STRUCTURE FOUNDATION EXPLORATION REPORT

Proposed ATB Old Main Street Bridge Replacement

Submitted to City of Conneaut
REPORT Date *October 2025*

Prepared by

CT consultants
engineers • architects • planners
A Verdantas Company

Ohio Department of Transportation

Structure Foundation Exploration Report

PREPARED FOR

City of Conneaut

294 Main Street, Conneaut, OH 4403

ISSUED: October 2025

CT consultants
engineers • architects • planners

October 27, 2025 CT Project No. 232245

City of Conneaut
294 Main Street
Conneaut, Ohio 44030

Re: Structure Foundation Exploration Report
Proposed ATB Old Main Street Bridge Replacement
Conneaut, Ohio

Dear City of Conneaut Representative:

Following is the report of our Geotechnical Subsurface Exploration performed by CT Consultants, Inc. (CT) for the referenced project. This study was performed in accordance with Proposal No. P220609, dated June 9, 2023, and was authorized with a Subconsultant Services Agreement, dated February 16, 2024.

This report contains the results of our studies, our engineering interpretation of the results with respect to the project characteristics, and our recommendations for design and construction of pavements and bridge foundations.

A draft report was submitted to city of Conneaut and ODOT in June 2024, for review and comment. Comments were received and are incorporated herein. As such, we are now submitting the previously provided report as "FINAL" in accordance with ODOT protocol. Should you have any questions regarding this report or require additional information, please contact our office.

Respectfully,

CT Consultants, Inc.

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STRUCTURE FOUNDATION EXPLORATION REPORT

PROPOSED ATB OLD MAIN STREET BRIDGE REPLACEMENT
CONNEAUT, OHIO

FOR

CITY OF CONNEAUT
294 MAIN STREET
CONNEAUT, OHIO 44030

SUBMITTED

OCTOBER 27, 2025
CT PROJECT NO. 232245

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1.0 INTRODUCTION

This report has been prepared for the proposed bridge replacement of the existing Old Main Street Bridge over the West Branch Conneaut Creek in Conneaut, Ohio. The project site is shown on the Site Location Map (Plate 1.0).

This study was performed in accordance with Proposal No. P220609, dated June 9, 2023, and was authorized with a Subconsultant Services Agreement, dated February 16, 2024.

1.1 Purpose and Scope of Exploration

The purpose of this exploration was to evaluate the subsurface conditions at the site. To accomplish this, CT performed four (4) test borings, field and laboratory soil testing and review of available geologic and soils data for the project area. The information provided in this data report will be incorporated into the final geotechnical exploration report which would be prepared in subsequent stages on the project.

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 GDM Section 600 "Subgrade" (January 2024)and provides our design and construction recommendations for pavements.

This report includes:

- A description of the existing surface materials, subsurface soils, and groundwater conditions encountered in the borings.
- Design recommendations for bridge foundations, associated shaft, walls, and pavements.
- Recommendations concerning soil and groundwater-related construction procedures such as subgrade preparation, earthwork, pavement and foundation construction, and related field testing.

1.2 Proposed Construction

The project includes the proposed bridge replacement of the existing Old Main Street Bridge over the West Branch Conneaut Creek in Conneaut, Ohio.

The proposed bridge will be a two-span, 160-foot-long composite prestressed box beam bridge supported on new abutments and 1 pier. The proposed abutments will be located out of the stream flow of Conneaut Creek and portions of the existing abutment will remain in front of the new abutments as scour countermeasures. It will also consist of a 10-foot-wide shared use path for pedestrian traffic across the bridge.

It is out understanding that the west abutment will be supported on drilled shafts whereas the new pier and east abutment will be supported on shallow foundations.

Information regarding traffic loads was not provided at the time of this report. New pavements are anticipated to consist of flexible (asphalt) and/or rigid (concrete) sections for roadways.

2.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1 General Geology and Hydrogeology

Physiographic Region

The project site at 132 Old Main St, Conneaut, OH, is situated within the Glaciated Allegheny Plateau, a sub-region of the larger Appalachian Plateau. This physiographic province is characterized by a landscape of rolling hills, dissected plateaus, and broad river valleys, which were heavily influenced by Pleistocene glaciation. The glaciation left behind a varied topography with significant deposits of glacial till and outwash. The area's topography and geomorphology are critical factors in bridge construction, particularly for structures with in-water piers, where understanding soil and bedrock conditions is essential for foundation design and stability.

Quaternary Deposits

The Quaternary deposits at the site primarily consist of glacial till, outwash, and lacustrine sediments. Glacial till, an unsorted mixture of clay, silt, sand, gravel, and boulders, is prevalent and typically exhibits low permeability and varying degrees of consolidation. Outwash deposits, composed of stratified sands and gravels, are found in areas influenced by glacial meltwater and are more permeable, often forming the primary aquifers. Lacustrine sediments, including fine-grained silts and clays, were deposited in glacial lakes and are typically found in low-lying areas.

NRCS Soil Survey

The USDA Natural Resource Conservation Service (NRCS) Web Soil Survey for the project area identifies the predominant upper-profile soil as Otego silt loam. These soils are derived from alluvium formed on floodplains and are considered moderately well-drained. The Otego silt loam has good drainage characteristics, making it suitable for various types of construction. However, its alluvial nature means it can be susceptible to changes in moisture content and may exhibit variable bearing capacity.

Aquifers

Aquifers in the Conneaut area are found within both the unconsolidated glacial deposits and the underlying bedrock formations. Unconfined aquifers in the outwash sands and gravels provide significant groundwater storage and transmission capacity, recharged by precipitation and surface water infiltration. Confined aquifers within the glacial till and lacustrine sediments may also be present, with groundwater flow controlled by the permeability and continuity of these deposits.

Bedrock

The bedrock underlying the site is primarily composed of Devonian and Mississippian-age sedimentary formations, including sandstones, shales, and siltstones. These formations were deposited in ancient marine and fluvial environments, resulting in varied lithologies with different degrees of consolidation and fracturing. The depth to bedrock in this area can vary significantly due to the glacial and post-glacial topography, but it is typically encountered at relatively shallow depths beneath the Quaternary deposits.

Based on ODNR mapping, no mining or probable karst is indicated for the project site.

2.2 Site Reconnaissance

CT performed site reconnaissance on October 11, 2023. The eastern part of the site consist of predominantly commercial and residential properties while the western side is undeveloped and consist of mature woods. predominantly of heavy wooded surrounding encapsulating the project site. There is a creek below the bridge.

In the immediate area of the bridge, the pavement along Old Main Street was observed to generally be in poor condition and heavily distressed. The pavements were observed to have transverse, longitudinal, and fatigue cracks that were generally not sealed. Alligator cracks were observed in multiple areas along the roadway area. Several Potholes were also observed along the edge of the bridge with smaller potholes along the entirety of the bridge. At the ends of the bridge there appears to be longitudinal cracks as well as depressions in the concrete The foundation structures of the bridge appear to be slightly weathered, but overall in good condition.

The existing bridge deck appeared to be surfaced with asphalt with the ends being concrete. The bridge is approximately 180 feet long and 20 feet wide. There is also a concrete sidewalk on the north side of the bridge along the entirety of the bridge. A metal medium between the concrete ends and the asphalt of the bridge appears to be slightly rusted as well.

Within the site area, the agricultural area surrounding the bridge slopes down to the creek on both sides of the bridge. The creek below the bridge was approximately 15 to 20 feet below the road surface. A railroad track also runs north-south just west of the bridge.

Overhead utility lines were observed but were just north of the bridge.

3.0 EXPLORATION

3.1 Historic Borings

Based on our research, historic boring information was not available for the alignment of Old Main Street within the vicinity of either project location.

3.2 Project Exploration Program

This exploration included four test borings, designated as Borings B-001-0-23 through B-004-0-23, performed from the period of October 17 through 19, 2023, by CT Consultants. The borings have been identified in accordance with ODOT protocol, but the "-0-23" portion of the nomenclature is generally omitted for discussion in this report. The borings were located in the field by CT in accordance with a proposed boring location plan. The approximate locations of the borings are shown on the Test Boring Location Plans (Plates 2.0).

Based on boring location dimensions from existing site features obtained by CT, Latitude, Longitude, and ground surface elevation were estimated using Google Earth. This data is shown on the Logs of Test Borings.

Borings B-001 and B-003 were performed as ODOT Type E1 borings and were extended to a depth of 6 feet below existing pavements as an ODOT type A Boring. Boring B-002 was performed as an ODOT Type E1 boring to a depth of 6 feet below the creek bottom for scour analysis. Boring B-004 was performed as an ODOT Type A roadway boring for subgrade evaluation.

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 services during the site preparation and pavement construction 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 using a truck-mounted rig with hollow-stem augers. All borings were continuously sampled for 6 feet using 18-inch split-spoon sample drives. Borings B-001 and B-003 were then sampled every 2½ feet until auger refusal. Boring B-2 was conducted from the bridge through a corehole in the bridge deck, with only one sample recovered from the creek bed before auger refusal. The samples were sealed in jars and transported to our laboratory for classification and testing.

Split-spoon 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. The number of blows per increment was recorded at each depth interval, 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 Nm-value, and is typically reported in blows per foot (bpf). The Nm-values were corrected to an equivalent rod energy ratio of 60 percent, N60. The hammer/rod energy ratio for the CME 75 Truck 844 mounted drilling rig was 72.9 percent and was last calibrated on February 20, 2023. The N60-values are presented on the attached Logs of Test Borings.

One (1) Shelby tube sample, designated ST on the Log of Test Boring, was obtained in Boring B-003 from 13 to 15 feet below existing grades. The Shelby tube sample was obtained 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, and the ends were capped and sealed. The sample was transported to our laboratory where it was extruded, classified, and tested.

Upon encountering auger refusal in Borings B-001, B-2 and B-003, two 5-foot rock core runs were completed using an NQ2 diamond-bit core barrel and coring techniques in general accordance with ASTM D 2113. 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 in length and greater to the total length of the run or rock unit thickness. The RQD is expressed for each bedrock unit to provide clarity of the overall quality of rock within the unit descriptions. The rock core samples are designated as “NQ2-1” and “NQ2-2” on the Log of Test Boring attached to this report.

Soil conditions encountered in the test borings are presented in the Logs of Test Borings, along with information related to sample data, SPT results, water conditions observed in the borings, and laboratory test data. 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.

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.

3.4 Laboratory Testing Program

All 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). Dry density determinations and unconfined compressive strength tests by the constant rate of strain method (ASTM D 2166) were performed on selected samples, including the Shelby tube sample. Unconfined compressive strength estimates were obtained for the remaining intact cohesive samples using a calibrated hand penetrometer. `

Laboratory testing was performed in accordance with ODOT GDM SECTION 600 “Subgrade” criteria, including mechanical soil classification consisting of an Atterberg limits test (ASTM D 4318) and a particle size analysis (ASTM D 6913 and D 7928) for two samples from each boring within 6 feet of the proposed subgrade. These test results are presented on the Logs of Test Borings and Grain Size Distribution sheets.

These test results are presented on the Logs of Test Borings.

4.0 FINDINGS

4.1 General Site Conditions

The project site is predominantly located along the Old Main Street bridge in Conneaut, Ohio. Roadway Grades in the project area ranged from Elevs. 591± to 595± in Borings B-001, B-003, and B-004 and 573± in Boring B-002.

All the borings were performed in existing pavements. The encountered surface materials consisted of asphalt ranging in thickness from approximately 1 to 8½ inches in Borings B-002, B-003, and B-004. Aggregate base material was present below the asphalt in Borings B-003 and B-004 and at the ground surface in Boring B-001 ranging in thickness from 4 to 15 ½ inches. The asphalt was underlain by concrete (bridge deck) in Boring B-002 that was approximately 31 inches thick.

The encountered pavement materials are summarized in the following table.

Table 4.1. Encountered Pavement and Subgrade Materials

Boring Number	Thickness (inches)		
	Asphalt	Aggregate Base	Concrete
B-001	-	4	-
B-002	1	-	31
B-003	8½	15½	-
B-004	4	8	-

In Borings B-001, B-003, and B-004 underlying the surface material, granular and cohesive fill material were encountered to depths of 5½ , 8½, and 4½ feet, respectively.

The granular fill material consisted of gravel and stone fragments with silt and sand (A-2-4) with varying amounts of clay. It should be noted that coal fragments were encountered in Boring B-4 within the granular fill layer. SPT N_{60}-values of 1 to 36 blows per foot (bpf), indicating very loose to dense compactness. Moisture contents ranged from 7 to 26 percent.

Isolated layers of Cohesive fill material were only encountered in Borings B-001 and B-004 with thickness on the order of 3 feet. The cohesive fill material consisted of sandy silt (A-4a) with varying amounts of coal fragments, gravel and clay. SPT N_{60}-values of 7 and 10 bpf were reported for the two samples recovered from the cohesive fill material.

4.2 General Soil and Rock Conditions

Based on the results of our field and laboratory tests, the subsoils encountered underlying the surface materials and existing fill materials can generally be characterized predominantly stiff to hard native cohesive soils underlain by native granular soil.

Stratum I consisted of predominantly stiff to hard cohesive soils encountered along the surface material in Borings B-001, B-003, and B-004 to depths of 8 feet, 16 feet and 7½ feet below existing grades, respectively. The Stratum I native cohesive soils consisted of silt and clay (A-6a) and sandy silt (A-4a) mixed with varying amounts of clay, gravel, and sand. SPT N_{60}-values ranged from 7 to 11 blows per foot (bpf). Unconfined compressive strength of 2,500 to 9,000 pounds per square foot (psf) (maximum reading obtainable using a hand penetrometer). Moisture contents ranged from 16 to 26 percent.

Stratum II consisted of predominantly medium dense native granular soils encountered underlying stratum I in Boring B-001 and B-003 to depths of 15½ feet and 18½ feet, respectively. The Stratum II granular soils consisted of fine sand (A-3), as well as coarse and fine sand (A-3a) mixed with varying amounts of sand, gravel, silt, and clay. A layer of decomposed bedrock sampled and classified as gravel and stone fragments (A-1-a) was encountered in Boring B-001. SPT N_{60}-values generally ranged from 16 to 17 blows per foot (bpf). Moisture contents ranged from 12 to 13 percent.

Shale bedrock was encountered underlying the native cohesive soils starting at approximately 15½ feet (Elev. 1038±), 23 feet (Elev. 570±) and 18½ feet (Elev. 573±) in Borings B-001, B-002 and B-003, respectively. Weathered rock that was able to be penetrated with the augers was encountered to depths of 24 (Elev. 569±) and 24½ feet (Elev. 567±) in Borings B-002 and B-003, respectively. This represents approximately 1 and 6 feet in Borings B-002 and B-003, respectively, of bedrock that was weathered and decomposed such that it was augerable. Within the weathered rock, the SPT generally resulted in split-spoon refusal (SSR, 50 or more blows for 6 inches or less penetration).

The depths of encountered weathered rock and auger refusal on more intact rock are summarized in the following table.

Table 4.2.A Summary of Encountered Rock Depths

Boring Number	Ground Surface Elevation (feet)	Top of Weathered Rock Depth (feet)	Top of Weathered Rock Elevation (feet)	Top of Corable Rock Depth (feet)	Top of Corable Rock Elevation (feet)
B-001	592.7	15.7	577	15.7	577
B-002	570.1	0	570	24	569.2
B-003	591.7	18.7	573	24.5	567.2

Upon encountering auger refusal, the bedrock was cored for 10 feet in each boring, using 5-foot intervals. The recovered rock consisted of slightly to moderately weathered shale. Data for the cored bedrock is summarized in the following table.

Table 4.2.B Summary of Cored Rock

Boring No.	Rock Core Run No.	Depth (feet)	Recovery (%)	RQD (%)	Slake Durability Index, S_{DI} (percent)	Comp. Strength (psi)
B-001	NQ2-8	15.7-20.7	80	0	80.6	2730, 2860
	NQ2-9	20.7-25.7	88	0		3350
B-002	NQ-2	24-29	100	0	83.5	2280, 5630
	NQ-3	29-34	90	0		5170
B-003	NQ2-12	24.5-29.5	95	0	80.9	7040-2581
	NQ2-23	29.5-34.5	100	0		4994-7473

Based on RQD values that were generally 0 percent, the rock mass quality in the cored bedrock profile can be generally characterized as very poor to poor. Based on compressive strength test results, the cored bedrock can be described as moderately strong to strong.

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

4.3 Groundwater Conditions

Groundwater was encountered during drilling in Borings B-001 and B-003 at depths of 8 feet and 20.3 feet, but not observed upon completion of drilling operations in any of the land borings. For Boring B-002, the top of water in the creek elevations was approximately 572 at the time of boring. It should be noted that the boreholes were drilled and backfilled within the same day, and stabilized water levels may not have occurred over this limited time period.

Apart from streamflow influences in the creek, it is our opinion that the "normal" groundwater level can generally be expected at depths corresponding to the bottom of the creek, on the order of 18 to 20 feet below roadway grades (Elev. 572±), or deeper (within bedrock). It should be noted that groundwater elevations can also fluctuate with seasonal and climatic influences, as well as streamflow conditions in the creek. Perched groundwater may be encountered within the pavement subbase, existing fill materials, or existing granular embankment materials that are underlain by relatively impermeable cohesive soils. Perched groundwater may also be encountered at the soil/bedrock interface. Therefore, the groundwater conditions may vary at different times of the year from those encountered during this exploration.

4.4 Scour Considerations

Scour considerations for the encountered subsoils should be made as part of the vertical and lateral load evaluations for the drilled shafts and rock sockets. There is no evidence of bed movement of the rock on which the piers and abutments are founded. Utilizing the clear-water methodology - HEC-18, Section 3.4 - for abutment scour, and as shown on Report ATB-MR-365-0.02, Dated October 2025, negative scour is predicted, indicating that the abutments will not be subjected to scour. Nevertheless, we have attached in Appendix B scour calculations sheet.

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 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 West Abutment Foundation – Drilled Shafts

The Old Main Street Bridge replacement is designed to feature a two-span 160-foot composite prestressed box beam bridge supported on new abutments and one (1) pier. Due to the proximity of bedrock to the proposed foundation pier caps and the nearby train tracks, the west abutment of the new bridge is planned to be supported by a deep foundation system, specifically drilled shafts socketed into bedrock. The design of the bridge foundations will adhere to LRFD methods. The maximum total factored vertical loads for abutments are specified to be 307.8 kips.

The anticipated top of shaft elevations (i.e., bottom of abutment) are projected to be approximately 583.8 for the western abutment. For the abutments, it is planned to utilize 42-inches diameter shaft above bedrock and 36-inches diameter shaft in the socket. 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.

5.1.1 Vertical Load Evaluations

The minimum prescribed rock socket length is 1.5 times the socket diameter. However, rock sockets should be increased to 5 feet in accordance with BDM 305.4.4.4, as the top of the rock was encountered within 10 feet of the bottom of the shaft cap. The proposed abutments are located out of the stream flow of the Conneaut Creek and the existing abutment stones will be removed and placed in front of the new abutment as scour countermeasures. Therefore, controlling scour elevation is not required for this site.

For detailed recommendations regarding rock socket lengths based on vertical resistance evaluations, please refer to the accompanying table.

Item	Boring B-001
	West Abutment
Recommended Minimum Rock Socket Length[1]	5 feet
Top of Rock Elevation (feet)	577
Bottom of Rock Socket Minimum Elevation (feet)	572

[1] Based on rock socket diameter of 36 inches as well as rock considerations discussed above.

Based on the rock conditions encountered in Boring B-001 for the Western Abutment, an unfactored unit tip resistance (q_p) of 1,073 kips per square foot (ksf) was calculated. Per LRFD guidance, this value was determined using an average of the compressive strength results at and within approximately 2 times the socket diameter below the end-bearing elevation. 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 535 ksf.** Using the planned 36-inch diameter socket, this design value would provide sufficient resistance for the indicated factored vertical load.

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. If such conditions are observed during socket installation at or in close proximity above the end-bearing elevation, it may be prudent to extend the sockets deeper.

The factored unit tip resistance evaluations presented above were 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 (for shafts installed beyond the less competent upper bedrock profile, into rock resulting at least in SSR), 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 expected presence of groundwater at the soil/rock interface, as well as the encountered fill materials, it is likely that temporary steel casing will be required to support the walls of the shaft and to control groundwater 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. 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 308 kips at the abutments. This load is resisted entirely by tip resistance. At the West Abutment the factored tip resistance is 3,782 kips.

5.1.2 Lateral Load Evaluations

For lateral load-deflection evaluations using software, such as LPILE, recommended design parameters are summarized in the following tables based on the conditions encountered in the borings. Design values are provided based on Borings B-001 so evaluations can be made for the West Abutment.

Table 5.1.2.A. Subsurface Conditions and Recommended Lateral Load-Deflection Parameters – Boring B-001 [West Abutment]

Approx. Depth (feet)	Approx. Elevation (feet)	Generalized Layer Description	Approx. Total Unit Weight[1] (pcf)	Approx. Internal Angle of Friction (deg)	Average Undrained Shear Strength, Su (psf)	Strain at 50% Maximum Stress, ε_{50}	Young's Modulus, Er (psi)	Rock Uniaxial Compressive Strength (psi)	k_{rm}
0 to 3	592.4 to 589.7	Medium stiff cohesive soils – Fill	118	-	875	0.007	–	–	–
3 to 5.5	589.7 to 587.4	Very loose to loose granular soil – Potential Fill	110	29	-	-	–	–	–
5.5 to 8	587.4 to 584.7	Stiff to very stiff cohesive soil – Native Soils	120	-	2,630	0.005	-	-	-
8 to 11.3	584.7 to 581.4	Medium dense granular soil – Native Soils	122	34	-	-	-	-	-
11.3 to 15.6	581.4 to 577.0	Augerable Shale as Bedrock	150	-	–	–	18,000	102.9	0.000029
15.6 to 25.7	577.0 to 567	Weak to slightly strong, weathered Shale as Bedrock RQD = 0%	155	-	-	-	265,067	2730 - 3350	0.000050

[1]Effective unit weight should be used below a depth of 20 feet (reduce by unit weight of water – 62.4 pcf).

A p–y analysis was performed in accordance with GDM Section 1501.7 using the parameters shown above. The vertical wall element was modeled from the proposed top of wall elevation to the estimated tip elevation, and fixity was achieved within the anticipated rock socket depth. The resulting head deflection of the vertical wall element was within the serviceability limit of 2 inches, satisfying the requirements of GDM Section 1501.6.

5.1.3 Scour Considerations

Scour considerations for the encountered subsoils should be made as part of the vertical and lateral load evaluations for the drilled shafts and rock sockets. The proposed abutments are located out of the stream flow of the Conneaut Creek and the existing abutment stones will be removed and placed in front of the new abutment as scour countermeasures. For scour depth, please refer to Section 4.4.

5.2 <u>Pier Foundation - Shallow Foundations on Weathered Bedrock</u>

For the pier support, it is planned that the bridge spread foundations be extended to bear on moderately weathered shale bedrock. Footing excavation should extend through highly weathered/fractured rock (particularly that which was augerable in the borings).

Based on the conditions encountered in Boring B-002, zero percent RQD and high recovery of 90 to 100 percent was determined for the rock extending to approximately Elev. 559.2. Furthermore, uniaxial compressive (UCS) strength of in the order of 2280 to 5630 psi was determined on the rock core samples collected at Elev. 569 to 565.

We understand that the bridge foundations will be designed using LRFD specifications. The following loads were provided. It was indicated that a foundation width of 9 feet was planned.

- Strength Limit State Maximum load: 10.5 ksf;
- Service Limit State Maximum load: 8.1 ksf

At the strength limit state, we recommend a nominal bearing resistance (q_n) of 99 ksf for foundations bearing on intact shale bedrock. At the strength limit state, the resistance factor (ϕ_b) is 0.45. Therefore, the factored bearing resistance (q_r) is 45 ksf. From a conventional allowable stress design comparison, this is roughly akin to calculating an ultimate bearing capacity and applying a factor of safety. This strength limit state bearing resistance is adequate based on the provided maximum strength limit state bearing pressure of 10.5 ksf.

Since the structure will be bearing on weathered bedrock, no adjustments to the bearing pressure are required at this structure location. The calculated unfactored bearing pressure of 99 ksf (0.69 ksi) is significantly lower than the estimated rock mass modulus of 57,120 ksf (605 ksi) , which satisfies the criterion outlined in GDM Section 1303.2.1— that bearing stress should be less than 50 times the rock mass modulus to assume negligible settlement. Supporting calculations for the rock mass modulus are provided in Appendix A.

Given that the foundation is resting on competent bedrock and the settlement is considered negligible, the service limit state bearing resistance is deemed adequate. We, therefore, anticipate that the **service limit state bearing resistance is adequate based on the provided maximum service limit state bearing pressure of 8.1 ksf.**

5.3 East Abutment Foundation - Shallow Foundations on Rock

For east abutment support, it is planned that the bridge spread foundations be extended to bear on augerable severely weathered shale bedrock. The augerable severely weathered bedrock is assumed to behave like cohesionless granular material. The bearing capacity, settlement, and overall stability of east abutment is computed based on this assumption.

We understand that the bridge foundations will be designed using LRFD specifications. The following loads were provided. It was indicated that a foundation width of 9 feet was planned.

- Strength Limit State Maximum load: 12.2 ksf;
- Service Limit State Maximum load: 8.6 ksf

At the strength limit state, we recommend a nominal bearing resistance (q_n) of 31.4 ksf for foundations bearing on augerable severely weathered bedrock. At the strength limit state, the resistance factor (ϕ_b) is 0.55. Therefore, the factored bearing resistance (q_r) is 17.3 ksf. From a conventional allowable stress design comparison, this is roughly akin to calculating an ultimate bearing capacity and applying a factor of safety. **This strength**

limit state bearing resistance is adequate based on the provided maximum strength limit state bearing pressure of 12.2 ksf.

Since the structure will be bearing on weathered bedrock, no adjustments to the bearing pressure are required at this structure location. The calculated unfactored bearing pressure of 17.3 ksf is significantly lower than the estimated rock mass modulus of 7,920 ksf, which satisfies the criterion outlined in GDM Section 1303.2.1—that bearing stress should be less than 50 times the rock mass modulus to assume negligible settlement. Supporting calculations for the rock mass modulus are provided in Appendix A.

Given that the foundation is resting on competent bedrock and the settlement is considered negligible, the service limit state bearing resistance is deemed adequate. We, therefore, **anticipate that the service limit state bearing resistance is adequate based on the provided maximum service limit state bearing pressure of 8.6 ksf.**

5.3.1 East Abutment – Overall Stability

East abutment is checked against for potential overturning and sliding as per LRFD Section 10.6.3.5 and 10.6.3.4. In order to perform these, we assumed the east abutment as a semi-gravity cantilever wall.

Overturning stability was evaluated by comparing the calculated eccentricity of the wall geometry to the maximum eccentricity with the resultant force. It was assumed that the backfill will consist of cohesive soils with a minimum effective internal angle of friction (ϕ') of 30 degrees behind the stabilized earth section of fill. As such, a coefficient of active earth pressure, K_a, of 0.33 was used for the overturning analysis at the abutment sections. Based on the analysis the abutment were determined to be adequate with regard to eccentricity, as presented in attached calculations in Appendix A.

The LRFD factored sliding resistance (R_R) is determined by ϕR_n, where R_n is the nominal sliding resistance on the base of the wall, and ϕ is the resistance factor. For semi gravity cantilever walls, ϕ = 1.0.

The abutment is anticipated to bear augerable weathered rock. We assumed the augerable weathered rock as cohesionless granular soils having an internal angel of friction of 30 degrees with no cohesion. The factored sliding resistance provided by the foundation base is 2,454 kips. Calculations are attached in Appendix A.

5.3.2 East Abutment Wingwall Foundation - Shallow Foundations on Soils

For the east abutment support, it is planned that the bridge spread foundations be extended to bear on the existing native soils or the underlying weathered bedrock.

Based on the conditions encountered in the borings, the soils at the anticipated foundation bearing elevation are expected to consist of:

- Stratum I – very stiff to hard native cohesive soils, or
- Stratum II - medium dense native granular soils, or
- Bedrock - Highly Weathered / Decomposed Shale

The native cohesive soils are considered generally suitable for support of the proposed abutment and will govern the bearing capacity design. However, with any installation within a creek area, there may be areas of encountered sediment at bearing elevations, which would require over-excavation. The bearing soils should be confirmed as being native cohesive soils with an unconfined compressive strength of at least 3,000 pounds per square foot (hand penetrometer reading of 1.5 or greater).

We understand that the abutment foundations will be designed using LRFD specifications. At the strength limit state, we recommend a nominal bearing resistance (q_n) of 8.98 ksf (undrained) and 54.9 ksf (drained) for the abutment base bearing on the native cohesive soils. As such, undrained conditions govern the design. At the strength limit state, the resistance factor (ϕ_b) is 0.55. Therefore, the factored bearing resistance (q_r) is 4.9 ksf (undrained) and 30.2 ksf (drained). From a conventional allowable stress design comparison, this is roughly akin to calculating an ultimate bearing capacity and applying a factor of safety.

Settlement of the abutment was calculated by conventional consolidation theory utilizing recompression indices for the over-consolidated soils, based on empirical relations using moisture content. Based on a bearing pressure of 4.9 ksf (undrained) and 30.2 ksf (drained), using the service limit state bearing resistance indicated above, total settlement was calculated to be on the order of ½ to ¾ inches.

Although not anticipated to be prevalent, if unsuitable bearing soils are encountered during culvert installation, over-excavation should extend through these materials to suitable bearing soils. The base of the over-excavation should be widened 6 inches for every foot of depth extending beyond the edge of the culvert. The over-excavated areas should be backfilled with lean concrete having a minimum compressive strength of 1,500 pounds per square inch (psi) or other flowable controlled-density fill having a minimum compressive strength of 300 psi. If foundations will be placed at the base of the over-excavation or the lean concrete fill option will be utilized, widening the footing over-excavation will not be required. If the controlled-density fill or aggregate fill option is utilized, the footing over-excavation shall be widened as discussed above.

5.3.3 Lateral Earth Pressures

Based on the conditions encountered in the borings performed for this investigation, the soils at the east (forward) abutment that will support the wingwall above the rock are predominately consist of either native cohesive or granular soils.

For wingwalls that are restrained at the top of the wall, lateral earth pressures should be assumed for "at-rest" conditions. It is anticipated that excavated on-site granular soils will comprise the majority of the backfill behind the existing abutment walls. For the encountered subsurface soils, an at-rest lateral earth pressure coefficient (K_o) of 0.48 should be used along with a total soil unit weight of 122 pounds per cubic foot (pcf) in determining the lateral pressure acting on the walls.

For the encountered subsurface soils, an active lateral earth pressure coefficient (K_a) of 0.32, and passive lateral earth pressure coefficient (K_p) of 3.12 should be used along with a total soil unit weight of 122 pcf in determining the lateral pressures acting on the walls.

Although unlikely, lateral loading due to hydrostatic pressures below the design groundwater depth should be included in design of below-grade walls. Depending on the design methodology, total lateral pressures would be the resultant of the hydrostatic pressures in combination with submerged (or "effective") unit weights of the soil. An effective unit weight of 57 pcf should be used for lateral earth pressure design below the design groundwater depth.

It should be noted that the above K-parameters may be used for general design of subsurface structures, retaining walls, and possible excavation support systems associated with the project. However, certain types of braced excavations may account for method-specific earth pressure distributions, for which the above parameters should be reviewed and utilized in the proper context of the design method/system.

Lateral load due to hydrostatic pressures below the design groundwater depth should be included in design of below-grade walls. Additionally, the earth pressures indicated above are based on a level backfill condition behind the culvert wall. If there are areas beyond the horizontal roadway portion of the backfill area that include sloping backfill behind the top of the wall, surcharge loading or equivalent higher earth pressure coefficients should be evaluated, based on backfill material, backfill slope, and proximity to the wall. In general, 50 percent of the vertical surcharge load may be assumed for lateral loading in the design of the wall.

Backfill for the abutment should be placed concurrently on both sides to avoid unbalanced forces that could cause sliding. If this method of backfilling is not possible and one side will be backfilled prior than the other, sliding can be evaluated as presented below.

We recommend that passive pressure be considered negligible at the toe of the wall due to the potential for erosion and/or freeze-thaw behavior that would significantly reduce reliance on passive earth pressure. As such, the LRFD nominal sliding resistance (R_R) is determined by $\phi_T R_T$, where RT is the nominal sliding resistance on the base of the footing.

For cohesive soils, nominal sliding resistance R_T is the lesser of the following:

- The cohesion (c) of the clay, for which we recommend c be taken as 3,000 psf, or
- Although not anticipated to be the case, where footings are supported on at least 6 inches of compacted granular material, one-half the normal stress on the interface between the footing and soil.

For sliding resistance on clays, the resistance factor ϕ_T should be taken as 0.85.

5.4 Subgrades and Pavements

An evaluation of the subgrade soils was completed in general accordance with ODOT Geotechnical Design Manual Section 600. As part of this evaluation, the ODOT "Subgrade Analysis" worksheet (V14.76, 1102/0611/2422) was completed and is attached to this report.

Final pavement grades are assumed to approximate existing grades. Based on the existing pavement cross-sections encountered in the borings, the proposed subgrade is presumed to be 12 inches below the existing top of pavement grades (represented as a 1 feet cut in the ODOT "Subgrade Analysis" worksheet).

Based on the GDM, 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. However, unsuitable Uncontrolled Fill (UCF) consisting of coal fragments was encountered in B-004 at 1.5 to 3 feet below existing ground surface, having a thickness of 24 inches. It, therefore, is recommended to remove and replace it with granular engineered fill.

The type and thickness of subgrade modification is determined by the GDM criteria based on the average, low SPT N_{60}-value (N60L) of the subgrade soils in a particular portion of the project area, hand penetrometer value, soil type, and moisture content. Based on these criteria, subgrade modification is anticipated.

Where 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. 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.

Due to the relatively small area for pavement replacement, sulfate content testing was not performed to evaluate potential concerns with global chemical stabilization. It was anticipated that undercutting and replacement with new granular engineered fill would be more economical for this project.

5.4.1 Flexible (Asphalt) and Rigid Pavement Design

In Boring B-004 underlying UCF and loose A-2-4 zones to a depth of 4½ feet below the existing ground surface. These soils may govern the overall subgrade support conditions. As such, we recommend that the selected replacement pavement section granular engineered fill. It should also be noted that the subgrades should be compacted to at least 100 percent of the maximum dry density as determined by ASTM D 698 (Standard Proctor).

All pavement design and paving operations should conform to ODOT specifications. The pavement and subgrade preparation procedures outlined in this report should result in 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 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.

For short projects where the pavement replacement is less than 300 feet, ODOT encourages to use Pavement Design Manual Appendix C. Based on the GDM Subgrade analysis, a design CBR value of 8 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 8

It should also be noted that the design CBR value is based on subgrades compacted to at least 100 percent of the maximum dry density as determined by ASTM D 698 (Standard Proctor).

All pavement design and paving operations should conform to ODOT specifications. The pavement and subgrade preparation procedures outlined in this report should result in 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.

5.5 Construction

5.5.1 Sedimentation 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 Preparation

Prior to proceeding with construction operations, site preparation activities should include the removal of any structures or substructures which are not appropriated for spillway protection, as well as topsoil, root systems, and vegetation from all proposed structure areas.

Replacement pavement subgrade preparation recommendations are provided in Subgrade and Pavement Section.

5.5.3 Excavations and Slopes

The sides of temporary excavations for subsurface drainage pipe, 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. For the relatively shallow depth of excavation activity anticipated for this project, laid-back slopes are likely to be most feasible and economical. In any case, applicable Occupational Safety and Health Administration (OSHA) safety standards must be followed.

Based on the test borings, it is likely that excavations will encounter a range of soil conditions that include the following OSHA designations:

- OSHA Type A soils (cohesive soils with unconfined compressive strengths of 3,000 pounds per square foot (psf) or greater),
- OSHA Type B soils (cohesive soils with unconfined compressive strengths greater than 1,000 psf but less than 3,000 psf), and
- OSHA Type C soils (cohesive soils with unconfined compressive strengths of 1,000 psf or less, granular soils, weathered bedrock, and existing fill materials).

For temporary excavations in Type A, B, C soils, side slopes should be constructed no steeper than ¾ horizontal to 1 vertical (¾H:1V), 1H:1V, and 1½H:1V, respectively. For situations where an excavation encounters a lower strength soil underlying a higher strength soil, the slope of the entire excavation is governed 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 excavation slopes, we recommend that grades be no steeper than 3H:1V without a more extensive geotechnical evaluation of the proposed construction plans and site conditions.

5.5.4 Rock Excavation

For bridge foundation installation, augerable weathered/fractured rock should be excavated. Additionally, the encountered rock in Boring B-002 and B-003 indicates rock excavation beyond the depth of auger refusal will likely be required in some areas to encounter proper foundation bearing material.

As stated in Section 5.2, RQD values of zero percent and high recovery of 90 to 100 percent were determined for the rock extending to approximate Elev. 567 in Boring B-

002 and B-003. As such, footings should be extended to the more suitable material that was encountered below this elevation in Boring B-003.

Based on test data from the rock cores, our evaluations indicate that the weathered/ fractured (augerable) shale bedrock and cored highly fractured to fractured bedrock may be rippable using conventional excavation equipment such as a backhoe or track excavator, with some assistance from pneumatic chippers, jackhammers, or hydraulic wedging equipment.

5.5.5 Construction Dewatering and Groundwater Control

Groundwater conditions encountered in the borings were summarized in Section 4.3. Apart from streamflow influences in the creek, it is our opinion that the "normal" groundwater level can generally be expected at depths corresponding to the bottom of the creek, on the order of 8 to 20 feet (Elevs. 584± to 571±). It should be noted that groundwater elevations can also fluctuate with seasonal and climatic influences, as well as streamflow conditions in the creek. Perched groundwater may be encountered within the pavement subbase, existing fill materials, or existing granular embankment materials that are underlain by relatively impermeable cohesive soils. Perched groundwater may also be encountered at the soil/bedrock interface.

It is our experience that adequate control of groundwater seepage or surface water run-off into shallow excavations should be achievable by minor dewatering systems, such as pumping from prepared sumps. As mentioned in Section 5.1, it is likely that temporary steel casing will be required to support the walls of the drilled shafts and to control groundwater seepage. In the event excessive seepage is encountered during construction, CT should be notified to evaluate whether other dewatering methods are required.

5.5.6 Fill

Material for engineered fill or backfill required to achieve design grades should meet ODOT Item 203 "Embankment Fill" placement and compaction requirements.

The upper profile on-site soils consist predominantly of cohesive soils. For these cohesive soils, a sheepsfoot roller should provide the most effective soil compaction. For new granular engineered fill or dense-graded aggregate pavement base materials, a vibratory smooth-drum roller would be required to provide effective compaction.

6.0 QUALIFICATION OF RECOMMENDATIONS

Our evaluation of design and construction conditions for the proposed bridge replacement and pavement reconstruction has been based on our understanding of the site and project information and the data obtained during our field exploration. The general subsurface conditions were based on interpretation of the data obtained at specific boring locations. Regardless of the thoroughness of a subsurface exploration, 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 for previously developed sites. Therefore, experienced geotechnical engineers should observe earthwork and foundation 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

SCALE IN FEET

0' 2,500' 5,000'

BASE PLAN "SITE AERIAL PLAN" DATED 11/6/2021 OBTAINED FROM GOOGLE EARTH.

SITE LOCATION MAP

PROPOSED ATB OLD MAIN STREET BRIDGE REPLACEMENT

CONNEAUT, OHIO

PREPARED FOR

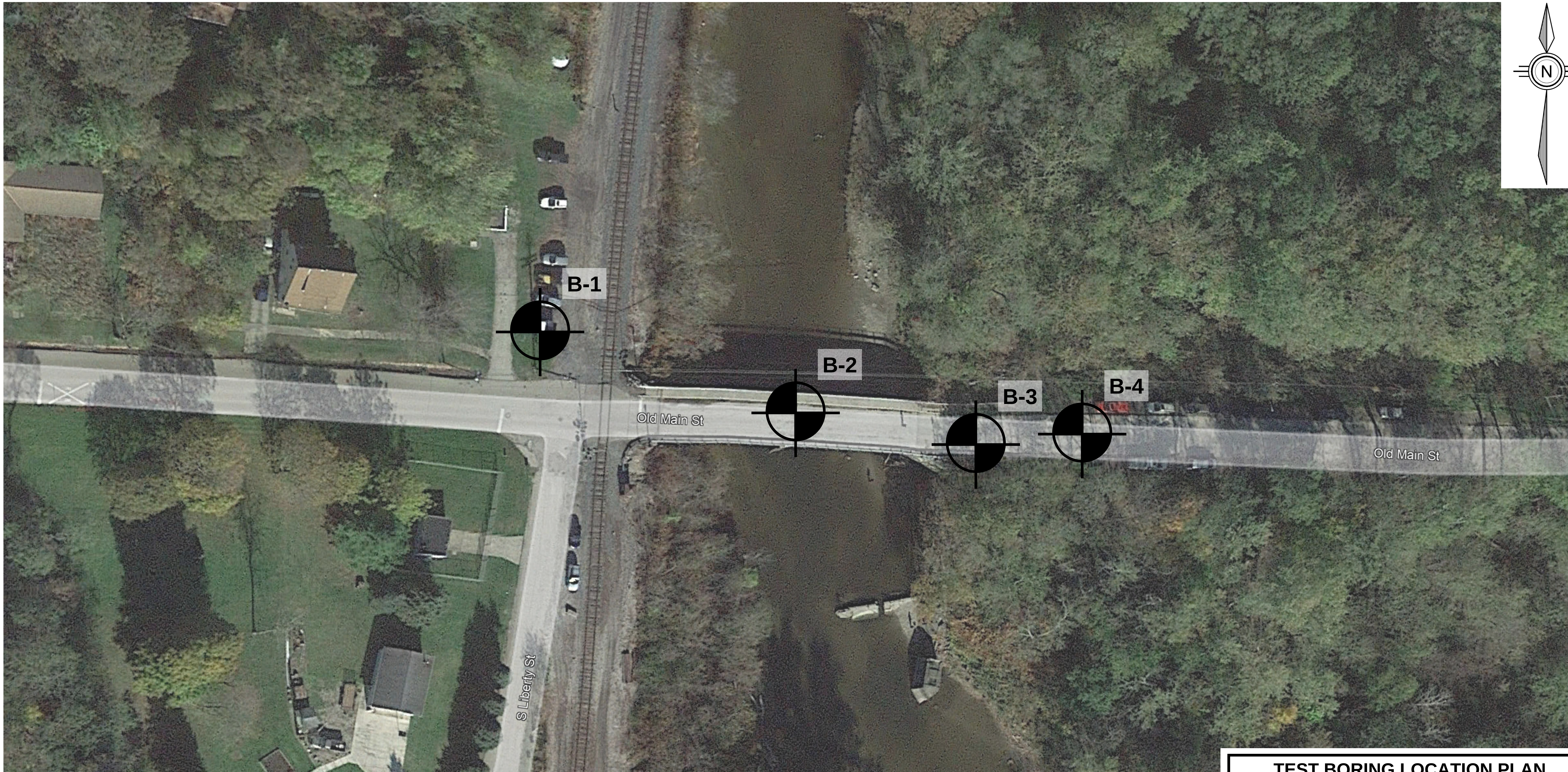
CITY OF CONNEAUT

CONNEAUT, OHIO

DRAWN:	RK / 04/19/23
REVISED:	---
PROJECT No:	232245

PLATE 1.0

CT consultants
engineers • architects • planners



LEGEND:

B-1

APPROXIMATE TEST BORING LOCATION

SCALE IN FEET

0' 75' 150'

BASE PLAN "SITE AERIAL PLAN" DATED 11/6/2021 OBTAINED FROM GOOGLE EARTH.

TEST BORING LOCATION PLAN

PROPOSED ATB OLD MAIN STREET BRIDGE REPLACEMENT

CONNEAUT, OHIO

PREPARED FOR

CITY OF CONNEAUT

CONNEAUT, OHIO

DRAWN:	RK / 04/19/23
REVISED:	---
PROJECT No:	232245

CT consultants

engineers • architects • planners

PLATE 2.0

Figures

Logs of Test Borings
Legend Key
Grain Size Distribution
Rock Core Photo Logs
Point Load Test Results
Slake Durability Test Results

PROJECT: OLD MAIN STREET BRIDGE TYPE: BRIDGE PID: 119471 SFN: 0461254 START: 10/19/23 END: 10/19/23	DRILLING FIRM / OPERATOR: CT CONSULTANTS / CW SAMPLING FIRM / LOGGER: TTL / KKC DRILLING METHOD: 3.25" HSA / NQ2 SAMPLING METHOD: SPT / NQ2	DRILL RIG: CME 75 TRUCK 844 HAMMER: CME AUTOMATIC CALIBRATION DATE: 2/20/23 ENERGY RATIO (%): 72.9	STATION / OFFSET: 10+32, 68' LT. ALIGNMENT: CL CONST. (OLD MAIN ST.) ELEVATION: 592.7 (NAVD88) EOB: 25.7 ft. LAT / LONG: 41.943784, -80.551162

EXPLORATION ID B-001-0-23

MATERIAL DESCRIPTION AND NOTES	ELEV. 592.7	DEPTHS
AGGREGATE BASE - 4 INCHES	592.4	
MEDIUM STIFF, BROWN/BLACK, **SANDY SILT**, SOME CLAY, LITTLE COAL FRAGMENTS, LITTLE GRAVEL, WET FILL	589.7	
VERY LOOSE, GRAY, **GRAVEL AND STONE FRAGMENTS WITH SAND AND SILT**, TRACE CLAY, DAMP FILL @4.8': LOOSE, DARK BROWN	587.4	
STIFF, BLUE/GRAY/BROWN, **SILT AND CLAY**, LITTLE GRAVEL, LITTLE SAND, MOIST	586.7	
VERY STIFF TO HARD, BLUE/GRAY/BROWN, **SILT AND CLAY**, LITTLE SAND, DAMP Qu - 28.9 PSI	584.7	W 584.7
MEDIUM DENSE, BROWN, **COARSE AND FINE SAND**, LITTLE CLAY, LITTLE SHALE FRAGMENTS, TRACE SILT, MOIST	581.4	
DENSE TO VERY DENSE, GRAY, **GRAVEL AND STONE FRAGMENTS**, MOIST TO DAMP **SHALE**, GRAY, SEVERELY WEATHERED, WEAK, HIGHLY FRACTURED. [INFERRED FROM DRILLING]	577.0	TR
SHALE, GRAY, HIGHLY WEATHERED, WEAK TO SLIGHTLY STRONG, JOINTED, HIGHLY FRACTURED, OPEN TO NARROW; RQD 0%, REC 72%. @17.1': Qu - 2,730 PSI @18.3': Qu - 2,860 PSI	572.0	
SHALE, GRAY, HIGHLY TO SLIGHTLY WEATHERED, WEAK TO SLIGHTLY STRONG, JOINTED, HIGHLY FRACTURED, OPEN TO NARROW; RQD 0%, REC 88%. @21.3': Qu - 3,350 PSI	567.0	EOB

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
4 3 3	7	100	SS-1	-	16	15	14	35	20	NP	NP	NP	26	A-4a (4)	
0 0 1	1	100	SS-2	-	38	14	15	29	4	NP	NP	NP	14	A-2-4 (0)	
1 1 2	4	100	SS-3A	-	-	-	-	-	-	-	-	-	-	A-2-4 (V)	
			SS-3B	1.25	-	-	-	-	-	-	-	-	20	A-6a (V)	
2 3 5	10	100	SS-4	4.00	0	3	7	22	68	32	21	11	19	A-6a (8)	
4 7 7	17	100	SS-5	-	-	-	-	-	-	-	-	-	13	A-3a (V)	
10 27 50/1"	-	100	SS-6	-	-	-	-	-	-	-	-	-	6	A-1-a (V)	
50/3"	-	100	SS-7	-	-	-	-	-	-	-	-	-	11	A-1-a (V)	
0		80	NQ2-8											CORE	
0		88	NQ2-9											CORE	

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PUMPED 7 CF CEMENT-BENTONITE GROUT

PROJECT: OLD MAIN STREET BRIDGE	DRILLING FIRM / OPERATOR: CT CONSULTANTS / CW	DRILL RIG: CME 75 TRUCK 844	STATION / OFFSET: 11+71, 6' RT.
TYPE: BRIDGE	SAMPLING FIRM / LOGGER: TTL / KKC	HAMMER: CME AUTOMATIC	ALIGNMENT: CL CONST. (OLD MAIN ST.)
PID: 119471 SFN: 0461254	DRILLING METHOD: 3.25" HSA / NQ2	CALIBRATION DATE: 2/20/23	ELEVATION: 570.1 (NAVD88) EOB: 10.9 ft.
START: 10/17/23 END: 10/17/23	SAMPLING METHOD: SPT / NQ2	ENERGY RATIO (%): 72.9	LAT / LONG: 41.943617, -80.550660

EXPLORATION ID: B-002-0-23

PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 570.1	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
SHALE, GRAY, SEVERELY WEATHERED, WEAK, HIGHLY FRACTURED.	569.2	TR 0.1	50/5"	-	100	SS-1	-	-	-	-	-	-	-	-	-	9	Rock (V)	
SHALE, GRAY, MODERATELY TO HIGHLY WEATHERED, SLIGHTLY TO MODERATELY STRONG, JOINTED, HIGHLY FRACTURED, NARROW; RQD 0%, REC 100%. @2.6': Qu - 2,280 PSI @5.5': Qu - 5,630 PSI	564.2	1–5	0		100	NQ2-2											CORE	
SHALE, GRAY, MODERATELY TO HIGHLY WEATHERED, SLIGHTLY TO MODERATELY STRONG, JOINTED, HIGHLY FRACTURED, NARROW; RQD 0%, REC 90%. @6.2': Qu - 5,170 PSI	559.2	6–10, EOB	0		90	NQ2-3											CORE	

NOTES: THE BORING IS DRILLED THROUGH THE BRIDGE DECK. BRIDGE DECK ELEVATION WAS 593.2 FEET.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 1 BAG ASPHALT PATCH; PUMPED 3 CF CEMENT-BENTONITE GROUT

PROJECT: OLD MAIN STREET BRIDGE	DRILLING FIRM / OPERATOR: CT CONSULTANTS / CW	DRILL RIG: CME 75 TRUCK 844	STATION / OFFSET: 12+67, 9' LT.	EXPLORATION ID B-003-0-23
TYPE: BRIDGE	SAMPLING FIRM / LOGGER: TTL / KKC	HAMMER: CME AUTOMATIC	ALIGNMENT: CL CONST. (OLD MAIN ST.)	
PID: 119471 SFN: 0461254	DRILLING METHOD: 3.25" HSA / NQ2	CALIBRATION DATE: 2/20/23	ELEVATION: 591.7 (NAVD88) EOB: 34.5 ft.	PAGE 1 OF 2
START: 10/18/23 END: 10/18/23	SAMPLING METHOD: SPT / ST / NQ2	ENERGY RATIO (%): 72.9	LAT / LONG: 41.943638, -80.550312	

MATERIAL DESCRIPTION AND NOTES	ELEV. 591.7
ASPHALT - 8.5 INCHES	591.0
AGGREGATE BASE - 15.5 INCHES	589.7
DENSE, GRAY/BROWN, **CRUSHED STONE WITH SAND**, LITTLE SILT, TRACE CLAY, DAMP FILL	588.7
LOOSE TO MEDIUM DENSE, BROWN, **CRUSHED STONE WITH SAND AND SILT**, TRACE CLAY, DAMP FILL @4.5': GRAY/BROWN	583.4
HARD, BROWN, **SANDY SILT**, SOME CLAY, TRACE GRAVEL, TRACE IRON OXIDE STAIN SEAM, MOIST	580.7
STIFF TO VERY STIFF, BROWN, **SANDY SILT**, SOME CLAY, TRACE GRAVEL, TRACE IRON OXIDE STAIN SEAM, DAMP @13': GRAY/BROWN, LITTLE GRAVEL	575.7
MEDIUM DENSE, BROWN, **FINE SAND**, LITTLE GRAVEL, TRACE SILT, TRACE CLAY, MOIST	573.0
MEDIUM DENSE, GRAY, **SEVERELY WEATHERED SHALE**, MOIST [INFERRED FROM DRILLING] @19.3': BLUE/GRAY	571.4
BLUE/GRAY, **WEATHERED SHALE SHALE**, GRAY, SEVERELY WEATHERED, WEAK, HIGHLY WEATHERED.	567.2
SHALE, GRAY, MODERATELY WEATHERED, SLIGHTLY TO MODERATELY STRONG, JOINTED, HIGHLY FRACTURED, NARROW; RQD 0%, REC 95%. @25.4': Qu - 7,042 PSI @28.1': Qu - 2,581 PSI	562.2

DEPTHS: TR (573.0); W 571.4

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
11 15 15	36	44	SS-1	-	50	23	9	16	2	NP	NP	NP	8	A-1-b (0)	
4 5 3	10	100	SS-2	-	-	-	-	-	-	-	-	-	7	A-2-4 (V)	
4 4 5	11	100	SS-3	-	28	21	16	31	4	NP	NP	NP	10	A-2-4 (0)	
4 4 4	10	100	SS-4	-	-	-	-	-	-	-	-	-	12	A-2-4 (V)	
2 3 3	7	100	SS-5	4.50	-	-	-	-	-	-	-	-	16	A-4a (V)	
4 5 4	11	100	SS-6	-	-	-	-	-	-	-	-	-	26	A-4a (V)	
		100	ST-7	2.75	10	15	15	39	21	25	18	7	18	A-4a (5)	
2 5 8	16	100	SS-8	-	-	-	-	-	-	-	-	-	12	A-3 (V)	
5 5 13	22	100	SS-9	-	-	-	-	-	-	-	-	-	13	Rock (V)	
50/5"	-	100	SS-10	-	-	-	-	-	-	-	-	-	9	Rock (V)	
50/5"	-	100	SS-11	-	-	-	-	-	-	-	-	-	6	Rock (V)	
0		95	NQ2-12											CORE	

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 10/23/25 14:39 - X:\PROJECTS\232245.GPJ

PID: 119471	SFN: 0461254	PROJECT: OLD MAIN STREET BRIDGE	STATION / OFFSET: 12+67, 9' LT.	START: 10/18/23	END: 10/18/23	PG 2 OF 2	B-003-0-23

MATERIAL DESCRIPTION AND NOTES	ELEV. 561.7	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
SHALE, GRAY, MODERATELY TO HIGHLY WEATHERED, SLIGHTLY TO MODERATELY STRONG, JOINTED, HIGHLY FRACTURED, NARROW; RQD 0%, REC 100%. *(continued)* @30.3': Qu - 4,994 PSI @32.1': Qu - 7,473 PSI	557.2	31 32 33 34 EOB	0		100	NQ2-13											CORE	

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.25 BAG ASPHALT PATCH; PUMPED 10 CF CEMENT-BENTONITE GROUT

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 10/23/25 14:39 - X:\PROJECTS\232245.GPJ

PROJECT: OLD MAIN STREET BRIDGE TYPE: SUBGRADE PID: 119471 SFN: 0461254 START: 10/18/23 END: 10/18/23	DRILLING FIRM / OPERATOR: CT CONSULTANTS / CW SAMPLING FIRM / LOGGER: TTL / KKC DRILLING METHOD: 3.25" HSA / NQ2 SAMPLING METHOD: SPT / NQ2	DRILL RIG: CME 75 TRUCK 844 HAMMER: CME AUTOMATIC CALIBRATION DATE: 2/20/23 ENERGY RATIO (%): 72.9	STATION / OFFSET: 13+21, 3' RT. ALIGNMENT: CL CONST. (OLD MAIN ST.) ELEVATION: 590.2 (NAVD88) EOB: 7.5 ft. LAT / LONG: 41.943589, -80.550067	EXPLORATION ID B-004-0-23 PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 590.2	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
ASPHALT - 4 INCHES	589.9																	
AGGREGATE BASE - 8 INCHES	589.2	1																
MEDIUM DENSE, BLACK/GRAY, **COAL FRAGMENTS**, TRACE SILT, TRACE CLAY, MOIST FILL	587.5	2	3 5 5	12	100	SS-1	-	-	-	-	-	-	-	-	-	15	UCF (V)	
LOOSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND AND SILT**, TRACE CLAY, DAMP FILL	585.7	3 4	3 2 2	5	100	SS-2	-	28	29	15	25	3	NP	NP	NP	7	A-2-4 (0)	
STIFF, BROWN, **SANDY SILT**, SOME GRAVEL, TRACE CLAY, DAMP FILL		5	5 5 3	10	100	SS-3	-	27	22	13	33	5	NP	NP	NP	10	A-4a (1)	
	583.4	6	7 6 3	11	100	SS-4A	-	-	-	-	-	-	-	-	-	-	A-4a (V)	
STIFF, BROWN, **SILT AND CLAY**, LITTLE SAND, MOIST	582.7	7 EOB				SS-4B	1.50	-	-	-	-	-	-	-	-	22	A-6a (V)	

NOTES: NONE

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

KEY TO SYMBOLS

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT OLD MAIN STREET BRIDGE
PID 119471
OGE NUMBER N/A
PROJECT TYPE STRUCTURE FOUNDATION

LITHOLOGIC SYMBOLS (Unified Soil Classification System)

- A-1-B: Ohio DOT: A-1-b, gravel and/or stone fragments with sand
- A-2-4: Ohio DOT: A-2-4, gravel and/or stone fragments with sand and silt
- F S — A-3: Ohio DOT: A-3, fine sand
- A-3A: Ohio DOT: A-3a, coarse and fine sand
- A-4A: Ohio DOT: A-4a, sandy silt
- A-6A: Ohio DOT: A-6a, silt and clay
- C — COAL: Ohio DOT: Coal or coal blossom
- PAVEMENT OR BASE: Ohio DOT: Pavement or Aggregate base
- SHALE: Ohio DOT: Shale
- WEATHERED SHALE: Ohio DOT: Highly or Severely Weathered Shale

SAMPLER SYMBOLS

- Thin Walled Undisturbed Sample

WELL CONSTRUCTION SYMBOLS

- Bentonite: Bottom of hole
- Soil Cuttings Backfill mixed with Bentonite Pellets or Chips
- Asphalt or Concrete Pavement Patch

ABBREVIATIONS

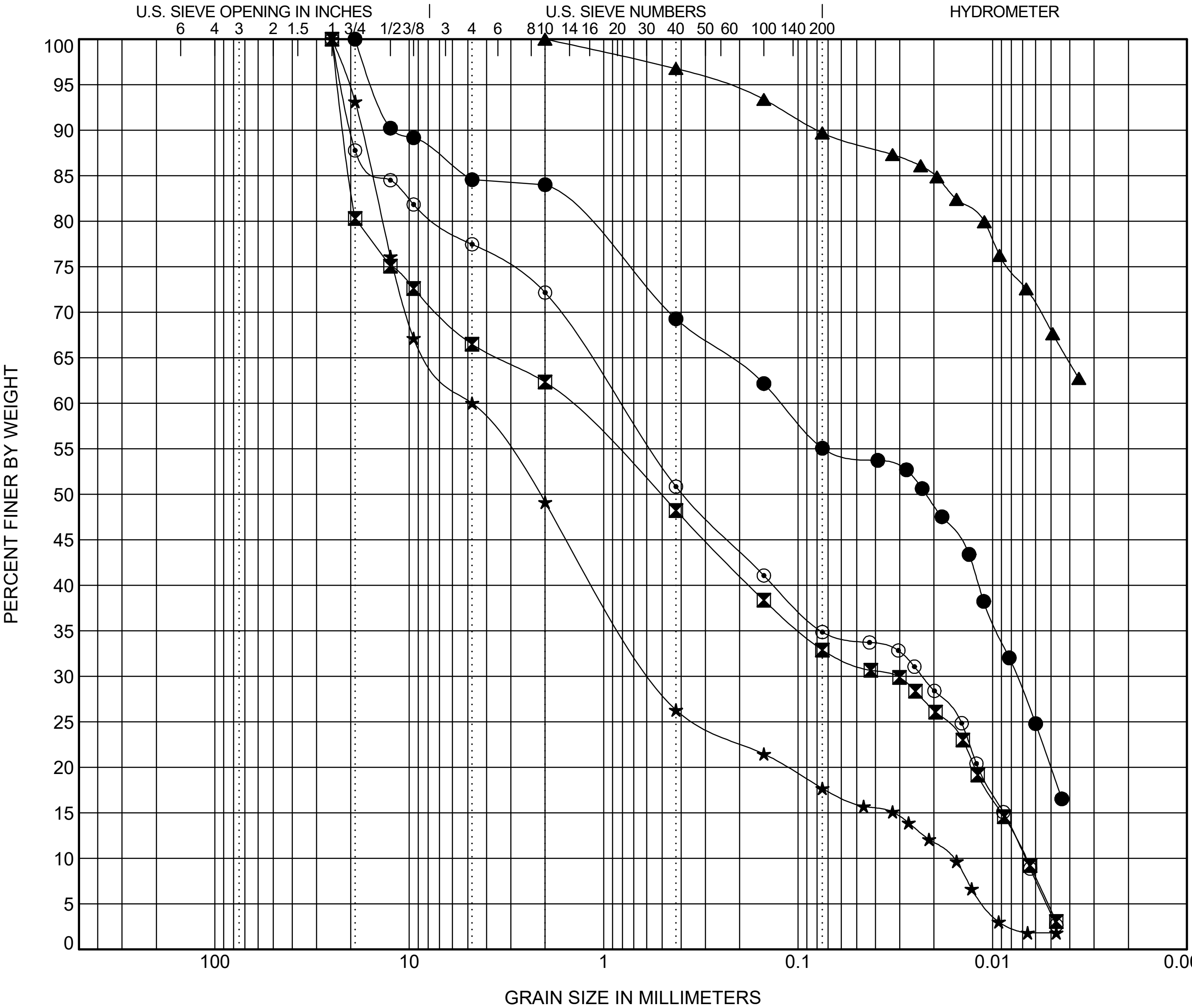
LL	- LIQUID LIMIT (%)
PI	- PLASTIC INDEX (%)
W	- MOISTURE CONTENT (%)
DD	- DRY DENSITY (PCF)
NP	- NON PLASTIC
-200	- PERCENT PASSING NO. 200 SIEVE
PP	- POCKET PENETROMETER (TSF)
TV	- TORVANE
PID	- PHOTOIONIZATION DETECTOR
UC	- UNCONFINED COMPRESSION
ppm	- PARTS PER MILLION
∇	Water Level at Time Drilling, or as Shown
▼	Water Level at End of Drilling, or as Shown
▼	Water Level After 24 Hours, or as Shown

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT OLD MAIN STREET BRIDGE **PID** 119471

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



COBBLES	GRAVEL	SAND coarse	SAND fine	SILT	CLAY

Specimen Identification		ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
● B-001-0-22	1.5	**A-4a ~ SANDY SILT with GRAVEL(ML)**	NP	NP	NP
⊠ B-001-0-22	3.0	**A-2-4 ~ SILTY SAND with GRAVEL(SM)**	NP	NP	NP
▲ B-001-0-22	6.0	**A-6a ~ LEAN CLAY(CL)**	32	21	11
★ B-003-0-22	1.5	**A-1-b ~ SILTY SAND with GRAVEL(SM)**	NP	NP	NP
⊙ B-003-0-22	4.5	**A-2-4 ~ SILTY SAND with GRAVEL(SM)**	NP	NP	NP

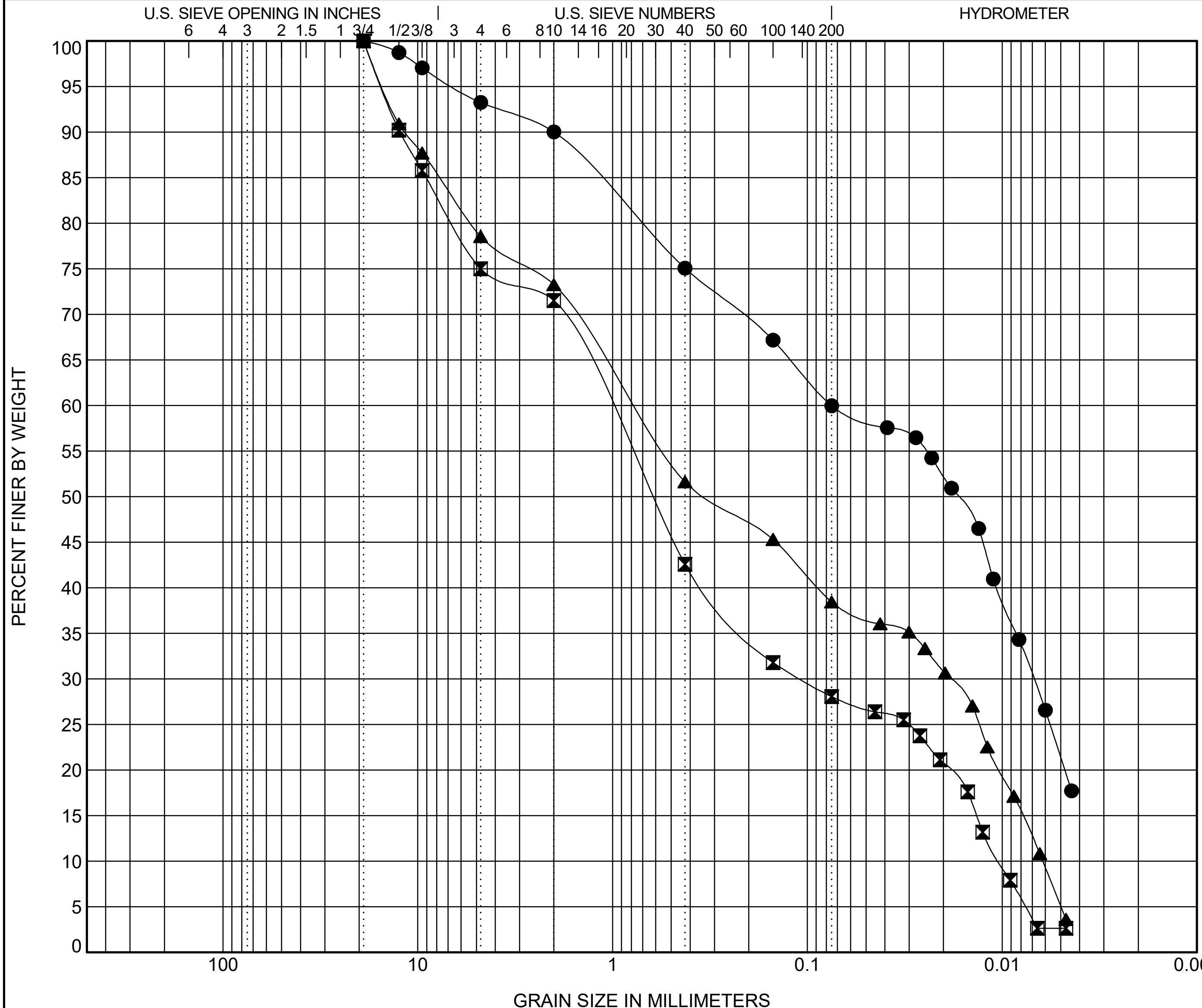
Specimen Identification		D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
● B-001-0-22	1.5	11.789	0.022	0.008		16	15	14	35	20		
⊠ B-001-0-22	3.0	21.749	0.516	0.032	0.007	38	14	15	29	4	0.10	231.27
▲ B-001-0-22	6.0	0.079				0	3	7	22	68		
★ B-003-0-22	1.5	17.585	2.144	0.546	0.016	50	23	9	16	2	3.93	296.43
⊙ B-003-0-22	4.5	19.973	0.388	0.023	0.007	28	21	16	31	4	0.09	121.94

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT OLD MAIN STREET BRIDGE **PID** 119471

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



Specimen Identification		ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
● B-003-0-22	13.0	**A-4a ~ SANDY SILTY CLAY(CL-ML)**	25	18	7
⧗ B-004-0-22	3.0	**A-2-4 ~ SILTY SAND with GRAVEL(SM)**	NP	NP	NP
▲ B-004-0-22	4.5	**A-4a ~ SILTY SAND with GRAVEL(SM)**	NP	NP	NP

Specimen Identification		D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
● B-003-0-22	13.0	1.995	0.017	0.007		10	15	15	39	21		
⧗ B-004-0-22	3.0	12.335	0.632	0.108	0.01	28	29	15	25	3	1.03	104.34
▲ B-004-0-22	4.5	11.575	0.325	0.019	0.006	27	22	13	33	5	0.07	125.25

Office of Geotechnical Engineering

B-001-0-23

Core Date: October 19, 2023					Ground Surface Elevation: 592.7’			
Run #:	Depth		Elevation		Recovery		RQD	
NQ2-8	15.7’	20.7’	577.0’	572.0’	48/60	80%	0/60	0%
NQ2-9	20.7’	25.7’	572.0’	567.0’	53/60	88%	0/60	0%
Old Main Street Bridge, PID 119471								

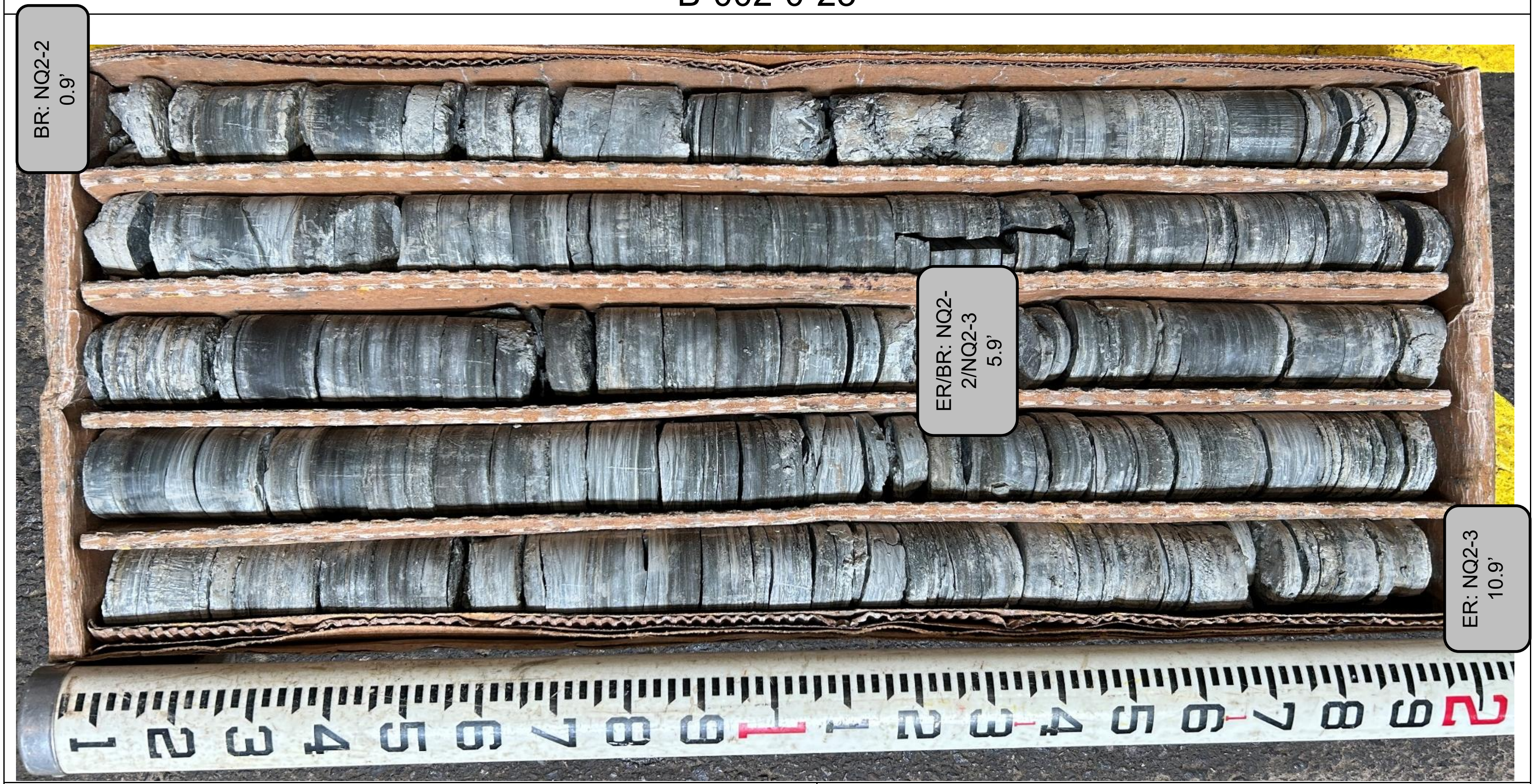
Prepared by:

CT Project No.: 232245

Ohio Department of Transportation
Division of Engineering

Office of Geotechnical Engineering

B-002-0-23



Core Date: October 17, 2023					Ground Surface Elevation: 570.1'			
Run #:	Depth		Elevation		Recovery		RQD	
NQ2-2	0.9'	5.9'	569.2'	564.2'	60/60	100%	0/60	0%
NQ2-3	5.9'	10.9'	564.2'	559.2'	54/60	90%	0/60	0%
Old Main Street Bridge, PID 119471								

Prepared by:

CT Project No.: 232245

B-003-0-23

Core Date: October 18, 2023					Ground Surface Elevation: 591.7'			
Run #:	Depth		Elevation		Recovery		RQD	
NQ2-12	24.5'	29.5'	567.2'	562.2'	57/60	95%	0/60	0%
NQ2-13	29.5'	34.5'	562.2'	557.2'	60/60	100%	0/60	0%
Old Main Street Bridge, PID 119471								

OHIO DEPARTMENT OF TRANSPORTATION
DIVISION OF ENGINEERING

Office of Geotechnical Engineering

PROJECT: ATB Old Main Street Bridge   DISTRICT No.:   PID No. 119471   Tech: KKC

Axial Point Load Strength Calc*: $Is = P / (D_e^2)$ $D_e^2 = 4A/\pi$ A = (WD')   Strength = Is * K   K= 23

Boring #	Sample Elevation (ft)	Material Type	W (in)	W (mm)	D (in)	D (mm)	L/D	Failure Load (kN)	Penetration (in)	Penetration (mm)	Is_{50} (MPa)	Is_{50} (psi)	Strength S, (MPa)	Strength S (psi)
B-1	575.6 – 575.5	Shale	1.98	50.3	1.30	33.0	0.7	1.50	0.176	4.5	0.76	**110.9**	18.9	2730
	574.4 – 574.3	Shale	1.98	50.3	1.26	32.0	0.6	1.5	0.184	4.7	0.79	**114.7**	19.7	2860
	571.4 – 571.3	Shale	1.98	50.3	1.40	35.6	0.7	2.00	0.175	4.4	0.95	**138.3**	23.1	3350
											Average Strength (Sc)			2980
B-2	567.5 – 567.4	Shale	1.98	50.3	1.40	35.6	0.7	1.50	0.053	1.3	0.66	**96.4**	15.7	2280
	564.6 – 564.5	Shale	1.98	50.3	1.35	34.3	0.7	3.50	0.076	1.9	1.62	**234.9**	38.8	5630
	563.9 – 563.8	Shale	1.98	50.3	1.30	33.0	0.7	3.00	0.110	2.8	1.46	**212.2**	35.6	5170
											Average Strength (Sc)			4360
B-3	566.3 – 566.2	Shale	1.98	50.3	1.28	32.5	0.6	4.00	0.115	2.9	1.98	**287.7**	48.6	7042
	563.6 – 563.5	Shale	1.98	50.3	1.25	31.8	0.6	1.50	0.058	1.5	0.73	**106.0**	17.8	2581
	561.4 – 561.3	Shale	1.98	50.3	1.34	34.0	0.7	3.00	0.108	2.7	1.42	**206.6**	34.4	4994
	559.6 – 559.5	Shale	1.98	50.3	1.42	36.1	0.7	4.50	0.185	4.7	2.13	**309.3**	51.5	7473
											Average Strength (Sc)			5523

Comments:

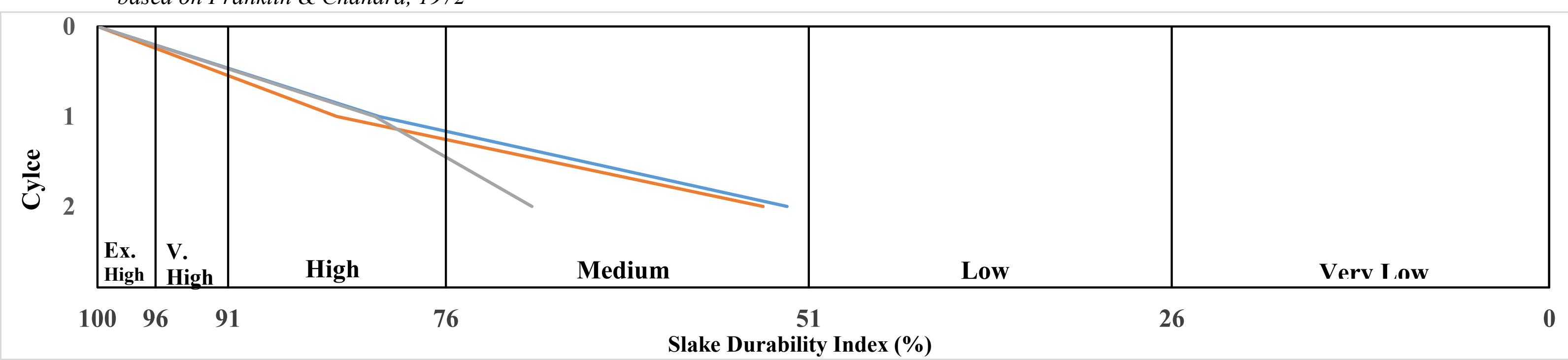
Laboratory Test Result Summary
Slake Durability of Shales and Other Similar Weak Rocks (ASTM D 4644)

CT Project No: 232245
Client: CITY OF CONNEAUT
Project Name: ATB OLD MAIN STREET BRIDGE
Location: Conneaut, Ohio

Technician: *KC*
Date Tested: *4/17/2024*

Sample No.	Boring ID	RC No.	Depth (ft)	As Received Moisture Content (%)	Slake Durability Index		I_{d2} Durability Classification*	I_{d2} Standard Description
					I_{d1} (%)	I_{d2} (%)		
1	B-1	NQ2-8	15.8'-16.5'	0.4	80.6	52.5	Medium	Type I
2	B-2	NQ2-2	24.2'-25'	0.4	83.5	54.1	Medium	Type I
3	B-3	NQ2-12	25.8'-26.5'	0.4	80.9	70.1	Medium	Type I

** based on Franklin & Chandra, 1972*



Laboratory Test Results
Slake Durability of Shales and Other Similar Weak Rocks (ASTM D 4644)

Technician: *KC*
Date Tested: *4/17/2024*

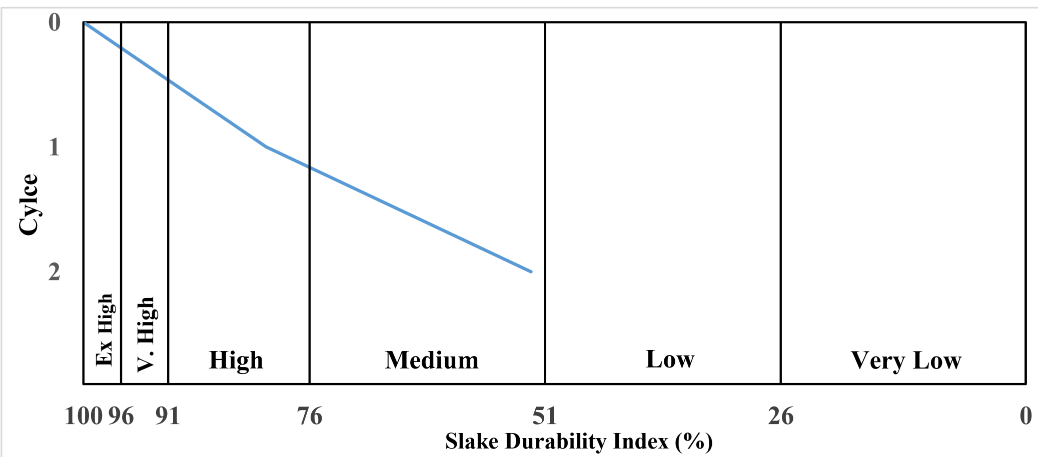
CT Project No: 232245
Client: CITY OF CONNEAUT
Project Name: ATB OLD MAIN STREET BRIDGE
Location: Conneaut, Ohio

Sample No.	Boring ID	RC No.	Depth (ft)
1	B-1	NQ2-8	15.8'-16.5'

As Received Moisture Content (%)	Slake Durability Index		I_{d2} Durability Classification*
	I_{d1} (%)	I_{d2} (%)	
0.4	80.6	52.5	Medium

** based on Franklin & Chandra, 1972*

I_{d2} Standard Description
Type I—Retained specimens remain virtually unchanged.



Sample Before Testing

Sample After Cycle 1

(No Photograph)

Sample After Cycle 2

Laboratory Test Results
Slake Durability of Shales and Other Similar Weak Rocks (ASTM D 4644)

Technician: *KC*
Date Tested: *4/17/2024*

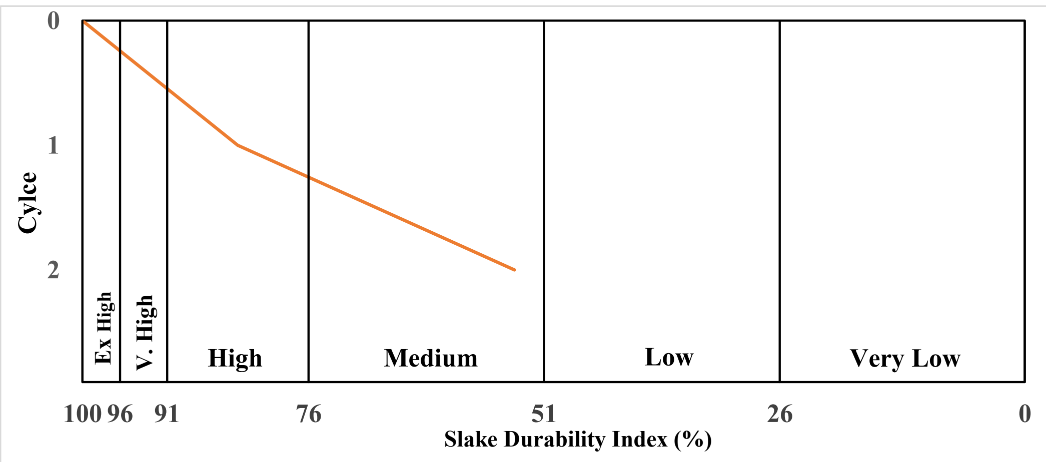
CT Project No: 232245
Client: CITY OF CONNEAUT
Project Name: ATB OLD MAIN STREET BRIDGE
Location: Conneaut, Ohio

Sample No.	Boring ID	RC No.	Depth (ft)
2	B-2	NQ2-2	24.2'-25'

As Received Moisture Content (%)	Slake Durability Index		I_{d2} Durability Classification*
	I_{d1} (%)	I_{d2} (%)	
0.4	83.5	54.1	Medium

** based on Franklin & Chandra, 1972*

I_{d2} Standard Description
Type I—Retained specimens remain virtually unchanged.



Sample Before Testing

Sample After Cycle 1

(No Photograph)

Sample After Cycle 2

Laboratory Test Results
Slake Durability of Shales and Other Similar Weak Rocks (ASTM D 4644)

Technician: *KC*
Date Tested: *4/17/2024*

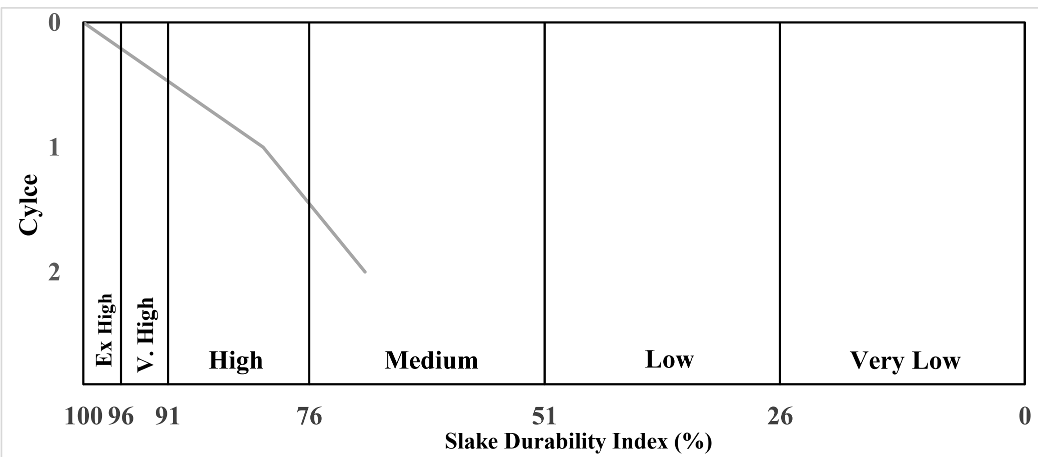
TTL Project No: 232245
Client: CITY OF CONNEAUT
Project Name: ATB OLD MAIN STREET BRIDGE
Location: Conneaut, Ohio

Sample No.	Boring ID	RC No.	Depth (ft)
3	B-3	NQ2-12	25.8'-26.5'

As Received Moisture Content (%)	Slake Durability Index		I_{d2} Durability Classification*
	I_{d1} (%)	I_{d2} (%)	
0.4	80.9	70.1	Medium

** based on Franklin & Chandra, 1972*

I_{d2} Standard Description
Type I—Retained specimens remain virtually unchanged.



Sample Before Testing

Sample After Cycle 1

(No Photograph)

Sample After Cycle 2

Laboratory Test Results
Slake Durability of Shales and Other Similar Weak Rocks (ASTM D 4644)

Technician: *KC*
Date Tested: *4/17/2024*

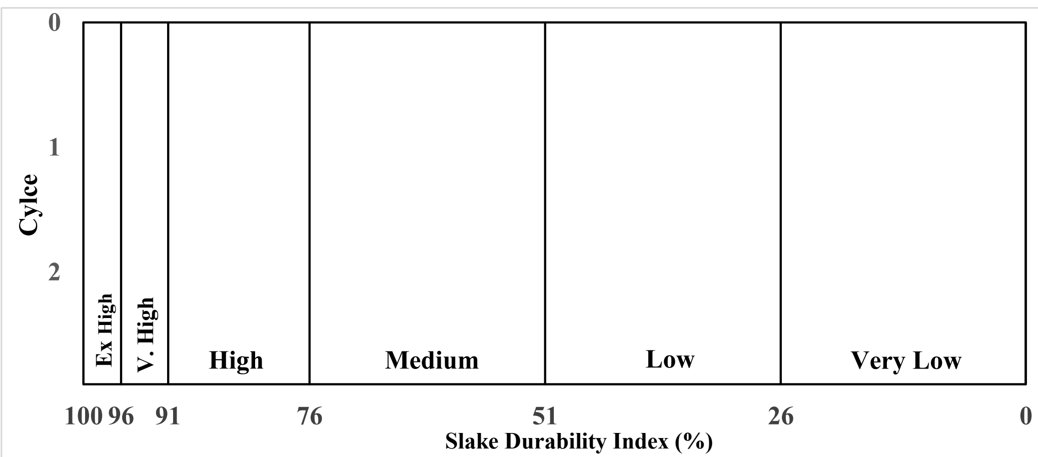
TTL Project No: 232245
Client: CITY OF CONNEAUT
Project Name: ATB OLD MAIN STREET BRIDGE
Location: Conneaut, Ohio

Sample No.	Boring ID	RC No.	Depth (ft)

As Received Moisture Content (%)	Slake Durability Index		I_{d2} Durability Classification*
	I_{d1} (%)	I_{d2} (%)	

** based on Franklin & Chandra, 1972*

I_{d2} Standard Description
Type I—Retained specimens remain virtually unchanged.



Sample Before Testing

Sample After Cycle 1

(No Photograph)

Sample After Cycle 2

Appendix A

Engineering Calculations
(Including ODOT Subgrade Analysis Spreadsheets)

Project No.:	**232245**
Project:	**ATB-Old Main Street Bridge**
Calcs by:	**MSI**
Date:	**2/11/2025**
Revision:	**1**
Date:	**msi, 10/3/2025**
Chekced:	**ihj, 10/23/2025**
Calcs:	**Drilled Shaft Rock Sockets - Vertical Resistance**
Location:	**ATB-Old Main Street**
Substructure:	**Rear (West) Abutment**
Boring(s):	B-001-0-23
Ground Surface Elevation (ft):	593.71
Bottom of Abutment Elev (ft):	583.8
Top of Rock Elevation (ft):	577
Length of Shaft in Soil (ft):	6.8
Shaft in Soil Diameter (in):	42
Shaft in Rock Diameter (in):	36
Shaft in Rock Diameter (ft):	3
End-Bearing at 1.5 x B	
Length of Socket (ft):	4.5
May increase Shaft in soil to 3.5 ft and socket to 3 ft diameter for lateral resistance	
Shaft in Rock Diameter (ft):	3
In this case, 1.5 x B	
Length of Socket (ft):	4.5
BDM 305.4.4.4, minimum 5' socket if rock within 10 ft of ground surface or bottom of shaft cap.	
As noted above, shaft in soil (ft):	6.8
Therefore, governing Length of Socket (ft):	5
End-Bearing Elev. (ft):	572

Calcs:	**Drilled Shaft Rock Sockets - Vertical Resistance**
Location:	**ATB-Old Main Street**
Substructure:	**Rear (West) Abutment**
Look at rock core Qu at bearing to	
2B below bearing:	
2B below foundation/shaft bearing Elev.:	566
Qu (psi):	2730
	2860
	3350
Use Average Qu (psi):	2980
Average Qu (ksf):	429
End-Bearing Resistance (AASHTO LRFD 10.8.3.5.4c-1)	
qp=2.5qu	
(Unfactored) qp (ksf):	**1073**
Resistance Factor (AASHTO LRFD Table 10.5.5.2.4-1)	
f=	0.5
Factored Bearing Resistance (ksf)=	536
Say, Factored Bearing Resistance (ksf)=	**535**
For 3 ft diameter socket,	
Available Resistance (kips)=	3782
Based on provided loading	
Indicated Total Factored Load (kips)=	307.8
Suitable Vertical Resistance?	**YES**
For 3 ft diameter socket,	
Available Resistance (kips)=	3782

Project No.:	232245
Project:	ATB-Old Main ST. Bridge
Calcs by:	msi
Date:	2/12/2025
Review:	
Revision:	1
Revision by:	msi, 10/3/2025
Checked by:	ihj, 10/23/2025
Calcs:	Drilled Shaft Rock Sockets - Lateral Resistance
Location:	Old Main St., Ashtabula, OH
Substructure:	Rear (West) Abutment
Boring(s):	B-001-0-22
GSE (ft):	593.71
Long-Term GWT (ft):	574.2
Bottom of Shaft Elev. (ft):	583.9

Soil

Layer	Soil Type	Top Depth (ft)	Bottom Depth (ft)	Top Elev. (ft)	Bottom Elev. (ft)	Avg. N60	HP (tsf)	Qu (tsf)
Layer 1 - Fill	**Med. Stiff, A-4a**	0	3	593.71	590.71	7	-	-
	Depth below bottom of Pier Cap:			-9.81	-6.81			
Total Unit Wt (pcf):	118	GDM Table 400-4		**Use**	**118**	**pcf**		
Su = N60 x 125 (N60<= 52 bpf) per GDM 404.1								
N60, Su (ksf)=	0.875							
HP, Su (ksf)=	-							
Qu, Su (ksf)=	-							
Say, Su (ksf)=	**0.875**							
Evaluation of Strain at half stress (epsilon 50) from LPILE 2018 Technical Manual								
Su = 1-2 ksf, epsilon 50 =	**0.007**							

Layer	Soil Type	Top Depth (ft)	Bottom Depth (ft)	Top Elev. (ft)	Bottom Elev. (ft)	N60	HP (tsf)	Qu (tsf)
Layer 2 - Fill	**Very loose to loose, A-3a**	3	5.5	590.71	588.21	2.5	-	-
	Depth below bottom of Pier Cap:			-6.81	-4.31			
Total Unit Wt (pcf):	110	GDM Table 400-4		**Use**	**110**	**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	4.25	ft						
sigma-v' (ksf):	0.49							
CN=	1.5	<2 so use:	1.5					
N160 (bpf)=	4							
AASHTO LRFD Table 10.4.6.2.4-1								
N160	Mid-Range Phi (deg)							
4	29.5							
		use	29.5	deg				
GDM Table 400-3 phi Adjustment								
A-3a	-0.5							
Phi (deg) =	**29**	< ODOT Maximum 46 deg, ok						
k Evaluation From LPILE 2018 Technical Manual								
Parameters:	Loose sand and silt							
Range of k-value (pci) =	3.5 to 10.4							
Very loose to loose range of N60	k (pci)							
0	3.5							
10	10.4							
Interpolate for 2.5 bpf for this layer:	5.2							
Say k (pci) =	**5**	**Sand (Reese)**						

Calcs:	**Drilled Shaft Rock Sockets - Lateral Resistance**							
Location:	**Old Main St., Ashtabula, OH**							
Substructure:	**Rear (West) Abutment**							
Layer	**Soil Type**	**Top Depth (ft)**	**Bottom Depth (ft)**	**Top Elev. (ft)**	**Bottom Elev. (ft)**	**N60**	**HP (tsf)**	**Qu (tsf)**
Layer 3	**Stiff to V. Stiff, A-6a**	3	8	590.71	585.71	10	2.63	-
	Depth below bottom of Pier Cap:			-6.81	-1.81			
Total Unit Wt (pcf):	120	GDM Table 400-4		**Use**	**120**	**pcf**		
Su = N60 x 125 (N60<= 52 bpf) per GDM 404.1				Based on Unit Wt for native A-4a in B-002.				
N60, Su (ksf)=	1.25							
HP, Su (ksf)=	2.63							
Say, Su (ksf)=	**2.63**							
Evaluation of Strain at half stress (epsilon 50) from LPILE 2018 Technical Manual								
Su = 2-4 ksf, epsilon 50 =	**0.005**							
Layer	**Soil Type**	**Top Depth (ft)**	**Bottom Depth (ft)**	**Top Elev. (ft)**	**Bottom Elev. (ft)**	**N60**	**HP (tsf)**	**Qu (tsf)**
Layer 4	**Medium dense, A-3a**	8	11.3	585.71	582.41	17	-	-
	Depth below bottom of Pier Cap:			-1.81	1.49			
Total Unit Wt (pcf):	122	GDM Table 400-4		**Use**	**122**	**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	9.65	ft						
sigma-v' (ksf):	1.33							
CN=	1.1	<2 so use:	1.1					
N160 (bpf)=	19							
AASHTO LRFD Table 10.4.6.2.4-1								
N160	Mid-Range Phi (deg)							
10	32.5							
30	37.5							
N160	Phi (deg)							
19	34.8	use	34.5	deg				
GDM Table 400-3 phi Adjustment								
A-3a	-0.5							
Phi (deg) =	**34**	< ODOT Maximum 46 deg, ok						
k Evaluation From LPILE 2018 Technical Manual								
Parameters:	Loose sand and silt							
Range of k-value (pci) =	8 to 27	k, submerged sand						
Medium dense range of N60	k (pci)							
11	8							
30	27							
Interpolate for 17 bpf for this layer:	14.0							
Say k (pci) =	**14**	**Sand (Reese)**						

Calcs:	**Drilled Shaft Rock Sockets - Lateral Resistance**									
Location:	**Old Main St., Ashtabula, OH**									
Substructure:	**Rear (West) Abutment**									
Augerable Weathered Bedrock										
Layer	**Rock Type**	**Top Depth (ft)**	**Bottom Depth (ft)**	**Top Elev. (ft)**	**Bottom Elev. (ft)**	**SPT Result**				
Layer 4	**Weathered Shale**	11.3	15.6	582.41	578.11	50/3"				
			Depth below bottom of Pier Cap:	1.49	5.79					
Total Unit Wt (pcf):	150-160	GDM Table 400-5		**Use**	**150**	**pcf**				
Qu based on SPT Results per GDM 404.3										
Qu (ksf)=0.092x(Nrate)90 (bpf)										
ER(%)=	72.9									
N72.5=50/5" x 12" =	200	bpf								
N90 = 72.5/90 x 120 bpf =	161	bpf								
Qu (ksf) =	14.8									
Qu (psi) =	102.9									
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 18,000 psi is 1%, then strain at half max stress (krm) is calculated by:										
Half max stress = Qu/2 =	52.0	psi								
krm = 1% x (52 psi / 18,000 psi) =	0.0029	%								
krm (decimal format) =	**0.000029**									
Bedrock										
Layer	**Soil Type**	**Top Depth (ft)**	**Bottom Depth (ft)**	**Top Elev. (ft)**	**Bottom Elev. (ft)**	**RQD (%)**	**Rec (%)**	**Avg. Qu (psi)**	**Total Unit Wt (pcf)**	
Layer 5	**Shale - Highly Weathered**	15.6	25.7	578.11	568.01	0	100	2981.67	150	at 17-21 ft
	Weak to Slightly Strong									
			Depth below bottom of Pier Cap:	5.79	15.89					
Total Unit Wt (pcf):	**150 - 160**	GDM Table 400-5		**Use**	**150**	**pcf**				
Qu (psi)=	**2,982**									
From GDM Table 400-6										
Qu (psi)	**E (psi)**									
2,250	**200,000**									
3,600	**320,000**									
Interpolation for Qu (psi) = 2982, E(psi):	**265,067**									
From GDM Table 400-6, say E (psi) =	**265,067**									
If Strain at 265067 psi is 1%, then strain at half max stress (krm) is calculated by:										
Half max stress = Qu/2 =	1,491	psi								
krm = 1% x (1491 psi / 265067 psi) =	0.0056	%								
krm (decimal format) =	**0.000056**									

==

LPile for Version 2022-12.012

License ID : 5279320353
License Type : (Office Cloud License)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

This software is licensed for exclusive use by:
Verdantas Inc.

==

This model was prepared by:
Muhammad.Iqbal

Files Used for Analysis

Path to file locations:
\Users\Muhammad.Iqbal\Documents\ATB Old Main Street - 09172025 ODOT Comments\L-Pile\Rear (West) Abutment Drilled Shaft\

Name of input data file:
36-inch Dia.lp12d

Name of output report file:
36-inch Dia.lp12o

Name of plot output file:
36-inch Dia.lp12p

Name of runtime message file:
36-inch Dia.lp12r

Date and Time of Analysis

Date: October 22, 2025   Time: 21:14:37

Problem Title

Project Name: ATB Old Main Street Bridtge

Job Number: 232245

Client: City of Conneaut

Engineer: msi

Description: Lateral Shaft Resistance - Rear Abutment

Program Options and Settings

Computational Options:
- Conventional Analysis

Engineering Units Used for Data Input and Computations:
- US Customary System Units (pounds, feet, inches)

Analysis Control Options:

- Maximum number of iterations allowed	=	500
- Deflection tolerance for convergence	=	1.0000E-05 in
- Maximum allowable deflection	=	100.0000 in
- Number of pile increments	=	100

Loading Type and Number of Cycles of Loading:
- Static loading specified

- Analysis uses p-y modification factors for p-y curves
- Analysis uses layering correction (Method of Georgiadis)
- Analysis includes loading by multiple distributed lateral loads acting on pile
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- Input of moment resistance at the pile tip not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:
- Output files use decimal points to denote decimal symbols.
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1
- No p-y curves to be computed and reported for user-specified depths
- Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined = 1
Total length of pile = 11.800 ft
Depth of ground surface below top of pile = 0.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	36.0000
2	11.800	36.0000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile
Cross-sectional Shape = Circular Pile
Length of section = 11.800000 ft

```
   Width of top of section                                =    36.000000 in
   Width of bottom of section                             =    36.000000 in
   Top Area                                               =        1018. sq. in
   Bottom Area                                            =        1018. sq. in
   Moment of Inertia at Top                               =       82448. in^4
   Moment of Inertia at Bottom                            =       82448. in^4
   Elastic Modulus                                        =     3604997. psi



--------------------------------------------------------------------------------
                       Soil and Rock Layering Information
--------------------------------------------------------------------------------

The soil profile is modelled using 3 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer              =       0.0000 ft
   Distance from top of pile to bottom of layer           =     5.000000 ft
   Effective unit weight at top of layer                  =   122.000000 pcf
   Effective unit weight at bottom of layer               =   122.000000 pcf
   Friction angle at top of layer                         =    34.000000 deg.
   Friction angle at bottom of layer                      =    34.000000 deg.
   Subgrade k at top of layer                             =       0.0000 pci
   Subgrade k at bottom of layer                          =       0.0000 pci

   NOTE: Default values for subgrade k will be computed for this layer.

Layer 2 is weak rock, p-y criteria by Reese, 1997

   Distance from top of pile to top of layer              =     5.000000 ft
   Distance from top of pile to bottom of layer           =     6.800000 ft
   Effective unit weight at top of layer                  =   150.000000 pcf
   Effective unit weight at bottom of layer               =   150.000000 pcf
   Uniaxial compressive strength at top of layer          =   103.000000 psi
   Uniaxial compressive strength at bottom of layer       =   103.000000 psi
   Initial modulus of rock at top of layer                =       18000. psi
   Initial modulus of rock at bottom of layer             =       18000. psi
   RQD of rock at top of layer                            =       0.0000 %
   RQD of rock at bottom of layer                         =       0.0000 %
   k rm of rock at top of layer                           =    0.0000290
   k rm of rock at bottom of layer                        =    0.0000290


Layer 3 is massive rock, p-y criteria by Liang et al., 2009

   Distance from top of pile to top of layer              =     6.800000 ft
   Distance from top of pile to bottom of layer           =    11.800000 ft
   Effective unit weight at top of layer                  =   155.000000 pcf
```

```
   Effective unit weight at bottom of layer            =   155.000000 pcf
   Uniaxial compressive strength at top of layer       =        2730. psi
   Uniaxial compressive strength at bottom of layer    =        3350. psi
   Poisson's ratio at top of layer                     =     0.180000
   Poisson's ratio at bottom of layer                  =     0.180000
   Option 1: Intact rock modulus at top of layer       =       0.0000 psi
             Intact rock modulus at bottom of layer    =       0.0000 psi
   Option 1: Geologic Strength Index for layer         =    30.000000
   Option 2: Rock mass modulus at top of layer         =     1.380000 psi
             Rock mass modulus at bottom of layer      =     1.450000 psi
   Option 2 will use the input value of rock mass modulus to compute the p-y curve
            in massive rock.
   The rock type is (sedimentary) shales, Hoek-Brown Material Constant mi = 6


 (Depth of the lowest soil layer extends 0.000 ft below the pile tip)


**** Warning - Possible Input Data Error ****

Values entered for effective unit weight of rock were outside the limits of
50 pcf to 150 pcf.

The maximum input value, in layer   1, for effective unit weight = 155.00 pcf

This data may be erroneous. Please check your data.



--------------------------------------------------------------------------------
                        Summary of Input Soil Properties
--------------------------------------------------------------------------------

Layer         Soil Type          Layer      Effective    Angle of     Uniaxial
              E50                    Rock Mass    Geologic     Int. Rock
Hoek-Brown
 Num.           Name             Depth       Unit Wt.    Friction        qu
RQD %         or           kpy        Modulus     Strength      Modulus     Material
    Poisson's
      (p-y Curve Type)        ft           pcf          deg.         psi
              krm          pci          psi         Index         psi       Index,
mi      Ratio
-----   -------------------   ----------   ----------   ----------   ----------
----------   ----------   ----------   ----------   ----------   ----------
----------   ----------
  1           Sand                  0.00     122.0000      34.0000       --
 --          --         default        --           --            0.00
0.00        0.00
         (Reese, et al.)         5.0000     122.0000      34.0000       --
 --          --         default        --           --            0.00
```

```
0.00         0.00
  2            Weak               5.0000     150.0000       --         103.0000
   0.00     2.90E-05       --           18000.       --             0.00
0.00         0.00
               Rock               6.8000     150.0000       --         103.0000
   0.00     2.90E-05       --           18000.       --             0.00
0.00         0.00
  3           Massive             6.8000     155.0000       --            2730.
 --           --           --           1.3800      30.0000         0.00
6.0000       0.1800
               Rock              11.8000     155.0000       --            3350.
 --           --           --                       30.0000         0.00
6.0000       0.1800
```

Modification Factors for p-y Curves

Distribution of p-y modifiers with depth defined using 2 points

Point No.	Depth X ft	p-mult	y-mult
1	0.000	0.8000	1.0000
2	8.000	0.8000	1.0000

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

```
Load     Load         Condition                 Condition            Axial Thrust
Compute Top y     Run Analysis
 No.     Type            1                         2                  Force, lbs
vs. Pile Length
```

```
-----   ----   --------------------   ----------------------   ----------------
---------------    ------------
   1     2     V =       63620. lbs   S =       0.0000 in/in           307800.
    No                Yes
```

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

---

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Moment-curvature properties were derived from elastic section properties

---

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	56362.
2	5.0000	5.0000	No	Yes	N.A.	N.A.
3	6.8000	6.8000	No	Yes	N.A.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Shear and Pile-head Rotation (Loading Type 2)

Shear force at pile head = 63620.0 lbs
Rotation of pile head = 0.000E+00 radians
Axial load at pile head = 307800.0 lbs

(Zero slope for this load indicates fixed-head conditions)

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	0.02969	-4369015.	63620.	0.00	1256.	2.97E+11	0.00	0.00	0.00
0.1180	0.02968	-4278924.	63617.	-2.06E-05	1237.	2.97E+11	-4.098	195.5222	0.00
0.2360	0.02964	-4188833.	63608.	-4.08E-05	1217.	2.97E+11	-8.184	391.0443	0.00
0.3540	0.02956	-4098750.	63594.	-6.05E-05	1197.	2.97E+11	-12.246	586.5665	0.00
0.4720	0.02946	-4008682.	63574.	-7.98E-05	1178.	2.97E+11	-16.274	782.0886	0.00
0.5900	0.02934	-3918639.	63548.	-9.87E-05	1158.	2.97E+11	-20.255	977.6108	0.00
0.7080	0.02918	-3828629.	63516.	-1.17E-04	1138.	2.97E+11	-24.179	1173.	0.00
0.8260	0.02901	-3738659.	63479.	-1.35E-04	1119.	2.97E+11	-28.036	1369.	0.00
0.9440	0.02880	-3648737.	63437.	-1.53E-04	1099.	2.97E+11	-31.816	1564.	0.00
1.0620	0.02857	-3558872.	63389.	-1.70E-04	1079.	2.97E+11	-35.508	1760.	0.00
1.1800	0.02832	-3469070.	63337.	-1.87E-04	1060.	2.97E+11	-39.105	1955.	0.00
1.2980	0.02804	-3379340.	63279.	-2.03E-04	1040.	2.97E+11	-42.596	2151.	0.00
1.4160	0.02775	-3289688.	63216.	-2.19E-04	1021.	2.97E+11	-45.973	2346.	0.00

1.5340	0.02742	-3200121.	63149.	-2.34E-04	1001.	2.97E+11	-49.228	2542.	0.00
1.6520	0.02708	-3110647.	63077.	-2.49E-04	981.5093	2.97E+11	-52.352	2737.	0.00
1.7700	0.02672	-3021271.	63000.	-2.64E-04	961.9968	2.97E+11	-55.339	2933.	0.00
1.8880	0.02633	-2931999.	62920.	-2.78E-04	942.5071	2.97E+11	-58.180	3128.	0.00
2.0060	0.02593	-2842838.	62836.	-2.92E-04	923.0415	2.97E+11	-60.868	3324.	0.00
2.1240	0.02551	-2753794.	62748.	-3.05E-04	903.6014	2.97E+11	-63.397	3519.	0.00
2.2420	0.02507	-2664870.	62656.	-3.18E-04	884.1877	2.97E+11	-65.761	3715.	0.00
2.3600	0.02461	-2576073.	62562.	-3.31E-04	864.8016	2.97E+11	-67.953	3910.	0.00
2.4780	0.02413	-2487407.	62464.	-3.43E-04	845.4440	2.97E+11	-69.968	4106.	0.00
2.5960	0.02364	-2398876.	62364.	-3.54E-04	826.1160	2.97E+11	-71.800	4301.	0.00
2.7140	0.02313	-2310484.	62261.	-3.66E-04	806.8183	2.97E+11	-73.444	4497.	0.00
2.8320	0.02260	-2222235.	62156.	-3.76E-04	787.5517	2.97E+11	-74.896	4693.	0.00
2.9500	0.02206	-2134131.	62049.	-3.87E-04	768.3169	2.97E+11	-76.152	4888.	0.00
3.0680	0.02151	-2046175.	61940.	-3.97E-04	749.1145	2.97E+11	-77.206	5084.	0.00
3.1860	0.02094	-1958370.	61830.	-4.06E-04	729.9449	2.97E+11	-78.055	5279.	0.00
3.3040	0.02035	-1870718.	61719.	-4.15E-04	710.8086	2.97E+11	-78.697	5475.	0.00
3.4220	0.01976	-1783219.	61608.	-4.24E-04	691.7059	2.97E+11	-79.127	5670.	0.00
3.5400	0.01915	-1695875.	61495.	-4.32E-04	672.6370	2.97E+11	-79.343	5866.	0.00
3.6580	0.01854	-1608687.	61383.	-4.40E-04	653.6021	2.97E+11	-79.343	6061.	0.00
3.7760	0.01791	-1521654.	61271.	-4.48E-04	634.6012	2.97E+11	-79.124	6257.	0.00
3.8940	0.01727	-1434777.	61159.	-4.55E-04	615.6343	2.97E+11	-78.684	6452.	0.00
4.0120	0.01662	-1348055.	61048.	-4.61E-04	596.7011	2.97E+11	-78.023	6648.	0.00
4.1300	0.01596	-1261486.	60938.	-4.68E-04	577.8015	2.97E+11	-77.139	6843.	0.00
4.2480	0.01530	-1175070.	60830.	-4.73E-04	558.9351	2.97E+11	-76.031	7039.	0.00
4.3660	0.01462	-1088803.	60723.	-4.79E-04	540.1014	2.97E+11	-74.698	7234.	0.00

4.4840	0.01394	-1002684.	60619.	-4.84E-04	521.2999	2.97E+11	-73.140	7430.	0.00
4.6020	0.01325	-916710.	60516.	-4.88E-04	502.5300	2.97E+11	-71.358	7625.	0.00
4.7200	0.01256	-830876.	60417.	-4.92E-04	483.7910	2.97E+11	-69.352	7821.	0.00
4.8380	0.01186	-745180.	60320.	-4.96E-04	465.0818	2.97E+11	-67.122	8016.	0.00
4.9560	0.01115	-659618.	60227.	-5.00E-04	446.4018	2.97E+11	-64.669	8212.	0.00
5.0740	0.01044	-574183.	58248.	-5.03E-04	427.7497	2.97E+11	-2730.	370161.	0.00
5.1920	0.00973	-494220.	54316.	-5.05E-04	410.2923	2.97E+11	-2824.	411127.	0.00
5.3100	0.00901	-419919.	50256.	-5.07E-04	394.0709	2.97E+11	-2911.	457434.	0.00
5.4280	0.00829	-351454.	46079.	-5.09E-04	379.1237	2.97E+11	-2988.	510336.	0.00
5.5460	0.00757	-288980.	41800.	-5.11E-04	365.4844	2.97E+11	-3055.	571525.	0.00
5.6640	0.00685	-232631.	37435.	-5.12E-04	353.1823	2.97E+11	-3110.	643354.	0.00
5.7820	0.00612	-182517.	33002.	-5.13E-04	342.2415	2.97E+11	-3152.	729201.	0.00
5.9000	0.00539	-138723.	28521.	-5.14E-04	332.6802	2.97E+11	-3177.	834118.	0.00
6.0180	0.00467	-101297.	24019.	-5.14E-04	324.5095	2.97E+11	-3183.	966043.	0.00
6.1360	0.00394	-70253.	19525.	-5.15E-04	317.7321	2.97E+11	-3165.	1138287.	0.00
6.2540	0.00321	-45555.	15078.	-5.15E-04	312.3399	2.97E+11	-3115.	1375174.	0.00
6.3720	0.00248	-27103.	10732.	-5.15E-04	308.3114	2.97E+11	-3023.	1726952.	0.00
6.4900	0.00175	-14712.	6563.	-5.15E-04	305.6063	2.97E+11	-2865.	2319055.	0.00
6.6080	0.00102	-8066.	2752.	-5.15E-04	304.1554	2.97E+11	-2518.	3496274.	0.00
6.7260	2.90E-04	-6470.	445.7268	-5.15E-04	303.8069	2.97E+11	-738.960	3603210.	0.00
6.8440	-4.39E-04	-6355.	-77.436	-5.15E-04	303.7818	2.97E+11	0.02865	92.3876	0.00
6.9620	-0.00117	-6240.	-77.362	-5.15E-04	303.7567	2.97E+11	0.07632	92.4657	0.00
7.0800	-0.00190	-6125.	-77.220	-5.15E-04	303.7316	2.97E+11	0.1241	92.5440	0.00
7.1980	-0.00263	-6009.	-77.011	-5.15E-04	303.7064	2.97E+11	0.1719	92.6225	0.00
7.3160	-0.00336	-5894.	-76.733	-5.15E-04	303.6811	2.97E+11	0.2198	92.7010	0.00

7.4340	-0.00409	-5777.	-76.388	-5.15E-04	303.6557	2.97E+11	0.2678	92.7797	0.00
7.5520	-0.00482	-5661.	-75.975	-5.15E-04	303.6302	2.97E+11	0.3159	92.8583	0.00
7.6700	-0.00555	-5543.	-75.493	-5.15E-04	303.6046	2.97E+11	0.3641	92.9370	0.00
7.7880	-0.00628	-5425.	-74.944	-5.15E-04	303.5788	2.97E+11	0.4123	93.0157	0.00
7.9060	-0.00701	-5306.	-74.325	-5.16E-04	303.5529	2.97E+11	0.4607	93.0944	0.00
8.0240	-0.00774	-5186.	-73.549	-5.16E-04	303.5267	2.97E+11	0.6364	116.4661	0.00
8.1420	-0.00847	-5065.	-72.605	-5.16E-04	303.5002	2.97E+11	0.6970	116.5641	0.00
8.2600	-0.00920	-4943.	-71.575	-5.16E-04	303.4735	2.97E+11	0.7577	116.6620	0.00
8.3780	-0.00993	-4818.	-70.459	-5.16E-04	303.4464	2.97E+11	0.8186	116.7599	0.00
8.4960	-0.01066	-4693.	-69.256	-5.16E-04	303.4189	2.97E+11	0.8795	116.8578	0.00
8.6140	-0.01139	-4565.	-67.968	-5.16E-04	303.3911	2.97E+11	0.9406	116.9556	0.00
8.7320	-0.01212	-4436.	-66.593	-5.16E-04	303.3628	2.97E+11	1.0017	117.0534	0.00
8.8500	-0.01285	-4304.	-65.131	-5.16E-04	303.3341	2.97E+11	1.0630	117.1512	0.00
8.9680	-0.01358	-4171.	-63.582	-5.16E-04	303.3049	2.97E+11	1.1243	117.2489	0.00
9.0860	-0.01431	-4035.	-61.947	-5.16E-04	303.2753	2.97E+11	1.1858	117.3467	0.00
9.2040	-0.01504	-3897.	-60.224	-5.16E-04	303.2451	2.97E+11	1.2473	117.4443	0.00
9.3220	-0.01577	-3756.	-58.414	-5.16E-04	303.2143	2.97E+11	1.3090	117.5420	0.00
9.4400	-0.01650	-3612.	-56.517	-5.16E-04	303.1830	2.97E+11	1.3707	117.6396	0.00
9.5580	-0.01723	-3466.	-54.532	-5.16E-04	303.1511	2.97E+11	1.4326	117.7372	0.00
9.6760	-0.01796	-3317.	-52.460	-5.16E-04	303.1186	2.97E+11	1.4946	117.8348	0.00
9.7940	-0.01869	-3165.	-50.300	-5.16E-04	303.0854	2.97E+11	1.5566	117.9324	0.00
9.9120	-0.01942	-3010.	-48.051	-5.16E-04	303.0515	2.97E+11	1.6188	118.0299	0.00
10.0300	-0.02015	-2852.	-45.715	-5.16E-04	303.0169	2.97E+11	1.6811	118.1274	0.00
10.1480	-0.02088	-2690.	-43.291	-5.16E-04	302.9816	2.97E+11	1.7435	118.2248	0.00
10.2660	-0.02161	-2524.	-40.778	-5.16E-04	302.9455	2.97E+11	1.8059	118.3223	0.00

```
   10.3840   -0.02234     -2356.    -38.176  -5.16E-04   302.9087   2.97E+11
1.8685   118.4197       0.00
   10.5020   -0.02307     -2183.    -35.486  -5.16E-04   302.8710   2.97E+11
1.9312   118.5170       0.00
   10.6200   -0.02380     -2006.    -32.707  -5.16E-04   302.8324   2.97E+11
1.9940   118.6144       0.00
   10.7380   -0.02453     -1826.    -29.839  -5.16E-04   302.7930   2.97E+11
2.0568   118.7117       0.00
   10.8560   -0.02526     -1641.    -26.882  -5.16E-04   302.7527   2.97E+11
2.1198   118.8090       0.00
   10.9740   -0.02600     -1452.    -23.835  -5.16E-04   302.7114   2.97E+11
2.1829   118.9062       0.00
   11.0920   -0.02673     -1259.    -20.700  -5.16E-04   302.6692   2.97E+11
2.2461   119.0034       0.00
   11.2100   -0.02746     -1061.    -17.474  -5.16E-04   302.6260   2.97E+11
2.3094   119.1006       0.00
   11.3280   -0.02819   -858.649    -14.160  -5.16E-04   302.5819   2.97E+11
2.3728   119.1978       0.00
   11.4460   -0.02892   -651.443    -10.755  -5.16E-04   302.5366   2.97E+11
2.4363   119.2949       0.00
   11.5640   -0.02965   -439.351     -7.260  -5.16E-04   302.4903   2.97E+11
2.4998   119.3920       0.00
   11.6820   -0.03038   -222.246     -3.675  -5.16E-04   302.4429   2.97E+11
2.5635   119.4891       0.00
   11.8000   -0.03111       0.00       0.00  -5.16E-04   302.3944   2.97E+11
2.6273    59.7931       0.00

* The above values of total stress are combined axial and bending stresses.


Output Summary for Load Case No. 1:

Pile-head deflection              =     0.02969370 inches
Computed slope at pile head       =       0.000000 radians
Maximum bending moment            =      -4369015. inch-lbs
Maximum shear force               =         63620. lbs
Depth of maximum bending moment   =       0.000000 feet below pile head
Depth of maximum shear force      =       0.000000 feet below pile head
Number of iterations              =             11
Number of zero deflection points =              1


--------------------------------------------------------------------------------
            Summary of Pile-head Responses for Conventional Analyses
--------------------------------------------------------------------------------

Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
```

Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	V, lb	63620.	S, rad	0.00	307800.	0.02969	0.00	63620.	-4369015.

Maximum pile-head deflection = 0.0296937010 inches
Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

Summary of Warning Messages

The following warning was reported 180 times

***** Warning *****

The input value for k_rm used by the weak rock criteria is smaller than 0.00005. This value is outside the recommended range of 0.00005 to 0.0005. Please check your input data for accuracy.

The following warning was reported 516 times

WARNING: The ratio of rock mass modulus to uniaxial compressive strength for massive rock appears to be outside the usual range of values.

The analysis ended normally.

```
================================================================================

                          LPile for Version 2022-12.012

                           License ID : 5279320353
                      License Type : (Office Cloud License)

                  Analysis of Individual Piles and Drilled Shafts
                 Subjected to Lateral Loading Using the p-y Method
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================================================================================


This model was prepared by:
Muhammad.Iqbal




--------------------------------------------------------------------------------
                             Files Used for Analysis
--------------------------------------------------------------------------------

Path to file locations:
\Users\Muhammad.Iqbal\Documents\ATB Old Main Street - 09172025 ODOT
Comments\L-Pile\Rear (West) Abutment Drilled Shaft\

Name of input data file:
36-inch Dia.lp12d

Name of output report file:
36-inch Dia.lp12o

Name of plot output file:
36-inch Dia.lp12p

Name of runtime message file:
36-inch Dia.lp12r


--------------------------------------------------------------------------------
                            Date and Time of Analysis
--------------------------------------------------------------------------------
```

Date: October 22, 2025   Time: 21:14:37

Problem Title

Project Name: ATB Old Main Street Bridtge

Job Number: 232245

Client: City of Conneaut

Engineer: msi

Description: Lateral Shaft Resistance - Rear Abutment

Program Options and Settings

Computational Options:
 - Conventional Analysis
Engineering Units Used for Data Input and Computations:
 - US Customary System Units (pounds, feet, inches)

Analysis Control Options:
 - Maximum number of iterations allowed   =   500
 - Deflection tolerance for convergence   =   1.0000E-05 in
 - Maximum allowable deflection   =   100.0000 in
 - Number of pile increments   =   100

Loading Type and Number of Cycles of Loading:
 - Static loading specified

- Analysis uses p-y modification factors for p-y curves
- Analysis uses layering correction (Method of Georgiadis)
- Analysis includes loading by multiple distributed lateral loads acting on pile
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- Input of moment resistance at the pile tip not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:
- Output files use decimal points to denote decimal symbols.
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1
- No p-y curves to be computed and reported for user-specified depths
- Print using wide report formats

Pile Structural Properties and Geometry

```
Number of pile sections defined                        =            1
Total length of pile                                   =       11.800 ft
Depth of ground surface below top of pile              =       0.0000 ft
```

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	36.0000
2	11.800	36.0000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

```
   Section 1 is an elastic pile
   Cross-sectional Shape                               =   Circular Pile
   Length of section                                   =    11.800000 ft
```

```
   Width of top of section                               =    36.000000 in
   Width of bottom of section                            =    36.000000 in
   Top Area                                              =        1018. sq. in
   Bottom Area                                           =        1018. sq. in
   Moment of Inertia at Top                              =       82448. in^4
   Moment of Inertia at Bottom                           =       82448. in^4
   Elastic Modulus                                       =     3604997. psi



--------------------------------------------------------------------------------
                       Soil and Rock Layering Information
--------------------------------------------------------------------------------

The soil profile is modelled using 3 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer             =       0.0000 ft
   Distance from top of pile to bottom of layer          =     5.000000 ft
   Effective unit weight at top of layer                 =   122.000000 pcf
   Effective unit weight at bottom of layer              =   122.000000 pcf
   Friction angle at top of layer                        =    34.000000 deg.
   Friction angle at bottom of layer                     =    34.000000 deg.
   Subgrade k at top of layer                            =       0.0000 pci
   Subgrade k at bottom of layer                         =       0.0000 pci

   NOTE: Default values for subgrade k will be computed for this layer.

Layer 2 is weak rock, p-y criteria by Reese, 1997

   Distance from top of pile to top of layer             =     5.000000 ft
   Distance from top of pile to bottom of layer          =     6.800000 ft
   Effective unit weight at top of layer                 =   150.000000 pcf
   Effective unit weight at bottom of layer              =   150.000000 pcf
   Uniaxial compressive strength at top of layer         =   103.000000 psi
   Uniaxial compressive strength at bottom of layer      =   103.000000 psi
   Initial modulus of rock at top of layer               =       18000. psi
   Initial modulus of rock at bottom of layer            =       18000. psi
   RQD of rock at top of layer                           =       0.0000 %
   RQD of rock at bottom of layer                        =       0.0000 %
   k rm of rock at top of layer                          =    0.0000290
   k rm of rock at bottom of layer                       =    0.0000290


Layer 3 is massive rock, p-y criteria by Liang et al., 2009

   Distance from top of pile to top of layer             =     6.800000 ft
   Distance from top of pile to bottom of layer          =    11.800000 ft
   Effective unit weight at top of layer                 =   155.000000 pcf
```

```
   Effective unit weight at bottom of layer            =   155.000000 pcf
   Uniaxial compressive strength at top of layer       =        2730. psi
   Uniaxial compressive strength at bottom of layer    =        3350. psi
   Poisson's ratio at top of layer                     =     0.180000
   Poisson's ratio at bottom of layer                  =     0.180000
   Option 1: Intact rock modulus at top of layer       =       0.0000 psi
             Intact rock modulus at bottom of layer    =       0.0000 psi
   Option 1: Geologic Strength Index for layer         =    30.000000
   Option 2: Rock mass modulus at top of layer         =     1.380000 psi
             Rock mass modulus at bottom of layer      =     1.450000 psi
   Option 2 will use the input value of rock mass modulus to compute the p-y curve
            in massive rock.
   The rock type is (sedimentary) shales, Hoek-Brown Material Constant mi = 6


 (Depth of the lowest soil layer extends 0.000 ft below the pile tip)


**** Warning - Possible Input Data Error ****

Values entered for effective unit weight of rock were outside the limits of
50 pcf to 150 pcf.

The maximum input value, in layer   1, for effective unit weight = 155.00 pcf

This data may be erroneous. Please check your data.



--------------------------------------------------------------------------------
                        Summary of Input Soil Properties
--------------------------------------------------------------------------------

Layer         Soil Type          Layer      Effective    Angle of     Uniaxial
              E50                    Rock Mass    Geologic     Int. Rock
Hoek-Brown
 Num.           Name             Depth       Unit Wt.    Friction        qu
RQD %         or          kpy         Modulus     Strength      Modulus     Material
    Poisson's
      (p-y Curve Type)        ft           pcf          deg.         psi
              krm         pci          psi         Index         psi       Index,
mi      Ratio
-----   -------------------   ----------   ----------   ----------   ----------
----------   ----------   ----------   ----------   ----------   ----------
----------   ----------
  1           Sand                  0.00     122.0000      34.0000       --
 --           --         default        --           --            0.00
0.00         0.00
      (Reese, et al.)         5.0000     122.0000      34.0000       --
 --           --         default        --           --            0.00
```

```
0.00          0.00
  2            Weak               5.0000     150.0000       --          103.0000
   0.00     2.90E-05       --            18000.       --             0.00
0.00          0.00
               Rock               6.8000     150.0000       --          103.0000
   0.00     2.90E-05       --            18000.       --             0.00
0.00          0.00
  3           Massive             6.8000     155.0000       --             2730.
 --           --           --            1.3800      30.0000         0.00
6.0000       0.1800
               Rock              11.8000     155.0000       --             3350.
 --           --           --                        30.0000         0.00
6.0000       0.1800
```

Modification Factors for p-y Curves

Distribution of p-y modifiers with depth defined using 2 points

Point No.	Depth X ft	p-mult	y-mult
1	0.000	0.8000	1.0000
2	8.000	0.8000	1.0000

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

```
Load     Load         Condition                  Condition            Axial Thrust
Compute Top y     Run Analysis
 No.     Type            1                           2                 Force, lbs
vs. Pile Length
```

1	2	V = 63620. lbs	S = 0.0000 in/in	307800.	No	Yes

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

---

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Moment-curvature properties were derived from elastic section properties

---

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	56362.
2	5.0000	5.0000	No	Yes	N.A.	N.A.
3	6.8000	6.8000	No	Yes	N.A.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Shear and Pile-head Rotation (Loading Type 2)

Shear force at pile head = 63620.0 lbs
Rotation of pile head = 0.000E+00 radians
Axial load at pile head = 307800.0 lbs

(Zero slope for this load indicates fixed-head conditions)

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	0.02969	-4369015.	63620.	0.00	1256.	2.97E+11	0.00	0.00	0.00
0.1180	0.02968	-4278924.	63617.	-2.06E-05	1237.	2.97E+11	-4.098	195.5222	0.00
0.2360	0.02964	-4188833.	63608.	-4.08E-05	1217.	2.97E+11	-8.184	391.0443	0.00
0.3540	0.02956	-4098750.	63594.	-6.05E-05	1197.	2.97E+11	-12.246	586.5665	0.00
0.4720	0.02946	-4008682.	63574.	-7.98E-05	1178.	2.97E+11	-16.274	782.0886	0.00
0.5900	0.02934	-3918639.	63548.	-9.87E-05	1158.	2.97E+11	-20.255	977.6108	0.00
0.7080	0.02918	-3828629.	63516.	-1.17E-04	1138.	2.97E+11	-24.179	1173.	0.00
0.8260	0.02901	-3738659.	63479.	-1.35E-04	1119.	2.97E+11	-28.036	1369.	0.00
0.9440	0.02880	-3648737.	63437.	-1.53E-04	1099.	2.97E+11	-31.816	1564.	0.00
1.0620	0.02857	-3558872.	63389.	-1.70E-04	1079.	2.97E+11	-35.508	1760.	0.00
1.1800	0.02832	-3469070.	63337.	-1.87E-04	1060.	2.97E+11	-39.105	1955.	0.00
1.2980	0.02804	-3379340.	63279.	-2.03E-04	1040.	2.97E+11	-42.596	2151.	0.00
1.4160	0.02775	-3289688.	63216.	-2.19E-04	1021.	2.97E+11	-45.973	2346.	0.00

1.5340	0.02742	-3200121.	63149.	-2.34E-04	1001.	2.97E+11
-49.228	2542.	0.00				
1.6520	0.02708	-3110647.	63077.	-2.49E-04	981.5093	2.97E+11
-52.352	2737.	0.00				
1.7700	0.02672	-3021271.	63000.	-2.64E-04	961.9968	2.97E+11
-55.339	2933.	0.00				
1.8880	0.02633	-2931999.	62920.	-2.78E-04	942.5071	2.97E+11
-58.180	3128.	0.00				
2.0060	0.02593	-2842838.	62836.	-2.92E-04	923.0415	2.97E+11
-60.868	3324.	0.00				
2.1240	0.02551	-2753794.	62748.	-3.05E-04	903.6014	2.97E+11
-63.397	3519.	0.00				
2.2420	0.02507	-2664870.	62656.	-3.18E-04	884.1877	2.97E+11
-65.761	3715.	0.00				
2.3600	0.02461	-2576073.	62562.	-3.31E-04	864.8016	2.97E+11
-67.953	3910.	0.00				
2.4780	0.02413	-2487407.	62464.	-3.43E-04	845.4440	2.97E+11
-69.968	4106.	0.00				
2.5960	0.02364	-2398876.	62364.	-3.54E-04	826.1160	2.97E+11
-71.800	4301.	0.00				
2.7140	0.02313	-2310484.	62261.	-3.66E-04	806.8183	2.97E+11
-73.444	4497.	0.00				
2.8320	0.02260	-2222235.	62156.	-3.76E-04	787.5517	2.97E+11
-74.896	4693.	0.00				
2.9500	0.02206	-2134131.	62049.	-3.87E-04	768.3169	2.97E+11
-76.152	4888.	0.00				
3.0680	0.02151	-2046175.	61940.	-3.97E-04	749.1145	2.97E+11
-77.206	5084.	0.00				
3.1860	0.02094	-1958370.	61830.	-4.06E-04	729.9449	2.97E+11
-78.055	5279.	0.00				
3.3040	0.02035	-1870718.	61719.	-4.15E-04	710.8086	2.97E+11
-78.697	5475.	0.00				
3.4220	0.01976	-1783219.	61608.	-4.24E-04	691.7059	2.97E+11
-79.127	5670.	0.00				
3.5400	0.01915	-1695875.	61495.	-4.32E-04	672.6370	2.97E+11
-79.343	5866.	0.00				
3.6580	0.01854	-1608687.	61383.	-4.40E-04	653.6021	2.97E+11
-79.343	6061.	0.00				
3.7760	0.01791	-1521654.	61271.	-4.48E-04	634.6012	2.97E+11
-79.124	6257.	0.00				
3.8940	0.01727	-1434777.	61159.	-4.55E-04	615.6343	2.97E+11
-78.684	6452.	0.00				
4.0120	0.01662	-1348055.	61048.	-4.61E-04	596.7011	2.97E+11
-78.023	6648.	0.00				
4.1300	0.01596	-1261486.	60938.	-4.68E-04	577.8015	2.97E+11
-77.139	6843.	0.00				
4.2480	0.01530	-1175070.	60830.	-4.73E-04	558.9351	2.97E+11
-76.031	7039.	0.00				
4.3660	0.01462	-1088803.	60723.	-4.79E-04	540.1014	2.97E+11
-74.698	7234.	0.00				

4.4840	0.01394	-1002684.	60619.	-4.84E-04	521.2999	2.97E+11	-73.140	7430.	0.00
4.6020	0.01325	-916710.	60516.	-4.88E-04	502.5300	2.97E+11	-71.358	7625.	0.00
4.7200	0.01256	-830876.	60417.	-4.92E-04	483.7910	2.97E+11	-69.352	7821.	0.00
4.8380	0.01186	-745180.	60320.	-4.96E-04	465.0818	2.97E+11	-67.122	8016.	0.00
4.9560	0.01115	-659618.	60227.	-5.00E-04	446.4018	2.97E+11	-64.669	8212.	0.00
5.0740	0.01044	-574183.	58248.	-5.03E-04	427.7497	2.97E+11	-2730.	370161.	0.00
5.1920	0.00973	-494220.	54316.	-5.05E-04	410.2923	2.97E+11	-2824.	411127.	0.00
5.3100	0.00901	-419919.	50256.	-5.07E-04	394.0709	2.97E+11	-2911.	457434.	0.00
5.4280	0.00829	-351454.	46079.	-5.09E-04	379.1237	2.97E+11	-2988.	510336.	0.00
5.5460	0.00757	-288980.	41800.	-5.11E-04	365.4844	2.97E+11	-3055.	571525.	0.00
5.6640	0.00685	-232631.	37435.	-5.12E-04	353.1823	2.97E+11	-3110.	643354.	0.00
5.7820	0.00612	-182517.	33002.	-5.13E-04	342.2415	2.97E+11	-3152.	729201.	0.00
5.9000	0.00539	-138723.	28521.	-5.14E-04	332.6802	2.97E+11	-3177.	834118.	0.00
6.0180	0.00467	-101297.	24019.	-5.14E-04	324.5095	2.97E+11	-3183.	966043.	0.00
6.1360	0.00394	-70253.	19525.	-5.15E-04	317.7321	2.97E+11	-3165.	1138287.	0.00
6.2540	0.00321	-45555.	15078.	-5.15E-04	312.3399	2.97E+11	-3115.	1375174.	0.00
6.3720	0.00248	-27103.	10732.	-5.15E-04	308.3114	2.97E+11	-3023.	1726952.	0.00
6.4900	0.00175	-14712.	6563.	-5.15E-04	305.6063	2.97E+11	-2865.	2319055.	0.00
6.6080	0.00102	-8066.	2752.	-5.15E-04	304.1554	2.97E+11	-2518.	3496274.	0.00
6.7260	2.90E-04	-6470.	445.7268	-5.15E-04	303.8069	2.97E+11	-738.960	3603210.	0.00
6.8440	-4.39E-04	-6355.	-77.436	-5.15E-04	303.7818	2.97E+11	0.02865	92.3876	0.00
6.9620	-0.00117	-6240.	-77.362	-5.15E-04	303.7567	2.97E+11	0.07632	92.4657	0.00
7.0800	-0.00190	-6125.	-77.220	-5.15E-04	303.7316	2.97E+11	0.1241	92.5440	0.00
7.1980	-0.00263	-6009.	-77.011	-5.15E-04	303.7064	2.97E+11	0.1719	92.6225	0.00
7.3160	-0.00336	-5894.	-76.733	-5.15E-04	303.6811	2.97E+11	0.2198	92.7010	0.00

7.4340	-0.00409	-5777.	-76.388	-5.15E-04	303.6557	2.97E+11	0.2678	92.7797	0.00
7.5520	-0.00482	-5661.	-75.975	-5.15E-04	303.6302	2.97E+11	0.3159	92.8583	0.00
7.6700	-0.00555	-5543.	-75.493	-5.15E-04	303.6046	2.97E+11	0.3641	92.9370	0.00
7.7880	-0.00628	-5425.	-74.944	-5.15E-04	303.5788	2.97E+11	0.4123	93.0157	0.00
7.9060	-0.00701	-5306.	-74.325	-5.16E-04	303.5529	2.97E+11	0.4607	93.0944	0.00
8.0240	-0.00774	-5186.	-73.549	-5.16E-04	303.5267	2.97E+11	0.6364	116.4661	0.00
8.1420	-0.00847	-5065.	-72.605	-5.16E-04	303.5002	2.97E+11	0.6970	116.5641	0.00
8.2600	-0.00920	-4943.	-71.575	-5.16E-04	303.4735	2.97E+11	0.7577	116.6620	0.00
8.3780	-0.00993	-4818.	-70.459	-5.16E-04	303.4464	2.97E+11	0.8186	116.7599	0.00
8.4960	-0.01066	-4693.	-69.256	-5.16E-04	303.4189	2.97E+11	0.8795	116.8578	0.00
8.6140	-0.01139	-4565.	-67.968	-5.16E-04	303.3911	2.97E+11	0.9406	116.9556	0.00
8.7320	-0.01212	-4436.	-66.593	-5.16E-04	303.3628	2.97E+11	1.0017	117.0534	0.00
8.8500	-0.01285	-4304.	-65.131	-5.16E-04	303.3341	2.97E+11	1.0630	117.1512	0.00
8.9680	-0.01358	-4171.	-63.582	-5.16E-04	303.3049	2.97E+11	1.1243	117.2489	0.00
9.0860	-0.01431	-4035.	-61.947	-5.16E-04	303.2753	2.97E+11	1.1858	117.3467	0.00
9.2040	-0.01504	-3897.	-60.224	-5.16E-04	303.2451	2.97E+11	1.2473	117.4443	0.00
9.3220	-0.01577	-3756.	-58.414	-5.16E-04	303.2143	2.97E+11	1.3090	117.5420	0.00
9.4400	-0.01650	-3612.	-56.517	-5.16E-04	303.1830	2.97E+11	1.3707	117.6396	0.00
9.5580	-0.01723	-3466.	-54.532	-5.16E-04	303.1511	2.97E+11	1.4326	117.7372	0.00
9.6760	-0.01796	-3317.	-52.460	-5.16E-04	303.1186	2.97E+11	1.4946	117.8348	0.00
9.7940	-0.01869	-3165.	-50.300	-5.16E-04	303.0854	2.97E+11	1.5566	117.9324	0.00
9.9120	-0.01942	-3010.	-48.051	-5.16E-04	303.0515	2.97E+11	1.6188	118.0299	0.00
10.0300	-0.02015	-2852.	-45.715	-5.16E-04	303.0169	2.97E+11	1.6811	118.1274	0.00
10.1480	-0.02088	-2690.	-43.291	-5.16E-04	302.9816	2.97E+11	1.7435	118.2248	0.00
10.2660	-0.02161	-2524.	-40.778	-5.16E-04	302.9455	2.97E+11	1.8059	118.3223	0.00

```
   10.3840   -0.02234     -2356.    -38.176  -5.16E-04   302.9087   2.97E+11
1.8685   118.4197       0.00
   10.5020   -0.02307     -2183.    -35.486  -5.16E-04   302.8710   2.97E+11
1.9312   118.5170       0.00
   10.6200   -0.02380     -2006.    -32.707  -5.16E-04   302.8324   2.97E+11
1.9940   118.6144       0.00
   10.7380   -0.02453     -1826.    -29.839  -5.16E-04   302.7930   2.97E+11
2.0568   118.7117       0.00
   10.8560   -0.02526     -1641.    -26.882  -5.16E-04   302.7527   2.97E+11
2.1198   118.8090       0.00
   10.9740   -0.02600     -1452.    -23.835  -5.16E-04   302.7114   2.97E+11
2.1829   118.9062       0.00
   11.0920   -0.02673     -1259.    -20.700  -5.16E-04   302.6692   2.97E+11
2.2461   119.0034       0.00
   11.2100   -0.02746     -1061.    -17.474  -5.16E-04   302.6260   2.97E+11
2.3094   119.1006       0.00
   11.3280   -0.02819   -858.649    -14.160  -5.16E-04   302.5819   2.97E+11
2.3728   119.1978       0.00
   11.4460   -0.02892   -651.443    -10.755  -5.16E-04   302.5366   2.97E+11
2.4363   119.2949       0.00
   11.5640   -0.02965   -439.351     -7.260  -5.16E-04   302.4903   2.97E+11
2.4998   119.3920       0.00
   11.6820   -0.03038   -222.246     -3.675  -5.16E-04   302.4429   2.97E+11
2.5635   119.4891       0.00
   11.8000   -0.03111       0.00       0.00  -5.16E-04   302.3944   2.97E+11
2.6273    59.7931       0.00

* The above values of total stress are combined axial and bending stresses.


Output Summary for Load Case No. 1:

Pile-head deflection              =     0.02969370 inches
Computed slope at pile head       =       0.000000 radians
Maximum bending moment            =      -4369015. inch-lbs
Maximum shear force               =         63620. lbs
Depth of maximum bending moment   =       0.000000 feet below pile head
Depth of maximum shear force      =       0.000000 feet below pile head
Number of iterations              =             11
Number of zero deflection points  =              1


--------------------------------------------------------------------------------
            Summary of Pile-head Responses for Conventional Analyses
--------------------------------------------------------------------------------

Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
```

```
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians


Load Load                Load                  Axial    Pile-head  Pile-head  Max
Shear Max Moment
Case Type   Pile-head    Type     Pile-head   Loading  Deflection  Rotation    in
Pile    in Pile
 No.  1      Load 1       2        Load 2       lbs      inches     radians      lbs
     in-lbs
---- ----- ---------- ---------- ---------- ---------- ---------- ----------
---------- ----------
  1  V, lb     63620.   S, rad         0.00    307800.    0.02969      0.00
63620.  -4369015.


Maximum pile-head deflection = 0.0296937010 inches
Maximum pile-head rotation   = 0.0000000000 radians = 0.000000 deg.

--------------------------------------------------------------------------------
                           Summary of Warning Messages
--------------------------------------------------------------------------------

The following warning was reported 180 times

***** Warning *****

The input value for k_rm used by the weak rock criteria is smaller than
0.00005. This value is outside the recommended range of 0.00005 to 0.0005.
Please check your input data for accuracy.

The following warning was reported 516 times

WARNING:  The ratio of rock mass modulus to uniaxial compressive strength
          for massive rock appears to be outside the usual range of values.


The analysis ended normally.
```

Project Name: ATB Old Main Street Bridge Replacement
Project Number: 232245
Calculated by: Bikal Sah
Reviewed By: Imad El. Hajjar / MS Iqbal 10/2/2025

Scour Determination

Upper Elevation Limit for Analysis = 588.09 feet, based on 100-year floodplain
Lower Elevation Limit for Analysis = 563.56 feet, based on 6 feet below bottom of river

Table 2. Scour Parameters for Cored Rock

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)
Forward Abutment												
B-001-0-22	B-001-0-22 (NQ2-8)	17.1-17.2	575.6-575.5	2730	80.6	0	155	37	20 to 30	1.13	5.62	269.0
B-001-0-22	B-001-0-22 (NQ2-8)	18.3-18.4	574.4-574.3	2860	80.6	0	155	37	20 to 30	1.18	5.75	275.3
B-001-0-22	B-001-0-22 (NQ2-9)	21.3-21.4	571.4-571.3	3350	80.6	0	155	37	20 to 30	1.39	6.22	298.0
Pier												
B-002-0-22	B-002-0-22 (NQ2-2)	2.6-2.7	567.5-567.4	2280	83.5	0	155	37	25 to 45	0.47	3.63	173.8
B-002-0-22	B-002-0-22 (NQ2-2)	5.5-5.6	564.6-564.5	5630	83.5	0	155	39	25 to 45	1.16	5.70	273.1
B-002-0-22	B-002-0-22 (NQ2-3)	6.2-6.3	563.9-563.8	5170	83.5	0	155	39	25 to 45	1.07	5.47	261.7
Rear Abutment												
B-003-0-22	B-003-0-22 (NQ2-12)	25.4-25.5	566.3-566.2	7042	80.9	0	155	39	20 to 35	1.46	6.38	305.5
B-003-0-22	B-003-0-22 (NQ2-12)	28.1-28.2	563.6-563.5	2581	80.9	0	155	37	20 to 35	0.53	3.86	184.9
B-003-0-22	B-003-0-22 (NQ2-13)	30.3-30.4	561.4-561.3	4994	80.9	0	155	39	20 to 35	1.03	5.37	257.2
B-003-0-22	B-003-0-22 (NQ2-13)	32.1-32.2	559.6-559.5	7473	80.9	0	155	39	20 to 35	1.55	6.57	314.7

[1] Qu is average of two tested specimens for NQ2.

Date: September 17, 2025

Project Name:	Old Main Street Bridge
Project No.	232245
Calculated by	MSI
Checked by	IHJ, 10/23/2025
Method	LRFD Shallow Foundation on Rock
Structure	Central Pier
Boring ID	B-002-0-23
Severely weathered weak and highly fractured gray shale is exposed at elevation of 570 feet.	
STRENGTH LIMIT STATE DESIGN	
As per Geotechnical Design Manual (GDM) Section 1303.3.3 • Bedrock slope of 2H:1V or less • Rock Mass Rating (RMR) ≤ 70 • q_u ≤7,500 psi then calculate drained shear strength properties (c' and ϕ') in accordance with Bieniawski (1989). The Bieniawski (1989) drained shear strength equations are as follows: c' = 0.104 x RMR (ksf) ϕ' = RMR/2 + 5° (deg)	
RMR = 39 q_u = 5,630 psi Unit Weight, γ = 155 pcf	RMR [4+3+20+12+0 = 39] is computed based on the recovered rock cores at Elevation 565 in boring B-002-0-23. Point load test at Elevation 565 feet resulted a UCS of 5,630 psi.
Drained Cohesion c' = 0.104 x 39 = 4.056 ksf = **4,056** psf	
Drained Friction Angle ϕ' = 39/2 + 5 = 24.5° ≈ **25**°	
Footing Dimensions Width, B = 9 feet Length, L = 35 feet Footing Depth, D_f = 3/12 feet	
Based on AASHTO 10.6.3.1.2a Nominal bearing resistance of spread footing on cohesionless soils	

$q_n = cN_{cm} + \gamma_q D_f N_{qm} C_{wq} + 0.5\gamma_f B N_{\gamma m} C_{w\gamma}$ (10.6.3.1.2a-1) in which: $N_{cm} = N_c s_c i_c$ (10.6.3.1.2a-2) $N_{qm} = N_q s_q d_q i_q$ (10.6.3.1.2a-3) $N_\gamma m = N_\gamma s_\gamma i_\gamma$ (10.6.3.1.2a-4)	
N_c = 20.7	AASHTO LRFD Table 10.6.3.1.2a-1
N_q = 10.7	AASHTO LRFD Table 10.6.3.1.2a-1
N_γ = 10.9	AASHTO LRFD Table 10.6.3.1.2a-1
i_c = 1	No inclination
i_q = 1	No inclination
i_γ = 1	No inclination
s_c = 1.1329	1+ (B/L)(N_q/N_c) AASHTO Table 10.6.3.1.2a-3
s_γ = 0.8971	1 - 0.4 x (B/L) AASHTO Table 10.6.3.1.2a-3
s_q = 1.1199	1+((B/L)tan ϕ) AASHTO Table 10.6.3.1.2a-3
d_q = 1	$d_q = 1 + 2\tan\phi_f(1-\sin\phi_f)^2 \arctan\left(\frac{D_f}{B}\right)$ (10.6.3.1.2a-10)
C_{wq} = 0.5	D_w = 0 AASHTO Table 10.6.3.1.2a-2
$C_{w\gamma}$ = 0.5	D_w = 0 AASHTO Table 10.6.3.1.2a-2
N_{cm} =20.7 x 1.1329 x1 = 23.4510	AASHTO LRFD 10.6.3.1.2a-2
N_{qm} = 10.7 x 1.1199 x 1 x 1 = 11.877	AASHTO LRFD 10.6.3.1.2a-3
$N_{\gamma m}$ = 10.9 x 0.8971 x 1 = 9.7784	AASHTO LRFD 10.6.3.1.2a-4
Nomila Bearing Resistance, q_n	AASHTO LRFD 10.6.3.1.2a-1
q_n = (4056 x 23.4510) + (155 x 0.25 x 11.877 x 0.5) + (0.5 x 155 x 9 x 9.7784 x 0.5) q_n = **98,827.15** psf = 99 ksf	
For Piers, footing on rock Reduction Factor, $ø_b$ = 0.45	AASHTO LRFD Table 10.5.5.2.2-1

Bearing Capacity Factored, q_r q_r = 98827.15 x 0.45 = 44,472.21 psf ≈ **45** ksf
SERVICE LIMIT STATE DESIGN
The calculated unfactored bearing pressure of 0.69 ksi is significantly lower than the estimated rock mass modulus of 605 ksi, which satisfies the criterion outlined in GDM Section 1303.2.1—that bearing stress should be less than 50 times the rock mass modulus to assume negligible settlement. Therefore, no adjustments to the bearing pressure are required at this structure location. Supporting calculations for the rock mass modulus are provided in the attached documentation.

By:	msi	Date:	10/22/2025	Checked:	IJH	Date:	10/23/2025

GENERAL FOUNDATION INFORMATION:

Forward Abutment Wall:

Bottom Elevation	572.8	ft	Approximate bearing elevation	
B	9	ft	Width	B is the governing direction for this structure
L	48	ft	Length	
$\gamma_{concrete}$	0.15	Kcf	Unit weight	

ECCENTRICITY, e, in Governing Direction GDM 1303.1.2

$e_B = \Sigma M / \Sigma V$

$e_B =$ 1.87 NOTE: At strength limit state. Provided by Structural Engineer.

LIMITING ECCENTRICITY, e_{Limit}

In Governing Direction AASHTO LRFD 10.6.3.3

$e_{Limit} = 0.45 * B$

$e_{Limit} =$ 4.05

EFFECTIVE FOOTING DIMENSIONS AASHTO LRFD 10.6.1.3

$B' = B - 2e_B$

B' = 5.26 ft.
B' = 5.00 ft. approx.

$L' = L - 2e_L$ NOTE: Not applicable

By:	msi	Date:	10/14/2025	Checked:	IJH	Date:	10/24/2025

GENERAL FOUNDATION INFORMATION:

Forward Abutment: Width W = 5.3' , Length L = 48' Width = 5.3' is effective width
Bearing at approxiamtely 572.8 feet.

GENERAL SOIL INFORMATION:

Anticipated Bearing Conditions:

Predominantly severely weathered rock.
The weathered rock is assumed to behave like cohesionless granular material.
The estimated UCS is 19.5 psi based on SPT data.

Based on Soil Strength Evaluation Spreadsheet,

USE c' =	0	ksf - cohesionless soil
USE ϕ'=	30	Degrees

Groundwater

Model groundwater in creek above foundation bearing elevation.

STRENGTH LIMIT STATE:

$q_R = \varphi_b * q_n$ (AASTHO LRFD 10.6.3.1.1-1)

q_R = factored resistance at strength limit state (ksf)
φ_b = resistance factor (Article 10.5.5.2.2)
q_n = nominal bearing resistance (ksf)

$q_n = cN_{cm} + \gamma_q D_f N_{qm} C_{wq} + 0.5\gamma_f B N_{gm} C_{wg}$ (AASTHO LRFD 10.6.3.1.2a-1)
$N_{cm} = N_c s_c i_c$ (AASTHO LRFD 10.6.3.1.2a-2)
$N_{qm} = N_q s_q d_q i_q$ (AASTHO LRFD 10.6.3.1.2a-3)
$N_{gm} = N_g s_g i_g$ (AASTHO LRFD 10.6.3.1.2a-4)

c = cohesion, undrained shear strength (ksf)
N_c = cohesion term (Table 10.6.3.1.2a-1)
N_q = surcharge term (Table 10.6.3.1.2a-1)
N_g = unit weight term (Table 10.6.3.1.2a-1)
g = total (moist) unit weight (kcf)
D_f = footing embedment depth (ft)
B = footing width (ft)
C_{wq} , C_{wg} = groundwater correction factors (Table 10.6.3.1.2a-2)
s_c , s_g , s_q = shape correction factors (Table 10.6.3.1.2a-3)
d_q = shear resistance thought cohesionless material correction factor (Table 10.6.3.1.2a-4)
i_c , i_g , i_q = inclination correction factors

By: msi Date: 10/14/2025 Checked: IJH Date: 10/24/2025

Setup	$c =$	0	ksf	assumed zero in cohesionless soil
	$\phi_f =$	30	degrees	
	$N_c =$	30.1	*unitless*	
	$N_q =$	18.4	*unitless*	for soil with a $\phi_f = 30$ Degrees
	$N_\gamma =$	22.4	*unitless*	
	$\gamma =$	0.125	kcf	(assumed in upper 1.5 feet above bearing)
	$D_f =$	0.3	ft	Minimum embedment of the footing into rock
	$B =$	5.3	ft	Width Stage 2 plans
	$L =$	48	ft	Length Stage 2 plans
	$D_w =$	0	ft	highest anticipated groundwater depth
	$C_{wq} =$	0.5	*unitless*	where $D_w = 0.0$ $1.5B + D_f =$ 8.2
	$C_{w\gamma} =$	0.5	*unitless*	(above D_f)
	$s_c =$	1.07	*unitless*	$s_c = 1 + (B/L)*(Nq/Nc)$ $s_c = 1 + (B/(5L))$
	$s_\gamma =$	0.96	*unitless*	for $\phi_f > 0$ $s_g = 1 - 0.4 * (B/L)$ for $\phi_f = 0$ $s_g = 1$
	$s_q =$	1.05	*unitless*	$s_q = 1+((B/L)*\tan\phi_f)$ $s_q = 1$
	$d_q =$	1.0	*unitless*	taken as 1 since cohesionless soil on top of weathered rock
	$i_c , i_\gamma , i_q =$	1.0	*unitless*	Assumed loaded without inclination

calculation	$N_{cm} = N_c s_c i_c$	= 30.1 * 1.067 * 1 =	32.117
	$N_{qm} = N_q s_q d_q i_q$	= 18.4 * 0.956 * 1 * 1	17.59
	$N_{gm} = N_g s_g i_g$	= 22.4 * 1.05 * 1 =	23.52

$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma B N_{\gamma m} C_{w\gamma}$ $cN_{cm} =$ 0

= (0*32.117) + (0.125*0.25*17.59*0.5) + (0.5*5.3*23.52*0.5) = $\gamma D_f N_{qm} C_{wq} =$ 0.275

= (0) + (0.275) + (31.164) = $0.5\gamma B N_{\gamma m} C_{w\gamma} =$ 31.164

$q_n =$ 31.44 ksf

$\varphi_b =$ 0.55 AASHTO LRFD Table 11.5.7-1 - SLS Res. Factor for Perm. Retain. Walls

$q_R = \varphi_b * q_n$ = 0.55 * 31.439 = 17.29 ksf

Factored resistance at the strength limit state for the ABUTMENT footing bearing in the WEATHERED BEDROCK is equal to 17.3 ksf

By: msi Date: 10/14/2025 Checked: IJH Date: 10/24/2025

SERVICE LIMIT STATE:

Based on a factored bearing pressure of 17.3 ksf.
Referring to GDM Section 1303.2.1, the settlement of foundations bearing on bedrock may be assumed to be negligible if the maximum Service Limit State bearing stress is less than fifty (50) times the rock mass modulus, Em.

The unfactored bearing pressure is 0.12 ksi which is significantly lower than estimated 50 x E_m of weathered rock which is 55 ksi. Therefore, no adjustments to the bearing pressure are required at this structure location.
Supporting calculations for the rock mass modulus are provided in the attached documentation.

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By: MSI Date: 10/20/2025 Checked: IHJ Date: 10/23/2025

GENERAL FOUNDATION INFORMATION:

Forward Abutment: Width W = 5.3' , Length L = 48' w = 5.3 is effective width
Bearing at approximately 572.8 feet.

GENERAL SOIL INFORMATION:

Anticipated Bearing Conditions:

Predominantly severely weathered rock.
The weathered rock is assumed to behave like cohesion less granular material.
The estimated UCS is 19.5 psi based on SPT data.

USE c' =	0	ksf	cohesionless soil
USE ϕ'=	30	Degrees	backfill material

Groundwater:

Model groundwater in creek above foundation bearing elevation.

FAILURE BY OVERTURNING:

Assumptions:

The rear abutment is assumed as semi-gravity cantilever wall

SETUP:

W	5.3	feet	*Width of fwd.abutment*
L	48	feet	*Length of fwd. abutment*
z	17.3	feet	*height of fwd. abutment*
b	25	feet	*length of the backfill material (assuming same as length of approach slab)*
$\gamma_{concrete}$	0.15	ksf	*Unit weight of the concrete*
P	12.16	ksf	*Maximum strength load for Fwd. Abutment*
V	5253	kips	*Estimated based on P (ksi) for Fwd. Abutment Wall with total area of 48' x 9'*
ϕ_f	30	degrees	*internal angle of friction of the soil (backfill)*
c	0	ksf	*cohesion of the soil (backfill)*
γ	0.125	kcf	*Unit weight of the soil (backfill)*
K_a	0.33	unitless	*Coefficient of active earth pressure (backfill)* **$K_a = \tan^2 (45^0 - \phi_f /2)$**

RESISTING MOMENTS:

A_{wall}	254.4	Area (W x L) of Wall (ft^2)	
$A_{wall\ base}$	75	Area (b x 3) of Wall Base (ft^2)	Based on the Fwd. Abutment wall sections
$A_{backfill}$	1200	Area (L x b) of Backfill (ft^2)	
P_{wall}	38.16	Weight / unit length of Wall (kip/ft)	
$P_{wall\ base}$	11.25	Weight / unit length of Wall Base (kip/ft)	
$P_{backfill}$	150	Weight / unit length of Backfill (kip/ft)	

X_{wall}	2.65	Moment Arm Wall (ft)	*Assumed half of the abutment width*
X_{wall} base	12.5	Moment Arm Wall Base (ft)	*Assumed half of the approach slab length*
$X_{backfill}$	12.5	Moment Arm Backfill (ft)	*Assumed half of the approach slab length*

Resisting Moment, M_r

M_{wall}	101.124
$M_{wall\ base}$	140.63
$M_{backfill}$	1875

M_{RT} 2,116.75 Total Resisting Moment (kip)

Overturning Moment, M_O

$P_a = 0.5 * K_a * \gamma * z^2$

$P_a =$ 6.24 kips/ft

$X_{backfill}$ 5.77 Moment Arm, typically taken at one-thrid of height of wall (ft)

$M_o = P_a * X_{backfill}$

$M_o =$ 35.96 kip

FACTOR OF SAFETY (FS):

$FS = M_r / M_O$

FS = 59

By: MSI Date: 10/20/2025 Checked: IHJ Date: 10/23/2025

GENERAL FOUNDATION INFORMATION:

Forward Abutment: Width W = 5.3' , Length L = 48' Width = 5.3' is effective width
Bearing at approximately 572.8 feet.

GENERAL SOIL INFORMATION:

Anticipated Bearing Conditions:

Predominantly severely weathered rock.
The weathered rock is assumed to behave like cohesion less granular material.
The estimated UCS is 19.5 psi based on SPT data.

USE c' =	0	ksf - cohesionless soil
USE ϕ'=	30	Degrees

Groundwater:

Model groundwater in creek above foundation bearing elevation.

FAILURE BY SLIDING:

AASHTO LRFD 10.6.3.4

$R_R = \phi R_n = \phi_\tau R_\tau + \phi_{ep} R_{ep}$ (kips) AASHTO LRFD 10.6.3.4-1

ϕ resistance factor (dim)
R_n nominal sliding resistance against failure by sliding (kips)
ϕ_τ resistance factor for shear resistance between soil and foundation specified in Table 10.5.5.2.2-1
R_τ nominal sliding resistance between soil and foundation (kips)
ϕ_{ep} resistance factor for passive resistance specified in Table 10.5.5.2.2-1
R_{ep} nominal passive resistance of soil available throughout the design life of the structure (kips)

for footings on the cohesionless soils

$R_\tau = CV \tan \phi_f$ AASHTO LRFD 10.6.3.4-2

C 1.0 for concrete cast against soil
0.8 for precast concrete footing
V total vertical force (kips)
ϕ_f internal friction angle of drained soil (degrees)

Since the forward abutment wall (assumed semi-gravity cantilever wall) rests on the severely weathered rock, assumed as cohesionless granular material, AASHTO LRFD 10.6.3.4-2 section is applicable.

SETUP:

C	1	unitless					
P	12.16	ksf	*Maximum strength load for Fwd. Abutment*				
V	5253	kips	*Estimated based on P (ksi) for Fwd. Abutment Wall with total area of 48' x 9'*				
ϕ_f	30	degrees	*Estimated based on overburden pressure and SPT data*				
ϕ_τ	0.8	unitless	*concrete on sand*	AASHTO	LRFD	Table	10.5.5.2.2.1
ϕ_{ep}	0.5	unitless		AASHTO	LRFD	Table	10.5.5.2.2.1
ϕ	1	unitless		AASHTO	LRFD	Table	11.5.7-1
γ	0.125	kcf	*Unit weight of the backfill material*				
K_p	3.00	unitless	*Coefficient of passive earth pressure (backfill)*	**$K_p = \tan^2(45^0 + \phi_f/2)$**			
z	17.3	feet	*height of the fwd. abutment based on the stage 2 plans*				
c	0	ksf	*cohesion of the soil (backfill)*				

Nominal Sliding Resistance between Soil and Foundation (kips):

$R_\tau =$ 3,032.89

Nominal Passive Resistance of Soil (kips):

$R_{ep} = 0.5 * \gamma * z^2 * K_p$ AASHTO LRFD Figure 3.11.5.4-1

$R_{ep} =$ 56.12

Nominal Sliding Resistance against Failure by Sliding (kips):

$R_n = \phi_\tau R_\tau + \phi_{ep} R_{ep}$

$R_n =$ 2,426.31 + 28.06

$R_n =$ 2,454.37

Factored Resistance against Sliding (kips): AASHTO LRFD 10.6.3.4

$R_R = \phi R_n$

$\boxed{R_R = 2{,}454.37}$

By:	msi	Date:	10/22/2025	Checked:	IJH	Date:	10/23/2025

GENERAL FOUNDATION INFORMATION:

Wingwall at Rear Abutment:

Bottom Elevation	580	ft	Approximate bearing elevation	
B	7	ft	Width	B is the governing direction for this structure
L	12.5	ft	Length	
$\gamma_{concrete}$	0.15	Kcf	Unit weight	

ECCENTRICITY, e, in Governing Direction GDM 1303.1.2

$e_B = \Sigma M / \Sigma V$

$e_B =$ 0.00

NOTE: At strength limit state. Provided by Structural Engineer.

LIMITING ECCENTRICITY, e_{Limit}

In Governing Direction AASHTO LRFD 10.6.3.3

$e_{Limit} = 0.45 * B$

$e_{Limit} =$ 3.15

EFFECTIVE FOOTING DIMENSIONS AASHTO LRFD 10.6.1.3

$B' = B - 2e_B$

B' = 7.00 ft.

B' = 7.00 ft. approx.

$L' = L - 2e_L$

NOTE: Not applicable

By:	IJH	Date:	2/26/2025	Checked:	IJH	Date:	10/23/2025
				Revision:	msi	Date:	10/7/2025

GENERAL FOUNDATION INFORMATION:

Wingwall at Forward Abutment: Width W = 7' , Length L = 12.5'
Bearing at approxiamtely 580 feet.

GENERAL SOIL INFORMATION:

Anticipated Bearing Conditions:

Predominantly Very Stiff to Hard Cohesive Soils underlain by 1' zone very dense sand, and then weathered bedrock.

Based on Soil Strength Evaluation Spreadsheet,

USE c =	1.5	ksf for these soils

Groundwater

Model groundwater in creek above foundation bearing elevation.

STRENGTH LIMIT STATE:

$q_R = \varphi_b * q_n$ (AASTHO LRFD 10.6.3.1.1-1)

- q_R = factored resistance at strength limit state (ksf)
- φ_b = resistance factor (Article 10.5.5.2.2)
- q_n = nominal bearing resistance (ksf)

$q_n = cN_{cm} + gD_fN_{qm}C_{wq} + 0.5gBN_{gm}C_{wg}$ (AASTHO LRFD 10.6.3.1.2a-1)

$N_{cm} = N_cs_ci_c$ (AASTHO LRFD 10.6.3.1.2a-2)

$N_{qm} = N_qs_qd_qi_q$ (AASTHO LRFD 10.6.3.1.2a-3)

$N_{gm} = N_gs_gi_g$ (AASTHO LRFD 10.6.3.1.2a-4)

- c = cohesion, undrained shear strength (ksf)
- N_c = cohesion term (Table 10.6.3.1.2a-1)
- N_q = surcharge term (Table 10.6.3.1.2a-1)
- N_g = unit weight term (Table 10.6.3.1.2a-1)
- g = total (moist) unit weight (kcf)
- D_f = footing embedment depth (ft)
- B = footing width (ft)
- C_{wq} , C_{wg} = groundwater correction factors (Table 10.6.3.1.2a-2)
- s_c , s_g , s_q = shape correction factors (Table 10.6.3.1.2a-3)
- d_q = shear resistance thought cohesionless material correction factor (Table 10.6.3.1.2a-4)
- i_c , i_g , i_q = inclination correction factors

By: IJH Date: 2/26/2025 Checked: IJH Date: 10/7/2025

Setup	c =	1.5	ksf		
	f_f =	0	degrees	assumed zero in cohesive soil	
	N_c =	5.14	*unitless*		
	N_q =	1.0	*unitless*	for soil with a f_f = 0 Degrees	
	N_g =	0.0	*unitless*		
	g =	0.125	kcf	(assumed in upper 1.5 feet above bearing)	
	D_f =	6.5	ft	(Depth below creek bottom)	
	B =	7	ft	Width	
	L =	12.5	ft	Length (measured on Google Earth)	
	D_w =	0	ft	highest anticipated groundwater depth	
	C_{wq} =	0.5	*unitless*	where D_w = 0.0	1.5B + D_f = 17
	C_{wg} =	0.5	*unitless*	(above $D_{f)}$	
	s_c =	1.112	*unitless*	$s_c = 1 + (B/(5L))$	$s_c = 1 + (B/(5L))(Nq/Nc)$
	s_g =	1.0	*unitless*	for $\phi_f = 0$ $s_g = 1$	for $\phi_f > 0$ $s_g = 1 - 0.4(B/L)$
	s_q =	1.0	*unitless*	$s_q = 1$	$s_q = 1 + ((B/L)\tan(f_f))$
	d_q =	1.0	*unitless*	taken as 1 since cohesive soil	D_f / B = 0.928571
	i_c , i_g , i_q =	1.0	*unitless*	Assumed loaded without inclination	

calculation $N_{cm} = N_c s_c i_c$ = 5.14 * 1.112 * 1 = 5.716

$N_{qm} = N_q s_q d_q i_q$ = 1 * 1 * 1 * 1 = 1

$N_{gm} = N_g s_g i_g$ = 0 * 1 * 1 = 0

$q_n = cN_{cm} + gD_fN_{qm}C_{wq} + 0.5gBN_{gm}C_{wg}$ — cN_{cm} = 8.574

= (1.5*5.716) + (0.125*6.5*1*0.5) + (0.5*7*0*0.5) = — $gD_fN_{qm}C_{wq}$ = 0.406

= (8.574) + (0.406) + (0) = — $0.5gBN_{gm}C_{wg}$ = 0

q_n = 8.98 ksf

φ_b = 0.55 AASHTO LRFD Table 11.5.7-1 - Strength Limit State Res. Factor for Perm. Retaining Wall:

$q_R = \varphi_b * q_n$ = 0.55 * 8.98 = 4.94 ksf

Factored resistance at the strength limit state for the wingwall footing bearing in the very stiff to hard cohesive soils is equal to 4.9 ksf

By: IJH Date: 2/26/2025 Checked: IJH Date: 10/7/2025

SERVICE LIMIT STATE:

At this structure location, the maximum service limit state bearing pressure is 3.0 ksf, and the maximum strength limit state bearing pressure is 3.8 ksf, both of which are significantly lower than the factored bearing resistance of 30.2 ksf.
Settlement analysis performed under the Service Limit State, using the applied bearing pressure of 30.2 ksf, yielded total settlements in the range of 0.54 to 0.73 inches, which is below the maximum allowable settlement of 1 inch typically considered acceptable per GDM Section 1303.2.
This confirms that the foundation design satisfies both serviceability and strength requirements, with the Service Limit State loads being well within the bounds of the factored bearing resistance.

Furthermore, we consider the settlement to be insignificant at the wingwall footing, especially since the wingwall is proposed to be structurally connected to the abutment and the abutment is bearing on rock.

No need to reduce BC

(see attached *Settlement Calculation*)

.

By: msi Date: 10/20/2025 Checked: IJH Date: 10/23/2025

GENERAL FOUNDATION INFORMATION:

Wingwall at Forward Abutment: Width W = 7' , Length L = 12.5'
Bearing at approxiamtely 580 feet.

GENERAL SOIL INFORMATION:

Anticipated Bearing Conditions:

Predominantly Very Stiff to Hard Soil underlain by 1' zone very dense sand and then weathered bedrock.

Based on Soil Strength Evaluation Spreadsheet,

USE c' =	0	ksf - cohesionless soil
USE ϕ'=	30	Degrees

Groundwater

Model groundwater in creek above foundation bearing elevation.

STRENGTH LIMIT STATE:

$q_R = \varphi_b * q_n$ (AASTHO LRFD 10.6.3.1.1-1)

q_R = factored resistance at strength limit state (ksf)
φ_b = resistance factor (Article 10.5.5.2.2)
q_n = nominal bearing resistance (ksf)

$q_n = cN_{cm} + \gamma_q D_f N_{qm} C_{wq} + 0.5\gamma_f B N_{gm} C_{wg}$ (AASTHO LRFD 10.6.3.1.2a-1)
$N_{cm} = N_c s_c i_c$ (AASTHO LRFD 10.6.3.1.2a-2)
$N_{qm} = N_q s_q d_q i_q$ (AASTHO LRFD 10.6.3.1.2a-3)
$N_{gm} = N_g s_g i_g$ (AASTHO LRFD 10.6.3.1.2a-4)

c = cohesion, undrained shear strength (ksf)
N_c = cohesion term (Table 10.6.3.1.2a-1)
N_q = surcharge term (Table 10.6.3.1.2a-1)
N_g = unit weight term (Table 10.6.3.1.2a-1)
g = total (moist) unit weight (kcf)
D_f = footing embedment depth (ft)
B = footing width (ft)
C_{wq} , C_{wg} = groundwater correction factors (Table 10.6.3.1.2a-2)
s_c , s_g , s_q = shape correction factors (Table 10.6.3.1.2a-3)
d_q = shear resistance thought cohesionless material correction factor (Table 10.6.3.1.2a-4)
i_c , i_g , i_q = inclination correction factors

By:	msi	Date:	10/20/2025	Checked:	IJH	Date:	10/23/2025

Setup	$c =$	0	ksf	assumed zero in cohesionless soil	
	$\phi_f =$	30	degrees		
	$N_c =$	30.1	*unitless*		
	$N_q =$	18.4	*unitless*	for soil with a $\phi_f = 30$ Degrees	
	$N_\gamma =$	22.4	*unitless*		
	$\gamma =$	0.125	kcf	(assumed in upper 1.5 feet above bearing)	
	$D_f =$	6.5	ft	Minimum embedment of the footing into rock	
	$B =$	7	ft	Width Stage 2 plans	
	$L =$	12.5	ft	Length Stage 2 plans	
	$D_w =$	0	ft	highest anticipated groundwater depth	
	$C_{wq} =$	0.5	*unitless*	where $D_w = 0.0$	$1.5B + D_f =$ 17
	$C_{w\gamma} =$	0.5	*unitless*	(above D_f)	
	$s_c =$	1.34	*unitless*	$s_c = 1 + (B/L)*(Nq/Nc)$	$s_c = 1 + (B/(5L))$
	$s_\gamma =$	0.78	*unitless*	for $\phi_f > 0$ $s_g = 1 - 0.4 * (B/L)$	for $\phi_f = 0$ $s_g = 1$
	$s_q =$	1.25	*unitless*	$s_q = 1+((B/L)*\tan\phi_f)$	$s_q = 1$
	$d_q =$	1.0	*unitless*	taken as 1 since cohesionless soil on top of weathered rock	
	i_c , i_γ , i_q =	1.0	*unitless*	Assumed loaded without inclination	

calculation

$N_{cm} = N_c s_c i_c$ = 30.1 * 1.342 * 1 = 40.394

$N_{qm} = N_q s_q d_q i_q$ = 18.4 * 0.776 * 1 * 1 = 14.278

$N_{gm} = N_g s_g i_g$ = 22.4 * 1.252 * 1 = 28.045

$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma B N_{\gamma m} C_{w\gamma}$ — $cN_{cm} =$ 0

= (0*40.394) + (0.125*6.5*14.278*0.5) + (0.5*7*28.045*0.5) = — $\gamma D_f N_{qm} C_{wq} =$ 5.8

= (0) + (5.8) + (49.079) = — $0.5\gamma B N_{\gamma m} C_{w\gamma} =$ 49.079

$q_n =$ 54.88 ksf

$\varphi_b =$ 0.55 AASHTO LRFD Table 11.5.7-1 - SLS Res. Factor for Perm. Retain. Walls

$q_R = \varphi_b * q_n$ = 0.55 * 54.879 = 30.18 ksf

Factored resistance at the strength limit state for the wingwall footing bearing in the very stiff to hard cohesive soils is equal to 30.2 ksf

By: msi Date: 10/20/2025 Checked: IJH Date: 10/23/2025

SERVICE LIMIT STATE:

At this structure location, the maximum service limit state bearing pressure is 3.0 ksf, and the maximum strength limit state bearing pressure is 3.8 ksf, both of which are significantly lower than the factored bearing resistance of 30.2 ksf.
Settlement analysis performed under the Service Limit State, using the applied bearing pressure of 30.2 ksf, yielded total settlements in the range of 0.54 to 0.73 inches, which is below the maximum allowable settlement of 1 inch typically considered acceptable per GDM Section 1303.2.
This confirms that the foundation design satisfies both serviceability and strength requirements, with the Service Limit State loads being well within the bounds of the factored bearing resistance.

Furthermore, we consider the settlement to be insignificant at the wingwall footing, especially since the wingwall is proposed to be structurally connected to the abutment and the abutment is bearing on rock.

No need to reduce BC (see attached *Settlement Calculation*)

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By:	msi	Date:	10/22/2025	Checked:	IJH	Date:	10/23/2025

GENERAL FOUNDATION INFORMATION:

Wingwall at Rear Abutment:

Bottom Elevation	580	ft	Approximate bearing elevation	
B	7	ft	Width	B is the governing direction for this structure
L	12.5	ft	Length	
$\gamma_{concrete}$	0.15	Kcf	Unit weight	

ECCENTRICITY, e, in Governing Direction GDM 1303.1.2

$e_B = \Sigma M / \Sigma V$

$e_B =$ 0.00

NOTE: At strength limit state. Provided by Structural Engineer.

LIMITING ECCENTRICITY, e_{Limit}

In Governing Direction AASHTO LRFD 10.6.3.3

$e_{Limit} = 0.45 * B$

$e_{Limit} =$ 3.15

EFFECTIVE FOOTING DIMENSIONS AASHTO LRFD 10.6.1.3

$B' = B - 2e_B$

B' = 7.00 ft.

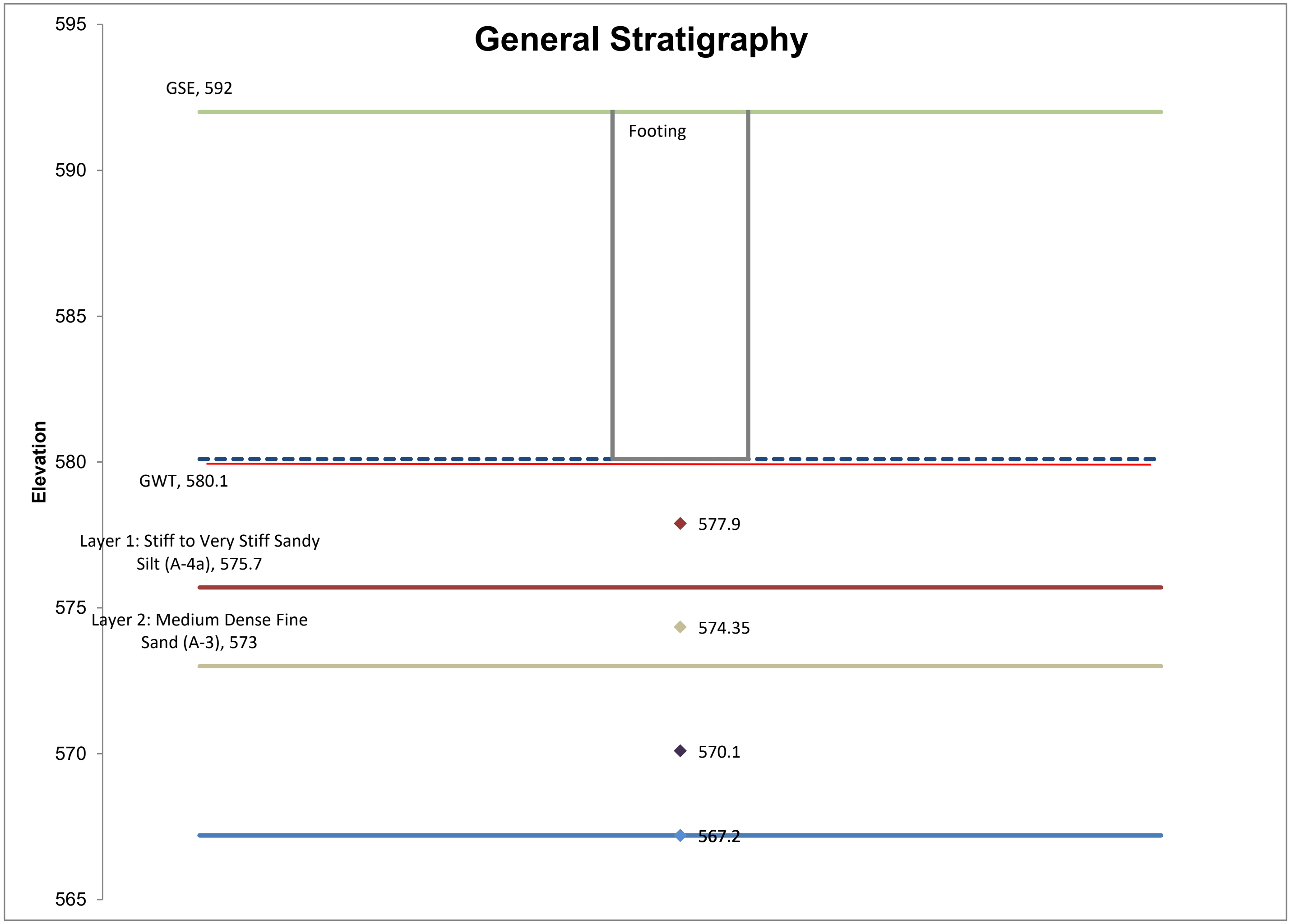
B' = 7.00 ft. approx.

$L' = L - 2e_L$ NOTE: Not applicable

Project Name:	ATB Old Main Bridge	Boring Number	B-003-0-21	Rev.	2	by:	msi
Project Number:	232245	Analysis Type	Boussinesq Continuous	Date:	10/8/2025	Checked by:	IHJ, 10/23/25
Calculated by:	IJH 2/26/25						

Wingwall - Indicated 4.9 ksf bearing pressure.

Layers and Soil Type	H (feet)	C_r	e_o	σ_v (psf)	z (feet)	b (feet)	(z-Df) / b	I_z	Δ p@ 4900 psf	(check) sigma v+ΔP	Δ H (inches)
Layer 1: Stiff to Very Stiff Sandy Silt (A-4a)	4.4	0.022	0.59	1696	2.2	7	0.3	0.9	4410	6106	0.41
Layer 2: Medium Dense Fine Sand (A-3)	2.7	0.012	0.45	1936	5.75	7	0.8	0.6	2940	4876	0.11
Layer 3: Rock	5.8	0.009333	0.43	2223	10	7	1.4	0.4	1960	4183	0.12
Layer 3: Rock	0	0	0.30	2419	12.9	7	1.8	0.3	1470	3889	0.00
Layer 3: Rock	0	0	0.30	2419	12.9	7	1.8	0.3	1470	3889	0.00
Layer 3: Rock	0	0	0.30	2419	12.9	7	1.8	0.3	1470	3889	0.00



Total Δ H (in.)	0.64
+15%	0.73
-15%	0.54

pc = s_u/(0.11+0.0037(PI))

s_u (psf)= 1,500

Max PI = 7

pc (psf)= 11,038

All σ_v + Δ P < pc, so all Cr

OHIO DEPARTMENT OF TRANSPORTATION

OFFICE OF GEOTECHNICAL ENGINEERING

PLAN SUBGRADES
Geotechnical Design Manual Section 600

Ashtabula, Old Main Street
119471

ATB-OLD MAIN STREET BRIDGE

CT/VERDANTAS

Prepared By: MSI
Date prepared: Thursday, October 23, 2025

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NO. OF BORINGS: 1

NO. OF DCPS:

Add DCP Test Data Worksheets

#	Boring ID	Alignment	Station	Offset	Dir	Drill Rig	ER	Boring EL.	Proposed Subgrade EL	Cut Fill
1	b-004-0-23	US-20	13+21	3	RIGHT	CME AUTOMATIC	73	590.2	589.2	1.0 C

OHIO DEPARTMENT OF TRANSPORTATION

Subgrade Analysis

V. 14.81 7/21/2025

#	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	SS-1	1.5	3.0	0.5	2.0	12									15	0	UCF			UCF	N_{60} & Mc	24''	12''	**REMOVE AND REPLACE 24-INCH UCF**
	004-0	SS-2	3.0	4.5	2.0	3.5	5						25	3	28	7	10	A-2-4	0			N_{60}			**204 Geotextile**
	23	SS-3	4.5	6.0	3.5	5.0	10						33	5	38	10	10	A-4a	5						
		SS-4	6.0	7.5	5.0	6.5	11	5								22	14	A-6a	10						

PID: 119471

County-Route-Section: Ashtabula, Old Main Street
No. of Borings: 1

Geotechnical Consultant: CT/VERDANTAS
Prepared By: MSI
Date prepared: 10/23/2025

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): Average(HP):	 21" 0"
Global Geogrid Average(N60L): Average(HP):	 15" 0"

Design CBR	8

% Samples within 3 feet of subgrade			
$N_{60} \leq 5$	25%	$HP \leq 0.5$	0%
$N_{60} < 12$	25%	$0.5 < HP \leq 1$	0%
$12 \leq N_{60} < 15$	25%	$1 < HP \leq 2$	0%
$N_{60} \geq 20$	0%	$HP > 2$	0%
M+	25%		
Rock	0%		
Unsuitable Soil	50%		

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

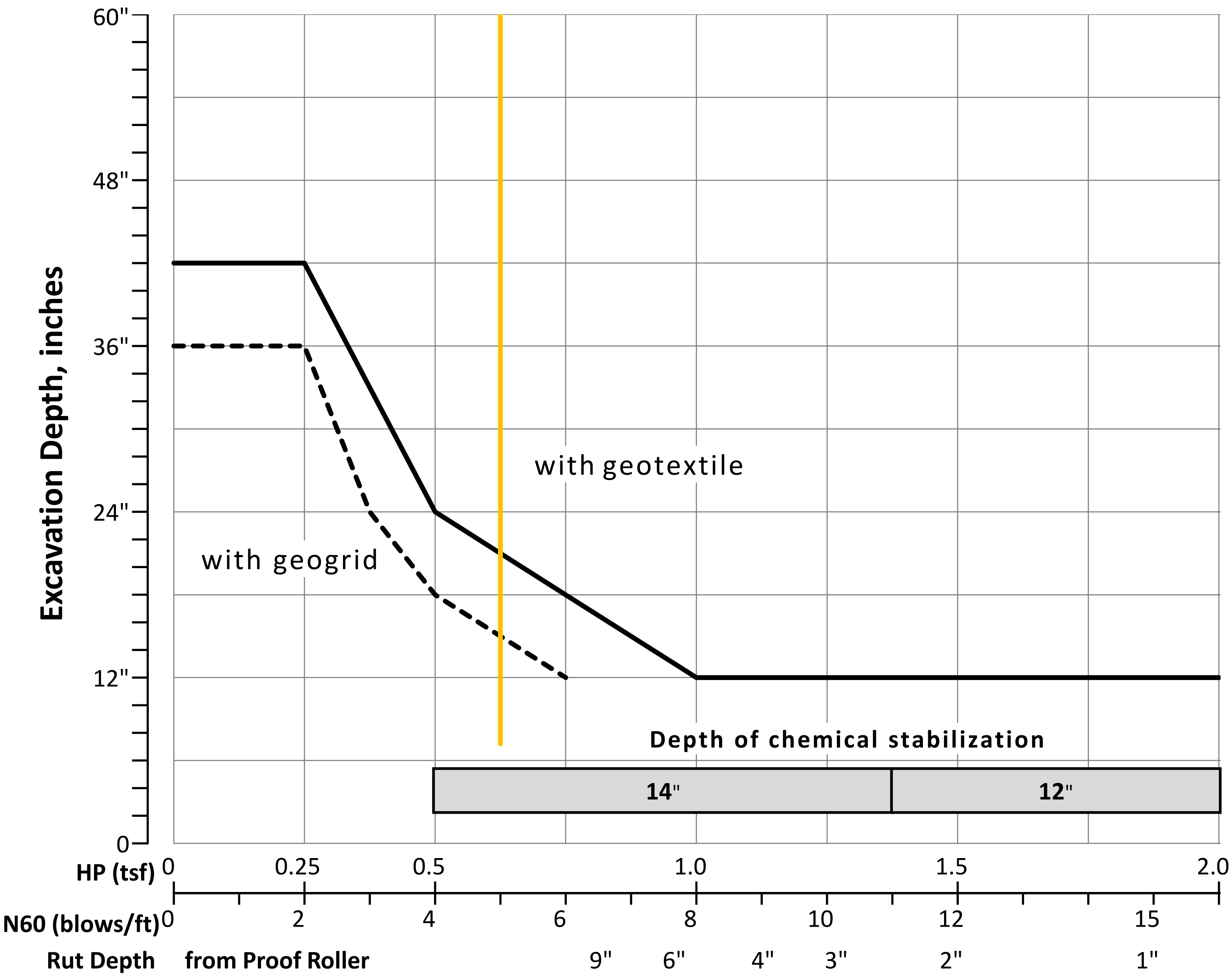
% Proposed Subgrade Surface	
Unstable & Unsuitable	150%
Unstable	100%
Unsuitable (Soil & Rock)	50%

	N_{60}	N_{60L}	HP	LL	PL	PI	Silt	Clay	P 200	M_C	M_{OPT}	GI
Average	10	5	NP	0	0	0	29	4	33	14	9	5
Maximum	12	5	NP	0	0	0	33	5	38	22	14	10
Minimum	5	5	NP	0	0	0	25	3	28	7	0	0

Classification Counts by Sample

ODOT Class	UCF	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	1	0	0	0	1	0	0	0	0	0	1	0	0	1	0	0	0	0	0	4
Percent	25%	0%	0%	0%	25%	0%	0%	0%	0%	0%	25%	0%	0%	25%	0%	0%	0%	0%	0%	100%
% Rock|Granular|Cohesive	25%	0%	50%									25%								100%
Surface Class Count	1	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2
Surface Class Percent	50%	0%	0%	0%	50%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	100%

Fig. 600-1 – Subgrade Stabilization



OVERRIDE TABLE

Calculated Average	New Values	Check to Override
NP	0.50	☐ HP
5.00	6.00	☐ N60L

Average HP

Average N_{60L}

Appendix B

Geotechnical Engineering Design Checklists

I. Geotechnical Design Checklists

Project: ATB Old Main St. Bridge (Conneaut)	**PDP Path:**
PID: 119471	**Review Stage:**

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: (Conneaut)	PID: 119471	Reviewer: IEH	Date: 3/17/2025

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	No historic boring were found at the project location.
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	Y	
c.	estimated amount of rock and soil, including the total for each for the entire program.	Y	
Planning – Exploration Number		(Y/N/X)	Notes:
11	Have the coordinates, stations and offsets of all explorations (borings, soundings, test pits, etc.) been identified?	Y	
12	Has each exploration been assigned a unique identification number, in the following format X-ZZZ-W-YY, as per Section 303.2 of the SGE?	Y	
13	When referring to historic explorations that did not use the identification scheme in 12 above, have the historic explorations been assigned identification numbers according to Section 303.2 of the SGE?	X	

II. Reconnaissance and Planning Checklist

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

III.C. Subgrade Checklist

C-R-S: (Conneaut)	PID: 119471	Reviewer: IEH	Date: 3/17/2025

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	
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 anticipated. Extent of Removal and replacement is shown in the report. 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?	X	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	Plans to be prepared by others.
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: (Conneaut)	PID: 119471	Reviewer: IEH	3/17/2025

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.

		(Y/N/X)	Notes:
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:	UCS	
	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	Y	
5	Have the recommended bottom of footing elevation and reason for this recommendation been provided?	Y	
a.	Has the recommended bottom of footing elevation taken scour from streams or other water flow into account?	N	Scour is not aticipated at that footing elevation.
6	Were representative sections analyzed for the entire length of the structure for the following:	Y	
a.	factored bearing resistance?	Y	
b.	factored sliding resistance?	N	Recommended soil parameters provided.
c.	eccentric load limitations (overturning)?	N	
d.	predicted settlement?	Y	
e.	overall (global) stability?	N	
7	Has the need for a shear key been evaluated?	N	
a.	If needed, have the details been included in the plans?	X	Plans to be prepared by others.
8	If special conditions exist (e.g. geometry, sloping rock, varying soil conditions), was the bottom of footing “stepped” to accommodate them?	X	Conditions not present.
9	Have the Service I and Maximum Strength Limit States for bearing pressure on soil or rock been provided?	Y	

IV.A Foundations of Structures Checklist

Spread Footings		(Y/N/X)	Notes:
10	If weak soil is present at the proposed foundation level, has the removal / treatment of this soil been developed and included in the plans?	X	Conditions not present
a.	Have the procedure and quantities related to this removal / treatment been included in the plans?	X	See response from Item 10, above.
Pile Structures		(Y/N/X)	Notes:
11	Are there piles on the project? If no, go to Question 17	N	
12	Has an appropriate pile type been selected?		
	Check the type selected:		
	H-pile (driven)		
	H-pile (prebored)		
	Cast In-place Reinforced Concrete Pipe		
	Micropile		
	Continuous Flight Auger (CFA)		
	other (describe other types)		
13	Have the estimated pile length or tip elevation and section (diameter) based on either the Ultimate Bearing Value (UBV) or the depth to top of bedrock been specified? Indicate method used.		
14	If scour is predicted, has pile resistance in the scour zone been neglected?		
15	Has a wave equation drivability analysis been performed as per BDM 305.3.1.2 to determine whether the pile can be driven to either the UBV, the pile tip elevation, or refusal on bedrock without overstressing the pile?		
16	If required for design, have sufficient soil parameters been provided and calculations performed to evaluate the:		
a.	Nominal unit tip resistance and maximum settlement of the piles?		
b.	Nominal unit side resistance for each contributing soil layer and maximum deflection of the piles?		
c.	Downdrag load on piles driven through new embankment or compressible soil layers, as per BDM 305.3.2.2?		
d.	Potential for and impact of lateral squeeze from soft foundation soils?		

IV.A Foundations of Structures Checklist

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

IV.A Foundations of Structures Checklist

Drilled Shafts		(Y/N/X)	Notes:
20	Are there drilled shafts on the project? If no, go to the next checklist.	Y	
21	Have the drilled shaft diameter and embedment length been specified?	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	
a.	total factored lateral shear?	Y	
b.	total factored bending moment?	Y	
c.	maximum deflection?	Y	
d.	reinforcement design?	X	
24	If a bedrock socket is required, has a minimum rock socket length equal to 1.5 times the rock socket diameter been used, as per BDM 305.4.2?	Y	
25	Generally, bedrock sockets are 6" smaller in diameter than the soil embedment section of the drilled shaft. Has this factor been accounted for in the drilled shaft design?	Y	
26	If scour is predicted, has shaft resistance in the scour zone been neglected?	✓	See response from Item 4a, above.
27	Has the site been assessed for groundwater influence?	N	
a.	If yes, and if artesian flow is a potential concern, does the design address control of groundwater flow during construction?	X	
28	Have all the proper items been included in the plans for integrity testing?	N	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?	N	
30	If necessary, have wet construction methods been specified?	N	
General		(Y/N/X)	Notes:
31	Has the need for load testing of the foundations been evaluated?	N	
a.	If needed, have details and plan notes for load testing been included in the plans?		

VI.B. Geotechnical Reports

C-R-S: (Conneaut)	**PID:** 119471	**Reviewer:** IEH	**Date:** 3/17/2025

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'?	X	This is a draft submittal.
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	gINT project file will be sent with final report.
5	Does the report cover format follow ODOT's Brand and Identity Guidelines Report Standards found at http://www.dot.state.oh.us/brand/Pages/default.aspx ?	Y	
6	Have all geotechnical reports being submitted been titled correctly as prescribed in Section 706.1 of the SGE?	Y	
Report Body		(Y/N/X)	Notes:
7	Do all geotechnical reports being submitted contain the following:	Y	
a.	an Executive Summary as described in Section 706.2 of the SGE?	Y	
b.	an Introduction as described in Section 706.3 of the SGE?	Y	
c.	a section titled "Geology and Observations of the Project," as described in Section 706.4 of the SGE?	Y	
d.	a section titled "Exploration," as described in Section 706.5 of the SGE?	Y	
e.	a section titled "Findings," as described in Section 706.6 of the SGE?	Y	
f.	a section titled "Analyses and Recommendations," as described in Section 706.7 of the SGE?	Y	
Appendices		(Y/N/X)	Notes:
8	Do all geotechnical reports being submitted contain all applicable Appendices as described in Section 706.8 of the SGE?	Y	
9	Do the Appendices present a site Boring Plan showing all boring locations as described in Section 706.8.1 of the SGE?	Y	

VI.B. Geotechnical Reports

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

VII. References

Publications - FHWA

Advanced Course on Slope Stability, Volume 1 and 2, Abramson, Lee, Boyce, Glenn, et al., Publication No. FHWA-SA-94-005 and 006

Corrosion/Degradation of Soil Reinforcement for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Elias, Publication No. FHWA-NHI-09-087

Geotechnical Engineering Circular No. 2 - Earth Retaining Systems, Sabitini, Elias, et al., Publication No. FHWA-SA-96-038

Geotechnical Engineering Circular No. 3 - LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations, Kavazanjian, Publication No. FHWA-NHI-11-032

Geotechnical Engineering Circular No. 4 - Ground Anchors and Anchor Systems, Sabitini, Pass and Bachus, Publication No. FHWA-IF-99-015

Geotechnical Engineering Circular No. 5 – Geotechnical Site Characterization, Loehr, et. al., Publication No. FHWA-NHI-16-072

Geotechnical Engineering Circular No. 6 – Shallow Foundations, Kimmerling, Publication No. FHWA-IF-02-054

Geotechnical Engineering Circular No. 7 – Soil Nail Walls Reference Manual, Lazarte, et. al., Publication No. FHWA-NHI-14-007

Geotechnical Engineering Circular No. 8 – Design and Construction of Continuous Flight Auger Piles, Brown, et. al., Publication No. FHWA-HIF-07-039

Geotechnical Engineering Circular No. 9 – Design and Analysis of Laterally Loaded Deep Foundations, Parkes, et. al., Publication No. FHWA-HIF-18-031

Geotechnical Engineering Circular No. 10 - Drilled Shafts: Construction Procedures and Design Methods, Brown, et. al., Publication No. FHWA-NHI-18-024

Geotechnical Engineering Circular No. 11 - Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Volume I and II, Berg, Christopher, and Samtani, Publication No. FHWA-NHI-10-024 and 025

Geotechnical Engineering Circular No. 12 - Design and Construction of Driven Pile Foundations, Volume I and II, Hannigan, Rausche, Likins, Robinson, and Becker, Publication No. FHWA-NHI-16-009 and 010

Geotechnical Engineering Circular No. 13 – Ground Modification Methods Reference Manual, Volume I and II, Schaefer, et. al., Publication No. FHWA-NHI-16-027 and 028

Geotechnical Engineering Circular No. 15 – Acceptance Procedures for Structural Foundations, Loehr, et. al., Publication No. FHWA-HIF-22-024

Geotechnical Instrumentation Reference Manual, Dunnicliff, NHI Course No. 13241 - Module 11

Prefabricated Vertical Drains: Volume 1: Engineering Guidelines, Rixner, Kraemer, and Smith, Publication No. FHWA-RD-86-168

Soils and Foundations Workshop, Reference Manual and Participant Workbook, Cheney and Chassie, Publication No. NHI-00-045

Soils and Foundations Reference Manual, Volume I and II, Samtani and Nowatzki, Publication No. NHI-06-088 and 089

Highway Subdrainage Design, Moulton, Publication No. FHWA-TS-80-224

Tiebacks, Weatherby, Publication No. FHWA/RD-82/047

VII. References

Publications - ODOT (www.dot.state.oh.us/drrc/)

Bridge Design Manual, Office of Structural Engineering
CADD Engineering Standards Manual, Office of CADD and Mapping
Construction and Material Specifications, Office of Construction Administration
Geotechnical Design Manual, Office of Geotechnical Engineering
Location and Design Manual: Volume 1 - Roadway Design, Office of Roadway Engineering
Location and Design Manual: Volume 3 - Highway Plans, Office of CADD and Mapping
Manual for Abandoned Underground Mine Inventory and Risk Assessment (AUMIRA), Office of Geotechnical Engineering
Pavement Design Manual, Office of Pavement Engineering
Specifications for Geotechnical Explorations, Office of Geotechnical Engineering

Publications - ODNR (www.dnr.state.oh.us/)

Bedrock Geology Map, DGS
Bedrock Structure Map, DGS
Geologic Map of Ohio, DGS
Quaternary Geology of Ohio, DGS

Bedrock Topography Map, DGS
Known and Probable Karst in Ohio, DGS
USGS Open File Map Series #78-1057 Landslides and Related Features, DGS

Other publications or information available from ODNR:

Bulletins	Boring logs	Measured geologic section(s)
Information Circulars	Water well logs	Report of Investigations

Publications – Other Organizations

AASHTO LRFD Bridge Design Specifications, Highway Subcommittee on Bridges and Structures, latest edition
Soil Survey, Natural Resources Conservation Service (https://www.nrcs.usda.gov/wps/portal/nrcs/main/soils/survey/)
Wetlands Mapper, National Wetlands Inventory (https://www.fws.gov/wetlands/data/Mapper.html)