# STRUCTURE FOUNDATION EXPLORATION

## Proposed Culvert Replacement

WOO 65-6.18, PID 107711

SR 65 over Existing Williamsburg Reservoir Outlet Structure to Maumee River

Washington Township, Wood County, Ohio

Submitted to Tetra Tech, Inc.
Date April *2025*

Prepared by

TTL associates inc
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April 2, 2025 **TTL Project No. 2130501**

Mr. David T. Charville, PE
Tetra Tech, Inc.
420 Madison Avenue, Suite 1001
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# Final Report
# Structure Foundation Exploration
# Proposed Culvert Replacement
# WOO-65-6.18, PID 107711
# Washington Township, Wood County, Ohio

Dear Mr. Charville:

Following is the final report of our structure foundation exploration performed by TTL Associates, Inc. (TTL) for the referenced site. This study was performed in accordance with TTL Proposal No. 2130501R, dated August 22, 2021, and was authorized with a Tetra Tech, Inc. Subconsultant agreement signed by you on September 22, 2021, for which you referenced Tetra Tech Project No. 200-12914-21002, Task 003.A.

A draft geotechnical report was submitted on May 26, 2022 for the replacement culvert planned at that time. A revised draft report was submitted on October 24, 2023 for the planned replacement with a bridge supported on driven piles to bedrock. Based on comments and loads provided on January 13, 2025, another revised draft report was provided on January 23, 2025 containing recommendations for support of the bridge using drilled shafts socketed into bedrock, due to the depth of potential scour that affected the driven pile design. This report also contains the results of our study, our engineering interpretation of the results with respect to the project characteristics, as well as our design and construction recommendations for the replacement bridge and associated pavements. We were notified on March 24, 2025 that no further changes to the report were requested. Therefore, this submittal is considered final.

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

Sincerely,

**TTL Associates, Inc.**

STATE OF OHIO
CHRISTOPHER P IOTT
E-69734
REGISTERED PROFESSIONAL ENGINEER

Imad El Hajjar, EI
Geotechnical Project Manager

Christopher P. Iott, P.E.
Chief Geotechnical Engineer

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# FINAL REPORT
# STRUCTURE FOUNDATION EXPLORATION
# PROPOSED CULVERT REPLACEMENT
# WOO-65-6.18, PID 107711
# SR 65 OVER EXISTING WILLIAMSBURG RESERVOIR OUTLET STRUCTURE TO MAUMEE RIVER
# WASHINGTON TOWNSHIP, WOOD COUNTY, OHIO

**FOR**

**TETRA TECH, INC.**
**420 MADISON AVENUE, SUITE 1001**
**TOLEDO, OHIO  43604**

**SUBMITTED**

**APRIL 2, 2025**
**TTL PROJECT NO. 2130501**

**TTL ASSOCIATES, INC.**
**1915 NORTH 12TH STREET**
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# EXECUTIVE SUMMARY

This structure foundation exploration report has been prepared for the proposed replacement with a new bridge of the existing culvert along State Route 65 (SR 65), in Washington Township, Wood County, Ohio, designated as WOO-65-6.18, PID No. 107711. This exploration included two test borings, each of which included a pavement core. A summary of the conclusions and recommendations of this study are as follows:

1. The pavement section encountered in Borings B-003 and B-004 consisted of approximately 14 inches and 8½ inches of asphalt, respectively, underlain by approximately 17¼ inches and 13 inches of aggregate base, respectively.

2. The subsoils encountered underlying the pavement materials can be generally described as predominantly cohesive soils overlying bedrock. The encountered subsoils consisted of predominantly native cohesive soils encountered underlying the pavement materials to depths of approximately 33½ feet. These soils consisted of sandy silt (A-4a), silt and clay (A-6a) and silty clay (A-6b). The upper portion of the cohesive soils exhibited generally stiff to very stiff consistency. This layer extended to a depth of 23 feet (Elev. 609±) in Boring B-003 and to a depth of 26½ feet (Elev. 606±) in Boring B-004. The lower portion of the cohesive soil profile exhibited generally hard consistency. Weathered dolomite that was able to be penetrated with augers was encountered underlying the hard cohesive soils to auger refusal at a depth of approximately 34½ feet in Boring B-004. Upon encountering auger refusal in Borings B-003 and B-004, the rock was cored for a total length of 5 feet. The cored bedrock consisted of slightly to moderately weathered dolomite.

3. During our site reconnaissance on October 19, 2021, water levels in the waterway were approximately 1 to 2 feet deep. The waterway bottom was approximately 13 to 15 feet below the road surface. Hence, based on our field observations, water was approximately 12 to 13 feet below the road surface (Elevs. 620± to 619±) during the more normal/non-rain-influenced observation date of October 19, 2021. Apart from streamflow influences in the waterway and nearby Williamsburg Reservoir, it is our opinion that the “normal” groundwater level can generally be expected at depths corresponding to the water elevation in the Maumee River. Based on google earth, the water elevation in the Maumee River in the vicinity of the project site is at approximate Elev 619± corresponding to approximately 13 to 13½ feet below existing grades along SR-65.

4. Based on the relatively shallow bedrock at the site and the potential for scour, we understand that the new bridge is planned to be supported by a deep foundation system consisting of drilled shafts socketed into bedrock. The maximum total factored load was indicated to be 365 kips. For the abutments, we have evaluated a 36-inch diameter shafts above bedrock and a socket diameter of 30 inches. **Using the planned 30-inch diameter socket, this design value would provide sufficient resistance for the indicated factored vertical load.** The minimum prescribed rock socket length is 1.5B, where B is the socket diameter. For the socket diameter of 30 inches indicated above, the minimum socket length would be 3.75 feet for bearing elevations on the order of Elevs. 595±. It was indicated that potential scour extends to within approximately 5 feet above the top of bedrock. Based on this information, additional socket length is not required based on BDM 305.4.1.1 scour

considerations. In any case, any structural requirement for the drilled shaft foundations to resist lateral loads or moments may increase the socket depth or diameter and should be evaluated on an individual shaft basis by Tetra Tech. Design soil and bedrock parameters are provided herein for lateral-load evaluations by Tetra Tech.

5. It should be noted that the values for socketed drilled shaft factored unit tip resistance listed in Section 5.1.1 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. **Since the replacement structure was originally planned as a culvert, only 5 feet of rock coring was performed in each boring. As such, the cored rock only extends approximately ½ socket diameter and 1 socket diameter below the anticipated end-bearing elevations at the Rear Abutment and Forward Abutment, respectively. It was indicated by TTL that, if the risk associated with non-cored rock within 2 socket diameters of the end-bearing elevation is not acceptable, additional rock coring would need to be performed. Subsequently, it was indicated by ODOT District 2 that the 5-foot rock core runs were suitable for this project.**

6. Based on the GB-1 analysis, a design CBR value of 6 percent was determined for the project. It should be noted that the CBR determination by the GB-1 spreadsheet is based on the average Group Index of all the evaluated subgrade samples, which was 9. Group indices for all the tested samples ranged from 0 to 16, which would correlate with a CBR value of 4 to 12 percent. Cohesive subgrade soils classified as ODOT A-4a, A-6a and A-6b were predominantly present within the upper 6 feet of the subgrade elevation in both borings. The average group index for the tested A-4a, A-6a and A-6b samples was also 9. Based on the average design value calculations from GB-1, it does not appear to be unconservative to use the GB-1 design CBR value of 6 percent for new pavement sections throughout the project area.

7. The GB-1 analysis indicates options for global chemical stabilization using cement to a depth of 14 inches or planned over-excavation of unsuitable subgrade soils to a depth of 12 inches and replacement with new granular engineered fill for the entire extent of the project. Due to the relatively small project area, global chemical stabilization is not anticipated to be economical compared to over-excavation and replacement with granular engineered fill. The GB-1 analysis spreadsheet indicates that rubblize and roll is not an option for this project.

This executive summary highlights our evaluations and recommendations and should only be utilized in conjunction with the accompanying report, including the detailed findings, analysis and recommendations, and qualifications presented herein.

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# 1.0 INTRODUCTION

This structure foundation exploration report has been prepared for the proposed replacement of the Williamsburg Reservoir outlet structure to Maumee River beneath State Route 65 (SR 65) in Washington Township, Wood County, Ohio, designated as WOO-65-6.18 PID No. 107711. The project is located near the southeast banks of Maumee River, approximately half way between Grand Rapids and Waterville,  between SR 235 and Brillhart Road, as shown on the attached Site Location Map (Plate 1.0).

This study was performed in accordance with TTL Proposal No. 2130501R, dated August 22, 2021, and was authorized with a Tetra Tech, Inc. Subconsultant agreement signed by Mr. David Charville, PE on September 22, 2021. The subconsultant agreement referenced Tetra Tech Project No. 200-12914-21002, Task 003.A. The revised draft report previously submitted on October 24, 2023 has been revised to include support of the bridge using drilled shafts socketed into bedrock rather than driven piles to bedrock due to the extent of potential scour at the bridge location that was of concern for the driven piles.

## 1.1 Purpose and Scope of Exploration

The purpose of this exploration was to evaluate the subsurface conditions relative to installation and support of a single-span bridge and associated pavement reconstruction at the referenced location. To accomplish this, TTL performed two test borings, both of which included a pavement core, field and laboratory soil testing, a geotechnical engineering evaluation of the test results, as well as review of available geologic and soils data for the project area.

This report summarizes our understanding of the proposed construction, describes the investigative and testing procedures utilized to evaluate the subsurface conditions at the site, and presents our findings from the field and laboratory testing. This report also presents our design and construction recommendations for bridge foundations consisting of drilled shafts socketed into bedrock, our evaluations and conclusions with respect to pavement subgrade conditions in accordance with ODOT GB-1 “Plan Subgrades” (January 15, 2021), as well as our design and construction recommendations for pavements.

This report includes:

- A description of the existing surface cover, subsurface soils, and groundwater conditions encountered in the borings.
- Design recommendations for bridge foundations and pavements.

- Recommendations concerning soil- and groundwater-related construction procedures such as site preparation, subgrade preparation in accordance with ODOT GB-1 criteria, earthwork, pavement construction, pile foundation installation, as well as related field testing.

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

The scope of this study did not include an environmental assessment of the surface or subsurface materials at this site.

## 1.2 Proposed Construction

It is our understanding that the existing outlet structure will be replaced with a new single-span bridge. The roadway grade and width will remain approximately the same as the current structure.

Due to potential scour extending to within approximately 5 feet of the top of bedrock, it is planned to support the bridge using drilled shaft socketed into bedrock rather than driven piles to bedrock. The total factored load for an individual drilled shaft was indicated to be 365 kips. The bottoms of the abutments are planned at Elevs. 624.25 and 625.48 for the rear and forward abutments, respectively.

# 2.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

## 2.1 General Geology and Hydrogeology

Published geologic maps from the Ohio Department of Natural Resources (ODNR) indicate that the project site is located near the borderline of the Maumee Lake Plains Physiographic Region and the Maumee Sand Plains District within that region. With the lack of encountered sand in the upper profile of the borings performed for this exploration, it is interpreted that the site is located within the Maumee Lake Plains Region. Within this region, the geologic deposits consist of Pleistocene-age silt, clay, and wave-planed clayey till overlying Silurian- and Devonian-age carbonate rocks and shales.

The lacustrine soils are generally characterized as mostly soft to medium stiff clays, often with a desiccated stiffer layer within the upper portion of the profile. The lacustrine deposits generally do not exhibit significant overconsolidation, although the desiccation effects induce some apparent overconsolidation within the near-surface soils. In addition to the clayey lake bottom deposits, alluvial deposits may be encountered overlying the till.

The glacial till, also referred to as moraine, was deposited by the advance and retreat of glacial ice. Due to the weight of the ice mass, the till deposits are moderately to highly over-consolidated, that is, the existing soil deposits have experienced a previous vertical stress significantly higher than the present effective vertical stress due to the remaining overlying soil strata in the profile. The till may contain cobbles and/or boulders left in the till soil matrix. Additionally, seams of granular soils may also be encountered within glacial tills. These granular seams may or may not be water bearing. In the Maumee Lake Plains Physiographic Region, the surface of the glacial till has generally experienced some reworking from wave action of the historic lake.

Bedrock in the project area is broadly mapped on the "Geologic Map of Ohio" as middle and lower Devonian-age dolomite of the Detroit River Group. Top of bedrock was encountered at depths of approximately 33½ feet below existing grades (Elev. 599±) in Borings B-003 and B-004.

Review of the ODNR "Ohio Karst Areas" map indicated that the site is not in an area of probable karst. A Review of the Ohio Department of Natural Resources (ODNR) Map of Mines indicated no historic mining activity in the vicinity of the site area.

The USDA Soil Conservation Service (SCS) "Soil Survey of Sandusky County, Ohio" indicates that the near-surface soils in the project area are mapped as Sloan Silt loam (Sna). These soils are comprised of a surface layer of silty and/or clayey loamy alluvium becoming a stratified loam to silty clay loam to gravelly sandy loam with depth which has been deposited as flats or backswamps on flood plains. SnA soils are considered very poorly drained with a very low permeability.

## 2.2 Site Reconnaissance

TTL performed site reconnaissance on October 19 and 28, 2021. The site is located in a predominantly wooded areas around State Route 65 (SR 65). Williamsburg Reservoir and its outlet to the Maumee River were located to the south of the culvert. A gravel lot and an apparent electrical transformer were located to the southwest of the culvert. Additionally, two retaining wall structures were located in the vicinity of the existing outlet structure, one supporting the gravel lot and a second one approximately 140 feet west of the culvert along the south side of SR 65. The concrete wall was approximately 150 feet in length and varied in height from approximately 2 to 4½ feet, tallest in the middle. Several large vertical cracks were noted along the wall extending from top to bottom. A section of the wall was apparently replaced and weepholes were observed to have been added along the bottom of that section, as the original wall section did not exhibit any apparent weepholes.

The retaining wall supporting the gravel lot varied in height from approximately 3 to 5½ feet, tallest in the corner nearest the culvert inlet.

In the immediate area of the culvert, the pavement along SR 65 was observed to generally be in good condition, albeit heavily weathered. Significant pavement distresses were not observed. Solar powered devices, presumably lane illuminators, were imbedded in the pavement. These lane illuminators were in poor condition, with the glass frosted/yellowed, glass broken/missing, or panted over by the lane divider. It is assumed that most or all of these devices were not in working order.

The existing culvert appeared to be made of corrugated metal sheets bolted together and was circular or semi-circular in cross-sectional shape. The culvert dimensions were approximately 8 feet tall from the top most part of the culvert to ditch bottom, approximately 16 feet wide at the bottom, and is approximately 60 feet in length. The culvert appeared to have light rust along the waterline. The slopes around the culvert were supported by gabions (rock-filled wire/mesh blocks). One pipe appeared to discharge into the ditch at the south end of the culvert.

The Williamsburg Reservoir outlet was rounded in shape and lined with concrete. The concrete appeared in fair condition. One concrete slab joint, approximately half way along the outlet, was noted to be significantly misaligned, creating a step down. Water loss or erosion of the supporting soils did not appear to be evident.

South of the culvert, the reservoir was observed to have sheet-piling installed with tie-backs. Along the sheet-piling, several depressions in the soils were noted. In the areas of these depressions, the tie-back structures were more exposed and the sheet-pile wall appeared to be slightly bowing outward into the reservoir. However, the backslopes appeared to generally be in good condition.

Southwest of the culvert, the reservoir was not observed to have sheet-piling. Instead, this area included stone, a few feet wide, placed at the water line. The slopes along this part of the reservoir appeared to be in good to fair condition. However, evidence of past minor surface sluffing/erosion was noted in a few areas along the backslopes.

Grades in the project area tended to increase in elevation in the southern direction. However, grades were diverse, due to the existence of multiple retaining walls and earthen structures associated with the reservoir. Grades along SR 65 were significantly lower in the area of the culvert than surrounding road grades. The waterway bottom was approximately 13 to 15 feet below the road surface with relatively steep slopes.

At the time of the October 19, 2021 reconnaissance, water levels at the culvert were approximately 1 to 2 feet deep. October 28, 2021 reconnaissance was performed shortly after significant rainfall in the area, and the water levels rose by approximately 3 to 4 feet. The rise in water levels flooded a small portion of the surrounding wooded areas.

# 3.0 EXPLORATION

## 3.1 Historic Borings

Review of ODOT records indicated that no historic test borings have been drilled within the project area.

## 3.2 Project Exploration Program

Two test borings, designated as Borings B-003-0-21 and B-004-0-21 were drilled by TTL on October 20 and 22, 2021. Pavement cores were also obtained at the boring locations. These borings and pavement cores are fully designated as Borings B-001-0-21 and B-002-0-21 in accordance with ODOT protocol, but the -0-21 portion of the nomenclature is generally omitted in the discussions within this report. Borings B-003 and B-004 were located in the northbound and southbound lanes of SR 65, respectively, drilled near the inlet and outlet sides of the culvert, respectively. The existing site features and approximate locations of the borings are presented on the Test Boring Location Plan (Plate 2.0).

Stationing and offsets at the boring locations were provided by Tetra Tech, Inc. Latitude, Longitude, and ground surface elevations were surveyed by TTL via a hand-held GPS device. The accuracy from the handheld GPS device was generally found to be approximately 2 to 6 inches horizontal, and approximately 4 to 12 inches vertical. These data are presented on the logs of test borings as well as in the following table.

**Table 3.2 General Boring Location Information**

| Boring Number | Centerline SR 65 Station (feet) | Offset (feet) | Ground Surface Elevation (feet) | Latitude (Degrees) | Longitude (Degrees) |
|---|---|---|---|---|---|
| B-003-0-21 | Sta. 326+12 | 8 Right | 632.1 | 41.455621 | -83.781212 |
| B-004-0-21 | Sta. 326+45 | 6 Left | 632.4 | 41.455682 | -83.781111 |

The two culvert borings (B-003 and B-004) were planned as Type E2a box culvert with a diameter/span of greater than 10 feet and sampled to 6 feet as a ODOT type A Borings per geotechnical investigative procedures outlined in Ohio Department of Transportation (ODOT) "Specifications for Geotechnical Explorations" (SGE).

Borings B-003 and B-004 were terminated after encountering auger refusal at a depth of approximately 33½ and 34½ feet below existing grade, respectively, then coring 5 feet of rock.

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

## 3.3 Boring Methods

The test borings performed during this exploration were drilled with a CME 75 truck-mounted drilling rig utilizing 3¼-inch inside diameter hollow-stem augers. During auger advancement, split-spoon drive samples were generally taken continuously to 6 feet below bottom of existing pavement, at 2½-foot intervals to a depth of 30 feet and at 5-foot intervals thereafter. The samples were sealed in jars and transported to our laboratory for further classification and testing.

Two (2) pavement cores were obtained in Borings B-003 and B-004 using a 4-inch diameter single-wall, diamond-tipped core barrel. After pavement coring was completed, a determination was made of the underlying aggregate base thickness. The recovered cores were photographed and retained by TTL.

Split-spoon (SS) soil samples were obtained by the Standard Penetration Test Method (ASTM D 1586). The Standard Penetration Test (SPT) consists of driving a 2-inch outside diameter split-spoon sampler into the soil with a 140-pound weight falling freely through a distance of 30 inches. The sampler was driven in three successive 6-inch increments, with the number of blows per increment being recorded. 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 $N_m$-value, and is typically reported in blows per foot (bpf). The $N_m$-values were corrected to an equivalent rod energy ratio of 60 percent, $N_{60}$. The calibrated hammer/rod energy ratio for the CME 75 truck-mounted drill rig utilized in this project was 66.0 percent, based on calibration on March 15, 2021. The $N_{60}$-values are presented on the attached Logs of Test Borings.

Two Shelby tube samples, designated ST on the Logs of Test Borings, were obtained from Borings B-003 (8 to 10 feet) and B-004 (16 to 18 feet). The Shelby tube samples were obtained by hydraulically advancing a 3-inch diameter, thin-walled sampler approximately 24 inches beyond the hollow-stem auger into undisturbed soil, in accordance with ASTM D 1587. The

Shelby tubes were then extracted from the subsoils, and the ends were capped and sealed. The samples were transported to our laboratory where they were extruded, classified, and tested.

Core samples of the bedrock were obtained from Borings B-003 and B-004, using an NQ2 diamond-bit core barrel and coring techniques in general accordance with ASTM D 2113. In each boring, one core run of five feet was completed immediately following auger refusal. Recovery of the core is expressed as the percentage ratio of the recovered rock length to the total length of the core run. The Rock Quality Designation (RQD) is the percentage ratio of the summed length of rock pieces 4 inches long and greater to the total length of the run. The rock core samples are designated as “NQ2” on the Logs of Test Borings. The recovered rock cores were visually classified using the ODOT Rock Classification System. The rock cores were also documented in a photographic core log, which is attached to this report.

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

### 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). A dry density determination and an unconfined compressive strength test by the constant rate of strain method (ASTM D 2166) were performed on the recovered Shelby tube samples. Unconfined compressive strength estimates were obtained for the remaining intact cohesive samples using a calibrated hand penetrometer. Atterberg limits tests (ASTM D 4318) and particle size analyses (ASTM D 6913 & D 7928) were performed on selected samples to determine soil classification and index properties. These test results are presented on the Logs of Test Borings, Grain Size Distribution sheets, and Unconfined Compression Test sheets.

Compressive strength tests (ASTM D 7012, Method C) were performed for selected rock core specimens. The results of these tests are presented on the Logs of Test Borings and Compressive Strength of Rock sheets attached to this report.

# 4.0 FINDINGS

## 4.1 General Site Conditions

The site is located along SR 65 between SR 235 and Brillhart Road, in Washington Township. In the project area, grades along SR 65 were on the order of Elev. 632±. The waterway bottom is on the order of 13 to 15 feet below the top of pavement, roughly on the order of Elev. 619± to 617±.

The surface materials encountered in Borings B-003 and B-004 consisted of approximately 14 inches and 8½ inches of asphalt, respectively, underlain by approximately 17¼ inches and 13 inches of aggregate base, respectively.

## 4.2 General Soil Conditions

Based on the results of our field and laboratory tests, the subsoils encountered underlying the pavement materials can be generally described as predominantly cohesive soils overlying bedrock. However, a localized zone of granular soils was encountered in Boring B-003.

The encountered subsoils consisted of predominantly native cohesive soils encountered underlying the pavement materials to a depth of approximately 33½ feet below existing grade (Elev. 599±), in both borings (auger refusal on bedrock in Boring B-003 and top of weathered rock in Boring B-004). These soils consisted of sandy silt (A-4a) with trace amounts of gravel and clay, silt and clay (A-6a) with varying amounts of sand and gravel, as well as silty clay (A-6b) with some to little sand and varying amounts of gravel. Trace wood and/or organics were noted for the samples encountered at depths corresponding to the waterway bottom.

The upper portion of the cohesive soils exhibited generally stiff to very stiff consistency. This layer extended to a depth of 23 feet (Elev. 609±) in Boring B-003 and to a depth of 26½ feet (Elev. 606±) in Boring B-004. SPT $N_{60}$-values generally ranged from 7 to 23 bpf. Unconfined compressive strengths generally ranged from 2,000 to 5,500 psf. Moisture contents generally ranged from 13 to 29 percent. Within this upper portion of soils, zones exhibiting **soft** or hard consistency and a seam of granular soils were encountered as follows:

- A localized zone exhibiting **soft** consistency with an SPT $N_{60}$-value of 4 bpf and an unconfined compressive strength of 1,000 psf was encountered in Boring B-003 from approximately 13 to 15 feet. This layer was encountered at a depth corresponding to the waterway bottom.

- Localized zones exhibiting hard consistency with an SPT $N_{60}$-value of 30 bpf along with unconfined compressive strengths greater than 9,000 psf (the highest obtainable strength using a hand penetrometer) were encountered in Boring B-004 from 4 to 6 feet and from 21½ to 23½ feet.

- A localized zone of **loose** granular native soils were encountered in Boring B-003 from 15 to 18 feet. These soils consisted of coarse and fine sand (A-3a) mixed with some clay and little silt. An SPT $N_{60}$-value of 8 bpf and a moisture content of 28 percent were determined for the sample obtained in this zone.

The lower portion of the cohesive soil profile exhibited hard consistency. Within this layer, SPT $N_{60}$-values varied from 28 to 46 bpf. Unconfined compressive strengths were greater than 9,000 psf. Moisture contents ranged from 13 to 15 percent.

Weathered shale that was able to be penetrated with augers was encountered underlying the hard cohesive soils in Boring B-004 to auger refusal at a depth of approximately 34½ feet (Elev. 598±).

Upon encountering auger refusal in Borings B-003 and B-004, the rock was cored using a 5-foot rock core run. The cored bedrock consisted of slightly to moderately weathered dolomite. The recovered material represented 100 percent and 98 percent of the core runs in Borings B-003 and B-004, respectively. RQD values of 93 percent and 95 percent were determined for the cores recovered from Borings B-003 and B-004, respectively. A photographic log of the recovered rock from each core is attached to this report. Compressive strengths for intact specimens of the cores ranged from 16,360 to 16,560 pounds per square inch (psi).

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

### 4.3 Groundwater Conditions

During our site reconnaissance on October 19, 2021, water levels in the waterway were approximately 1 to 2 feet deep. During our site reconnaissance on October 28, 2021, shortly after a significant rain in the area, the water levels rose by approximately 3 to 4 feet. The waterway bottom was approximately 13 to 15 feet below the road surface. Hence, based on our field observations, water was approximately 12 to 13 feet below the road surface (Elevs. 620± to 619±) during the more normal/non-rain-influenced observation date of October 19, 2021.

Groundwater was initially encountered during drilling at a depth of 18 feet (Elev. 614±) in Boring B-003 and at a depth of 17 feet (Elev. 615±) in Boring B-004. Water was noted upon completion of drilling and rock coring operations at a depth of 21½ feet (Elev. 610±) in Boring B-001 and at a depth of 13 feet (Elev. 619±) in Boring B-004. However, these water levels were affected by water introduced during rock coring. It should be noted that the boreholes were drilled and sealed within the same day, and stabilized water levels may not have occurred over this limited time period.

Apart from streamflow influences in the waterway and nearby Williamsburg Reservoir, it is our opinion that the "normal" groundwater level can generally be expected at depths corresponding to the water elevation in the Maumee River. Based on google earth, the water elevation in the Maumee River in the vicinity of the project site is at approximate Elev 619± corresponding to approximately 13 to 13½ feet below existing grades along SR-65. However, groundwater elevations can fluctuate with seasonal and climatic influences, will also be particularly affected locally by water levels in the waterway, reservoir, and the nearby Maumee River. Therefore, groundwater conditions may vary at different times of the year from those encountered during this exploration.

### 4.4 Remedial Measures

It should be noted that the values for factored unit tip resistance of socketed drilled shafts listed in Section 5.1.1 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. **Since the replacement structure was originally planned as a culvert, only 5 feet of rock coring was performed in each boring. As such, the cored rock only extends approximately ½ socket diameter and 1 socket diameter below the anticipated end-bearing elevations at the Rear Abutment and Forward Abutment, respectively. It was indicated by TTL that, if the risk associated with non-cored rock within 2 socket diameters of the end-bearing elevation is not acceptable, additional rock coring would need to be performed. Subsequently, it was indicated by ODOT District 2 that the 5-foot rock core runs were suitable for this project.**

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

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

The GB-1 “Subgrade Analysis” worksheet (V14.5, 01/18/19) indicates that over-excavation of unsuitable subgrade soils to a depth of 12 inches both east and west of the new culvert installation, and replacement with new granular engineered fill, are anticipated to be required based on the conditions encountered in Boring B-003 and B-004. Due to the limited extent of the required remediation, global stabilization is not anticipated to be economical.

# 5.0 ANALYSES AND RECOMMENDATIONS

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

## 5.1 Bridge Foundations

### 5.1.1 Drilled Shaft Rock Socket Vertical Resistance

We understand that the bridge foundations will be designed using LRFD methods. Based on the relatively shallow bedrock at the site and the potential for scour, we understand that the new bridge is planned to be supported by a deep foundation system consisting of drilled shafts socketed into bedrock. The maximum total factored load was indicated to be 365 kips. Recommendations are provided herein for the smallest diameter drilled shafts that may be constructed in accordance with the ODOT Bridge Design Manual (BDM).

The minimum diameter for drilled shafts that support pier columns is 42 inches, although there are no pier columns for this project. Drilled shafts that support abutments may be 36 inches in diameter. The diameter of bedrock sockets for drilled shafts are generally 6 inches less than the diameter of the shaft above the bedrock elevation. Regardless of shaft diameter, reinforcing steel cages should be based on the bedrock socket diameter.

For the abutments, we have evaluated a 36-inch diameter shafts above bedrock and a socket diameter of 30 inches. Based on the rock conditions encountered in Boring B-003, for the Rear (Southwest) Abutment, an unfactored unit tip resistance ($q_p$) of 5,962 kips per square foot (ksf) was calculated. Based on the design methodologies utilized to evaluate unfactored unit tip resistance and AASHTO LRFD Table 10.5.5.2.4-1, a resistance factor of 0.50 should be utilized for design for tip resistance. As such, the factored unit tip resistance was calculated to be 2,980 ksf. **Using the planned 30-inch diameter socket, this design value would provide sufficient resistance for the indicated factored vertical load.** The maximum factored load to be supported by the Rear Abutment drilled shafts is 365 kips. This load is resisted entirely by tip resistance. At the Rear Abutment, the factored tip resistance is 14,628 kips.

Based on the rock conditions encountered in Boring B-004 for the Forward (Northeast) Abutment, an unfactored unit tip resistance ($q_p$) of 5,890 kips per square foot (ksf) was calculated. Based on the design methodologies utilized to evaluate unfactored unit tip resistance and AASHTO LRFD Table 10.5.5.2.4-1, a resistance factor of 0.50 should be utilized for design for tip resistance. As such, the factored unit tip resistance was calculated to be 2,945 ksf. **Using the planned 30-inch diameter socket, this design value would provide sufficient resistance for the indicated factored vertical load.** The maximum factored load to be supported by the Forward Abutment drilled shafts is 365 kips. This load is resisted entirely by tip resistance. At the Rear Abutment, the factored tip resistance is 14,456 kips.

It should be noted that the values for factored unit tip resistance listed above are based on bearing in competent rock that does not contain adverse jointing, open solution cavities, or joints that are filled with weathered material that would affect the bearing resistance of the rock, within a distance equal to two socket diameters below the tip of the drilled shaft rock socket. **Since the replacement structure was originally planned as a culvert, only 5 feet of rock coring was performed in each boring. As such, the cored rock only extends approximately ½ socket diameter and 1 socket diameter below the anticipated end-bearing elevations at the Rear Abutment and Forward Abutment, respectively. It was indicated by TTL that, if the risk associated with non-cored rock within 2 socket diameters of the end-bearing elevation is not acceptable, additional rock coring would need to be performed. Subsequently, it was indicated by ODOT District 2 that the 5-foot rock core runs were suitable for this project.**

The minimum prescribed rock socket length is 1.5B, where B is the socket diameter. For the socket diameter of 30 inches indicated above, the minimum socket length would be 3.75 feet for bearing elevations on the order of Elevs. 595±. It was indicated that potential scour extends to within approximately 5 feet above the top of bedrock. Based on this information, additional socket length is not required based on BDM 305.4.1.1 scour considerations. In any case, any structural requirement for the drilled shaft foundations to resist lateral loads or moments may increase the socket depth or diameter and should be evaluated on an individual shaft basis by Tetra Tech.

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

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

| Item | Rear (Southwest) Abutment (B-003) | Forward (Northeast) Abutment (B-004) |
|---|---|---|
| Minimum Rock Socket Length[(1)] (feet) | 3.75 | 3.75 |
| Top of Rock Elevation (feet) | 598.6 | 598.9 |
| Bottom of Rock Socket Elevation (feet) | 594.85 | 595.15 |

(1) Based on 1.5 times rock socket diameter of 30 inches.

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

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

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

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

Although cobbles or boulders were not noted in the borings performed for this exploration, they may be encountered at this site. Therefore, provisions should be made by the contractor to remove any obstructions, including 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 TTL 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 365 kips at the abutments. This load is resisted entirely by tip resistance. At the Rear

> Abutment and Forward Abutment, the factored tip resistance is 14,628 kips and 14,456 kips, respectively.
>
> Perform integrity testing on all of the drilled shafts at the abutments by Thermal Integrity Profiling (TIP). Perform TIP testing pre ASTM D7949, “Standard Test Methods for Thermal Integrity Profiling of Concrete Deep Foundations,” Method B, and per Supplemental Specification 894.

### 5.1.2 Lateral Load Soil and Rock Design Parameters

For lateral load-deflection evaluations using software, such as LPILE, recommended design parameters are provided in Attachment A of this report, based on the conditions encountered in the borings. For the portion of the drilled shaft below the groundwater table (estimated at or slightly above the water level in Maumee River (Elev. 620±), the effective unit weight must be considered (i.e., reduce the total unit weight by the unit weight of water, 62.4 pounds per cubic foot). These LPILE inputs are being provided for the structural engineer to evaluate a suitable, economical socket length and diameter, as well as to modify steel reinforcement conditions.

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

> LATERALLY LOADED DRILLED SHAFTS: The maximum factored lateral load and bending moment to be supported by each drilled shaft are * kips, and * kip-feet, respectively. These loads produce a maximum factored bending moment of * kip-feet, and a maximum factored shear of * kips, within the drilled shaft.
>
> * Complete the loads and dimensions in this note. If the maximum factored lateral loading of drilled shafts varies between substructure units, specify the drilled shaft groups and locations separately in the note.

## 5.2 Subgrades and Pavements

### 5.2.1 GB-1 “Plan Subgrades” Evaluation

An evaluation of the subgrade soils was completed in general accordance with ODOT Geotechnical Bulletin GB-1 “Plan Subgrades” (January 15, 2021). As part of this evaluation, the ODOT “Subgrade Analysis” worksheet (V14.5, 01/18/19) 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 approximately 2 to 2½ feet below the existing top of pavement grades (represented as a 2 to 2.5 feet cut in the ODOT "Subgrade Analysis" worksheet).

Based on GB-1, soils classified as ODOT A-4b, A-2-5, A-5, A-7-5, A-8a, A-8b, or rock have been designated as being problematic with respect to pavement subgrade support. None of these soil types were encountered at planned subgrade elevations in the borings performed for this exploration.

Based on GB-1 criteria, subgrade soils with moisture contents greater than 3 percent above optimum likely indicate the presence of unstable subgrade that may require some form of subgrade modification. Approximately 50 percent of the tested subgrade soil samples were greater than 3 percent above the optimum as determined using GB-1 criteria. Approximately 65 percent of the samples with moisture contents greater than 3 percent above optimum had moisture contents greater than or equal to 5 percent above optimum. Thus, where moisture contents were wet of optimum, they were appreciably wet of optimum. These data indicate that scarification and aeration methods may not be feasible to achieve satisfactory proof rolling and stabilization of the predominantly cohesive subgrades. However, scarification and aeration methods may be utilized if construction schedule will allow such soil modification.

The type and thickness of subgrade modification is determined by GB-1 criteria based on the average, low SPT $N_{60}$-value ($N_{60L}$) of the subgrade soils in a particular portion of the project area, hand penetrometer value, soil type, and moisture content. Based on these criteria, both borings (B-003 and B-004) contained subgrade soils which indicated subgrade modification is likely to be required. Subgrade modification for these borings was indicated by ODOT GB-1 to include planned undercutting of 12 inches of the existing subgrade and replacement with granular engineered fill.

Although ODOT GB-1 indicates that global cement stabilization to a depth of 14 inches could be considered, due to the relatively small project area, global chemical stabilization is not anticipated to be economical compared to over-excavation and replacement with granular engineered fill. In any case, sulfate content tests for tested subgrade samples were on the order of 370 parts per million (ppm) and 360 ppm, which would not preclude the use of global chemical stabilization. The GB-1 analysis spreadsheet indicates that rubblize and roll is not an option for this project.

It should be noted that GB-1 analyses are used as a pre-construction tool to plan subgrade modification alternatives. Actual subgrade modification will depend on field observations of proof-rolling conditions at the time of construction. Changes in soil moisture content could create more or less favorable subgrade conditions that may result in adjustments to subgrade modification or soil stabilization requirements at the time of construction.

### 5.2.2 Flexible (Asphalt) Pavement Design

Based on the GB-1 analysis, a design CBR value of 6 percent was determined for the project. It should be noted that the CBR determination by the GB-1 spreadsheet is based on the **average** Group Index of all the evaluated subgrade samples, which was 9. Group indices for all the tested samples ranged from 0 to 16, which would correlate with a CBR value of 4 to 12 percent. Cohesive subgrade soils classified as ODOT A-4a, A-6a and A-6b were predominantly present within the upper 6 feet of the subgrade elevation in both borings. The average group index for the tested A-4a, A-6a and A-6b samples was also 9. Based on the average design value calculations from GB-1, it does not appear to be unconservative to use the GB-1 design CBR value of 6 percent for new pavement sections throughout the project area.

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) or verified as stable through proof-rolling in accordance with Section 5.3.2 of this report.

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

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

## 5.3 Construction

### 5.3.1 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.

### 5.3.2 Site and subgrade Preparation

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

Upon completion of the clearing and undercutting activities, all areas that are to receive fill, or that have been excavated to proposed final subgrade elevation, should be inspected by a geotechnical engineer. Pavement subgrades should be proof rolled in accordance with ODOT CMS 204.06.

Any unsuitable materials observed during the inspection and proof-rolling operations should be undercut and replaced with compacted fill, or stabilized in place utilizing conventional remedial measures such as discing, aeration, and recompaction. As stated previously, based on the conditions encountered during our exploration, where subgrade soil moisture contents were wet of optimum, they were significantly wet of optimum. As such, scarification and aeration methods may not be feasible to achieve satisfactory proof rolling and stabilization of the predominantly cohesive subgrades. However, scarification and aeration methods may be utilized if areas where granular subgrades wet of optimum are present, provided weather conditions and construction schedule will allow such soil modification.

The GB-1 analysis indicates options for global chemical stabilization using cement to a depth of 14 inches, or planned over-excavation of unsuitable subgrade soils to a depth of 12 inches and replacement with new granular engineered fill for the entire extent of the project. Due to the relatively small project area, global chemical stabilization is not anticipated to be economical compared to over-excavation and replacement with granular engineered fill.

### 5.3.3 Temporary Excavations and Permanent Slopes

The sides of the temporary excavations for bridge/abutment installation should be adequately sloped to provide stable sides and safe working conditions. Otherwise, the excavation must be properly braced against lateral movements. In any case, applicable Occupational Safety and Health Administration (OSHA) standards must be followed. It is the responsibility of the installation contractor to develop appropriate installation methods and specify pertinent equipment prior to commencement of work, and to obtain the services of a geotechnical engineer to design or approve sloped or benched excavations and/or lateral bracing systems as required by OSHA criteria.

Although the encountered cohesive soils and anticipated "normal" groundwater level below the anticipated extents of abutment installation excavation elevations should be generally conducive to stable excavation slopes, provisions should be made for the abutment construction to proceed as a sloped-bank excavation, or as a steeper trench-type cut with properly designed and installed lateral bracing.

If the excavation is to be performed with sloped banks, adequate stable slopes must be provided in accordance with OSHA criteria. Based on the test borings, it is likely that excavations will encounter a range of soil conditions that include the following OSHA designations:

- Type A soils (cohesive soils with unconfined compressive strengths of 3,000 pounds per square foot (psf) or greater) and
- Type B soils (cohesive soils with unconfined compressive strengths greater than 1,000 psf but less than 3,000 psf).

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

For permanent excavations and slopes, we recommend that grades generally be no steeper than 3H:1V.

### 5.3.4 Construction Dewatering and Groundwater Control

Groundwater conditions encountered during our exploration are summarized in Section 4.3.

During construction, methods should be taken to divert the waterway flow around the construction area for any work in the channel.

Based on the soil characteristics and groundwater conditions encountered in the borings and apart from streamflow influences in the waterway and nearby reservoir, it is our opinion that the “normal” groundwater level can generally be expected at depths on the order of 13 to 13½ feet below existing roadway grades (Elevs. 619±).

If construction does not occur during a particularly wet period, adequate control of groundwater seepage into excavations should be achievable by minor dewatering systems, such as pumping from prepared sumps. Even at depths a few feet below the “normal” groundwater level, control of groundwater using sumps should be feasible due to the predominantly cohesive nature of the encountered soils and their associated low permeability, but will require due diligence by the contractor to maintain a stable subgrade condition at the bottom of the excavation. If granular soils are encountered below the water table, diligent dewatering will be required. Installation of multiple well points may be required in addition to pumping from prepared sumps. Additionally, sheetpile cutoff walls may be considered to extend through the granular soils into the underlying clay soils for groundwater management.

Based on the location of the proposed excavation relative to the waterway, it is likely that the headwall foundation excavations will encounter saturated subgrade conditions including groundwater seepage. In addition to dewatering measures, the contractor may need to incorporate a thin mat of lean concrete over the bottom of the excavation to avoid loss of subgrade strength and excessive undercutting of the bearing soils from groundwater seepage or surface run off.

### 5.3.5 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 of predominantly native cohesive soils. For the cohesive soils, a sheepsfoot roller should provide the most effective soil compaction. Where existing pavement base materials remain or new dense-graded aggregate pavement base materials are placed, 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 foundations 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, TTL 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.

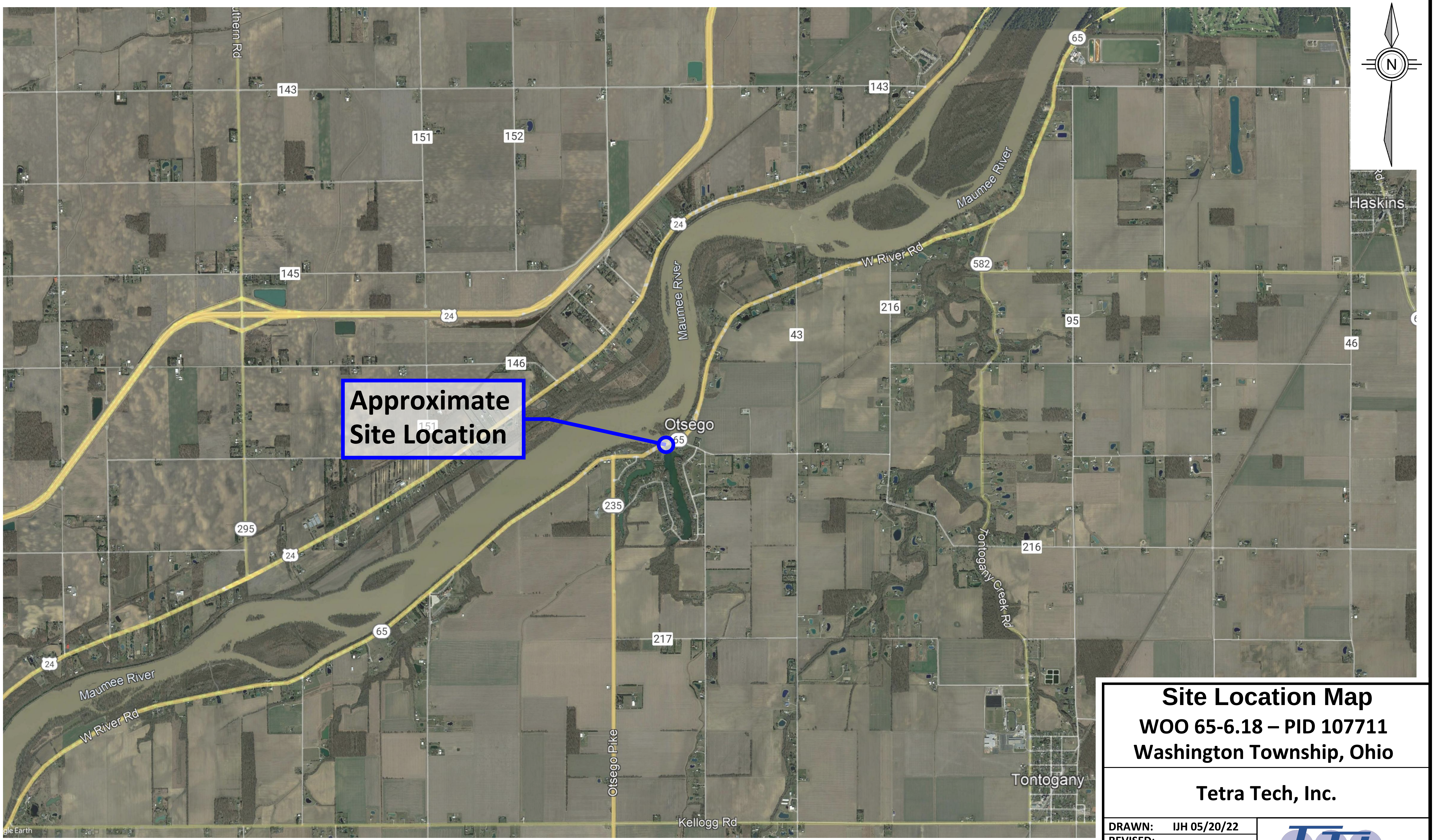
Our professional services have been performed, our findings derived, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices. This warranty is in lieu of all other warranties either expressed or implied. TTL is not responsible for the conclusions,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

# Plates

| | |
|---|---|
| **Plate 1.0** | **Site Location Map** |
| **Plate 2.0** | **Test Boring Location Plan** |

# Plates

| | |
|---|---|
| Plate 1.0 | Site Location Map |
| Plate 2.0 | Test Boring Location Plan |



**Notes:** Aerial Basemap obtained From Google Earth and dated 03/20/2021.

## Site Location Map

**WOO 65-6.18 – PID 107711**

**Washington Township, Ohio**

**Tetra Tech, Inc.**

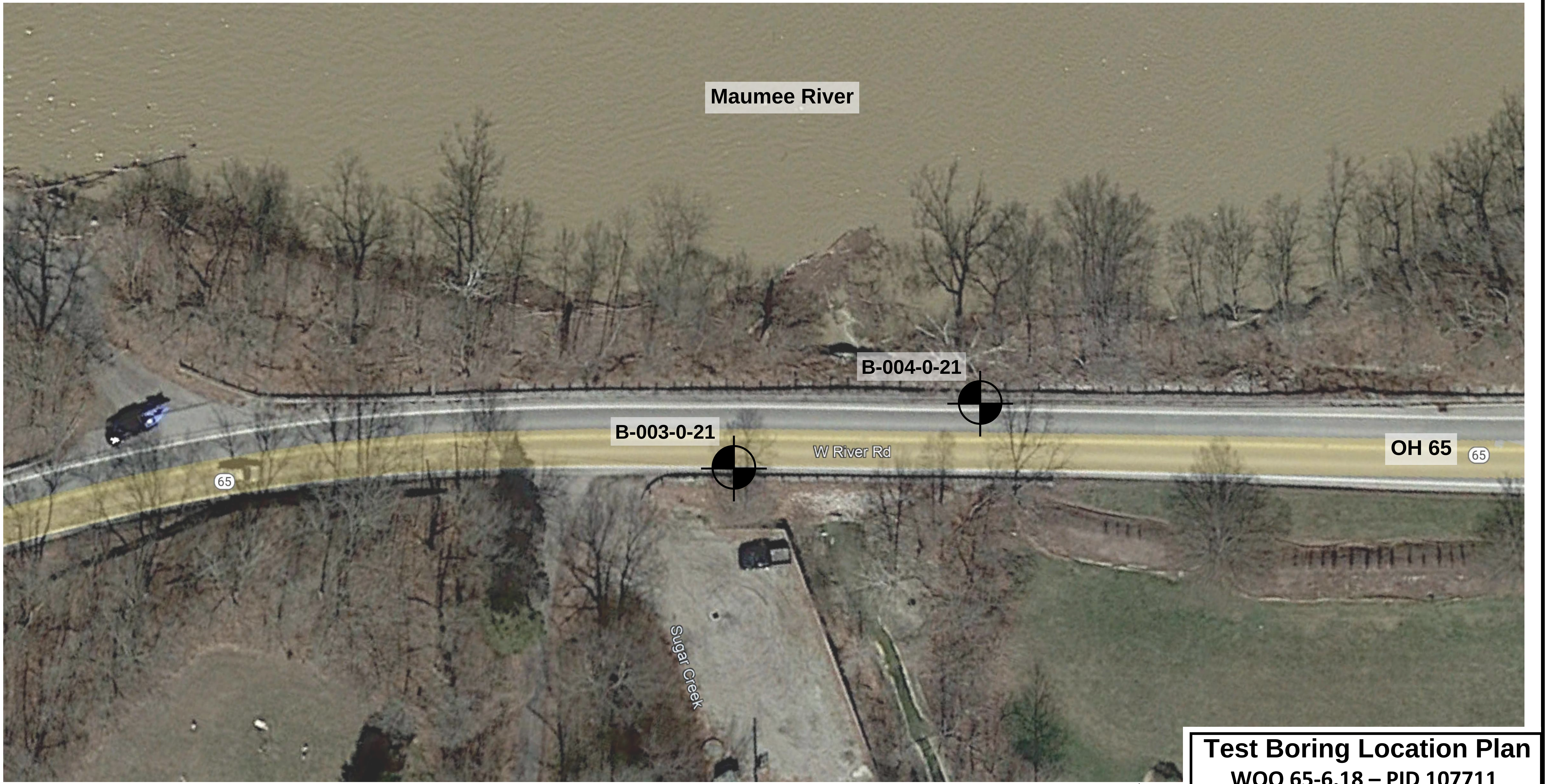
**DRAWN:** IJH 05/20/22

**REVISED:** ---

**Project No.:** 2130501

**Drawing No.:**
**Plate 1.0**

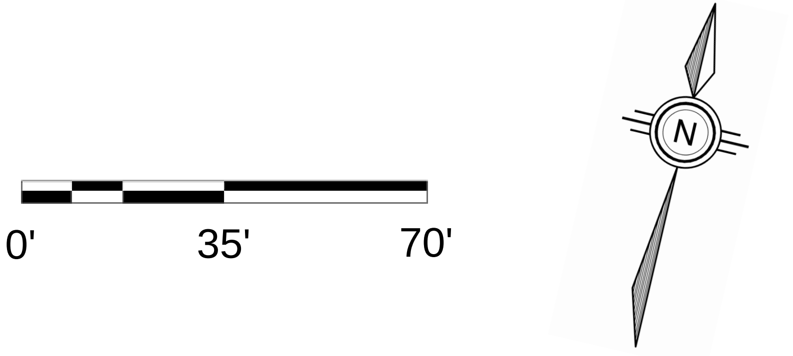
TTL associates inc
Environmental, Geotechnical
Engineering & Testing



**Legend:**

**B-003-0-21**

Approximate Pavement Core/
Test Boring Location



**Notes:** Aerial Basemap obtained From Google Earth and dated 03/20/2021.

# Test Boring Location Plan

## WOO 65-6.18 – PID 107711
## Washington Township, Ohio

**Tetra Tech, Inc.**

DRAWN: IJH 05/20/22
REVISED: ---
Project No.: 2130501
Drawing No.: Plate 2.0

TTL associates inc
Environmental, Geotechnical
Engineering & Testing

# Figures

Logs of Test Borings
Legend Key
Grain Size Distribution Curves
Undisturbed Sample Unconfined Compressive Strength Test Results
Pavement Core Photographic Logs
Rock Core Photographic Logs
Rock Core Unconfined Compressive Strength Test Results

| | | | | |
|---|---|---|---|---|
| PROJECT: WOO-65-06.18<br>TYPE: CULVERT<br>PID: 107711 SFN:<br>START: 10/20/21 END: 10/20/21 | DRILLING FIRM / OPERATOR: TTL / TB<br>SAMPLING FIRM / LOGGER: TTL / KKC<br>DRILLING METHOD: 3.25" HSA / NQ2<br>SAMPLING METHOD: SPT / ST / NQ2 | DRILL RIG: CME 75 TRUCK 844<br>HAMMER: CME AUTOMATIC<br>CALIBRATION DATE: 3/15/21<br>ENERGY RATIO (%): 66 | STATION / OFFSET: 326+12, 8' RT.<br>ALIGNMENT: CL R/W & CONST. SR 65<br>ELEVATION: 632.1 (NAVD88) EOB: 38.5 ft.<br>LAT / LONG: 41.455621, -83.781212 | EXPLORATION ID<br>B-003-0-21<br>PAGE<br>1 OF 2 |

| ***MATERIAL DESCRIPTION AND NOTES*** | ELEV. 632.1 | DEPTHS | SPT/ RQD | $N_{60}$ | REC (%) | SAMPLE ID | HP (tsf) | GRADATION (%) GR | CS | FS | SI | CL | ATTERBERG LL | PL | PI | WC | ODOT CLASS (GI) | SO4 ppm | HOLE SEALED |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ASPHALT - 14 INCHES | 630.9 | | | | | | | | | | | | | | | | | | |
| AGGREGATE BASE - 17.25 INCHES | 629.5 | | 10 8 5 | 14 | 67 | SS-1 | - | 66 | 6 | 7 | 16 | 5 | 30 | 15 | 15 | 3 | A-2-6 (0) | 370 | |
| STIFF, BROWN, **SILT AND CLAY**, SOME SAND, TRACE GRAVEL, MOIST | | | 3 4 4 | 9 | 89 | SS-2 | 1.75 | - | - | - | - | - | - | - | - | 21 | A-6a (V) | - | |
| @4': LITTLE SAND | 626.6 | | 3 4 3 | 8 | 100 | SS-3 | 1.50 | 6 | 7 | 9 | 21 | 57 | 32 | 17 | 15 | 18 | A-6a (10) | - | |
| STIFF TO VERY STIFF, BROWN, **SILTY CLAY**, SOME SAND, TRACE GRAVEL, MOIST | | | 6 6 7 | 14 | 100 | SS-4 | 2.50 | - | - | - | - | - | - | - | - | 17 | A-6b (V) | - | |
| @8': LITTLE SAND, Qu = 15.7 PSI = 2260 PSF | | | | | 100 | ST-5 | 2.75 | 2 | 6 | 11 | 21 | 60 | 35 | 19 | 16 | 17 | A-6b (10) | - | |
| @11': SOME SAND | 619.3 | | 4 6 7 | 14 | 100 | SS-6 | 2.75 | - | - | - | - | - | - | - | - | 19 | A-6b (V) | - | |
| SOFT, GRAY, **SILT AND CLAY**, LITTLE SAND, TRACE WOOD, MOIST TO WET | 617.1 | | 1 2 2 | 4 | 100 | SS-7 | 0.50 | - | - | - | - | - | - | - | - | 29 | A-6a (V) | - | |
| LOOSE, GRAY, **COARSE AND FINE SAND**, SOME CLAY, LITLTE SILT, WET | 614.1 | W 614.1 | 2 3 4 | 8 | 100 | SS-8 | - | - | - | - | - | - | - | - | - | 28 | A-3a (V) | - | |
| VERY STIFF, BROWN, **SILT AND CLAY**, LITTLE SAND, TRACE GRAVEL, MOIST | | | 3 5 10 | 17 | 100 | SS-9 | 2.75 | - | - | - | - | - | - | - | - | 26 | A-6a (V) | - | |

| PID: 107711 | SFN: | PROJECT: WOO-65-06.18 | STATION / OFFSET: 326+12, 8' RT. | START: 10/20/21 | END: 10/20/21 | PG 2 OF 2 | B-003-0-21 |
|---|---|---|---|---|---|---|---|

| *MATERIAL DESCRIPTION AND NOTES* | ELEV. 611.1 | DEPTHS | SPT/ RQD | $N_{60}$ | REC (%) | SAMPLE ID | HP (tsf) | GRADATION (%) GR | CS | FS | SI | CL | ATTERBERG LL | PL | PI | WC | ODOT CLASS (GI) | SO4 ppm | HOLE SEALED |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 610.6 | ▼ | | | | | | | | | | | | | | | | | |
| VERY STIFF, GRAY, **SILTY CLAY**, LITTLE SAND, MOIST | 609.1 | 22 | 7<br>10<br>11 | 23 | 100 | SS-10 | 2.50 | - | - | - | - | - | - | - | - | 21 | A-6b (V) | - | |
| HARD, GRAY, **SILT AND CLAY**, LITTLE SAND, TRACE GRAVEL, MOIST | | 24 | 4<br>10<br>15 | 28 | 100 | SS-11 | 4.50 | 5 | 6 | 8 | 20 | 61 | 29 | 15 | 14 | 14 | A-6a (10) | - | |
| | | 27 | 10<br>16<br>20 | 40 | 100 | SS-12 | 4.50 | - | - | - | - | - | - | - | - | 14 | A-6a (V) | - | |
| | 598.6 | 29 | 10<br>12<br>15 | 30 | 100 | SS-13 | 4.50 | - | - | - | - | - | - | - | - | 15 | A-6a (V) | - | |
| **DOLOMITE**, LIGHT GRAY, SLIGHTLY TO MODERATELY WEATHERED, VERY STRONG, VERY FINE GRAINED, VERY THIN TO THIN BEDDED, JOINTED - SLIGHTLY TO MODERATELY FRACTURED, TIGHT. DOLOMITE, Qu = 16560 PSI<br>@ 35.2': 1 TO 2 INCH HIGHLY FRACTURED SEAM | 593.6 | TR<br>36<br>EOB | 93 | | 100 | NQ2 | | | | | | | | | | 0 | CORE | | |

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; PUMPED 11 CF BENTONITE GROUT

PROJECT: WOO-65-06.18
TYPE: CULVERT
PID: 107711 SFN:
START: 10/22/21 END: 10/22/21

DRILLING FIRM / OPERATOR: TTL / TB
SAMPLING FIRM / LOGGER: TTL / KKC
DRILLING METHOD: 3.25" HSA / NQ2
SAMPLING METHOD: SPT / ST / NQ2

DRILL RIG: CME 75 TRUCK 844
HAMMER: CME AUTOMATIC
CALIBRATION DATE: 3/15/21
ENERGY RATIO (%): 66

STATION / OFFSET: 326+45, 6' LT.
ALIGNMENT: CL R/W & CONST. SR 65
ELEVATION: 632.4 (NAVD88) EOB: 39.3 ft.
LAT / LONG: 41.455682, -83.781111

EXPLORATION ID
B-004-0-21

PAGE
1 OF 2

| ***MATERIAL DESCRIPTION AND NOTES*** | ELEV. 632.4 | DEPTHS | SPT/ RQD | $N_{60}$ | REC (%) | SAMPLE ID | HP (tsf) | GRADATION (%) GR | CS | FS | SI | CL | ATTERBERG LL | PL | PI | WC | ODOT CLASS (GI) | SO4 ppm | HOLE SEALED |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ASPHALT - 8.5 INCHES | 631.7 | | | | | | | | | | | | | | | | | | |
| AGGREGATE BASE - 13 INCHES | 630.6 | | | | | | | | | | | | | | | | | | |
| STIFF, BROWN, **SILT AND CLAY**, SOME GRAVEL, LITTLE SAND, MOIST | | | 7<br>9<br>7 | 18 | 89 | SS-1 | - | - | - | - | - | - | - | - | - | 4 | A-1-b (V) | 360 | |
| | 628.4 | | 7<br>5<br>5 | 11 | 22 | SS-2 | - | 23 | 9 | 7 | 16 | 45 | 31 | 16 | 15 | 16 | A-6a (7) | - | |
| HARD, BROWN, **SANDY SILT**, TRACE CLAY, TRACE GRAVEL, MOIST | 626.6 | | 3<br>5<br>5 | 11 | 78 | SS-3 | 4.50 | - | - | - | - | - | - | - | - | 16 | A-4a (V) | - | |
| VERY STIFF, BROWN, **SILT AND CLAY**, LITTLE SAND, TRACE GRAVEL, MOIST | | | 6<br>7<br>7 | 15 | 100 | SS-4 | 2.50 | 9 | 7 | 12 | 21 | 51 | 30 | 16 | 14 | 16 | A-6a (9) | - | |
| | | | 3<br>9<br>5 | 15 | 100 | SS-5 | 2.25 | - | - | - | - | - | - | - | - | 19 | A-6a (V) | - | |
| | | | 5<br>7<br>10 | 19 | 100 | SS-6 | 3.25 | - | - | - | - | - | - | - | - | 15 | A-6a (V) | - | |
| | 619.4 | ▼ | | | | | | | | | | | | | | | | | |
| STIFF, GRAY, **SILTY CLAY**, LITTLE SAND, TRACE ORGANICS, MOIST | | | 2<br>3<br>3 | 7 | 100 | SS-7 | 1.00 | - | - | - | - | - | - | - | - | 29 | A-6b (V) | - | |
| @17': BROWN/GRAY, TRACE GRAVEL, Qu = 22.8 PSI = 3283 PSF | | W 615.4 | | | 100 | ST-8 | - | 3 | 5 | 6 | 20 | 66 | 39 | 17 | 22 | 19 | A-6b (13) | - | |
| | | | 4<br>6<br>8 | 15 | 100 | SS-9 | 2.50 | - | - | - | - | - | - | - | - | 18 | A-6b (V) | - | |

| PID: 107711 | SFN: | PROJECT: WOO-65-06.18 | STATION / OFFSET: 326+45, 6' LT. | START: 10/22/21 | END: 10/22/21 | PG 2 OF 2 | B-004-0-21 |
|---|---|---|---|---|---|---|---|

| MATERIAL DESCRIPTION AND NOTES | ELEV. 611.4 | DEPTHS | SPT/ RQD | N60 | REC (%) | SAMPLE ID | HP (tsf) | GRADATION (%) GR | CS | FS | SI | CL | ATTERBERG LL | PL | PI | WC | ODOT CLASS (GI) | SO4 ppm | HOLE SEALED |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 610.9 | | | | | | | | | | | | | | | | | | |
| HARD, GRAY, **SANDY SILT**, LITTLE GRAVEL, TRACE CLAY, MOIST | 608.9 | 22<br>23 | 9<br>14<br>13 | 30 | 100 | SS-10 | 4.50 | - | - | - | - | - | - | - | - | 15 | A-4a (V) | - | |
| VERY STIFF, GRAY, **SILT AND CLAY**, LITTLE SAND, TRACE GRAVEL, MOIST | 605.9 | 24<br>25<br>26 | 5<br>4<br>9 | 14 | 100 | SS-11 | 2.50 | - | - | - | - | - | - | - | - | 13 | A-6a (V) | - | |
| HARD, GRAY, **SILTY CLAY**, SOME SAND, TRACE GRAVEL, MOIST | | 27<br>28 | 15<br>16<br>26 | 46 | 100 | SS-12 | 4.50 | - | - | - | - | - | - | - | - | 13 | A-6b (V) | - | |
| | 598.9 | 29<br>30<br>31<br>32<br>33 | 13<br>15<br>17 | 35 | 100 | SS-13 | 4.50 | 4 | 6 | 15 | 24 | 51 | 30 | 13 | 17 | 14 | A-6b (11) | - | |
| GRAY, **WEATHERED DOLOMITE WITH SAND**, SILT, **AND CLAY** | 598.1 | TR | 50/3" | - | 100 | SS-14 | - | - | - | - | - | - | - | - | - | 8 | A-2-6 (V) | - | |
| **DOLOMITE**, LIGHT GRAY, SLIGHTLY TO MODERATELY WEATHERED, VERY STRONG, VERY FINE GRAINED, VERY THIN TO THIN BEDDED, JOINTED - MODERATELY FRACTURED, TIGHT. @34.3': Qu = 16360 PSI<br>@ 35.2': SLIGHLTY FRACTURED | 593.1 | 34<br>35<br>36<br>37<br>38<br>39<br>EOB | 95 | | 98 | NQ2 | | | | | | | | | | 0 | CORE | | |

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; PUMPED 11 CF BENTONITE GROUT

# LEGEND KEY

## Ohio Department of Transportation Soil Symbols

|  |  |  |  |
|---|---|---|---|
| ▨ A-1-a - Gravel and/or Stone Fragments | ▨ A-1-b - Gravel and/or Stone Fragments with Sand | ▨ A-2-4,A-2-5 - Gravel and/or Stone Fragments with Sand and Silt | ▨ A-2-6,A-2-7 - Gravel and/or Stone Fragments with Sand, Silt and Clay |
| ▨ A-3 - Fine Sand | ▨ A-3a - Coarse and Fine Sand | ▥ A-4a - Sandy Silt | ▨ A-4b - Silt |
| ▨ A-5 - Elastic Silt and Clay | ▨ A-6a - Silt and Clay | ▤ A-6b - Silty Clay | ▨ A-7-5 - Elastic Clay |
| ▦ A-7-6 - Clay | ▨ A-8a - Organic Silt | ▨ A-8b - Organic Clay | ▩ Asphalt |
| ▧ Sod and/or Topsoil | ▨ Concrete | ▨ Random Fill | ■ Peat |
| ▨ Dolomite | ▨ Weathered Dolomite | ▨ Limestone | ▨ Weathered Limestone |
| ▨ Sandstone | ▨ Weathered Sandstone | ▤ Shale | ▨ Weathered Shale |

## Sample Symbols

|  |  |  |  |
|---|---|---|---|
| ☒ SS - Split Spoon | ■ ST - Shelby Tube | ▯ RC - Rock Core | ▯ GS - Geoprobe Sleeve |
|  | ▯ AU - Auger Cuttings | ▯ GB - Grab |  |

Notes:

1. Exploratory borings B-003-0-21 and B-004-0-21 were drilled on October 20 and 22, 2021, using 3¼-inch diameter hollow-stem augers. Pavement cores were obtained using a 4-inch inside diameter thin-wall core bit. Upon encountering auger refusal, a rock core run was performed using an NQ2 diamond-bit core barrel.

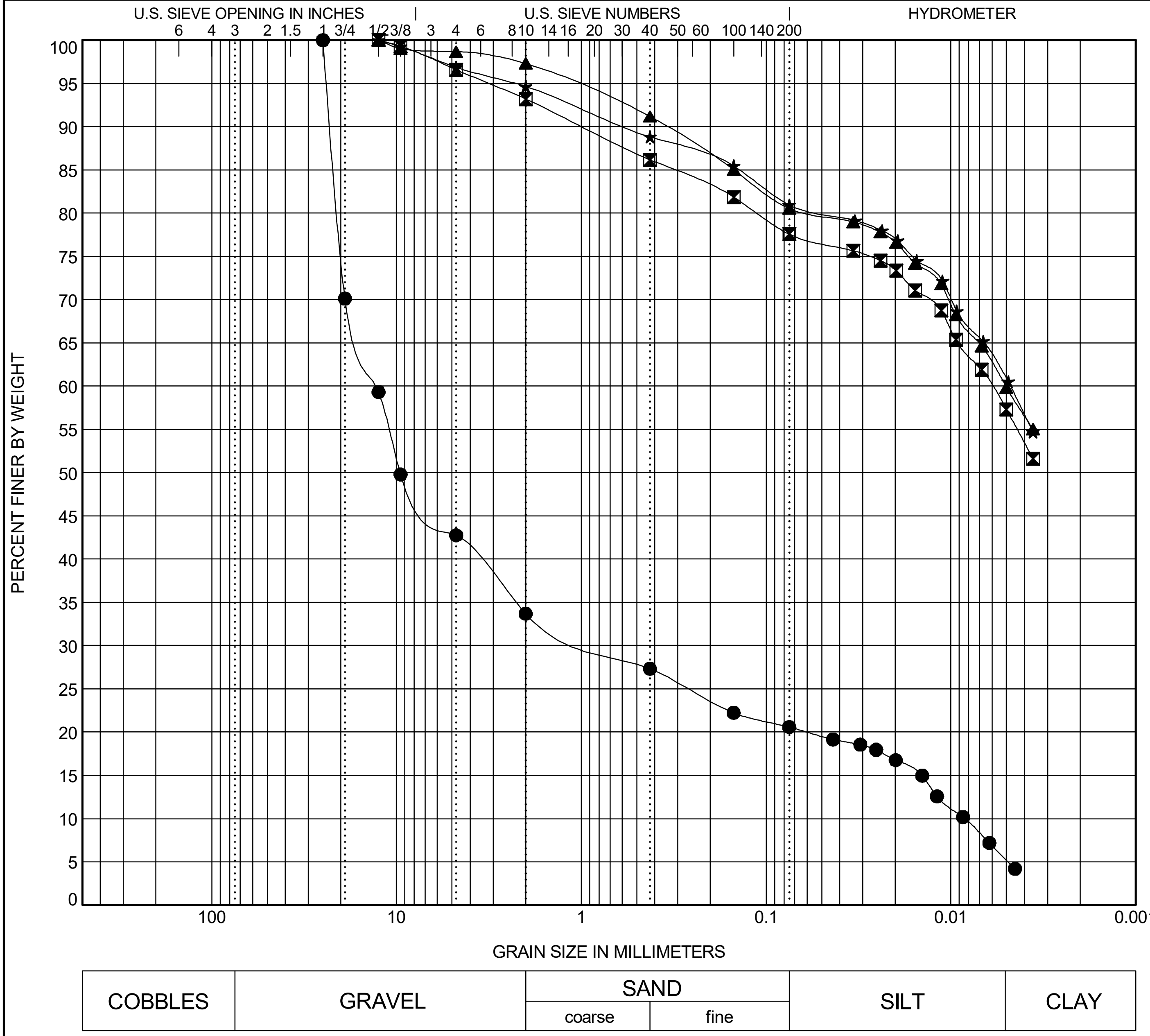
2. These logs are subject to the limitations, conclusions, and recommendations in the report and should not be interpreted separate from the report.

3. The borings were located in the field by TTL in accordance with the Proposed Boring Location Plan, attached to the proposal via a hand-held GPS. Stationing and offsets, were provided by Tera Tech, Inc. Latitude, Longitude, and ground surface elevations were surveyed by TTL via a hand-held GPS. The accuracy from the handheld GPS device was generally found to be approximately 2 to 6 inches horizontal, and approximately 4 to 12 inches vertical.

# GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

**PROJECT** WOO-65-06.18 **PID** 107711
**OGE NUMBER** N/A **PROJECT TYPE** STRUCTURE FOUNDATION



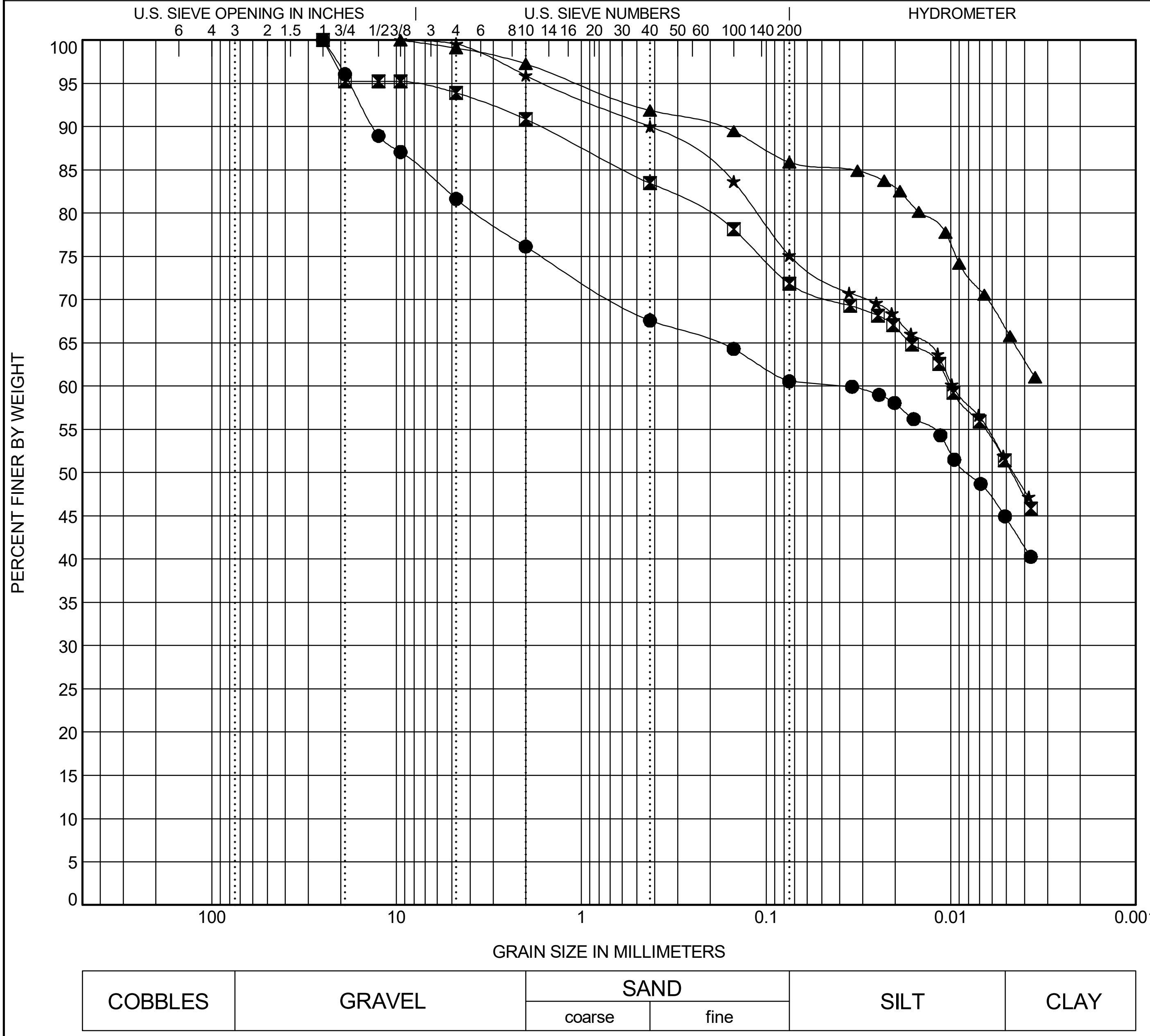
| Specimen Identification | | ODOT (Modified AASHTO) ~ USCS Classification | LL | PL | PI |
|---|---|---|---|---|---|
| ● **B-003-0-21** | **1.0** | **A-2-6 ~ CLAYEY GRAVEL with SAND(GC)** | **30** | **15** | **15** |
| ☒ **B-003-0-21** | **4.0** | **A-6a ~ LEAN CLAY with SAND(CL)** | **32** | **17** | **15** |
| ▲ **B-003-0-21** | **8.0** | **A-6b ~ LEAN CLAY with SAND(CL)** | **35** | **19** | **16** |
| ★ **B-003-0-21** | **23.5** | **A-6a ~ LEAN CLAY with SAND(CL)** | **29** | **15** | **14** |
| | | | | | |

| Specimen Identification | | D90 | D50 | D30 | D10 | %G | %CS | %FS | %M | %C | Cc | Cu |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● **B-003-0-21** | **1.0** | **22.806** | **9.559** | **0.815** | **0.008** | **66** | **6** | **7** | **16** | **5** | **6.14** | **1520.59** |
| ☒ **B-003-0-21** | **4.0** | **0.986** | | | | **6** | **7** | **9** | **21** | **57** | | |
| ▲ **B-003-0-21** | **8.0** | **0.348** | | | | **2** | **6** | **11** | **21** | **60** | | |
| ★ **B-003-0-21** | **23.5** | **0.581** | | | | **5** | **6** | **8** | **20** | **61** | | |
| | | | | | | | | | | | | |

# GRAIN SIZE DISTRIBUTION

**OHIO DEPARTMENT OF TRANSPORTION**
**OFFICE OF GEOTECHNICAL ENGINEERING**

**PROJECT** WOO-65-06.18
**PID** 107711
**OGE NUMBER** N/A
**PROJECT TYPE** STRUCTURE FOUNDATION



| Specimen Identification | | | ODOT (Modified AASHTO) ~ USCS Classification | LL | PL | PI |
|---|---|---|---|---|---|---|
| ● | **B-004-0-21** | **2.5** | **A-6a ~ SANDY LEAN CLAY with GRAVEL(CL)** | **31** | **16** | **15** |
| ⊠ | **B-004-0-21** | **5.5** | **A-6a ~ LEAN CLAY with SAND(CL)** | **30** | **16** | **14** |
| ▲ | **B-004-0-21** | **16.0** | **A-6b ~ LEAN CLAY(CL)** | **39** | **17** | **22** |
| ★ | **B-004-0-21** | **28.5** | **A-6b ~ LEAN CLAY with SAND(CL)** | **30** | **13** | **17** |
| | | | | | | |

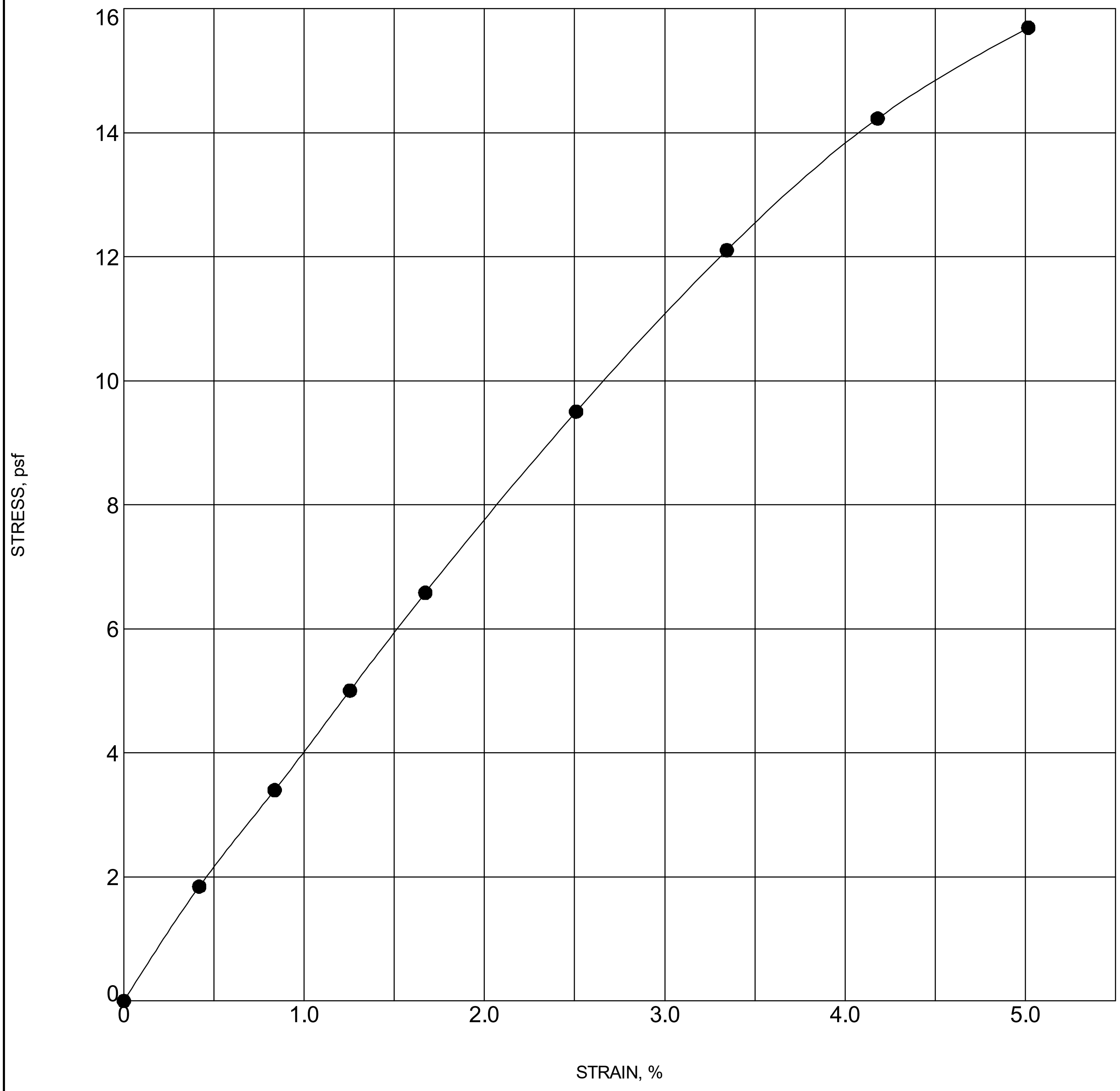
| Specimen Identification | | | D90 | D50 | D30 | D10 | %G | %CS | %FS | %M | %C | Cc | Cu |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | **B-004-0-21** | **2.5** | **13.301** | **0.008** | | | **23** | **9** | **7** | **16** | **45** | | |
| ⊠ | **B-004-0-21** | **5.5** | **1.663** | **0.005** | | | **9** | **7** | **12** | **21** | **51** | | |
| ▲ | **B-004-0-21** | **16.0** | **0.188** | | | | **3** | **5** | **6** | **20** | **66** | | |
| ★ | **B-004-0-21** | **28.5** | **0.423** | **0.005** | | | **4** | **6** | **15** | **24** | **51** | | |
| | | | | | | | | | | | | | |

# UNCONFINED COMPRESSION TEST

**OHIO DEPARTMENT OF TRANSPORTION**
**OFFICE OF GEOTECHNICAL ENGINEERING**

**PROJECT** WOO-65-06.18 **PID** 107711

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



| | Specimen Identification | | Classification | $\gamma_d$ | MC% |
|---|---|---|---|---|---|
| ● | **B-003-0-21** | **8.0** | **A-6b** | **117** | **17** |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |

# UNCONFINED COMPRESSION TEST

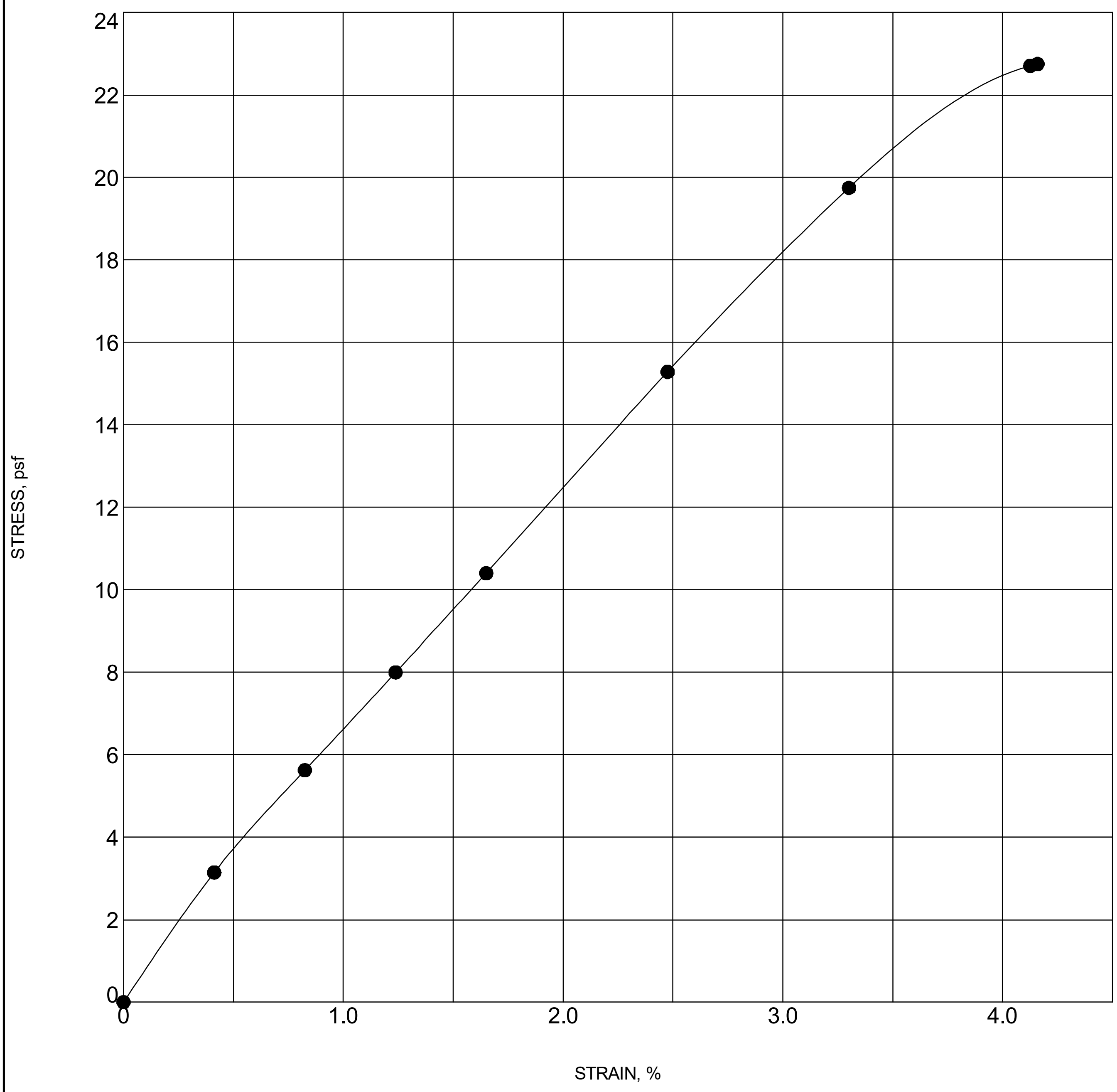
**OHIO DEPARTMENT OF TRANSPORTION**
**OFFICE OF GEOTECHNICAL ENGINEERING**

**PROJECT** WOO-65-06.18

**PID** 107711

**OGE NUMBER** N/A

**PROJECT TYPE** STRUCTURE FOUNDATION



| | Specimen Identification | | Classification | $\gamma_d$ | MC% |
|---|---|---|---|---|---|
| ● | **B-004-0-21** | **16.0** | **A-6b** | **116** | **19** |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |

# CORE LOG for B-003-0-21

Project: WOO 65-06.18 – PID 107711
Project Location: Washington Township, Ohio
TTL Project No. 2130501
Core Date: October 20, 2021

| | | |
|---|---|---|
| ASPHALT THICKNESS (in) | = | 14.0 |
| STONE THICKNESS (in) | = | 17.25 |
| CORE BARREL DIAMETER (in) | = | 4.0 |

**VISUAL DESCRIPTION:**

# CORE LOG for B-004-0-21

Project: WOO 65-06.18 – PID 107711
Project Location: Washington Township, Ohio
TTL Project No. 2130501
Core Date: October 22, 2021

| | | |
|---|---|---|
| ASPHALT THICKNESS (in) | = | 8.5 |
| STONE THICKNESS (in) | = | 13.0 |
| CORE BARREL DIAMETER (in) | = | 4.0 |

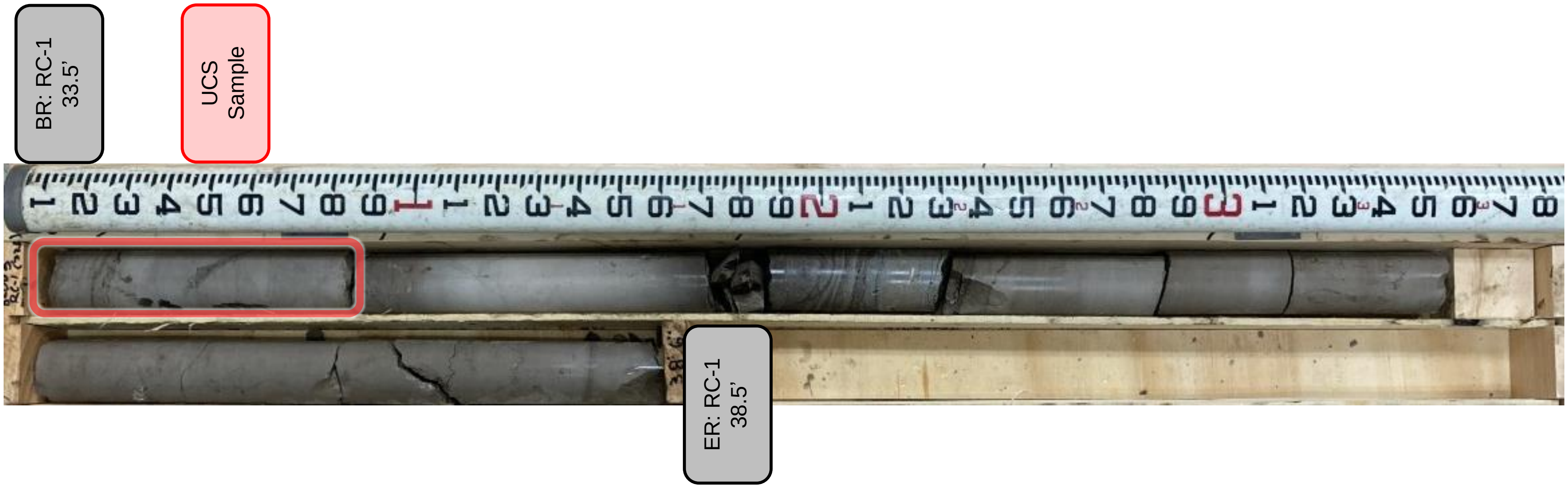
**VISUAL DESCRIPTION:**

Apparent delamination at approximately 5.5 Inches below top of pavement.

OHIO DEPARTMENT OF TRANSPORTATION
DIVISION OF ENGINEERING

Office of Geotechnical Engineering

# B-003-0-21



| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Core Date: October 20 , 2021 | | | | | Ground Surface Elevation: 632.1 feet (NAVD88) | | | |
| Run #: | Depth | | Elevation | | Recovery | | RQD | |
| RC-1 | 33.5’ | 38.5’ | 598.6 | 593.6 | 60/60 | 100% | 56/60 | 93% |
| WOO-65-6.18, PID 107711 | | | | | | | | |

Prepared by

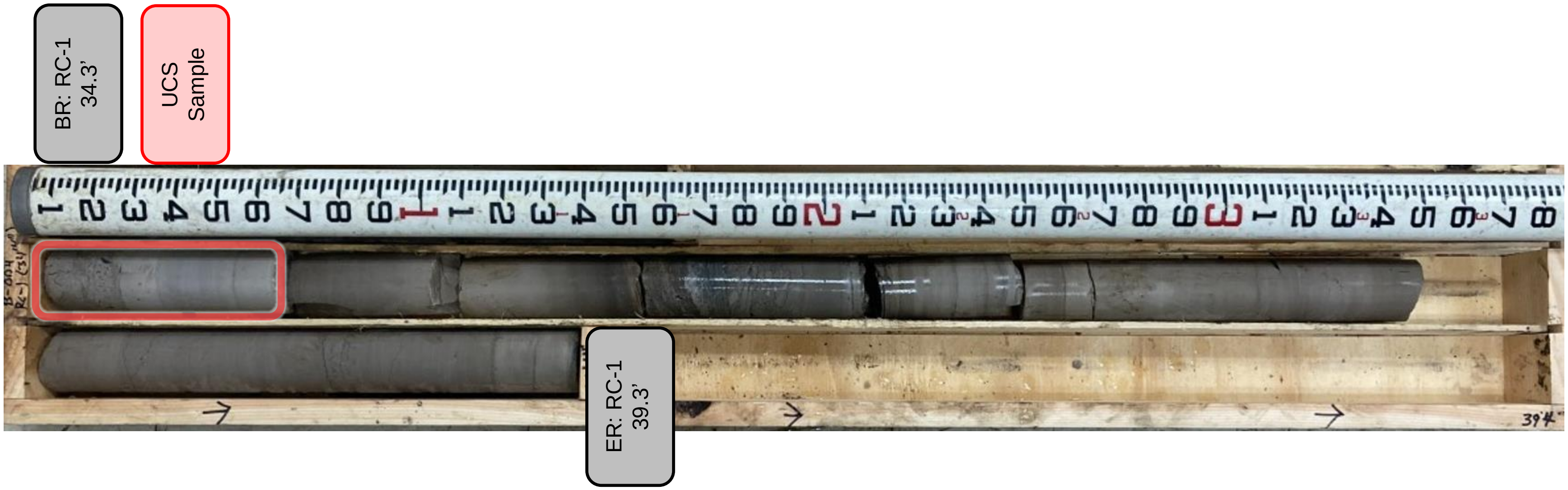
associates inc
Environmental, Geotechnical Engineering & Testing

TTL Project No.: 2130501

OHIO DEPARTMENT OF TRANSPORTATION
DIVISION OF ENGINEERING

Office of Geotechnical Engineering

# B-004-0-21



| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Core Date: October 22, 2021 | | | | | Ground Surface Elevation: 632.4 feet (NAVD88) | | | |
| Run #: | Depth | | Elevation | | Recovery | | RQD | |
| RC-1 | 34.3’ | 39.3’ | 598.1 | 593.1 | 59/60 | 98% | 57/60 | 95% |
| WOO-65-6.18, PID 107711 | | | | | | | | |

Prepared by TTL associates inc

TTL Project No.: 2130501

# Compressive Strength of Rock
# ASTM D 7012, Method C

| | | | |
|---|---|---|---|
| PROJECT | WOO-65-6.18, PID 107711 | TTL PROJECT NUMBER | 2130501 |
| LOCATION | Washington Township, Ohio | | |
| CLIENT | Tetra Tech | | |
| BORING NUMBER | B-003-0-21 | SAMPLE NUMBER | 3 (RC-1) |
| SAMPLE DEPTH (FEET) | 33.5 – 38.5 | SPECIMEN DEPTH (FEET) | 33.5 |

| | |
|---|---|
| ROCK DESCRIPTION | Dolomite, Light Gray, Slightly to Moderately Weathered, Very Strong, Very Fine Grained, Very Thin to Thin Bedded, Jointed - Slightly to Moderately Fractured, Tight. |

| | |
|---|---|
| LENGTH (INCHES) | 4.11 |
| DIAMETER (INCHES) | 1.99 |
| LENGTH / DIAMETER | 2.06 |
| CORRECTION FACTOR | 1.0 |
| AREA (SQ. IN.) | 3.11 |

| | |
|---|---|
| MASS (GRAMS) | 525.8 |
| UNIT WEIGHT (LBS/CU. FT.) | 156.7 |
| | |
| MAXIMUM LOAD (LBS) | 51.510 |
| COMPRESSIVE STRENGTH (PSI) | 16,560 |

TEST SPECIMEN PHOTO

TEST SPECIMEN PHOTO

# Compressive Strength of Rock
# ASTM D 7012, Method C

| | | | |
|---|---|---|---|
| PROJECT | WOO-65-6.18, PID 107711 | TTL PROJECT NUMBER | 2130501 |
| LOCATION | Washington Township, Ohio | | |
| CLIENT | Tetra Tech | | |
| BORING NUMBER | B-004-0-21 | SAMPLE NUMBER | 4 (RC-1) |
| SAMPLE DEPTH (FEET) | 34.3 – 39.3 | SPECIMEN DEPTH (FEET) | 34.3 |

| | |
|---|---|
| ROCK DESCRIPTION | Dolomite, Light Gray, Slightly to Moderately Weathered, Very Strong, Very Fine Grained, Very Thin to Thin Bedded, Jointed - Moderately Fractured, Tight. |

| | |
|---|---|
| LENGTH (INCHES) | 4.10 |
| DIAMETER (INCHES) | 1.99 |
| LENGTH / DIAMETER | 2.06 |
| CORRECTION FACTOR | 1.0 |
| AREA (SQ. IN.) | 3.11 |

| | |
|---|---|
| MASS (GRAMS) | 520.1 |
| UNIT WEIGHT (LBS/CU. FT.) | 155.4 |
| | |
| MAXIMUM LOAD (LBS) | 50,880 |
| COMPRESSIVE STRENGTH (PSI) | 16,360 |

TEST SPECIMEN PHOTO

TEST SPECIMEN PHOTO

# APPENDIX A

# Engineering Calculations

| | |
|---|---|
| CT Project No.: | 229121 |
| Project: | WOO-65-06.18 |
| Calcs by: | CPI |
| Date: | 1/20/2025 |
| | |
| Calcs: | Drilled Shaft Rock Sockets - Vertical Resistance |
| Location: | SR65 over Williamsburg Reservoir Outlet to Maumee River |
| Substructure: | Rear Abutment |
| | |
| Boring(s): | B-003-0-21 |
| Ground Surface Elevation (ft): | 632.1 |
| Bottom of Pier Cap Elev (ft): | 624.25 |
| Top of Rock Elevation (ft): | 598.6 |
| Length of Shaft in Soil (ft): | 25.65 |
| Shaft in Soil Diameter (in): | 36 |
| *(Minimum 42" for Pier Columns, and 36" for others.)* | |
| | |
| Shaft in Rock Diameter (in): | 30 |
| Shaft in Rock Diameter (ft): | 2.5 |
| End-Bearing Elev. at 1.5 x B (ft): | 594.85 |
| Length of Socket (ft): | 3.75 |
| | |
| 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): | 25.65 |
| Therefore, not governing. | |
| | |
| | |
| Structural indicates Scour to: | Not extending to bedrock |
| BDM 305.4.1.1, for end-bearing shafts/sockets in non-scour resistant bedrock, | |
| extend socket to penetrate a minimum of 5 feet or 10 feet below scour elevation in rock for | |
| Qu>2.5ksi and Qu<2.5ksi, respectively. | |
| No scour in rock so not governing. | |
| | |

| | |
|---|---|
| Calcs: | Drilled Shaft Rock Sockets - Vertical Resistance |
| Location: | SR65 over Williamsburg Reservoir Outlet to Maumee River |
| Substructure: | Rear Abutment |
| | |
| Look at RC Qu at bearing to | |
| 2B below bearing: | |
| 2B below bearing Elev.: | 589.85 |
| Qu (psi): | 16560 |
| *Only one tested sample* | |
| | |
| Use Average Qu (psi): | 16560 |
| Average Qu (ksf): | 2385 |
| | |
| End-Bearing Resistance (AASHTO LRFD 10.8.3.5.4c-1) | |
| qp=2.5qu | |
| (Unfactored) qp (ksf): | 5962 |
| Resistance Factor (AASHTO LRFD Table 10.5.5.2.4-1) | |
| ϕ= | 0.5 |
| Factored Bearing Resistance (ksf)= | 2981 |
| Say, Factored Bearing Resistance (ksf)= | 2980 |
| | |
| Indicated Total Factored Load (kips)= | 365 |
| For 2.5 ft diameter socket, | |
| Available Resistance (kips)= | 14628 |
| Suitable Vertical Resistance? | YES |
| | |

| | |
|---|---|
| CT Project No.: | 229121 |
| Project: | WOO-65-06.18 |
| Calcs by: | CPI |
| Date: | 1/20/2025 |
| | |
| Calcs: | Drilled Shaft Rock Sockets - Vertical Resistance |
| Location: | SR65 over Williamsburg Reservoir Outlet to Maumee River |
| Substructure: | Forward Abutment |
| | |
| Boring(s): | B-004-0-21 |
| Ground Surface Elevation (ft): | 632.4 |
| Bottom of Pier Cap Elev (ft): | 625.48 |
| Top of Rock Elevation (ft): | 598.9 |
| Length of Shaft in Soil (ft): | 26.58 |
| Shaft in Soil Diameter (in): | 36 |
| *(Minimum 42" for Pier Columns, and 36" for others.)* | |
| | |
| Shaft in Rock Diameter (in): | 30 |
| Shaft in Rock Diameter (ft): | 2.5 |
| End-Bearing Elev. at 1.5 x B (ft): | 595.15 |
| Length of Socket (ft): | 3.75 |
| | |
| 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): | 26.58 |
| Therefore, not governing. | |
| | |
| | |
| Structural indicates Scour to: | Not extending to bedrock |
| BDM 305.4.1.1, for end-bearing shafts/sockets in non-scour resistant bedrock, | |
| extend socket to penetrate a minimum of 5 feet or 10 feet below scour elevation in rock for | |
| Qu>2.5ksi and Qu<2.5ksi, respectively. | |
| No scour in rock so not governing. | |
| | |

| | |
|---|---|
| Calcs: | Drilled Shaft Rock Sockets - Vertical Resistance |
| Location: | SR65 over Williamsburg Reservoir Outlet to Maumee River |
| Substructure: | Forward Abutment |
| | |
| Look at RC Qu at bearing to | |
| 2B below bearing: | |
| 2B below bearing Elev.: | 590.15 |
| Qu (psi): | 16360 |
| *Only one tested sample* | |
| | |
| Use Average Qu (psi): | 16360 |
| Average Qu (ksf): | 2356 |
| | |
| End-Bearing Resistance (AASHTO LRFD 10.8.3.5.4c-1) | |
| qp=2.5qu | |
| (Unfactored) qp (ksf): | 5890 |
| Resistance Factor (AASHTO LRFD Table 10.5.5.2.4-1) | |
| $\phi$= | 0.5 |
| Factored Bearing Resistance (ksf)= | 2945 |
| Say, Factored Bearing Resistance (ksf)= | 2945 |
| | |
| Indicated Total Factored Load (kips)= | 365 |
| For 2.5 ft diameter socket, | |
| Available Resistance (kips)= | 14456 |
| Suitable Vertical Resistance? | YES |
| | |

|  |  |  |  |  |  |  |  |  |
|---|---|---|---|---|---|---|---|---|
| Project Name: | WOO-65-6.18 - Proposed Culvert Replacement |  |  |  |  |  |  |  |
| Project Number: | 229121 |  |  |  |  |  |  |  |
| Calculated by: | CRO 1/16/2025 |  | Reviewed by: | CPI 1/19/2025 |  |  |  |  |
|  |  |  | Rev2.: | CPI 1/21/2025 |  |  |  |  |
| Calcs: | Drilled Shaft Rock Sockets - Lateral Resistance |  |  |  |  |  |  |  |
| Location: | B-003-0-21 |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |  |
| GSE (ft): | 632.1 |  |  |  |  |  |  |  |
| Long-Term GWT (ft): | 620 | Approximate Maumee River "Normal" Elevation |  |  |  |  |  |  |
| Bottom of Pier Cap Elev. (ft): | 624.25 |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |  |
| Soil |  |  |  |  |  |  |  |  |
| Layer | Soil Type | Top Depth (ft) | Bottom Depth (ft) | Top Elev. (ft) | Bottom Elev. (ft) | Avg. N60 | Avg. HP (tsf) | Qu (tsf) |
| Layer 1 | Stiff A-6a | 0 | 5.5 | 632.1 | 626.6 | 8.5 | 1.625 | - |
|  |  |  | Depth below bottom of Pier Cap: | -7.85 | -2.35 |  |  |  |
| Total Unit Wt (pcf): | 118 | GDM Table 400-4 |  | Use | 120 | pcf |  |  |
| Su = N60 x 125 (N60<= 52 bpf) per GDM 404.1 |  |  |  |  |  |  |  |  |
| Su (ksf)= | 1.06 |  |  |  |  |  |  |  |
| Su (ksf) = HP (tsf) |  |  |  |  |  |  |  |  |
| Su (ksf)= | 1.63 |  |  |  | Recommended Su (ksf) = | 1.3 |  |  |
|  |  |  |  |  |  |  |  |  |
| Evaluation of Strain at half stress (epsilon 50) from LPILE 2018 Technical Manual |  |  |  |  |  |  |  |  |
| Su = 1-2 ksf, epsilon 50 = | 0.007 | Stiff Clay w/o Free Water (Reese) |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |  |
| Layer | Soil Type | Top Depth (ft) | Bottom Depth (ft) | Top Elev. (ft) | Bottom Elev. (ft) | Avg. N60 | Avg. HP (tsf) | Qu (tsf) |
| Layer 2 | Stiff to Very Stiff A-6b | 5.5 | 12.8 | 626.6 | 619.3 | 14 | 2.67 | 1.13 |
|  |  |  | Depth below bottom of Pier Cap: | -2.35 | 4.95 |  |  |  |
| Total Unit Wt (pcf): | 122 | GDM Table 400-4 |  | Use | 120 | pcf |  |  |
| Su = N60 x 125 (N60<= 52 bpf) per GDM 404.1 |  |  |  |  |  |  |  |  |
| Su (ksf)= | 1.75 |  |  |  |  |  |  |  |
| Su (ksf) = HP (tsf) |  |  |  |  |  |  |  |  |
| Su (ksf)= | 2.67 |  |  |  | Recommended Su (ksf) = | 1.7 |  |  |
| Su (ksf) = Qu (tsf) |  |  |  |  |  |  |  |  |
| Su (ksf)= | 1.13 |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |  |
| Evaluation of Strain at half stress (epsilon 50) from LPILE 2018 Technical Manual |  |  |  |  |  |  |  |  |
| Su = 1-2 ksf, epsilon 50 = | 0.007 | Stiff Clay w/o Free Water (Reese) |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |  |
| Layer | Soil Type | Top Depth (ft) | Bottom Depth (ft) | Top Elev. (ft) | Bottom Elev. (ft) | N60 | HP (tsf) | Qu (tsf) |
| Layer 3 | Soft A-6a | 12.8 | 15 | 619.3 | 617.1 | 4 | 0.5 | - |
|  |  |  | Depth below bottom of Pier Cap: | 4.95 | 7.15 |  |  |  |
| Total Unit Wt (pcf): | 112 | GDM Table 400-4 |  | Use | 110 | pcf |  |  |
| Su = N60 x 125 (N60<= 52 bpf) per GDM 404.1 |  |  |  |  |  |  |  |  |
| Su (ksf)= | 0.5 |  |  |  |  |  |  |  |
| Su (ksf) = HP (tsf) |  |  |  |  |  |  |  |  |
| Su (ksf)= | 0.50 |  |  |  | Recommended Su (ksf) = | 0.5 |  |  |
|  |  |  |  |  |  |  |  |  |
| Evaluation of Strain at half stress (epsilon 50) from LPILE 2018 Technical Manual |  |  |  |  |  |  |  |  |
| Soft, epsilon 50 = | 0.020 | Soft Clay (Matlock) |  |  |  |  |  |  |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Calcs: | Drilled Shaft Rock Sockets - Lateral Resistance | | | | | | | |
| Location: | B-003-0-21 | | | | | | | |
| | | | | | | | | |
| Layer | Soil Type | Top Depth (ft) | Bottom Depth (ft) | Top Elev. (ft) | Bottom Elev. (ft) | N60 | HP (tsf) | Qu (tsf) |
| Layer 4 | Loose A-3a | 15 | 18 | 617.1 | 614.1 | 8 | - | - |
| | | | Depth below bottom of Pier Cap: | 7.15 | 10.15 | | | |
| Total Unit Wt (pcf): | 120 | GDM Table 400-4 | | Use | 120 | pcf | | |
| Internal Angle of Friction Determination (GDM 404.2): | | | | | | | | |
| N160 (bpf)=CN*N60 | AASHTO LRFD 10.4.6.2.4 | | | | | | | |
| CN=0.77log(40/sigma-v'), with CN<2.0 | | | | | | | | |
| CN at | 16.5 | ft | | | | | | |
| sigma-v' (ksf): | 1.68 | | | | | | | |
| CN= | 1.1 | <2 so use: | 1.1 | | | | | |
| N160 (bpf)= | 8 | | | | | | | |
| AASHTO LRFD Table 10.4.6.2.4-1 | | | | | | | | |
| N160 | Mid-Range Phi (deg) | | | | | | | |
| 4 | 29.5 | | | | | | | |
| 10 | 32.5 | | | | | | | |
| N160 | Phi (deg) | | | | | | | |
| 8 | 31.7 | use | 31.5 | deg | | | | |
| GDM Table 400-3 phi Adjustment | | | | | | | | |
| A-3a | -0.5 | | | | | | | |
| Phi (deg) = | 31 | < ODOT Maximum 46 deg, ok | | | | | | |
| | | | | | | | | |
| k Evaluation From LPILE 2018 Technical Manual | | | | | | | | |
| Parameters: | loose, submerged sand | | | | | | | |
| Range of k-value (pci) = | 2.1 to 6.4 | | | | | | | |
| Loose range of N60 | k (pci) | | | | | | | |
| 0 | 2.1 | | | | | | | |
| 10 | 6.4 | | | | | | | |
| Interpolate for 8 bpf for this layer: | 5.5 | | | | | | | |
| Say k (pci) = | 6 | Sand (Reese) | | | | | | |
| | | | | | | | | |
| Layer | Soil Type | Top Depth (ft) | Bottom Depth (ft) | Top Elev. (ft) | Bottom Elev. (ft) | Avg. N60 | Avg. HP (tsf) | Qu (tsf) |
| Layer 5 | Very Stiff A-6a and A-6b | 18 | 23 | 614.1 | 609.1 | 20.0 | 2.625 | - |
| | | | Depth below bottom of Pier Cap: | 10.15 | 15.15 | | | |
| Total Unit Wt (pcf): | 125 | GDM Table 400-4 | | Use | 125 | pcf | | |
| Su = N60 x 125 (N60<= 52 bpf) per GDM 404.1 | | | | | | | | |
| Su (ksf)= | 2.5 | | | | | | | |
| Su (ksf) = HP (tsf) | | | | | | | | |
| Su (ksf)= | 2.63 | | | | Recommended Su (ksf) = | 2.5 | | |
| | | | | | | | | |
| Evaluation of Strain at half stress (epsilon 50) from LPILE 2018 Technical Manual | | | | | | | | |
| Su = 2-4 ksf, epsilon 50 = | 0.005 | Stiff Clay w/o Free Water (Reese) | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| Layer | Soil Type | Top Depth (ft) | Bottom Depth (ft) | Top Elev. (ft) | Bottom Elev. (ft) | Avg. N60 | Avg. HP (tsf) | Qu (tsf) |
| Layer 6 | Hard A-6a | 23 | 33.5 | 609.1 | 598.6 | 32.7 | 4.5 | - |
| | | | Depth below bottom of Pier Cap: | 15.15 | 25.65 | | | |
| Total Unit Wt (pcf): | 128 | GDM Table 400-4 | | Use | 130 | pcf | | |
| Su = N60 x 125 (N60<= 52 bpf) per GDM 404.1 | | | | | | | | |
| Su (ksf)= | 4.08 | | | | | | | |
| Su (ksf) = HP (tsf) | | | | | | | | |
| Su (ksf)= | 4.50 | | | | Recommended Su (ksf) = | 4.3 | | |
| | | | | | | | | |
| Evaluation of Strain at half stress (epsilon 50) from LPILE 2018 Technical Manual | | | | | | | | |
| Su = 4-6 ksf, epsilon 50 = | 0.004 | Stiff Clay w/o Free Water (Reese) | | | | | | |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Calcs: | Drilled Shaft Rock Sockets - Lateral Resistance | | | | | | | |
| Location: | B-003-0-21 | | | | | | | |
| Bedrock | | | | | | | | |
| Layer | Soil Type | Top Depth (ft) | Bottom Depth (ft) | Top Elev. (ft) | Bottom Elev. (ft) | RQD (%) | Rec (%) | Qu (psi) |
| Layer 7 | Dolomite - Very Strong | 33.5 | 38.5 | 598.6 | 593.6 | 93 | 100 | 16560 |
| | Moderately Frac. | | | | | | | |
| | | | Depth below bottom of Pier Cap: | 25.65 | 30.65 | | | |
| Total Unit Wt (pcf): | 165-175 | GDM Table 400-5 | | Use | 155 | pcf | | |
| Lab Tested Unit Wt (pcf): | 157 | For B-003 tested sample | | | | | | |
| Qu (psi)= | 16560 | Value for tested sample in B-003 | | | | | | |
| From GDM Table 400-6, say E (psi) = | 1400000 | | | | | | | |
| If Strain at 1400000 psi is 1%, then strain at half max stress (krm) is calculated by: | | | | | | | | |
| Half max stress = Qu/2 = | 8280 | psi | | | | | | |
| krm = 1% x (8280 psi / 1400000 psi) = | 0.0059 | % | | | | | | |
| krm (decimal format) = | 0.000059 | Weak Rock (Reese) | | | | | | |

Project Name: WOO-65-6.18 - Proposed Culvert Replacement
Project Number: 229121
Calculated by: CRO 1/16/2025
Reviewed by: CPI 1/19/2025
Rev2.: CPI 1/21/2025

Calcs: Drilled Shaft Rock Sockets - Lateral Resistance
Location: B-004-0-21

GSE (ft): 632.4
Long-Term GWT (ft): 620 Approximate Maumee River "Normal" Elevation
Bottom of Pier Cap Elev. (ft): 625.48

Soil

| Layer | Soil Type | Top Depth (ft) | Bottom Depth (ft) | Top Elev. (ft) | Bottom Elev. (ft) | N60 | HP (tsf) | Qu (tsf) |
|---|---|---|---|---|---|---|---|---|
| Layer 1 | Stiff A-6a | 0 | 4 | 632.4 | 628.4 | 11 | - | - |
| | | | Depth below bottom of Pier Cap: | -6.92 | -2.92 | | | |
| Total Unit Wt (pcf): | 120 | GDM Table 400-4 | | Use | 120 | pcf | | |
| Su = N60 x 125 (N60<= 52 bpf) per GDM 404.1 | | | | | | | | |
| Su (ksf)= | 1.375 | | Recommended Su (ksf) = | | | 1.3 | | |
| Evaluation of Strain at half stress (epsilon 50) from LPILE 2018 Technical Manual | | | | | | | | |
| Su = 1-2 ksf, epsilon 50 = | 0.007 | Stiff Clay w/o Free Water (Reese) | | | | | | |

| Layer | Soil Type | Top Depth (ft) | Bottom Depth (ft) | Top Elev. (ft) | Bottom Elev. (ft) | N60 | HP (tsf) | Qu (tsf) |
|---|---|---|---|---|---|---|---|---|
| Layer 2 | Hard A-4a | 4 | 5.8 | 628.4 | 626.6 | 11 | 4.5 | - |
| | | | Depth below bottom of Pier Cap: | -2.92 | -1.12 | | | |
| Total Unit Wt (pcf): | 120 | GDM Table 400-4 | | Use | 120 | pcf | | |
| Su = N60 x 125 (N60<= 52 bpf) per GDM 404.1 | | | | | | | | |
| Su (ksf)= | 1.375 | | | | | | | |
| Su (ksf) = HP (tsf) | | | | | | | | |
| Su (ksf)= | 4.50 | | Recommended Su (ksf) = | | | 1.3 | | |
| Evaluation of Strain at half stress (epsilon 50) from LPILE 2018 Technical Manual | | | | | | | | |
| Su = 1-2 ksf, epsilon 50 = | 0.007 | Stiff Clay w/o Free Water (Reese) | | | | | | |

| Layer | Soil Type | Top Depth (ft) | Bottom Depth (ft) | Top Elev. (ft) | Bottom Elev. (ft) | Avg. N60 | Avg. HP (tsf) | Qu (tsf) |
|---|---|---|---|---|---|---|---|---|
| Layer 3 | Very Stiff A-6a | 5.8 | 13 | 626.6 | 619.4 | 16.3 | 2.67 | - |
| | | | Depth below bottom of Pier Cap: | -1.12 | 6.08 | | | |
| Total Unit Wt (pcf): | 122 | GDM Table 400-4 | | Use | 120 | pcf | | |
| Su = N60 x 125 (N60<= 52 bpf) per GDM 404.1 | | | | | | | | |
| Su (ksf)= | 2.04 | | | | | | | |
| Su (ksf) = HP (tsf) | | | | | | | | |
| Su (ksf)= | 2.67 | | Recommended Su (ksf) = | | | 2.4 | | |
| Evaluation of Strain at half stress (epsilon 50) from LPILE 2018 Technical Manual | | | | | | | | |
| Su = 2-4 ksf, epsilon 50 = | 0.005 | Stiff Clay w/o Free Water (Reese) | | | | | | |

| Layer | Soil Type | Top Depth (ft) | Bottom Depth (ft) | Top Elev. (ft) | Bottom Elev. (ft) | Avg. N60 | Avg. HP (tsf) | Qu (tsf) |
|---|---|---|---|---|---|---|---|---|
| Layer 4 | Stiff A-6b | 13 | 21.5 | 619.4 | 610.9 | 11.0 | 1.75 | 1.64 |
| | | | Depth below bottom of Pier Cap: | 6.08 | 14.58 | | | |
| Total Unit Wt (pcf): | 122 | GDM Table 400-4 | | Use | 120 | pcf | | |
| Su = N60 x 125 (N60<= 52 bpf) per GDM 404.1 | | | | | | | | |
| Su (ksf)= | 1.375 | | | | | | | |
| Su (ksf) = HP (tsf) | | | | | | | | |
| Su (ksf)= | 1.75 | | Recommended Su (ksf) = | | | 1.6 | | |
| Su (ksf) = Qu (tsf) | | | | | | | | |
| Su (ksf)= | 1.64 | | | | | | | |
| Evaluation of Strain at half stress (epsilon 50) from LPILE 2018 Technical Manual | | | | | | | | |
| Su = 1-2 ksf, epsilon 50 = | 0.007 | Stiff Clay w/o Free Water (Reese) | | | | | | |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Calcs: | Drilled Shaft Rock Sockets - Lateral Resistance | | | | | | | |
| Location: | B-004-0-21 | | | | | | | |
| Layer | Soil Type | Top Depth (ft) | Bottom Depth (ft) | Top Elev. (ft) | Bottom Elev. (ft) | N60 | HP (tsf) | Qu (tsf) |
| Layer 5 | Hard A-4a | 21.5 | 23.5 | 610.9 | 608.9 | 30 | 4.5 | - |
| | | | Depth below bottom of Pier Cap: | 14.58 | 16.58 | | | |
| Total Unit Wt (pcf): | 128 | GDM Table 400-4 | | Use | 125 | pcf | | |
| Su = N60 x 125 (N60<= 52 bpf) per GDM 404.1 | | | | | | | | |
| Su (ksf)= | 3.75 | | | | | | | |
| Su (ksf) = HP (tsf) | | | | | | | | |
| Su (ksf)= | 4.50 | | | | Recommended Su (ksf) = | 4.0 | | |
| Evaluation of Strain at half stress (epsilon 50) from LPILE 2018 Technical Manual | | | | | | | | |
| Su = 2-4 ksf, epsilon 50 = | 0.005 | Stiff Clay w/o Free Water (Reese) | | | | | | |
| Layer | Soil Type | Top Depth (ft) | Bottom Depth (ft) | Top Elev. (ft) | Bottom Elev. (ft) | N60 | HP (tsf) | Qu (tsf) |
| Layer 6 | Very Stiff A-6a | 23.5 | 26.5 | 608.9 | 605.9 | 14 | 2.5 | - |
| | | | Depth below bottom of Pier Cap: | 16.58 | 19.58 | | | |
| Total Unit Wt (pcf): | 122 | GDM Table 400-4 | | Use | 120 | pcf | | |
| Su = N60 x 125 (N60<= 52 bpf) per GDM 404.1 | | | | | | | | |
| Su (ksf)= | 1.75 | | | | | | | |
| Su (ksf) = HP (tsf) | | | | | | | | |
| Su (ksf)= | 2.50 | | | | Recommended Su (ksf) = | 2.0 | | |
| Evaluation of Strain at half stress (epsilon 50) from LPILE 2018 Technical Manual | | | | | | | | |
| Su = 1-2 ksf, epsilon 50 = | 0.007 | Stiff Clay w/o Free Water (Reese) | | | | | | |
| Layer | Soil Type | Top Depth (ft) | Bottom Depth (ft) | Top Elev. (ft) | Bottom Elev. (ft) | Avg. N60 | Avg. HP (tsf) | Qu (tsf) |
| Layer 7 | Hard A-6b | 26.5 | 33.5 | 605.9 | 598.9 | 40.5 | 4.5 | - |
| | | | Depth below bottom of Pier Cap: | 19.58 | 26.58 | | | |
| Total Unit Wt (pcf): | 128 | GDM Table 400-4 | | Use | 130 | pcf | | |
| Su = N60 x 125 (N60<= 52 bpf) per GDM 404.1 | | | | | | | | |
| Su (ksf)= | 5.06 | | | | | | | |
| Su (ksf) = HP (tsf) | | | | | | | | |
| Su (ksf)= | 4.50 | | | | Recommended Su (ksf) = | 5.0 | | |
| Evaluation of Strain at half stress (epsilon 50) from LPILE 2018 Technical Manual | | | | | | | | |
| Su = 4-6 ksf, epsilon 50 = | 0.004 | Stiff Clay w/o Free Water (Reese) | | | | | | |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Calcs: | Drilled Shaft Rock Sockets - Lateral Resistance | | | | | | | |
| Location: | B-004-0-21 | | | | | | | |
| Augerable Weathered Bedrock | | | | | | | | |
| Layer | Rock Type | Top Depth (ft) | Bottom Depth (ft) | Top Elev. (ft) | Bottom Elev. (ft) | SPT Result | | |
| Layer 8 | Weathered Dolomite | 33.5 | 34.3 | 598.9 | 598.1 | 50/3" | | |
| | | | Depth below bottom of Pier Cap: | 26.58 | 27.38 | | | |
| Total Unit Wt (pcf): | 165-175 | GDM Table 400-5 | | Use | 155 | pcf | | |
| | 155 | Average of Tested Values for the project. | | | | | | |
| Qu based on SPT Results per GDM 404.3 | | | | | | | | |
| Qu (ksf)=0.092x(Nrate)90 (bpf) | | | | | | | | |
| ER(%)= | 66 | | | | | | | |
| N90=50/3" x 12" = | 200 | bpf | | | | | | |
| N90 = 66/90 x 200 bpf = | 146.7 | bpf | | | | | | |
| Qu (ksf) = | 13.5 | | | | | | | |
| Qu (psi) = | 93.7 | | | | | | | |
| Estimate E based on GDM Table 400-6 | | | | | | | | |
| Lowest Qu = 200 psi, indicated as E = 18,000 psi | | | | | | | | |
| Use E (psi) = | 18000 | | | | | | | |
| If Strain at 18,000 psi is 1%, then strain at half max stress (krm) is calculated by: | | | | | | | | |
| Half max stress = Qu/2 = | 46.9 | psi | | | Say RQD (%)= | 0 | | |
| krm = 1% x (47 psi / 18000 psi) = | 0.0026 | % | | | | | | |
| krm (decimal format) = | 0.000026 | Weak Rock (Reese) | | | | | | |
| Bedrock | | | | | | | | |
| Layer | Soil Type | Top Depth (ft) | Bottom Depth (ft) | Top Elev. (ft) | Bottom Elev. (ft) | RQD (%) | Rec (%) | Qu (psi) |
| Layer 9 | Dolomite - Very Strong | 34.3 | 39.3 | 598.1 | 593.1 | 95 | 98 | 16360 |
| | Moderately Frac. | | | | | | | |
| | | | Depth below bottom of Pier Cap: | 27.38 | 32.38 | | | |
| Total Unit Wt (pcf): | 165-175 | GDM Table 400-5 | | Use | 155 | pcf | | |
| Lab Tested Unit Wt (pcf): | 155 | For B-004 tested sample | | | | | | |
| Qu (psi)= | 16360 | Value for tested sample in B-004 | | | | | | |
| From GDM Table 400-6, say E (psi) = | 1400000 | | | | | | | |
| If Strain at 1400000 psi is 1%, then strain at half max stress (krm) is calculated by: | | | | | | | | |
| Half max stress = Qu/2 = | 8180 | psi | | | | | | |
| krm = 1% x (8180 psi / 1400000 psi) = | 0.0058 | % | | | | | | |
| krm (decimal format) = | 0.000058 | Weak Rock (Reese) | | | | | | |

# OHIO DEPARTMENT OF TRANSPORTATION

## OFFICE OF GEOTECHNICAL ENGINEERING

# PLAN SUBGRADES
## Geotechnical Bulletin GB1

## WOO 65-6.18
## 107711
## SR 65 over the existing Williamsburg Reservoir outlet structure to Maumee River Proposed Culvert Replacement

## TTL Associates, Inc

**Prepared By:** Imad El Hajjar, EI
**Date prepared:** Monday, May 23, 2022

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**NO. OF BORINGS:** 2

| # | Boring ID | Alignment | Station | Offset | Dir | Drill Rig | ER | Boring EL. | Proposed Subgrade EL | Cut Fill |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | B-003-0-21 | SR-65 | 326+12 | 8 | RIght | CME 75 Truck Mounted | 66 | 632.1 | 629.6 | 2.5 C |
| 2 | B-004-0-21 | SR-65 | 326+45 | 6 | Left | CME 75 Truck Mounted | 66 | 632.4 | 630.4 | 2.0 C |

OHIO DEPARTMENT OF TRANSPORTATION

# Subgrade Analysis

V. 14.5 1/18/2019

| # | 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-2 | 2.5 | 4.0 | 0.0 | 1.5 | 9 | | 1.75 | | | | | | | 21 | 14 | A-6a | 10 | | | HP & Mc | | 12" | 12"<br>204 Geotextile |
| | 003-0 | SS-3 | 4.0 | 5.5 | 1.5 | 3.0 | 8 | | 1.5 | 32 | 17 | 15 | 21 | 57 | 78 | 18 | 14 | A-6a | 10 | | | HP & Mc | | | |
| | 21 | SS-4 | 5.5 | 8.0 | 3.0 | 5.5 | 14 | | 2.5 | | | | | | | 17 | 16 | A-6b | 16 | | | | | | |
| | | ST-5 | 8.0 | 10.0 | 5.5 | 7.5 | | 8 | 2.75 | 35 | 19 | 16 | 21 | 60 | 81 | 17 | 16 | A-6b | | | | | | | |
| 2 | B | SS-1 | 1.0 | 2.5 | -1.0 | 0.5 | 18 | | NP | | | | | | | 4 | 6 | A-1-b | 0 | 360 | | | | | 12"<br>204 Geotextile |
| | 004-0 | SS-2 | 2.5 | 4.0 | 0.5 | 2.0 | 11 | | - | 31 | 16 | 15 | 16 | 45 | 61 | 16 | 14 | A-6a | 7 | | | $N_{60}$ | | 12" | |
| | 21 | SS-3 | 4.0 | 5.5 | 2.0 | 3.5 | 11 | | 4.5 | | | | | | | 16 | 10 | A-4a | 8 | | | $N_{60}$ & Mc | | | |
| | | SS-4 | 5.5 | 7.0 | 3.5 | 5.0 | 15 | 11 | 2.5 | 30 | 16 | 14 | 21 | 51 | 72 | 16 | 14 | A-6a | 9 | | | | | | |

Ohio Department of Transportation

**Subgrade Analysis**

V. 14.5 1/18/2019

**PID:** 107711

**County-Route-Section:** WOO 65-6.18
**No. of Borings:** 2

**Geotechnical Consultant:** TTL Associates, Inc
**Prepared By:** Imad El Hajjar, EI
**Date prepared:** 5/23/2022

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

| Excavate and Replace Stabilization Options | |
|---|---|
| Global Geotextile<br>Average(N60L):<br>Average(HP): | <br>12"<br>0" |
| Global Geogrid<br>Average(N60L):<br>Average(HP): | <br>0"<br>0" |

| | |
|---|---|
| Design CBR | 6 |

| % Samples within 6 feet of subgrade | | | |
|---|---|---|---|
| $N_{60} \le 5$ | 0% | $HP \le 0.5$ | 0% |
| $N_{60} < 12$ | 50% | $0.5 < HP \le 1$ | 0% |
| $12 \le N_{60} < 15$ | 13% | $1 < HP \le 2$ | 25% |
| $N_{60} \ge 20$ | 0% | $HP > 2$ | 50% |
| M+ | 38% | | |
| Rock | 0% | | |
| Unsuitable | 0% | | |

| Excavate and Replace at Surface | |
|---|---|
| Average | 12" |
| Maximum | 12" |
| Minimum | 12" |

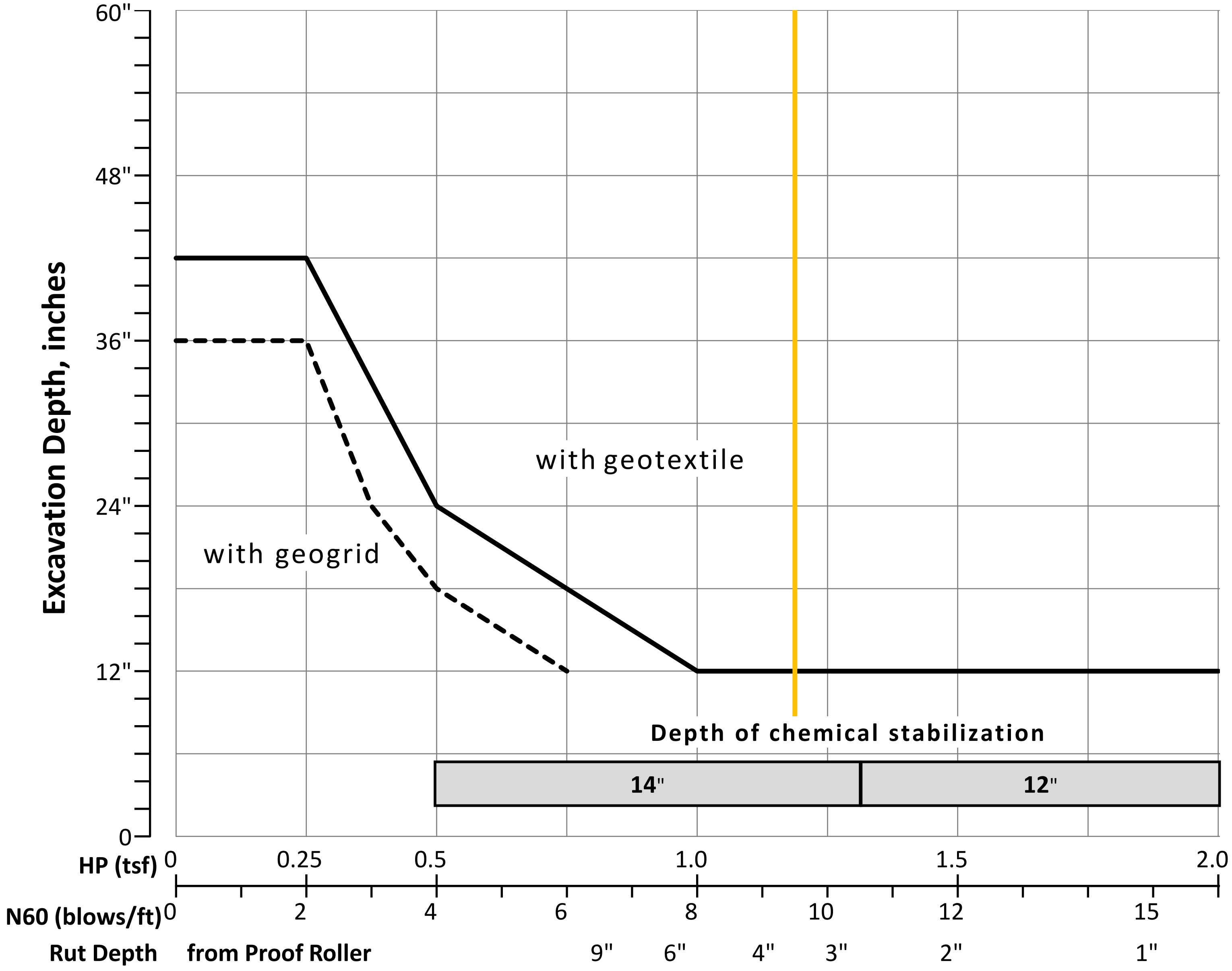
| % Proposed Subgrade Surface | |
|---|---|
| Unstable & Unsuitable | 80% |
| Unstable | 80% |
| Unsuitable | 0% |

| | $N_{60}$ | $N_{60L}$ | HP | LL | PL | PI | Silt | Clay | P 200 | $M_C$ | $M_{OPT}$ | GI |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Average | 12 | 10 | 2.58 | 32 | 17 | 15 | 20 | 53 | 73 | 16 | 13 | 9 |
| Maximum | 18 | 11 | 4.50 | 35 | 19 | 16 | 21 | 60 | 81 | 21 | 16 | 16 |
| Minimum | 8 | 8 | 1.50 | 30 | 16 | 14 | 16 | 45 | 61 | 4 | 6 | 0 |

## Classification Counts by Sample

| ODOT Class | Rock | A-1-a | A-1-b | A-2-4 | A-2-5 | A-2-6 | A-2-7 | A-3 | A-3a | A-4a | A-4b | A-5 | A-6a | A-6b | A-7-5 | A-7-6 | A-8a | A-8b | Totals |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Count | 0 | 0 | 1 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 0 | 0 | 4 | 2 | 0 | 0 | 0 | 0 | 8 |
| Percent | 0% | 0% | 13% | 0% | 0% | 0% | 0% | 0% | 0% | 13% | 0% | 0% | 50% | 25% | 0% | 0% | 0% | 0% | 100% |
| % Rock\|Granular\|Cohesive | 0% | 25% | | | | | | | | 75% | | | | | | | | | 100% |
| Surface Class Count | 0 | 0 | 1 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 0 | 0 | 3 | 0 | 0 | 0 | 0 | 0 | 5 |
| Surface Class Percent | 0% | 0% | 20% | 0% | 0% | 0% | 0% | 0% | 0% | 20% | 0% | 0% | 60% | 0% | 0% | 0% | 0% | 0% | 100% |

## GB1 Figure B – Subgrade Stabilization

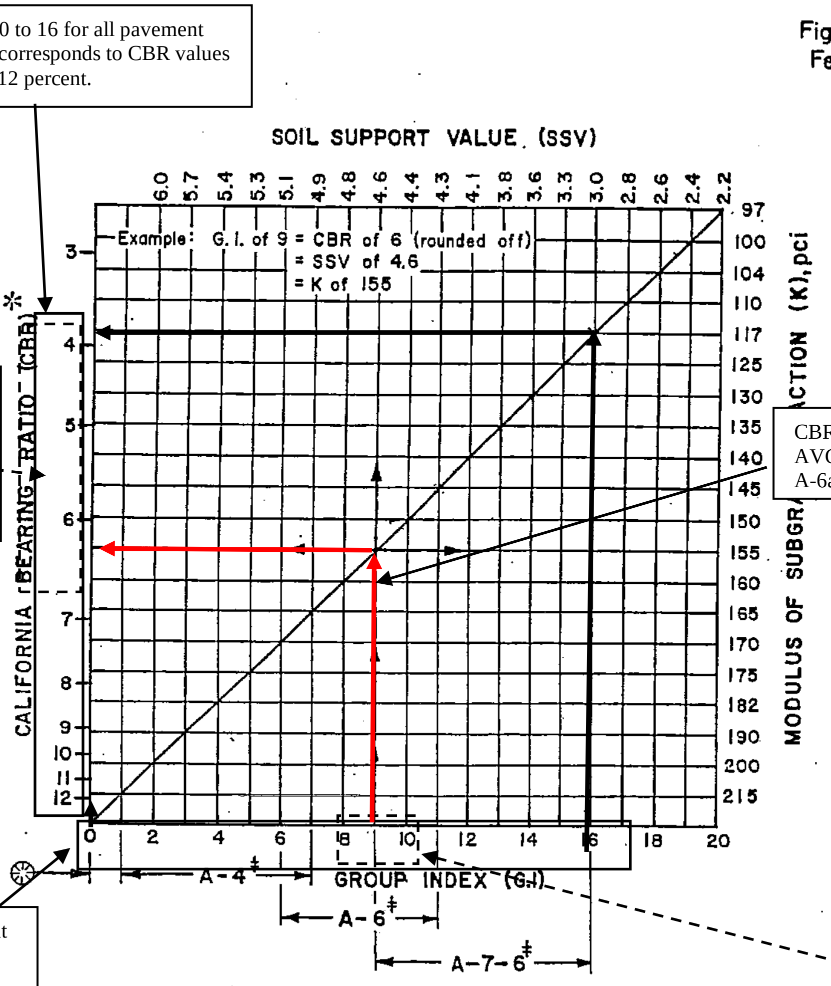


**OVERRIDE TABLE**

| Calculated Average | New Values | Check to Override |
|---|---|---|
| 2.58 | | ☐ HP |
| 9.50 | | ☐ N60L |

| | |
|---|---|
| **Average HP** | ▬ |
| **Average $N_{60L}$** | ▬ |

# WOO 65-6.18



Range of GI from 0 to 16 for all pavement subgrade samples corresponds to CBR values ranging from 4 to 12 percent.

Range of GI from 8 to 16 for tested **A-4a, A-6a and A-6b** pavement subgrade samples corresponds to CBR values of 4 to 6 percent.

CBR = 6 for the project based on AVG GI of 9 for all tested A-4a, A-6a and A-6b soils.

Range of GI for all pavement subgrade samples: 0 to 16

Range of GI for tested **A-4a**, **A-6a** and **A-6b** pavement subgrade samples: 8 to 10, with average of 9

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

‡ Usual range of AASHTO Classes.

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

CORRELATION CHART FOR
SUBGRADE STRENGTHS

# APPENDIX B

## Geotechnical Engineering Design Checklists

# I. Geotechnical Design Checklists

Project: WOO 65-6.18 PDP Path:

PID: 117402 Review Stage: 1

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

# II. Reconnaissance and Planning Checklist

| | | | |
|---|---|---|---|
| C-R-S: WOO 65-6.18 | PID: 107711 | Reviewer: IJH | Date: 10/23/2023 |

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

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

# II. Reconnaissance and Planning Checklist

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

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

# II. Reconnaissance and Planning Checklist

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

# III.C. Subgrade Checklist

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| C-R-S: | WOO 65-6.18 | PID: | 117402 | Reviewer: | IJH | Date: | 10/23/2023 |

*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 <u>Geotechnical Bulletin 1: Plan Subgrades (GB1)</u>? | 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 (soil profile), do the plans specify that these materials need to be removed and replaced or chemically stabilized? | X | None present. |
| a. If these materials are to be removed and replaced, have the station limits, depth, and lateral limits for the planned removal been provided? | X | |
| 3 If there is any rock, shale, or coal present at the proposed subgrade (C&MS 204.05), do the plans specify the removal of the material? | X | Rock deeper than 24 inches below anticipated subgrade elevation so removal not required. |
| a. If removal of any rock, shale, or coal is required, have the station limits, depth, and lateral limits for the planned removal of the material at proposed subgrade been provided? | | |

## III.C. Subgrade Checklist

| Subgrade | | (Y/N/X) | Notes: |
|---|---|---|---|
| 4 | In accordance with GB1, do the SPT ($N_{60}$)/HP values and existing moisture contents for the proposed subgrade soils indicate the need for subgrade stabilization? | N | |
| a. | If removal and replacement is applicable, has the detail of subgrade removal been shown on the plans, including depth of removal, station limits, lateral extent, replacement material, and plan notes (Item 204 - Subgrade Compaction and Proof Rolling)? | Y | Removal and replacement is not anticipated.<br>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.<br>Plans to be prepared by others. |
| | Indicate type of chemical 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: WOO 65-6.18 | PID: 117402 | Reviewer: CPI | Date: 1/22/2025 |

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

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

# IV.A Foundations of Structures Checklist

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

# IV.A Foundations of Structures Checklist

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

# IV.A Foundations of Structures Checklist

| Drilled Shafts | | (Y/N/X) | Notes: |
|---|---|---|---|
| 20 | Are there drilled shafts on the project? If no, go to the next checklist. | Y | |
| 21 | Have the drilled shaft diameter and embedment length been specified? | Y | |
| 22 | Have the recommended drilled shaft diameter and embedment been developed based on the nominal unit side resistance and nominal unit tip resistance for vertical loading situations? | Y | For tip resistance only. |
| 23 | For shafts undergoing lateral loading, have the following been determined: | | |
| a. | total factored lateral shear? | N | LPILE Parameters provided to structural |
| b. | total factored bending moment? | N | LPILE Parameters provided to structural |
| c. | maximum deflection? | N | LPILE Parameters provided to structural |
| d. | reinforcement design? | N | LPILE Parameters provided to structural |
| 24 | If a bedrock socket is required, has a minimum rock socket length equal to 1.5 times the rock socket diameter been used, as per BDM 305.5.2? | Y | |
| 25 | Generally, bedrock sockets are 6" smaller in diameter than the soil embedment section of the drilled shaft. Has this factor been accounted for in the drilled shaft design? | Y | |
| 26 | If scour is predicted, has shaft resistance in the scour zone been neglected? | X | Scour extends to 5 feet above top of rock, so not a design consideration for socket resistance or embedment. |
| 27 | Has the site been assessed for groundwater influence? | Y | Effective unit weight below Maumee River level of Elev. 620+/-. |
| a. | If yes, and if artesian flow is a potential concern, does the design address control of groundwater flow during construction? | X | Recommended temporary casing. Recommended use of tremie concrete installation, if needed. |
| 28 | Have all the proper items been included in the plans for integrity testing? | X | Recommended in the report. Plans still in production. Will address in Geo Review Letter. |
| 29 | If special construction features (e.g., slurry, casing, load tests) are required, have all the proper items been included in the plans? | X | Recommended in the report. Plans still in production. Will address in Geo Review Letter. |
| 30 | If necessary, have wet construction methods been specified? | X | Abutments beyond waterway channel. |
| **General** | | **(Y/N/X)** | **Notes:** |
| 31 | Has the need for load testing of the foundations been evaluated? | X | |
| a. | If needed, have details and plan notes for load testing been included in the plans? | X | |

## VI.B. Geotechnical Reports

| | | | |
|---|---|---|---|
| C-R-S: WOO 65-6.18 | PID: 117402 | Reviewer: CPI | Date: 4/2/2025 |

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

## VI.B. Geotechnical Reports

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