STRUCTURE FOUNDATION EXPLORATION

Proposed Bridge Replacement

Bridge Street Over Middle Branch Portage River

Sylvania, Lucas County, Ohio

Submitted to Tetra Tech

FINAL REPORT Date *May 2025*

Prepared by

CT consultants
engineers • architects • planners
A Verdantas Company

Ohio Department of Transportation

CT consultants
engineers • architects • planners

May 30, 2025 CT Project No. 231797

Mr. David T. Charville, PE
Tetra Tech
420 Madison Ave, Suite 1001
Toledo, Ohio 43604

Re: ***Final Report***
Structure Foundation Exploration
Proposed Bridge Replacement
Bridge Street over Middle Branch Portage River
Village of Pemberville, Ohio

Dear Mr. Charville:

Following is the final report of the structure foundation exploration performed by CT Consultants, Inc. (CT) for the referenced project conducted for Tetra Tech. This study was performed in accordance with CT Proposal No. P231797R3, dated September 13, 2023, and authorized via a Tetra Tech Subconsultant Services Agreement dated October 27, 2023.

A “draft” version of the report, dated September 19, 2024, was previously provided for review by Tetra Tech and ODOT. It was indicated that there were no comments regarding the draft report. This final report contains the results of our study, our engineering interpretation of the results with respect to the project characteristics, as well as our design and construction recommendations for bridge foundations and pavements.

Should you have any questions regarding this report or require additional information, please contact our office.

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Respectfully,
CT Consultants, Inc.

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FINAL REPORT
STRUCTURE FOUNDATION EXPLORATION
PROPOSED BRIDGE REPLACEMENT
BRIDGE STREET OVER MIDDLE BRANCH PORTAGE RIVER
VILLAGE OF PEMBERVILLE, OHIO

FOR

TETRA TECH
420 MADISON AVE, SUITE 1001
TOLEDO, OHIO 43604

SUBMITTED

MAY 30, 2025
CT PROJECT NO. 231797

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1.0 INTRODUCTION

This structure foundation exploration report has been prepared for the replacement of the existing bridge along Bridge Street over Middle Branch Portage River in the Village of Pemberville, Ohio. The general project area is shown on the Site Location Map (Plate 1.0).

This report describes the investigative and testing procedures, presents the findings, discusses our evaluations and conclusions, and provides our recommendations for bridge foundations.

This report has been prepared for Tetra Tech. This study was performed in accordance with CT Proposal No. P231797R3, dated September 13, 2023, and authorized via a Tetra Tech Subconsultant Services Agreement dated October 27, 2023.

1.1 Purpose and Scope of Exploration

The purpose of this exploration was to evaluate the subsurface conditions relative to the design and construction of foundations for a new bridge structure at the referenced location. To accomplish this, CT performed four test borings, field and laboratory soil testing, a geotechnical engineering evaluation of the test results, and a review of available geologic and soils data for the project area.

This report summarizes our understanding of the proposed construction, describes the investigative and testing procedures utilized to evaluate the subsurface conditions at the site, and presents our findings from the field and laboratory testing. This report also presents our evaluations and conclusions in accordance with ODOT Bridge Design Manual, as well as provides our design and construction recommendations for foundations for the proposed bridge replacement structure.

This report includes:

> A description of the subsurface soil, rock, and groundwater conditions encountered in the borings.

> Design recommendations for bridge foundations.

> Recommendations concerning soil-, rock-, and groundwater-related construction procedures such as site preparation, earthwork, foundation and pavement construction, as well as related field testing.

Appendix B includes pertinent ODOT Geotechnical Engineering Design Checklists that apply to the scope of this report.

This exploration did not include an environmental assessment of the surface or subsurface materials at the site.

1.2 Proposed Construction

It is our understanding that the existing bridge is indicated to be a three-span structure 160 feet in length, bearing on shallow spread foundations. Currently the bridge includes a barricaded closure in an abundance of caution for the exterior bridge beams beneath the sidewalks which showed section loss.

It is planned to support the structure using drilled shafts socketed into bedrock at the abutments. Maximum total factored loads are shown in the following table

Table 1.2. Maximum Factored Load

Item	Rear (West) Abutment (B-001)	Pier 1 (West Pier) (B-002)	Pier 2 (East Pier) (B-003)	Forward (East) Abutment (B-004)
Maximum Factored Load (feet)	247	456	456	247

2.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1 General Geology and Hydrogeology

Published geologic maps from the Ohio Department of Natural Resources (ODNR) indicate that the project site is located in the Woodville Lake-Plain Reefs Physiographic District of the Maumee Lake Plains Region of Ohio. Within this district, the geologic deposits consist of Wisconsinan-age, wave-planed clayey glacial till, lacustrine (lake-bed) deposits, and sand.

At the project site, the sandy soils, as well as lacustrine deposits may have been eroded by the Portage River or removed and replaced with fill as part of the previous bridge construction. Alluvial deposits associated with the Portage River may also be encountered at the site.

The glacial till, also referred to as moraine, was deposited by the advance and retreat of glacial ice. Due to the weight of the ice mass, the till deposits are moderately to highly over-consolidated, that is, the existing soil deposits have experienced a previous vertical stress significantly higher than the effective vertical stress presently caused by the remaining overlying soil strata in the profile. Additionally, within the glacial till, it is not uncommon to encounter cobbles, boulders, and seams of granular soils, which may or may not be water bearing.

Underlying the soils, bedrock in the project area is broadly mapped on the “Geologic Map of Ohio” as Silurian-age Monroe limestone. Borings performed for this investigation encountered bedrock at elevations varying from approximate Elevs. 637 to 630.The USDA Natural Resource Conservation Service (NRCS) Web Soil Survey indicates that soils in the project area are predominantly mapped as Eel silt Loam (EmA) and Haney Loam (HdA). The Eel silt loam soils consist of loamy alluvium formed on rises on flood plains, natural levees on flood plains, as well as flats on flood plains , and are characterized as moderately well drained. The Haney loam soils consist of loamy glaciolacustrine deposits formed along beach ridges on lake plains, flats on lake plains, as well as rises on lake plains , and are characterized as moderately well drained.

2.2 Observations of the Project

Based on the original plans for the existing bridge structure prepared by the Wood County Engineer's Office, the existing bridge consists of a three-span structure 160 feet in length, bearing on shallow spread foundations. The ten-year High Water Level is indicated at Elev. 644.5, and the channel bottom is indicated at Elev. 633.

Currently the bridge includes a barricaded closure in an abundance of caution for the exterior bridge beams beneath the sidewalks which showed section loss. The central, interior bridge beams were suitable for support of CT's drilling rig.

CT performed a site reconnaissance on December 8, 2023. Cracks were observed along each of the abutments, with one abutment wingwall observed to have translated away from the superstructure. Spill through sections were observed to contain rip rap, sediment, and debris. Gravel appeared below the west and east spans to have accumulated to a level above the existing stream at the time of our reconnaissance. The observable portions of the existing bridge piers appeared in generally good condition.

The existing asphalt pavements appeared in fair condition, with longitudinal and transverse cracks which had been sealed. The existing bridge deck appeared in fair condition, with occasional transverse cracking.

Surrounding land usage consisted of residential developments.

3.0 EXPLORATION

3.1 Historic Borings

Historic boring data was not available at this site.

3.2 Project Exploration Program

This exploration included four test borings, designated as Borings B-001-0-23 through B-004-0-23, performed by CT from the period of December 11, 2023 to January 4, 2024. These borings are fully designated in accordance with ODOT protocol, but the “-0-23” portion of the nomenclature is generally omitted for ease of identification in the discussions within this report. The borings were located in the field by CT based on the proposed boring location plan provided with the proposal for this project. Two of the borings were performed with pavement cores, one behind each abutment, and two of the borings were performed offset from the existing pier locations. The approximate locations of the borings and existing structure are shown on the Test Boring Location Plan (Plate 2.0).

Stations, offsets, and ground surface elevations, at the boring locations were provided by Tetra Tech. Latitude and Longitude were estimated using Google Earth. These data are presented on the logs of test borings.

In accordance with the ODOT Specifications for bridge structure explorations, each of the borings were extended to auger refusal at depths ranging from 10 to 17 feet below existing grades. Upon encountering auger refusal, the borings were then advanced by coring 10 feet into the underlying bedrock, with the exception of Boring B-001 having cored 15 feet into underlying rock due to poor recovery/Rock Quality Designation in the upper profile rock.

Experience indicates that the actual subsoil conditions at a site could vary from those generalized on the basis of test borings made at specific locations. Therefore, it is essential that a geotechnical engineer be retained to provide soil engineering and inspection services during the site preparation, excavation, and foundation phases of the proposed project. This is to observe compliance with the design concepts, specifications, and recommendations, and to allow design changes in the event subsurface conditions differ from those anticipated prior to the start of construction.

3.3 Boring Methods

The test borings performed during this exploration were drilled with a CME 550 ATV-mounted drilling rig. The borings were extended utilizing 3¼-inch inside diameter hollow-stem augers. During auger advancement, samples were collected continuously using 18-inch sample drives to evaluate gradation for subgrade analyses as well as for scour potential. The samples were sealed in jars and transported to our laboratory for further classification and testing.

Split-spoon (SS) soil samples were obtained by the Standard Penetration Test Method (ASTM D 1586). The Standard Penetration Test (SPT) consists of driving a 2-inch outside diameter split-spoon sampler into the soil with a 140-pound weight falling freely through a distance of 30 inches. The sampler was driven in three successive 6-inch increments, with the number of blows per increment being recorded, and these data are presented under the "SPT" column on the Logs of Test Borings attached to this report. The sum of the number of blows required to advance the sampler the second and third 6-inch increments is termed the Standard Penetration Resistance, or N_m-value, and is typically reported in blows per foot (bpf). The N_m-values were corrected to an equivalent rod energy ratio of 60 percent, N_{60}. The hammer/rod energy ratio was 75.2 percent for the CME 550 ATV-mounted drill rig utilized on this project, based on calibration performed on February 20, 2023. The N_{60}-values are presented on the attached Logs of Test Borings attached to this report. In conjunction with published data and typical correlations, the N_{60}-values can be evaluated as a measure of soil compactness/consistency as well as shear strength and bearing capacity.

A Shelby tube sample, designated ST on the Logs of Test Borings, was attempted in Boring B-004 (5½ to 7½ feet) by hydraulically advancing a 3-inch diameter, thin-walled sampler approximately 24 inches beyond the hollow-stem auger into relatively undisturbed soil in accordance with ASTM D 1587. The Shelby tube was then extracted from the subsoils but resulted in no recovery.

Core samples of the bedrock were obtained from each boring, using an NQ2 diamond-bit core barrel and coring techniques in general accordance with ASTM D 2133. In Boring B-001, two core runs were completed immediately following auger refusal for a total depth of 15 feet. In Borings B-002, B-003, and B-004, two rock core runs were performed for a total of 10 feet. Recovery of the core is expressed as the percentage ratio of the recovered rock length to the total length of the core run. The Rock Quality Designation (RQD) is the percentage ratio of the summed length of rock pieces 4 inches long and greater to the total length of the run. The rock core samples are designated as "NQ2-1 and NQ2-2" on the Logs of Test Borings. The recovered rock cores were visually classified using the ODOT Rock Classification System. The rock cores were also documented by photographic core logs which are attached to this report in Appendix C.

Soil and rock conditions encountered in the test borings are presented in the Logs of Test Borings along with information related to sample data, SPT results and corresponding N_{60}-values, water conditions observed in the borings, and laboratory test data. Field and laboratory data were incorporated into gINT™ software for presentation purposes. It should be noted that these logs have been prepared on the basis of laboratory classification and testing as well as field logs of the encountered soils and rock.

3.4 Laboratory Testing Program

All soil samples were visually or manually classified in accordance with the ODOT Soil Classification System. All samples of the subsoils were also tested in our laboratory for moisture content (ASTM D 2216). Atterberg limits tests (ASTM D 4318) and/or particle size analyses (ASTM D 6913 and D 7928) were performed on selected samples to determine soil classification and index properties. Dry density determinations and unconfined compressive strength tests (ASTM D 2166) by the constant rate of strain method were performed on selected intact cohesive split-spoon samples. Unconfined compressive strength estimates were obtained for the remaining intact cohesive samples using a calibrated hand penetrometer. Additionally, sulfate content testing (ODOT Supplement 1122) was performed on two samples, B-001 (SS-1) and B-002 (SS-1), one from the uppermost 3 feet of the subgrade soils in each of the pavement borings. These test results are presented on the Logs of Test Borings attached to this report.

Recovered rock core specimens were visually classified in general accordance with Ohio Department of Transportation (ODOT) "Specifications for Geotechnical Explorations" (SGE) criteria. Selected intact rock specimens were tested for unconfined compressive strength in accordance with ASTM D 7012, Method C. Results of these tests are presented on the Logs of Test Borings and the attached Unconfined Compressive Strength Test results.

It should be noted that the specimens were prepared using a table saw to obtain flat perpendicular ends with respect to the longitudinal specimen, then the ends were capped using capping compound to ensure they were relatively flat. The planeness of the bearing surfaces of the specimens were checked by means of a straightedge and feeler gauge, and the capped surfaces were determined to be plane within 0.002 inches (0.05 mm). The surfaces of the specimens in contact with the lower bearing block of the testing machine were similarly evaluated for perpendicularity to the axis by less than 1 degree (approximately equivalent to a deviance of 0.07 inches along a 4-inch specimen). ASTM D 7012 requires that we indicate the sample was not prepared using specialized equipment per ASTM D 4543, and that the reported results may differ from those obtained using a test specimen prepared per ASTM D 4543. However, the difference should be insignificant for strong rock, such as encountered for this project, but the difference can be more pronounced for weak rock.

4.0 FINDINGS

4.1 General Site Conditions

The project site is located along Bridge Street, at the crossing of Portage River, between Water Street and Brierley Avenue, in the Village of Pemberville, Wood County, Ohio. Roadway grades in the project area are generally level, with ground surface elevations at the boring locations on the order of Elev. 611.

Borings B-001 and B-004 were performed within the roadway, and the encountered surface materials consisted of approximately 3 to 6½ inches of asphalt underlain by approximately 5 to 11½ inches of aggregate base.

Borings B-002 and B-003 were performed within the bridge deck and the encountered surface materials consisted of concrete approximately 8 inches in thickness.

Granular existing fill materials were encountered underlying the pavement materials in Borings B-001 and B-004 to depths of 3 feet and feet below existing grade (Elevs. 643± and 642±), respectively. The granular existing fill materials consisted of predominantly crushed stone (or gravel and stone fragments) with varying amounts of sand, silt, and clay. SPT N_{60}-values ranged from 15 to 24 bpf, indicating medium dense compactness. Moisture contents ranged from 4 to 8 percent.

4.2 General Soil Conditions

The subsoils encountered underlying the surface and fill materials can be generally described as a layer of cohesive till underlain granular soils, overlying bedrock.

Stratum I consisted of predominantly stiff to very stiff cohesive till deposits encountered underlying the existing fill materials in Borings B-001 and B-004 to depths of 10 feet and 7 feet below existing grade (Elevs. 636± and 639±), respectively. These cohesive soils consisted of sandy silt (ODOT A-4a). SPT N_{60}-values generally ranged from 10 to 25 blows per foot (bpf). Unconfined compressive strengths generally ranged from 3,000 to 7,500 pounds per square foot (psf). Moisture contents ranged from 14 to 20 percent.

Stratum II consisted of predominantly medium dense granular soils encountered at the river bottom in Boring B-002 as well as underlying Stratum I in Boring B-004 to depths of 17 feet and 9 feet (Elevs. 630± and 637±), respectively. These granular soils consisted of gravel and stone fragments (ODOT A-1-a) as well as gravel and stone fragments with sand (ODOT A-1-b). SPT N_{60}-values of 26 bpf and 25 bpf were determined for these granular soils. Moisture contents of 11 percent and 14 percent were determined for the recovered samples.

Additional descriptions of the stratigraphy encountered in the borings are presented on the Logs of Test Borings.

4.3 General Bedrock Conditions

Augerable weathered dolomite was encountered at the river bottom in Boring B-003 as well as underlying the subsoils in the remaining borings at depths ranging from 9 to 17 feet (Elev. 637± to 630±), extending to depths of ranging from 10½ to 18 feet (Elev. 635± to 629±).

Underlying the weathered dolomite, auger refusal on dolomite bedrock was encountered. The rock was cored in each of the borings for a total length of 10 to 15 feet. The cored bedrock consisted of highly weathered to moderately weathered dolomite. The rock core data obtained from the borings are summarized as follows:

Table 4.3. Rock Core Data

Boring Number	Rock Core Number	Depth (feet)	Elevation (feet)	Recovery (%)	RQD (%)	Unconfined Compressive Strength (psi)
B-001	NQ2-1	12.0-22.0	633.8-623.8	40	0	-
	NQ2-2	22.0-27.0	623.8-618.8	100	40	10,990
B-002	NQ2-1	18.0-23.0	629.4-624.4	100	30	5,510
	NQ2-2	23.0-28.0	624.4-619.4	95	75	-
B-003	NQ2-1	15.5-20.5	631.8-626.8	100	57	-
	NQ2-2	20.5-25.5	626.8-631.8	100	95	15,570
B-004	NQ2-1	10.5-15.5	635.2-630.2	100	17	-
	NQ2-2	15.5-20.5	630.2-625.2	80	65	5,560

RQD values typically ranged from 17 to 75 percent, indicating that the overall rock mass quality can be generally described as very poor to fair. Unconfined compressive strength results ranged from 5,510 to 15,570 pounds per square inch (psi), indicating moderately strong to very strong bedrock.

Additional descriptions of the stratigraphy encountered in the borings are presented on the Logs of Test Borings. Photographs of the rock cores are attached to this report in Appendix C.

4.4 Groundwater Conditions

Provided drawings for the existing structure indicate the ten-year High Water Level at Elev. 644.5.

Groundwater was initially encountered during drilling and observed at the completion of drilling in Borings B-002 and B-003 at a depths 14 feet below existing grade (Elev. 633±). It should be noted that each boring was drilled and sealed within the same day. Therefore, stabilized water levels may not have occurred over this limited time period. Instrumentation was not installed to observe long-term groundwater levels.

Based on the soil characteristics and water conditions encountered in the borings, it is our opinion that "normal" groundwater levels at this structure location will generally occur at or slightly above the streamflow levels in Portage River. It should be noted that groundwater elevations can also fluctuate with seasonal and climatic influences, as well as streamflow conditions in the river. In particular, "perched" groundwater conditions may be present within the existing fill materials underlain by relatively impermeable cohesive soils, as well as at the soil/bedrock interface. Therefore, the groundwater conditions may vary at different times of the year from those encountered during this exploration.

4.5 Gradation Results for Potential Scour Evaluations

Scour considerations for the encountered subsoils and bedrock should be made as part of the vertical and lateral load evaluations for the drilled shafts and rock sockets. Scour calculations were completed in accordance with the ODOT Bridge Design Manual and ODOT Geotechnical Design Manual, incorporating geotechnical scour parameters such as critical shear stress, erodibility index, and geological strength index. Additionally, rock quality parameters, including slake durability test results (ASTM D4644-16) and unconfined compressive strength tests (ASTM D7012 Method C), were provided.

The preliminary Slake Durability test results indicate the corable bedrock section is scour-resistant. Accordingly, the top of competent bedrock at each boring location is established as the controlling scour elevation. The scour calculations are attached in Appendix A for review by the project's hydrological professional.

However, it was subsequently communicated that methods found in FHWA Hydraulic Engineering Circular No. 18 (HEC 18) will be used to determine bridge scour, which relies on a geotechnical scour number empirically derived from modified slake durability test results to calculate rock abrasion scour. As this procedure is not commonly used on ODOT projects and is not mentioned in the ODOT SGE, it was not included in the scope of this subsurface investigation. If required, CT can perform the modified slake durability test and provide a Geotechnical scour number under separate cover.

4.6 Remedial Measures

Based on the relative proximity of bedrock to the proposed foundation pier/pile caps, we understand that the new bridge is planned to be supported by a deep foundation system consisting of drilled shafts socketed into bedrock. The planned extension for the sockets is typically 1.5 times the shaft diameter. However, rock sockets should be increased to 5 feet below top of competent rock (i.e., top of corable bedrock). In accordance with BDM 305.4.1.1 requirements, drilled shafts must extend at least 5 feet below the controlling scour elevation.

The ODOT "Subgrade Analysis" worksheet (V14.6, 02/11/22) indicates that over-excavation of unsuitable subgrade soils and replacement with new granular engineered fill are not anticipated.

During construction, temporary sheet-pile cutoff walls or cofferdams to direct streamflow may be required to manage groundwater in addition to pumping from prepared sumps. It is likely that temporary steel casing will be required to support the walls of the drilled shafts and to control groundwater seepage.

5.0 ANALYSES AND RECOMMENDATIONS

The following analyses and recommendations are based on our understanding of the proposed construction and upon the data obtained during our field exploration. If the project information or location as outlined is incorrect or should change significantly, a review of these recommendations should be made by CT.

5.1 Bridge Foundations

Consideration was given to spread foundations bearing on bedrock for the bridge. However, based on our evaluations, none of the samples of the upper potential bearing rock met all of the criterion required to be considered scour-resistant rock in accordance with ODOT Bridge Design Manual (BDM) Section 305.2.1.2.b. The RQD values, RMR values, and GSI values were lower than the minimum requirements. These structures are now planned to be supported by drilled shafts socketed into bedrock.

5.1.1 Drilled Shaft Rock Socket Vertical Resistance

We understand that the bridge foundations will be designed using LRFD methods. Based on the relative proximity of bedrock to the proposed foundation pier/pile caps, as well as the potential for scour, we understand that the new bridge is planned to be supported by a deep foundation system consisting of drilled shafts socketed into bedrock. Maximum total factored loads associated with individual drilled shafts were not available at the time of preparing this memo. Recommendations are provided herein for the smallest diameter drilled shafts that may be constructed in accordance with the ODOT Bridge Design Manual (BDM).

The minimum diameter for drilled shafts that support pier columns is 42 inches. Likewise, drilled shafts that support abutments may be 36 inches in diameter. The diameter of bedrock sockets for drilled shafts are generally 6 inches less than the diameter of the shaft above the bedrock elevation. Regardless of shaft diameter, reinforcing steel cages should be based on the bedrock socket diameter.

For the abutments, we have evaluated a 36-inch diameter shafts above bedrock and a socket diameter of 30 inches. At the pier locations, we have evaluated 42-inch diameter shafts for the full depth below the bottom of the river without a reduction in shaft diameter for the socket due to minimal overburden.

Based on the rock conditions encountered in Boring B-001, as well as considering rock conditions in nearby Boring B-002, for the Rear (West) Abutment, an unfactored unit tip resistance (q_p) of 1,984 kips per square foot (ksf) was calculated. Based on the design methodologies utilized to evaluate unfactored unit tip resistance and AASHTO LRFD Table 10.5.5.2.4-1, a resistance factor of 0.50 should be utilized for design for tip resistance. As such, the factored unit tip resistance was calculated to be 992 ksf. **Using the planned 30-inch diameter socket, this design value would provide sufficient resistance for the indicated factored vertical load.** The maximum factored load to be supported by the read abutment drilled shafts is 247 kips at the abutments. This load is resisted entirely by tip resistance. At the Rear Abutment, the factored tip resistance is 4868 kips.

Based on the rock conditions encountered in Boring B-002 for the Pier 1 (West Pier), an unfactored unit tip resistance (q_p) of 1,984 kips per square foot (ksf) was calculated. Based on the design methodologies utilized to evaluate unfactored unit tip resistance and AASHTO LRFD Table 10.5.5.2.4-1, a resistance factor of 0.50 should be utilized for design for tip resistance. As such, the factored unit tip resistance was calculated to be 992 ksf. **Using the planned 42-inch diameter socket, this design value would provide sufficient resistance for the indicated factored vertical load.** The maximum factored load to be supported by the West Pier drilled shafts is 576 kips at the abutments. This load is resisted entirely by tip resistance. At the West Pier, the factored tip resistance is 9542 kips.

Based on the rock conditions encountered in Boring B-003 for the Pier 2 (East Pier), an unfactored unit tip resistance (q_p) of 5,605 kips per square foot (ksf) was calculated. Based on the design methodologies utilized to evaluate unfactored unit tip resistance and AASHTO LRFD Table 10.5.5.2.4-1, a resistance factor of 0.50 should be utilized for design for tip resistance. As such, the factored unit tip resistance was calculated to be 2,803 ksf. **Using the planned 42-inch diameter socket, this design value would provide sufficient resistance for the indicated factored vertical load.** The maximum factored load to be supported by the East Pier drilled shafts is 576 kips at the abutments. This load is resisted entirely by tip resistance. At the East Pier, the factored tip resistance is 26,964 kips.

Based on the rock conditions encountered in Boring B-004 for the Forward (East) Abutment, an unfactored unit tip resistance (q_p) of 2,002 kips per square foot (ksf) was calculated. Based on the design methodologies utilized to evaluate unfactored unit tip resistance and AASHTO LRFD Table 10.5.5.2.4-1, a resistance factor of 0.50 should be utilized for design for tip resistance. As such, the factored unit tip resistance was calculated to be 1,001 ksf. **Using the planned 30-inch diameter socket, this design value would provide sufficient resistance for the indicated factored vertical load.** The maximum factored load to be supported by the East Pier drilled shafts is 576 kips at the abutments. This load is resisted entirely by tip resistance. At the East Pier, the factored tip resistance is 26,964 kips.

It should be noted that the values for factored unit tip resistance listed above are based on bearing in competent rock that does not contain adverse jointing, open solution cavities, or joints that are filled with weathered material that would affect the bearing resistance of the rock, within a distance equal to two socket diameters below the tip of the drilled shaft rock socket.

The minimum prescribed rock socket length is 1.5B, where B is the socket diameter. For the abutment and pier socket diameters indicated above, the minimum socket length would be 3.75 feet for the abutments and 5.25 feet for the piers for bearing elevations on the order of Elevs. 633± to 625±. In any case, any structural requirement for the drilled shaft foundations to resist lateral loads or moments may increase the socket depth or diameter and should be evaluated on an individual shaft basis by Tetra Tech.

A summary of the recommended rock socket lengths based on vertical resistance evaluations is provided in the following table.

Table 5.5.1. Minimum Rock Socket Length Based on Vertical Load Considerations

Item	Rear (West) Abutment (B-001)	Pier 1 (West Pier) (B-002)	Pier 2 (East Pier) (B-003)	Forward (East) Abutment (B-004)
Minimum Rock Socket Length$^{(1)(2)}$ (feet)	7	6	6	6.5
Top of Rock Elevation (feet)	635.8	630.4	632.8	636.7
Bottom of Rock Socket Elevation (feet)	628.8	624.4	626.8	630.2

(1) Based on rock socket diameters of 30 inches for the abutments and 42 inches for the piers, as well as rock considerations discussed above.
(2) Rock socket length based on BDM 305.4.1.1 requirements, drilled shafts must extend at least 5 feet below the controlling scour elevation.

The factored unit tip resistance was based on rock conditions. We recommend the structural engineer also consider any limiting conditions associated with the stress limitations of the concrete.

It should be noted that the provided factored unit bearing resistance reflects end-bearing conditions only. Typically, design based on end-bearing alone is considered when sound bedrock underlies highly weathered rock. Conversely, design based on side shear resistance alone is considered when the drilled shaft cannot be adequately cleaned, or where large movement of the shaft would be required to mobilize the end bearing. For this project, significant movement is not expected to be required to mobilize the end bearing, and it is assumed that due diligence will be exercised to install the shafts in a cleaned drill hole.

Drilled shafts should be constructed in accordance with ODOT Construction and Material Specifications (CMS) Item 524. It is also recommended that the center-to-center spacing between adjacent shafts be no less than 2 shaft diameters.

Due to the presence of groundwater, it is likely that temporary steel casing will be required to support the walls of the shaft and to control water seepage. If significant seepage is encountered and cannot be suitably pumped to dewater the drilled shaft, concrete will require placement by tremie methods. As the steel casing is withdrawn during concreting, sufficient concrete should be maintained above the bottom of the casing to counteract any hydrostatic head. Care must be taken during concreting and removal of any temporary liner so as to avoid the possibility of soil intrusions. The contractor should submit procedures for installation prior to the start of work.

Although cobbles or boulders were not noted in the borings performed for this exploration, they may be encountered at this site. Additionally, although not encountered, debris may be present in existing fill materials. Therefore, provisions should be made by the contractor to remove any obstructions, including debris, cobbles or boulders, if they are encountered during the drilling operations.

Drilled shafts should be clean and free of all loose material prior to the placement of concrete. A CT representative should verify that shafts are bearing on competent materials and that installation procedures meet specifications.

Based on ODOT guidelines, foundation plans should contain the following typical notes:

> The maximum factored load to be supported by each drilled shaft is 247 kips at the abutments. This load is resisted entirely by tip resistance. At the Rear Abutment and Forward Abutment, the factored tip resistance is 4,868 kips and 4,913 kips, respectively.
>
> The maximum factored load to be supported by each drilled shaft is 576 kips at the piers. This load is resisted entirely by tip resistance. At Pier 1 and Pier 2, the factored tip resistance is 9,542 kips and 26,964 kips, respectively.

5.1.2 Lateral Load Soil and Rock Design Parameters

For lateral load-deflection evaluations using software, such as LPILE, recommended design parameters are provided in the attached tables based on the conditions encountered in the borings. For the portion of the drilled shaft below the groundwater table (estimated at or slightly above the water level in Middle Branch Portage River), the effective unit weight must be considered (i.e., reduce the total unit weight by the unit weight of water, 62.4 pounds per cubic foot). These LPILE inputs are being provided for the structural engineer to evaluate a suitable, economical socket length and diameter, as well as to modify steel reinforcement conditions.

5.2 GDM Section 600 "Plan Subgrades" Evaluation

5.2.1 Subgrade Analysis Worksheet

Based on the encountered pavement sections, our evaluations considered an average pavement cross-section of 13 inches (approximately 1.1 feet) for the proposed pavements.

Based on GDM Section 600, soils classified as ODOT A-4b, A-2-5, A-5, A-7-5, A-8a, A-8b, or rock have been designated as being problematic with respect to pavement subgrade support. None of these soil types were encountered at planned subgrade elevations in the borings performed for this exploration.

Based on GDM Section 600 criteria, subgrade soils with moisture contents greater than 3 percent above optimum likely indicate the presence of unstable subgrade that may require some form of subgrade modification. For this site, approximately 75 percent of tested cohesive subgrade soil samples and approximately 25% of the tested granular subgrade soil samples were greater than 3 percent above the optimum as determined using GDM Section 600 criteria.

It should be noted that all of the evaluated samples with moisture contents greater than 3 percent above optimum had moisture contents greater than or equal to 5 percent above optimum. Thus, where moisture contents were wet of optimum, they were appreciably wet of optimum. These data indicate that scarification and aeration methods may not be feasible to achieve satisfactory proof rolling and stabilization of the predominantly cohesive subgrades. However, scarification and aeration methods may be utilized in areas where granular subgrades wet of optimum are present, provided weather conditions and construction schedule will allow such soil modification.

The type and thickness of subgrade modification is determined by GDM Section 600 criteria based on the average, low SPT N_{60}-value (N_{60L}) of the subgrade soils in a particular portion of the project area, hand penetrometer values, soil type, and moisture content. Based on these criteria, none of the borings contained subgrade soils which indicated subgrade modification is likely to be required. Possible alternatives for those areas where modification of the subgrade soils is indicated could include the following, using GDM Section 600 criteria based on the encountered conditions:

> Undercut to a depth of 12 inches and replacement with geotextile and granular engineered fill in those areas, or

> Global chemical stabilization to a depth of 14 inches using cement.

5.2.2 Construction Considerations

Undercut and Replacement Option

If undercut and replacement is utilized, all fill should consist of ODOT Item 304 Aggregate Base or Item 703.16C, Granular Material Type B or Type C. As prescribed by GDM Section 600 criteria, excavate unstable subgrades to 18 inches beyond the edge of the surface of the pavement, paved shoulders, or paved medians, including under new curbs and gutters. Always drain the excavation to an underdrain, catch basin, or pipe. It is recommended that geotextile fabric (referenced in ODOT Item 204, and specified as ODOT Item 712.09, Type D) be utilized on the subgrade at the bottom of the undercut zone. Although not anticipated to be required based on the conditions encountered in the borings and the proposed sections and grades, if particularly unstable subgrades are encountered during construction, or undercuts exceed approximately 18 inches, a geogrid could be used to reduce the total undercut and replacement of the unsuitable soils by 6 inches. Do not use geotextile or geogrid in the areas of underdrains.

Additionally, if the ongoing cement shortage precludes the use of cement for chemical stabilization, subgrade modification should consider over-excavation of unsuitable subgrade soils and replacement with new granular engineered fill.

Chemical Stabilization Option

For projects where the total length of required undercuts is equal to or greater than 0.1 mile, it is common that global chemical stabilization to a depth of 14 inches can be more economical compared to over-excavation and replacement with new granular engineered fill.

As previously noted, none of the borings contained subgrade soils which indicated subgrade modification is likely to be required. GDM Section 600 indicates that, if it is determined that 30 percent or more of the subgrade area must be stabilized, consideration should be given to stabilizing the entire project (global chemical stabilization). As such, it is anticipated that global chemical stabilization would not be an economical modification option.

As required by GDM Section 600, sulfate content tests (ODOT Supplement 1122) were performed on a sample typically within the upper 3 feet of the subgrade elevation in each boring. The sulfate content test results are summarized in the following table.

Table 5.2.2.2. Sulfate Content Test Results

Boring Number	Sulfate Content (mg/kg)
B-001	<100
B-004	<100

GDM Section 600 indicates that chemical stabilization cannot be utilized when sulfate contents for the majority of the samples exceed 3,000 parts per million (ppm), or individual soil samples exhibit sulfate contents of greater than 5,000 ppm. Based on the tested samples, sulfate content will not preclude the use of chemical stabilization for this project.

General

It should be noted that subgrade analyses are used as a pre-construction tool to plan subgrade modification alternatives. **Actual subgrade modification will depend on field observations of proof-rolling conditions at the time of construction.** Changes in soil moisture content could create more or less favorable subgrade conditions that may result in adjustments to subgrade modification or soil stabilization requirements at the time of construction.

5.2.3 Planned Subgrade Modification Recommendation

Based on the subgrade analysis and our understanding of the project, it is anticipated that over-excavation and replacement with new granular engineered fill to a depth of 12 inches will be more economical compared to global chemical stabilization.

5.3 Flexible (Asphalt) Pavement Design

Based on the subgrade analysis, a design CBR value of 9 percent was determined for the project. It should be noted that the CBR determination by the subgrade analysis spreadsheet is based on the average Group Index of all the evaluated samples, which was 3. Group indices for the evaluated samples ranged from 0 to 8, which would correlate with a CBR value of 6 to 12 percent. Based on the average design value calculations from the subgrade analysis spreadsheet, it does not appear to be unconservative to use the spreadsheet design CBR value of 9 percent for new pavement sections throughout the project area.

All pavement design and paving operations should conform to ODOT specifications. The pavement and subgrade preparation procedures outlined in this report should result in a reasonably workable and satisfactory pavement. It should be recognized, however, that all pavements need repairs or overlays over time as a result of progressive yielding under repeated loading for a prolonged period.

It is recommended that proof rolling, placement of aggregate base, and placement of asphalt be performed within as short a time period as possible. Exposure of the aggregate base to rain, snow, or freezing conditions may lead to deterioration of the subgrade and/or base materials due to excessive moisture conditions and to difficulties in achieving the required compaction.

5.4 Construction Dewatering and Groundwater Control

Groundwater conditions encountered in the borings were summarized in Section 4.4. Based on the soil characteristics and moisture conditions encountered in the borings, it is our opinion that "normal" groundwater levels in the vicinity of Portage River will generally occur at or slightly above the "normal" flow levels in Portage River. It should be noted that groundwater elevations can also fluctuate with seasonal and climatic influences, as well as streamflow conditions in the river. Additionally, perched water may be present in granular soils that are underlain by relatively impermeable cohesive soils, as well as at the soil/bedrock interface.

Groundwater seepage, perched water, and surface water runoff into shallow excavations in predominantly cohesive soils should be controllable by pumping from prepared sumps. If excavations extend below the groundwater level in granular soils, installation of multiple well points may be required in addition to pumping from prepared sumps. Installation of the intermediate piers in Portage River may require temporary cofferdams to divert streamflow to manage groundwater in addition to pumping from prepared sumps. Otherwise, steel casing may also be used to help facilitate groundwater control. In any case, as mentioned in Section 5.1.1, it is likely that temporary steel casing will be required to support the walls of the drilled shafts, in addition to facilitating control groundwater seepage. In the event excessive seepage is encountered during construction, TTL should be notified to evaluate whether other dewatering methods are required.

5.5 Construction

5.5.1 Sediment and Erosion Control

In planning the implementation of earthwork operations, special consideration should be given to provide measures to prevent or reduce soil erosion and the subsequent sedimentation into nearby waterways. These measures may include some or all of the following:

1. Scheduling of earthwork operations such that erodible areas are kept as small as possible and are exposed for the shortest possible time.
2. Using special grading practices, along with diversion or interceptor structures, to reduce the amount of run-off water from an erodible area.
3. Providing vegetative buffer zones, filter berms, or sedimentation basins to trap sediment from surface run-off water.

A specific and detailed soil erosion and sedimentation control program and permits may be required by local, state, or federal regulatory agencies.

5.5.2 Site and Subgrade Preparation

Site and subgrade preparation activities should conform to ODOT Construction and Materials Specifications (CMS) Item 201 and 204 specifications. Site preparation activities should include the removal of vegetation, topsoil, root mats, pavements, and other deleterious non-soil materials from all proposed roadway areas. The actual amount of required stripping should be determined in the field by a geotechnical engineer or qualified representative.

Upon completion of the clearing and undercutting activities, all areas that are to receive fill, or that have been excavated to proposed final subgrade elevation, should be inspected by a geotechnical engineer. Prior to performing undercutting or subgrade stabilization, pavement subgrades should be proof rolled in accordance with ODOT CMS 204.06 to confirm the depth and extent of subgrade modifications required, followed by subgrade modification in accordance with ODOT CMS 204.06.

The subgrade analysis indicates options for "planned" subgrade modification of either global chemical stabilization to a depth of 14 inches or over-excavation to a depth of 12 inches of unsuitable subgrade soils and replacement with geotextile and new granular engineered fill. As indicated in Section 5.2.3, planned subgrade modifications are recommended to consist of global chemical stabilization.

5.5.3 Fill

Material for engineered fill or backfill required to achieve design grades should meet ODOT Item 203 "Embankment Fill" placement and compaction requirements. Borrow materials used for fill at subgrade elevations should be similar to the encountered existing subgrade soils to maintain the subgrade support properties associated with the recommended design CBR value and k-value for pavement design.

The upper profile on-site soils predominantly consist of cohesive soils, although granular soils were also encountered at pavement subgrade elevations. For the cohesive soils, a sheepsfoot roller should provide the most effective soil compaction. Where granular soils are encountered or new dense-graded aggregate pavement base materials are placed, a vibratory smooth-drum roller would be required to provide effective compaction.

5.5.4 Excavations and Slopes

The sides of temporary excavations for utility installations and other construction should be adequately sloped to provide stable sides and safe working conditions. Otherwise, the excavation must be properly braced against lateral movements. In any case, applicable Occupational Safety and Health Administration (OSHA) safety standards must be followed.

Based on the encountered soils, excavation may encounter the following OSHA type soils:

- Type A soils (native cohesive soils with unconfined compressive strengths of 3,000 pounds per square foot (psf) or greater),

- Type B soils (native cohesive soils with unconfined compressive strengths greater than 1,000 psf but less than 3,000 psf, cohesive embankment fill, as well as dry rock that is not stable), and
- Type C soils (granular soils, submerged soil, as well as submerged rock).

For temporary excavations in Type A, B, and C soils, side slopes must be no steeper than ¾ horizontal to 1 vertical (¾H:1V), 1H:1V, and 1½H:1V, respectively. For situations where a higher strength soil is underlain by a lower strength soil and the excavation extends into the lower strength soil, the slope of the entire excavation is governed by that required by the lower strength soil. In all cases, flatter slopes may be required if lower strength soils or adverse seepage conditions are encountered during construction.

For permanent excavations and slopes, we recommend that grades generally be no steeper than 3H:1V. Based on the provided plans, embankment slopes are generally planned to be 2H:1V. It should be noted that ODOT routinely uses 2H:1V slopes for roadway embankments. While these steeper slopes may used, it should be noted that the embankment faces are more prone to erosion and sloughing. Additional discussions regarding GB-2 “Special Benching” and slope stability were presented in Sections 5.1.1 and 5.1.2, respectively.

6.0 QUALIFICATION OF RECOMMENDATIONS

Our evaluation of bridge foundation and roadway pavement design and construction conditions has been based on our understanding of the site and project information and the data obtained during our field investigation. The general subsurface conditions were based on interpretation of the subsurface data at specific boring locations. Regardless of the thoroughness of a subsurface investigation, there is the possibility that conditions between borings will differ from those at the boring locations, that conditions are not as anticipated by the designers, or that the construction process has altered the soil conditions. This potential is increased at previously developed sites. Therefore, experienced geotechnical engineers should observe earthwork construction to confirm that the conditions anticipated in design are noted. Otherwise, CT assumes no responsibility for construction compliance with the design concepts, specifications, or recommendations.

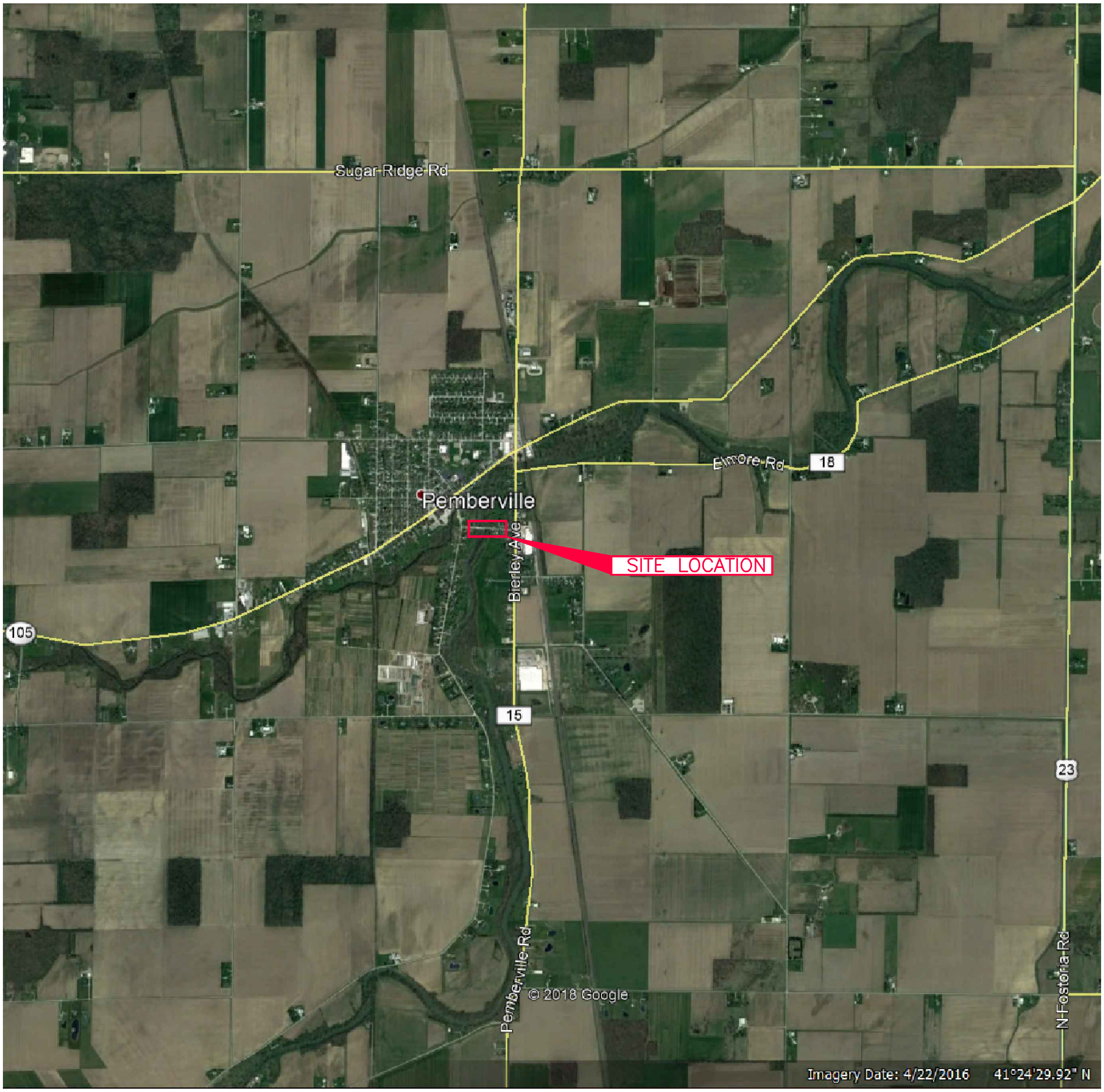
The design recommendations in this report have been developed on the basis of the previously described project characteristics and subsurface conditions. If project criteria or locations change, a qualified geotechnical engineer should be permitted to determine whether the recommendations must be modified. The findings of such a review will be presented in a supplemental report.

The nature and extent of variations between the borings may not become evident until the course of construction. If such variations are encountered, it will be necessary to reevaluate the recommendations of this report after on-site observations of the conditions.

Our professional services have been performed, our findings derived, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices. This warranty is in lieu of all other warranties either expressed or implied. CT is not responsible for the conclusions, opinions, or recommendations of others based on this data.

Plates

Plate 1.0 Site Location Map
Plate 2.0 Test Boring Location Plan



LEGEND

🟥 APPROXIMATE SITE LOCATION

N

APPROXIMATE SCALE – FEET

0 3,000 6,000

PLATE 1.0
SITE LOCATION MAP

PROPOSED BRIDGE REPLACEMENT
BRIDGE STREET OVER MIDDLE BRANCH PORTAGE RIVER
PEMBERVILLE, OHIO

PREPARED FOR
TETRA TECH
TOLEDO, OHIO

DRAWN	TRR/1-16-24	CHECKED	KCH/2-6-24
REVISED		APPROVED	
JOB NO.	231797		
DRAWING NUMBER			
B_231797-01-PLATE 1 SM			

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consultants
engineers
architects
planners

LEGEND

B-001-0-23 ⊕ APPROXIMATE TEST BORING LOCATION

APPROXIMATE SCALE – FEET
0 60 120

PLATE 2
TEST BORING LOCATION PLAN
PROPOSED BRIDGE REPLACEMENT
BRIDGE STREET OVER MIDDLE BRANCH PORTAGE RIVER
PEMBERVILLE, OHIO

PREPARED FOR
TETRA TECH
TOLEDO, OHIO

DRAWN	TRR/1-17-24	CHECKED	KCH/2-6-24
REVISED		APPROVED	
JOB NO.	231797		
DRAWING NUMBER	**B_231797-01-PLATE 2 BLP**		

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engineers
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planners

Figures

Logs of Test Borings
Legend Key
Rock Core Photo Log
Grain Size Distribution Curves
Rock Core Unconfined Compressive Strength Test Results

PROJECT: BRIDGE STREET BRIDGE
TYPE: BRIDGE
PID: N/A SFN: 8758948
START: 12/11/23 END: 12/13/23

DRILLING FIRM / OPERATOR: CT / JP
SAMPLING FIRM / LOGGER: CT / KKC
DRILLING METHOD: 3.25" HSA / NQ2
SAMPLING METHOD: SPT / NQ2

DRILL RIG: CME 550X ATV
HAMMER: CME AUTOMATIC
CALIBRATION DATE: 2/20/23
ENERGY RATIO (%): 75.2

STATION / OFFSET: 57+56, 4' LT.
ALIGNMENT: BRIDGE STREET
ELEVATION: 645.8 (NAVD88) EOB: 27.0 ft.
LAT / LONG: 41.409169, -83.457336

EXPLORATION ID
B-001-0-23

PAGE
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MATERIAL DESCRIPTION AND NOTES	ELEV. 645.8	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI				
ASPHALT - 3 INCHES	645.5																		
AGGREGATE BASE - 5 INCHES	645.1																		
MEDIUM DENSE, GRAY, **CRUSHED STONE WITH SAND**, SOME SILT, TRACE CLAY, DAMP FILL	642.8		16 8 8	20	100	SS-1	-	54	15	9	20	2	NP	NP	NP	8	A-1-b (0)	<100	
			7 5 5	13	100	SS-2A	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	-	
STIFF TO VERY STIFF, BROWN, **SANDY SILT**, LITTLE GRAVEL, LITTLE CLAY, DAMP	641.8					SS-2B	3.75	13	6	32	38	11	22	17	5	14	A-4a (3)	-	
MEDIUM STIFF TO STIFF, BROWN/GRAY, **SANDY SILT**, TRACE CLAY, TRACE GRAVEL, MOIST @5.5': BROWN @7.0' TO 8.0': Qu = 1,755 PSF	637.8		2 2 3	6	100	SS-3	1.50	2	5	41	45	7	23	15	8	20	A-4a (3)	-	
			3 3 3	8	100	SS-4	2.00	-	-	-	-	-	-	-	-	17	A-4a (V)	-	
			3 3 6	11	100	SS-5	2.00	-	-	-	-	-	-	-	-	17	A-4a (V)	-	
STIFF TO VERY STIFF, BROWN, **SANDY SILT**, "AND" GRAVEL, LITTLE CLAY, DAMP @8.5': VERY STIFF	635.8		10 10 10	25	100	SS-6	-	42	11	9	26	12	22	16	6	14	A-4a (1)	-	
BROWN, **WEATHERED DOLOMITE**, "AND" SAND, SOME SILT, TRACE CLAY	633.8	TR	50/4"	-	100	SS-7	-	29	15	23	30	3	23	17	6	10	A-2-4 (0)	-	
DOLOMITE, GRAY, HIGHLY WEATHERED, STRONG, JOINTED - HIGHLY FRACTURED TO FRACTURED, TIGHT, ROUGH; RQD 0%, REC 40%.	623.8		0		40	NQ2-1											CORE		
DOLOMITE, GRAY, HIGHLY WEATHERED, STRONG, JOINTED - FRACTURED TO MODERATELY FRACTURED, TIGHT, ROUGH; RQD 40%, REC 100%. @24.5' TO 24.9': Qu=10,990 PSI, γ_{DRY}=162.7 PCF @25.9' TO 26.7': CRYSTALLINE	618.8	EOB	40		100	NQ2-2											CORE		

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.25 BAG ASPHALT PATCH; AUGER CUTTINGS MIXED WITH 1.5 BAGS BENTONITE CHIPS

PROJECT: BRIDGE STREET BRIDGE
TYPE: BRIDGE
PID: N/A SFN: 8758948
START: 1/4/24 END: 1/4/24

DRILLING FIRM / OPERATOR: CT / CW
SAMPLING FIRM / LOGGER: CT / KKC
DRILLING METHOD: 3.25" HSA / NQ2
SAMPLING METHOD: SPT / NQ2

DRILL RIG: CME 550X ATV
HAMMER: CME AUTOMATIC
CALIBRATION DATE: 2/20/23
ENERGY RATIO (%): 75.2

STATION / OFFSET: 58+26, 4' RT.
ALIGNMENT: BRIDGE STREET
ELEVATION: 647.4 (NAVD88) EOB: 28.0 ft.
LAT / LONG: 41.409134, -83.457104

EXPLORATION ID
B-002-0-23

PAGE
1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 647.4	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI				
CONCRETE BRIDGE DECK - 8 INCHES	646.7																		
BOTTOM OF BRIDGE DECK TO TOP OF WATER	633.4	▼ 633.4																	
WATER	631.9																		
MEDIUM DENSE, GRAY, **GRAVEL AND STONE FRAGMENTS**, SOME SAND, LITTLE SILT, TRACE CLAY, MOIST	630.4	TR	5 7 14	26	89	SS-1	-	64	11	11	11	3	NP	NP	NP	11	A-1-a (0)	-	
GRAY, **WEATHERED DOLOMITE**, SOME SAND, LITTLE SILT, TRACE CLAY	629.4		34 50/1"	-	100	SS-2	-	58	15	12	14	1	NP	NP	NP	11	A-1-a (0)	-	
DOLOMITE, GRAY, MODERATELY WEATHERED, STRONG, CRYSTALLINE, JOINTED - HIGHLY FRACTURED TO FRACTURED, TIGHT, ROUGH; RQD 13%, REC 100%.	626.7		30		100	NQ2-1											CORE		
DOLOMITE, GRAY, MODERATELY WEATHERED, MODERATELY STRONG, SANDY LAMINAE, JOINTED - FRACTURED TO MODERATELY FRACTURED, TIGHT, ROUGH; RQD 67%, REC 100%. @21.2' TO 21.8': Qu=5,510 PSI, γ_{DRY}=151.6 PCF @24.8': VUGGY AND CRYSTALLINE	619.4	EOB	75		95	NQ2-2											CORE		

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: AUGER CUTTINGS MIXED WITH 0.5 BAG BENTONITE CHIPS; PLACED 0.25 BAG QUICKCRETE

PROJECT: BRIDGE STREET BRIDGE
TYPE: BRIDGE
PID: N/A SFN: 8758948
START: 1/4/24 END: 1/4/24

DRILLING FIRM / OPERATOR: CT / CW
SAMPLING FIRM / LOGGER: CT / KKC
DRILLING METHOD: 3.25" HSA / NQ2
SAMPLING METHOD: SPT / NQ2

DRILL RIG: CME 550X ATV
HAMMER: CME AUTOMATIC
CALIBRATION DATE: 2/20/23
ENERGY RATIO (%): 75.2

STATION / OFFSET: 58+96, 0' LT.
ALIGNMENT: BRIDGE STREET
ELEVATION: 647.3 (NAVD88) EOB: 25.5 ft.
LAT / LONG: 41.409127, -83.456794

EXPLORATION ID
B-003-0-23

PAGE
1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 647.3	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	SO4 ppm	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI	WC			
CONCRETE BRIDGE DECK - 8 INCHES	646.6																		
BOTTOM OF BRIDGE DECK TO TOP OF WATER	633.3	▼ 633.3																	
WATER	632.8	TR																	
GRAY, **WEATHERED DOLOMITE**, SOME SAND, LITTLE SILT, TRACE CLAY	631.8	15	15 50/5"	-	91	SS-1	-	65	16	8	10	1	NP	NP	NP	11	A-1-a (0)	-	
DOLOMITE, GRAY, SLIGHTLY WEATHERED, STRONG, JOINTED, FRACTURED TO MODERATELY FRACTURED, TIGHT, ROUGH, NEAR-VERTICAL FRACTURING; RQD 59%, REC 100%.	628.0	18	57		100	NQ2-1											CORE		
DOLOMITE, GRAY, SLIGHTLY WEATHERED, VERY STRONG, SANDY LAMINAE, JOINTED, MODERATELY FRACTURED TO SLIGHTLY FRACTURED, TIGHT, ROUGH; RQD 86%, REC 100%. @20.1' TO 20.5': Qu=15,570 PSI, γ_{DRY}=158.6 PCF	621.8	23 EOB	95		100	NQ2-2											CORE		

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: AUGER CUTTINGS MIXED WITH 0.5 BAG BENTONITE CHIPS; PLACED 0.25 BAG QUICKCRETE

PROJECT: BRIDGE STREET BRIDGE
TYPE: BRIDGE
PID: N/A SFN: 8758948
START: 1/3/24 END: 1/3/24

DRILLING FIRM / OPERATOR: CT / CW
SAMPLING FIRM / LOGGER: CT / KKC
DRILLING METHOD: 3.25" HSA / NQ2
SAMPLING METHOD: SPT / ST / NQ2

DRILL RIG: CME 550X ATV
HAMMER: CME AUTOMATIC
CALIBRATION DATE: 2/20/23
ENERGY RATIO (%): 75.2

STATION / OFFSET: 59+66, 4' RT.
ALIGNMENT: BRIDGE STREET
ELEVATION: 645.7 (NAVD88) EOB: 20.5 ft.
LAT / LONG: 41.409108, -83.456498

EXPLORATION ID
B-004-0-23

PAGE
1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 645.7	DEPTHS
ASPHALT - 6.5 INCHES	645.1	
AGGREGATE BASE - 11.5 INCHES	644.2	
MEDIUM DENSE, GRAY, **GRAVEL AND STONE FRAGMENTS**, SOME SAND, LITTLE SILT, TRACE CLAY, DAMP FILL	643.2	
MEDIUM DENSE, GRAY, **GRAVEL AND STONE FRAGMENTS WITH SAND**, LITTLE SILT, TRACE CLAY, DAMP FILL	641.7	
STIFF TO VERY STIFF, BROWN, **SANDY SILT**, LITTLE CLAY, LITTLE GRAVEL, MOIST	638.7	
MEDIUM DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, LITTLE SILT, LITTLE DOLOMITE FRAGMENTS, TRACE CLAY, MOIST TO WET	636.7	TR
BROWN, **WEATHERED DOLOMITE**, SOME SAND, LITTLE SILT, TRACE CLAY	635.2	
DOLOMITE, GRAY, SLIGHTLY WEATHERED, STRONG, JOINTED - HIGHLY FRACTURED TO FRACTURED, TIGHT, ROUGH; RQD 15%, REC 100%.	629.5	
DOLOMITE, GRAY, SLIGHTLY WEATHERED, MODERATELY STRONG, JOINTED - SLIGHTLY FRACTURED, TIGHT, ROUGH; RQD 75%, REC 100%. @16.2' TO 17.0': Qu=5,560 PSI, γ_{DRY}=154.1 PCF	625.2	EOB

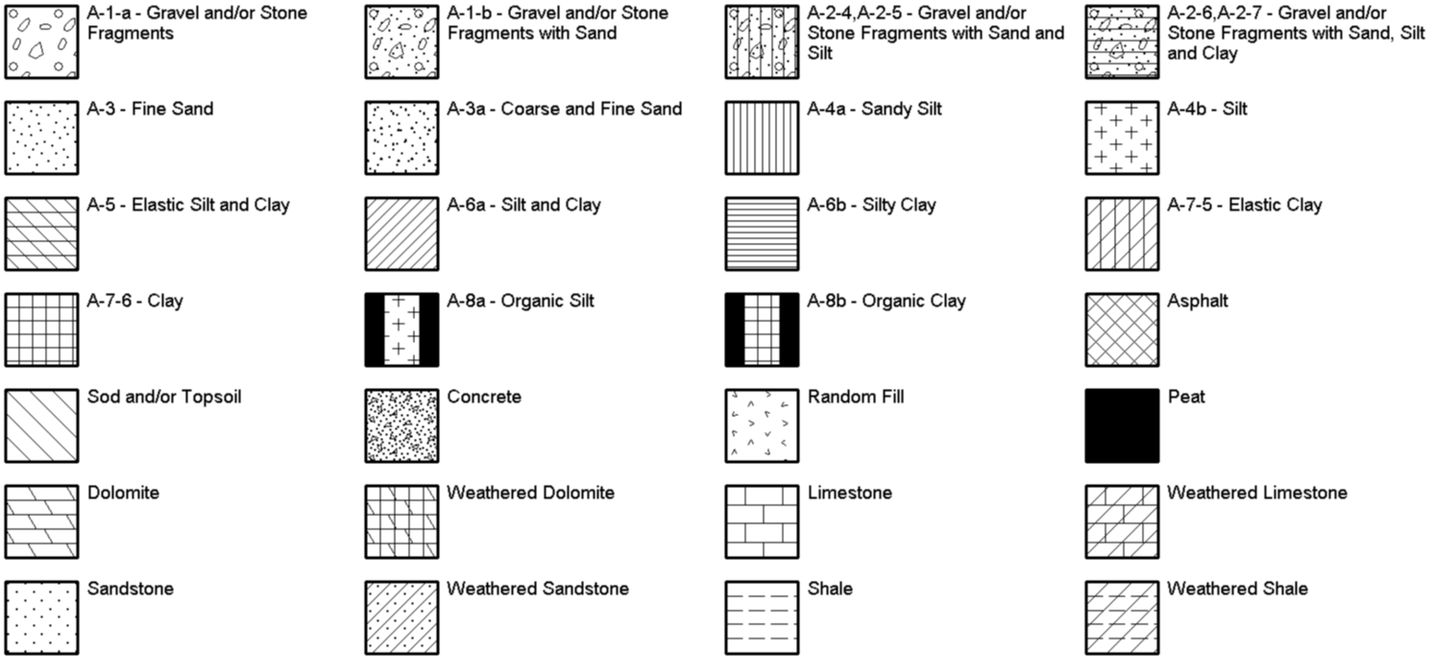
SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
22 12 7	24	89	SS-1	-	66	15	7	11	1	NP	NP	NP	4	A-1-a (0)	<100	
9 7 5	15	100	SS-2	-	50	13	15	19	3	NP	NP	NP	8	A-1-b (0)	-	
3 3 5	10	100	SS-3	2.75	11	2	26	43	18	22	15	7	18	A-4a (5)	-	
		0	ST-4A	-	-	-	-	-	-	-	-	-	-	A-4a (V)	-	
			ST-4B	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	-	
4 9 11	25	100	SS-5	-	48	19	10	19	4	NP	NP	NP	14	A-1-b (0)	-	
6 14 50/5"	-	100	SS-6	-	-	-	-	-	-	-	-	-	7	A-1-a (V)	-	
17		100	NQ2-1											CORE		
65		80	NQ2-2											CORE		

NOTES: NONE

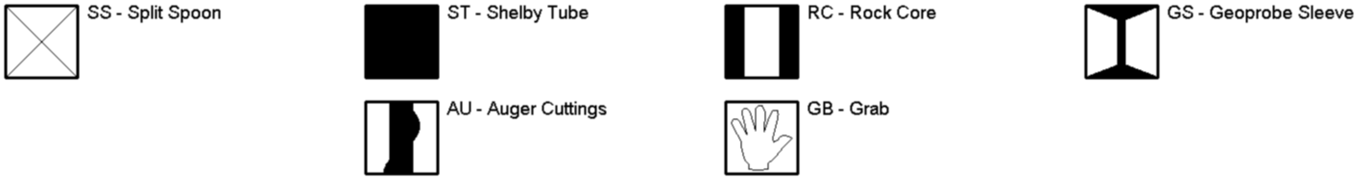
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.25 BAG ASPHALT PATCH; AUGER CUTTINGS MIXED WITH 1 BAG BENTONITE CHIPS

LEGEND KEY

Ohio Department of Transportation Soil Symbols



Sample Symbols



Notes:

1. Exploratory borings were drilled during the period from December 13, 2023 to January 4, 2024, using 3¼-inch diameter hollow-stem augers. Rock coring was performed using NQ-2 sized core barrel.

2. These logs are subject to the limitations, conclusions, and recommendations in the report and should not be interpreted separate from the report.

3. The borings were located in the field by CT in accordance with the Proposed Boring Location Plan, attached to the proposal.

4. Latitude, Longitude, ground surface elevation, stationing and offsets for all borings were provided by Tetratech based on field survey.

OHIO DEPARTMENT OF
TRANSPORTATION
DIVISION OF ENGINEERING

Office of Geotechnical Engineering

B-001-0-23

Core Date:	December 12 and 13, 2023				Ground Surface Elevation:	646.0'		
Run #:	Depth		Elevation		Recovery		RQD	
NQ2-1	12.0'	22.0'	634.0'	624.0'	48/120	40%	0/120	0%
NQ2-2	22.0'	27.0'	624.0'	619.0'	60/60	100%	24/60	40%

PROPOSED BRIDGE REPLACEMENT - BRIDGE STREET BRIDGE

Prepared by

CT Project No.: 231797

OHIO DEPARTMENT OF TRANSPORTATION
DIVISION OF ENGINEERING

Office of Geotechnical Engineering

B-002-0-23

Core Date:	January 4, 2024				Ground Surface Elevation: 646.0’			
Run #:	Depth		Elevation		Recovery		RQD	
NQ2-1	18.0'	23.0'	628.0'	623.0'	60/60	100%	18/60	30%
PROPOSED BRIDGE REPLACEMENT - BRIDGE STREET BRIDGE								

Prepared by CT

CT Project No.: 231797

B-002-0-23

Core Date:	January 4, 2024				Ground Surface Elevation: 628.2’			
Run #:	Depth		Elevation		Recovery		RQD	
NQ2-2	23.0'	28.0'	623.0'	618.0'	57/60	95%	45/60	75%
PROPOSED BRIDGE REPLACEMENT - BRIDGE STREET BRIDGE								

B-003-0-23

Core Date: January 4, 2024					Ground Surface Elevation: 646.0'			
Run #:	Depth		Elevation		Recovery		RQD	
NQ2-1	15.5'	20.5'	630.5'	625.5'	60/60	100%	34/60	57%
PROPOSED BRIDGE REPLACEMENT - BRIDGE STREET BRIDGE								

B-003-0-23

Core Date:	January 4, 2024				Ground Surface Elevation:	646.0'		
Run #:	Depth		Elevation		Recovery		RQD	
NQ2-2	20.5'	25.5'	625.5'	620.5'	60/60	100%	57/60	95%
PROPOSED BRIDGE REPLACEMENT - BRIDGE STREET BRIDGE								

Prepared by

OHIO DEPARTMENT OF TRANSPORTATION
DIVISION OF ENGINEERING

Office of Geotechnical Engineering

B-004-0-23

Core Date: January 3, 2024					Ground Surface Elevation: 646.0'			
Run #:	Depth		Elevation		Recovery		RQD	
NQ2-1	10.5'	15.5'	635.5'	630.5'	60/60	100%	10/60	17%
PROPOSED BRIDGE REPLACEMENT - BRIDGE STREET BRIDGE								

Prepared by

CT Project No.: 231797

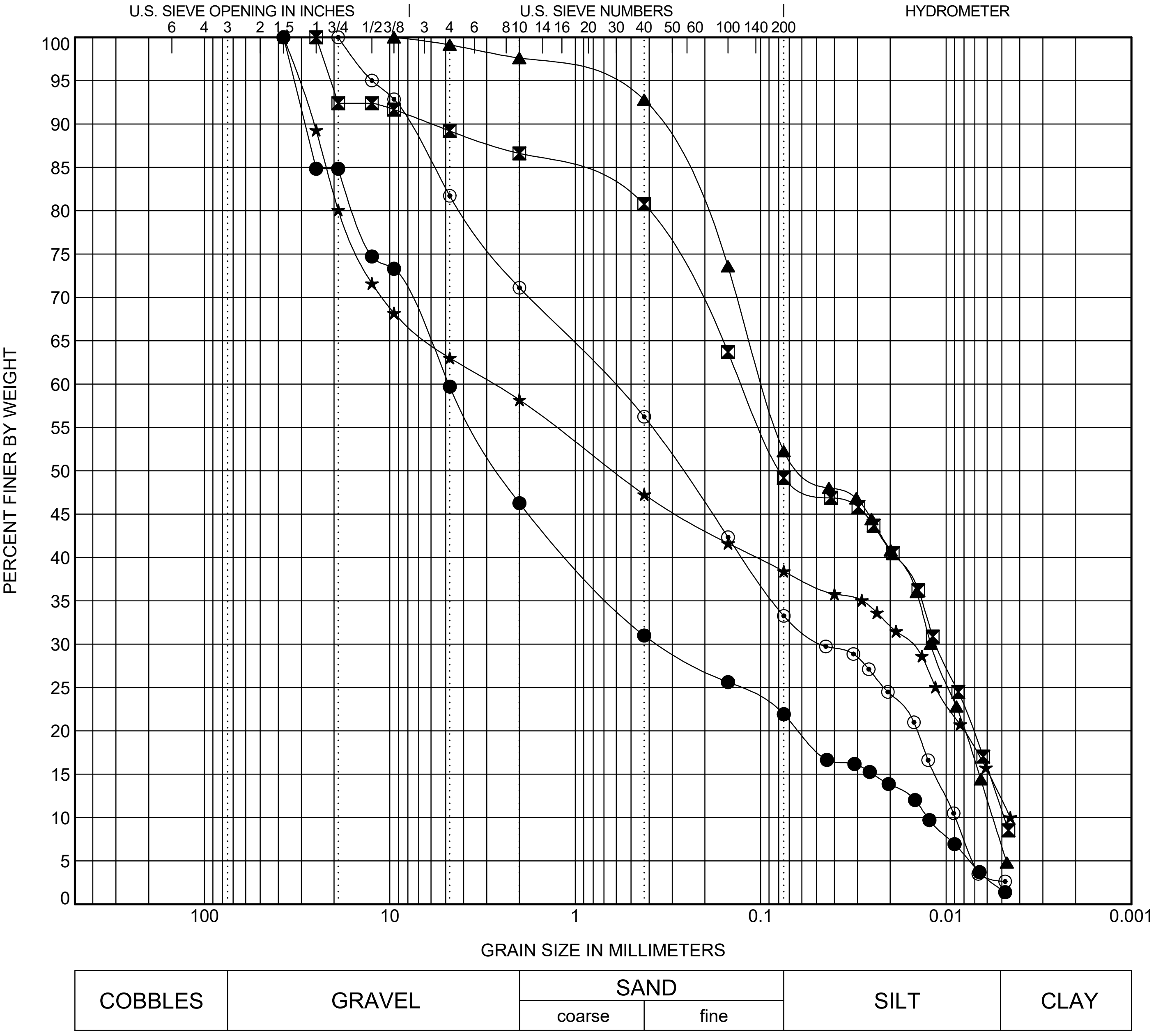
B-004-0-23

Core Date: January 3, 2024					Ground Surface Elevation: 646.0’			
Run #:	Depth		Elevation		Recovery		RQD	
NQ2-2	15.5'	20.5'	630.5'	625.5'	48/60	80%	39/60	65%
PROPOSED BRIDGE REPLACEMENT - BRIDGE STREET BRIDGE								

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT PEMBERVILLE BRDG. REPL.
PID 118539
OGE NUMBER N/A
PROJECT TYPE STRUCTURE FOUNDATION



Specimen Identification			ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
●	**B-001-0-23**	**1.0**	**A-1-b ~ SILTY GRAVEL with SAND(GM)**	**NP**	**NP**	**NP**
☒	**B-001-0-23**	**3.0**	**A-4a ~ SILTY, CLAYEY SAND(SC-SM)**	**22**	**17**	**5**
▲	**B-001-0-23**	**4.0**	**A-4a ~ SANDY LEAN CLAY(CL)**	**23**	**15**	**8**
★	**B-001-0-23**	**8.5**	**A-4a ~ SILTY, CLAYEY GRAVEL with SAND(GC-GM)**	**22**	**16**	**6**
⊙	**B-001-0-23**	**10.0**	**A-2-4 ~ SILTY, CLAYEY SAND with GRAVEL(SC-SM)**	**23**	**17**	**6**

Specimen Identification			D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
●	**B-001-0-23**	**1.0**	**28.699**	**2.544**	**0.351**	**0.013**	**54**	**15**	**9**	**20**	**2**	**2.03**	**383.25**
☒	**B-001-0-23**	**3.0**	**5.95**	**0.078**	**0.011**	**0.005**	**13**	**6**	**32**	**38**	**11**	**0.21**	**25.89**
▲	**B-001-0-23**	**4.0**	**0.365**	**0.056**	**0.012**	**0.006**	**2**	**5**	**41**	**45**	**7**	**0.27**	**17.22**
★	**B-001-0-23**	**8.5**	**25.678**	**0.627**	**0.016**		**42**	**11**	**9**	**26**	**12**		
⊙	**B-001-0-23**	**10.0**	**7.953**	**0.266**	**0.046**	**0.009**	**29**	**15**	**23**	**30**	**3**	**0.38**	**70.63**

Compressive Strength of Rock ASTM D 7012, Method C

PROJECT	Proposed Bridge Replacement - Bridge Street	CT PROJECT NUMBER	231797
LOCATION	Village of Pemberville, Ohio		
CLIENT	Tetra Tech		
BORING NUMBER	B-001-0-23	SAMPLE NUMBER	NQ2-2
SAMPLE DEPTH (FEET)	22.0 TO 27.0	SPECIMEN DEPTH (FEET)	24.5 TO 24.9

ROCK DESCRIPTION	DOLOMITE, GRAY, HIGHLY WEATHERED, STRONG, JOINTED - FRACTURED TO MODERATELY FRACTURED, TIGHT, ROUGH

LENGTH (INCHES)	3.47	MASS (GRAMS)	460.80
DIAMETER (INCHES)	1.99	UNIT WEIGHT (LBS/CU. FT.)	162.7
LENGTH / DIAMETER	1.74		
CORRECTION FACTOR	1.0	MAXIMUM LOAD (LBS)	34,870
AREA (SQ. IN.)	3.11	COMPRESSIVE STRENGTH (PSI)	10,990

TEST SPECIMEN PHOTO

TEST SPECIMEN PHOTO

Compressive Strength of Rock ASTM D 7012, Method C

PROJECT	Proposed Bridge Replacement - Bridge Street	CT PROJECT NUMBER	231797
LOCATION	Village of Pemberville, Ohio		
CLIENT	Tetra Tech		
BORING NUMBER	B-002-0-23	SAMPLE NUMBER	NQ2-1
SAMPLE DEPTH (FEET)	18.0 TO 23.0	SPECIMEN DEPTH (FEET)	21.2 TO 21.8

ROCK DESCRIPTION	DOLOMITE, GRAY, MODERATELY WEATHERED, MODERATELY STRONG, SANDY LAMINAE, JOINTED - FRACTURED TO MODERATELY FRACTURED, TIGHT, ROUGH

LENGTH (INCHES)	3.99	MASS (GRAMS)	474.20
DIAMETER (INCHES)	1.95	UNIT WEIGHT (LBS/CU. FT.)	151.6
LENGTH / DIAMETER	2.05		
CORRECTION FACTOR	1.0	MAXIMUM LOAD (LBS)	16,450
AREA (SQ. IN.)	2.99	COMPRESSIVE STRENGTH (PSI)	5,510

TEST SPECIMEN PHOTO

TEST SPECIMEN PHOTO

Compressive Strength of Rock ASTM D 7012, Method C

PROJECT	Proposed Bridge Replacement - Bridge Street	CT PROJECT NUMBER	231797
LOCATION	Village of Pemberville, Ohio		
CLIENT	Tetra Tech		
BORING NUMBER	B-003-0-23	SAMPLE NUMBER	NQ2-1
SAMPLE DEPTH (FEET)	15.5 TO 20.5	SPECIMEN DEPTH (FEET)	20.1 TO 20.5

ROCK DESCRIPTION	DOLOMITE, GRAY, SLIGHTLY WEATHERED, VERY STRONG, SANDY LAMINAE, JOINTED, MODERATELY FRACTURED TO SLIGHTLY FRACTURED, TIGHT, ROUGH;

LENGTH (INCHES)	3.74	MASS (GRAMS)	474.60
DIAMETER (INCHES)	1.97	UNIT WEIGHT (LBS/CU. FT.)	158.6
LENGTH / DIAMETER	1.90		
CORRECTION FACTOR	1.0	MAXIMUM LOAD (LBS)	47,950
AREA (SQ. IN.)	3.05	COMPRESSIVE STRENGTH (PSI)	15,570

TEST SPECIMEN PHOTO

TEST SPECIMEN PHOTO

Compressive Strength of Rock ASTM D 7012, Method C

Project	Proposed Bridge Replacement - Bridge Street	CT Project Number	231797
Location	Village of Pemberville, Ohio		
Client	Tetra Tech		
Boring Number	B-004-0-23	Sample Number	NQ2-2
Sample Depth (feet)	15.5 TO 20.5	Specimen Depth (feet)	16.2 TO 17.0

Rock Description	DOLOMITE, GRAY, SLIGHTLY WEATHERED, MODERATELY STRONG, JOINTED - SLIGHTLY FRACTURED, TIGHT, ROUGH

Length (inches)	4.00	Mass (grams)	493.20
Diameter (inches)	1.97	Unit Weight (lbs/cu. ft.)	154.1
Length / Diameter	2.03		
Correction Factor	1.0	Maximum Load (lbs)	16,960
Area (sq. in.)	3.05	Compressive Strength (psi)	5,560

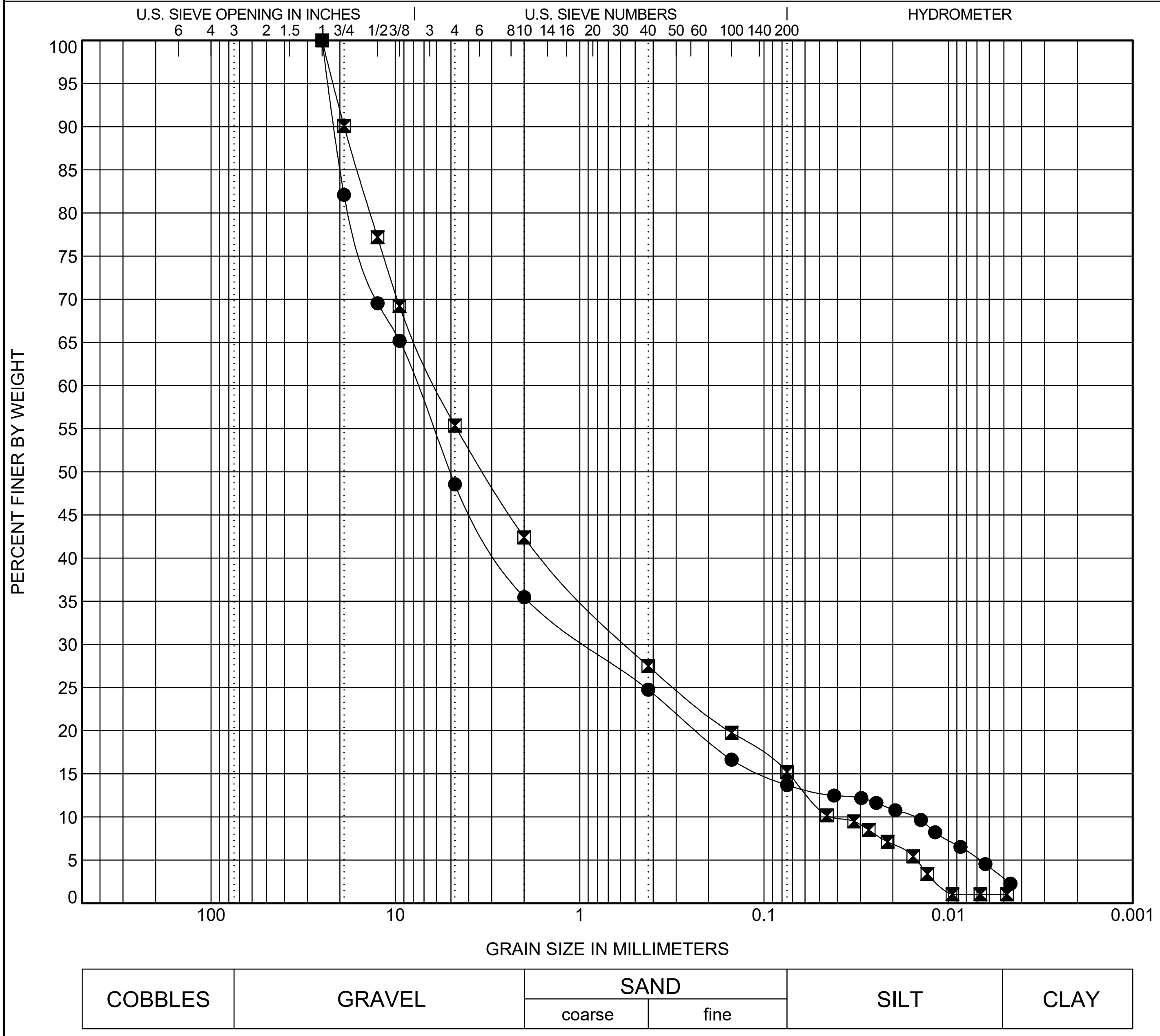
Test Specimen Photo

Test Specimen Photo

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT PEMBERVILLE BRDG. REPL.
PID 118539
OGE NUMBER N/A
PROJECT TYPE STRUCTURE FOUNDATION



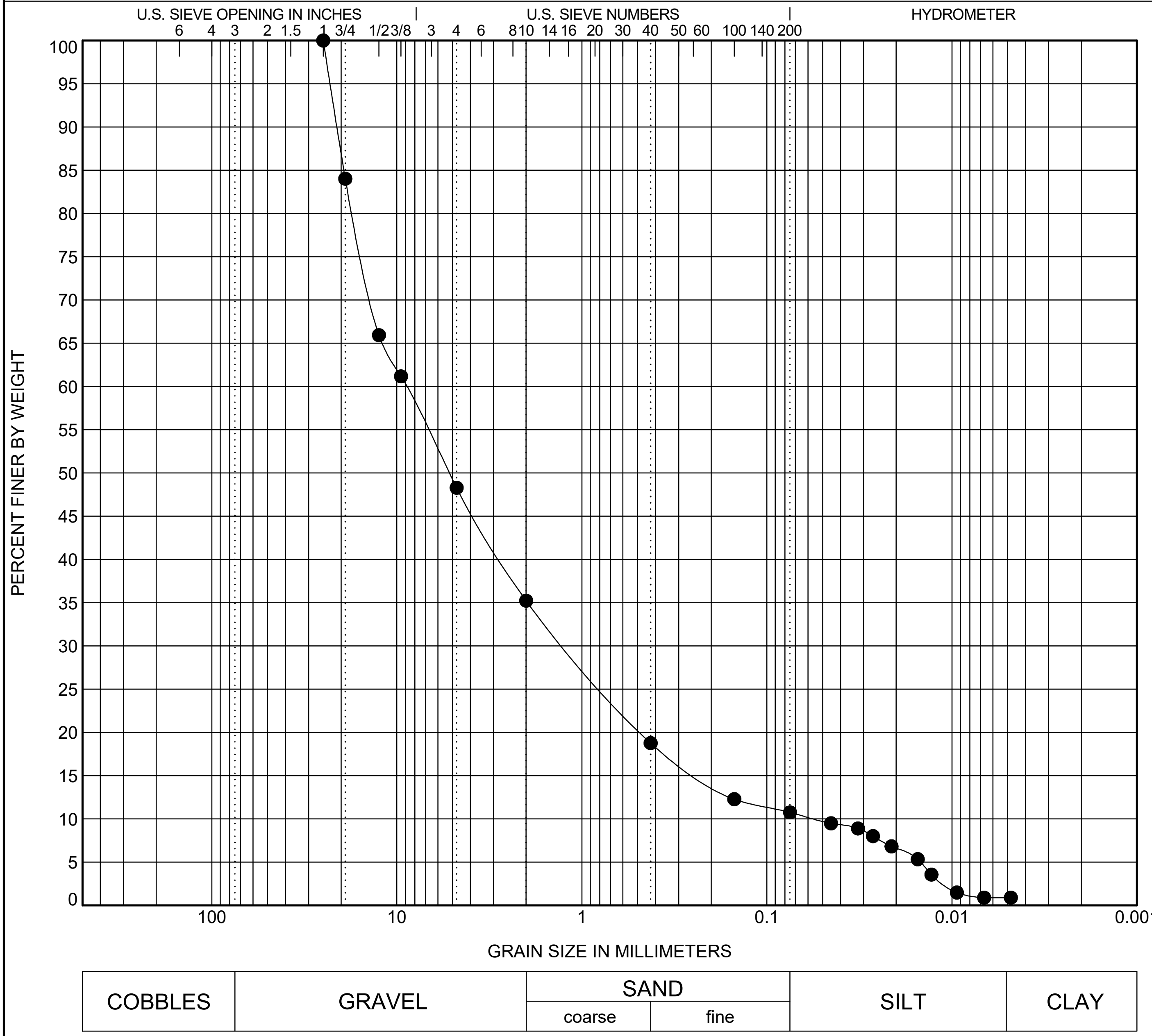
Specimen Identification		ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
●	**B-002-0-23 15.5**	**A-1-a ~ SILTY GRAVEL with SAND(GM)**	**NP**	**NP**	**NP**
☒	**B-002-0-23 17.0**	**A-1-a ~ SILTY GRAVEL with SAND(GM)**	**NP**	**NP**	**NP**

Specimen Identification		D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
●	**B-002-0-23 15.5**	**21.447**	**5.046**	**0.908**	**0.016**	**64**	**11**	**11**	**11**	**3**	**6.90**	**490.37**
☒	**B-002-0-23 17.0**	**18.942**	**3.32**	**0.552**	**0.042**	**58**	**15**	**12**	**14**	**1**	**1.22**	**143.29**

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT PEMBERVILLE BRDG. REPL.
PID 118539
OGE NUMBER N/A
PROJECT TYPE STRUCTURE FOUNDATION



	Specimen Identification		ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
●	**B-003-0-23**	**14.5**	**A-1-a ~ POORLY GRADED GRAVEL with SILT and SAND(GP-GM)**	**NP**	**NP**	**NP**

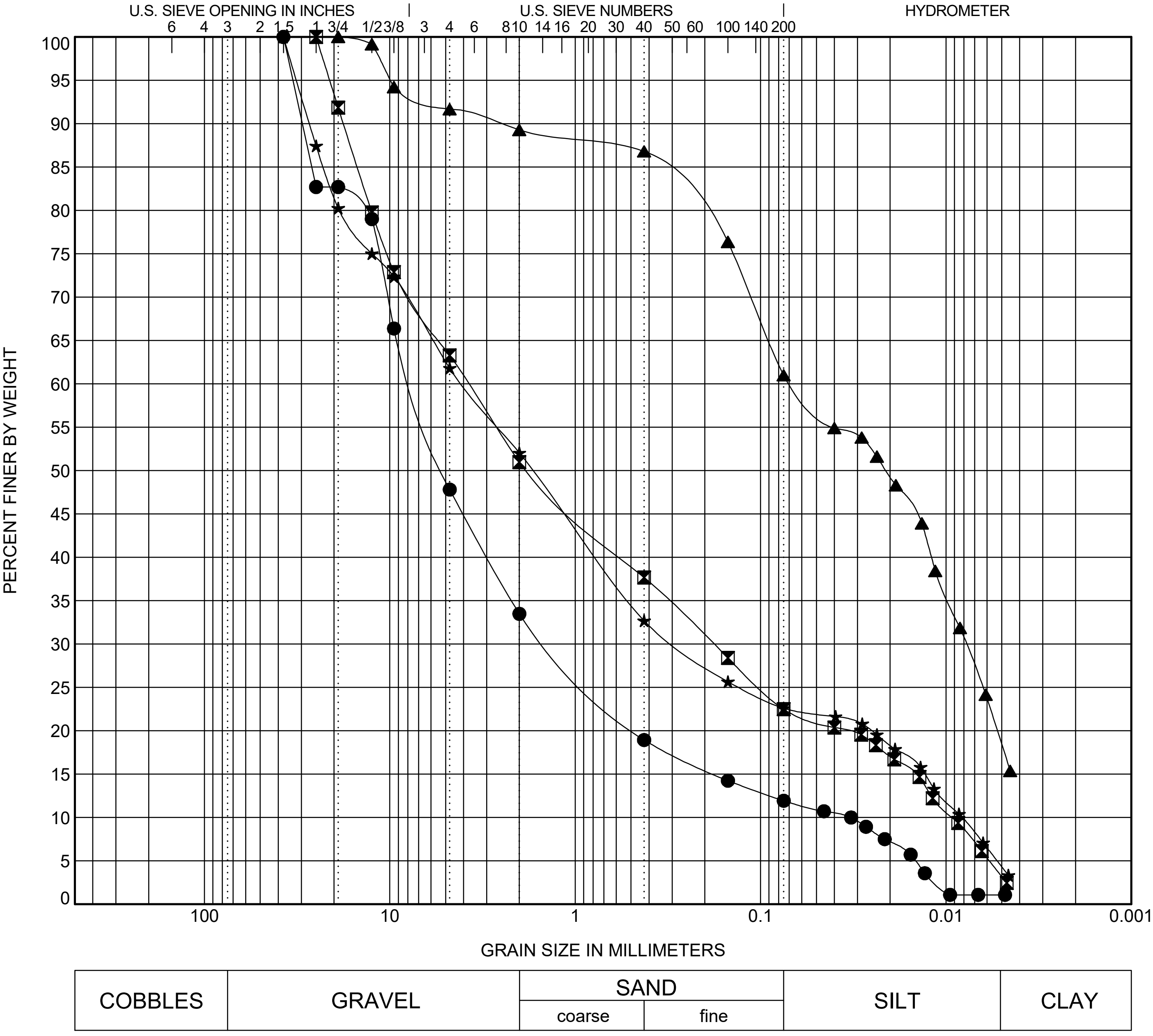
	Specimen Identification		D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
●	**B-003-0-23**	**14.5**	**21.057**	**5.209**	**1.224**	**0.055**	**65**	**16**	**8**	**10**	**1**	**3.03**	**161.04**

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT PEMBERVILLE BRDG. REPL. **PID** 118539

OGE NUMBER N/A **PROJECT TYPE** STRUCTURE FOUNDATION



	Specimen Identification		ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
●	**B-004-0-23**	**1.0**	**A-1-a ~ POORLY GRADED GRAVEL with SILT and SAND(GP-GM)**	**NP**	**NP**	**NP**
☒	**B-004-0-23**	**2.5**	**A-1-b ~ SILTY SAND with GRAVEL(SM)**	**NP**	**NP**	**NP**
▲	**B-004-0-23**	**4.0**	**A-4a ~ SANDY SILTY CLAY(CL-ML)**	**22**	**15**	**7**
★	**B-004-0-23**	**7.5**	**A-1-b ~ SILTY SAND with GRAVEL(SM)**	**NP**	**NP**	**NP**

	Specimen Identification		D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
●	**B-004-0-23**	**1.0**	**29.666**	**5.155**	**1.381**	**0.033**	**66**	**15**	**7**	**11**	**1**	**7.81**	**229.47**
☒	**B-004-0-23**	**2.5**	**17.814**	**1.789**	**0.18**	**0.009**	**50**	**13**	**15**	**19**	**3**	**0.93**	**409.24**
▲	**B-004-0-23**	**4.0**	**2.606**	**0.021**	**0.008**		**11**	**2**	**26**	**43**	**18**		
★	**B-004-0-23**	**7.5**	**27.151**	**1.699**	**0.286**	**0.008**	**48**	**19**	**10**	**19**	**4**	**2.46**	**493.85**

APPENDIX A

Engineering Calculations

Project Name: Proposed Bridge Replacement - Bridge Street Bridge
Project Numbe 231797
Calculated by: KCH 01/29/2024
Reviewed By: IJH 09/06/2024

Scour Determination - Rear (West) Abutment

Upper Elevation Limit for Analysis = 645 feet, based on 100-year floodplain
Lower Elevation Limit for Analysis = 624.5 feet, based on 6 feet below bottom of river

Table 1. Scour Parameters for Soils - Rear (West) Abutment

Boring Number	Sample Number	Sample Depth (feet)	Sample Approximate Elevation (feet)	ODOT Soil Class	Fines (<75 µm) (percent)	PI (percent)	w (percent)	q_u[1] (psf)	D_{50} (mm)	D_{95} (mm)	Critical Shear Stress, τ_c (psf)	Critical Shear Stress, τ_c (Pa)
B-001	SS-1	1.0 to 2.5	645.0 to 643.5	A-1-b	22	NP	8	-	2.5441	32.8057	0.053	2.54
B-001	SS-2B	3.0 to 4.0	643.0 to 642.0	A-4a	49	5	14	1,500	0.078	20.869	0.046	2.18
B-001	SS-3	4.0 to 5.5	642.0 to 640.5	A-4a	52	8	20	1,500	0.0556	0.8613	0.047	2.22
B-001	SS-6	8.5 to 10.0	637.5 to 636.0	A-4a	38	6	14	6,250	0.6266	31.031	0.063	2.95
B-001	SS-7	10.0 to 10.3	636.0 to 635.7	A-2-4	33	6	10	-	0.2664	12.4692	0.006	0.27

[1] For cohesive samples which were not intact for an unconfined compressive strength test or a hand penetrometer value, or where such tests were affected by desiccation, q_u was estimated by N_{60}x250.

Table 2. Scour Parameters for Rock - Rear (West) Abutment

Boring Number	Sample Number	Sample Depth (feet)	Sample Approximate Elevation (feet)	Unconfined Compressive Strength, Q_u (psi)	Slake Durability Index, S_{DI} (percent)	Rock Quality Designation, RQD (percent)	Unit Weight (pcf)	Rock Mass Rating, RMR (Superseded by GSI)	Geologic Strength Index, GSI	Erodibility Index, K	Critical Shear Stress, τ_c (psf)	Critical Shear Stress, τ_c (Pa)
B-001	NQ2-1	12.0 to 22.0	634.0 to 624.0	-	90.6	0	150	-	35 to 45	5	11.88	568.9

Scour Determination - Pier 1 (West)

Upper Elevation Limit for Analysis = 645 feet, based on 100-year floodplain
Lower Elevation Limit for Analysis = 624.5 feet, based on 6 feet below bottom of river

Project Name: Proposed Bridge Replacement - Bridge Street Bridge
Project Numbe 231797
Calculated by: KCH 01/29/2024
Reviewed By: IJH 09/06/2024

Table 3. Scour Parameters for Soils - Pier 1 (West)

Boring Number	Sample Number	Sample Depth (feet)	Sample Approximate Elevation (feet)	ODOT Soil Class	Fines (<75 µm) (percent)	PI (percent)	w (percent)	q_u (psf)	D_{50} (mm)	D_{95} (mm)	Critical Shear Stress, τ_c (psf)	Critical Shear Stress, τ_c (Pa)
B-002	SS-1	15.5 to 17.0	630.5 to 629.0	A-1-a	14	NP	11	-	5.046	23.1553	0.105	5.05
B-002	SS-2	17.0 to 17.6	629.0 to 628.4	A-1-a	15	NP	11	-	3.3205	21.7658	0.069	3.32

Table 4. Scour Parameters for Rock - Pier 1 (West)

Boring Number	Sample Number	Sample Depth (feet)	Sample Approximate Elevation (feet)	Unconfined Compressive Strength, Q_u (psi)	Slake Durability Index, S_{DI} (percent)	Rock Quality Designation, RQD (percent)	Unit Weight (pcf)	Rock Mass Rating, RMR (Superseded by GSI)	Geologic Strength Index, GSI	Erodibility Index, K	Critical Shear Stress, τ_c (psf)	Critical Shear Stress, τ_c (Pa)
B-002	NQ2-1	18.0 to 23.0	628.0 to 623.0	-	90.6	30	-	-	45 to 55	633	133.02	6,369.0

Project Name: Proposed Bridge Replacement - Bridge Street Bridge
Project Numbei 231797
Calculated by: KCH 01/29/2024
Reviewed By: IJH 09/06/2024

Scour Determination - Pier 2 (East)

Upper Elevation Limit for Analysis = 645 feet, based on 100-year floodplain
Lower Elevation Limit for Analysis = 624.5 feet, based on 6 feet below bottom of river

Table 5. Scour Parameters for Soils - Pier 2 (East)

Boring Number	Sample Number	Sample Depth (feet)	Sample Approximate Elevation (feet)	ODOT Soil Class	Fines (<75 µm) (percent)	PI (percent)	w (percent)	q_u (psf)	D_{50} (mm)	D_{95} (mm)	Critical Shear Stress, τ_c (psf)	Critical Shear Stress, τ_c (Pa)
B-003	SS-1	14.5 to 15.4	631.5 to 630.6	A-1-a	11	NP	11	-	5.2089	22.9437	0.109	5.21

Table 6. Scour Parameters for Rock - Pier 2 (East)

Boring Number	Sample Number	Sample Depth (feet)	Sample Approximate Elevation (feet)	Unconfined Compressive Strength, Q_u (psi)	Slake Durability Index, S_{DI} (percent)	Rock Quality Designation, RQD (percent)	Unit Weight (pcf)	Rock Mass Rating, RMR (Superseded by GSI)	Geologic Strength Index, GSI	Erodibility Index, K	Critical Shear Stress, τ_c (psf)	Critical Shear Stress, τ_c (Pa)
B-003	NQ2-1	15.5 to 20.5	630.5 to 625.5	-	90.6	57	-	-	60 to 70	3399	308.22	14,757.6
B-003	NQ2-2	20.5 to 25.5	625.5 to 620.5	15,570	90.6	95	158.6	-	70 to 90	5666	397.91	19,051.9

Project Name: Proposed Bridge Replacement - Bridge Street Bridge
Project Numbe 231797
Calculated by: KCH 01/29/2024
Reviewed By: IJH 09/06/2024

Scour Determination - Forward (East) Abutment

Upper Elevation Limit for Analysis = 645 feet, based on 100-year floodplain
Lower Elevation Limit for Analysis = 624.5 feet, based on 6 feet below bottom of river

Table 7. Scour Parameters for Soils - Forward (East) Abutment

Boring Number	Sample Number	Sample Depth (feet)	Sample Approximate Elevation (feet)	ODOT Soil Class	Fines (<75 µm) (percent)	PI (percent)	w (percent)	q_u[1] (psf)	D_{50} (mm)	D_{95} (mm)	Critical Shear Stress, τ_c (psf)	Critical Shear Stress, τ_c (Pa)
B-004	SS-1	1.0 to 2.5	645.0 to 643.5	A-1-a	12	NP	4	-	5.1554	33.3535	0.108	5.16
B-004	SS-2	2.5 to 4.0	643.5 to 642.0	A-1-b	22	NP	8	-	1.789	21.1207	0.037	1.79
B-004	SS-3	4.0 to 5.5	642.0 to 640.5	A-4a	61	7	18	2,500	0.021	9.9267	0.083	3.89
B-004	SS-5	8.5 to 10.0	637.5 to 636.0	A-1-b	23	NP	14	-	1.6987	31.9085	0.035	1.70

[1] For cohesive samples which were not intact for an unconfined compressive strength test or a hand penetrometer value, or where such tests were affected by desiccation, q_u was estimated by N_{60}x250.

Table 7. Scour Parameters for Rock - Forward (East) Abutment

Boring Number	Sample Number	Sample Depth (feet)	Sample Approximate Elevation (feet)	Unconfined Compressive Strength, Q_u (psi)	Slake Durability Index, S_{DI} (percent)	Rock Quality Designation, RQD (percent)	Unit Weight (pcf)	Rock Mass Rating, RMR (Superseded by GSI)	Geologic Strength Index, GSI	Erodibility Index, K	Critical Shear Stress, τ_c (psf)	Critical Shear Stress, τ_c (Pa)
B-004	NQ2-1	10.5 to 15.5	635.5 to 630.5	-	90.6	17	-	-	35 to 45	362	100.59	4,816.1
B-004	NQ2-2	15.5 to 20.5	630.5 to 625.5	10,990	90.6	65	154.1	-	60 to 70	1384	196.68	9,417.3

Project Name: Prop. Bridge Replacement - Bridge Street

Drilled Shaft Socket Evaluations - Load and Resistance Factor Design

Project Number: 231797

Calculated by: KCH 2/9/2024

Reviewed by: CPI 2/15/2024

Boring: B-001-0-23 - Rear (West) Abutment

shaft diameter, B = 3 feet = 36 inches

socket diameter, B = 2.5 feet = 30 inches

minimum socket length 5 feet

GSE = 645.8 feet

TR = 12 feet

TR Elev.(ft)= 633.8

+5 feet = 17 feet

Tip Elev. 628.80 feet

$$q_p = 2.5q_u \qquad (10.8.3.5.4c\text{-}1)$$

qu = 10,990 psi B-001 (NQ2-2 Sample), for rock zone starting approximately 8 feet below tip.

However, for nearest boring without RQD=0 at tip elevation

qu = 5,510 psi B-002 (NQ2-2 Sample) Conservatively use this value

= 793,440 psf

qp = 2.5*qu = 1,983,600 psf = 1984 ksf

Resistance Factor, phi = 0.5 Table 10.5.5.2.4-1, tip resistance in rock

phi*qp = **992** **ksf**

A*phi*qp = **4,868** **kips** tip only, 10.8.3.5.4c-1 method, use for design

consider strength of concrete

f'c = 4000 psi

= 576 ksf

0.33*f'c*A = 933 kips allowable strength of concrete; structural engineer to consider any limiting conditions associated with the stress limitations of the concrete

Available Resistance (kips)= 4868

Based on provided loading

Indicated Total Factored Load (kips)= 247

Suitable Vertical Resistance? YES

Project Name: Prop. Bridge Replacement - Bridge Street

Drilled Shaft Socket Evaluations - Load and Resistance Factor Design

Project Number: 231797

Calculated by: KCH 2/9/2024

Reviewed by: CPI 2/15/2024

Boring: B-002-0-23 - Pier 1 (West Pier)

shaft diameter, B = 3.5 feet = 42 inches

socket diameter, B = 3.5 feet = 42 inches

minimum socket length 5 feet

very little overburden,
do not size down 6 inches

GSE = 647.4 feet

TR = 18 feet

TR Elev.(ft)= 629.4

+5 feet = 23 feet

Tip Elev. 624.40 feet

$$q_p = 2.5q_u \qquad (10.8.3.5.4c\text{-}1)$$

qu = 5,510 psi B-002 (NQ2-1 Sample)

= 793,440 psf

qp = 2.5*qu = 1,983,600 psf = 1984 ksf

Resistance Factor, phi = 0.5 Table 10.5.5.2.4-1, tip resistance in rock

phi*qp = **992** **ksf**

A*phi*qp = **9,542** **kips** tip only, 10.8.3.5.4c-1 method, use for design

consider strength of concrete

f'c = 4000 psi

= 576 ksf

0.33*f'c*A = 1829 kips allowable strength of concrete; structural engineer to consider any limiting conditions associated with the stress limitations of the concrete

Available Resistance (kips)= 9542

Based on provided loading

Indicated Total Factored Load (kips)= 456

Suitable Vertical Resistance? YES

Project Name: Prop. Bridge Replacement - Bridge Street

Drilled Shaft Socket Evaluations - Load and Resistance Factor Design

Project Number: 231797

Calculated by: KCH 2/9/2024

Reviewed by: CPI 2/15/2024

Boring: B-003-0-23 - Pier 2 (East Pier)

shaft diameter, B =	3.5	feet	= 42	inches	(N/A, see below)
socket diameter, B =	3.5	feet	= 42	inches	
minimum socket length	5	feet		no overburden, do not size down 6 inches	
GSE =	647.3	feet			
TR =	15.5	feet	TR Elev.(ft)=	631.8	
+5 feet =	20.5	feet			
Tip Elev.	626.80	feet			

$$q_p = 2.5 q_u \qquad (10.8.3.5.4c\text{-}1)$$

qu =	15,570	psi	B-003 (NQ2-1 Sample)		
=	2,242,080	psf			
qp = 2.5*qu =	5,605,200	psf	=	5605	ksf
Resistance Factor, phi =		0.5	Table 10.5.5.2.4-1, tip resistance in rock		
phi*qp =	**2,803**	**ksf**			
A*phi*qp =	**26,964**	**kips**	tip only, 10.8.3.5.4c-1 method, use for design		

consider strength of concrete

f'c =	4000	psi	
=	576	ksf	
0.33*f'c*A =	1829	kips	allowable strength of concrete; structural engineer to consider any limiting conditions associated with the stress limitations of the concrete

Available Resistance (kips)= 26964

Based on provided loading

Indicated Total Factored Load (kips)= 456

Suitable Vertical Resistance? YES

Project Name: Prop. Bridge Replacement - Bridge Street

Drilled Shaft Socket Evaluations - Load and Resistance Factor Design

Project Number: 231797
Calculated by: KCH 2/9/2024
Reviewed by: CPI 2/15/2024
Boring: B-004-0-23 - Forward (East) Abutment

shaft diameter, B =	3	feet	= 36	inches
socket diameter, B =	2.5	feet	= 30	inches
minimum socket length	5	feet		
GSE =	645.7	feet		
TR =	10.5	feet	TR Elev.(ft)=	635.2
+5 feet =	15.5	feet		
Tip Elev.	630.20	feet		

$$q_p = 2.5 q_u \quad (10.8.3.5.4c\text{-}1)$$

qu = 5,560 psi B-004 (NQ2-2 Sample)
= 800,640 psf

qp = 2.5*qu = 2,001,600 psf = 2002 ksf

Resistance Factor, phi = 0.5 Table 10.5.5.2.4-1, tip resistance in rock
phi*qp = **1,001** **ksf**
A*phi*qp = **4,913** **kips** tip only, 10.8.3.5.4c-1 method, use for design

consider strength of concrete
f'c = 4000 psi
= 576 ksf
0.33*f'c*A = 933 kips allowable strength of concrete; structural engineer to consider any limiting conditions associated with the stress limitations of the concrete

Available Resistance (kips)= 4913
Based on provided loading
Indicated Total Factored Load (kips)= 247
Suitable Vertical Resistance? YES

Project Name: Prop. Bridge Replacement - Bridge Street
Project Number: 231797
Calculated by: KCH 2/9/2024 Reviewed by: CPI 2/15/2024

Calcs: **Drilled Shaft Rock Sockets - Lateral Resistance**
Location: B-001-0-23 - Rear (West) Abutment

GSE (ft): 645.8
Long-Term GWT (ft): 633 approximate river elevation
Bottom of Pier Cap Elev. (ft): 636.8 assumed, based on existing bridge plans

Soil

Layer	**Soil Type**	**Top Depth (ft)**	**Bottom Depth (ft)**	**Top Elev. (ft)**	**Bottom Elev. (ft)**	**N60**	**HP (tsf)**	**Qu (tsf)**
Layer 1	**Very Stiff A-4b**	8.5	10	637.3	635.8	25	-	-
			Depth below bottom of Pier Cap:	-0.5	1			
Total Unit Wt (pcf):	125	GDM Table 400-4		**Use**	**125**	**pcf**		

Su = N60 x 125 (N60<= 52 bpf) per GDM 404.1
Su (ksf)= **3.125**

Evaluation of Strain at half stress (epsilon 50) from LPILE 2018 Technical Manual
P-Y Curve Type:
Su = 2-4 ksf, epsilon 50 = **0.005** **Stiff Clay w/o Free Water (Reese)**

Augerable Weathered Bedrock

Layer	**Rock Type**	**Top Depth (ft)**	**Bottom Depth (ft)**	**Top Elev. (ft)**	**Bottom Elev. (ft)**	**SPT Result**
Layer 2	**Weathered Dolomite**	10	12	635.8	633.8	50/4"
			Depth below bottom of Pier Cap:	1	3	
Total Unit Wt (pcf):	165-175	GDM Table 400-5		**Use**	**155**	**pcf**
	157	Average of Tested Values for the project.				

Qu based on SPT Results per GDM 404.3
Qu (ksf)=0.092x(Nrate)90 (bpf)
ER(%)= 75.2
N75.2 = 150 bpf
N90 = 75.2/90 x 150 bpf = 125 bpf
Qu (ksf) = 11.5
Qu (psi) = 80

Estimate E based on GDM Table 400-6
Lowest Qu = 200 psi, indicated as E = 18,000 psi
Use E (psi) = **18,000**

If Strain at 18000 psi is 1%, then strain at half max stress (krm) is calculated by:
Half max stress = Qu/2 = 40 psi
krm = 1% x (40 psi / 18000 psi) = 0.0022 %
krm (decimal format) = **0.000022** P-Y Curve Type, RQD =0%:
Weak Rock (Reese)

Bedrock

Layer	**Soil Type**	**Top Depth (ft)**	**Bottom Depth (ft)**	**Top Elev. (ft)**	**Bottom Elev. (ft)**	**RQD (%)**	**Rec (%)**	**Qu (psi)**
Layer 3	**Dolomite - Strong**	12	22	633.8	623.8	0	40	No Test
	Highly Frac. to Frac.							
			Depth below bottom of Pier Cap:	3	13			
Total Unit Wt (pcf):	165-175	GDM Table 400-5		**Use**	**155**	**pcf**		
	157	Average of Tested Values for the project.						

Qu (psi)= **5510** Conservatively using lower value from close B-002 tested specimen instead of relatively high value from deeper sample in B-001 or higher average value of all samples
From GDM Table 400-6, say E (psi) = **450000**

If Strain at 450000 psi is 1%, then strain at half max stress (krm) is calculated by:
Half max stress = Qu/2 = 2755 psi
krm = 1% x (2755 psi / 450000 psi) = 0.0061 %
krm (decimal format) = **0.000061** P-Y Curve Type:
Weak Rock (Reese)

Calcs:	**Drilled Shaft Rock Sockets - Lateral Resistance**									
Location:	B-001-0-23 - Rear (West) Abutment									
Layer	**Soil Type**	**Top Depth (ft)**	**Bottom Depth (ft)**	**Top Elev. (ft)**	**Bottom Elev. (ft)**	**RQD (%)**	**Rec (%)**	**Qu (psi)**	**Total Unit Wt (pcf)**	
Layer 4	**Dolomite - Strong**	22	27	623.8	618.8	40	100	10990	162.7	at 24.5 ft
	Frac. To Mod Frac.									
		Depth below bottom of Pier Cap:		13	18					
Total Unit Wt (pcf):	165-175	GDM Table 400-5		**Use**	**160**	**pcf**				
	162.7	Tested Value								
Qu (psi)=	**10990**	Tested value								
From GDM Table 400-6, say E (psi) =	**900000**									
If Strain at 900000 psi is 1%, then strain at half max stress (krm) is calculated by:										
Half max stress = Qu/2 =	5495	psi								
krm = 1% x (5495 psi / 900000 psi) =	0.0061	%								
krm (decimal format) =	**0.000061**	P-Y Curve Type:								
		Weak Rock (Reese)								

Project Name: Prop. Bridge Replacement - Bridge Street
Project Number: 231797
Calculated by: KCH 2/12/2024 Reviewed by: CPI 2/16/2024

Calcs: Drilled Shaft Rock Sockets - Lateral Resistance
Location: B-002-0-23 - Pier 1 (West Pier)

GSE (ft): 647.4
Long-Term GWT (ft): 633 approximate river elevation
Bottom of Pier Cap Elev. (ft): 629.4 assumed, based on existing bridge plans

Soil

Layer	**Soil Type**	**Top Depth (ft)**	**Bottom Depth (ft)**	**Top Elev. (ft)**	**Bottom Elev. (ft)**	**N60**	**HP (tsf)**	**Qu (tsf)**
Layer 1	**Medium Dense A-1-a**	15.5	17	631.9	630.4	26	-	-
		Depth below bottom of Pier Cap:		-2.5	-1			
Total Unit Wt (pcf):	128	GDM Table 400-4		**Use**	**125**	**pcf**		
Internal Angle of Friction Determination (GDM 404.2):								
N160 (bpf)=CN*N60	AASHTO LRFD 10.4.6.2.4							
CN=0.77log(40/sigma-v'), with CN<2.0								
CN at	16.25	ft						
sigma-v' (ksf):	0.05							
CN=	2.3	>2 so use:	2.0					
N160 (bpf)=	52							
AASHTO LRFD Table 10.4.6.2.4-1								
N160	Mid-Range Phi (deg)							
50	40.5							
N160	Phi (deg)							
52	40.5	use	40.5	deg				
GDM Table 400-3 phi Adjustment								
A-1-a	+2.5							
Phi (deg) =	**43**	< ODOT Maximum 46 deg, ok						
k Evaluation From LPILE 2018 Technical Manual								
Parameters:	Medium Dense, Submerged Sand							
Range of k-value (pci) =	8.0 - 27.0							
Med Dense range of N60	k (pci)							
11	8							
30	27							
Interpolate for 26 bpf for this layer:	23.0	P-Y Curve Type:						
Say k (pci) =	**23**	**Sand (Reese)**						

Augerable Weathered Bedrock

Layer	**Rock Type**	**Top Depth (ft)**	**Bottom Depth (ft)**	**Top Elev. (ft)**	**Bottom Elev. (ft)**	**SPT Result**
Layer 2	**Weathered Dolomite**	17	18	630.4	629.4	50/1"
		Depth below bottom of Pier Cap:		-1	0	
Total Unit Wt (pcf):	165-175	GDM Table 400-5		**Use**	**155**	**pcf**
	157	Average of Tested Values for the project.				
Qu based on SPT Results per GDM 404.3						
Qu (ksf)=0.092x(Nrate)90 (bpf)						
ER(%)=	75.2					
N75.2 =	600	bpf				
N90 = 75.2/90 x 600 bpf =	501	bpf				
Qu (ksf) =	46.1					
Qu (psi) =	320					
Estimate E based on GDM Table 400-6						
Qu (psi)	Ei (psi)					
200	18000					
360	32000					
Interpolate for 320 psi for this layer:	28526					
Use E (psi) =	**28,525**					
If Strain at 28525 psi is 1%, then strain at half max stress (krm) is calculated by:						
Half max stress = Qu/2 =	160	psi				
krm = 1% x (160 psi / 28525 psi) =	0.0056	%				
krm (decimal format) =	**0.000056**	P-Y Curve Type, RQD =0%:				
		Weak Rock (Reese)				

Bedrock

Calcs:	**Drilled Shaft Rock Sockets - Lateral Resistance**									
Location:	B-002-0-23 - Pier 1 (West Pier)									
Layer	**Soil Type**	**Top Depth (ft)**	**Bottom Depth (ft)**	**Top Elev. (ft)**	**Bottom Elev. (ft)**	**RQD (%)**	**Rec (%)**	**Qu (psi)**		
Layer 3	**Dolomite - Strong**	18	20.7	629.4	626.7	13	100	No Test		
	Highly Frac. to Frac.									
			Depth below bottom of Pier Cap:	0	2.7					
Total Unit Wt (pcf):	165-175	GDM Table 400-5		**Use**	**155**	**pcf**				
	157	Average of Tested Values for the project.								
Qu (psi)=	**5510**	Value for tested sample deeper in B-002								
From GDM Table 400-6, say E (psi) =	**450000**									
If Strain at 450000 psi is 1%, then strain at half max stress (krm) is calculated by:										
Half max stress = Qu/2 =	2755	psi								
krm = 1% x (2755 psi / 450000 psi) =	0.0061	%								
krm (decimal format) =	**0.000061**	P-Y Curve Type:								
		Weak Rock (Reese)								
Layer	**Soil Type**	**Top Depth (ft)**	**Bottom Depth (ft)**	**Top Elev. (ft)**	**Bottom Elev. (ft)**	**RQD (%)**	**Rec (%)**	**Qu (psi)**	**Total Unit Wt (pcf)**	
Layer 4	**Dolomite - Strong**	20.7	28	626.7	619.4	67	100	5510	151.6	at 21.2 ft
	Frac. To Mod Frac.									
			Depth below bottom of Pier Cap:	2.7	10					
Total Unit Wt (pcf):	165-175	GDM Table 400-5		**Use**	**155**	**pcf**				
	151.6	Tested Value								
	157	Average of Tested Values for the project.								
Qu (psi)=	**5510**	Tested value								
From GDM Table 400-6, say E (psi) =	**450000**									
If Strain at 450000 psi is 1%, then strain at half max stress (krm) is calculated by:										
Half max stress = Qu/2 =	2755	psi								
krm = 1% x (2755 psi / 450000 psi) =	0.0061	%								
krm (decimal format) =	**0.000061**	P-Y Curve Type:								
		Weak Rock (Reese)								

Project Name: Prop. Bridge Replacement - Bridge Street
Project Number: 231797
Calculated by: KCH 2/12/2024 Reviewed by: CPI 2/16/2024

Calcs: **Drilled Shaft Rock Sockets - Lateral Resistance**
Location: B-003-0-23 - Pier 2 (East Pier)

GSE (ft): 647.3
Long-Term GWT (ft): 633 approximate river elevation
Bottom of Pier Cap Elev. (ft): 629.3 assumed, based on existing bridge plans

Augerable Weathered Bedrock

Layer	Rock Type	Top Depth (ft)	Bottom Depth (ft)	Top Elev. (ft)	Bottom Elev. (ft)	SPT Result
Layer 2	**Weathered Dolomite**	14.5	15.5	632.8	631.8	50/5"
			Depth below bottom of Pier Cap:	-3.5	-2.5	
Total Unit Wt (pcf):	165-175	GDM Table 400-5		**Use**	**155**	**pcf**
	157	Average of Tested Values for the project.				
Qu based on SPT Results per GDM 404.3						
Qu (ksf)=0.092x(Nrate)90 (bpf)						
ER(%)=	75.2					
N75.2 =	120	bpf				
N90 = 75.2/90 x 120 bpf =	100	bpf				
Qu (ksf) =	9.2					
Qu (psi) =	64					
Estimate E based on GDM Table 400-6						
Lowest Qu = 200 psi, indicated as E = 18,000 psi						
Use E (psi) =	**18,000**					
If Strain at 18000 psi is 1%, then strain at half max stress (krm) is calculated by:						
Half max stress = Qu/2 =	32	psi				
krm = 1% x (32 psi / 18000 psi) =	0.0018	%				
krm (decimal format) =	**0.000018**	P-Y Curve Type, RQD =0%:				
		Weak Rock (Reese)				

Calcs:	**Drilled Shaft Rock Sockets - Lateral Resistance**									
Location:	B-003-0-23 - Pier 2 (East Pier)									
Bedrock										
Layer	**Soil Type**	**Top Depth (ft)**	**Bottom Depth (ft)**	**Top Elev. (ft)**	**Bottom Elev. (ft)**	**RQD (%)**	**Rec (%)**	**Qu (psi)**		
Layer 3	**Dolomite - Strong**	15.5	19.3	631.8	628	59	100	No Test		
	Frac. to Mod. Frac.									
			Depth below bottom of Pier Cap:	-2.5	1.3					
Total Unit Wt (pcf):	165-175	GDM Table 400-5		**Use**	**155**	**pcf**				
	157	Average of Tested Values for the project.								
Qu (psi)=	**9408**	Average of Tested Values for the project.								
From GDM Table 400-6, say E (psi) =	**900000**									
If Strain at 900000 psi is 1%, then strain at half max stress (krm) is calculated by:										
Half max stress = Qu/2 =	4704	psi								
krm = 1% x (4704 psi / 900000 psi) =	0.0052	%								
krm (decimal format) =	**0.000052**	P-Y Curve Type:								
		Weak Rock (Reese)								
Layer	**Soil Type**	**Top Depth (ft)**	**Bottom Depth (ft)**	**Top Elev. (ft)**	**Bottom Elev. (ft)**	**RQD (%)**	**Rec (%)**	**Qu (psi)**	**Total Unit Wt (pcf)**	
Layer 4	**Dolomite - Strong**	19.3	25.5	628	621.8	86	100	15570	158.6	at 20.1 ft
	Frac. To Mod Frac.									
			Depth below bottom of Pier Cap:	1.3	7.5					
Total Unit Wt (pcf):	165-175	GDM Table 400-5		**Use**	**155**	**pcf**				
	158.6	Tested Value								
Qu (psi)=	**15570**	Tested value								
From GDM Table 400-6, say E (psi) =	**1400000**									
If Strain at 1400000 psi is 1%, then strain at half max stress (krm) is calculated by:										
Half max stress = Qu/2 =	7785	psi								
krm = 1% x (7785 psi / 1400000 psi) =	0.0056	%								
krm (decimal format) =	**0.000056**	P-Y Curve Type:								
		Weak Rock (Reese)								

Project Name: Prop. Bridge Replacement - Bridge Street
Project Number: 231797
Calculated by: KCH 2/9/2024
Reviewed by: CPI 2/16/2024

Calcs: Drilled Shaft Rock Sockets - Lateral Resistance
Location: B-004-0-23 - Forward (East) Abutment

GSE (ft): 645.7
Long-Term GWT (ft): 633 approximate river elevation
Bottom of Pier Cap Elev. (ft): 636.7 assumed, based on existing bridge plans

Soil

Layer	Soil Type	Top Depth (ft)	Bottom Depth (ft)	Top Elev. (ft)	Bottom Elev. (ft)	N60	HP (tsf)	Qu (tsf)
Layer 1	**Medium Dense A-1-b**	7	9	638.7	636.7	25	-	-
			Depth below bottom of Pier Cap:	-2	0			
Total Unit Wt (pcf):	128	GDM Table 400-4		**Use**	**125**	**pcf**		
Internal Angle of Friction Determination (GDM 404.2):								
N160 (bpf)=CN*N60	AASHTO LRFD 10.4.6.2.4							
CN=0.77log(40/sigma-v'), with CN<2.0								
CN at	8	ft						
sigma-v' (ksf):	1.00							
CN=	1.2	<2 so use:	1.2					
N160 (bpf)=	31							
AASHTO LRFD Table 10.4.6.2.4-1								
N160	Mid-Range Phi (deg)							
30	37.5							
50	40.5							
N160	Phi (deg)							
31	37.6	use	37.5	deg				
GDM Table 400-3 phi Adjustment								
A-1-b	+1.5							
Phi (deg) =	**39**	< ODOT Maximum 46 deg, ok						
k Evaluation From LPILE 2018 Technical Manual								
Parameters:	Medium Dense, Moist Sand							
Range of k-value (pci) =	13.0 - 40.0							
Med Dense range of N60	k (pci)							
11	13							
30	40							
Interpolate for 25 bpf for this layer:	33	P-Y Curve Type:						
Say k (pci) =	**33**	**Sand (Reese)**						

Augerable Weathered Bedrock

Layer	Rock Type	Top Depth (ft)	Bottom Depth (ft)	Top Elev. (ft)	Bottom Elev. (ft)	SPT Result
Layer 2	**Weathered Dolomite**	9	10.5	636.7	635.2	50/5"
			Depth below bottom of Pier Cap:	0	1.5	
Total Unit Wt (pcf):	165-175	GDM Table 400-5		**Use**	**155**	**pcf**
	157	Average of Tested Values for the project.				
Qu based on SPT Results per GDM 404.3						
Qu (ksf)=0.092x(Nrate)90 (bpf)						
ER(%)=	75.2					
N75.2 =	120	bpf				
N90 = 75.2/90 x 120 bpf =	100	bpf				
Qu (ksf) =	9.2					
Qu (psi) =	64					
Estimate E based on GDM Table 400-6						
Lowest Qu = 200 psi, indicated as E = 18,000 psi						
Use E (psi) =	**18,000**					
If Strain at 18000 psi is 1%, then strain at half max stress (krm) is calculated by:						
Half max stress = Qu/2 =	32.0	psi				
krm = 1% x (32 psi / 18000 psi) =	0.0018	%				
krm (decimal format) =	**0.000018**	P-Y Curve Type, RQD =0%:				
		Weak Rock (Reese)				

Calcs:	**Drilled Shaft Rock Sockets - Lateral Resistance**									
Location:	B-004-0-23 - Forward (East) Abutment									
Bedrock										
Layer	**Soil Type**	**Top Depth (ft)**	**Bottom Depth (ft)**	**Top Elev. (ft)**	**Bottom Elev. (ft)**	**RQD (%)**	**Rec (%)**	**Qu (psi)**		
Layer 3	**Dolomite - Strong**	10.5	16.2	635.2	629.5	15	100	No Test		
	Highly Frac. to Frac.									
		Depth below bottom of Pier Cap:		1.5	7.2					
Total Unit Wt (pcf):	165-175	GDM Table 400-5		**Use**	**155**	**pcf**				
	157	Average of Tested Values for the project.								
Qu (psi)=	**5560**	Value for tested sample deeper in B-002								
From GDM Table 400-6, say E (psi) =	**450000**									
If Strain at 450000 psi is 1%, then strain at half max stress (krm) is calculated by:										
Half max stress = Qu/2 =	2780	psi								
krm = 1% x (2780 psi / 450000 psi) =	0.0062	%								
krm (decimal format) =	**0.000062**	P-Y Curve Type:								
		Weak Rock (Reese)								
Layer	**Soil Type**	**Top Depth (ft)**	**Bottom Depth (ft)**	**Top Elev. (ft)**	**Bottom Elev. (ft)**	**RQD (%)**	**Rec (%)**	**Qu (psi)**	**Total Unit Wt (pcf)**	
Layer 4	**Dolomite - Strong**	16.2	20.5	629.5	625.2	40	100	5560	154.1	at 16.2 ft
	Slightly Frac.									
		Depth below bottom of Pier Cap:		7.2	11.5					
Total Unit Wt (pcf):	165-175	GDM Table 400-5		**Use**	**155**	**pcf**				
	154.1	Tested Value								
Qu (psi)=	**5560**	Tested value								
From GDM Table 400-6, say E (psi) =	**450000**									
If Strain at 450000 psi is 1%, then strain at half max stress (krm) is calculated by:										
Half max stress = Qu/2 =	2780	psi								
krm = 1% x (2780 psi / 450000 psi) =	0.0062	%								
krm (decimal format) =	**0.000062**	P-Y Curve Type:								
		Weak Rock (Reese)								

OHIO DEPARTMENT OF TRANSPORTATION

OFFICE OF GEOTECHNICAL ENGINEERING

PLAN SUBGRADES
Geotechnical Bulletin GB1

Bridge Street

Between Water Street and Brierley Avenue
Village of Pemberville, Wood County, Ohio

CT Consultants, Inc.

Prepared By: Katherine C. Hennicken, P.E.
Date prepared: Friday, April 26, 2024

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NO. OF BORINGS: 2

#	Boring ID	Alignment	Station	Offset	Dir	Drill Rig	ER	Boring EL.	Proposed Subgrade EL	Cut Fill
1	B-001-0-23	Bridge Street	57+56	4	Lt	CME 550X	75	645.8	644.7	1.1 C
2	B-004-0-23	Bridge Street	59+66	4	Rt	CME 550X	75	645.7	644.6	1.1 C

OHIO DEPARTMENT OF TRANSPORTATION

Subgrade Analysis

V. 14.6 2/11/2022

#	Boring	Sample	Sample Depth		Subgrade Depth		Standard Penetration		HP (tsf)	Physical Characteristics						Moisture		Ohio DOT		Sulfate Content (ppm)	Problem		Excavate and Replace (Item 204)		Recommendation (Enter depth in inches)
			From	To	From	To	N_{60}	N_{60L}		LL	PL	PI	% Silt	% Clay	P200	M_C	M_{OPT}	Class	GI		Unsuitable	Unstable	Unsuitable	Unstable	
1	B 001-0 23	SS-1	1.0	3.0	-0.1	1.9	20	6		NP	NP	NP	20	2	22	8	6	A-1-b	0	100					**NO UNDERCUT**
		SS-2B	3.0	4.0	1.9	2.9	13		3.75	22	17	5	38	11	49	14	12	A-4a	3						
		SS-3	4.0	5.5	2.9	4.4	6		1.5	23	15	8	45	7	52	20	10	A-4a	3						
		SS-4	5.5	7.0	4.4	5.9	8		2							17	10	A-4a	8						
2	B 004-0 23	SS-1	1.0	2.5	-0.1	1.4	24	10		NP	NP	NP	11	1	12	4	6	A-1-a	0	100					**NO UNDERCUT**
		SS-2	2.5	4.0	1.4	2.9	15			NP	NP	NP	19	3	22	8	6	A-1-b	0						
		SS-3	4.0	7.0	2.9	5.9	10		2.75	22	15	7	43	18	61	18	10	A-4a	5						
		ST-4B	7.0	7.5	5.9	6.4	25									14	6	A-1-b							

OHIO DEPARTMENT OF TRANSPORTATION

Subgrade Analysis

V. 14.6 2/11/2022

PID:

County-Route-Section: Bridge Street
No. of Borings: 2

Geotechnical Consultant: CT Consultants, Inc.
Prepared By: Katherine C. Hennicken, P.E.
Date prepared: 4/26/2024

Chemical Stabilization Options		
320	Rubblize & Roll	No
206	Cement Stabilization	Option
	Lime Stabilization	No
206	Depth	14"

Excavate and Replace Stabilization Options	
Global Geotextile	
Average(N60L):	12"
Average(HP):	0"
Global Geogrid	
Average(N60L):	0"
Average(HP):	0"

Design CBR	9

% Samples within 6 feet of subgrade			
$N_{60} \leq 5$	0%	$HP \leq 0.5$	0%
$N_{60} < 12$	38%	$0.5 < HP \leq 1$	0%
$12 \leq N_{60} < 15$	13%	$1 < HP \leq 2$	25%
$N_{60} \geq 20$	38%	$HP > 2$	25%
M+	0%		
Rock	0%		
Unsuitable	0%		

Excavate and Replace at Surface	
Average	
Maximum	0"
Minimum	0"

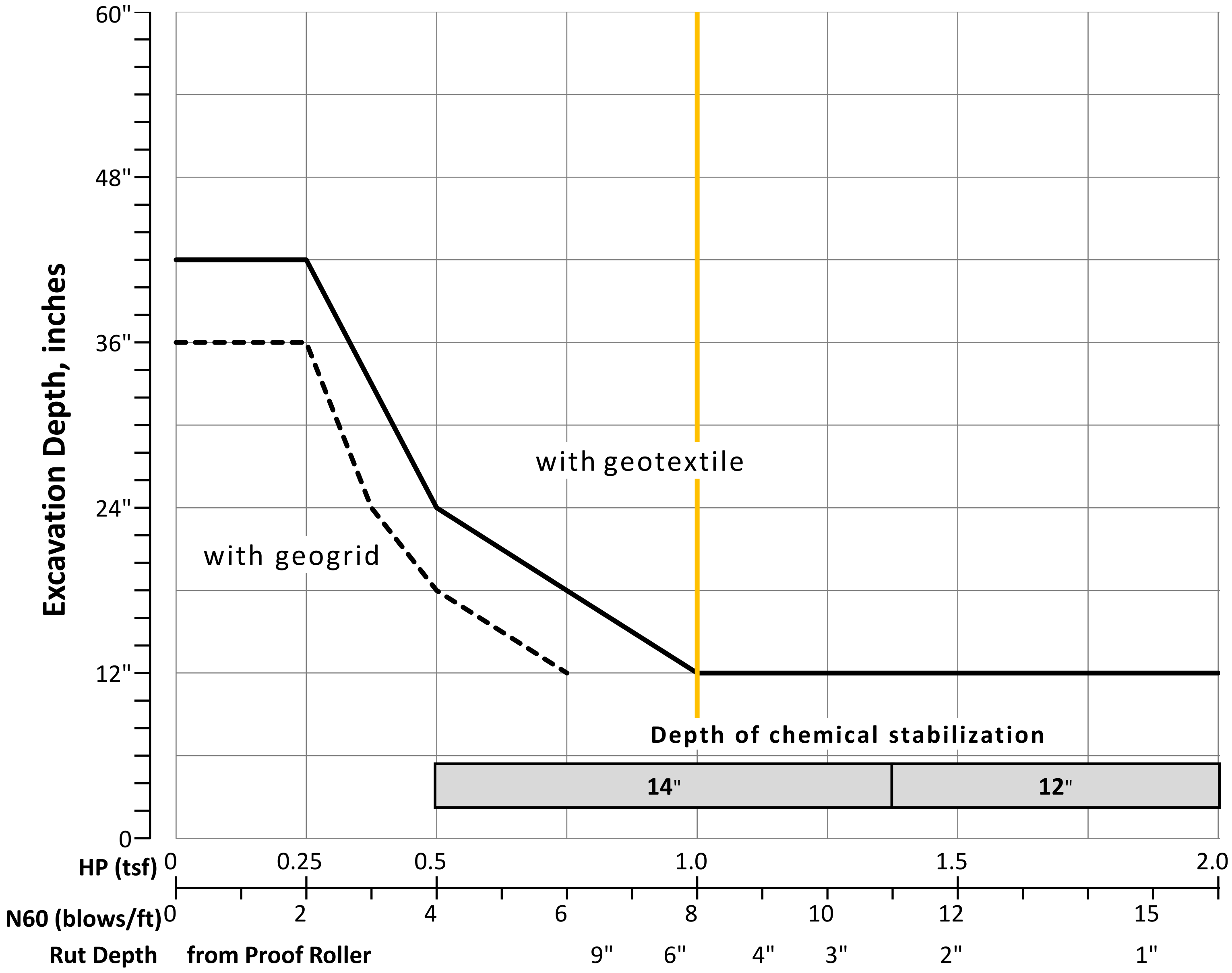
% Proposed Subgrade Surface	
Unstable & Unsuitable	0%
Unstable	0%
Unsuitable	0%

	N_{60}	N_{60L}	HP	LL	PL	PI	Silt	Clay	P 200	M_C	M_{OPT}	GI
Average	15	8	2.50	22	16	7	29	7	36	13	8	3
Maximum	25	10	3.75	23	17	8	45	18	61	20	12	8
Minimum	6	6	1.50	22	15	5	11	1	12	4	6	0

Classification Counts by Sample

ODOT Class	Rock	A-1-a	A-1-b	A-2-4	A-2-5	A-2-6	A-2-7	A-3	A-3a	A-4a	A-4b	A-5	A-6a	A-6b	A-7-5	A-7-6	A-8a	A-8b	Totals
Count	0	1	3	0	0	0	0	0	0	4	0	0	0	0	0	0	0	0	8
Percent	0%	13%	38%	0%	0%	0%	0%	0%	0%	50%	0%	0%	0%	0%	0%	0%	0%	0%	100%
% Rock|Granular|Cohesive	0%	100%									0%								100%
Surface Class Count	0	1	2	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	4
Surface Class Percent	0%	25%	50%	0%	0%	0%	0%	0%	0%	25%	0%	0%	0%	0%	0%	0%	0%	0%	100%

GB1 Figure B – Subgrade Stabilization



OVERRIDE TABLE

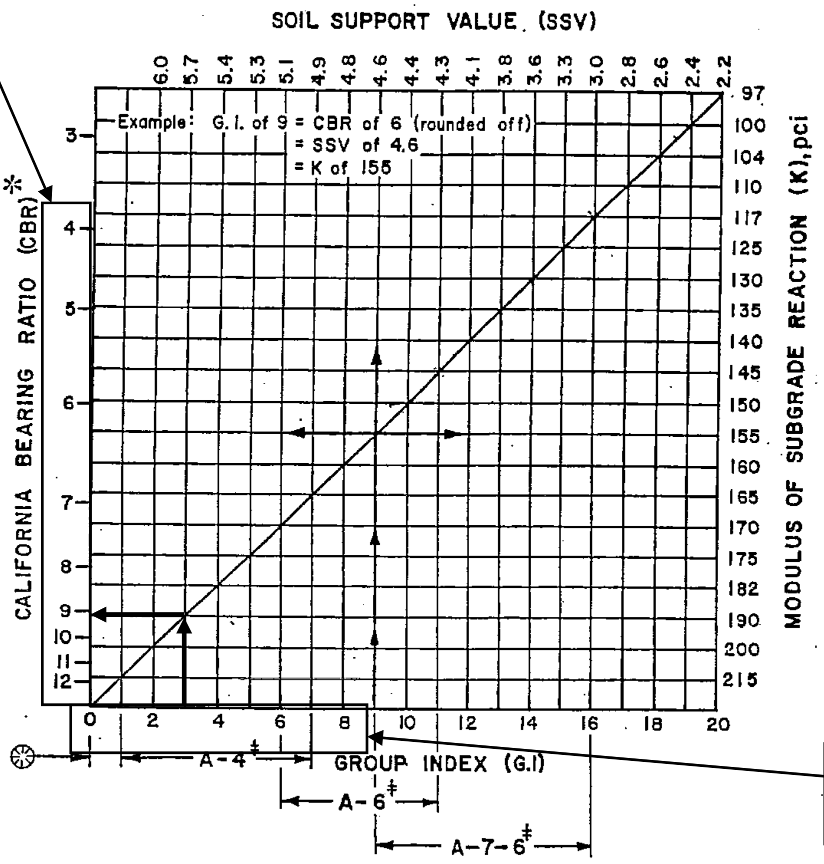
Calculated Average	New Values	Check to Override
2.50		☐ HP
8.00		☐ N60L

Average HP ▬

Average N_{60L} ▬

Range of GI from 0 to 8 for pavement subgrade samples corresponds to CBR values ranging from 6 to 12 percent. Average GI of 3 corresponds with CBR of 9 percent.

Fig.1301-3
Feb.1978



Range of GI for pavement subgrade samples: 0 to 8 for A-1-a, A-1-b, and A-4a, soils. Average GI was 3.

⊛ AASHTO Classes A-1, A-2 & A-3 lie below 0. SSV=6-10; K=200+.

‡ Usual range of AASHTO Classes.

* 5-1/2 Lb. hammer, 12" drop, 4 layers, 45 blows per layer, compacted at optimum moisture as determined by AASHTO T-99.

CORRELATION CHART FOR
SUBGRADE STRENGTHS

Based on the subgrade analysis, a design CBR value of 9 percent was determined for the project. It should be noted that the CBR determination by the subgrade analysis spreadsheet is based on the **average** Group Index of all the evaluated samples, which was 3. Group indices for the evaluated samples ranged from 0 to 8, which would correlate with a CBR value of 6 to 12 percent. Based on the average design value calculations from the subgrade analysis spreadsheet, it does not appear to be unconservative to use the spreadsheet design CBR value of 9 percent for new pavement sections throughout the project area.

APPENDIX B

Geotechnical Engineering Design Checklists

I. Geotechnical Design Checklists

Project: Bridge Street Bridge **PDP Path:**

PID: **Review Stage:** Final

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

II. Reconnaissance and Planning Checklist

C-R-S: Bridge Street Bridge	**PID:** 0	**Reviewer:** IJH	**Date:** 9/19/2024

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

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

II. Reconnaissance and Planning Checklist

Planning - General		(Y/N/X)	Notes:
10	Have the scaled boring plans, showing all project and historic borings, and a schedule of borings in tabular format, been submitted to the District Geotechnical Engineer?	Y	Included with proposal.
	The schedule of borings should present the following information for each boring:		
a.	exploration identification number	Y	
b.	location by station and offset	X	Station and offset were not available during planning.
c.	estimated amount of rock and soil, including the total for each for the entire program.	Y	

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

II. Reconnaissance and Planning Checklist

Planning – Boring Types		(Y/N/X)	Notes:
14	Based on Sections 303.3 to 303.7.6 of the SGE, have the location, depth, and sampling requirements for the following boring types been determined for the project?	Y	The borings were conducted following ODOT Type E1 standards for a bridge, with sampling performed in the upper 6 feet as an ODOT Type A Roadway Boring. To gather comprehensive subsurface data for the scour analysis, continuous sampling was extended to encounter bedrock.
	Check all boring types utilized for this project:		
	Existing Subgrades (Type A)	✓	
	Roadway Borings (Type B)		
	Embankment Foundations (Type B1)		
	Cut Sections (Type B2)		
	Sidehill Cut Sections (Type B3)		
	Sidehill Cut-Fill Sections (Type B4)		
	Sidehill Fill Sections on Unstable Slopes (Type B5)		
	Geohazard Borings (Type C)		
	Lakes, Ponds, and Low-Lying Areas (Type C1)		
	Peat Deposits, Compressible Soils, and Low Strength Soils (Type C2)		
	Uncontrolled Fills, Waste Pits, and Reclaimed Surface Mines (Type C3)		
	Underground Mines (C4)		
	Landslides (Type C5)		
	Rock Slope (Type C6)		
	Karst (Type C7)		
	Proposed Underground Utilities (Type D)		
	Structure Borings (Type E)		
	Bridges (Type E1)	✓	
	Culverts (Type E2 a,b,c)		
	Retaining Walls (Type E3 a and b)		
	Noise Barrier (Type E4)		
	CCTV & High Mast Lighting Towers (Type E5)		
	Buildings and Salt Domes (Type E6)		

III.C. Subgrade Checklist

C-R-S:	Bridge Street Bridge	**PID:**	0	**Reviewer:**	IJH	**Date:**	9/19/2024

Use this Checklist in conjunction with the Subgrade design guidance in GDM Section 600
If you do not have any subgrade work on the project, you do not have to fill out this checklist.

Subgrade		(Y/N/X)	Notes:
1	Has the subsurface exploration adequately characterized the soil or rock according to GDM Section 600?	Y	
a.	Has each sample been visually classified and inspected for the presence of gypsum? Has a moisture content been performed on each sample?	Y	
b.	Has mechanical classification (Plastic Limit (PL), Liquid Limit (LL), and gradation testing) been done on at least two samples from each boring within six feet of the proposed subgrade?	Y	
c.	Has the sulfate content of at least one sample from each boring within 3 feet of the proposed subgrade been determined, per Supplement 1122, Determining Sulfate Content in Soils?	Y	
d.	Has the sulfate content of all samples that exhibit gypsum crystals been determined?	X	No gypsum observed in samples.
e.	Have A-2-5, A-4b, A-5, A-7-5, A-8a, or A-8b soils within the top 3 feet of the proposed subgrade been mechanically classified?	X	None present.
2	If soils classified as A-2-5, A-4b, A-5, A-7-5, A-8a, or A-8b, or having a LL>65, are present at the proposed subgrade (geotechnical profile), do the plans specify that these materials need to be removed and replaced or chemically stabilized?	X	None present.
a.	If these materials are to be removed and replaced, have the station limits, depth, and lateral limits for the planned removal been provided?	X	
3	If there is any rock, shale, or coal present at the proposed subgrade (C&MS 204.05), do the plans specify the removal of the material?	X	Rock deeper than 24 inches below anticipated subgrade elevation so removal not required.
a.	If removal of any rock, shale, or coal is required, have the station limits, depth, and lateral limits for the planned removal of the material at proposed subgrade been provided?		

III.C. Subgrade Checklist

Subgrade	(Y/N/X)	Notes:
4 In accordance with GDM Section 600, do the SPT (N_{60})/HP values and existing moisture contents for the proposed subgrade soils indicate the need for subgrade stabilization?	N	
a. If removal and replacement is applicable, has the detail of subgrade removal been shown on the plans, including depth of removal, station limits, lateral extent, replacement material, and plan notes (Item 204 - Subgrade Compaction and Proof Rolling)?	Y	Removal and replacement is not anticipated. Plans to be prepared by others.
b. If chemical stabilization is applicable, has the detail of this treatment been shown on the plans, including depth, percentage of chemical, station limits, lateral extent, and plan notes?	N	Chemical stabilization not anticipated to be economical. Plans to be prepared by others.
Indicate type of chemcial stabilization specified:		
cement stabilization		
lime stabilization		
5 If removal and replacement has been specified, do the plans include Plan Note G121 from L&D3?	X	
6 If drainage or groundwater is an issue with the proposed subgrade, has an appropriate drainage system (e.g., pipe, underdrains) been provided?	X	Plans to be prepared by others.
7 Has an appropriate quantity of Proof Rolling (C&MS 204.06) and has Plan Note G111 from L&D3 been included in the plans?	X	Plans to be prepared by others.
8 Has a design CBR value been provided?	Y	

IV.A Foundations of Structures Checklist

C-R-S:	Bridge Street Bridge	**PID:**	0	**Reviewer:**		**Date:**	9/19/2024

Use this Checklist in conjunction with the bridge foundation design guidance in GDM Section 1300
If you do not have such a foundation or structure on the project, you do not have to fill out this checklist.

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

IV.A Foundations of Structures Checklist

Spread Footings		(Y/N/X)	Notes:
10	If weak soil is present at the proposed foundation level, has the removal / treatment of this soil been developed and included in the plans?		
a.	Have the procedure and quantities related to this removal / treatment been included in the plans?		
Pile Structures		(Y/N/X)	Notes:
11	Are there piles on the project? If no, go to Question 17	N	
12	Has an appropriate pile type been selected?		
	Check the type selected:		
	H-pile (driven)		
	H-pile (prebored)		
	Cast In-place Reinforced Concrete Pipe		
	Micropile		
	Continuous Flight Auger (CFA)		
	other (describe other types)		
13	Have the estimated pile length or tip elevation and section (diameter) based on either the Ultimate Bearing Value (UBV) or the depth to top of bedrock been specified? Indicate method used.		
14	If scour is predicted, has pile resistance in the scour zone been neglected?		
15	Has a wave equation drivability analysis been performed as per BDM 305.3.1.2 to determine whether the pile can be driven to either the UBV, the pile tip elevation, or refusal on bedrock without overstressing the pile?		
16	If required for design, have sufficient soil parameters been provided and calculations performed to evaluate the:		
a.	Nominal unit tip resistance and maximum settlement of the piles?		
b.	Nominal unit side resistance for each contributing soil layer and maximum deflection of the piles?		
c.	Downdrag load on piles driven through new embankment or compressible soil layers, as per BDM 305.3.2.2?		
d.	Potential for and impact of lateral squeeze from soft foundation soils?		

IV.A Foundations of Structures Checklist

Pile Structures		(Y/N/X)	Notes:
17	If piles are to be driven to strong bedrock (Q_u >7.5 ksi) or through very dense granular soils or overburden containing boulders, have “pile points” been recommended in order to protect the tips of the steel piling, as per BDM 305.3.5.6?	X	
18	If subsurface obstacles exist, has preboring been recommended to avoid these obstructions?	X	
19	If piles will be driven through 15 feet or more of new embankment, has preboring been specified as per BDM 305.3.5.7?	X	

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:	Y	Lateral load-deflection parameters provided to structural engineer.
a.	total factored lateral shear?		
b.	total factored bending moment?		
c.	maximum deflection?		
d.	reinforcement design?		
24	If a bedrock socket is required, has a minimum rock socket length equal to 1.5 times the rock socket diameter been used, as per BDM 305.4.2?	Y	Yes, then deeper embedment required for shallow bedrock considerations.
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?	✓	Scour has been neglected in determining shaft capacities.
27	Has the site been assessed for groundwater influence?	Y	
a.	If yes, and if artesian flow is a potential concern, does the design address control of groundwater flow during construction?	X	
28	Have all the proper items been included in the plans for integrity testing?	X	Plans to be prepared by others.
29	If special construction features (e.g., slurry, casing, load tests) are required, have all the proper items been included in the plans?	X	Plans to be prepared by others. Provided recommendations in geotechnical report.
30	If necessary, have wet construction methods been specified?	Y	
General		(Y/N/X)	Notes:
31	Has the need for load testing of the foundations been evaluated?	X	
a.	If needed, have details and plan notes for load testing been included in the plans?		

VI.B. Geotechnical Reports

C-R-S: Bridge Street Bridge	**PID:** 0	**Reviewer:**	**Date:** 5/30/2025

General		(Y/N/X)	Notes:
1	Has an electronic copy of all geotechnical submissions been provided to the District Geotechnical Engineer (DGE)?	Y	
2	Has the first complete version of a geotechnical report being submitted been labeled as 'Draft'?	Y	
3	Subsequent to ODOT's review and approval, has the complete version of the revised geotechnical report being submitted been labeled 'Final'?	y	
4	Has the boring data been submitted in a native format that is DIGGS (Data Interchange for Geotechnical and Geoenvironmental) compatable? gINT files meet this demand?	y	
5	Does the report cover format follow ODOT's Brand and Identity Guidelines Report Standards found at http://www.dot.state.oh.us/brand/Pages/default.aspx ?	Y	
6	Have all geotechnical reports being submitted been titled correctly as prescribed in Section 706.1 of the SGE?	Y	
Report Body		(Y/N/X)	Notes:
7	Do all geotechnical reports being submitted contain the following:		
a.	an Executive Summary as described in Section 706.2 of the SGE?	Y	
b.	an Introduction as described in Section 706.3 of the SGE?	Y	
c.	a section titled "Geology and Observations of the Project," as described in Section 706.4 of the SGE?	Y	
d.	a section titled "Exploration," as described in Section 706.5 of the SGE?	Y	
e.	a section titled "Findings," as described in Section 706.6 of the SGE?	Y	
f.	a section titled "Analyses and Recommendations," as described in Section 706.7 of the SGE?	Y	
Appendices		(Y/N/X)	Notes:
8	Do all geotechnical reports being submitted contain all applicable Appendices as described in Section 706.8 of the SGE?	Y	
9	Do the Appendices present a site Boring Plan showing all boring locations as described in Section 706.8.1 of the SGE?	Y	

VI.B. Geotechnical Reports

Appendices		(Y/N/X)	Notes:
10	Do the Appendices include boring logs and color pictures of rock, if applicable, as described in Section 706.8.2 of the SGE?	Y	
11	Do the Appendices include reports of undisturbed test data as described in Section 706.8.3 of the SGE?	Y	
12	Do the Appendices include calculations in a logical format to support recommendations as described in Section 706.8.4 of the SGE?	Y	