STRUCTURE FOUNDATION EXPLORATION

WOO-582-16.35, PID 107717

Proposed Culvert Replacement

Middleton Pike and Pemberville Road

Troy Township, Wood County, Ohio

Submitted to *ODOT District 2*

Date *July 2023*

Prepared by

TTL associates inc

Environmental, Geotechnical Engineering & Testing

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July 21, 2023

TTL Project No. 231368

Ms. Jorey Summersett, P.E.
ODOT District 2
317 East Poe Road
Bowling Green, Ohio 43402

Final Report
Structure Foundation Exploration
Proposed Culvert Replacement
WOO-582-16.35, PID 107717
Middleton Pike and Pemberville Road
Troy Township, Wood County, Ohio

Dear Ms. Summersett:

Following is the report of our structure foundation exploration performed by TTL Associates, Inc. (TTL) for the referenced site. This study was performed in accordance with Proposal No. P231368, dated May 31, 2023, and was authorized by ODOT Agreement No. 37607, referencing Encumbrance Number 741256, dated June 8, 2023.

A "Draft" report was provided to you on July 17, 2023. This final report incorporates comments regarding standard headwall design assumptions. This report contains the results of our study, our engineering interpretation of the results with respect to the project characteristics, as well as our design and construction recommendations for the replacement culvert.

The soil and rock samples collected during this exploration will be stored at our laboratory for 90 days from the date of this report. The samples will be discarded after this time unless you request that they be saved or delivered to you.

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

Sincerely,

TTL Associates, Inc.

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FINAL REPORT
STRUCTURE FOUNDATION EXPLORATION
PROPOSED CULVERT REPLACEMENT
WOO-582-16.35, PID 107717
MIDDLETON PIKE AND PEMBERVILLE ROAD
TROY TOWNSHIP, WOOD COUNTY, OHIO

FOR

ODOT DISTRICT 2
317 EAST POE ROAD
BOWLING GREEN, OHIO 43402

SUBMITTED

JULY 21, 2023
TTL PROJECT NO. 231368

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EXECUTIVE SUMMARY

This structure foundation exploration report has been prepared for the proposed replacement of the culvert along Middleton Pike (State Route 582), in Troy Township, Wood County, Ohio, designated as WOO-582-16.35, PID 107717. This exploration included two test borings laboratory testing, and engineering evaluations for support for the proposed culverts and headwall foundations. A summary of the conclusions and recommendations of this study are as follows:

1. The surface materials encountered in Borings B-001 and B-002 consisted of asphalt on the order of 10 inches and 8 inches in thickness, underlain by aggregate base on the order of 8 inches and 9 inches in thickness respectively.

2. Based on the results of our field and laboratory tests, the subsoils encountered underlying the existing pavement materials can be generally be described as two strata of predominantly cohesive soils with varying strength and moisture characteristics. The soils at the site were underlain by dolomite bedrock. **Stratum I** consisted of stiff to very stiff cohesive soils encountered underlying the existing pavement materials to depths of 7.2 feet and 7.5 feet below existing grade. The Stratum I cohesive soils consisted of silt and clay (ODOT A-6b). **Stratum II** consisted of very stiff to hard cohesive soils (commonly referred to as “hardpan” in this region) encountered underlying Stratum I in Borings B-001 and B-002 to depths of 19.7 feet and 19.9 feet, respectively. The Stratum I cohesive soils consisted of silt and clay (ODOT A-6b), as well as sandy silt (ODOT A-4a). A zone of granular soils was encountered within each of the borings from approximately 16 to 18 feet. The granular soils consisted of gravel and stone fragments with sand and silt (ODOT A-2-4), as well as gravel and stone fragments with sand, silt, and clay (ODOT A-2-6). Underlying the soils, dolomite bedrock was encountered. The rock core data obtained from the borings are summarized as follows:

Rock Core Data					
Boring Number	**Rock Core Number**	**Depth (feet)**	**Approximate Elevation (feet)**	**Recovery (%)**	**RQD (%)**
B-001	NQ2-1	19.7 – 24.7	626.3 – 621.3	100	85
	NQ2-2	24.7 – 29.7	621.3 – 616.3	100	83
B-002	NQ2-1	19.9 – 24.9	626.2 – 621.2	100	78
	NQ2-2	24.9 – 29.9	621.2 – 616.2	100	85

3. During this exploration, groundwater was initially encountered during drilling at a depth of 18 feet below existing grade. Groundwater was observed upon completion of drilling at depths of 12 feet and 9.7 feet. Apart from streamflow influences in the creek, it is our opinion that the “normal” groundwater level can generally be expected at at depths on the order of 11 feet or lower.

4. Based on the conditions encountered in the borings, the soils encountered within the alignment of the culvert and at the headwall bearing elevations are anticipated to consist of Stratum I stiff to very stiff near the transition to Stratum II hard cohesive soils, all which are considered generally suitable for the support of the proposed culvert and headwalls.

5. We understand that the culvert headwall will be designed using LRFD specifications. At the **service** limit state, the factored bearing resistance was determined to be 8 ksf. Settlement associated with this bearing resistance was calculated to be on the order of 1 to 1½ inches. To reduce total calculated settlement to 1 inch or less, we recommend a service limit state factored bearing resistance (q_r) of 4 ksf. At the **strength** limit state, the factored bearing resistance was calculated to be 10.7 ksf. The bearing soils should be confirmed as being native cohesive soils with an unconfined compressive strength of at least 8,000 pounds per square foot (hand penetrometer reading of 4.0 tsf or greater).

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 culvert along Middleton Pike (State Route 582), in Troy Township, Wood County, Ohio, designated as WOO-582-16.35, PID 107717. The culvert is located approximately ½ mile north of the intersection with Devils Hole Road, as shown on the attached Site Location Map (Plate 1.0).

This study was performed in accordance with Proposal No. P231368, dated May 31, 2023, and was authorized by Ohio Department of Transportation Agreement No. 37607, referencing Encumbrance Number 741256, dated June 8, 2023.

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 culvert at the referenced location. To accomplish this, two test borings, field and laboratory soil testing, a geotechnical engineering evaluation of the test results, and review of available geologic and soils data for the project area were performed.

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 provides our design and construction recommendations for culvert support.

This report includes:

- A description of the existing surface cover, subsurface soils, rock, and groundwater conditions encountered in the borings.
- Design recommendations for culvert support.
- Recommendations concerning soil-, rock-, and groundwater-related construction procedures such as site preparation, earthwork, culvert 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 culvert will be replaced with a new box culvert with full height headwalls. The proposed culvert will have a 9-feet span, 6-feet rise, and 75-feet length. The culvert inverts are indicated to be Elevs. 640.82 and 640.66 at the inlet and outlet, respectively. The headwall footing is shown at the inlet to bear at Elev. 638.57, with a width of 5.5feet, and a total length of approximate 32.15 feet. At the outlet, the headwall footing is shown to bear at Elev. 638.40, with a width of 6.25 feet, and a total length of approximately 35.21 feet.

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 within the Woodville Lake-Plain Reefs Physiographic District of the Huron-Erie Lake Plains Section of Ohio. Within this district, the predominant geologic deposits consist of Pleistocene-age silt, clay, and wave-planed clayey till over Silurian-age and Devonian-age carbonate bedrock and shale.

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

Bedrock in the project area is broadly mapped on the "Geologic Map of Ohio" as Silurian-age Monroe limestone and shale. Specific to the project site, the uppermost carbonate rock formation is mapped as Guelph (Niagaran) dolomite. Bedrock was encountered within the borings performed for this investigation at approximate Elev. 626.

2.1.1 Generalized Near-Surface Soils

The USDA Natural Resource Conservation Service (NRCS) Web Soil Survey indicates that soils in the project area are predominantly mapped as Hoytville clay loam. The Hoytville clay loam soils formed in wave-worked till plains. These soils generally consist of clayey till over dense till. The Hoytville clay loam soils are considered very poorly drained soils, with very low permeability.

2.2 Site Reconnaissance

TTL performed site reconnaissance on June 16, 2023. The existing culvert consists of corrugated metal pipe, approximately 6 ½ feet in diameter. The water depth was approximately 10 inches, flowing from south to north. Visible erosion was observed within the crushed stone shoulder along the eastern edge of Pemberville Road.

The existing asphalt pavements along Pemberville Road and Middleton Pike appeared in fair to poor condition with horizontal and longitudinal cracking.

Surrounding land usage was predominantly agricultural, with occasional rural residences.

3.0 EXPLORATION

3.1 Historic Borings

Historic borings were not available within the project vicinity.

3.2 Project Exploration Program

Two test borings, designated as Borings B-001-0-23 and B-002-0-23 were drilled by TTL on July 10 and 12, 2023. These borings are fully designated in accordance with ODOT protocol, but the -0-23 portion of the nomenclature is generally omitted in the discussions within this report. Boring B-001 was located near the outlet side of the culvert. Boring B-002 was located near the inlet side of the culvert. The proposed culvert and approximate locations of the borings are presented on the Test Boring Location Plan (Plate 2.0).

Latitude, Longitude, and ground surface elevations at the boring locations were surveyed by TTL via a hand-held GPS. These data are presented on the logs of test borings, and are summarized in the following table. Stations and offsets were estimated based on the Stage 2 drawings provided by ODOT District 2.

Table 3.2 Boring Location Information

Boring Number	Ground Surface Elevation (feet)	Latitude (Degrees)	Longitude (Degrees)	Reference Alignment	Station (feet)	Offset (feet)
B-001-0-23	646.0	41.450113	-83.453827	SR 582	86276	5 LT
B-002-0-23	646.1	41.449937	-83.453686	SR 582	86275	7 RT

Borings B-001 and B-002 were planned as Type E2b structure borings per geotechnical investigative procedures outlined in Ohio Department of Transportation (ODOT) “Specifications for Geotechnical Explorations” (SGE). The borings were extended to auger refusal and extended 10 feet into the underlying bedrock using coring methods.

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 550X ATV-mounted drilling rig, as well as a Diedrich D70 track-mounted drilling rig, each utilizing 3¼-inch inside diameter hollow-stem augers. During auger advancement, split-spoon drive samples were generally taken at 2½-foot intervals to auger refusal. The samples were sealed in jars and transported to our laboratory for further classification and testing.

Split-spoon (SS) soil samples were obtained by the Standard Penetration Test Method (ASTM D 1586). The Standard Penetration Test (SPT) consists of driving a 2-inch outside diameter split-spoon sampler into the soil with a 140-pound weight falling freely through a distance of 30 inches. The sampler was driven in three successive 6-inch increments, with the number of blows per increment being recorded. 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 550X ATV-mounted drill rig utilized in this project was 75.2 percent, based on calibration on February 20, 2023. The calibrated hammer/rod energy ratio for the Diedrich D70 track-mounted drill rig utilized in this project was 90.0 percent, based on calibration on April 13, 2022. The N_{60}-values are presented on the attached Logs of Test Borings.

Shelby tube samples, designated ST on the Logs of Test Borings, were obtained from Borings B-001 (6 to 8 feet) and B-002 (6 to 8 feet). Each Shelby tube sample was 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 each boring, using an NQ2 diamond-bit core barrel and coring techniques in general accordance with ASTM D 2113. Two 5-foot core runs were completed immediately following auger refusal in both borings. 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 "RC" on the Logs of Test Borings. The recovered rock cores were visually classified using the ODOT Rock Classification System. The rock cores were also documented by photographic core logs which are attached to this report.

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). Dry density determinations and unconfined compressive strength tests by the constant rate of strain method (ASTM D 2166) were performed on 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 and 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 and Unconfined Compression Test sheet.

4.0 FINDINGS

4.1 General Site Conditions

The site is located along State Route 582 (SR 582), approximately ½ mile north of the intersection with Devils Hole Road. In the project area, grades along SR 582 were on the order of Elev. 646.

The surface materials encountered in Borings B-001 and B-002 consisted of asphalt on the order of 10 inches and 8 inches in thickness, underlain by aggregate base on the order of 8 inches and 9 inches in thickness respectively.

4.2 General Soil Conditions

Based on the results of our field and laboratory tests, the subsoils encountered underlying the existing pavement materials can be generally be described as two strata of predominantly cohesive soils with varying strength and moisture characteristics. The soils at the site were underlain by dolomite bedrock.

Stratum I consisted of stiff to very stiff cohesive soils encountered underlying the existing pavement materials to depths of 7.2 feet and 7.5 feet below existing grade. The Stratum I cohesive soils consisted of silt and clay (ODOT A-6b). SPT N_{60}-values ranged from 6 to 15 blows per foot (bpf). Unconfined compressive strengths were on the order of 2,500 pounds per square foot (psf) or greater, with the higher unconfined compressive strength estimates indicating desiccation and/or some transition to the underlying hard soils. Moisture contents varied from 14 to 24 percent.

Stratum II consisted of very stiff to hard cohesive soils (commonly referred to as "hardpan" in this region) encountered underlying Stratum I in Borings B-001 and B-002 to depths of 19.7 feet and 19.9 feet, respectively. The Stratum I cohesive soils consisted of silt and clay (ODOT A-6b), as well as sandy silt (ODOT A-4a). The upper samples from this stratum exhibited SPT N_{60}-values ranging from 24 to 72 bpf. The SPT for the samples obtained just above the encountered rock resulted in split-spoon refusal (50 or more blows for six inches or less penetration). Unconfined compressive strengths ranged from 6,000 psf to greater than 9,000 psf (the highest obtainable reading using a calibrated hand penetrometer). Moisture contents ranged from 11 to 18 percent.

A zone of granular soils was encountered within each of the borings from approximately 16 to 18 feet. The granular soils consisted of gravel and stone fragments with sand and silt (ODOT A-2-4), as well as gravel and stone fragments with sand, silt, and clay (ODOT A-2-6). SPT N_{60}-values of 34 bpf and 66 bpf, indicating dense to very dense compactness, and moisture contents of 9 percent and 7 percent were determined for these soils.

Underlying the soils, dolomite bedrock was encountered. In each of the borings, the bedrock was cored for a total length of approximately 10 feet, starting from the depth in the bedrock profile where auger refusal was encountered. The rock core data obtained from the borings are summarized as follows:

Table 4.2. Rock Core Data

Boring Number	Rock Core Number	Depth (feet)	Approximate Elevation (feet)	Recovery (%)	RQD (%)
B-001	NQ2-1	19.7 – 24.7	626.3 – 621.3	100	85
	NQ2-2	24.7 – 29.7	621.3 – 616.3	100	83
B-002	NQ2-1	19.9 – 24.9	626.2 – 621.2	100	78
	NQ2-2	24.9 – 29.9	621.2 – 616.2	100	85

RQD values generally ranged from 78 to 85 percent, indicating that the overall rock mass quality in the upper profile can be generally described as good.

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

4.3 Groundwater Conditions

During this exploration, groundwater was initially encountered during drilling at a depth of 18 feet below existing grade. Groundwater was observed upon completion of drilling at depths of 12 feet and 9.7 feet. 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 tributary, it is our opinion that the “normal” groundwater level can generally be expected at depths on the order of 11 feet or lower. However, groundwater elevations can fluctuate with seasonal and climatic influences, and will also be particularly affected locally by water levels in the tributary. Therefore, groundwater conditions may vary at different times of the year from those encountered during this exploration.

4.4 Remedial Measures

Based on the conditions encountered in the borings, the soils encountered within the alignment of the culvert and at the headwall bearing elevations are anticipated to consist of Stratum I stiff to very stiff near the transition to Stratum II hard cohesive soils, all which are considered generally suitable for the support of the proposed culvert and headwalls. However, with any installation within a tributary, there may be areas of encountered sediment at bearing elevations, which would require over-excavation.

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 Culvert Support and Installation

It is our understanding that the existing culvert will be replaced with a new box culvert with full height headwalls. The proposed culvert will have a 9-feet span, 6-feet rise, and 75-feet length. The culvert inverts are indicated to be Elevs. 640.82 and 640.66 at the inlet and outlet, respectively.

5.1.1 Culvert Support

Based on the conditions encountered in Borings B-001 and B-002, the soils at the anticipated invert depths are expected to consist of a thin layer of Stratum I stiff to very stiff near the transition to Stratum II hard cohesive soils, all which are considered generally suitable for the support of the proposed culvert, using bedding materials in accordance with ODOT Construction and Material Specifications (CMS) and manufacturer's guidelines.

Although not anticipated to be prevalent, if unsuitable soils are encountered at the invert elevations, they must be undercut to firm subgrade conditions. In areas of extensive poor subgrade conditions, unsuitable soils should be undercut as needed to establish a stable base for support of the culvert. The undercut zones should be replaced with engineered fill, properly placed and compacted as outlined in Section 5.4 of this report. If saturated soil or groundwater seepage is encountered, we recommend that a coarse, open-graded aggregate be utilized (ODOT Table 703.01-1, No. 57 or No. 67 stone).

We recommend that the culverts be installed as soon as practical after excavation operations and that water not be allowed to pond in the excavation. If it is necessary to leave the exposed subgrade open for any extended period of time, the contractor may need to undercut the subgrade soils below the design bearing elevation, and replace the bottom of the excavation with 12 inches of stone to facilitate dewatering and maintenance of a firm subgrade condition. Should an excavation be allowed to collect and pond water, it may be necessary to undercut saturated or unstable subgrade and replace it with additional stone.

Along the proposed culvert alignment, we recommend that the trench excavation at the invert elevation of the proposed culvert be inspected by a geotechnical engineer or qualified representative. This is to confirm that the bearing soils are consistent with those encountered in the test borings, and that the exposed materials are capable of supporting the proposed culvert and/or that engineered fill has been properly placed and compacted.

5.1.2 Open-Cut Installation Methods

The sides of the temporary excavations for culvert 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 at or near the culvert invert the culvert invert should be generally conducive to stable excavation slopes, provisions should be made for the culvert installation to proceed as a sloped-bank excavation, or as a steeper trench-type cut with properly designed and installed lateral bracing. The latter system may include the use of a portable trench box or a sliding trench shield.

If the excavation is to be performed with sloped banks, adequate stable slopes must be provided in accordance with OSHA criteria. The soils encountered in the test borings within the anticipated depth of excavations may include:

- OSHA Type A soils (cohesive soils with unconfined compressive strengths of 3,000 pounds per square foot (psf) or greater), and
- OSHA 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 and B soils, side slopes must be constructed no steeper than ¾ horizontal to 1 vertical (¾H:1V) and 1H: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 for the lower strength soil. In all cases, flatter slopes may be required if lower strength soils or adverse seepage conditions are encountered during construction. Depending on streamflow and tributary water levels at the time of construction, it may be advantageous to utilize temporary sheetpiling to support excavations that will extend below stream level.

5.2 Headwall Foundations and Spread Foundations

The headwall footing is shown at the inlet to bear at Elev. 638.57, with a width of 5.5 feet, and a total length of approximate 32.15 feet. At the outlet, the headwall footing is shown to bear at Elev. 638.40, with a width of 6.25 feet, and a total length of approximately 35.21 feet.

Based on the conditions encountered in the borings, the soils at the anticipated culvert headwall bearing elevation are expected to consist of Stratum II very stiff to hard native cohesive soils, as well as dense to very dense native granular soil zones, which are considered generally suitable for support of the proposed headwall foundations. The bearing soils should be confirmed as being native cohesive soils with an unconfined compressive strength of at least 8,000 pounds per square foot (hand penetrometer reading of 4.0 tsf or greater).

We understand that the culvert bearing slab will be designed using LRFD specifications. At the **service** limit state, a nominal (unfactored) bearing resistance (q_n) of 8 kips per square foot (ksf) was determined for the culvert base bearing in Stratum II very stiff to hard native cohesive soils. At the service limit state, the resistance factor (ϕ_b) is 1.0. Therefore, the factored bearing resistance (q_r) is 8 ksf. From a conventional allowable stress design comparison, this is roughly akin to using an allowable bearing pressure. **A reduced service limit state factored bearing resistance of 4 ksf would need to be utilized for design, to maintain calculated settlement of 1 inch or less, if required for the structure.**

At the **strength** limit state, we recommend a nominal bearing resistance (q_n) of 21.3 ksf for the culvert base bearing in Stratum II very stiff to hard native cohesive soils. At the strength limit state, the resistance factor (ϕ_b) is 0.5. Therefore, the factored bearing resistance (q_r) is 10.7 ksf. From a conventional allowable stress design comparison, this is roughly akin to calculating an ultimate bearing capacity and applying a factor of safety.

Settlement of the culvert was calculated by conventional consolidation theory utilizing recompression indices for the over-consolidated cohesive soils based on empirical relations using moisture content. Based on a bearing pressure of 8 ksf, using the service limit state bearing resistance indicated above, total settlement was calculated to be on the order of 1 to 1½ inches. To reduce total calculated settlement to 1 inch or less, we recommend a service limit state factored bearing resistance (q_r) of 4 ksf.

Although not anticipated to be prevalent, if unsuitable bearing soils are encountered during culvert installation, over-excavation should extend through these materials to suitable bearing soils. The base of the over-excavation should be widened 6 inches for every foot of depth extending beyond the edge of the culvert. For the relatively high strength limit state factored bearing resistance of 10.7 ksf and service limit state factored bearing resistance of 8 ksf (if utilized) indicated above, the over-excavated areas should be backfilled with lean concrete having a minimum compressive strength of 1,500 pounds per square inch (psi) or other flowable controlled-density fill having a minimum compressive strength of 300 psi. If design incorporates a strength limit state and service limit state factored bearing resistance of 4 ksf or less, then dense-graded aggregate may be utilized for backfill. The aggregate should be placed and compacted as described in Section 5.4.3. If foundations will be placed at the base of the over-excavation or the lean concrete fill option will be utilized, widening the footing over-excavation will not be required. If the controlled-density fill option is utilized, the footing over-excavation shall be widened as discussed above.

For culvert walls that are restrained at the top of the wall, lateral earth pressures should be assumed for "at-rest" conditions. It is anticipated that excavated on-site cohesive soils will comprise the majority of the backfill behind the new culvert walls. For the cohesive soils, an active earth pressure coefficient (k_a) of 0.5 should be used in determining the lateral pressure acting on the walls, along with a total (moist) soil unit weight of 135 pounds per cubic foot (pcf). Alternatively, an equivalent fluid weight of 67.5 pcf may be used for the "at-rest" case design.

If lower at-rest earth pressures are preferred for structural reasons, we recommend that a select, free-draining granular fill (such as No. 57 or 67 stone) be utilized for the entire culvert backfill zone extending to the surface from the base of the wall at 45 degrees. For these granular fill types, k_o may be taken as 0.4, and the soil unit weight may be assumed as 120 pcf. Alternatively, an equivalent fluid weight of 50 pcf may be used for these granular fills.

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

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

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

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

- The cohesion (c) of the clay, for which we recommend c be taken as 4,000 psf, or
- Although not anticipated to be the case, where footings are supported on at least 6 inches of compacted granular material, one-half the normal stress on the interface between the footing and soil.

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

We recommend all slopes on the toe side of the headwall have erosion protection, such as vegetated topsoil, riprap, and/or man-made materials. Seeding of the exterior slopes should be completed as soon as possible after construction is complete.

5.2.1 <u>Standard Headwall Foundation Considerations</u>

It was indicated that slightly modified ODOT standard concrete headwalls for precast box culverts (Sheet HWDD-1) will be utilized for this project. The standard concrete headwalls are indicated to be based on design using a minimum undrained shear strength (s_u), or cohesion (c), of 1,500 pounds per square foot (psf) when the walls are bearing on cohesive soils. The design s_u or c value for the very stiff to hard cohesive bearing soils encountered during this investigation is 4,000 psf, which meets the minimum design requirement. Likewise, the standard concrete headwalls are indicated to be based on an internal angle of friction (drained) of ϕ' = 28 degrees for foundation soil. Based on correlations with plasticity index and N_{60} values, we estimate a ϕ' of at least 29 degrees for the foundation soil, which also meets the minimum design requirement.

It should be noted that the standard headwall design values are based on backfill with a slope not exceeding 2 horizontal to 1 vertical (2H:1V) consisting of soil with an internal angle of friction (ϕ) of at least 30 degrees and a total soil unit weight of 120 pounds per cubic foot (pcf) or less. As such, the backfill behind headwalls should not consist of on-site excavated cohesive soils, since they do not meet these criteria. Rather, a select, free-draining granular fill (such as No. 57 or 67 stone) could be utilized. For these granular fill types, ϕ may be taken as 37 degrees, and the soil unit weight may be assumed as 120 pcf. This material should be placed for the entire headwall backfill zone extending to the surface from the base of the wall at 63 degrees from the horizontal [Slip Line/Failure Envelope of 45 + (ϕ/2) degrees for active earth pressure condition].

We recommend all slopes on the toe side of the wall have erosion protection, such as vegetated topsoil, riprap, and/or man-made materials. Seeding of the exterior slopes should be completed as soon as possible after construction is complete.

5.3 Construction Dewatering and Groundwater Control

Groundwater conditions at the culvert locations were previously discussed in Section 4.3. Based on the soil characteristics and groundwater conditions encountered in Borings B-001 and B-002, it is our opinion that the "normal" groundwater level can generally be expected at or below a depth of approximately 11 feet below roadway grades.

If construction does not occur during a particularly wet period, adequate control of groundwater seepage into excavations extending only a few feet below the "normal" groundwater level should be achievable by minor dewatering systems, such as pumping from prepared sumps. Even at greater depths 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.

Based on the location of the proposed excavations relative to the existing tributary, it is possible 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 due to groundwater seepage or surface run-off.

Other seepage or surface water run-off control measures, including sheetpile wall cutoff or cofferdam installation, may be required to divert or dewater the tributaries if "stream flow" occurs during construction.

5.4 Construction

5.4.1 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.

5.4.2 Site Preparation

Site and subgrade preparation activities should conform to ODOT CMS Item 204 specifications.

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.

After installation of the culvert and backfilling operations, adjoining pavement subgrades exposed as part of the culvert excavation should be proof rolled in accordance with ODOT CMS 204.06.

5.4.3 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 cohesive fill materials and native cohesive soils. For these 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

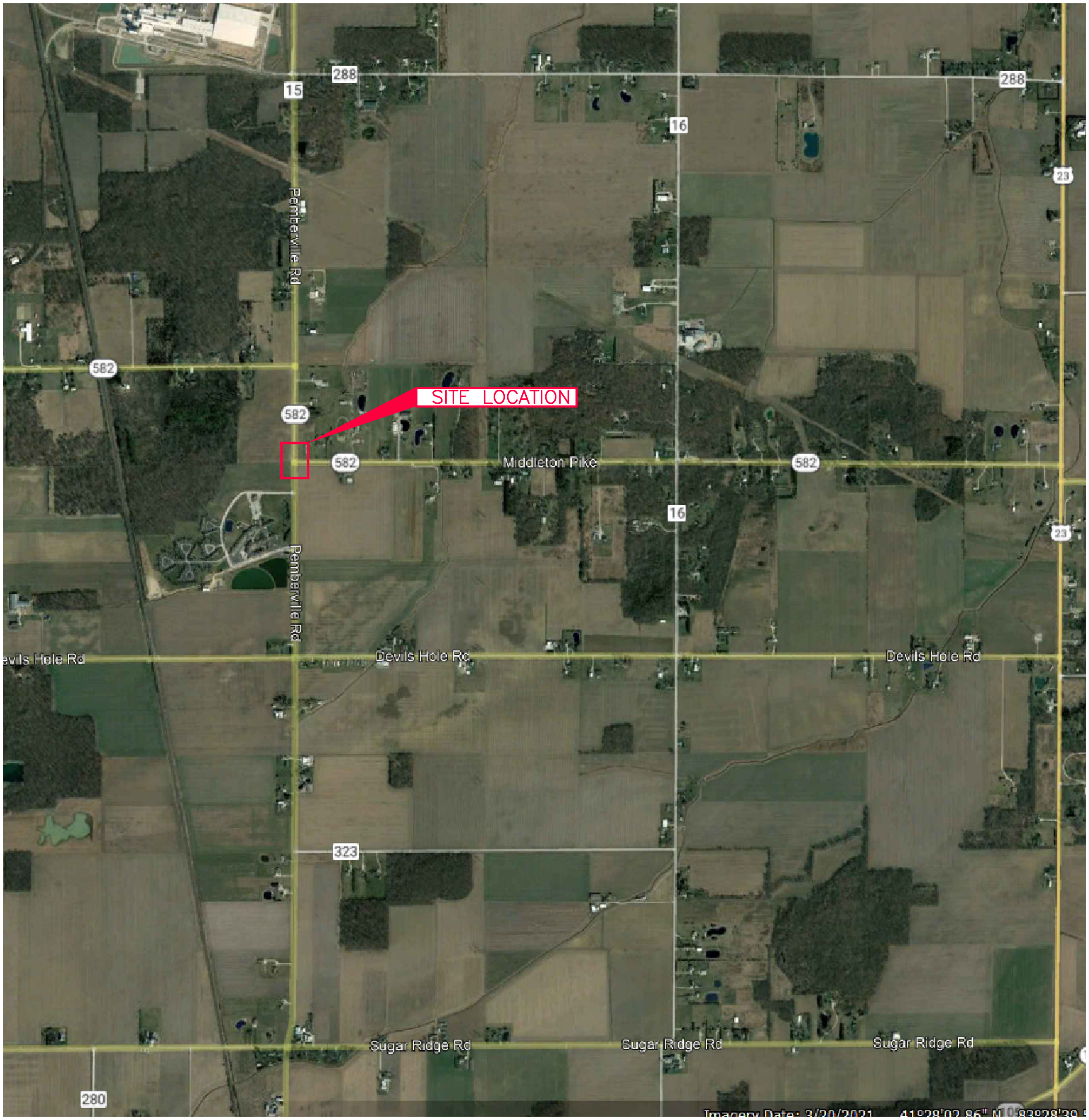
Our evaluation of design and construction conditions for the proposed culvert replacement has been based on our understanding of the site and project information and the data obtained during our field exploration. The general subsurface conditions were based on interpretation of the data obtained at 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 at 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



LEGEND

APPROXIMATE SITE LOCATION

N

APPROXIMATE SCALE – FEET

0 2,200 4,400

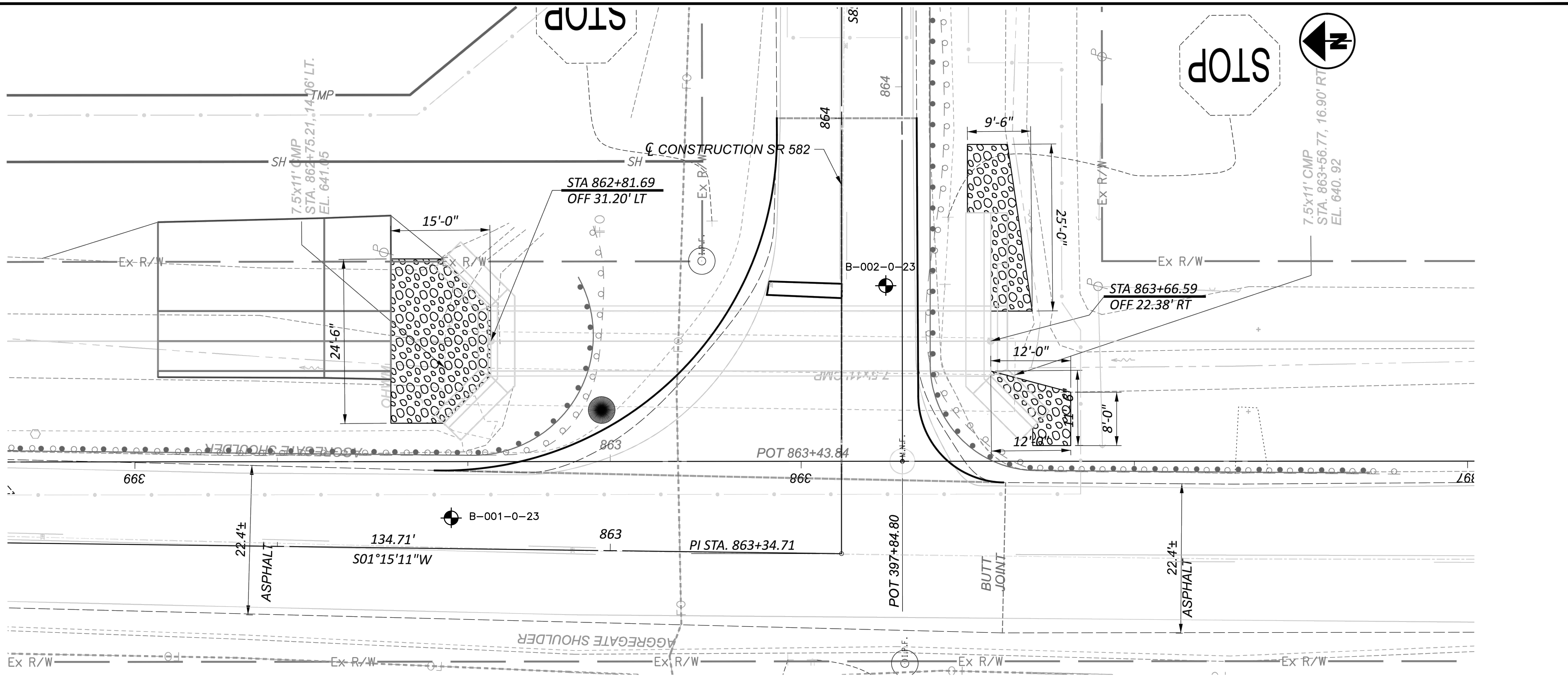
PLATE 1.0
SITE LOCATION MAP

WOO–582–16.35, PID 107717
PROPOSED CULVERT REPLACEMENT
TROY TOWNSHIP, WOOD COUNTY, OHIO

PREPARED FOR
ODOT DISTRICT 2
BOWLING GREEN, OHIO

DRAWN	TRR/7–14–23	CHECKED	KCH/7–14–23
REVISED		APPROVED	
JOB NO.	231368		
DRAWING NUMBER	**231368–01G**		

TTL associates inc
Environmental, Geotechnical Engineering & Testing



LEGEND

B-001-0-23 APPROXIMATE TEST BORING LOCATION

APPROXIMATE SCALE – FEET

0 15 30

BASE DRAWING PROVIDED VIA EMAIL FROM ODOT DISTRICT 2 ON MAY 25, 2023.

PLATE 2.0
TEST BORING LOCATION PLAN
WOO–582–16.35, PID 107712
PROPOSED CULVERT REPLACEMENT
TROY TOWNSHIP, WOOD COUNTY, OHIO

PREPARED FOR
ODOT DISTRICT 2
BOWLING GREEN, OHIO

DRAWN	TRR/7–14–23	CHECKED	KCH/7–14–23
REVISED		APPROVED	
JOB NO.	231368		
DRAWING NUMBER	231368–02G		

TTL associates inc
Environmental, Geotechnical Engineering & Testing

FIGURES

PROJECT: WOO-582-16.35 TYPE: CULVERT PID: 107717 SFN: 8707170 START: 7/12/23 END: 7/12/23	DRILLING FIRM / OPERATOR: TTL / TB SAMPLING FIRM / LOGGER: TTL / KKC DRILLING METHOD: 3.25" HSA / NQ2 SAMPLING METHOD: SPT / ST / NQ	DRILL RIG: CME 550X ATV HAMMER: CME AUTOMATIC CALIBRATION DATE: 2/20/23 ENERGY RATIO (%): 75.2	STATION / OFFSET: 862+76, 5' LT. ALIGNMENT: SR 582 ELEVATION: 646.0 (NAVD88) EOB: 29.7 ft. LAT / LONG: 41.450113, -83.453827	EXPLORATION ID B-001-0-23 PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 646.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
ASPHALT - 10 INCHES	645.2																	
AGGREGATE BASE - 8 INCHES	644.5																	
STIFF TO VERY STIFF, BROWN/GRAY, **SILTY CLAY**, LITTLE SAND, TRACE GRAVEL, TRACE ORGANICS, DAMP	643.0	1 - 2	8 / 7 / 5	15	89	SS-1	3.50	-	-	-	-	-	-	-	-	14	A-6b (V)	
STIFF TO VERY STIFF, GRAY, **SILTY CLAY**, SOME SAND, TRACE GRAVEL, MOIST		3 - 5	2 / 2 / 3	6	100	SS-2	3.00	-	-	-	-	-	-	-	-	20	A-6b (V)	
@6.0' TO 7.2': γ_{WET}=121.9 PCF, γ_{DRY}=23.7 PCF @6': BROWN/GRAY, TRACE IRON OXIDE STAIN SEAM	638.8	6 - 7			92	ST-3A	1.25	2	4	18	26	50	33	16	17	24	A-6b (11)	
		7 - 8				ST-3B	>4.5	-	-	-	-	-	-	-	-	-		
HARD, BROWN/GRAY, **SILTY CLAY**, SOME SAND, TRACE GRAVEL, TRACE IRON OXIDE STAIN SEAM, DAMP		9 - 10	12 / 12 / 29	51	100	SS-4	>4.5	-	-	-	-	-	-	-	-	11	A-6b (V)	
@11': GRAY	633.0	11 - 12	9 / 15 / 20	44	100	SS-5	>4.5	6	5	17	24	48	30	14	16	11	A-6b (10)	
VERY STIFF, GRAY, **SILTY CLAY**, LITTLE SAND, TRACE GRAVEL, TRACE DOLOMITE FRAGMENTS, TRACE CALCITE STAIN SEAM, DAMP	630.0	14 - 15	7 / 7 / 12	24	100	SS-6	3.75	-	-	-	-	-	-	-	-	11	A-6b (V)	
DENSE, GRAY, **GRAVEL AND STONE FRAGMENTS WITH SAND AND SILT**, TRACE CLAY, TRACE CALCITE STAIN SEAM, DAMP	628.0	16 - 17; W 628.0	5 / 11 / 16	34	89	SS-7	-	40	13	17	28	2	19	11	8	9	A-2-4 (0)	
VERY STIFF TO HARD, GRAY, **SANDY SILT**, SOME CLAY. TRACE GRAVEL, TRACE DOLOMITE FRAGMENTS, MOIST	626.3	19; TR	5 / 7 / 50/2"	-	100	SS-8	3.00	-	-	-	-	-	-	-	-	18	A-4a (V)	
DOLOMITE, GRAY, MODERATELY WEATHERED, STRONG, VUGGY, CRYSTALLINE, JOINTED - MODERATELY FRACTURED TO SLIGHTLY FRACTURED, TIGHT; RQD 84%, REC 100%. @22.9' TO 23.4': HIGHLY FRACTURED TO FRACTURED FRAGMENTS		20 - 25	85		100	NQ2-1											CORE	
@27.5' TO 28.1': HIGHLY FRACTURED TO FRACTURED FRAGMENTS	616.3	25 - 29; EOB	83		100	NQ2-2											CORE	

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.25 BAG ASPHALT PATCH; PUMPED 8 CF CEMENT-BENTONITE GROUT

PROJECT: WOO-582-16.35	DRILLING FIRM / OPERATOR: TTL / TB	DRILL RIG: DIEDRICH D70 TRACK	STATION / OFFSET: 862+75, 7' RT.
TYPE: CULVERT	SAMPLING FIRM / LOGGER: TTL / KKC	HAMMER: DIEDRICH AUTOMATIC	ALIGNMENT: SR 582
PID: 107717 SFN: 8707170	DRILLING METHOD: 3.25" HSA / NQ2	CALIBRATION DATE: 4/13/22	ELEVATION: 646.1 (NAVD88) EOB: 29.9 ft.
START: 7/10/23 END: 7/12/23	SAMPLING METHOD: SPT / ST / NQ	ENERGY RATIO (%): 90	LAT / LONG: 41.449937, -83.453686

EXPLORATION ID B-002-0-23

MATERIAL DESCRIPTION AND NOTES	ELEV. 646.1	DEPTHS
ASPHALT - 8 INCHES	645.4	
AGGREGATE BASE - 9 INCHES	644.7	
STIFF, BROWN/GRAY, **SILTY CLAY**, SOME SAND, TRACE ORGANICS, MOIST	642.6	
STIFF, GRAY/BROWN, **SILTY CLAY**, SOME SAND, TRACE IRON OXIDE STAIN SEAM, TRACE CALCITE STAIN SEAM, DAMP TO MOIST @6': STIFF TO VERY STIFF, BROWN/GRAY, MOIST @6.0' TO 7.5': Qu=3,845 PSF, γ_{WET}=132.7 PCF, γ_{DRY}=111.3 PCF	638.6	
VERY STIFF, BROWN/GRAY, **SILTY CLAY**, SOME SAND, TRACE GRAVEL, TRACE IRON OXIDE STAIN SEAM, DAMP @8.5': HARD, LITTLE SAND @11': GRAY, TRACE CALCITE STAIN SEAM	633.1	▼ (9.5)
VERY STIFF TO HARD, GRAY, **SILTY CLAY**, SOME SAND, TRACE GRAVEL, MOIST	630.1	
VERY DENSE, DARK GRAY/GRAY, **GRAVEL AND STONE FRAGMENTS WITH SAND, SILT, AND CLAY**, DAMP	628.1	W 628.1
HARD, GRAY, **SANDY SILT**, SOME CLAY, TRACE GRAVEL, TRACE DOLOMITE FRAGMENTS, TRACE CALCITE STAIN SEAM, WET (FREE WATER NOTED)	626.2	TR
DOLOMITE, GRAY, MODERATELY WEATHERED, STRONG, VUGGY, CRYSTALLINE, JOINTED - FRACTURED TO MODERATELY FRACTURED, TIGHT; RQD 52%, REC 100%. @19.9' TO 20.4': HIGHLY FRACTURED FRAGMENTS @21.3': HIGHLY FRACTURED FRAGMENTS @23.6' TO 23.7': HIGHLY FRACTURED FRAGMENTS	622.4	
DOLOMITE, GRAY, MODERATELY WEATHERED, STRONG, VUGGY, CRYSTALLINE, JOINTED - MODERATELY FRACTURED TO SLIGHTLY FRACTURED, TIGHT; RQD 100%, REC 100%. @29.1' TO 29.4': FRACTURED SEGMENT	616.2	EOB

SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
6 4 5	14	89	SS-1	>4.5	0	4	19	27	50	34	17	17	20	A-6b (11)	
4 5 5	15	100	SS-2	>4.5	-	-	-	-	-	-	-	-	16	A-6b (V)	
		79	ST-3A	2.50	4	6	18	26	46	32	16	16	19	A-6b (10)	
			ST-3B	3.00	-	-	-	-	-	-	-	-	-		
8 16 23	59	89	SS-4	>4.5	-	-	-	-	-	-	-	-	12	A-6b (V)	
10 20 28	72	89	SS-5	>4.5	-	-	-	-	-	-	-	-	11	A-6b (V)	
5 9 10	29	100	SS-6	>4.5	4	7	18	25	46	31	15	16	16	A-6b (10)	
18 20 24	66	100	SS-7	-	-	-	-	-	-	-	-	-	7	A-2-6 (V)	
50	-	100	SS-8	-	-	-	-	-	-	-	-	-	11	A-4a (V)	
78		100	NQ2-1											CORE	
85		100	NQ2-2											CORE	

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.25 BAG ASPHALT PATCH; PUMPED 8 CF CEMENT-BENTONITE GROUT

LEGEND KEY

LITHOLOGIC SYMBOLS
(Unified Soil Classification System)

A-2-4: Ohio DOT: A-2-4, gravel and/or stone fragments with sand and silt

A-2-6: Ohio DOT: A-2-6, gravel and/or stone fragments with sand, silt and clay

A-4A: Ohio DOT: A-4a, sandy silt

A-6B: Ohio DOT: A-6b, silty clay

DOLOMITE: Ohio DOT: Dolomite

PAVEMENT OR BASE: Ohio DOT: Pavement or Aggregate base

SAMPLER SYMBOLS

Thin Walled Undisturbed Sample

WELL CONSTRUCTION SYMBOLS

Bentonite: Bottom of hole

Asphalt or Concrete Pavement Patch

Notes:

1. Exploratory borings were performed on July 10 and 12, 2023, utilizing 3¼-inch inside diameter hollow-stem augers and NQ2 rock core barrels.

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 test borings were located in the field by TTL based on the Stage 2 drawings. Latitude and longitude coordinates, and ground surface elevations were surveyed by TTL utilizing a handheld GPS device. Stations and offsets were estimated based on the provided Stage 2 drawings.

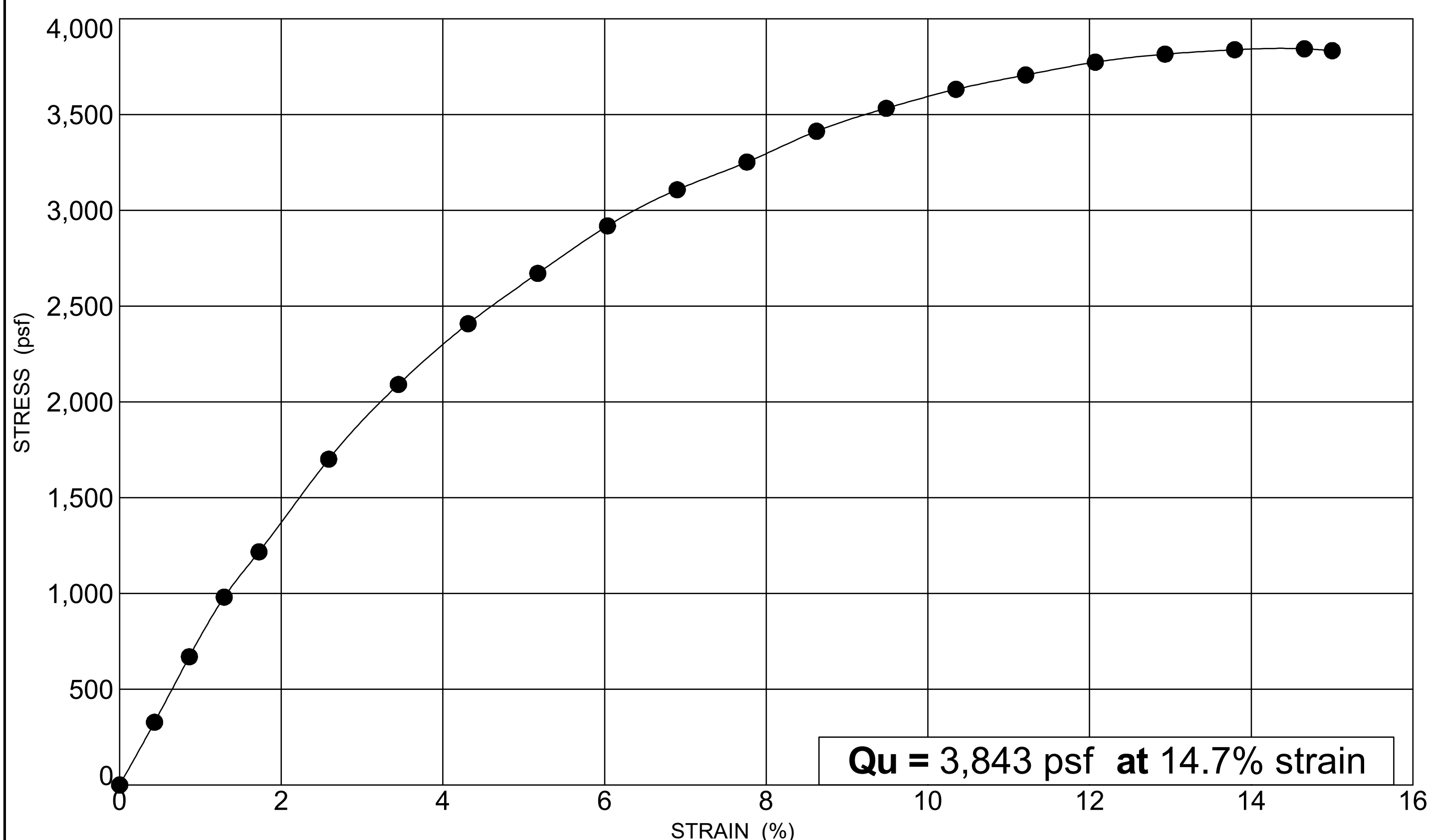
OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

UNCONFINED COMPRESSION TEST
AASHTO T - 208

PROJECT WOO-582-16.35 **PID** 107717
OGE NUMBER N/A **PROJECT TYPE** STRUCTURE FOUNDATION

SAMPLE IDENTIFICATION

BORING ID: B-002-0-23 SAMPLE ID: ST-3A
STATION: 862+75, 7' RT. DEPTH: 6.0 - 7.5 feet



SPECIMEN FAILURE SKETCHES OR PHOTOGRAPHS

FRONT VIEW SIDE VIEW

SPECIMEN DETAILS

HEIGHT: 147.300 mm
DIAMETER: 73.200 mm
WET UNIT WT: 132.65 pcf
DRY UNIT WT: 111.29 pcf

TESTED BY: KKC 7/13/2023

CLASSIFICATION RESULTS

GRADATION (%)

GR	CS	FS	SI	CL
4	6	18	26	46

ATTERBERG LIMITS / MOISTURE

LL	PL	PI	WC
32	16	16	19

ODOT CLASS: A-6b HP (tsf): 2.5
DESCRIPTION:

OHDOT UNCONFINED COMPRESSION - OH DOT.GDT - 7/17/23 10:05 - S:\PROJECTS\231368.GPJ

Ohio Department of Transportation
Division of Engineering

Office of Geotechnical Engineering

B-001-0-23

Core Date: July 12, 2023					Ground Surface Elevation: 646.0'			
Run #:	Depth		Elevation		Recovery		RQD	
NQ2-1	19.7'	24.7'	626.3'	621.3'	60/60	100%	51/60	85%
NQ2-2	24.7'	29.7'	621.3'	616.3'	60/60	100%	50/60	83%
WOO-582-16.35, PID 107717								

Prepared by TTL associates inc
Environmental, Geotechnical Engineering & Testing

TTL Project No.: 231368

Office of Geotechnical Engineering

B-002-0-23

Core Date: July 12, 2023					Ground Surface Elevation: 646.1’			
Run #:	Depth		Elevation		Recovery		RQD	
NQ2-1	19.9’	24.9’	626.2’	621.2’	60/60	100%	47/60	78%
NQ2-2	24.9’	29.9’	621.2’	616.2’	60/60	100%	51/60	85%
WOO-582-16.35, PID 107717								

Prepared by

TTL Project No.: 231368

Appendix A:
Engineering Calculations

GENERAL FOUNDATION INFORMATION:

The headwall footing is shown at the inlet to bear at Elev. 638.57, with a width of 5.5 feet, and a total length of approximate 32.15 feet. At the outlet, the headwall footing is shown to bear at Elev. 638.40, with a width of 6.25 feet, and a total length of approximately 35.21 feet.

Consider the headwall footing for the inlet, which is a smaller footprint, and therefore the conservative one to use for evaluations of bearing capacity.

L = 32.15 feet
B = 5.5 feet

GENERAL SOIL INFORMATION:

Boring	Existing GSE at Boring	Invert Elev.	Df
B-001	646	640.66	2.26
B-002	646.1	640.82	2.25

For both the inlet and outlet, the foundations are expected to bear in:
Stratum II very stiff to hard cohesive soils

Boring	Avg N60	N60*125	Avg HP (tsf)	Avg PI	f1*	Su (psf) by f1*
B-001	40	5000	4.3	16	5.5	4656.3
B-002	53	6625	4.1	16	5.5	6169.6

*f1 by ODOT GDM Table 400-1

USE c =	4	ksf for this analysis

Groundwater

Model groundwater in tributary above foundatation bearing elevation.

STRENTH LIMIT STATE:

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

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

$q_n = cN_{cm} + q_n$ = nominal bearing resistance (ksf) (AASTHO LRFD 10.6.3.1.2a-1)

$N_{cm} = N_c s_c i_c$ (AASTHO LRFD 10.6.3.1.2a-2)

$N_{qm} = N_q s_q d_q i_q$ (AASTHO LRFD 10.6.3.1.2a-3)

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ (AASTHO LRFD 10.6.3.1.2a-4)

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

Setup

Bearing in Very Stiff to Hard cohesive soils

$c =$	4	ksf	
$\phi_f =$	0	degrees	assumed zero in cohesive soil
$N_c =$	5.14	*units*	
$N_q =$	1.0	*units*	for soil with a $\phi_f =$ 0 Degrees
$N_\gamma =$	0.0	*units*	
$\gamma =$	0.073	kcf	(0.135 soil - 0.062 water)
$D_f =$	2.25	ft	
$B =$	5.50	ft	Width
$L =$	32.15	ft	Length; $1.5B + D_f =$ 10.5
$D_w =$	0	ft	highest anticipated groundwater depth
$C_{wq} =$	0.5	*units*	where $D_w = 0.0$
$C_{w\gamma} =$	0.5	*units*	(above D_f)
$s_c =$	1.03	*units*	$s_c = 1 + (B/(5L))$
$s_g =$	1.00	*units*	for $\phi_f = 0$ $s_\gamma = 1$
$s_q =$	1.00	*units*	$s_q = 1$
$d_q =$	1.0	*units*	taken as 1 since cohesive soil
calculation i_c , i_γ , $i_q =$	1.0	*units*	Assumed loaded without inclination

for $\phi_f > 0$: $s_c = 1 + (B/(5L))(Nq/Nc)$; $s_g = 1 - 0.4(B/L)$; $s_q = 1 + ((B/L)\tan(\phi_f))$

$D_f / B =$ 0.409091

$N_{cm} = N_c s_c i_c$ = 5.14 * 1.034 * 1 = 5.315

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

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 0 * 1 * 1 = 0

$cN_{cm} =$ 21.26

$\gamma D_f N_{qm} C_{wq} =$ 0.082

$0.5\gamma B N_{\gamma m} C_{w\gamma} =$ 0

$q_n = cN_{cm} + gD_f N_{qm} C_{wq} + 0.5gBN_{\gamma m} C_{w\gamma}$

= (4*5.315) + (0.0726*2.25*1*0.5) + (0.5*5.5*0*0.5) =

= (21.26) + (0.082) + (0) =

$q_n =$ 21.342 ksf

$\varphi_b =$ 0.5 based on theoretical method (Munfakh et al., 2001), in clay

$q_R = \varphi_b * q_n$ = 0.5 * 21.342 = 10.671 ksf

Factored resistance at the strength limit state for the proposed half height headwall at the inlet is equal to 10.7 ksf

SERVICE LIMIT STATE:

Based on : (Table C10.6.2.6.1-1)

"Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State" Table

Stratum II very stiff to hard cohesive soils

within applicable borings and depths:

Consistency	Soil Type	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use*
"Very Dense"	Lean Clay (CL)	6-12	8

* recomented value based on Table C10.6.2.6.1-1

φ_b = 1

Factored bearing resistance = 8 ksf

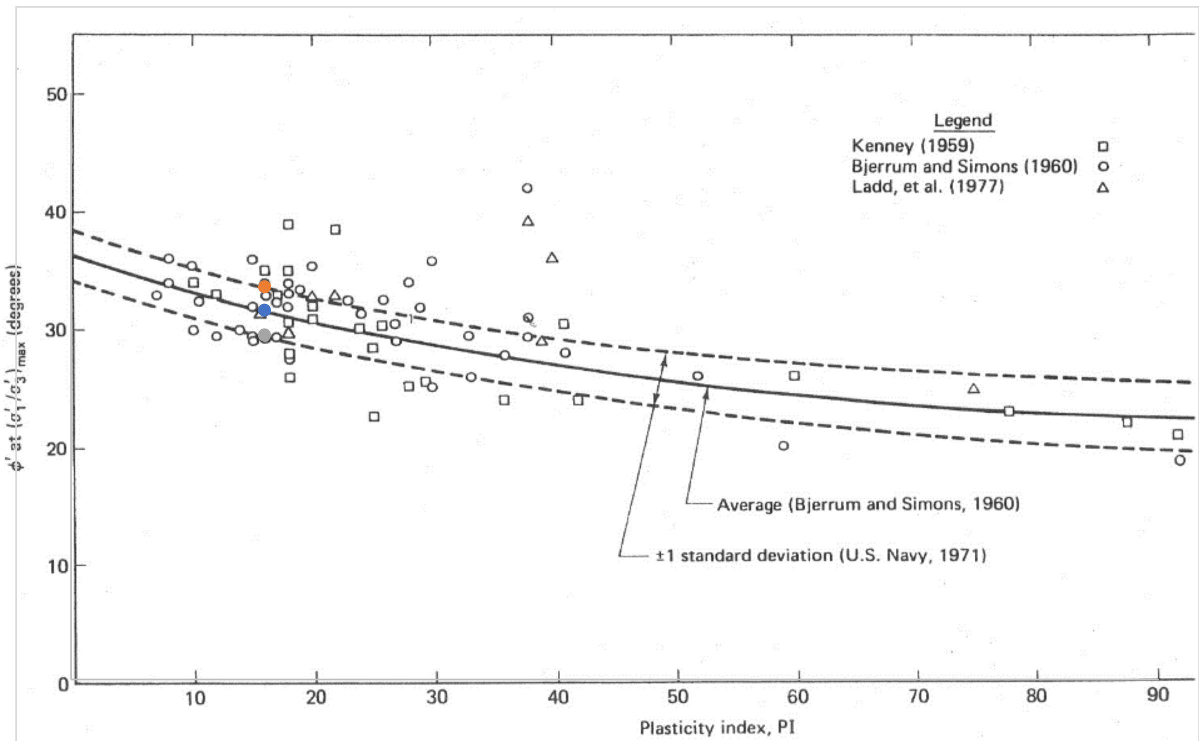
Limit to 4 ksf based on settlement ≤1" (see attached *Settlement Calculation*)

Project	WOO-582-16.35, PID 107717	
By	KCH	
Date	7/19/2023	
Subject	Estimating ϕ' (drained) for native cohesive soils	

Stratum	II	
PI	16	
Average ϕ'	31.5	based on graphical correlations below
low ϕ'	29.4	based on graphical correlations below
high ϕ'	33.5	based on graphical correlations below

compare with a "sandy silt" or a "silt" (A-4a or A-4b)

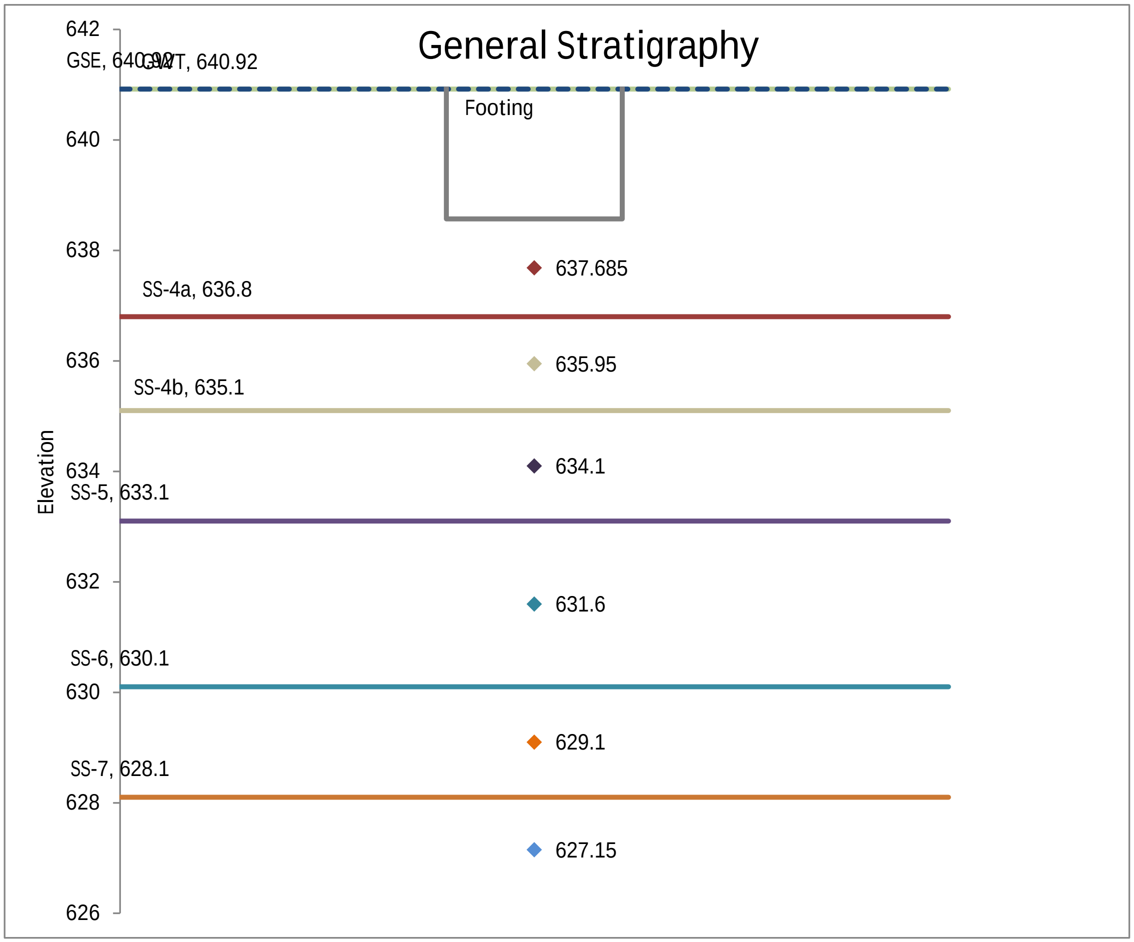
N_{60}	40	
ϕ'	36.5	deg (AASHTO LRFD Table 10.4.6.2.4-1, GDM Table 400-3)
		Linear Inerpolate between phi values and N_{60} values from Table 10.4.6.2.4-1, minus 2.5 degrees



Emperical correlation between ϕ' and PI from triaxial compression tests on normally consolidated undisturbed clays (after US Navy, 1971, and Ladd e. al., 1977)

Project Name: WOO-582-16.35, PID 107717 Boring Number B-002
Project Number: 231368 Analysis Type Rectangular
Calculated by: KCH 7/19/2023

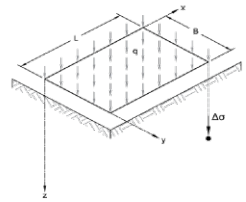
Layer	H (feet)	C_r	e_o	sigma v (psf)	z (feet)	b (feet)	(z-Df) / b	I_z	delta p@	8000 psf	(check) sigma v+ΔP	delta H (inches)		C'	delta H w/C'
SS-4a	1.77	0.012	0.40	235	0.885	6.25	0.1	0.248	7930		8164	0.28		0	-
SS-4b	1.7	0.012	0.40	361	2.62	6.25	0.4	0.217	6951		7312	0.23		0	-
SS-5	2	0.011	0.39	495	4.47	6.25	0.7	0.171	5485		5980	0.21		0	-
SS-6	3	0.016	0.45	677	6.97	6.25	1.1	0.125	4004		4680	0.33		0	-
SS-7	2	-	0.30	863	9.47	6.25	1.5	0.095	3049		3912	-		245	0.06
SS-8	1.9	0.011	0.70	1014	11.42	6.25	1.8	0.079	2526		3540	0.08		0	-



Total delta H (in.)	1.19			
+15%	1.37			
-15%	1.01			

Project Name:	WOO-582-16.35, PID 107717	Boring Number	B-002
Project Number:	231368	Analysis Type	Rectangular
Calculated by:	KCH 7/19/2023		

G (assumed)	2.7	
GSE	640.92	Tributary Bottom
GWT	640.92	At or above Tributary Bottom
Bearing Elev	638.57	
D_f	2.35	ft
Footing Width, B	6.25	ft
Length, L	32.21	ft
P	8000	psf
Rig	Diedrich D70	ER 90



$$\Delta\sigma = qI_B$$

$$I_B = \frac{1}{4\pi}\left[\frac{2|mn|\sqrt{V}}{V+V_1} \times \frac{V+1}{V} + \tan^{-1}\left(\frac{2|mn|\sqrt{V}}{V-V_1}\right) + \beta\right]$$

$$V = m^2 + n^2 + 1$$

$$V_1 = (mn)^2$$

$$\beta \left(\text{when } \tan^{-1}\left(\frac{2|mn|\sqrt{V}}{V-V_1}\right) \le 0\right) = \pi$$

$$\beta \left(\text{when } \tan^{-1}\left(\frac{2|mn|\sqrt{V}}{V-V_1}\right) > 0\right) = 0$$

$$m = \frac{B}{z}$$

$$n = \frac{L}{z}$$

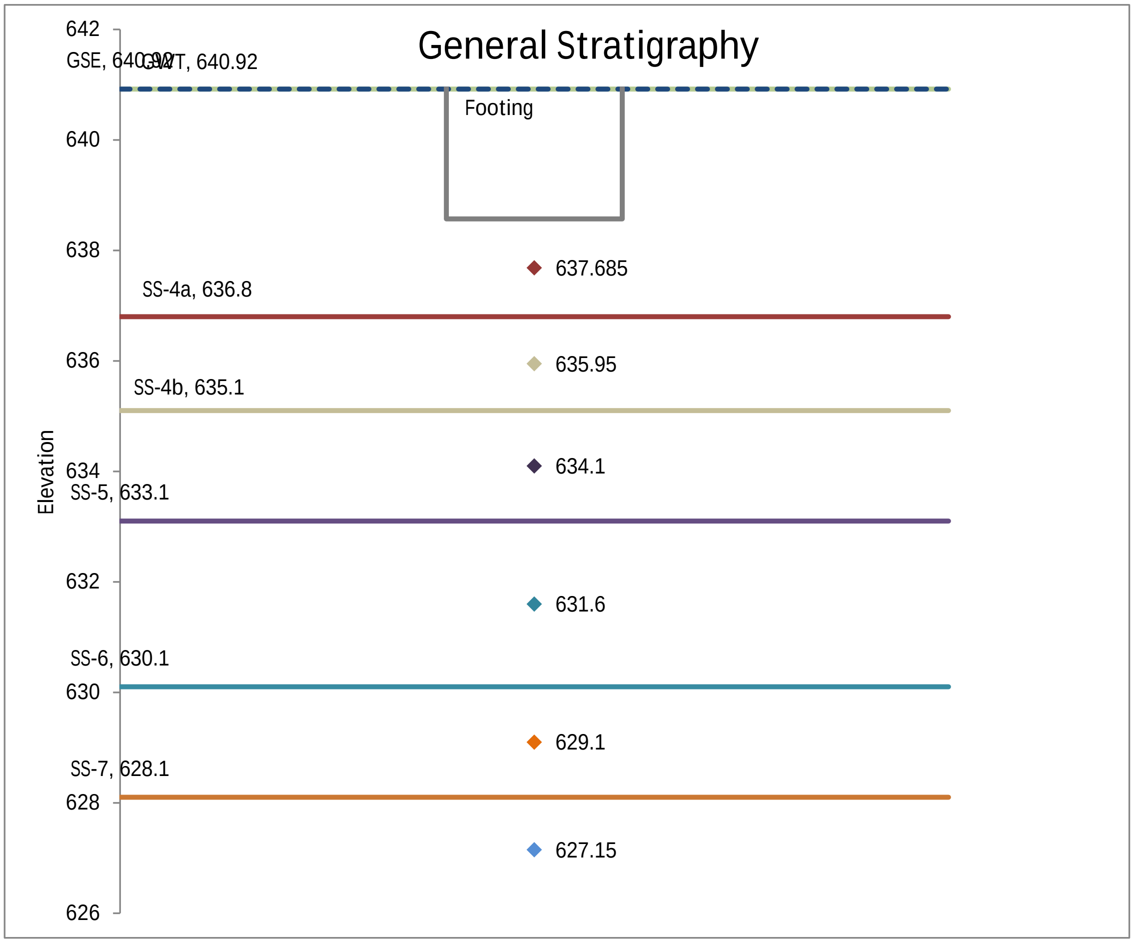
	Bot. Elev.	Centroid (C) Elev.	H (ft)	z below footing	z below GSE	γ_T (pcf)	γ_d (pcf)	H_{GWT-C}	w	e_o	Depth of Influence = (z-D_f)/B	m = 0.5*B/z	n = 0.5*L/z	I_z*	σ_v' (psf)	V	V1	$\tan^{-1}\left(\frac{2\|mn\|\sqrt{V}}{V-V_1}\right)$	Beta	N'/N	N_m	N_{60}	N'	C'
SS-4a	636.8	637.685	1.77	0.885	3.235	135	121	3.235	12	0.40	0.14	3.5	18.2	0.248	235	345	4129	-0.56	3.14	2.63		0	0	
SS-4b	635.1	635.95	1.7	2.62	4.97	135	121	4.97	12	0.40	0.4	1.2	6.1	0.217	361	40	54	-1.43	3.14	2.24		0	0	
SS-5	633.1	634.1	2	4.47	6.82	135	122	6.82	11	0.39	0.7	0.7	3.6	0.171	495	14.5	6.3	1.17	0.00	1.96		0	0	
SS-6	630.1	631.6	3	6.97	9.32	135	116	9.32	16	0.45	1.1	0.4	2.3	0.125	677	6.5	1.1	0.77	0.00	1.68		0	0	
SS-7	628.1	629.1	2	9.47	11.82	140	131	11.82	7	0.30	1.5	0.3	1.7	0.095	863	4.0	0.31	0.55	0.00	1.46	66	99	145	245
SS-8	626.2	627.15	1.9	11.42	13.77	140	99	13.77	11	0.70	1.8	0.27	1.4	0.079	1014	3.1	0.15	0.43	0.00	1.32		0	0	

*Note: Influence factors are multiplied by 4 in calculation of delta p

Project Name: WOO-582-16.35, PID 107717 Boring Number B-002
Project Number: 231368 Analysis Type Rectangular
Calculated by: KCH 7/19/2023

Layer	H (feet)	C_r	e_o	sigma v (psf)	z (feet)	b (feet)	$\frac{(z-Df)}{b}$	I_z	delta p@	4000 psf	(check) sigma v+ΔP	delta H (inches)		C'	delta H w/C'
SS-4a	1.77	0.012	0.40	235	0.885	6.25	0.1	0.248	3965		4200	0.23		0	-
SS-4b	1.7	0.012	0.40	361	2.62	6.25	0.4	0.217	3475		3836	0.18		0	-
SS-5	2	0.011	0.39	495	4.47	6.25	0.7	0.171	2743		3238	0.15		0	-
SS-6	3	0.016	0.45	677	6.97	6.25	1.1	0.125	2002		2678	0.24		0	-
SS-7	2	-	0.30	863	9.47	6.25	1.5	0.095	1524		2388	-		245	0.04
SS-8	1.9	0.011	0.70	1014	11.42	6.25	1.8	0.079	1263		2277	0.05		0	-

Total delta H (in.)	0.89			
+15%	1.03			
-15%	0.76			



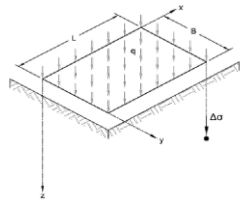
nominal 1 inch or less settlement
for 4000 psf pressure

settlement limited to 1 inch or less

TTL associates inc

Project Name:	WOO-582-16.35, PID 107717	Boring Number	B-002
Project Number:	231368	Analysis Type	Rectangular
Calculated by:	KCH 7/19/2023		

G (assumed)	2.7	
GSE	640.92	Tributary Bottom
GWT	640.92	At or above Tributary Bottom
Bearing Elev	638.57	
D_f	2.35	ft
Footing Width, B	6.25	ft
Length, L	32.21	ft
P	4000	psf
Rig	Diedrich D70	ER 90



$$\Delta\sigma \ldots\ldots = qI_B$$

$$I_B \ldots\ldots = \frac{1}{4\pi}\left[\frac{2|mn|\sqrt{V}}{V+V_1} \times \frac{V+1}{V} + \tan^{-1}\left(\frac{2|mn|\sqrt{V}}{V-V_1}\right) + \beta\right]$$

$$V \ldots\ldots = m^2 + n^2 + 1$$

$$V_1 \ldots\ldots = (mn)^2$$

$$\beta \left(\text{when } \tan^{-1}\left(\frac{2|mn|\sqrt{V}}{V-V_1}\right) \le 0\right) \ldots\ldots = \pi$$

$$\beta \left(\text{when } \tan^{-1}\left(\frac{2|mn|\sqrt{V}}{V-V_1}\right) > 0\right) \ldots\ldots = 0$$

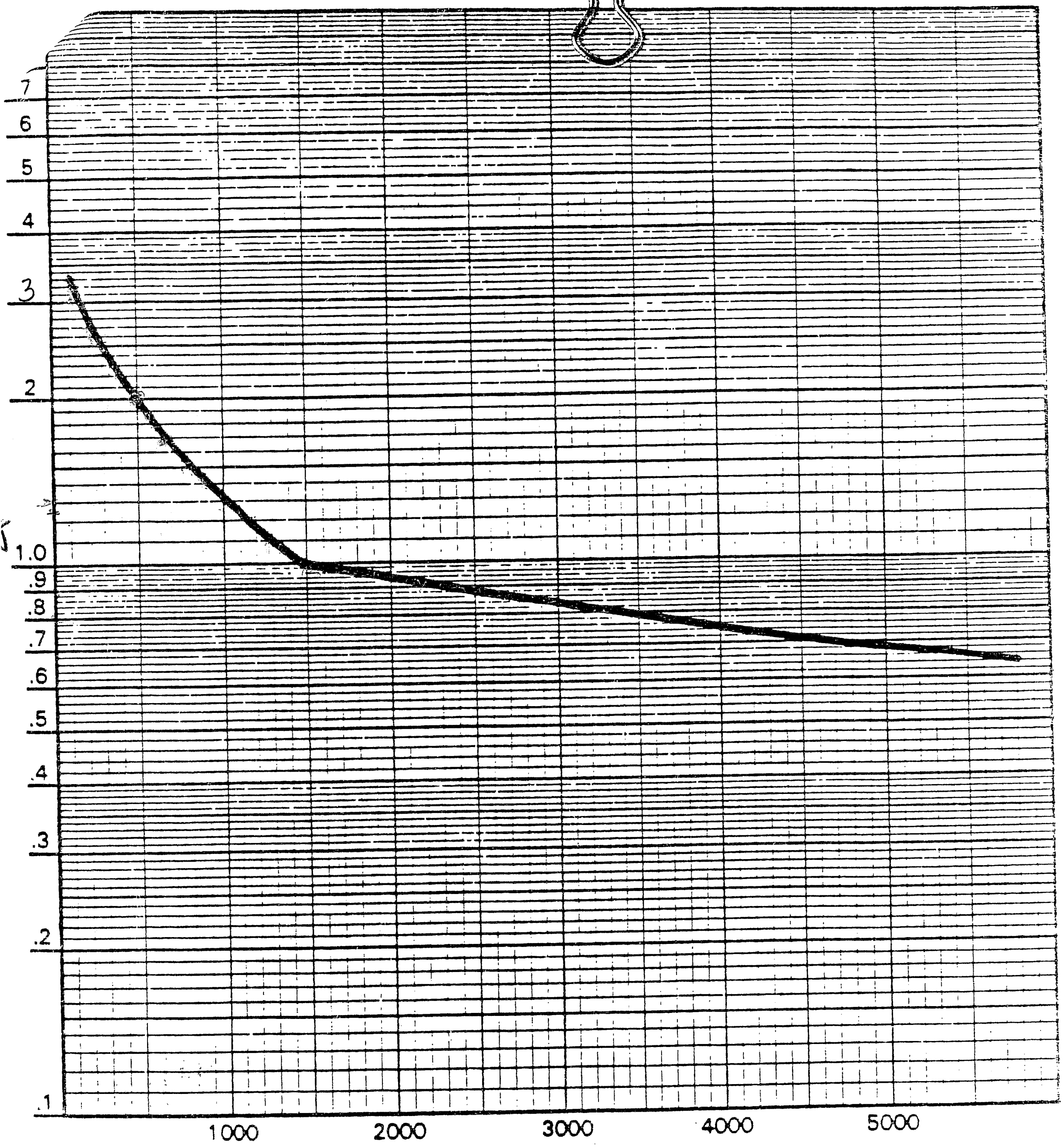
$$m \ldots\ldots = \frac{B}{z}$$

$$n \ldots\ldots = \frac{L}{z}$$

	Bot. Elev.	Centroid (C) Elev.	H (ft)	z below footing	z below GSE	γ_T (pcf)	γ_d (pcf)	H_{GWT-C}	w	e_o	Depth of Influence = $(z-D_f)/B$	m = 0.5*B/z	n = 0.5*L/z	I_z*	σ_v' (psf)	V	V1	$\tan^{-1}\left(\frac{2\lvert mn\rvert\sqrt{V}}{V-V_1}\right)$	Beta	N'/N	N_m	N_{60}	N'	C'
SS-4a	636.8	637.685	1.77	0.885	3.235	135	121	3.235	12	0.40	0.14	3.5	18.2	0.248	235	345	4129	-0.56	3.14	2.63		0	0	
SS-4b	635.1	635.95	1.7	2.62	4.97	135	121	4.97	12	0.40	0.4	1.2	6.1	0.217	361	40	54	-1.43	3.14	2.24		0	0	
SS-5	633.1	634.1	2	4.47	6.82	135	122	6.82	11	0.39	0.7	0.7	3.6	0.171	495	14.5	6.3	1.17	0.00	1.96		0	0	
SS-6	630.1	631.6	3	6.97	9.32	135	116	9.32	16	0.45	1.1	0.4	2.3	0.125	677	6.5	1.1	0.77	0.00	1.68		0	0	
SS-7	628.1	629.1	2	9.47	11.82	140	131	11.82	7	0.30	1.5	0.3	1.7	0.095	863	4.0	0.31	0.55	0.00	1.46	66	99	145	245
SS-8	626.2	627.15	1.9	11.42	13.77	140	99	13.77	11	0.70	1.8	0.27	1.4	0.079	1014	3.1	0.15	0.43	0.00	1.32		0	0	

*Note: Influence factors are multiplied by 4 in calculation of delta p

settlement limited to 1 inch or less

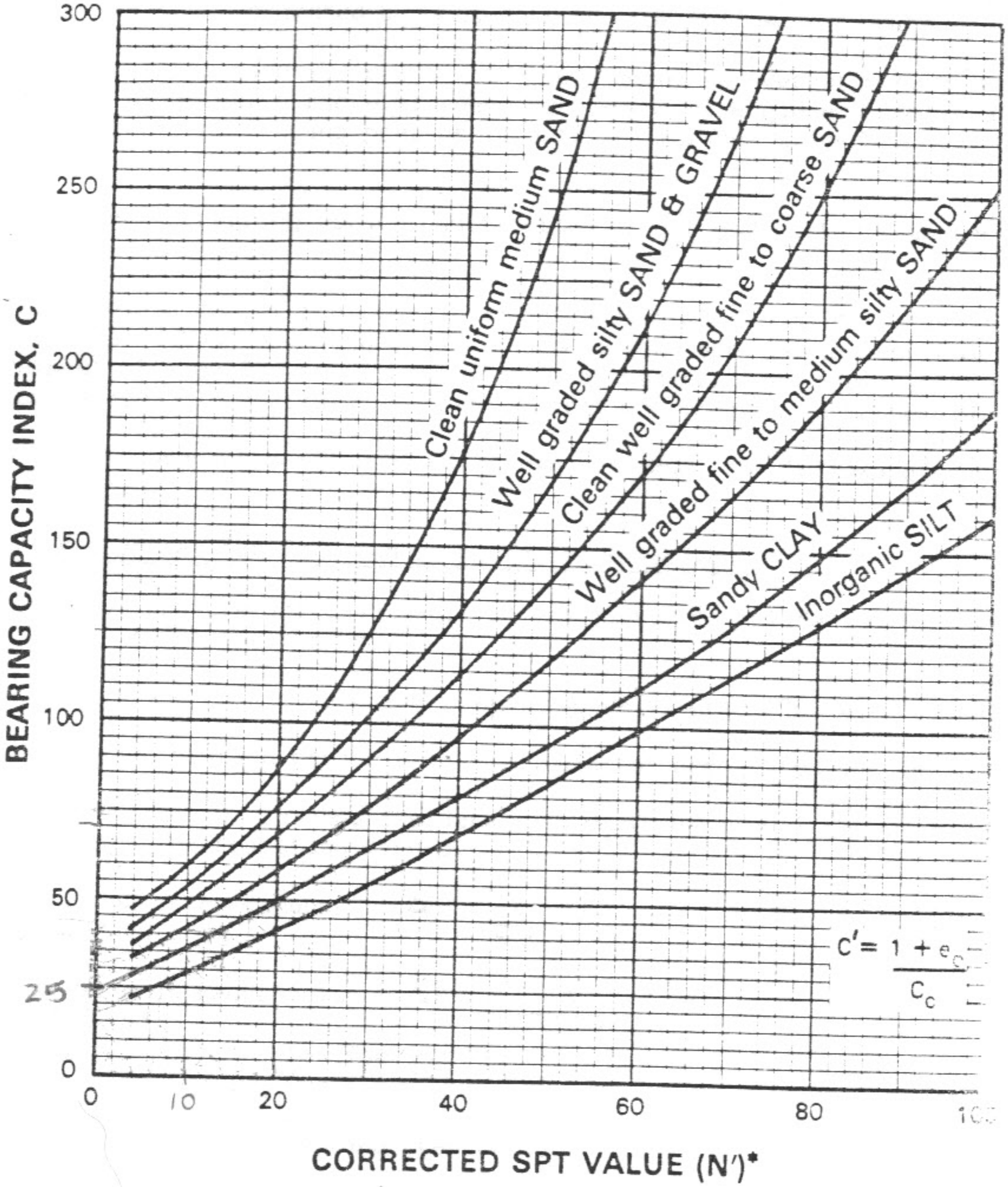


ere: N' = Corrected SPT Blow Count
N = SPT value
P_O = Existing effective vertical overburden pressure

Reference: Based on 1967, Bazaraa, The Use of Standard Penetration Test for Estimating Settlement of Shallow Foundation on Sand.

FIGURE 12 ***CORRECTING SPT (N) BLOW COUNTS***

$\frac{N'}{N} = .99$ $\frac{N'}{N} = 1.3$



*N'—*SPT (N) Value Corrected for Overburden Pressure.*

Reference: Hough, "Compr... as a Basis for So... Value" ASCE 19...

FIGURE 13

BEARING CAPACITY INDEX (C') VALUES FOR GRANUL...

K_p K_a

Lateral Earth Pressure Coefficients

TABLE 5.3. EARTH PRESSURE COEFFICIENTS.

$\phi°$	$\tan\phi$	$\tan(45° + \phi/2)$	$\tan(45° - \phi/2)$	$\tan^2(45° + \phi/2)$	$\tan^2(45° - \phi/2)$
0	0	1.000	1.000	1.000	1.000
1	0.017	1.018	0.983	1.036	0.966
2	0.035	1.036	0.966	1.073	0.933
3	0.052	1.054	0.949	1.111	0.901
4	0.070	1.072	0.933	1.149	0.870
5	0.087	1.091	0.916	1.190	0.839
6	0.105	1.111	0.900	1.234	0.810
7	0.123	1.130	0.885	1.277	0.783
8	0.140	1.150	0.869	1.322	0.755
9	0.158	1.171	0.854	1.371	0.729
10	0.176	1.192	0.839	1.420	0.704
11	0.194	1.213	0.824	1.472	0.680
12	0.213	1.235	0.810	1.525	0.656
13	0.231	1.257	0.795	1.580	0.633
14	0.249	1.280	0.781	1.638	0.610
15	0.268	1.303	0.767	1.698	0.589
16	0.287	1.327	0.754	1.761	0.568
17	0.306	1.351	0.740	1.826	0.548
18	0.325	1.376	0.727	1.894	0.528
19	0.344	1.402	0.713	1.965	0.509
20	0.364	1.428	0.700	2.040	0.490
21	0.384	1.455	0.687	2.117	0.472
22	0.404	1.483	0.675	2.198	0.455
23	0.424	1.511	0.662	2.283	0.438
24	0.445	1.540	0.649	2.371	0.422
25	0.466	1.570	0.637	2.464	0.406
26	0.488	1.600	0.625	2.561	0.390
27	0.510	1.632	0.613	2.676	0.376
28	0.532	1.664	0.601	2.770	0.361
29	0.554	1.698	0.583	2.882	0.347
30	0.577	1.732	0.577	3.000	0.333
31	0.601	1.767	0.566	3.124	0.320
32	0.625	1.804	0.554	3.255	0.307
33	0.649	1.842	0.543	3.392	0.295
34	0.675	1.881	0.532	3.537	0.283
35	0.700	1.921	0.521	3.690	0.271
36	0.727	1.963	0.510	3.852	0.260
37	0.754	2.006	0.499	4.023	0.249
38	0.781	2.050	0.488	4.204	0.238
39	0.810	2.097	0.477	4.395	0.228
40	0.839	2.145	0.466	4.599	0.217
41	0.869	2.194	0.456	4.815	0.208
42	0.900	2.246	0.445	5.045	0.198
43	0.933	2.300	0.435	5.289	0.189
44	0.968	2.356	0.424	5.550	0.180
45	1.000	2.414	0.414	5.828	0.172
46	1.036	2.475	0.404	6.126	0.163
47	1.072	2.539	0.304	6.445	0.155
48	1.111	2.605	0.384	6.786	0.147

Assume Clay in this area except Tricon

$K_o = 1 - \sin 30 = 0.50$

Tricon 2.4 K_p soft to med stiff clay, 2.0 stiff to hard clay

where σ_v is the vertical stress, acting on a plane parallel to the soil surface, and σ_s is the stress, parallel to the soil surface, acting on a vertical plane. The principal stresses are

$$\sigma_1 = z\gamma \frac{1 + \sin\phi}{1 + \sin\nu}$$

$$\sigma_3 = z\gamma \frac{1 - \sin\phi}{1 + \sin\nu} \tag{5.16}$$

The angle of the sliding surface with the horizontal (Fig. 5.13) is

$$\cos(\alpha_0 - \phi - \omega) = \frac{\sin\omega}{\sin\phi} \tag{5.17}$$

The σ_s value for the passive case is given by

$$\sigma_s = z\gamma \tan^2(45° + \nu/2)\cos\epsilon$$

If $c \neq 0$, the distribution of the stresses is the depth and the sliding surfaces are not p responding formulas are rather cumbersome a graphical procedure is more advantageou distribution of the conjugate stresses along and the values and the directions of the p for both the active and passive case.

The graphical solution ... w

Fig. 5 ... be
lines ... ga
curves ... y
into st ... y
straigh ...
cubic ...

Select Granular

$K_a = 0.25$, $\gamma_T = 120$

331942 African Safari

Rpts/Retwall

5.3 ST... IN...

For cases in which the material has no ... ure condition is expressed by $\tau_s = \sigma \tan$ ... Table 1), stresses will be developed by ... on the boundaries of the mass. The st... applied to cases in which the weight ... negligible in comparison with these exte... The basic problem is the determinat... load on a weightless truncated wedge distributed load on its sides (Fig. 5.15) to produce plastic state in the whole we...

$$p = p_0 K_p e^{2\vartheta \tan\phi} + c \cot\phi\,(K_p \ldots$$

The quantity p/c is shown in Fig. ... the angle ϑ, for $\phi = 30°$, for slip surf... cases ($\vartheta = 0, 90°, 180°$) and $p_0 = 0$; th... confined compression strength, the ... strip footing (Prandtl's formula), and ... of a cleft. The numerical values of p ... cial cases can be found in Table 5.4.

If the uniformly distributed load ... the vertical, then p/c is given by the functions of ϕ and ϵ. The angle α_c becomes equal to 0° if

$$\tan\epsilon_l = \tan\phi \frac{e^{[(\pi/2)-\phi]}}{e^{[(\pi/2)-\phi]}\tan \ldots}$$

If $\epsilon = \epsilon_l$, a sliding in the horizonta... values of p/c for the range $0 \leq \epsilon \leq$ ... as functions of $\tan\epsilon$ in Fig. 5.18.

TABLE 5.4. VALUES OF p/c FO... OF INTERNAL FRICTION.

ϕ	$\vartheta = 0$	$\vartheta = \pi/$...
0°	2.000	5.1...
10	2.384	8....
20	2.860	14....
25	3.142	20....
30	3.464	30....
35	3.842	46.
40	4.290	74
45	4.828	133

Appendix B:
Geotechnical Engineering Design Checklists

I. Geotechnical Design Checklists

Project: WOO-582-16.35	PDP Path:
PID: 107717	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-582-16.35	PID: 107717	Reviewer: KCH	Date: 7/19/2023

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	Y	
c.	estimated amount of rock and soil, including the total for each for the entire program.	Y	
Planning – Exploration Number		**(Y/N/X)**	**Notes:**
11	Have the coordinates, stations and offsets of all explorations (borings, 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)		

IV.A Foundations of Structures Checklist

C-R-S: WOO-582-16.35	PID: 107717	Reviewer: KCH	Date: 7/19/2023

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?	X	
	Check method used:		
	laboratory shear tests		
	other (describe other methods)		
Spread Footings		(Y/N/X)	Notes:
4	Are there spread footings on the project? If no, go to Question 11	Y	
5	Have the recommended bottom of footing elevation and reason for this recommendation been provided?	Y	
a.	Has the recommended bottom of footing elevation taken scour from streams or other water flow into account?	x	not a requirement of culverts
6	Were representative sections analyzed for the entire length of the structure for the following:	Y	
a.	factored bearing resistance?	Y	
b.	factored sliding resistance?	N	Recommended soil parameters provided
c.	eccentric load limitations (overturning)?	N	
d.	predicted settlement?	Y	
e.	overall (global) stability?	N	
7	Has the need for a shear key been evaluated?	N	
a.	If needed, have the details been included in the plans?	X	Plans to be prepared by others.
8	If special conditions exist (e.g. geometry, sloping rock, varying soil conditions), was the bottom of footing “stepped” to accommodate them?	X	Conditions not present.
9	Have the Service I and Maximum Strength Limit States for bearing pressure on soil or rock been provided?	Y	

IV.A Foundations of Structures Checklist

Spread Footings		(Y/N/X)	Notes:
10	If weak soil is present at the proposed foundation level, has the removal / treatment of this soil been developed and included in the plans?	N	Plans to be prepared by others. Removal/treatment of weak soils recommended in Geotechnical Report.
a.	Have the procedure and quantities related to this removal / treatment been included in the plans?	X	See response for Item 10, above.
Pile Structures		**(Y/N/X)**	**Notes:**
11	Are there piles on the project? If no, go to Question 17	N	
12	Has an appropriate pile type been selected?		
	Check the type selected:		
	H-pile (driven)		
	H-pile (prebored)		
	Cast In-place Reinforced Concrete Pipe		
	Micropile		
	Continuous Flight Auger (CFA)		
	other (describe other types)		
13	Have the estimated pile length or tip elevation and section (diameter) based on either the Ultimate Bearing Value (UBV) or the depth to top of bedrock been specified? Indicate method used.		
14	If scour is predicted, has pile resistance in the scour zone been neglected?		
15	Has a wave equation drivability analysis been performed as per BDM 305.4.1.2 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.	N	
21	Have the drilled shaft diameter and embedment length been specified?		
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?		
23	For shafts undergoing lateral loading, have the following been determined:		
a.	total factored lateral shear?		
b.	total factored bending moment?		
c.	maximum deflection?		
d.	reinforcement design?		
24	If a bedrock socket is required, has a minimum rock socket length equal to 1.5 times the rock socket diameter been used, as per BDM 305.5.2?		
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?		
26	If scour is predicted, has shaft resistance in the scour zone been neglected?		
27	Has the site been assessed for groundwater influence?		
a.	If yes, and if artesian flow is a potential concern, does the design address control of groundwater flow during construction?		
28	Have all the proper items been included in the plans for integrity testing?		
29	If special construction features (e.g., slurry, casing, load tests) are required, have all the proper items been included in the plans?		
30	If necessary, have wet construction methods been specified?		
General		(Y/N/X)	Notes:
31	Has the need for load testing of the foundations been evaluated?	X	
a.	If needed, have details and plan notes for load testing been included in the plans?		

VI.B. Geotechnical Reports

C-R-S: WOO-582-16.35	PID: 107717	Reviewer: KCH	Date: 7/19/2023

General		(Y/N/X)	Notes:
1	Has an electronic copy of all geotechnical submissions been provided to the District Geotechnical Engineer (DGE)?	Y	
2	Has the first complete version of a geotechnical report being submitted been labeled as 'Draft'?	Y	
3	Subsequent to ODOT's review and approval, has the complete version of the revised geotechnical report being submitted been labeled 'Final'?	Y	
4	Has the boring data been submitted in a native format that is DIGGS (Data Interchange for Geotechnical and Geoenvironmental) compatable? gINT files may be used for this.	Y	
5	Does the report cover format follow ODOT's Brand and Identity Guidelines Report Standards found at http://www.dot.state.oh.us/brand/Pages/default.aspx ?	Y	
6	Have all geotechnical reports being submitted been titled correctly as prescribed in Section 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	