STRUCTURE FOUNDATION EXPLORATION

LUC-64-8.49, PID 96000

Proposed Dam Pipe and Headwall Replacement

SR 64, South of Monclova Road (TR 95)

Swanton Township, Lucas County, Ohio

Submitted to *ODOT District 2*

Date *August 2024*

Prepared by

TTL associates inc
Environmental, Geotechnical Engineering & Testing

Ohio Department of Transportation

1915 North 12th Street
Toledo, OH 43604-5305
T 419-324-2222
F 419-241-1808
www.ttlassoc.com

August 1, 2024

TTL Project No. 232085

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

FINAL Rev1 Report
Structure Foundation Exploration
LUC-64-8.49, PID 96000
Proposed Dam Pipe and Headwall Replacement
SR 64, South of Monclova Road (TR 95)
Swanton Township, Lucas County, Ohio

Dear Ms. Summersett:

Following is the revised 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. P232085R, dated October 5, 2023, and was authorized by ODOT Agreement No. 37607, dated October 23, 2023, with Encumbrance number 741859.

A "draft" version of this report was provided January 29, 2024. On July 9, 2024, it was indicated that there were no comments regarding the draft report. This final report contains the results of our study, our engineering interpretation of the results with respect to the project characteristics, as well as our design and construction recommendations for the replacement headwalls. This revision incorporates updated ground surface elevations at boring locations based on survey information from the ORD plan and profile.

The soil samples collected during this exploration will be stored at our laboratory through completion and ODOT approval of Stage 2 plans. 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.

Katherine C. Hennicken, P.E.
Senior Geotechnical Engineer

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

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

H:\2023\232085\PHASE\01 Geotech\Reports and Other Deliverables\232085 FINAL Rev1 Geotech Report - LUC-64-0.85 PID 96000 Dam Pipe and Headwall Replacement.docx

FINAL REV1 REPORT
STRUCTURE FOUNDATION EXPLORATION
LUC-64-8.49, PID 96000
PROPOSED DAM PIPE AND HEADWALL REPLACEMENT
SR 64, SOUTH OF MONCLOVA ROAD (TR 95)
SWANTON TOWNSHIP, LUCAS COUNTY, OHIO

FOR

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

SUBMITTED

AUGUST 1, 2024
TTL PROJECT NO. 232085

TTL ASSOCIATES, INC.
1915 NORTH 12TH STREET
TOLEDO, OHIO 43604
(419) 324-2222
(419) 321-6257 FAX

EXECUTIVE SUMMARY

This structure foundation exploration report has been prepared for the proposed replacement of the headwalls for a pipe crossing of a tributary of Swan Creek beneath Waterville Swanton Road (State Route 64) in Swanton Township, Lucas County, Ohio, designated as LUC-64-8.49, PID 96000. This exploration included two test borings, laboratory testing, and engineering evaluations for support for the proposed headwall foundations. A summary of the conclusions and recommendations of this study are as follows:

1. Borings B-001 and B-002 were performed within the existing roadway, and the surface materials encountered consisted of asphalt approximately 9½ inches in thickness, underlain by aggregate base approximately 12½ inches in thickness.

2. Underlying the pavement materials in Borings B-001 and B-002, existing granular **fill** materials were encountered to depths of 11 feet and 8½ feet (approximate Elevs. 657 and 659), respectively. The existing fill materials consisted of fine sand or coarse and fine sand. A zone of cohesive existing fill materials was encountered immediately below the granular existing fill materials in Boring B-002 to a depth of 11 feet (approximate Elev. 657). The cohesive existing fill materials consisted of silt and clay.

3. Based on the results of our field and laboratory tests, the subsoils encountered underlying the existing fill materials can be generally described as stratum of granular alluvium underlain by two strata of native cohesive soils with varying strength and moisture characteristics. **Stratum I** consisted of **organic-containing loose** native granular soils interpreted as alluvium encountered underlying the existing fill materials in Borings B-001 and B-002 to depths of 18½ feet and 15 feet below roadway grades (approximate Elevs. 650 and 653), respectively. The granular soils consisted of fine sand (ODOT A-3), as well as coarse and fine sand (ODOT A-3a). **Stratum II** consisted of predominantly medium stiff to stiff cohesive soils encountered underlying Stratum I in Borings B-001 and B-002 to depths of 33½ feet and 38½ feet (approximate Elevs. 635 and 630), respectively. The Stratum II cohesive soils consisted of sandy silt (ODOT A-4a), silty clay (ODOT A-6b), as well as silt and clay (ODOT A-6a). **Stratum III** consisted of predominantly stiff to very stiff cohesive soils encountered underlying Stratum II in both borings extending to termination at a depth of 45 feet. The Stratum III cohesive soils consisted of silty clay (ODOT A-6b).

4. During this exploration, groundwater was initially encountered during drilling in each boring at a depth of 7 feet below roadway grade (approximate Elev. 661). Groundwater was not observed upon completion of drilling within either borehole. Based on the soil characteristics and groundwater conditions encountered in the borings, it is our opinion that the “normal” groundwater level will generally be encountered at or slightly above the water level of the tributary, corresponding to a depth of approximately 7 feet or deeper (approximate Elev. 661 or lower) below roadway grades at the time of this investigation.

5. Based on the conditions encountered in the borings, the soils encountered at the anticipated headwall foundation elevations are anticipated to consist of existing fill materials, organic-laden granular soils, as well as granular soils with trace or less organics. Some zones of

soft cohesive soils may also be encountered. Where granular existing fill materials, organic-laden granular soils, **soft** cohesive soils, or other unsuitable bearing soils are encountered during culvert and headwall installation, over-excavation should extend through these materials to suitable bearing soils. The existing soil conditions are not considered suitable for standard headwalls, unless over-excavation and replacement with new engineered fill is provided. Alternatively, use of a deep foundation system may be considered. Additional guidance and discussion is presented in Section 5.1.

6. Recommended soil parameters for use in temporary braced excavation design by others are provided in Section 5.2.4.

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.

TABLE OF CONTENTS

Page No.

TABLE OF CONTENTS (CONT.)

1.0 INTRODUCTION

This structure foundation exploration report has been prepared for the proposed replacement of the headwalls for a pipe crossing of a tributary of Swan Creek beneath Waterville Swanton Road (State Route 64) in Swanton Township, Lucas County, Ohio, designated as LUC-64-8.49, PID 96000. The pipe crossing is located approximately 900 feet south of the intersection with Monclova Road (Township Road 95), as shown on the attached Site Location Map (Plate 1.0).

This study was performed in accordance with TTL Proposal No. P232085R, dated October 5, 2023, and was authorized by Ohio Department of Transportation Agreement No. 37607, dated October 23, 2023, with Encumbrance number 741859.

1.1 <u>Purpose and Scope of Exploration</u>

The purpose of this exploration was to evaluate the subsurface conditions relative to suitability of use of standard headwalls, average soil properties for use by others in temporary bracing design, as well as OSHA temporary excavation slope requirements for the headwall replacement project at the referenced location. To accomplish this, TTL performed 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.

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 replacement headwalls.

This report includes:

- A description of the existing surface cover, subsurface soils, and groundwater conditions encountered in the borings.
- Design recommendations for headwall support.
- Recommendations concerning soil- and groundwater-related construction procedures such as site preparation, earthwork, headwall 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 planned improvements consist of drainage pipe and headwall replacement for the dam for a tributary of Swan Creek at the project site. It is planned to replace the headwalls with ODOT standard headwalls (pending confirmation of suitable soil conditions). It was indicated that sheetpiling/cofferdams may be required for the proposed replacement project.

Based on the provided historic plan drawings for the existing dam pipe and headwall, the pipe inverts are indicated to be approximate Elev. 657. The existing culvert consists of a box stone structure, with an opening of approximately 4 feet by three feet in area below the road as shown on plan drawings from 1921. More recent drawings show a conduit inserted into each opening of the structure below the road, extending out to the toes of the dam, and supported on stone headwall foundations.

It is assumed the headwalls will be designed as full-height headwalls, for which required minimum foundation soil properties for use of standard headwalls requires an angle of internal friction (ϕ) of 28 degrees or a shear strength (s_u) of 1,500 pounds per square foot, as shown on ODOT standard drawings HW 1.1 and HWDD-1.

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) Division of Geological Survey indicate that the project site is located within the Huron-Erie Lake Plains Section of physiographic regions in Ohio, specifically in the Maumee Sand Plains Region.

The Maumee Sand Plains consist of late Wisconsinan-age sand overlying lacustrine deposits and clay till. At the project site, alluvial deposits associated with the tributary of Swan Creek are also present.

Sandy beach lacustrine deposits are typically encountered overlying the more predominant lacustrine silts and clays. The lacustrine soils are generally characterized as mostly soft to medium stiff silts and 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.

The glacial tills, also referred to as moraine, were 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 tills, it is not uncommon to encounter cobbles, boulders, and seams of granular soils, which may or may not be water bearing.

Bedrock at the site is Devonian-age, broadly mapped as Olentangy Limestone and Shale in this area of Lucas County, Ohio. In particular, the upper bedrock is mapped as Tenmile Creek dolomite. Based on available bedrock topography maps, the top of bedrock was mapped near approximate Elev. 600, approximately 60 to 70 feet below existing roadway grades.

Based on the ODNR mining maps, no mining is indicated in the project area. Based on the ODNR Ohio Karst Areas map, the site is not located in an area of probable karst.

2.1.1 Generalized Near-Surface Soils

The USDA Natural Resources Conservation Services (NRCS) Web Soil Survey indicates that soils in the project area are mapped as Oakville fine sand. The Oakville fine sands formed in ridges on moraines, beach ridges on lake plains, beach ridges on outwash plains, dunes on moraines, dunes on lake plains, as well as dunes on outwash plains, and consist of Sandy eolian deposits. These soils are considered well drained with very high permeabilities.

2.2 Site Reconnaissance

TTL performed site reconnaissance on December 6, 2023. The site is located within the Oak Openings Metropark, and is predominantly wooded.

In the area of the existing dam, the existing asphalt pavement appeared in poor to condition through the center of the pavement, with less poor conditions toward the shoulders. Notably, sealed longitudinal and semi-transverse cracks were visible throughout the center portion of the pavement. The shoulder areas exhibited some longitudinal cracking and minor crumbling of the edges. The slopes of the dam beyond the shoulders appeared in generally good condition with maintained grass cover, and minimal vegetation at the water's edge.

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 December 27, 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 in the southbound lane of State Route 64 (SR 64). Boring B-002 was located in the northbound lane of SR 64. The existing site features and locations of the borings are presented on the Test Boring Location Plan (Plate 2.0).

Latitude and Longitude at the boring locations were surveyed by TTL via a hand-held GPS. Ground surface elevations at the boring locations have been assigned based on the provided ORD plan and profile, since the handheld GPS ground surface elevation was found to be too low compared to survey-level grades available in the ORD file. Stationing and offsets were estimated from the provided plan based on surrounding site features. These data are presented on the logs of test borings.

Borings B-001 and B-002 were planned as Type E3b structure borings per geotechnical investigative procedures outlined in Ohio Department of Transportation (ODOT) "Specifications for Geotechnical Explorations" (SGE). Each of the borings were terminated at the planned depth of 45 feet below the roadway elevation (approximate Elev. 623), meeting the request of borings extending at least to Elev. 626.

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 at 2½-foot intervals to a depth of 35 feet, and at 5-foot intervals thereafter to boring termination. 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 75 truck-mounted drill rig utilized in this project was 72.9 percent, based on calibration on February 20, 2023. 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 (31 to 33 feet) and B-002 (28 to 30 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.

Soil conditions encountered in the test borings are presented in the Logs of Test Borings, along with information related to sample data, SPT results, water conditions observed in the borings, and laboratory test data. In conjunction with published data and typical correlations, the N_{60}-values can be evaluated as a measure of soil compactness/consistency as well as shear strength.

Field and laboratory data were incorporated into gINT™ software for presentation purposes. It should be noted that these logs have been prepared on the basis of laboratory classification and testing as well as field logs of the encountered soils 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 selected 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. Organic determinations by Loss-on-Ignition (ASTM D 2974) were performed on two samples from Boring B-001 (SS-6 and SS-7). These test results are presented on the Logs of Test Borings and Unconfined Compression Test sheet.

A single-point unconsolidated-undrained (UU) triaxial compressive strength test (ASTM D 2850) was performed on the intact cohesive Shelby tube sample from Boring B-001 (ST-13). This test was performed using a confining stress corresponding to the approximate overburden pressure (effective vertical stress) of the sample internal midpoint. The results of this test are presented on the Logs of Test Borings and UU data sheets attached to this report.

4.0 FINDINGS

4.1 General Site Conditions

Borings B-001 and B-002 were performed within the existing roadway, and the surface materials encountered consisted of asphalt approximately 9½ inches in thickness, underlain by aggregate base approximately 12½ inches in thickness.

Underlying the pavement materials in Borings B-001 and B-002, existing granular **fill** materials were encountered to depths of 11 feet and 8½ feet (approximate Elevs. 657 and 659), respectively. The existing fill materials consisted of fine sand or coarse and fine sand. Non-soil materials within the existing fill materials consisted of coal fragments and wood, in trace quantities. Within the granular existing fill materials, SPT N_{60}-values ranged from 4 to 12 blows per foot (bpf), indicating **very loose** to medium dense compactness. Moisture contents ranged from 8 to 23 percent.

A zone of cohesive existing **fill** materials was encountered immediately below the granular existing fill materials in Boring B-002 to a depth of 11 feet (approximate Elev. 657). The cohesive existing fill materials consisted of silt and clay. Non-soil materials within the existing fill materials consisted of organics, in trace quantities. An SPT N_{60}-value of 2 blows per foot (bpf), indicating **soft** consistency, and a moisture content of 31 percent were determined for the sample obtained from this zone.

4.2 General Soil Conditions

Based on the results of our field and laboratory tests, the subsoils encountered underlying the existing fill materials can generally be described as a stratum of granular alluvium underlain by two strata of native cohesive soils with varying strength and moisture characteristics.

Stratum I consisted of **organic-containing loose** native granular soils interpreted as alluvium encountered underlying the existing fill materials in Borings B-001 and B-002 to depths of 18½ feet and 15 feet below roadway grades (approximate Elevs. 650 and 653), respectively. The granular soils consisted of fine sand (ODOT A-3), as well as coarse and fine sand (ODOT A-3a). SPT N_{60}-values ranged from 5 to 10 blows per foot (bpf). Moisture contents were on the order of 22 to 23 percent for samples which did not contain organics, and were approximately 68 percent and 83 percent for samples which were highly organic (SS-6 and SS-7 from Boring B-001). Within the highly organic samples, organic contents of 11.8 percent and 20.7 percent were determined.

Stratum II consisted of predominantly medium stiff to stiff cohesive soils encountered underlying Stratum I in Borings B-001 and B-002 to depths of 33½ feet and 38½ feet (approximate Elevs. 635 and 630), respectively. The Stratum II cohesive soils consisted of sandy silt (ODOT A-4a), silty clay (ODOT A-6b), as well as silt and clay (ODOT A-6a). SPT N_{60}-values typically ranged from 5 to 13 bpf. Unconfined compressive strengths generally ranged from 1,500 to 4,000 psf. Lower unconfined compressive strengths on the order of 500 psf were determined for two samples near the top of this stratum, indicative of **soft** consistency. Moisture contents varied from 21 to 34 percent.

Stratum III consisted of stiff to very stiff cohesive soils encountered underlying Stratum II. Borings B-001 and B-002 were terminated within Stratum III at a depth of 45 feet. The Stratum III cohesive soils consisted of silty clay (ODOT A-6b). SPT N_{60}-values ranged from 15 to 19 bpf. Unconfined compressive strengths generally ranged from 3,000 to 4,500 psf. Moisture contents varied from 15 to 18 percent.

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 7 feet below roadway grade (approximate Elev. 661). Groundwater was not observed upon completion of drilling within either borehole. 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.

Based on the soil characteristics and groundwater conditions encountered in the borings, it is our opinion that the "normal" groundwater level will generally be encountered at or slightly above the water level of the tributary, corresponding to a depth of approximately 7 feet or deeper (approximate Elev. 661 or lower) below roadway grades at the time of this investigation. 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 at the anticipated headwall foundation elevations are anticipated to consist of existing **fill** materials, **organic-laden** granular soils, **soft** cohesive soils, as well as granular soils with trace or less organics. Where granular existing fill materials, organic-laden granular soils, soft cohesive soils, or other unsuitable bearing soils are encountered during headwall installation, over-excavation should extend through these materials to suitable bearing soils. The foundation bearing soils were found to not be suitable for standard headwalls, unless over-excavation and replacement with new engineered fill is provided. Alternatively, use of a deep foundation system may be considered. Additional guidance and discussion is presented in Section 5.1.

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 Soil Parameters for Headwall Support

It was indicated that ODOT standard headwalls are planned to be utilized for this project. Existing pipe inverts are indicated at approximate Elev. 657, corresponding to approximately 11 feet below existing roadway grades. Headwall footings are assumed to bear slightly deeper than the inverts, corresponding to approximately 12 to 17 feet below existing roadway grades.

Existing fill materials, which are unsuitable for support of the proposed headwall foundations, extended to approximate Elev. 657 in each of the borings. Below this elevation in Boring B-001, **organic-laden** soils extended to approximate Elev. 650. These soils would also be considered unsuitable for support of the headwall foundations. Additionally, **soft** cohesive soils were encountered underlying the fill in Boring B-002 to Elev. 657± and underlaying the organic-laden soils in Boring B-001 to Elev. 648±. If within 4 times the headwall foundation width of the bearing elevation, these soils should also be over-excavated and replaced with new engineered fill as described below.

Where unsuitable bearing soils are encountered during headwall installation, over-excavation should extend through these materials to suitable bearing soils. The base of the over-excavation should be widened 1 foot for every foot of depth, centered longitudinally along the headwall. The over-excavated areas should be backfilled with dense-graded aggregate. The aggregate should be placed and compacted as described in Section 5.2.6. Alternatively, the over-excavated areas could be backfilled with flowable controlled-density fill having a minimum compressive strength of 300 psi.

Due to the depth of unsuitable bearing materials, as well as the presence of relatively shallow groundwater associated with the tributary of Swan Creek, consideration may be given to support of headwall footings on deep foundations. Average soil properties associated with the soil layers at the site are summarized in Section 5.2.4 of this report.

Based on the conditions encountered in the borings, the soils anticipated below the existing fill materials, organic-laden soils, and soft cohesive soils are expected to consist of predominantly medium stiff to stiff native cohesive soils, as well as loose native granular soils. These soils are considered generally suitable for support of headwall foundations, albeit not standard headwall foundations (as discussed below).

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 medium stiff to stiff cohesive bearing soils is 1,000 psf, which **does not** meet the minimum design requirement. However, these soils may be sufficiently deep after excavation and replacement of unsuitable soils, such that new backfill placed from the base of the over-excavation up to the original bearing elevation improves foundation support, depending on the final planned depth and size of the headwall foundation. If over-excavation of unsuitable soils extends at least 1 times the headwall spread foundation width below the bearing elevation, the minimum requirements for standard headwalls would be met for bearing materials consisting of new granular engineered fill. After over-excavation of unsuitable soils, a design ϕ value for granular new “embankment” fill may be estimated as 32 degrees, which meets the minimum design requirement for granular bearing soils as discussed below.

The standard concrete headwalls are indicated to be based on design using a minimum internal angle of friction (ϕ) of 28 degrees when the walls are bearing on granular soils. The design ϕ value for the loose granular bearing soils is 28.5 degrees, which meets the minimum design requirement. In any case, excavation into granular soils may further loosen the near-surface materials. As such, we recommend granular bearing soils be re-compacted in-place using a hand-operated plate compactor or backhoe-mounted vibratory compactor (hoe-pac) prior to placement or steel reinforcement and foundation concrete. Care and diligence will be required to lower the groundwater table at least one foot below the bearing elevation to provide compactive effort. Otherwise, over-excavation and replacement with additional granular engineered fill would be required.

5.2 Construction

5.2.1 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.

5.2.2 Site Preparation

Prior to proceeding with construction operations, all structures, pavements, topsoil, root systems, vegetation, and other deleterious non-soil materials should be removed from the proposed construction areas.

5.2.3 Temporary Excavations and Permanent Slopes

The sides of the temporary excavations for headwall 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 should be generally conducive to stable excavation slopes, the anticipated “normal” groundwater level is anticipated to roughly coincide with water levels within the tributary of Swan Creek. As such, seepage may occur in open excavations for headwall installation which could affect the stability of the excavation slopes. Provisions should be made for the headwall installation to proceed as a sloped-bank excavation, or as a steeper trench-type cut with properly designed and installed lateral bracing. Any excavations greater than 20 feet deep should be evaluated by a registered professional engineer.

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 B soils (native cohesive soils with unconfined compressive strengths greater than 1,000 psf but less than 3,000 psf), and
- OSHA Type C soils (cohesive soils with unconfined compressive strengths of 1,000 psf or less, granular soils, and fill materials).

For temporary excavations in Type B and C soils, side slopes must be constructed no steeper than 1H:1V and 1½H:1V, respectively. **At this site, we expect the majority of temporary excavations will require a 1½H:1V slope. 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. It should be noted that ODOT routinely uses 2H:1V slopes for roadway embankments. These steeper slopes could be used, with recognition that the embankment faces are more prone to erosion and sloughing.

5.2.4 Support of Excavations

Where existing structures, underground utilities, and embankments are located within a distance from the excavation equal to approximately twice the depth of the excavation, an adequate system of sheet piling, lateral bracing, trench boxes, or an alternate construction procedure may be required to prevent lateral movements that may cause settlement of these entities. Sheet piling may also be used in combination with laid-back slopes limited to the upper portion of the profile to avoid an excessively large, open excavation. Sheet piling may also be considered for coffer dam purposes, as discussed in the following section.

Design of sheet-pile cutoff walls or H-pile and lagging systems should be the responsibility of the contractor, since their installation and performance is integrally tied to the contractor's means and methods of construction. In any case, applicable OSHA standards must be followed. It is the responsibility of the installation contractor to develop appropriate installation methods and equipment specifications prior to commencement of work, and to obtain the services of a qualified engineer to design or approve sloped or benched excavations and/or lateral bracing systems as required by OSHA criteria. In addition, OSHA requires that excavations with open-cut slopes higher than 20 feet, or braced excavation support systems such as sheetpiling or cofferdams be reviewed and designed by a registered professional engineer.

Retaining structures or walls that are not restrained at the top of the wall, such as temporary sheeting, should be designed for active lateral earth pressure condition. An active earth pressure coefficient (k_a) of 0.36 may be used for design. A passive earth pressure coefficient (k_p) of 2.8 may be utilized for the portion of the wall that is below the excavation bottom. It should be noted that some wall movement or horizontal displacement is typically associated with active and passive earth pressure conditions. In particular, appreciable movements are needed to mobilize the **full** (theoretical) passive pressure of the soil. Specific bracing systems selected by the contractor may have variations of lateral earth pressure (and associated coefficients) that range between the active and passive cases.

In determining lateral earth pressures, a total unit weight of 120 pounds per cubic foot (pcf) may be utilized for the upper-profile cohesive soils and granular soils. Below the groundwater table, effective ("submerged") unit weights should be utilized by reducing the total unit weights by the unit weight of water (62.4 pcf). Additionally, hydrostatic pressures should be considered below the groundwater or streamflow level(s).

It should also be noted that the above earth pressures are based on a level backfill condition behind the retaining wall. In areas where appreciable sloping materials will be present behind the top of the wall, surcharge loading or equivalent higher earth pressure coefficients should be evaluated, based on the sloping material, backfill slope, and proximity to the wall. In general, 50 percent of the vertical surcharge load should be used for lateral loading in the design of the wall.

If required for sheetpiling design (or deep foundation alternative design as discussed in Section 5.1), average soil properties associated with the site soils are summarized in the following table.

Table 5.2.4. Average Soil Properties			
Soil Layer	**Total Unit Weight (pcf)**	**Undrained Shear Strength (Su) or Cohesion (c) (ksf)**	**Internal Angle of Friction (ϕ) (degrees)**
Predominantly Granular Existing Fill Materials	120	-	28.5
Stratum I Organic-Laden Granular Soils	120	-	28.5
Stratum II Predominantly Medium Stiff to Stiff Cohesive Soils	120	1,000	-
Stratum III Predominantly Stiff to Very Stiff Cohesive Soils	122	1,750	-

5.2.5 Construction Dewatering and Groundwater Control

Groundwater conditions encountered during our exploration are summarized in Section 4.3. Based on the soil characteristics and groundwater conditions encountered in the borings, it is our opinion that the “normal” groundwater level will generally be encountered at or slightly above the water level of the tributary, corresponding to a depth of approximately 7 feet or deeper (approximate Elev. 661 or lower) below roadway grades at the time of this investigation.

If construction does not occur during a particularly wet period, adequate control of groundwater seepage into shallow excavations above the groundwater level should be achievable by minor dewatering systems, such as pumping from prepared sumps. Due to the presence of granular soils below the “normal” water level, coffer dams are anticipated to be required for excavations adjacent to the tributary of Swan Creek at this site.

Based on the location of the proposed excavation relative to the tributary, it is likely that the headwall installation 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. For areas that require over-excavation and replacement with new granular engineered fill, the granular fill should be generally suitable as a working platform for preparation of steel reinforcement and placement of concrete as long as diligent dewatering activities are being provided. Installation of well points along with multiple sumps and pumps may be required, even with installation of the coffer dam.

5.2.6 Fill

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

The upper profile on-site soils consist of predominantly granular fill materials. For these soils, for new granular embankment fill, and where existing pavement base materials remain (should the project include excavations extending into the roadway), a vibratory smooth-drum roller would be required to provide effective compaction.

6.0 QUALIFICATION OF RECOMMENDATIONS

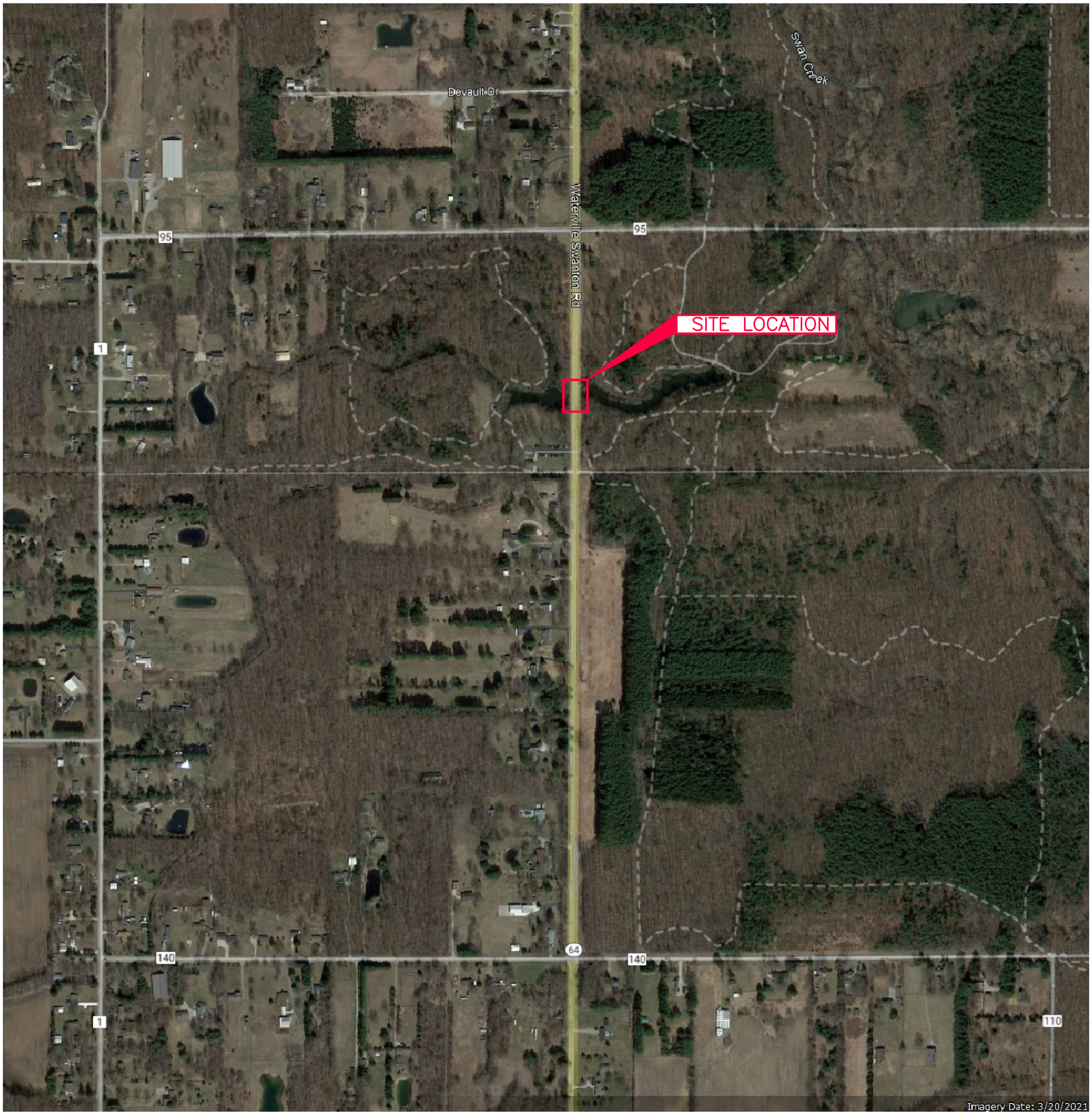
Our evaluation of design and construction conditions for the proposed headwalls replacement has been based on our understanding of the site and project information and the data obtained during our field exploration. The general subsurface conditions were based on interpretation of the data obtained at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

0 860 1,720

PLATE 1.0
SITE LOCATION MAP
LUC–64–8.49, PID 96000
SR 64, SOUTH OF MONCLOVA ROAD (TR 95)
SWANTON TOWNSHIP, LUCAS COUNTY, OHIO

PREPARED FOR
ODOT DISTRICT 2
BOWLING GREEN, OHIO

DRAWN	TRR/1–23–24	CHECKED	KCH/1–25–24
REVISED		APPROVED	
JOB NO.	232085		
DRAWING NUMBER			
B_232085-01-PLATE 1 SM			

TTL associates inc
Environmental, Geotechnical Engineering & Testing

LEGEND

B-001-0-23 APPROXIMATE TEST BORING LOCATION

APPROXIMATE SCALE – FEET

0 30 60

PLATE 2.0

TEST BORING LOCATION PLAN

LUC-64-8.49, PID 96000
SR 64, SOUTH OF MONCLOVA ROAD (TR 95)
SWANTON TOWNSHIP, LUCAS COUNTY, OHIO

PREPARED FOR

ODOT DISTRICT 2
BOWLING GREEN, OHIO

DRAWN	TRR/1-23-24	CHECKED	KCH/1-25-24
REVISED		APPROVED	
JOB NO.	232085		
DRAWING NUMBER	**B_232085-01-PLATE 2 BLP**		

TTL associates Inc
Environmental, Geotechnical Engineering & Testing

FIGURES

PROJECT: LUC-64-08.49	DRILLING FIRM / OPERATOR: CT / CW	DRILL RIG: CME 75 TRUCK 844	STATION / OFFSET: 567+68, 8' LT.	EXPLORATION ID B-001-0-23
TYPE: CULVERT	SAMPLING FIRM / LOGGER: CT / KKC	HAMMER: CME AUTOMATIC	ALIGNMENT: SR 64	
PID: 96000 SFN: N/A	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 2/20/23	ELEVATION: 668.5 (NAVD88) EOB: 45.0 ft.	PAGE 1 OF 2
START: 12/27/23 END: 12/27/23	SAMPLING METHOD: SPT / ST	ENERGY RATIO (%): 72.9	LAT / LONG: 41.557472, -83.873257	

MATERIAL DESCRIPTION AND NOTES	ELEV. 668.5
ASPHALT - 9.5 INCHES	667.7
AGGREGATE BASE - 12.5 INCHES	666.7
LOOSE, BROWN, **FINE SAND**, TRACE SILT, MOIST FILL @7': VERY LOOSE, GRAY/BROWN, WET (FREE WATER NOTED) @8.5': BLACK/BROWN, TRACE GRAVEL, TRACE COAL FRAGMENTS, TRACE WOOD, (FREE WATER NOTED)	657.5
LOOSE, GRAY, **COARSE AND FINE SAND**, LITTLE SILT, TRACE CLAY, TRACE GRAVEL, WET @13.5': GRAY/BLACK, LITTLE PEAT, HIGHLY ORGANIC @13.5' TO 15.0': ORGANIC CONTENT = 11.8%	652.5
LOOSE, GRAY/BLACK, **FINE SAND**, SOME PEAT, TRACE SILT, TRACE CLAY, HIGHLY ORGANIC, WET @16.0 TO 17.5': ORGANIC CONTENT = 20.7%	650.0
SOFT TO MEDIUM STIFF, GRAY, **SANDY SILT**, "AND" CLAY, WET	648.5
STIFF, GRAY, **SILTY CLAY**, LITTLE SAND, WET @23.5': TRACE SAND	642.5

DEPTHS: W 661.5 (at 7 ft)

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
7 16 16	39	100	SS-1A	-	-	-	-	-	-	-	-	-	3	A-1-b (V)	
			SS-1B	-	-	-	-	-	-	-	-	-	-	A-3 (V)	
2 2 2	5	100	SS-2	-	-	-	-	-	-	-	-	-	8	A-3 (V)	
2 2 1	4	100	SS-3	-	-	-	-	-	-	-	-	-	23	A-3 (V)	
2 2 1	4	100	SS-4	-	-	-	-	-	-	-	-	-	21	A-3 (V)	
0 2 2	5	100	SS-5	-	1	5	78	14	2	NP	NP	NP	22	A-3a (0)	
1 1 3	5	100	SS-6	-	-	-	-	-	-	-	-	-	68	A-3a (V)	
3 4 4	10	100	SS-7	-	-	-	-	-	-	-	-	-	83	A-3a (V)	
1 1 3	5	100	SS-8	0.25	0	3	16	43	38	26	19	7	30	A-4a (8)	
6 6 5	13	100	SS-9	1.50	0	3	11	25	61	25	9	16	28	A-6b (10)	
4 5 5	12	100	SS-10	1.25	-	-	-	-	-	-	-	-	24	A-6b (V)	

PID: 96000	SFN: N/A	PROJECT: LUC-64-08.49	STATION / OFFSET: 567+68, 8' LT.	START: 12/27/23	END: 12/27/23	PG 2 OF 2	B-001-0-23

MATERIAL DESCRIPTION AND NOTES	ELEV. 642.5	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
STIFF TO VERY STIFF, GRAY, **SILT AND CLAY**, TRACE SAND, MOIST	640.5	27	6 7 7	17	100	SS-11	1.00	0	2	5	25	68	29	15	14	29	A-6a (10)	
STIFF, GRAY, **SANDY SILT**, "AND" CLAY, WET		28 – 29	3 3 4	9	100	SS-12	1.50	-	-	-	-	-	-	-	-	27	A-4a (V)	
@31': DAMP @31.0' TO 33.0': Su = 2,280 PSF (STIFF)	635.0	30 – 33			100	ST-13	2.75	0	1	6	43	50	27	22	5	22	A-4a (8)	
STIFF, GRAY, **SILTY CLAY**, LITTLE SAND, TRACE GRAVEL, DAMP		34	4 5 7	15	100	SS-14	1.50	-	-	-	-	-	-	-	-	18	A-6b (V)	
@38.5' TO 40.0': Qu = 2,040 PSF		35 – 39	3 5 7	15	100	SS-15	1.50	-	-	-	-	-	-	-	-	17	A-6b (V)	
@43.5': VERY STIFF, SOME SAND	623.5	40 – 44	4 6 10	19	100	SS-16	2.25	-	-	-	-	-	-	-	-	16	A-6b (V)	
		EOB 45																

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.25 BAG ASPHALT PATCH; PUMPED 13 CF CEMENT-BENTONITE GROUT

PROJECT: LUC-64-08.49	DRILLING FIRM / OPERATOR: CT / CW	DRILL RIG: CME 75 TRUCK 844	STATION / OFFSET: 567+67, 10' RT.
TYPE: CULVERT	SAMPLING FIRM / LOGGER: CT / KKC	HAMMER: CME AUTOMATIC	ALIGNMENT: SR 64
PID: 96000 SFN: N/A	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 2/20/23	ELEVATION: 668.4 (NAVD88) EOB: 45.0 ft.
START: 12/27/23 END: 12/27/23	SAMPLING METHOD: SPT / ST	ENERGY RATIO (%): 72.9	LAT / LONG: 41.557470, -83.873192

EXPLORATION ID B-002-0-23

MATERIAL DESCRIPTION AND NOTES	ELEV. 668.4	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
ASPHALT - 9.5 INCHES	667.6																	
AGGREGATE BASE - 12.5 INCHES	666.6	1	13 13 10	28	100	SS-1A	-	-	-	-	-	-	-	-	-	4	A-1-b (V)	
MEDIUM DENSE, BROWN, **COARSE AND FINE SAND**, LITTLE SILT, TRACE CLAY, TRACE GRAVEL, WET FILL		2				SS-1B	-	-	-	-	-	-	-	-	-	-	A-3a (V)	
		3																
		4	6 5 5	12	100	SS-2	-	1	3	78	16	2	NP	NP	NP	15	A-3a (0)	
		5																
		6																
@7': LOOSE		W 661.4 7	3 3 1	5	100	SS-3	-	-	-	-	-	-	-	-	-	21	A-3a (V)	
	659.9	8																
SOFT, GRAY, **SILT AND CLAY**, "AND" SAND, TRACE GRAVEL, TRACE ORGANICS, MOIST FILL		9	0 1 1	2	100	SS-4	-	6	5	44	23	22	34	21	13	31	A-6a (3)	
	657.4	10																
LOOSE, GRAY, **FINE SAND**, TRACE SILT, TRACE CLAY, MOIST		11 12	0 1 3	5	100	SS-5	-	-	-	-	-	-	-	-	-	23	A-3 (V)	
		13																
@13.5': DRILLERS NOTED TREE ROOT	653.4	14	4 3 2	6	0	SS-6	-	-	-	-	-	-	-	-	-	-	A-3 (V)	
STIFF, GRAY, **SANDY SILT**, TRACE CLAY, TRACE GRAVEL, WET		15																
@16': (FREE WATER NOTED)		16 17	3 3 5	10	100	SS-7	-	1	7	43	45	4	21	16	5	34	A-4a (3)	
	649.9	18																
SOFT TO MEDIUM STIFF, GRAY, **SILTY CLAY**, SOME SAND, WET		19	3 2 2	5	100	SS-8	0.25	-	-	-	-	-	-	-	-	33	A-6b (V)	
	647.4	20																
STIFF, GRAY, **SILT AND CLAY**, TRACE SAND, TRACE GRAVEL, WET TO MOIST		21 22	5 5 6	13	100	SS-9	0.75	-	-	-	-	-	-	-	-	27	A-6a (V)	
		23																
@23.5': MOIST		24	3 4 6	12	100	SS-10	1.50	-	-	-	-	-	-	-	-	22	A-6a (V)	
		25																

PID: 96000	SFN: N/A	PROJECT: LUC-64-08.49	STATION / OFFSET: 567+67, 10' RT.	START: 12/27/23	END: 12/27/23	PG 2 OF 2	B-002-0-23

MATERIAL DESCRIPTION AND NOTES	ELEV. 642.4	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
STIFF, GRAY, **SILT AND CLAY**, TRACE SAND, TRACE GRAVEL, WET TO MOIST *(continued)* @26': VERY STIFF		27	7 6 7	16	100	SS-11	-	-	-	-	-	-	-	-	-	28	A-6a (V)	
@28.0' TO 30.0': Qu = 2,036 PSF		28 – 30			88	ST-12	2.00	1	2	7	23	67	32	20	12	24	A-6a (9)	
	637.4	31																
MEDIUM STIFF TO STIFF, GRAY, **SILT AND CLAY**, TRACE SAND, MOIST @31.0' TO 32.5': Qu = 1,595 PSF		32	2 3 3	7	100	SS-13	1.25	-	-	-	-	-	-	-	-	21	A-6a (V)	
@33.5': TRACE GRAVEL @33.5' TO 35.0': Qu = 1,720 PSF		34	2 3 5	10	100	SS-14	1.00	1	2	5	25	67	32	20	12	26	A-6a (9)	
	629.9	38																
STIFF TO VERY STIFF, GRAY, **SILTY CLAY**, SOME SAND, TRACE GRAVEL, DAMP		39	5 6 7	16	100	SS-15	1.50	-	-	-	-	-	-	-	-	15	A-6b (V)	
		44	5 7 9	19	100	SS-16	1.75	-	-	-	-	-	-	-	-	15	A-6b (V)	
	623.4	EOB 45																

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.25 BAG ASPHALT PATCH; PUMPED 13 CF CEMENT-BENTONITE GROUT

LEGEND KEY

LITHOLOGIC SYMBOLS
(Unified Soil Classification System)

- A-3: Ohio DOT: A-3, fine sand
- A-3A: Ohio DOT: A-3a, coarse and fine sand
- A-4A: Ohio DOT: A-4a, sandy silt
- A-6A: Ohio DOT: A-6a, silt and clay
- A-6B: Ohio DOT: A-6b, silty clay
- 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 December 27, 2023, utilizing 3¼-inch inside diameter hollow-stem augers.

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

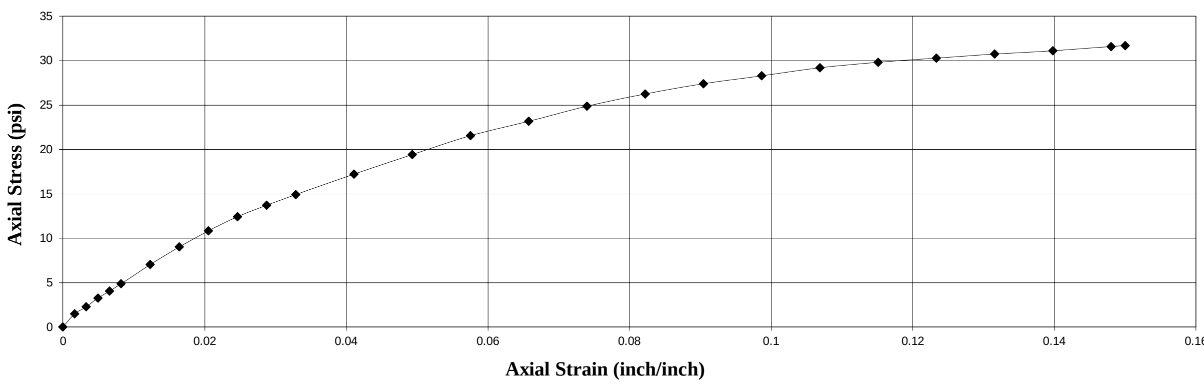
3. Latitude, Longitude, and ground surface elevations at the as-drilled boring locations were surveyed by TTL via a hand-held GPS. Stationing and offsets were estimated from the provided plan based on surrounding site features.

Unconsolidated - Undrained Triaxial Shear Strength Test

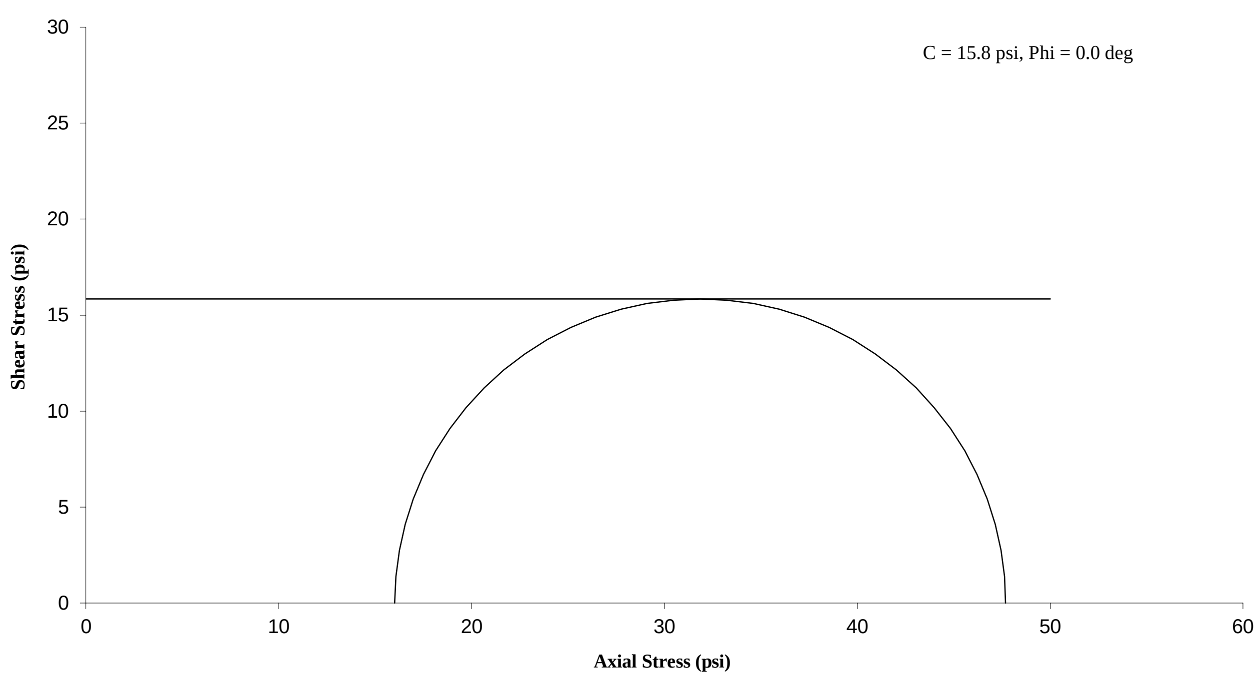
ASTM D 2850

General Sample Data		Triaxial Specimen Data			
TTL Project No.:	232085	Symbol	◆	■	●
Project:	LUC-64-8.49, PID 96000	Init. Specimen Height (in.)	6.08	-	-
Sample ID:	B-001-0-23 ST-13	Init. Specimen Diameter (in.)	2.88	-	-
Sample Interval:	31.0 - 33.0'	Init. Moisture Content* (%)	21.5	-	-
Soil Description:	Gray SANDY SILT, "And" Clay, A-4a (8)	Init. Dry Unit Weight (pcf)	106.2	-	-
Liquid Limit:	27	Init. Void Ratio	0.62	-	-
Plastic Limit:	22	Init. Degree of Saturation (%)	96	-	-
Plasticity Index:	5	Minor Principal Stress (psi)	16.0	-	-
Specific Gravity:	2.75 (Assumed)	Deviator Stress at Failure (psi)	31.7	-	-
Rate of Strain:	0.03 Inches per Minute	Major Principal Stress (psi)	47.7	-	-
Failure Criteria:	Peak Deviator Stress or Deviator Stress at 15% Axial Strain	Axial Strain at Failure (%)	15.0	-	-

Stress/Strain



Mohr Circle Plot



UNCONSOLIDATED, UNDRAINED COMPRESSIVE STRENGTH
OF COHESIVE SOILS IN TRIAXIAL COMPRESSION (ASTM D 2850)

Project: LUC-64-8.49, PID 96000
Client: ODOT District 2
Sample ID: B-001-0-23 ST-13
TTL Project No.: 232085

Date: 1/10/2024
File: 232085B-001-0-23ST-13
Depth: 31.0 - 33.0'
Specimen ID: "B" (31.5 - 32.0 Feet)

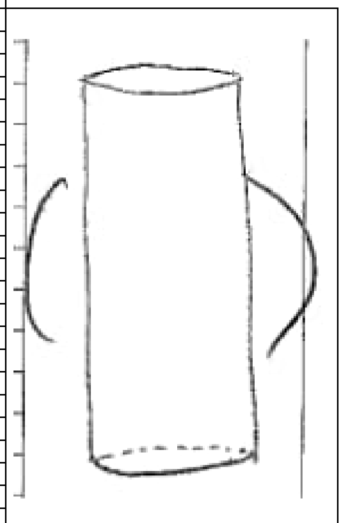
SAMPLE PROPERTIES

Visual Description: Gray SANDY SILT, "And" Clay, A-4a (8)
Diameter: 2.88 in.
Area: 6.514 in^2
Length: 6.08 in.
Initial Void Ratio: 0.62
Chamber Pressure: 16 psi

Initial Dry Unit Weight of Sample: 106.2 pcf
Initial Moisture Content: 21.5 %
Specific Gravity (assumed): 2.75
Initial Degree of Saturation: 96 %
Proving Ring Number: 1155-12-13322

STRESS-STRAIN DATA

Speciman Deformation (in)	Vertical Strain	Proving Ring Reading	Piston Load (lbs)	Corrected Area (in^2)	Deviator Stress (psi)
0.000	0.000	0.0	0.0	6.514	0.0
0.010	0.002	14.0	9.6	6.525	1.5
0.020	0.003	21.5	14.7	6.536	2.3
0.030	0.005	31.0	21.3	6.547	3.2
0.040	0.007	38.5	26.4	6.558	4.0
0.050	0.008	46.5	31.9	6.568	4.9
0.075	0.012	67.5	46.3	6.596	7.0
0.100	0.016	87.0	59.7	6.623	9.0
0.125	0.021	105.0	72.0	6.651	10.8
0.150	0.025	121.0	83.0	6.679	12.4
0.175	0.029	134.0	91.9	6.707	13.7
0.200	0.033	146.5	100.5	6.736	14.9
0.250	0.041	170.5	117.0	6.794	17.2
0.300	0.049	194.0	133.1	6.853	19.4
0.350	0.058	217.0	148.9	6.912	21.5
0.400	0.066	235.5	161.6	6.973	23.2
0.450	0.074	255.0	174.9	7.035	24.9
0.500	0.082	271.5	186.2	7.098	26.2
0.550	0.090	286.0	196.2	7.162	27.4
0.600	0.099	298.0	204.4	7.228	28.3
0.650	0.107	310.5	213.0	7.294	29.2
0.700	0.115	320.0	219.5	7.362	29.8
0.750	0.123	328.0	225.0	7.431	30.3
0.800	0.132	336.0	230.5	7.501	30.7
0.850	0.140	343.5	235.6	7.573	31.1
0.900	0.148	352.0	241.5	7.646	31.6
0.912	0.150	354.0	242.8	7.664	31.7



Sketch of Tested Specimen

RESULTS

Maximum Deviator Stress 31.7 psi

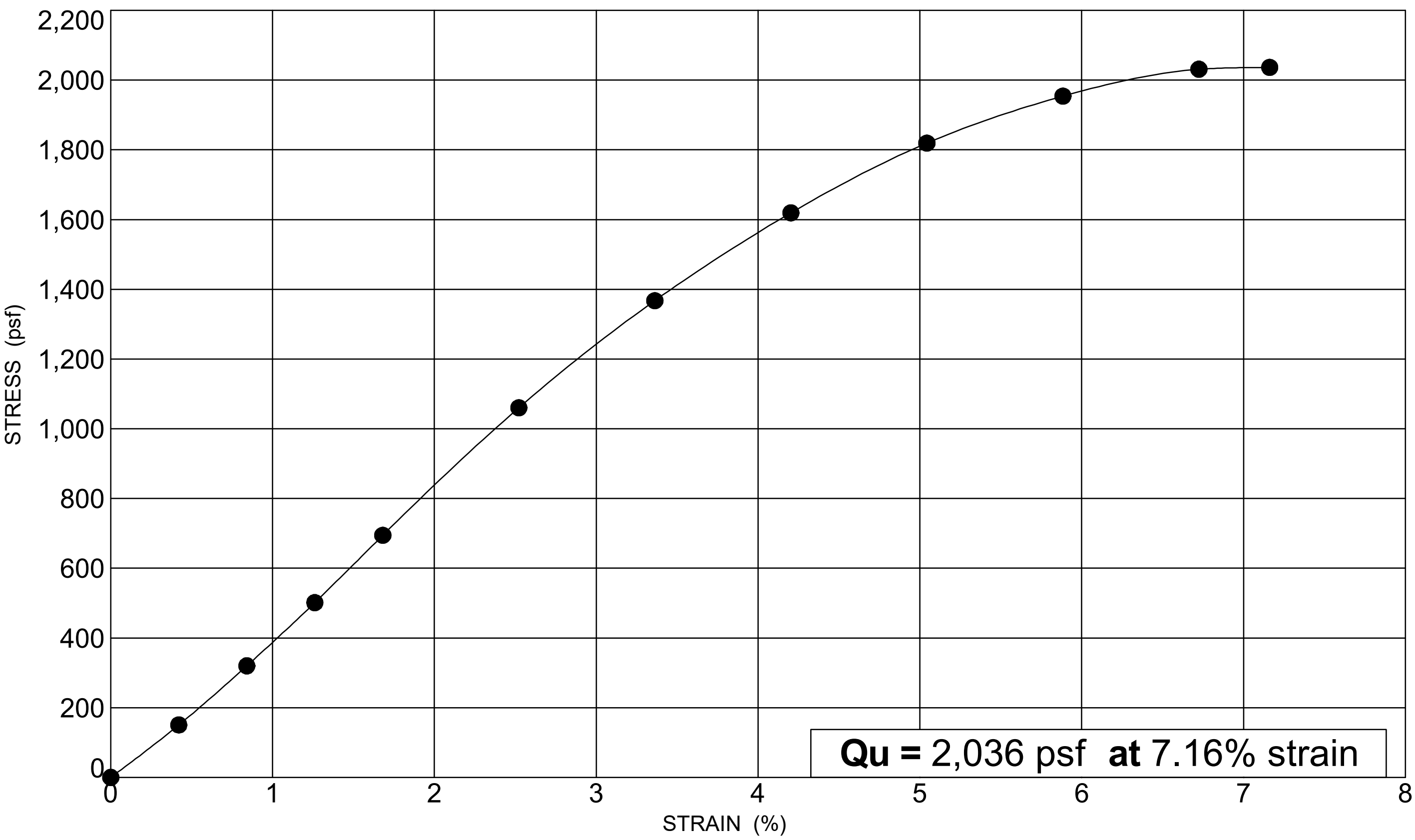
UNCONFINED COMPRESSION TEST
AASHTO T - 208

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT LUC-64-08.49 **PID** 96000
OGE NUMBER N/A **PROJECT TYPE** STRUCTURE FOUNDATION

SAMPLE IDENTIFICATION

BORING ID: B-002-0-23 SAMPLE ID: ST-12
STATION: 567+67, 14' RT. DEPTH: 28.0 - 30.0 feet



SPECIMEN FAILURE SKETCHES OR PHOTOGRAPHS

FRONT VIEW SIDE VIEW

SPECIMEN DETAILS

HEIGHT: 151.100 mm
DIAMETER: 72.400 mm
WET UNIT WT: 128.57 pcf
DRY UNIT WT: 103.43 pcf

TESTED BY: RS 1/9/2024

CLASSIFICATION RESULTS

GRADATION (%)

GR	CS	FS	SI	CL
1	2	7	23	67

ATTERBERG LIMITS / MOISTURE

LL	PL	PI	WC
32	20	12	24

ODOT CLASS: A-6a HP (tsf): 2.0
DESCRIPTION:

OHDOT UNCONFINED COMPRESSION - OH DOT.GDT - 1/25/24 12:25 - S:\PROJECTS\232085.GPJ

Appendix A:
Engineering Calculations

TTL Project No. 232085					
LUC-64-8.49, PID 96000					
Calculation By: KCH 1/26/2024					
Cohesive Soil Strength Evaluations					
Predominantly Medium Stiff to Stiff Cohesive Bearing Soils					
Upper 10 ft of cohesive bearing materials, from 10 to 25 feet below existing grades.	**N60**	**HP (tsf)**			
	5	0.25			
	13	1.5			
	12	1.25			
	10	-			
	5	0.25			
	13	0.75			
	12	1.5			
Minimum:	**5**	**0.25**			
c (psf): N60x250/2=	625	psf			
c (psf)=		250	psf		
Average:	**10.0**	**0.9**			
c (psf): N60x250/2=	1,250	psf			
c (psf)=		917	psf		
Average of Min., c =	438	psf			
Average of Avg., c =	1,083	psf			
c from UU =	2,280	psf	(at depth likely >2B)		
Conservatively, say su = c =		**1,000**	**psf (native soils)**		
For over-excavate and replace embankment fill, say su = 2,500 psf per GDM Table 500-2.					
Based on average N60 of 10 bpf, dry density = 120 pcf per GDM Table 400-4.					

TTL Project No. 232085					
LUC-64-8.49, PID 96000					
Calculation By: KCH 1/29/2024					
Cohesive Soil Strength Evaluations					
Consider Stratum III Cohesive Soils if deeper soils needed for support					
Upper 10 ft of Stratum III, from 33.5 to 45 feet below existing grades in Boring B-001, and from 38.5 to 45 feet in Boring B-002.	**N60**	**HP (tsf)**	**UCS (tsf)**		
	15	1.5			
	15	1.5	1.02		
	19	2.25			
	16	1.5			
	19	1.75			
Minimum:	**15**	**1.5**	**1.02**		
c (psf): N60x250/2=	1,875	psf			
c (psf)=		1,500	1,020	psf	
Average:	**16.8**	**1.7**			
c (psf): N60x250/2=	2,100	psf			
c (psf)=		1,700	1,020	psf	
Average of Min., c =	1,688	psf			
Average of Avg., c =	1,900	psf			
Conservatively, say su = c =		**1,750**	**psf (native soils)**		
Based on average N60 of 17 bpf, dry density = 122 pcf per GDM Table 400-4.					

TTL Project No. 232085
LUC-64-8.49, PID 96000
Calculation By: KCH 1/26/2024

Granular Phi Angle Evaluations - Foundation Soils

Over-excavate & replace organic-laden soils in Boring B-001; consider loose granular soils in Boring B-002 for bearing approximately 10 to 15 ft below existing grades.
Granular Layer consited of ODOT A-3 soils in B-002
Ground water generally encountered at 7'
Granular bearing soils, N60 = 5 to 6 bpf. ODOT GDM Table 400-4, total unit weight = 118 to 120 pcf. Say 120 pcf.

Geotechnical Design Manual

404.2 Granular Soils
For granular soils, use SPT N1₆₀ and the relative percentage of fine or coarse materials in the soil/soil classification, in accordance with the AASHTO LRFD Article 10.4.6.2.4 to estimate the drained friction angle of the soil. Use the middle of the range of friction angles presented on each line in AASHTO LRFD Table 10.4.6.2.4-1, and apply the adjustment according to Table 400-3:

Table 400-3: ϕ' Adjustment

Soil Class	Adjustment
A-1-a	+2.5°
A-1-b	+1.5°
A-2-4	+0.5°
A-2-5	-0.5°
A-2-6	-0.5°
A-2-7	-0.5°
A-3	-1.5°
A-3a	-0.5°
A-4a	-2.5°
A-4b	-2.5°

The values of SPT $N1_{60}$ on each line of AASHTO LRFD Table 10.4.6.2.4-1 may be linearly interpolated for intermediate values. For example, SPT $N1_{60}$ = 4 corresponds to a middle-range value of 29.5°, and SPT $N1_{60}$ = 10 corresponds to a middle-range value of 32.5°; therefore, SPT $N1_{60}$ = 8 corresponds to a middle-range value of approximately 31.5° by linear interpolation. If the soil was a Sandy Silt (A-4a) Soil class, the drained friction angle can be approximated as 31.5° - 2.5° = 29°.

For very dense granular soils, do not use a drained friction angle of $\phi' > 45°$. Considering the correlations published by Meyerhof (1956) and Bowles (1977), and the limits of the tabulated data in AASHTO LRFD Table 10.4.6.2.4-1 and publication FHWA-NHI-16-072 (GEC 5), we consider this to be a reasonably conservative limit for very dense granular soil.

AASHTO LRFD

$C_N = [0.77 \log_{10}(40/\sigma'_v)]$, and $C_N < 2.0$

σ'_v = vertical effective stress (ksf)

$$N1_{60} = C_N N_{60} \quad (10.4.6.2.4\text{-}3)$$

The drained friction angle of granular deposits should be determined based on the following correlation.

Table 10.4.6.2.4-1—Correlation of *SPT* $N1_{60}$ Values to Drained Friction Angle of Granular Soils (modified after Bowles, 1977)

$N1_{60}$	ϕ_f
<4	25–30
4	27–32
10	30–35
30	35–40
50	38–43

Effective Overburden Pressure - Soil Boring Location for $N1_{60}$ Adjustment

Layer	Depth (ft)	Thichness	γ_{TOTAL} (pcf)	Pressure* (psf)
1	7	7	120	840
2	15	8	57.6	230.4
3				
4				
5				
Leave Blank				
*Last layer pressure contribution taken in middle of layer			Sum	1.07 ksf

C_N = 0.77*LOG10(40/ksf) =	1.21
N_{60} =	5
$N1_{60} = C_N * N_{60}$ =	6

from Table 10.4.6.2.4-1	$N1_{60}$	Average Phi	Interpolate (linear)	
	4	29.5		
	10	32.5	Average	30
Find	6			
	Phi ajustment	Ajusted average Phi		
A-3	-1.5	28.5		

use Phi = 28.5 degrees

For over-excavate and replace, new embankment fill, phi = 32 degrees per GDM Table 500-2.

TTL Project No. 232085
LUC-64-8.49, PID 96000
Calculation By: KCH 1/26/2024

Granular Phi Angle Evaluations - Temporary Sheeting

Consider loose granular existing fill materials, which are predominant in upper 10 to 15 feet below existing grade, which will be retained soils.

Ground water generally encountered at 7'

Geotechnical Design Manual

404.2 Granular Soils

For granular soils, use SPT $N1_{60}$ and the relative percentage of fine or coarse materials in the soil/soil classification, in accordance with the AASHTO LRFD Article 10.4.6.2.4 to estimate the drained friction angle of the soil. Use the middle of the range of friction angles presented on each line in AASHTO LRFD Table 10.4.6.2.4-1, and apply the adjustment according to Table 400-3:

Table 400-3: ϕ' Adjustment

Soil Class	Adjustment
A-1-a	+2.5°
A-1-b	+1.5°
A-2-4	+0.5°
A-2-5	-0.5°
A-2-6	-0.5°
A-2-7	-0.5°
A-3	-1.5°
A-3a	-0.5°
A-4a	-2.5°
A-4b	-2.5°

The values of SPT $N1_{60}$ on each line of AASHTO LRFD Table 10.4.6.2.4-1 may be linearly interpolated for intermediate values. For example, SPT $N1_{60}$ = 4 corresponds to a middle-range value of 29.5°, and SPT $N1_{60}$ = 10 corresponds to a middle-range value of 32.5°; therefore, SPT $N1_{60}$ = 8 corresponds to a middle-range value of approximately 31.5° by linear interpolation. If the soil was a Sandy Silt (A-4a) Soil class, the drained friction angle can be approximated as 31.5° - 2.5° = 29°.

For very dense granular soils, do not use a drained friction angle of $\phi' > 45°$. Considering the correlations published by Meyerhof (1956) and Bowles (1977), and the limits of the tabulated data in AASHTO LRFD Table 10.4.6.2.4-1 and publication FHWA-NHI-16-072 (GEC 5), we consider this to be a reasonably conservative limit for very dense granular soil.

AASHTO LRFD

C_N = $[0.77 \log_{10}(40/\sigma'_v)]$, and $C_N < 2.0$

σ'_v = vertical effective stress (ksf)

$$N1_{60} = C_N N_{60} \qquad (10.4.6.2.4\text{-}3)$$

The drained friction angle of granular deposits should be determined based on the following correlation.

Table 10.4.6.2.4-1—Correlation of SPT $N1_{60}$ Values to Drained Friction Angle of Granular Soils (modified after Bowles, 1977)

$N1_{60}$	ϕ_f
<4	25–30
4	27–32
10	30–35
30	35–40
50	38–43

Effective Overburden Pressure - Soil Boring Location for $N1_{60}$ Adjustment

Layer	Depth (ft)	Thichness	γ_{TOTAL} (pcf)	Pressure* (psf)
1	7	7	120	840
2	10	3	57.6	86.4
3				
4				
5				
Leave Blank				

Sum 0.93 ksf

approx. midpoint = 0.465 ksf

C_N = 0.77*LOG10(40/ksf) =	1.49
N_{60} =	4
$N1_{60} = C_N * N_{60}$ =	6

from Table 10.4.6.2.4-1	$N1_{60}$	Average Phi	Interpolate (linear)	
	4	29.5		
	10	32.5	Average	30
Find	6			
	Phi ajustment	Ajusted average Phi		
A-3	-1.5	28.5		

use Phi = 28.5 degrees

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
[illegible]	[illegible]	1.428	0.700	2.040	0.490
[illegible]	[illegible]	1.455	0.687	2.117	0.472
[illegible]	[illegible]	1.483	0.675	2.198	0.455
[illegible]	[illegible]	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	0.966	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	0.394	6.445	0.155
48	1.111	2.605	0.384	6.786	0.147

Granular Existing Fill and Native Granular Phi = 28.5 deg

Predom granular behind the sheeting, some granular below mudline

Assume Clay

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

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

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

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

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

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 advantageo… 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 … Fig. 5… lines … curves … into st… straigh… cubic p…

Select Granular

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

331912 African Safari

Rpts /Retwall

5.3 ST… IN…

…e material has no … …essed by $\tau_s = \sigma \tan$ … …l be developed by … … the mass. The st… … which the weight … applied to cases in which these exte… negligible in comparison with these exte… determinat…

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 $p_0 = 0$; t… 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_0 … 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: LUC-64-8.49	PDP Path:	
PID: 96000	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: LUC-64-8.49	PID: 96000	Reviewer: KCH	Date: 1/26/2024

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

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

II. Reconnaissance and Planning Checklist

Planning - General		(Y/N/X)	Notes:
10	Have the scaled boring plans, showing all project and historic borings, and a schedule of borings in tabular format, been submitted to the District Geotechnical Engineer?	Y	Included with proposal.
	The schedule of borings should present the following information for each boring:		
a.	exploration identification number	Y	
b.	location by station and offset	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: LUC-64-8.49	PID: 96000	Reviewer: KCH	Date: 1/26/2024

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?	N	
6	Were representative sections analyzed for the entire length of the structure for the following:	Y	
a.	factored bearing resistance?	Y	Recommended soil parameters provided
b.	factored sliding resistance?	Y	Recommended soil parameters provided
c.	eccentric load limitations (overturning)?	N	
d.	predicted settlement?	N	
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?	X	Recommended soil parameters 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?	X	Plans to be prepared by others.
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?	N	
a.	If needed, have details and plan notes for load testing been included in the plans?		

VI.B. Geotechnical Reports

C-R-S:	LUC-64-8.49	PID: 96000	Reviewer: CPI	Date: 7/31/2024

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 report
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 being provided with this submittal
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	