TRC
Columbus, Ohio

Final Report
Structure Foundation Exploration
Proposed Bridge Replacement
LUC-120-11.32, PID 102940
Central Avenue over Ottawa River
Toledo, Lucas County, Ohio

June 2020

TTL associates inc
Environmental, Geotechnical
Engineering & Testing

TTL associates inc
Environmental, Geotechnical Engineering & Testing

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

June 24, 2020 **TTL Project No. 1771201**

Mr. Christopher M. Hay, P.E.
TRC
781 Science Boulevard, Suite 200
Columbus, Ohio 43230

Final Report
Structure Foundation Exploration
Proposed Bridge Replacement
LUC-120-11.32, PID 102940
Central Avenue over Ottawa River
Toledo, Lucas County, Ohio

Dear Mr. Hay:

Following is the report of the structure foundation exploration performed by TTL Associates, Inc. (TTL) at the site of the referenced project. This exploration was performed in general accordance with TTL Proposal No. 1771201, dated November 26, 2018, and was initially authorized by Mr. Doug Miller of TRC via email on January 17, 2019. The exploration was formally authorized with a Subconsultant Service Agreement dated February 19, 2019, along with Purchase Order 134035, dated February 25, 2019.

A “draft” version of the report, dated August 23, 2019, was previously provided. This final report contains the results of our study, our engineering interpretation of the results with respect to the project characteristics, and our recommendations for design and construction of foundations and pavements. This report also incorporates responses to comments provided by ODOT regarding our preliminary memoranda submittals and final design loads provided by TRC.

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.
Geotechnical Engineer

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

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

T:\Geotech\Projects 2018\1771201…\Report\1771201 FINAL rpt LUC-120-11.32 Bridge Replacement Toledo OH.doc

FINAL REPORT
STRUCTURE FOUNDATION EXPLORATION
PROPOSED BRIDGE REPLACEMENT
LUC-120-11.32, PID 102940
CENTRAL AVENUE OVER OTTAWA RIVER
TOLEDO, LUCAS COUNTY, OHIO

FOR

TRC
781 SCIENCE BOULEVARD, SUITE 200
COLUMBUS, OHIO 43230

SUBMITTED

JUNE 24, 2020

TTL PROJECT NO. 1771201

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 replacement of the existing Central Avenue Bridge over Ottawa River in Toledo, Lucas County, Ohio. This exploration included two structure borings. A summary of the conclusions and recommendations of this study are as follows:

1. The borings were performed within the roadway, and the encountered surface materials consisted of a composite section of asphalt underlain by concrete, which was underlain by crushed stone.

2. Cohesive **fill** materials were encountered underlying the surface materials to depths of 4 feet below existing grade (Elev. 607±) in Boring B-001 and approximately 3½ feet (Elev. 607±) in Boring B-002. Granular **fill** materials were encountered underlying the cohesive fill materials to a depth of 8 feet (Elev. 603±) in Boring B-002. The granular fill materials consisted of predominantly crushed stone.

3. The subsoils encountered underlying the surface and fill materials can be generally described as predominantly cohesive soils exhibiting varying strength and moisture characteristics, with zones of granular soils, overlying bedrock. **Stratum I** consisted of predominantly medium stiff to stiff cohesive soils encountered underlying the surface and fill materials in Borings B-001 and B-002 to depths of 11 feet below existing grade (Elev. 600±) and 23 feet (Elev. 588±). In Boring B-002, zones of predominantly **loose** to medium dense granular soils were encountered within Stratum I at depths of 11 to 14 feet (Elevs. 600± to 597±) and 18 to 20 feet (Elevs. 593± to 591±). **Stratum II** consisted of predominantly very stiff to hard cohesive soils encountered underlying Stratum I in Boring B-001 to a depth of approximately 27 feet (Elev. 584±). In Boring B-002, a zone of predominantly medium dense granular soils was encountered underlying Stratum II to a depth of 26½ feet (Elev. 584±).

4. Augerable weathered dolomite was encountered underlying the subsoils in Borings B-001 and B-002 at depths of approximately 27 feet (Elev. 584±) and 26½ feet (Elev. 584±), respectively, extending to depths of approximately 27½ feet (Elev. 583½) and 29½ feet (Elev. 581±), respectively.

5. Underlying the weathered dolomite, auger refusal on dolomite bedrock was encountered. The rock was cored in each of the borings for a total length of approximately 10 feet. The cored bedrock consisted of slightly weathered to moderately weathered dolomite. **The driller noted tool drop from approximately 32 to 33 feet in Boring B-002, possibly indicating a soil-filled zone or a void.**

6. Provided drawings for the existing structure, dated 1968, indicate normal water level at Elev. 591.8± and high water level at Elev. 600.5±. Based on the soil characteristics and moisture conditions encountered in the borings, it is our opinion that "normal" groundwater levels at this structure location will generally occur at Elevs. 600± to 595±, corresponding to depths at or slightly above the streamflow levels in Ottawa River.

7. It should be noted that ODOT design methods recommend that the contribution of skin friction be neglected in the upper 2 feet of the rock socket. However, with the exception of a half-foot zone, the upper portion of the bedrock extending approximately 4½ feet below top of rock in Boring B-001 exhibited an RQD of 0 percent, prior to encountering more competent bedrock. In Boring B-002, the upper approximately 3½ feet of rock was penetrable with augers. Additionally, in Boring B-002, a potential void or zone of soil-filled joint(s) was encountered from 5½ to 6½ feet below top of bedrock, with the underlying approximately 1 foot exhibiting an RQD of only 15 percent. Therefore, competent rock was not encountered until a depth of approximately 7½ feet below top of bedrock.

8. We recommend that the contribution of resistance be modeled starting below the 4½ feet zone (B-001) and the 7½ feet zone (B-002) of particularly weathered/fractured (and possibly void containing in the case of Boring B-002) rock. Recommendations for bridge foundations are provided in Section 5.1.

9. Using GB-1 criteria based on the encountered conditions, no planned subgrade modification was indicated. If planned subgrade modification was indicated, GB-1 indicated an option for global chemical stabilization to a depth of 14 inches using cement. Since no planned subgrade modifications are indicated, we recommend consideration be given to over-excavation and replacement with new granular engineered fill, if required during construction.

10. A design CBR value of 10 percent was calculated for the project area based on the GB-1 "Subgrade Analysis" worksheet, which considers an average condition of all of the soil types included in the GB-1 analysis. Group indices for the tested samples ranged from 0 to 10, which would correlate with a CBR value of 6 to 12 percent. Based on the proximity of the cohesive soils with higher GI, and associated lower CBR support, to the subgrade elevation in the western portion of the project area, consideration should be given to use of a lower CBR value. Group Indices associated with these soils tend to correlate with the lower CBR values of 6 to 7 percent compared to the GB-1 Design CBR value that was calculated based on the **average** Group Index value. These clays may govern the overall subgrade conditions. **As such, we recommend that the selected replacement pavement section incorporate a design CBR value of 6 percent**, or as an alternate, check that the pavement design is not sensitive to a variation in CBR value from 6 to 9 percent for the design traffic loading.

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

TABLE OF CONTENTS (Continued)

PLATES

FIGURES

APPENDICIES

1.0 INTRODUCTION

This structure foundation exploration report has been prepared for the replacement of the existing Central Avenue Bridge over Ottawa River in Toledo, Lucas County, Ohio. The project area is located approximately 850 feet west of Valleyview Drive. The general project area is shown on the Site Location Map (Plate 1.0).

This exploration was performed in general accordance with TTL Proposal No. 1771201, dated November 26, 2018, and was initially authorized by Mr. Douglas D. Miller, P.E., S.I. of TRC via email on January 17, 2019. The exploration was formally authorized with a Subconsultant Service Agreement dated February 19, 2019, along with Purchase Order 134035 dated February 25, 2019.

1.1 Purpose and Scope of Exploration

The purpose of this exploration was to evaluate the subsurface conditions relative to the design and construction of foundations for a new bridge structure, as well as design and construction of pavements 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 a review of available geologic and soils data for the project area.

This report summarizes our understanding of the proposed construction, describes the investigative and testing procedures utilized to evaluate the subsurface conditions at the site, and presents our findings from the field and laboratory testing. This report also presents our evaluations and conclusions in accordance with ODOT GB-1 “Plan Subgrades” (January 18, 2019), as well as provides our design and construction recommendations for foundations for the proposed bridge replacement structure.

This report includes:

- A description of the subsurface soil, rock, and groundwater conditions encountered in the borings.
- Design recommendations for bridge foundations and pavements.
- Recommendations concerning soil-, rock-, and groundwater-related construction procedures such as site preparation, earthwork, foundation and pavement construction, as well as related field testing.

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

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

1.2 Proposed Construction

It is our understanding that the existing two-span bridge will be replaced with a new three-span structure. It is planned to support the structure using drilled shafts socketed into bedrock. Maximum total factored loads were indicated to be 687 kips for abutments and 1,101 kips for piers.

Roadway approach grades west of the bridge will be raised an average of approximately 1.1 foot. Negligible grade change is planned east of the bridge. New pavement cross-sections will be on the order of 1.5 feet in thickness.

2.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1 General Geology and Hydrogeology

Published geologic maps from the Ohio Department of Natural Resources (ODNR) indicate that the project site is located in the Maumee Sand Plains District of the Huron-Erie Lake Plains Physiographic Region of Ohio. Within this district, the predominant geologic deposits consist of sandy beach ridge and outwash soils overlying lacustrine (lake-bed) sediments and glacial till.

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

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

Underlying the soils, bedrock consists of sedimentary formations deposited during the late Silurian Age of the Paleozoic Era. The Toledo area is broadly mapped as the Monroe formation, and specific to the project site, the bedrock is identified as Tymochtee dolomite. Within the predominantly dolomite formation, interbedded shales may also be present. Bedrock in the project vicinity is mapped at approximately Elevs. 580 to 570. In the borings completed for this exploration, bedrock was encountered at depths on the order of 26½ to 27 feet below existing grades (Elev. 584±).

The USDA Natural Resource Conservation Service (NRCS) Web Soil Survey indicates that soils in the project area are predominantly mapped as Eel loam and Sisson loam. The Eel loam soils formed in alluvium on flood plains, and are characterized as moderately well drained. The Sisson loam soils formed along deltas on lake plains, and are characterized as well drained.

2.2 Observations of the Project

Based on the original plans for the existing bridge structure prepared by the State of Ohio Department of Highways (now ODOT), dated February 1928, the existing bridge consists of a two-span structure, with each span approximately 64 feet in length. The bridge is shown to bear on footings to rock at Elevs. 582± to 581±. The Normal Water Level is indicated at Elev. 591.84, and the river bottom is indicated at Elev. 590.7.

TTL performed site reconnaissance on March 22, 2019 and June 23, 2019. We observed the bridge consisted of two spans. A utility duct was located along the northern side of the bridge. A sewer outlet was present along the western bank, south of the bridge. Silt build-up was present along the eastern portion of the river, south of the bridge. A soil island was present around the northern portion of the pier, and the island contained mature trees.

Pavements were in generally good condition. Roadway grades at the crossing were lower than grades to the east and west of the crossing.

3.0 EXPLORATION

3.1 Historic Borings

Borings were performed during 1968 along the Central Avenue alignment for roadway construction. One test boring was performed within 100 feet of Ottawa River, west of the western bank. The boring was indicated at Sta. 598+75, offset 35 feet right. No boring number was indicated. Therefore, the boring is identified on the attached Test Boring Location Plan (Plate 2.0) as Boring B-598-0-68. Underlying topsoil at the surface of Boring B-598-0-68, sandy silt (ODOT A-4a) was encountered to a depth of 4 feet (Elev. 608±), underlain by coarse and fine sand (ODOT A-3a) to a depth of 6 feet (Elev. 606±), which was underlain by silt and clay (ODOT A-6a) to termination at a depth of 10 feet (Elev. 602±).

Another boring performed in 1968 was located at Sta. 603+20, offset 25 feet right. This is beyond the project area shown on Plate 2.0. However, boring data for the nearby borings are provided in Appendix D "Historic Borings." The boring performed at Sta. 603+20 was extended to termination at a depth of 20 feet (Elev. 591±). This boring encountered predominantly sandy silt (ODOT A-4a), although zones of coarse and fine sand (ODOT A-3a) were also encountered.

3.2 Project Exploration Program

This exploration included two test borings, designated as Borings B-001-0-18 and B-002-0-18, performed by TTL on April 3 and 4, 2019. These borings are fully designated as Borings B-001-0-18 and B-002-0-18 in accordance with ODOT protocol, but the "-0-18" portion of the nomenclature is generally omitted for ease of identification in the discussions within this report. The borings were located in the field by TTL based on the provided plan for the existing structure and coordination with TRC. The borings were performed through the existing roadway, with one located behind each existing abutment. The approximate locations of the borings are shown on the Test Boring Location Plan (Plate 2.0).

Boring B-002 was initially advanced to a depth of 17½ feet below existing grade, and then was offset slightly since the augers were not plumb in the initial borehole. The boring is shown on one test boring log since the boring offset was not due to subsurface conditions. The ground surface elevation at the original and offset locations varied by less than 0.1 foot. The boring data provided on the boring log is based on the offset location, since foundations to bedrock are anticipated and the bedrock was encountered in the offset boring.

Stations, offsets, coordinates, ground surface elevations, and coordinates at the boring locations were provided by TRC. This boring data, as well as boring termination depths, are summarized in the following table.

Table 3.2 General Boring Information

Boring Number	Location	Latitude (degrees)	Longitude (degrees)	Station	Offset	Ground Surface Elevation (feet)	Boring Termination Depth (feet)	Boring Termination Elevation (feet)
B-001	Rear Abutment	41.676806	-83.660462	598+82	24’ RT	611.0	37.3[1]	573.7
B-002	Forward Abutment	41.676952	-83.659611	601+16	21’ LT	610.7	39.7[1]	571.0

[1] Includes 10 feet of rock coring.

In accordance with the ODOT Specifications for Geotechnical Explorations (SGE, 2007 and current revisions), the borings were planned ODOT Type E1 structure borings, planned to encounter bedrock, with the upper portion of the borings planned as ODOT Type A roadway borings. Borings B-001 and B-002 encountered auger refusal at depths of 27.3 feet and 29.7 feet below existing grades, respectively. Upon encountering auger refusal, the borings were then advanced by coring 10 feet into the underlying bedrock.

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

3.3 Boring Methods

The test borings performed during this exploration were drilled with a CME 550 ATV-mounted drilling rig. The borings were extended utilizing 3¼-inch inside diameter hollow-stem augers. During auger advancement, samples were generally taken at 2½-foot intervals that were planned to a depth of 30 feet, although auger refusal was encountered in each boring prior to obtaining this depth. Continuous sampling to obtain roadway subgrade soil samples was performed for 6 feet starting at a depth of 1 foot below existing grade using 18-inch sample drives. Additional sampling for evaluation of potential scour was performed starting at approximately 22½ feet below existing grades using 18-inch sample drives. The samples were sealed in jars and transported to our laboratory for further classification and testing.

Split-spoon (SS) soil samples were obtained by the Standard Penetration Test Method (ASTM D 1586). The Standard Penetration Test (SPT) consists of driving a 2-inch outside diameter split-spoon sampler into the soil with a 140-pound weight falling freely through a distance of 30 inches. The sampler was driven in three successive 6-inch increments, with the number of blows per increment being recorded, and these data are presented under the "SPT" column on the Logs of Test Borings attached to this report. The sum of the number of blows required to advance the sampler the second and third 6-inch increments is termed the Standard Penetration Resistance, or N_m-value, and is typically reported in blows per foot (bpf). The N_m-values were corrected to an equivalent rod energy ratio of 60 percent, N_{60}. The hammer/rod energy ratio was 77.3 percent for the CME 550 ATV-mounted drill rig utilized on this project, based on calibration performed on February 20, 2019. The N_{60}-values are presented on the attached Logs of Test Borings, as well as the Tabulation of Test Data sheets attached to this report. In conjunction with published data and typical correlations, the N_{60}-values can be evaluated as a measure of soil compactness/consistency as well as shear strength and bearing capacity.

Shelby tube samples, designated ST on the Logs of Test Borings, were obtained in Borings B-001 (8 to 10 feet) and B-002 (18 to 20 feet) by hydraulically advancing a 3-inch diameter, thin-walled sampler approximately 24 inches beyond the hollow-stem auger into relatively undisturbed soil in accordance with ASTM D 1587. The Shelby 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. In Boring B-001, two 5-foot core runs were completed immediately following auger refusal. In Boring B-002, three rock core runs were performed for a total of 10 feet. Recovery of the core is expressed as the percentage ratio of the recovered rock length to the total length of the core run. The Rock Quality Designation (RQD) is the percentage ratio of the summed length of rock pieces 4 inches long and greater to the total length of the run. The rock core samples are designated as "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 in Appendix C.

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

3.4 Laboratory Testing Program

All soil samples were visually or manually classified in accordance with the ODOT Soil Classification System. All samples of the subsoils were also tested in our laboratory for moisture content (ASTM D 2216). Atterberg limits tests (ASTM D 4318) and particle size analyses (ASTM D 422) were performed on selected samples to determine soil classification and index properties. Dry density determinations and unconfined compressive strength tests (ASTM D 2166) by the constant rate of strain method were performed on selected intact cohesive split-spoon samples. Unconfined compressive strength estimates were obtained for the remaining intact cohesive samples using a calibrated hand penetrometer. These test results are presented on the Logs of Test Borings and Tabulation of Test Data sheets attached to this report.

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

4.0 FINDINGS

4.1 General Site Conditions

The project site is located along Central Avenue, at the crossing of Ottawa River, approximately 850 feet west of Valleyview Drive, in Toledo, Lucas County, Ohio. Roadway grades in the project area are generally level, with ground surface elevations at the boring locations on the order of Elev. 611.

The borings were performed within the roadway, and the encountered surface materials consisted of approximately 6 inches of asphalt underlain by approximately 8 to 9 inches of concrete, underlain by 6 to 11 inches of crushed stone. An approximately 3 inches zone of crushed stone with sand, silt and clay was encountered underlying the “clean” crushed stone base layer in Boring B-001.

Cohesive **fill** materials were encountered underlying the surface materials to depths of 4 feet below existing grade (Elev. 607±) in Boring B-001 and approximately 3½ feet (Elev. 607±) in Boring B-002. The cohesive fill materials consisted of sandy silt with trace clay, and varying amounts of crushed stone. SPT N_{60}-values ranged from 14 to 41 blows per foot (bpf), indicating generally very stiff to hard consistency. Moisture contents ranged from 11 to 14 percent.

Granular **fill** materials were encountered underlying the cohesive fill materials to a depth of 8 feet (Elev. 603±) in Boring B-002. The granular fill materials consisted of predominantly crushed stone with sand and varying amount of silt and clay, as well as coarse and fine sand with some silt, crushed stone and trace clay. SPT N_{60}-values ranged from 14 to 31 bpf, indicating medium dense to dense compactness. Moisture contents ranged from 9 to 12 percent.

4.2 General Soil Conditions

The subsoils encountered underlying the surface and fill materials can be generally described as predominantly cohesive soils exhibiting varying strength and moisture characteristics, with zones of granular soils, overlying bedrock.

Stratum I consisted of predominantly medium stiff to stiff cohesive soils encountered underlying the surface and fill materials in Borings B-001 and B-002 to depths of 11 feet below existing grade (Elev. 600±) and 23 feet (Elev. 588±). These cohesive soils consisted of sandy silt (ODOT A-4a) as well as silt and clay (ODOT A-6a). SPT N_{60}-values ranged from 6 to 13 blows per foot (bpf). Unconfined compressive strengths generally ranged from 1,000 to 3,000 pounds per square foot (psf). Moisture contents ranged from 15 to 22 percent.

In Boring B-002, zones of predominantly **loose** to medium dense granular soils were encountered within Stratum I at depths of 11 to 14 feet (Elevs. 600± to 597±) and 18 to 20 feet (Elevs. 593± to 591±). These granular soils consisted of coarse and fine sand (ODOT A-3a). An SPT N_{60}-value of 9 bpf was determined for the upper-profile granular zone. Moisture contents of 16 percent and 20 percent were determined for the granular soil zones.

Stratum II consisted of predominantly very stiff to hard cohesive soils encountered underlying Stratum I in Boring B-001 to a depth of approximately 27 feet (Elev. 584±). These cohesive soils consisted of sandy silt (ODOT A-4a), silt and clay (ODOT A-6a), and silty clay (ODOT A-6b). SPT N_{60}-values ranged from 17 to 94 bpf. Unconfined compressive strengths generally ranged from 5,235 psf to greater than 9,000 psf (the maximum obtainable reading using a calibrated hand penetrometer). Moisture contents ranged from 9 to 16 percent.

In Boring B-002, a zone of predominantly medium dense granular soils was encountered underlying Stratum II to a depth of 26½ feet (Elev. 584±). These granular soils consisted of gravel and stone fragments with sand (ODOT A-1-b) as well as coarse and fine sand (ODOT A-3a). SPT N_{60}-values of 15 bpf and 28 bpf were determined for these granular soils. Moisture contents of 11 percent and 23 percent were determined for the recovered samples.

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

4.3 General Bedrock Conditions

Augerable weathered dolomite was encountered underlying the subsoils in Borings B-001 and B-002 at depths of approximately 27 feet (Elev. 584±) and 26½ feet (Elev. 584±), respectively, extending to depths of approximately 27½ feet (Elev. 583½) and 29½ feet (Elev. 581±), respectively.

Underlying the weathered dolomite, auger refusal on dolomite bedrock was encountered. The rock was cored in each of the borings for a total length of 10 feet. The cored bedrock consisted of slightly weathered to moderately weathered dolomite. **The driller noted tool drop from approximately 32 to 33 feet in Boring B-002, possibly indicating a soil-filled zone or a void.** The rock core data obtained from the borings are summarized as follows:

Table 4.3. Rock Core Data							
Boring Number	**Rock Core Number**	**Depth (feet)**	**Approximate Elevation (feet)**	**Recovery (%)**	**RQD (%)**	**Unconfined Compressive Strength Test Specimen Depth (feet)**	**Unconfined Compressive Strength (psi)**
B-001	RC-1	27.3 – 32.3	583.7 – 578.7	55	28	27.3	11,650
	RC-2	32.3 – 37.3	578.7 – 573.7	100	78	32.7 35.0	3,350 5,380
B-002	RC-1	29.7 – 33.3	581.0 – 577.4	86	38	29.7	19,250
	RC-2	33.3 – 34.7	577.4 – 576.0	89	33	33.9	10,140
	RC-3	34.7 – 39.7	576.0 – 571.0	100	78	35.3	4,950

RQD values ranged from 28 to 78 percent, indicating that the overall rock mass quality in the upper profile can be generally described as poor transitioning to good with increased depth. Unconfined compressive strength results generally ranged from 4,950 to 11,650 pounds per square inch (psi), indicating moderately strong to strong bedrock. An unconfined compressive strength of 3,350 psi was determined for a specimen from boring B-001 (RC-2), indicating slightly strong bedrock.

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

4.4 Groundwater Conditions

Provided drawings for the existing structure, dated 1968, indicate normal water level at Elev. 591.8± and high water level at Elev. 600.5±.

Groundwater was initially encountered during drilling in Borings B-001 and B-002 at depths of 25½ feet below existing grade (Elev. 585.5) and 23 feet (Elev. 587.7), respectively. Groundwater was present in the borings at shallower depths upon completion of the drilling and rock coring operations, but these levels may have been affected by water introduced into the borings for rock coring operations. It should be noted that each boring was drilled and sealed within the same day. Therefore, stabilized water levels may not have occurred over this limited time period. Instrumentation was not installed to observe long-term groundwater levels.

Based on the soil characteristics and moisture conditions encountered in the borings, it is our opinion that "normal" groundwater levels at this structure location will generally occur at Elevs. 600± to 595±, corresponding to depths at or slightly above the streamflow levels in Ottawa River. It should be noted that groundwater elevations can also fluctuate with seasonal and climatic influences, as well as streamflow conditions in the river. Therefore, the groundwater conditions may vary at different times of the year from those encountered during this exploration.

4.5 Gradation Results for Potential Scour Evaluations

Bridge foundations are planned to provide resistance to vertical loads via drilled shafts socketed into bedrock. The upper portion of the bedrock was weathered and highly fractured such that there is potential for scour. However, as will be discussed in Section 5.1.1 of this report, we recommend resistance not be considered in these weathered/highly fractured zones that extend approximately 4½ to 7½ feet below top of rock. In any case, potential scour considerations may be applicable to lateral load evaluations.

Based on the provided plans, Ottawa River bottom is at approximately Elev. 590 or higher. Therefore, approximately 5½ feet or more of soil is anticipated above top of rock. If final design includes potential scour associated with abutment or pier drilled shaft foundations, gradation results are provided below for evaluation of potential scour of the soils at and below the river bottom.

Particle size analyses were performed on selected samples from Borings B-001 and B-002, obtained within a depth of approximately 6 feet below the indicated river bottom elevation. The particle size analyses were performed to determine D_{50} values of the soils to facilitate scour analysis. Based on the tested samples, D_{50} values ranged from 0.015 millimeters (mm) to 1.22 mm. The results for the soil samples within the estimated potential scour zone are summarized as follows:

Table 4.5. Gradation Results for Potential Scour Evaluation

Boring Number (Associated Abutment)	Sample Number	Sample Depth (feet)	Approximate Sample Elevation (feet)	D_{50} (mm)
B-001 (Rear Abutment)	SS-11	22.5 – 24.0	588.5 – 587	0.018
	SS-12	24.0 – 25.5	587 – 585.5	0.015
	SS-13	25.5 – 26.8	585.5 – 584	0.015
B-002 (Forward Abutment)	SS-10	21.0 – 22.5	589.5 – 588	0.076
	SS-11	22.5 – 24.0	588 – 586.5	0.199
	SS-12	24.0 – 25.5	586.5 – 585	1.217

It should be noted that specific borings were not drilled for the piers as part of this exploration. Recovered soil samples evaluated for potential scour were from borings performed behind the existing abutments. As such, actual soil conditions and potential scour at the piers may vary from the conditions encountered in the borings performed near the abutments.

It is our understanding that design considerations for pier foundations will be based on scour removal of the soils at the river bottom, as well as the upper portion of the bedrock profile that was weathered/highly fractured. Similar to the non-contributing zones of weathered/highly fractured rock for vertical resistance for the rock socket bridge foundations (Section 5.1.1 of this report), rock being considered for potential scour in design consists of the upper 4½ feet and 7½ feet of the rock profile encountered in Borings B-001 and B-002, respectively.

4.6 Remedial Measures

Based on the relative proximity of bedrock to the proposed foundation pier/pile caps, we understand that the new bridge is planned to be supported by a deep foundation system consisting of drilled shafts socketed into bedrock.

It should be noted that ODOT design methods recommend that the contribution of skin friction be neglected in the upper 2 feet of the rock socket. However, with the exception of a half-foot zone, the upper portion of the bedrock extending approximately 4½ feet below top of rock in Boring B-001 exhibited an RQD of 0 percent, prior to encountering more competent bedrock. In Boring B-002, the upper approximately 3½ feet of rock was penetrable with augers. Additionally, in Boring B-002, a potential void or zone of soil-filled joint(s) was encountered from 5½ to 6½ feet below top of bedrock, with the underlying approximately 1 foot exhibiting an RQD of only 15 percent. Therefore, competent rock was not encountered until a depth of approximately 7½ feet below top of bedrock.

We recommend that the contribution of resistance be modeled starting below the 4½ feet and 7½ feet zones of particularly weathered/fractured (and possibly void containing in the case of Boring B-002) rock. It is our understanding that pier foundation evaluations will consider scour removal of the soils at the river bottom, as well as these zones of weathered/fractured rock.

Using GB-1 criteria based on the encountered conditions, no planned subgrade modification was indicated. If planned subgrade modification was indicated, GB-1 indicated an option for global chemical stabilization to a depth of 14 inches using cement. Since no planned subgrade modifications are indicated, we recommend consideration be given to over-excavation and replacement with new granular engineered fill, if required during construction.

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

5.0 ANALYSES AND RECOMMENDATIONS

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

5.1 Bridge Foundations

5.1.1 Vertical Load Evaluations

We understand that the bridge foundation will be designed using LRFD methods. Based on the relative proximity of bedrock to the proposed foundation pier/pile caps, we understand that the new bridge is planned to be supported by a deep foundation system consisting of drilled shafts socketed into bedrock.

It should be noted that ODOT design methods recommend that the contribution of skin friction be neglected in the upper 2 feet of the rock socket. However, with the exception of a half-foot zone, the upper portion of the bedrock extending approximately 4½ feet below top of rock in Boring B-001 exhibited an RQD of 0 percent, prior to encountering more competent bedrock. In Boring B-002, the upper approximately 3½ feet of rock was penetrable with augers. Additionally, in Boring B-002, a potential void or zone of soil-filled joint(s) was encountered from 5½ to 6½ feet below top of bedrock, with the underlying approximately 1 foot exhibiting an RQD of only 15 percent. Therefore, competent rock was not encountered until a depth of approximately 7½ feet below top of bedrock.

We recommend that the contribution of resistance be modeled starting below the 4½ feet zone (B-001) and the 7½ feet zone (B-002) of particularly weathered/fractured (and possibly void containing in the case of Boring B-002) rock.

Settlement associated with drilled shafts socketed into intact bedrock is expected to be ½ inch or less. Commentary in AASHTO LRFD Bridge Design Specifications (C.10.8.3.5.4d) indicates that axial capacity is typically taken solely in skin friction for rock sockets exhibiting 0.4 inches or less movement. Therefore, we have based on evaluation on resistance provided solely by skin friction.

Recommended design values for evaluation of drilled shafts socketed into dolomite bedrock are summarized in the following tables. Design values are provided based on Borings B-001 and B-002 so evaluations can be made for substructures considering the more pertinent boring location.

Table 5.1.1.A. Rock Socket Design Parameters – Boring B-001 (Rear Abutment & Pier 1)

Rock Zone		**Approximate Depth Range (feet)**	**Elevation Range (feet)**	**Unfactored Unit Side Resistance (qs) [ksf]**	**Factored Unit Side Resistance (ksf)[1]**
1	Predominantly Augerable Weathered/Fractured Rock (RQD = 0%)	27 to 31½	584 to 579½	-	-
2	More competent rock RQD = 70%, UCS = 3,350 psi	31½ to 35	579½ to 576	18	10
3	More competent rock RQD = 100%, UCS = 5,380 psi	35 to 37½	576 to 573½	22	12
4	Rock beyond exploration in Boring B-001	37½+	573½-	Presumed same as Layer 3	

[1] Based of ϕ_{stat} = 0.55

Table 5.1.1.B. Rock Socket Design Parameters – Boring B-002 (Forward Abutment and Pier 2)

Rock Zone		**Approximate Depth Range (feet)**	**Elevation Range (feet)**	**Unfactored Unit Side Resistance (qs) [ksf]**	**Factored Unit Side Resistance (ksf)[1]**
1	Predominantly Augerable Weathered/Fractured Rock (RQD = 0% to 50%) and Potential Void or Soil-Filled Joint(s)	26½ to 34	584 to 576½	-	-
2	More competent rock RQD = 76%, UCS ≥ 4,950 psi	34 to 39½	576½ to 571	22	12
3	Rock beyond exploration in Boring B-002	39½+	571-	Presumed same as Layer 2	

[1] Based of ϕ_{stat} = 0.55

For design considerations, estimation of drilled shaft resistances in the dolomite bedrock was based on AASHTO 10.8.3.5.4, using research by Horvath and Kenney (1979) as well as O‘Neill and Reese (1999). Estimation of reduction factors to account for jointing in rock were based on AASHTO Table 10.8.3.5.4b-1 (O’Neill and Reese, 1999) for analysis of shaft resistance, q_s. Fractured rock mass parameters to account for jointing in end-bearing capacity were based on AASHTO Table 10.4.6.4-4 (Hoek and Brown, 1988) considering rock mass quality observed in the recovered rock cores. Per ODOT guidance for drilled shafts, Class S Modified concrete with a 28-day strength (f'_c) of 4,000 pounds per square inch (psi) was incorporated into the analysis. Based on the design methodologies utilized to evaluate unfactored unit side resistance and AASHTO LRFD Table 10.5.5.2.4-1, a resistance factor of 0.55 for side resistance should be utilized for design, as indicated for Tables 5.1.A and 5.1.B.

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

Based on the indicated pier maximum total factored load of 1,101 kips, our calculations indicate that suitable resistance can be provided using a 3-foot diameter rock socket extending a minimum of 15 feet into rock considering Boring B-001 (Pier 1) and extending a minimum of 17½ feet into rock considering Boring B-002 (Pier 2). Based on the indicated maximum abutment total factored load of 687 kips, our calculations indicate that suitable resistance can be provided using a 2½-foot diameter rock socket extending a minimum of 12½ feet into rock considering Boring B-001 (Rear Abutment) and extending a minimum of 15 feet into rock considering Boring B-002 (Forward Abutment). Any structural requirement for the drilled shaft foundations to resist lateral loads or moments may increase the socket depth or diameter and should be evaluated on an individual shaft basis. Recommended soil parameters for these analyses are provided in Section 5.1.2.

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

Due to the presence of groundwater, as well as the granular soils encountered in Boring B-002, it is likely that temporary steel casing will be required to support the walls of the shaft and to control groundwater seepage. If significant seepage is encountered and cannot be suitably pumped to dewater the drilled shaft, concrete will require placement by tremie methods. As the steel casing is withdrawn during concreting, sufficient concrete should be maintained above the bottom of the casing to counteract any hydrostatic head. Care must be taken during concreting and removal of any temporary liner so as to avoid the possibility of soil intrusions. The contractor should submit procedures for installation prior to the start of work.

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

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

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

> The maximum factored load to be supported by each drilled shaft is 1,101 kips at Pier 1. Theoretically, this load is resisted entirely by side resistance within a portion of the bedrock socket, without any tip resistance. The factored resistance developed by side resistance is 1,121 kips, assumed to act along the bottom 10½ feet of the bedrock socket for Pier 1, assuming a minimum 15-foot socket embedment.

> The maximum factored load to be supported by each drilled shaft is 1,101 kips at Pier 2. Theoretically, this load is resisted entirely by side resistance within a portion of the bedrock socket, without any tip resistance. The factored resistance developed by side resistance is 1,131 kips, assumed to act along the bottom 10 feet of the bedrock socket for Pier 2, assuming a minimum 17½-foot socket embedment.

> The maximum factored load to be supported by each drilled shaft is 687 kips at the Rear Abutment. Theoretically, this load is resisted entirely by side resistance within a portion of the bedrock socket, without any tip resistance. The factored resistance developed by side resistance is 699 kips, assumed to act along the bottom 8 feet of the bedrock socket for the Rear Abutment, assuming a minimum 12½-foot socket embedment.

> The maximum factored load to be supported by each drilled shaft is 687 kips at the Forward Abutment. Theoretically, this load is resisted entirely by side resistance within a portion of the bedrock socket, without any tip resistance. The factored resistance developed by side resistance is 707 kips, assumed to act along the bottom 7½ feet of the bedrock socket for the Forward Abutment, assuming a minimum 15-foot socket embedment.

5.1.2 Lateral Load Evaluations

For lateral load-deflection evaluations using software, such as LPILE, recommended design parameters are summarized in the following tables based on the conditions encountered in the borings. Design values are provided based on Borings B-001 and B-002 so evaluations can be made for substructures considering the more pertinent boring location.

Table 5.1.2.A. Subsurface Conditions and Recommended Lateral Load-Deflection Parameters – Boring B-001 (Rear Abutment and Pier 1)

Average Depth (feet)	Approximate Elevation (feet)	Generalized Layer Description	Approximate Total Unit Weight[1] (pcf)	Average Undrained Shear Strength, Su (psf)	Strain at 50% Maximum Stress, ε_{50}	Young's Modulus, Er (psi)	Rock Uniaxial Compressive Strength (psi)	k_{rm}
0 to 4	611 to 607	Very Stiff Cohesive Embankment Fill	130	3,500	0.005	–	–	–
4 to 11	607 to 600	Medium Stiff to Stiff Cohesive Soils	130	1,000	0.010	–	–	–
11 to 21	600 to 590	Very Stiff to Hard Cohesive Soils	135	3,000	0.005	–	–	–
21 to 27	590 to 584	Hard Cohesive Soils	130	4,500	0.005	–	–	–
27 to 27½	584 to 583½	Weathered Dolomite[2]	135	4,500	0.005	–	–	–
27½ to 31½	583½ to 579½	Dolomite Bedrock RQD = 0%	150	–	–	500,000	3,000	0.0005
31½ to 35	579½ to 576	Dolomite Bedrock RQD = 70%	150	–	–	500,000	3,350	0.0005
35 to 37½	576 to 573½	Dolomite Bedrock RQD = 100%	150	–	–	500,000	5,380	0.0005
37½ and deeper	573½ and deeper	Beyond exploration in Boring B-001	Presumed same as layer above.					

[1] Effective unit weight should be used below a depth of 16 feet (reduce by unit weight of water – 62.4 pcf).

[2] Model as hard cohesive soil.

Table 5.1.2.B. Subsurface Conditions and Recommended Lateral Load-Deflection Parameters – Boring B-002 (Forward Abutment and Pier 2)

Average Depth (feet)	Approximate Elevation (feet)	Generalized Layer Description	Approximate Total Unit Weight[1] (pcf)	Average Undrained Shear Strength, Su (psf)	Strain at 50% Maximum Stress, ε_{50}	Young's Modulus, Er (psi)	Rock Uniaxial Compressive Strength (psi)	k_{rm}
0 to 3½	610½ to 607	Very Stiff Cohesive Embankment Fill	130	1,750	0.007	–	–	–
3½ to 8	607 to 602½	Medium Dense to Dense Granular Soils	120	ϕ=36°	k=90 pci	–	–	–
8 to 11	602½ to 599½	Medium Stiff Cohesive Soils	130	750	0.010	–	–	–
11 to 14	599½ to 596½	Loose Granular Soils	120	ϕ=30°	k=20 pci	–	–	–
14 to 23	596½ to 587½	Medium Stiff to Stiff Cohesive Soils	130	1,000	0.010	–	–	–
23 to 26½	587½ to 584	Medium Dense Granular Soils	120	ϕ=36°	k=60 pci	–	–	–
26½ to 29½	584 to 581	Weathered Dolomite[2]	135	4,500	0.005	–	–	–
29½ to 31½	581 to 579	Dolomite Bedrock RQD = 50%	150	–	–	500,000	3,000	0.0005
31½ to 34	579 to 576½	Dolomite Bedrock RQD = 15% And Potential Soil Infill Zone[2]	135	4,500	0.005	–	–	–
34 to 39½	576½ to 571	Dolomite Bedrock RQD = 76%	150	–	–	500,000	4,950	0.0005
39½ and deeper	571 and deeper	Beyond exploration in Boring B-002	Presumed same as layer above.					

[1] Effective unit weight should be used below a depth of 11 feet (reduce by unit weight of water – 62.4 pcf).

[2] Model as hard cohesive soil.

It is our understanding that design considerations for pier foundations will be based on scour removal of the soils at the river bottom as well as the upper portion of the bedrock profile consisting of weathered/highly fractured rock. For design, the scour depth below top of rock is considered 4½ feet (to a depth of 31½ feet below top of pavement) and 7½ feet (to a depth of 34 feet below top of pavement) in Borings B-001 and B-002, respectively.

5.2 GB-1 "Plan Subgrades" Evaluation

ODOT Geotechnical Bulletin GB-1 "Plan Subgrades" (January 18, 2019) was utilized to evaluate the subgrade soils encountered in Borings B-001 and B-002, which were located in the roadway. Evaluations included completion of the ODOT "Subgrade Analysis" worksheet (V.14.5).

Roadway approach grades west of the bridge will be raised an average of approximately 1.1 foot. Negligible grade change is planned east of the bridge. New pavement cross-sections will be on the order of 1.5 feet in thickness.

The conditions encountered in Boring B-001 were used to model the subgrade conditions at the beginning of project where new pavement grades will tie into existing pavement grades. On the GB-1 spreadsheet, this boring scenario was labeled as Boring B-001-1-18, and subgrade was considered 1.5 feet below top of existing pavement. For the western portion of the project where grades will be raised an average of 1.1 feet, the spreadsheet evaluation was identified as Boring B-001-0-18, and subgrade was considered at a depth of 0.4 feet. Since final grades were not varying significantly from existing grades east of the bridge, the analysis for Boring B-002-0-18 included subgrade at a depth of 1.5 feet.

Based on GB-1, soils classified as ODOT A-4b, A-2-5, A-5, A-7-5, A-8a, A-8b, or rock have been designated as being problematic with respect to pavement subgrade support. None of these soils types were encountered within in the upper 6 feet of the subgrade soils during this exploration. The subgrade materials tested during this exploration were found to consist of A-2-4, A-2-6, A-3a, A-4a, and A-6a soils.

The moisture content for only two of the evaluated samples within the upper 6 feet of the subgrade were greater than 3 percent higher than optimum as determined using GB-1 criteria. Based on GB-1 criteria, subgrade soils with moisture contents greater than 3 percent above optimum are likely to require modification. Both of the evaluated samples with moisture contents greater than 3 percent above optimum had moisture contents greater than 5 percent above optimum. One of these samples consisted of cohesive soils, for which scarification and aeration methods may not be feasible to achieve timely satisfactory proof rolling and stabilization of subgrades, depending on the construction schedule and seasonal conditions during subgrade preparation. The other wet sample was granular material, which is generally conducive to scarification and aeration methods, provided the construction schedule can facilitate this operation.

The type and depth of subgrade modification is determined by GB-1 criteria based on soil type, moisture content, and the average, low SPT N_{60}-value (N_{60L}) of the subgrade soils in a particular portion of the project area. Using GB-1 criteria based on the encountered conditions, no planned subgrade modification was indicated.

If planned subgrade modification was indicated, GB-1 indicated an option for global chemical stabilization to a depth of 14 inches using cement. Since no planned subgrade modifications are indicated, we recommend consideration be given to over-excavation and replacement with new granular engineered fill, if required during construction.

If undercut and replacement is performed, the fill should consist of ODOT Item 703.16C, Granular Material Type B or Type C. In all cases, geotextile fabric (referenced in ODOT Item 204, and specified as ODOT Item 712.09, Type D) should be utilized on the subgrade at the bottom of the undercut zone.

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

Sulfate content tests performed on the SS-2 samples from Borings B-001 and B-002 indicated less than 100 parts per million (ppm). Therefore, if it is decided to utilize global chemical stabilization, sulfate content would not preclude use of this method.

5.3 Flexible (Asphalt) Pavement Design

Based on the GB-1 analysis for Borings B-001 and B-002, a design CBR value of 10 percent was determined for the project area. The CBR value calculated by the "Subgrade Analysis" worksheet is based on an average condition of all of the soil types included in the GB-1 analysis. Group indices for the tested samples ranged from 0 to 10, which would correlate with a CBR value of 6 to 12 percent. However, it should be noted that, based on Boring B-001 located west of the bridge, ODOT A-6a soils were encountered at a depth of approximately 2½ feet below top of subgrade, when considering the beginning of the project where pavement grades will meet existing grades, and at a depth of approximately 3½ feet below top of subgrade for the portion of the project where grades will be raised approximately 1 foot. Group Indices associated with these soils tend to correlate with the lower CBR values of 6 to 7 percent compared to the GB-1 Design CBR value that was calculated based on the **average** Group Index value. These clays may govern the overall subgrade conditions. **As such, we recommend that the selected replacement pavement section incorporate a design CBR value of 6 percent**, or as an alternate, check that the pavement design is not sensitive to a variation in CBR value from 6 to 9 percent for the design traffic loading.

It should be noted that the CBR value is based on subgrade compacted to at least 100 percent of the maximum dry density as determined by ASTM D 698 (Standard Proctor) or verified as stable through proof rolling in accordance with Section 5.5.2 of this report.

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

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

5.4 Construction Dewatering and Groundwater Control

Groundwater conditions encountered in the borings were summarized in Section 4.4. Based on the soil characteristics and moisture conditions encountered in the borings, it is our opinion that “normal” groundwater levels at this structure location will generally occur at Elevs. 600± to 595±, corresponding to depths at or slightly above the streamflow levels in Ottawa River. It should be noted that groundwater elevations can also fluctuate with seasonal and climatic influences, as well as streamflow conditions in the river.

Groundwater seepage, perched water, and surface water runoff into these excavations should be controllable by pumping from prepared sumps. Installation of the piers in the Ottawa River may require temporary sheet-pile cutoff walls or cofferdams to divert streamflow to manage groundwater in addition to pumping from prepared sumps. As mentioned in Section 2.1.1, it is likely that temporary steel casing will be required to support the walls of the drilled shafts and to control groundwater seepage. In the event excessive seepage is encountered during construction, TTL should be notified to evaluate whether other dewatering methods are required.

5.5 Construction

5.5.1 Sedimentation and Erosion Control

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

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

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

5.5.2 Site Preparation and Earthwork

Site and subgrade preparation activities should conform to ODOT CMS Item 204 specifications (Subgrade Compaction and Proof Rolling). 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.

After installation of the bridge foundations and backfilling operations, pavement subgrades should be proof rolled in accordance with ODOT CMS 204.06. Using GB-1 criteria based on the encountered conditions, no planned subgrade modification was indicated. If planned subgrade modification was indicated, GB-1 indicated an option for global chemical stabilization to a depth of 14 inches using cement. Since no planned subgrade modifications are indicated, we recommend consideration be given to over-excavation and replacement with new granular engineered fill, if required during construction.

5.5.3 Fill

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

The upper profile on-site soils consist of granular and cohesive soils. As such, the contractor should be prepared to use a sheepsfoot roller to provide effective compaction of the cohesive soils and a smooth-drum roller for effective compaction of the granular soils. In narrow utility or footing excavations, the on-site cohesive soils may be difficult to compact; therefore, a clean granular material may be required in these areas.

5.5.4 Excavations and Slopes

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

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

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

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

For permanent excavations and slopes, we recommend that grades generally be no steeper than 3H:1V. It should be noted that ODOT routinely uses 2H:1V slopes for roadway embankments and spill-through sections. While these steeper slopes may be used, it is our experience that the embankment faces on these slopes are more prone to erosion and sloughing. All slopes along the channel of Ottawa River should be lined with rip-rap or other channel erosion protection.

6.0 QUALIFICATION OF RECOMMENDATIONS

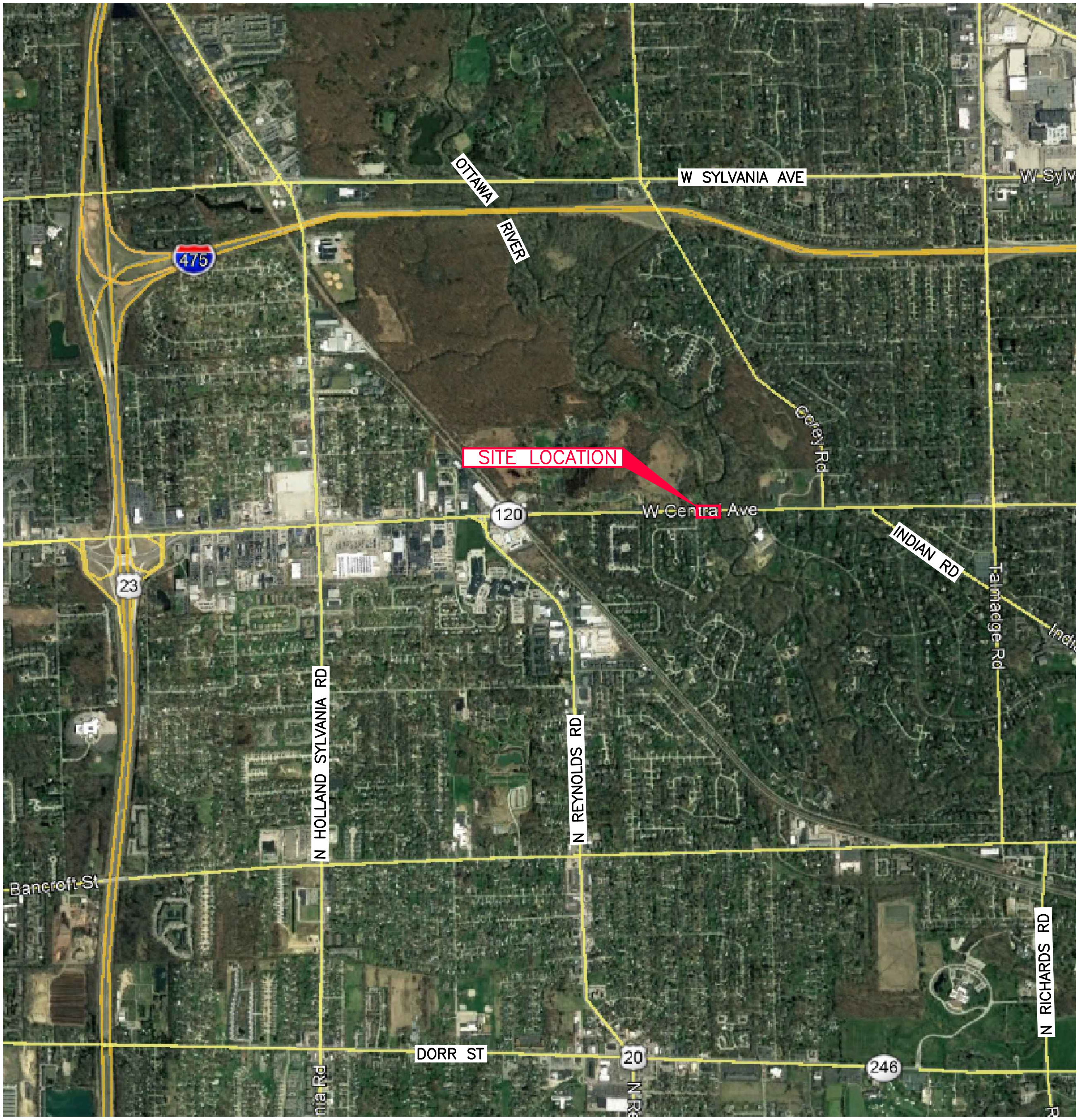
Our evaluation of bridge foundation and roadway pavement design and construction conditions has been based on our understanding of the site and project information and the data obtained during our field investigation. The general subsurface conditions used were based on interpretation of the subsurface data at specific boring locations. Regardless of the thoroughness of a subsurface investigation, there is the possibility that conditions between borings will differ from those at the boring locations, that conditions are not as anticipated by the designers, or that the construction process has altered the soil conditions. 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 2,500 5,000

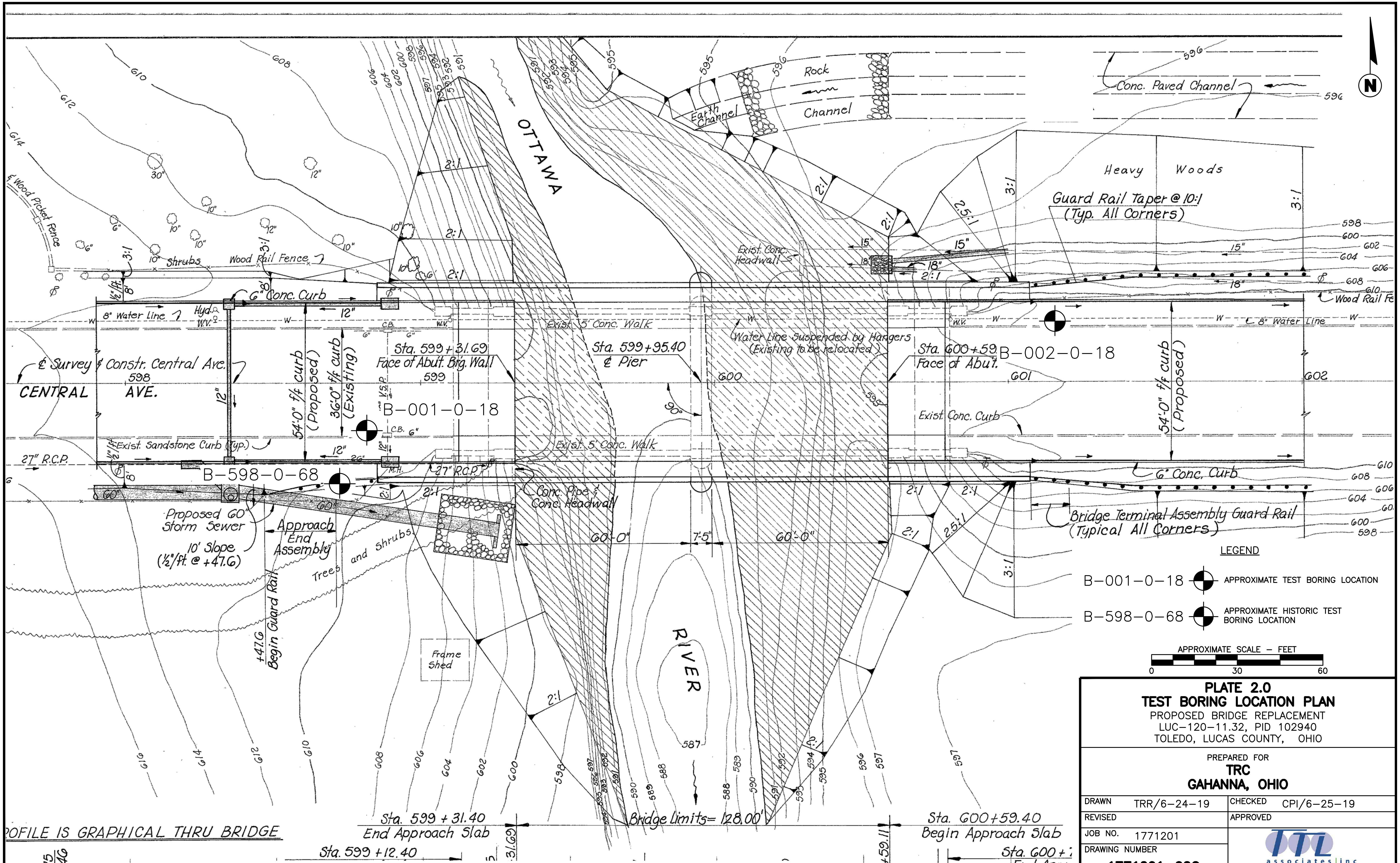
PLATE 1.0
SITE LOCATION MAP

PROPOSED BRIDGE REPLACEMENT
LUC–120–11.32, PID 102940
TOLEDO, LUCAS COUNTY, OHIO

PREPARED FOR
TRC
GAHANNA, OHIO

DRAWN	TRR/6–24–19	CHECKED	CPI/6–25–19
REVISED		APPROVED	
JOB NO.	1771201		
DRAWING NUMBER	**1771201–01G**		

TTL associates inc
Environmental, Geotechnical Engineering & Testing



BASE DRAWING "SITE PLAN" PREPARED BY CHARLES L. BARBER & ASSOCIATES, INC., DATED AUGUST 19, 1968.

PLATE 2.0
TEST BORING LOCATION PLAN
PROPOSED BRIDGE REPLACEMENT
LUC-120-11.32, PID 102940
TOLEDO, LUCAS COUNTY, OHIO

PREPARED FOR
TRC
GAHANNA, OHIO

DRAWN	TRR/6-24-19	CHECKED	CPI/6-25-19
REVISED		APPROVED	
JOB NO.	1771201		
DRAWING NUMBER	**1771201-02G**		

TTL associates inc
Environmental, Geotechnical Engineering & Testing

FIGURES

PROJECT: LUC-120-11.32	DRILLING FIRM / OPERATOR: TTL / TB	DRILL RIG: CME 550X ATV	STATION / OFFSET: 598+82, 24' RT.	EXPLORATION ID B-001-0-18
TYPE: BRIDGE	SAMPLING FIRM / LOGGER: TTL / KKC	HAMMER: CME AUTOMATIC	ALIGNMENT: SR 120	
PID: 102940 SFN:	DRILLING METHOD: 3.25" HSA / NQ	CALIBRATION DATE: 2/20/19	ELEVATION: 611.0 (NAVD88) EOB: 37.3 ft.	PAGE 1 OF 2
START: 4/4/19 END: 4/4/19	SAMPLING METHOD: SPT/ST/NQ	ENERGY RATIO (%): 77.3	LAT / LONG: 41.676806, -83.660462	

MATERIAL DESCRIPTION AND NOTES	ELEV. 611.0	DEPTHS
ASPHALT - 6 INCHES	610.5	
CONCRETE - 9 INCHES	609.7	
CRUSHED STONE - 11 INCHES	608.8	
MEDIUM DENSE, GRAY, **CRUSHED STONE WITH SAND, SILT, AND CLAY**, MOIST **FILL**	608.5	
VERY STIFF TO HARD, BROWN, **SANDY SILT**, SOME CRUSHED STONE AND TRACE CLAY, DAMP **FILL**	607.0	
MEDIUM STIFF, BROWN, **SILT AND CLAY**, SOME SAND AND LITTLE GRAVEL, MOIST	605.0	
STIFF, BROWN, **SILT AND CLAY**, SOME SAND AND TRACE GRAVEL, MOIST	600.0	
VERY STIFF TO HARD, BROWN, **SILT AND CLAY**, LITTLE SAND AND TRACE GRAVEL, DAMP @13': VERY STIFF, SOME SAND	595.0	
HARD, GRAY, **SILTY CLAY**, LITTLE SAND AND TRACE GRAVEL, DAMP	592.5	
VERY STIFF TO HARD, GRAY, **SILTY CLAY**, LITTLE SAND AND TRACE GRAVEL, DAMP	590.0	▼ 591.7

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	HOLE SEALED
					GR	CS	FS	SI	CL	LL	PL	PI			
7 6 5	14	67	SS-1A	NP	-	-	-	-	-	-	-	-	6	A-2-4 (V)	
			SS-1B	NP	-	-	-	-	-	-	-	-	9	A-2-6 (V)	
4 16 16	41	67	SS-2	3.50	21	16	20	35	8	24	16	8	14	A-4a (2)	
6 3 3	8	67	SS-3	0.63*	13	12	12	22	41	27	16	11	17	A-6a (6)	
4 4 6	13	78	SS-4	1.50	-	-	-	-	-	-	-	-	20	A-6a (V)	
		71	ST-5	NI	1	7	20	22	50	28	16	12	18	A-6a (8)	
4 6 7	17	100	SS-6	>4.5	-	-	-	-	-	-	-	-	16	A-6a (V)	
7 9 11	26	100	SS-7	NI	2	8	15	26	49	30	19	11	16	A-6a (8)	
11 14 18	41	100	SS-8	>4.5	-	-	-	-	-	-	-	-	13	A-6b (V)	
5 9 12	27	100	SS-9	4.5*	-	-	-	-	-	-	-	-	11	A-6b (V)	

PID: 102940	SFN:	PROJECT: LUC-120-11.32	STATION / OFFSET: 598+82, 24' RT.	START: 4/4/19	END: 4/4/19	PG 2 OF 2	B-001-0-18

MATERIAL DESCRIPTION AND NOTES	ELEV. 590.0	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
HARD, GRAY, **SILTY CLAY**, LITTLE SAND AND TRACE GRAVEL, DAMP	588.5	22	22 25 40	84	100	SS-10	>4.5	-	-	-	-	-	-	-	-	9	A-6b (V)	
HARD, GRAY, **SANDY SILT**, LITTLE CLAY AND TRACE DOLOMITE FRAGMENTS, DAMP		23	12 14 20	44	100	SS-11	>4.5	5	14	21	41	19	NP	NP	NP	9	A-4a (5)	
@24': SOME CLAY		24 25	12 35 38	94	100	SS-12	2.62*	7	11	17	43	22	19	14	5	9	A-4a (6)	
@25.5': WET SAND SEAM, LITTLE DOLOMITE FRAGMENTS	584.1	W 585.5 26	13 30 50/4"	-	88	SS-13	3.75	10	8	14	44	24	20	15	5	12	A-4a (7)	
GRAY, **WEATHERED DOLOMITE**	583.7	TR 27	50/2"	-	100	SS-14	NP	-	-	-	-	-	-	-	-	6	Rock (V)	
DOLOMITE, GRAY, SLIGHTLY WEATHERED, STRONG, JOINTED - MODERATELY FRACTURED, TIGHT; RQD 100%. @27.3': COMPRESSIVE STRENGTH = 11,650 PSI	583.1	28	28		55	RC-1											CORE	
DOLOMITE, GRAY, HIGHLY WEATHERED, SLIGHTLY STRONG, BRECCIATED, JOINTED - HIGHLY FRACTURED, TIGHT; RQD 0%.	579.6	29 30 31																
DOLOMITE, GRAY, SLIGHTLY WEATHERED, SLIGHTLY STRONG, VUGGY AND CRYSTALLINE, JOINTED - FRACTURED TO MODERATELY FRACTURED, TIGHT; RQD 70%. @32.3' TO 32.7': HIGHLY FRACTURED FRAGMENTS @32.7': COMPRESSIVE STRENGTH = 3,350 PSI	576.0	32 33 34	78		100	RC-2											CORE	
DOLOMITE, GRAY, SLIGHTLY WEATHERED, MODERATELY STRONG, JOINTED - SLIGHTLY FRACTURED, TIGHT; RQD 100%. @35': COMPRESSIVE STRENGTH = 5,380 PSI	573.7	35 36 37 EOB																

NOTES: "*" - UNCONFINED STRENGTH DETERMINED BY ASTM D 2166, "NP" - NON PLASTIC, "NI" - NOT INTACT

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

PROJECT: LUC-120-11.32
TYPE: BRIDGE
PID: 102940 SFN:
START: 4/3/19 END: 4/3/19

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

DRILL RIG: CME 550X ATV
HAMMER: CME AUTOMATIC
CALIBRATION DATE: 2/20/19
ENERGY RATIO (%): 77.3

STATION / OFFSET: 601+16, 21' LT.
ALIGNMENT: SR 120
ELEVATION: 610.7 (NAVD88) EOB: 39.75 ft.
LAT / LONG: 41.676952, -83.659611

EXPLORATION ID
B-002-0-18

PAGE
1 OF 2

MATERIAL DESCRIPTION AND NOTES	ELEV. 610.7
ASPHALT - 6 INCHES	610.2
CONCRETE - 8 INCHES	609.5
CRUSHED STONE - 6 INCHES	609.0
VERY STIFF, BROWN, **SANDY SILT**, TRACE CLAY AND CRUSHED STONE, MOIST **FILL**	607.0
DENSE, GRAY/BROWN, **COARSE AND FINE SAND**, SOME SILT, CRUSHED STONE, AND TRACE CLAY, MOIST **FILL**	606.7
DENSE, GRAY, **CRUSHED STONE WITH SAND AND SILT**, MOIST **FILL**	605.2
MEDIUM DENSE, GRAY, **CRUSHED STONE WITH SAND, SILT, AND CLAY**, MOIST **FILL**	602.7
MEDIUM STIFF, BROWN, **SILT AND CLAY**, SOME SAND AND TRACE GRAVEL, MOIST	599.7
LOOSE, GRAY, **COARSE AND FINE SAND**, LITTLE SILT AND CLAY, MOIST	596.7
STIFF, GRAY, **SANDY SILT**, SOME GRAVEL AND LITTLE CLAY, DAMP	594.2
MEDIUM STIFF TO STIFF, GRAY, **SANDY SILT**, SOME CLAY AND TRACE GRAVEL, MOIST	592.7
BROWN/GRAY, **COARSE AND FINE SAND**, LITTLE SILT, TRACE GRAVEL, AND CLAY, WET	590.7
MEDIUM STIFF TO STIFF, GRAY, **SANDY SILT**, TRACE CLAY, MOIST	

DEPTHS: ▼ 592.2

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)
					GR	CS	FS	SI	CL	LL	PL	PI		
6 7 4	14	67	SS-1A	NP	-	-	-	-	-	-	-	-	9	A-2-4 (V)
			SS-1B	NI	2	21	39	32	6	NP	NP	NP	11	A-4a (1)
6 7 14	27	78	SS-2	NP	20	6	50	23	1	20	15	5	17	A-3a (0)
10 15 9	31	72	SS-3	NP	-	-	-	-	-	-	-	-	9	A-2-4 (V)
9 4 7	14	56	SS-4	NP	-	-	-	-	-	-	-	-	12	A-2-6 (V)
1 2 3	6	78	SS-5	0.75	-	-	-	-	-	-	-	-	22	A-6a (V)
3 3 4	9	100	SS-6	NP	-	-	-	-	-	-	-	-	16	A-3a (V)
3 4 3	9	100	SS-7	NI	22	8	23	35	12	17	16	1	15	A-4a (2)
3 4 5	12	89	SS-8	0.49*	1	5	25	43	26	23	15	8	18	A-4a (7)
		100	ST-9	NP	1	6	76	16	1	NP	NP	NP	20	A-3a (0)

HOLE SEALED

PID: 102940 | SFN: | PROJECT: LUC-120-11.32 | STATION / OFFSET: 601+16, 21' LT. | START: 4/3/19 | END: 4/3/19 | PG 2 OF 2 | B-002-0-18

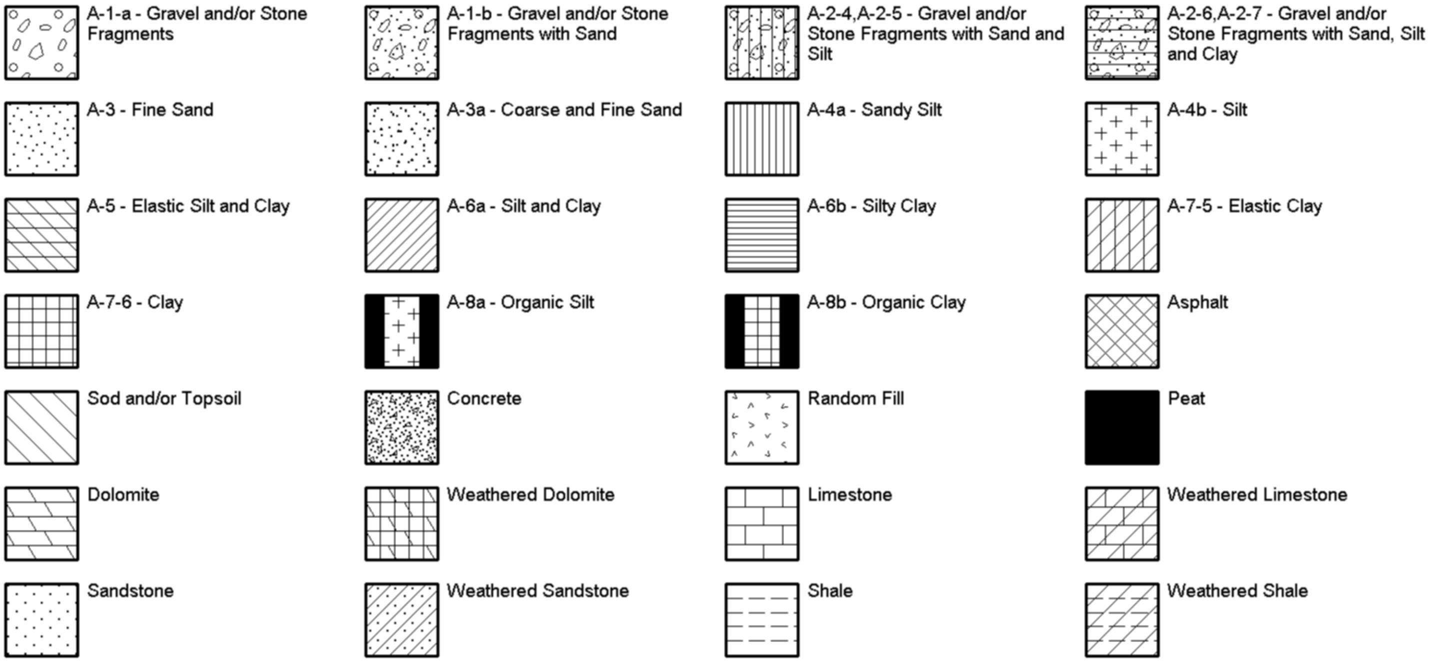
MATERIAL DESCRIPTION AND NOTES	ELEV. 589.7	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
MEDIUM STIFF TO STIFF, GRAY, **SANDY SILT**, TRACE CLAY, MOIST *(continued)*	587.7	W 587.7	2 4 6	13	78	SS-10	0.75	0	2	48	49	1	19	17	2	20	A-4a (3)	
MEDIUM DENSE, GRAY, **COARSE AND FINE SAND**, SOME SILT, TRACE GRAVEL, AND CLAY, WET @23' TO 24': MODERATELY ORGANIC (ORGANIC CONTENT = 5.3%) @23.7': TRACE DOLOMITE FRAGMENTS	586.2		2 4 8	15	100	SS-11	NP	1	8	66	24	1	NP	NP	NP	23	A-3a (0)	
MEDIUM DENSE, GRAY, **GRAVEL AND STONE FRAGMENTS WITH SAND**, LITTLE SILT AND TRACE CLAY, WET (FREE WATER NOTED)	584.2	TR	6 9 13	28	78	SS-12	NP	44	20	14	16	6	NP	NP	NP	11	A-1-b (0)	
GRAY, **WEATHERED DOLOMITE**, WET (FREE WATER NOTED) @27': (FREE WATER NOTED)	581.0		12 14 36	64	94	SS-13	NP	-	-	-	-	-	-	-	-	10	Rock (V)	
			50/5"	-	80	SS-14	NP	-	-	-	-	-	-	-	-	12	Rock (V)	
DOLOMITE, GRAY, SLIGHTLY WEATHERED, STRONG, JOINTED - FRACTURED TO MODERATELY FRACTURED, TIGHT; RQD 50%. @29.8': COMPRESSIVE STRENGTH = 19,250 PSI	579.0		38		86	RC-1											CORE	
DOLOMITE, GRAY, MODERATELY WEATHERED, MODERATELY STRONG, JOINTED - HIGHLY FRACTURED TO FRACTURED, TIGHT; RQD 15%. @32' TO 33': TOOL DROP NOTED BY DRILLERS	576.8		33		89	RC-2											CORE	
DOLOMITE, GRAY, SLIGHTLY WEATHERED, MODERATELY STRONG TO STRONG, CRYSTALLINE, JOINTED - FRACTURED TO MODERATELY FRACTURED, TIGHT; RQD 76%. @33.9': COMPRESSIVE STRENGTH = 10,140 PSI @35.3': COMPRESSIVE STRENGTH = 4,950 PSI	571.0	EOB	78		100	RC-3											CORE	

NOTES: "*" - UNCONFINED STRENGTH DETERMINED BY ASTM D 2166, "NP" - NON PLASTIC, "NI" - NOT INTACT

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

LEGEND KEY

Ohio Department of Transportation Soil Symbols



<u>Notes:</u>

1. Exploratory borings were performed on April 3 and 4, 2019, using 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. The borings were located in the field by TTL Associates, Inc. Stations, offsets, ground surface elevations, and coordinates at the boring locations were provided by TRC.

4. HP (tsf):
   “*” = Unconfined Compressive Strength Test per ASTM D 2166
   NP = Non-Plastic.
   NI = Not Intact.

PROJECT: LUC-120-11.32 Bridge Replacement, Toledo, Ohio

TTL Associates, Inc.

PROJECT NO: 1771201

TABULATION OF TEST DATA

Boring Number	Sample Number	Sample Interval Depth (Feet)	Measured Standard Penetration , N_m (Blows per Foot)	Corrected Standard Penetration, N_{60} (Blows per Foot)	Natural Moisture Content (% of Dry Weight)	In-Place Dry Density (Pounds per Cubic Foot)	Unconfined Compressive Strength (Pounds per Square Foot)		Particle Size Distribution (%)					Atterberg Limits (%)			ODOT Soil Classification
									Gravel	Coarse Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index	
B-001-0-18	SS-1	1.0-2.5	11	14													
		1.0-2.2			5.8												
		2.2-2.5			9.0												
	SS-2	2.5-4.0	32	41	14.4		7,000		21	16	20	35	8	24	16	8	A-4a (2)
	SS-3	4.0-5.5	6	8	16.5	111.4	*1,265		13	12	12	22	41	76	16	11	A-6a (6)
	SS-4	5.5-7.0	10	13	20.4		3,000										
	ST-5	8.0-10.0			18.0				1	7	20	22	50	28	16	12	A-6a (8)
	SS-6	11.0-12.5	13	17	16.3		9,000+										
	SS-7	13.5-15.0	20	26	16.1	104.6			2	8	15	26	49	30	19	11	A-6a (8)
	SS-8	16.0-17.5	32	41	13.2		9,000+										
	SS-9	18.5-20.0	21	27	10.9	123.7	*8,990										
	SS-10	21.0-22.5	65	84	9.0		9,000+										
	SS-11	22.5-24.0	34	44	9.1	117.4	9,000+		5	14	21	41	19	NON-PLASTIC			A-4a (5)
	SS-12	24.0-25.5	73	94	9.1	117.5	*5,235		7	11	17	43	22	19	14	5	A-4a (6)
	SS-13	25.5-26.8	SSR	-	11.5		7,500		10	8	14	44	24	20	15	5	A-4a (7)
	SS-14	27.0-27.2	SSR	-	5.5												
	RC-1	27.3-32.3	60” RUN WITH 55% RECOVERY, 28% RQD, UCS = 11,650 PSI @ 27.3 FT														
	RC-2	32.3-37.3	60” RUN WITH 100% RECOVERY, 78% RQD, UCS = 3,350 PSI @ 32.7 FT, UCS = 5,380 PSI @ 35.0 FT														

SSR = Split-Spoon Refusal

Unconfined compressive strength (UCS) generally derived from a calibrated hand penetrometer. UCS denoted with “*” determined by ASTM D 2166.

PROJECT: LUC-120-11.32 Bridge Replacement, Toledo, Ohio	**TTL Associates, Inc.**	PROJECT NO: 1771201

TABULATION OF TEST DATA

Boring Number	Sample Number	Sample Interval Depth (Feet)	Measured Standard Penetration , N_m (Blows per Foot)	Corrected Standard Penetration, N_{60} (Blows per Foot)	Natural Moisture Content (% of Dry Weight)	In-Place Dry Density (Pounds per Cubic Foot)	Unconfined Compressive Strength (Pounds per Square Foot)		Particle Size Distribution (%)					Atterberg Limits (%)			ODOT Soil Classification
									Gravel	Coarse Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index	
B-002-0-18	SS-1	1.0-2.5	11	14													
		1.0-1.7			8.5												
		1.7-2.5			11.0				2	21	39	62	6	NON-PLASTIC			A-4a (1)
	SS-2	2.5-4.0	21	27	16.9				20	6	50	23	1	20	15	5	A-3a (0)
	SS-3	4.0-5.5	24	31	8.6												
	SS-4	5.5-7.0	11	14	11.9												
	SS-5	8.5-10.0	5	6	22.2		1,500										
	SS-6	11.0-12.5	7	9	15.9												
	SS-7	13.5-15.0	7	9	14.9	107.0			22	8	23	35	12	17	16	1	A-4a (2)
	SS-8	16.0-17.5	9	12	18.2	108.4	*985		1	5	25	43	26	23	15	8	A-4a (7)
	ST-9	18.0-20.0			20.1	99.9			1	6	76	16	1	NON-PLASTIC			A-3a (0)
	SS-10	21.0-22.5	10	13	19.6		1,500		0	2	48	49	1	19	17	2	A-4a (3)
	SS-11	22.5-24.0	12	15	22.8				1	8	66	24	1	NON-PLASTIC			A-3a (0)
	SS-12	24.0-25.5	22	28	10.9				44	20	14	16	6	NON-PLASTIC			A-1-b (0)
	SS-13	25.5-27.0	50	64	10.3												
	SS-14	27.0-27.4	SSR	-	12.3												
	RC-1	29.8-33.3	42” RUN WITH 86% RECOVERY, 38% RQD, UCS = 19,250 PSI @ 29.7 FT														
	RC-2	33.3-34.8	18” RUN WITH 89% RECOVERY, 33% RQD, UCS = 10,140 PSI @ 33.9 FT														
	RC-3	34.8-39.8	60” RUN WITH 100% RECOVERY, 78% RQD, UCS = 4,950 PSI @ 35.3 FT														

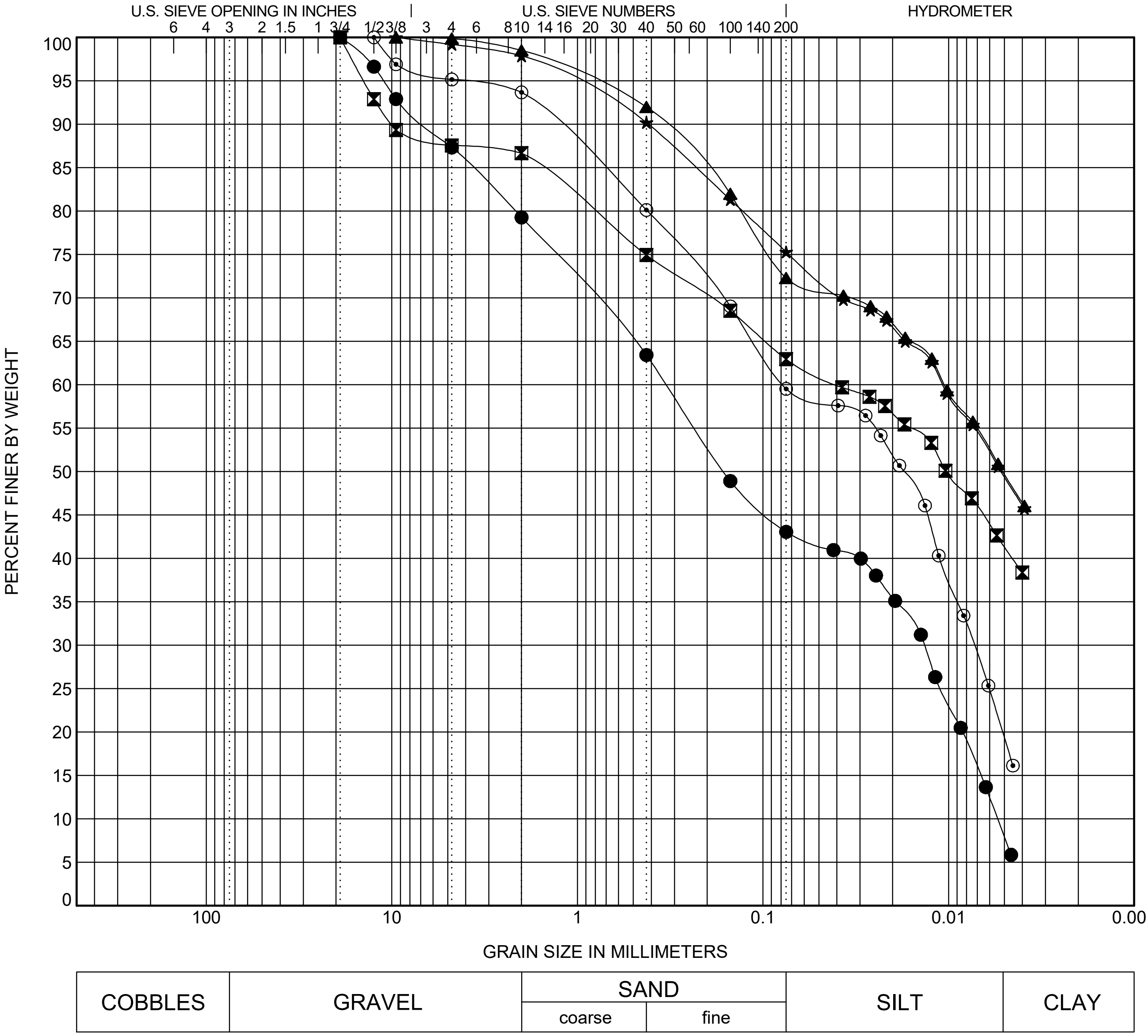
SSR = Split-Spoon Refusal

Unconfined compressive strength (UCS) generally derived from a calibrated hand penetrometer. UCS denoted with “*” determined by ASTM D 2166.

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT LUC-120-11.32
PID 102940
OGE NUMBER N/A
PROJECT TYPE STRUCTURE FOUNDATION



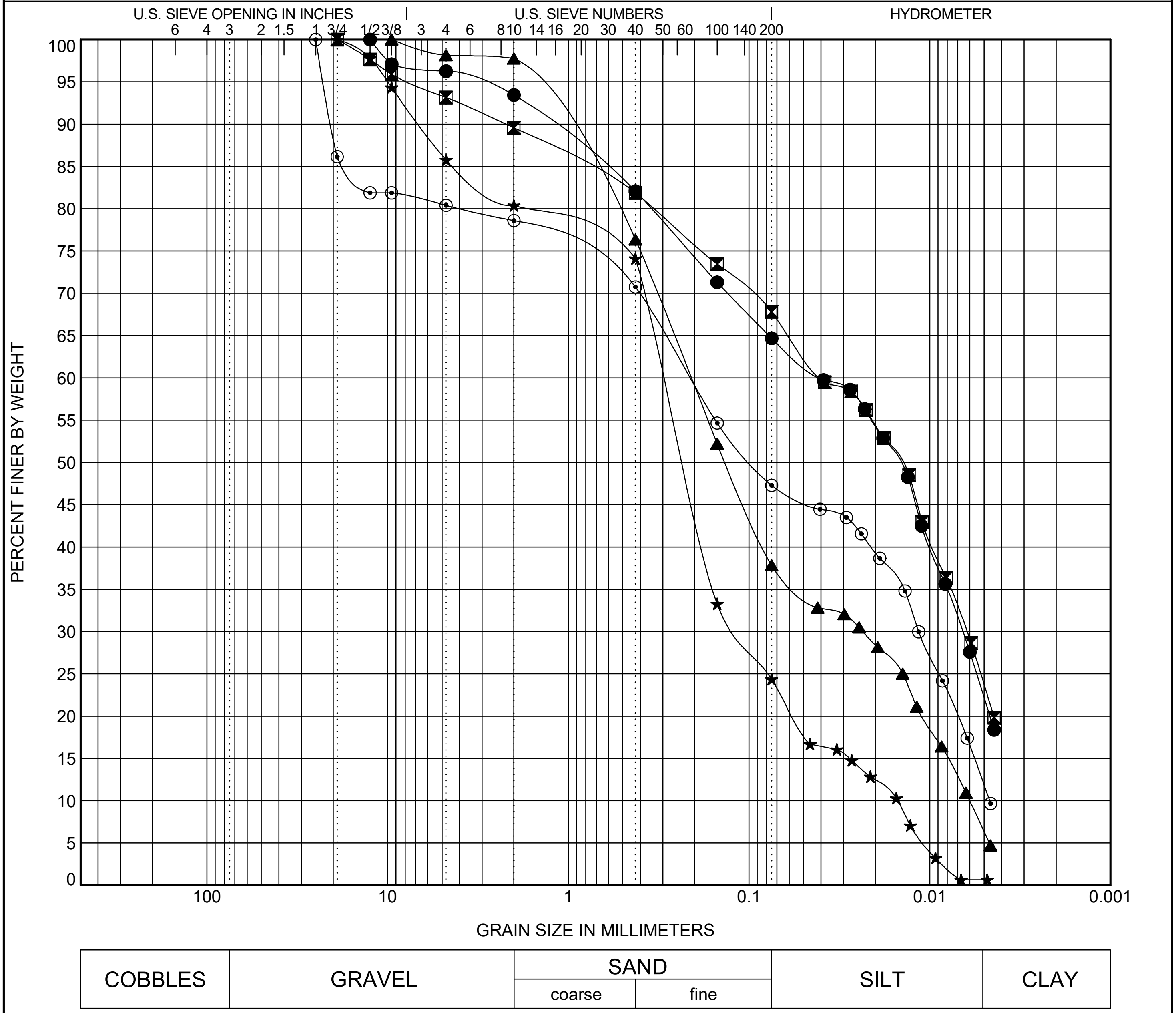
Specimen Identification			ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
●	**B-001-0-18**	**2.5**	**A-4a ~ CLAYEY SAND(SC)**	**24**	**16**	**8**
⊠	**B-001-0-18**	**4.0**	**A-6a ~ SANDY LEAN CLAY(CL)**	**27**	**16**	**11**
▲	**B-001-0-18**	**8.0**	**A-6a ~ LEAN CLAY with SAND(CL)**	**28**	**16**	**12**
★	**B-001-0-18**	**13.5**	**A-6a ~ LEAN CLAY with SAND(CL)**	**30**	**19**	**11**
⊙	**B-001-0-18**	**22.5**	**A-4a ~ SANDY SILT(ML)**	**NP**	**NP**	**NP**

Specimen Identification			D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
●	**B-001-0-18**	**2.5**	**6.622**	**0.162**	**0.013**	**0.005**	**21**	**16**	**20**	**35**	**8**	**0.10**	**61.16**
⊠	**B-001-0-18**	**4.0**	**10.006**	**0.01**			**13**	**12**	**12**	**22**	**41**		
▲	**B-001-0-18**	**8.0**	**0.346**	**0.005**			**1**	**7**	**20**	**22**	**50**		
★	**B-001-0-18**	**13.5**	**0.415**	**0.005**			**2**	**8**	**15**	**26**	**49**		
⊙	**B-001-0-18**	**22.5**	**1.315**	**0.018**	**0.007**		**5**	**14**	**21**	**41**	**19**		

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT LUC-120-11.32 **PID** 102940
OGE NUMBER N/A **PROJECT TYPE** STRUCTURE FOUNDATION



Specimen Identification			ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
●	**B-001-0-18**	**24.0**	**A-4a ~ SANDY SILTY CLAY(CL-ML)**	**19**	**14**	**5**
⊠	**B-001-0-18**	**25.5**	**A-4a ~ SANDY SILTY CLAY(CL-ML)**	**20**	**15**	**5**
▲	**B-002-0-18**	**1.5**	**A-4a ~ SILTY SAND(SM)**	**NP**	**NP**	**NP**
★	**B-002-0-18**	**2.5**	**A-3a ~ SILTY, CLAYEY SAND(SC-SM)**	**20**	**15**	**5**
⊙	**B-002-0-18**	**13.5**	**A-4a ~ SILTY SAND with GRAVEL(SM)**	**17**	**16**	**1**

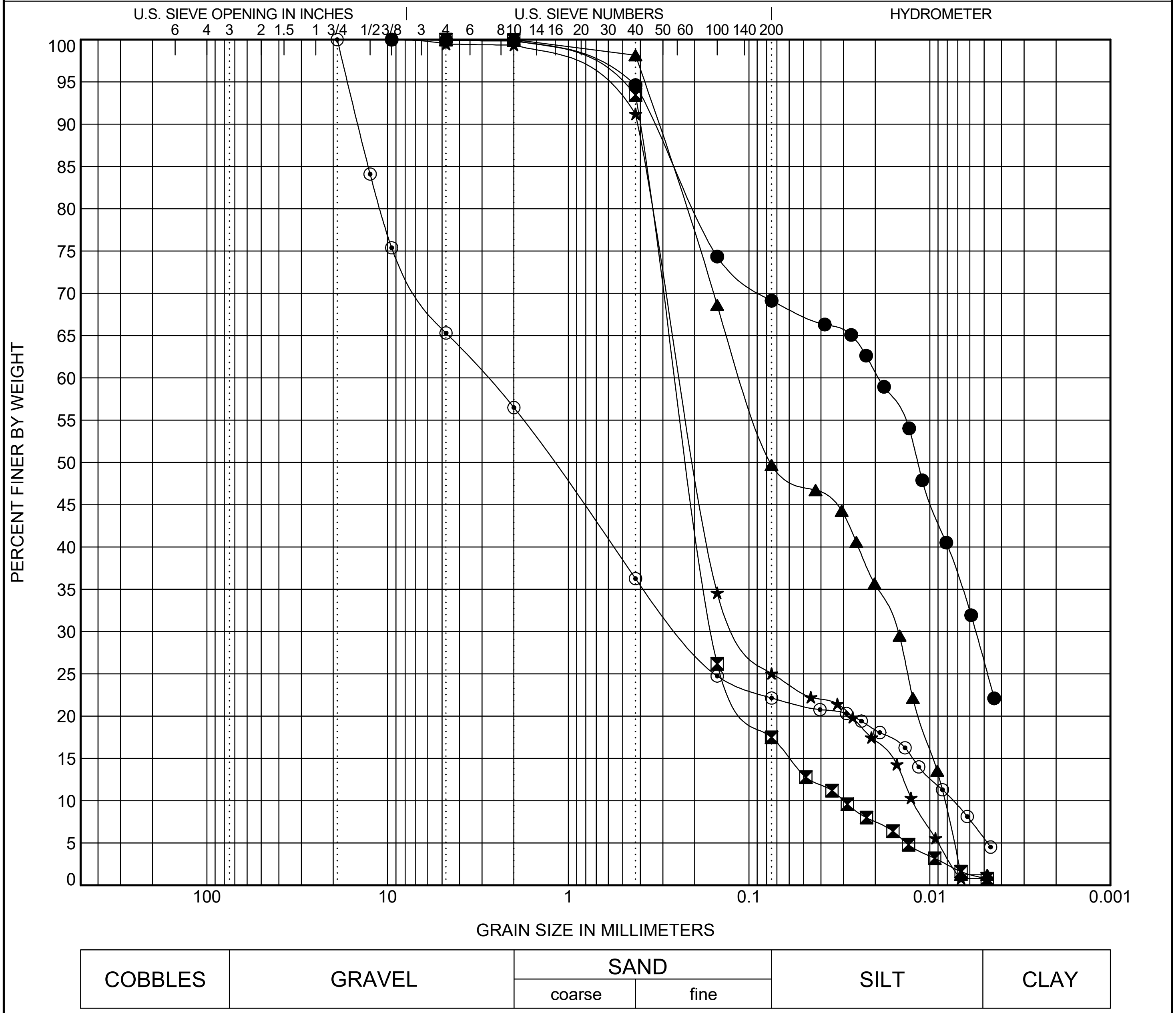
Specimen Identification			D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
●	**B-001-0-18**	**24.0**	**1.252**	**0.015**	**0.007**		**7**	**11**	**17**	**43**	**22**		
⊠	**B-001-0-18**	**25.5**	**2.213**	**0.015**	**0.006**		**10**	**8**	**14**	**44**	**24**		
▲	**B-002-0-18**	**1.5**	**1.141**	**0.135**	**0.023**	**0.006**	**2**	**21**	**39**	**32**	**6**	**0.44**	**34.94**
★	**B-002-0-18**	**2.5**	**6.686**	**0.23**	**0.116**	**0.015**	**20**	**6**	**50**	**23**	**1**	**3.03**	**19.69**
⊙	**B-002-0-18**	**13.5**	**20.503**	**0.097**	**0.012**	**0.005**	**22**	**8**	**23**	**35**	**12**	**0.13**	**45.50**

GRAIN SIZE - OH DOT.GDT - 8/21/19 10:26 - S:\PROJECTS\1771201.GPJ

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT LUC-120-11.32
PID 102940
OGE NUMBER N/A
PROJECT TYPE STRUCTURE FOUNDATION



	Specimen Identification		ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
●	**B-002-0-18**	**16.0**	**A-4a ~ SANDY LEAN CLAY(CL)**	**23**	**15**	**8**
⧗	**B-002-0-18**	**18.0**	**A-3a ~ SILTY SAND(SM)**	**NP**	**NP**	**NP**
▲	**B-002-0-18**	**21.0**	**A-4a ~ SILTY SAND(SM)**	**19**	**17**	**2**
★	**B-002-0-18**	**22.5**	**A-3a ~ SILTY SAND(SM)**	**NP**	**NP**	**NP**
⊙	**B-002-0-18**	**24.0**	**A-1-b ~ SILTY SAND with GRAVEL(SM)**	**NP**	**NP**	**NP**

	Specimen Identification		D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
●	**B-002-0-18**	**16.0**	**0.335**	**0.012**	**0.006**		**1**	**5**	**25**	**43**	**26**		
⧗	**B-002-0-18**	**18.0**	**0.403**	**0.217**	**0.159**	**0.03**	**1**	**6**	**76**	**16**	**1**	**3.33**	**8.42**
▲	**B-002-0-18**	**21.0**	**0.319**	**0.076**	**0.015**	**0.008**	**0**	**2**	**48**	**49**	**1**	**0.25**	**13.13**
★	**B-002-0-18**	**22.5**	**0.416**	**0.199**	**0.108**	**0.012**	**1**	**8**	**66**	**24**	**1**	**3.89**	**19.26**
⊙	**B-002-0-18**	**24.0**	**14.601**	**1.217**	**0.241**	**0.007**	**44**	**20**	**14**	**16**	**6**	**2.76**	**377.70**

Appendix A:
Engineering Calculations
(Including GB-1 Spreadsheet)

Date 6/15/20
Page 1 of 1

Project Name LUC-120-11.32
Project No. 1771201
By CPI
Checked by/Date 6/22/2020
Subject Drilled Shafts Socketed into Bedrock – LOADS

Piers: Max TFL = 1101 kips (From TRC 5/12/20)
(Total Factored Load)

Abutments: Max TFL = 687 kips (From TRC 5/27/20)

TTL associates inc
Environmental Geotechnical Engineering & Testing

Rev. CPI
6/16/20

Date 6/23/19
Page 1 of 5

Project Name LUC-120-11.32
Project No. 1771201
By CPI
Checked by/Date KGH 6/22/2020
Subject Drilled Shafts Socketed into Bedrock

B-001

Top of Rock 26.9' (584.1)
Auger Refusal 27.3' (583.7)
RQD = 100% + UCS = 11,650 psi to only 27.9' (0.6' zone)
then RQD = 0% to 31.4' (3.5' zone)
then RQD = 70% + UCS = 3,350 psi to 35.0' (3.6' zone)
then RQD = 100% + UCS = 5,380 psi to TUM @ 37.3'

31.4'-26.9' = 4.5' below TR

ODOT prescribes no use of resistance in upper 2'. Then it is 0% RQD. Therefore,
base design on rock @ 31.4-35' w/ RQD = 70% + UCS = 3350 psi
35' + deeper w/ RQD = 100% + UCS = 5380 psi

B-002

Top of Rock 26.5' (584.2)
Auger Refusal 29.7' (581.0)
RQD = 50% + UCS = 19250 psi to 31.0'
TOOLING DROP 32' to 33'
33'-33.9' still in RQD = 15% zone
33.9' to TUM @ 39.7' RQD = 76%, UCS = 4,950 to 10,170 psi

within 2' noncontributory zone.

Consider no contribution

Use for design - RQD = 76%
UCS = 4950 psi
@ 33.9' + deeper

End Bearing Resistance
Table 10.4.6.4-1 RMR = 4 + 17 + 10 + 12 + 4 - 0 = 47
Table 10.4.6.4-4 w/ RMR = 47 ~ 44 Carbonate Rock (A), m = 0.128, s = 9 x 10^-5
10.8.3.5.4c Tip Resistance

$q_p = 2.5 q_u$ for intact, tightly jointed rock w/ socket > 1.5B
B-001 q_u = 775 ksf (35'+) → q_p = 775(2.5) = 1937 ksf
B-002 q_u = 713 ksf (34'+) → q_p = 713(2.5) = 1783 ksf
} Too High

$q_p = \left[\sqrt{s} + \sqrt{m\sqrt{s} + s}\right] q_u$ for jointed rock w/in 2B

B-001 $q_p = \left[\sqrt{9\times10^{-5}} + \sqrt{0.128\sqrt{9\times10^{-5}} + \cancel{0.128}\ 9\times10^{-5}}\right](775\text{ ksf})$ = ~~286 ksf~~ 35 ksf
B-002: 35 ~~286~~ × 713/775 = ~~263 ksf~~ 32 ksf

$\phi_{Tip} = 0.50$ Factored q_p = ~~143 ksf~~ + ~~131 ksf~~ 17.5 ksf + 16 ksf, respectively, * See below (~~Too aggressive~~ Lower than Presumptive Sound dolomite rock presumptive typically 40 ksf)

*C10.8.3.5.4d: Axial Capacity of Shaft Socketed in rock solely carried w/ side friction until get shaft movement of 0.4". Expect ≈ 0.4" or less settlement so use skin friction only

Date 6/23/19
Page 2 of 5

Project Name LUC-120-11.32 Central over Ottawa River
Project No. 1771201
By CPZ
Checked by/Date KCH 6/22/2020
Subject Drilled shafts socketed in to Bedrock

LRFD Bridge Design Specs: 10.8.3.5.4b – Side Resistance for Rock Sockets

$$q_s = 0.65\,\alpha_E\,p_a\left(\frac{q_u}{p_a}\right)^{0.5} < 7.8\,p_a\left(\frac{f'_c}{p_a}\right)^{0.5}$$

, f'_c = 28 day Strength (ksi) = 4 ksi

q_u = UCS

B-001: 31.4-35'; UCS = 3350 psi $\times \frac{\text{kip}}{1000\text{ lb}} \times \frac{144\text{ in}^2}{\text{ft}^2}$ = 482 ksf
RQD = 70%

B-001: 35'+ UCS = 5380 psi $\times \frac{\text{kip}}{1000} \times \frac{144\text{ in}^2}{\text{ft}^2}$ = 775 ksf
RQD = 100%

B-002: 33.9'+ UCS = 4950 psi $\times \frac{\text{kip}}{1000} \times \frac{144\text{ in}^2}{\text{ft}^2}$ = 713 ksf
(~34'+)
RQD = 76%

p_a (ksf) = atmospheric Pressure = 2.12 ksf

α_E: reduction factor for jointing in rock.

RQD = 70% to 100% → Table 10.4.6.5-1 E_m/E_i = 0.70 to 1.00, respectively (closed joints)

B-001, 31.4' to 35' RQD = 0.70 = E_m/E_i → $\alpha_E = 0.8 + (1-0.8)\left(\frac{0.7-0.5}{1-0.5}\right) = 0.88$
70% →

B-001, 35'+ : RQD = 100% ⇒ E_m/E_i = 1.00 → α_E = 1.0

B-002, 33.9'+ : RQD = 76% → E_m/E_i : 0.76 → $\alpha_E = 0.8 + (1-0.8)\left(\frac{0.76-0.5}{1.0-0.5}\right) = 0.90$

B-001, 31.4'-35': $q_s = 0.65(0.88)(2.12\text{ ksf})\left(\frac{482\text{ ksf}}{2.12\text{ ksf}}\right)^{0.5} = 18.3 < 22.7$
$< 7.8(2.12\text{ ksf})(4\text{ ksf}/2.12\text{ ksf})^{0.5} = 22.7$ ~18 ksf

B-001, 35'+ : $q_s = 0.65(1.0)(2.12\text{ ksf})(775\text{ ksf}/2.12\text{ ksf})^{0.5} = 26.3 < 22.7$
$< 7.8(2.12)(4/2.12)^{0.5} = 22.7$ ~22 ksf

B-002, 33.9'+, $q_s = 0.65(0.90)(2.12\text{ ksf})(713/2.12)^{0.5} = 22.7 < 22.7$
$< 7.8(2.12)(4/2.12)^{0.5} = 22.7$ ~22 ksf

ϕ_{side} = 0.55 Factored q_s = 10 ksf, 12 ksf, + 12 ksf, respectively

TTL associates inc
Environmental, Geotechnical Engineering & Testing

Rev. CPI 6/16/20

Date 6/23/18
Page 3 of 5

Project Name LUC-120-11.32
Project No. 1771201
By CPI
Checked by/Date KCH 6/22/2020
Subject Drilled Shfts socketed into Bedrock

Piers

~~Alt 2 Max P~~ Factored ~~= 861k~~ TFL = 1101k

B-001: ~~31½-35' = 3½' @ 10ksf~~

See Spreadsheet Pier 1 B-001
3' diameter socket to 42' (Elev. 569)
(Socket = 15')
4.7' below cored depth)
Termination elevation

~~Min Shft Dia for piers~~ supporting columns ~~= 42" = 3.5' → socket = 3'~~

~~$\pi(3')(3.5') \times 10 ksf = 330k \rightarrow 861 - 330 = 531k$ left~~

~~$\frac{531k}{12ksf} = 44.25 ft^2 \div \pi(3') = 4.7' = L$ in 12ksf material~~

~~Go to 35' + 4.7' = 39.7' (Elev. 571.3)~~

~~(~2½' below cored rock)~~

~~Top of Rock = 26.9' → L = 12.8'~~
~~AR @ 27.3' → L~~ ~~below AR~~ ~~= 12.4'~~

Although don't expect end-bearing to kick in w/ 0.4" or less settlement/shaft movement, look at End-Bear @ 35': (lowermost cored rock) (assuming same at 42') $\frac{\pi(3')^2}{4} \times$ ~~143~~ 17.5 ksf = ~~1010~~ 123 kips ~~for 861k needed~~, if were to mobilize end-bearing. ~~However, 143 ksf is too aggressive to count on safety.~~

Look @ end-bearing @ 31½' as on pg 1 of calcs.

RMR: 2 + 13 + 5 (to 10) + 12 + 4 - 0 = 36 (to 41)

Say RMR ~23 conservatively → m = 0.029, s = 3×10⁻⁶

$q_p = \left[\sqrt{3\times10^{-6}} + \sqrt{0.029\sqrt{3\times10^{-6}} + 3\times10^{-6}}\right](q_u = 482 ksf) = 0.009(482 ksf) = 4 ksf$

too Low
only 0.9% of UCS.

For RMR = 44 parameters m = 0.128 + s = 9×10⁻⁵,

$q_p = \left[\sqrt{9\times10^{-5}} + \sqrt{0.128\sqrt{9\times10^{-5}} + 9\times10^{-5}}\right](482 ksf) = 0.0456 \times 482 = 22 ksf$

$\phi = 0.50$, q_p factored = 11ksf

Presumptive rock end-bearing typically 20ksf (RQD<25)

$\frac{\pi(3^2)}{4}(11 ksf) = 78$ ~~ksf~~ kips CPI 8/21/19 → still need to extend to at least 35' for higher end-bearing or larger shaft/socket.

NOT APPLICABLE

B-002 34'±: $\frac{861k}{12ksf} = 71.75 ft^2 \div \pi(3') = 7.6' \rightarrow 41½' (Elev. 569.2)$ (~2' below cored rock)

Top of Rock = 26.5' L = 15'
AR @ 29.7' L below AR = 11.8'

For 4' Dia, 71.75 ÷ π(4) = 5.7'
For 5' Dia, 71.75 ÷ π(5) = 4.6'
} only saves 2 to 3' depth

See Next Page

TTL associates Inc
Environmental, Geotechnical Engineering & Testing

Date 6/16/20
Page 4 of 5

Project Name LUC-120-11.32
Project No. 1771201
By CPI
Checked by/Date KCH 6/22/2020
Subject Drilled Shafts Socketed into Rock

Piers Continued

Pier 2 (B-002) : (See Spreadsheet) 3' diameter socket to 44' (Elev. 566.7)
(socket: 17.5') 4½' below cored depth/ Term. Elev.

Although don't expect end-bearing to kick in w/0.4" or less settlement/socket movement, Look at end-bearing possible contribution: $\frac{\pi(3')^2}{4}(16\,ksf) = 113\,kips$, if were to mobilize end-bearing.

(See Next Page for Abutments)

TTL associates inc
Environmental, Geotechnical Engineering & Testing

Rev. CPI 6/16/20

Date 6/24/19
Page 5 of 5

Project Name LUC-120-11.32
Project No. 1771201
By CPI
Checked by/Date KCH 6/22/2020
Subject Drilled Shafts Socketed into Rock

Abutments

Max Total Factored Load ~~(Alt 1)~~ ≈ ~~955k~~ 687K

Min dia for Abutment Shaft = 36" & socket = 30" (2.5').

B-001 : ~~31½ to 35' : $\frac{\pi(2.5')^2}{4}$ (3.5')(10 ksf) = 171 kips~~

~~Still need 955 − 171 = 784k~~

~~@35' + q_p factored = 12 ksf → $\frac{784k}{12\ ksf}$ = 65.3 ft² ÷ π(2.5') = 8.3'~~

~~Go to 35 + 8.3 = 43.3' (Elev. 567.7) (~6' below cored depth of rock)~~

~~Top of Rock = 26.9' → L = 16.4'~~

See Spreadsheet B-001 Rear Abutment
2.5' diameter rock socket to 39½' (Elev. 571.5)
Socket = 12.5' (~2' below bottom of core/termination Elevation)

If need end ~~Be~~ Bearing & move > 0.4", would get $\frac{\pi(2.5)^2}{4}$ (17.5 ksf) = 86 kips (Do not include)

B-002 : ~~34' ; q_p factored = 12 ksf ; $\frac{955k}{12\ ksf}$ = 79.6 ft² ÷ π(2.5) = 10.1'~~

~~Depth = 34 + 10.1 = 44.1' (Elev. 566.6) (~4½' below cored depth of rock)~~

~~Top of rock = 26.5' ; L = 17.6'~~

~~For 3½' dia shaft w/ 3' socket, L mobilized = 8.5' (10.1 − 8.5 ⇒ Save only 1.6' embedment)~~

See Spreadsheet B-002 Fwd Abutment

2.5' diameter rock socket to 41½' (Elev. 569.2)

Socket = 15' (~2' below bottom of core / termination depth)

If get more than 0.4" settlement, end-bearing component would be

$\frac{\pi(2.5)^2}{4}$ (16 ksf) = 78 kips (Do not include in design since δ likely ≤ 0.4")

TTL Associates, Inc.						
Drilled Shaft Socket Length Evaluations						
Project Number	**1771201**					
Project Description	**LUC-120-11.32, PID 102940**					
Evaluated By/Date	CPI/6-15-2020					
Location:	**Pier 1**					
Boring:	**B-001**					
Total Factored Load (kips)	**1101**					
Socket Diameter (feet)	**3**		**3.5**		**4**	
Contributory Side Resistance						
Rock Layer	1		1		1	
Top Depth (ft)	27		27		27	
Bottom Depth (ft)	31.5		31.5		31.5	
Length of Layer (ft)	4.5		4.5		4.5	
Factored Unit Side Resistance (ksf)	0		0		0	
Layer Side Resistance (kips)	0		0		0	
Rock Layer	2		2		2	
Top Depth (ft)	31.5		31.5		31.5	
Bottom Depth (ft)	35		35		35	
Length of Layer (ft)	3.5		3.5		3.5	
Factored Unit Side Resistance (ksf)	10		10		10	
Layer Side Resistance (kips)	329.9		384.8		439.8	
Rock Layer	3		3		3	
Top Depth (ft)	35		35		35	
Bottom Depth (ft)	37.5		37.5		37.5	
Length of Layer (ft)	2.5		2.5		2.5	
Factored Unit Side Resistance (ksf)	12		12		12	
Layer Side Resistance (kips)	282.7		329.9		377.0	
Rock Layer	4		4		4	
Top Depth (ft)	37.5		37.5		37.5	
Bottom Depth (ft)	42		40.5		39.5	
Length of Layer (ft)	4.5		3		2	
Factored Unit Side Resistance (ksf)	12		12		12	
Layer Side Resistance (kips)	508.9		395.8		301.6	
Total Side Resistance (kips)	1121.5		1110.6		1118.4	
Socket Length (ft)*	**15**		**13.5**	**	**12.5**	**
Volume of Socket (cu ft)	**106.0**		**129.9**	**	**157.1**	**

***Socket length evaluated at half foot increments until total side resistance meets or exceeds Total Factored Load.**

**Although shorter embedment, volume is much greater than 3' diameter. Smaller diameter, deeper socket
may be more economical. However, bigger diameter may be needed for lateral loading considerations.

TTL Associates, Inc.						
Drilled Shaft Socket Length Evaluations						
Project Number	**1771201**					
Project Description	**LUC-120-11.32, PID 102940**					
Evaluated By/Date	CPI/6-15-2020					
Location:	**Pier 2**					
Boring:	**B-002**					
Total Factored Load (kips)	**1101**					
Socket Diameter (feet)	**3**		**3.5**		**4**	
Contributory Side Resistance						
Rock Layer	1		1		1	
Top Depth (ft)	26.5		26.5		26.5	
Bottom Depth (ft)	34		34		34	
Length of Layer (ft)	7.5		7.5		7.5	
Factored Unit Side Resistance (ksf)	0		0		0	
Layer Side Resistance (kips)	0		0		0	
Rock Layer	2		2		2	
Top Depth (ft)	34		34		34	
Bottom Depth (ft)	39.5		39.5		39.5	
Length of Layer (ft)	5.5		5.5		5.5	
Factored Unit Side Resistance (ksf)	12		12		12	
Layer Side Resistance (kips)	622.0		725.7		829.4	
Rock Layer	3		3		3	
Top Depth (ft)	39.5		39.5		39.5	
Bottom Depth (ft)	44		42.5		41.5	
Length of Layer (ft)	4.5		3		2	
Factored Unit Side Resistance (ksf)	12		12		12	
Layer Side Resistance (kips)	508.9		395.8		301.6	
Total Side Resistance (kips)	1131.0		1121.5		1131.0	
Socket Length (ft)*	**17.5**		**16**	**	**15**	**
Volume of Socket (cu ft)	**123.7**		**153.9**	**	**188.5**	**
***Socket length evaluated at half foot increments until total side resistance meets or exceeds Total Factored Load.**						
**Although shorter embedment, volume is much greater than 3' diameter. Smaller diameter, deeper socket						
may be more economical. However, bigger diameter may be needed for lateral loading considerations.						

TTL Associates, Inc.						
Drilled Shaft Socket Length Evaluations						
Project Number	**1771201**					
Project Description	**LUC-120-11.32, PID 102940**					
Evaluated By/Date	CPI/6-15-2020					
Location:	**Rear Abutment**					
Boring:	**B-001**					
Total Factored Load (kips)	**687**					
Socket Diameter (feet)	**2.5**		**3**		**3.5**	
Contributory Side Resistance						
Rock Layer	1		1		1	
Top Depth (ft)	27		27		27	
Bottom Depth (ft)	31.5		31.5		31.5	
Length of Layer (ft)	4.5		4.5		4.5	
Factored Unit Side Resistance (ksf)	0		0		0	
Layer Side Resistance (kips)	0		0		0	
Rock Layer	2		2		2	
Top Depth (ft)	31.5		31.5		31.5	
Bottom Depth (ft)	35		35		35	
Length of Layer (ft)	3.5		3.5		3.5	
Factored Unit Side Resistance (ksf)	10		10		10	
Layer Side Resistance (kips)	274.9		329.9		384.8	
Rock Layer	3		3		3	
Top Depth (ft)	35		35		35	
Bottom Depth (ft)	37.5		37.5		37.5	
Length of Layer (ft)	2.5		2.5		2.5	
Factored Unit Side Resistance (ksf)	12		12		12	
Layer Side Resistance (kips)	235.6		282.7		329.9	
Rock Layer	4		4			
Top Depth (ft)	37.5		37.5			
Bottom Depth (ft)	39.5		38.5			
Length of Layer (ft)	2		1			
Factored Unit Side Resistance (ksf)	12		12			
Layer Side Resistance (kips)	188.5		113.1			
Total Side Resistance (kips)	699.0		725.7		714.7	
Socket Length (ft)*	**12.5**		**11.5**	**	**10.5**	**
Volume of Socket (cu ft)	**61.4**		**81.3**	**	**101.0**	**

***Socket length evaluated at half foot increments until total side resistance meets or exceeds Total Factored Load.**

**Although shorter embedment, volume is much greater than 3' diameter. Smaller diameter, deeper socket may be more economical. However, bigger diameter may be needed for lateral loading considerations.

TTL Associates, Inc.						
Drilled Shaft Socket Length Evaluations						
Project Number	**1771201**					
Project Description	**LUC-120-11.32, PID 102940**					
Evaluated By/Date	CPI/6-15-2020					
Location:	**Forward Abutment**					
Boring:	**B-002**					
Total Factored Load (kips)	**687**					
Socket Diameter (feet)	**2.5**		**3**		**3.5**	
Contributory Side Resistance						
Rock Layer	1		1		1	
Top Depth (ft)	26.5		26.5		26.5	
Bottom Depth (ft)	34		34		34	
Length of Layer (ft)	7.5		7.5		7.5	
Factored Unit Side Resistance (ksf)	0		0		0	
Layer Side Resistance (kips)	0		0		0	
Rock Layer	2		2		2	
Top Depth (ft)	34		34		34	
Bottom Depth (ft)	39.5		39.5		39.5	
Length of Layer (ft)	5.5		5.5		5.5	
Factored Unit Side Resistance (ksf)	12		12		12	
Layer Side Resistance (kips)	518.4		622.0		725.7	
Rock Layer	3		3			
Top Depth (ft)	39.5		39.5			
Bottom Depth (ft)	41.5		40.5			
Length of Layer (ft)	2		1			
Factored Unit Side Resistance (ksf)	12		12			
Layer Side Resistance (kips)	188.5		113.1			
Total Side Resistance (kips)	706.9		735.1		725.7	
Socket Length (ft)*	**15**		**14**	**	**13**	**
Volume of Socket (cu ft)	**73.6**		**99.0**	**	**125.1**	**
***Socket length evaluated at half foot increments until total side resistance meets or exceeds Total Factored Load.**						
**Although shorter embedment, volume is much greater than 3' diameter. Smaller diameter, deeper socket						
may be more economical. However, bigger diameter may be needed for lateral loading considerations.						

TTL associates inc
Environmental, Geotechnical Engineering & Testing

Date 6/24/19
Page 1 of 3

Project Name LUC-120-11.32
Project No. 1771201
By CPI
Checked by/Date
Subject Soil properties for LPILE

B-001

0-4' Pavement then Very Stiff to Hard Cohesive Fill (Embankment Fill)
$N_{60} = 41$ 41 × 250 = 10,250 UCS → c = 5125 psf
UCS = 3.5 tsf → c = 3500 psf
say c = 3500 psf → $\epsilon_{50} = 0.005$ $\gamma_T = 130$ pcf

4-11' Med stiff to stiff Cohesive

$N_{60} = 8, 13$ 8 × 250 = 2000 psf UCS → c = 1000 psf
UCS = 0.63*, 1.50 tsf — Avg = 1.1 tsf → c = 1100 psf
w/ c = 1000 psf, → ϵ_{50} = 0.010

c = 1000 psf
$\gamma_T = 130$ pcf

11'- 21' Very stiff to Hard Cohesive

N_{60} = 17, 26, ~~41~~, 27 Avg. = 23 × 250 = 5750 UCS → c = 2875 psf
UCS = >4.5, >4.5, 4.5* tsf → c = 4500 psf +
Say c = 3,000 psf → ϵ_{50} = 0.005 γ_T = 135 pcf
* Use γ' At and Below 16' (Elev 595)

21-27' Hard Cohesive Soils
← disturbed top of silt layer from SS + Drilling?
N_{60} = 84, 44, 94, SSR (100) → Avg = 80 × 250 = 20,000 UCS → c = 10,000 psf
UCS = >4.5, >4.5, 2.62*, 3.75 Avg = 3.84 tsf → c = 3840 psf
→ Disturbed silt SS samples?
For H.P. >4.5, say c = 4500 psf | γ_T = 130 pcf ϵ_{50} = 0.005

27½- 31½' Weathered/Fractured Rock (RQD = 0%)
γ_T = 150 pcf Er = 500,000 psi, UCS = 3,000 psi (~ lowest tested reading)
(Tested specimen w/ UCS = 11,650 psi not indicative of RQD = 0% material)
krm = 0.0005

27 to 27½' Augerable Rock → Model as Hard cohesive soil but γ_T = 135 pcf
(Same as layer from 21 to 27')

31½ to 35' Dolomite Bedrock (RQD = 70%)
γ_T = 150 pcf, Er = 500,000 psi, UCS = 3,350 psi, krm = 0.0005

35'+ Dolomite Bedrock (RQD = 100%)
γ_T = 150 pcf, Er = 500,000 psi, UCS = 5380 psi, krm = 0.0005

TTL associates inc
Environmental, Geotechnical Engineering & Testing

Date 6/25/19
Page 2 of 3

Project Name LUC-120-11.32
Project No. 177120 1
By CPI
Checked by/Date
Subject Soil Properties for LPILE

B-002

0-3½ (610½-607)
Pavement then Very Stiff Cohesive Fill (Embankment Fill)
$N_{60}=14$ $14\times250=3500$psf $\rightarrow c=1750$psf
UCS = NT
Say $c=1750$psf, $\varepsilon_{50}=0.007$, $\gamma_T=130$

3½-8' (607-602½)
Med Dense to Dense Granular $\gamma_T=120$pcf
$N_{60}=31, 14$ Use $N=14$
$\sigma_v'{}_{6'}\approx 4(130)+2(120)=760$psf ~ 5psi $\rightarrow \phi\approx36°$ for med dense to dense
Med Dense sand above GWT $\rightarrow k=90$pci
(Fig 10.20) Attached

8-11' (602½-599½)
Med Stiff Cohesive
$N_{60}=6\times250=1500$psf UCS $\rightarrow c=750$psf
UCS = 0.75 tsf $\rightarrow c=750$ psf
$\underline{c=750\text{psf}}\rightarrow\varepsilon_{50}=0.010$ $\gamma_T=130$pcf

11-14' (599½-596½)
Loose Granular $\gamma_T=120$pcf
$\underbrace{N_{60}=9,\ \sigma_v'{}_{12'}}\approx 4(130)+4(120)+3(130)+1(120-62.4)=1448$psf ~ 10psi
$\rightarrow \phi\approx30°$
Loose Sand submerged, $k=20$pci
* Use γ' at and below 11' (Elev. 599½)

14-23' (596½-587½)
Pred. Medium Stiff to Stiff Cohesive
$N_{60}=9, 12, 13$ Avg $=11\times250=2750$psf $\rightarrow c=1375$
UCS: NT + 0.49, 0.75 $\rightarrow c=500$ to 750?
↑ silt SS samples ↗
Say $c=1000$psf, $\varepsilon_{50}=0.010$, $\gamma_T=130$pcf

23 to 26½' (587½-584)
Medium Dense Granular $\gamma_T=120$
$N_{60}=15, 28\rightarrow$ Avg $=21$
$\sigma_v'{}_{24'}=1448+2(120-62.4)+9(130-62.4)+1(120-62.4)=2230$psf ~ 15psi
$N_{60}=21+\sigma_v'=15$psi $\rightarrow\phi\approx36°$ med dense submerged $k=60$pci

26½ to 29½' (584-581)
Augerable Rock $\rightarrow$ Model as Hard Cohesive Soil (Same as B-001 21 to 27)
but $\gamma_T=135$pcf

29½ to 31½' (581-579)
Dolomite Bedrock (RQD = 50%)
$\gamma_T=150$pcf $E_r=500{,}000$psi, UCS $=3{,}000$psi (~ lowest tested reading)
(Tested specimen w/ UCS = 19,250psi too agressive for this material)
$k_{rm}=0.0005$

Environmental, Geotechnical Engineering & Testing

Date 6/25/19

Page 3 of 3

Project Name LVC-120-11.32

Project No. 1771201

By CPI

Checked by/Date

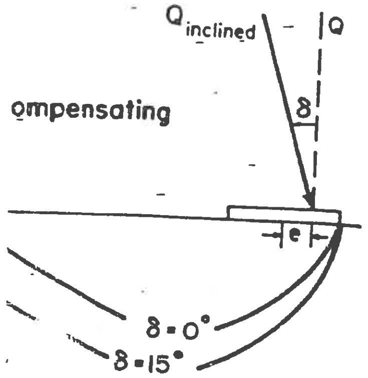
Subject Soil Properties for LPILE

B-002 (continued)

31½' - 34' (579 - 576½)	Dolomite Rock (RQD=15%) w/ 32'-33' Zone Potential Void or Soil in-fill Model as hard cohesive soil (Same as B-001 21 to 27') but $\gamma_T = 135$ pcf
34'+ B-002 (Term @ Elev. 571)	Dolomite Rock (RQD=76%) $\gamma_T = 150$ pcf $E_r = 500{,}000$ psi ; UCS = 4950 psi $k_{rm} = 0.0005$

Bearing in Granular Soils
(Finding ϕ)

ıclined

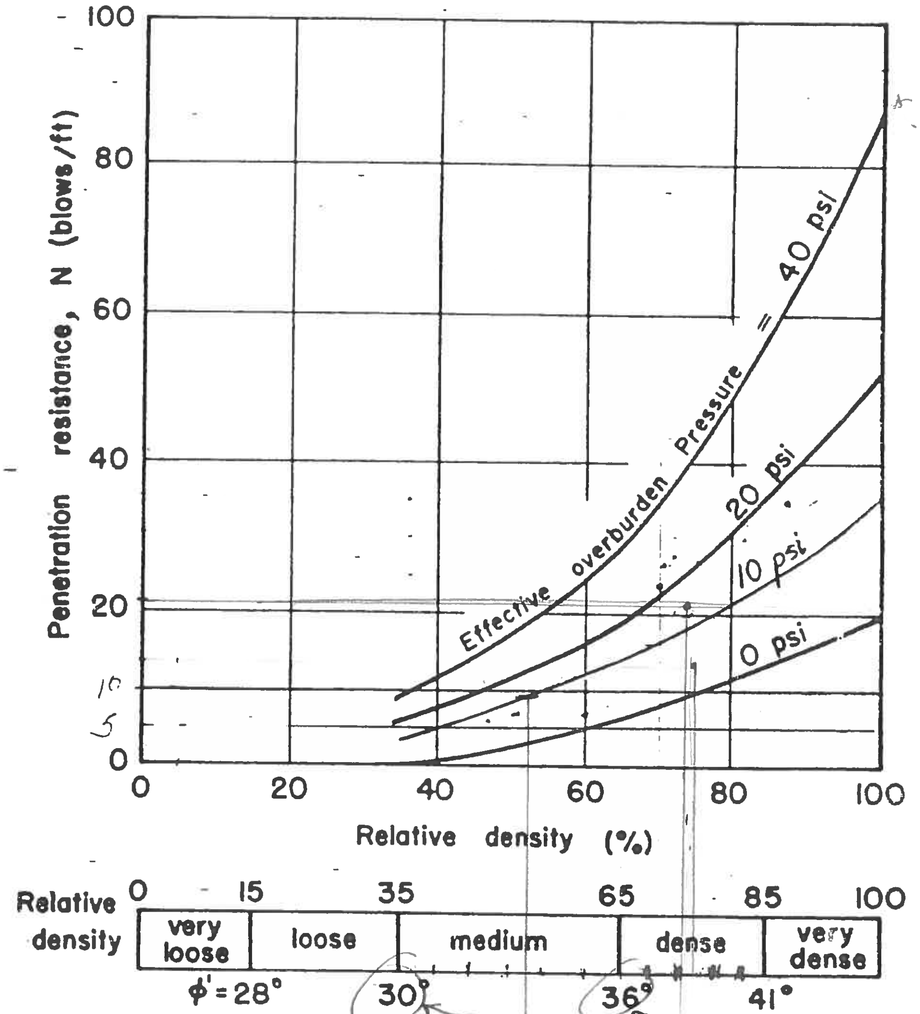


cohesionless materials is in-
o evaluate the *in sītu* relative
the *standard penetration test**
ıetration of the soil has been
y and the confining pressure.
:tween these parameters. This
in size distribution and grain
n the position of the ground-
m our discussion of shearing
tion test occurs very rapidly,
st of the soil. If the granular
rease produces an increase in
h is too high. Conversely the
the groundwater table will
owly applied loadings.

: of most test boring operations. It
bottom of a bore hole using a 140-lb
hammer required to drive the tube a
tion resistance, N.

The relationship between ϕ' and relative density for the particular material in question can then be determined in the laboratory. A very approximate correlation between ϕ' and relative density, which is conservative in many cases, is also given in Figure 10.20. This table indicates a unique relationship between relative density and ϕ' although, for a variety of soils there will be a variation in ϕ' at a given relative density. Hence it is only an approximate guide and must be used with caution.

As we shall see subsequently, except for the unusual case of a narrow footing on loose cohesionless material with a groundwater table close to the



For Design
33°
For Design
38

σ'_v const

Fig. 10.20—Relationship between standard penetration resistance, relative density, and angle of shearing resistance, for sands. (After Gibbs and Holtz, Meyerhof, 1956.)

a. Unit Weight

Values of effective unit weight for each soil depth are entered in standard units of force per unit volume. The program will linearly interpolate values of unit weight located between two specified soil depths, but can also accept step changes whenever the depth values are repeated, such as at the water table. The last entry of Unit Weight should be at the same depth as the bottom of the last soil layer.

b. *k* Value for Soil Layers

This is the value for the constant *k* used in the equation $E_s = kx$. This constant is in units of force per cubic length and depends on the type of soil and lateral loading imposed to the pile group. It has two different uses: (i) to define the initial (maximum) value of E_s on internally-generated *p-y* curves of stiff clays with free water and/or sands; and (ii) to initialize the E_s array for the first iteration of pile analysis.

Suggested values of the parameter *k* used for sands are given in Table 3.2. Suggested values of the parameter *k* used for clays are given in Table 3.3.

c. Undrained Shear Strength

Values of undrained shear strength, c_u, for clays and silts at each depth are entered in standard units of force per unit area. The undrained shear strength is not needed for sand layers. The undrained shear strength is generally taken as half of the unconfined compressive strengths.

Relative Density	**Loose**	**Medium**	**Dense**
Submerged Sand	20 lb/in3 5,430 KPa/m	60 lb/in3 16,300 KPa/m	125 lb/in3 33,900 KPa/m
Sand Above WT	25 lb/in3 6,790 KPa/m	90 lb/in3 24,430 KPa/m	225 lb/i3 61,000 KPa/m

Table 3.2 Soil-Modulus Parameter *k* for Sands

Consistency of Clay	ε_{50}
Soft	0.020
Medium	0.010
Stiff	0.005

Table 3.4 Values of ε_{50} for Clays

Su (psf)	Avgerage Undrained Shear Strength (kPa)	ε_{50}
1044 – 2088	50-100	0.007
2088 – 4177	100-200	0.005
6265 – 8354	300-400	0.004

Table 3.5 Values of ε_{50} for Stiff Clays

f. Elastic Modulus for Weak Rock

The mass modulus for weak rock should be entered for this value. This value may be measured in the field using an appropriate test or may be obtained from the product of the modulus reduction ratio and Young's modulus measured on intact rock specimens in the laboratory

g. Unconfined Compressive Strength for Rock/Weak Rock

This value is the unconfined compressive strength of weak rock at the specified depth. Values at elevations between the top and bottom elevations will be determined by linear interpolation.

Any input values that are considered to be unreasonable are flagged in the output file and a warning dialog box is displayed. However, the analysis is performed normally.

h. Rock Quality Designation for Weak Rock

The secondary structure of the weak rock is described using the Rock

Quality Designation (RQD).

Enter the value of RQD in percent for the weak rock.

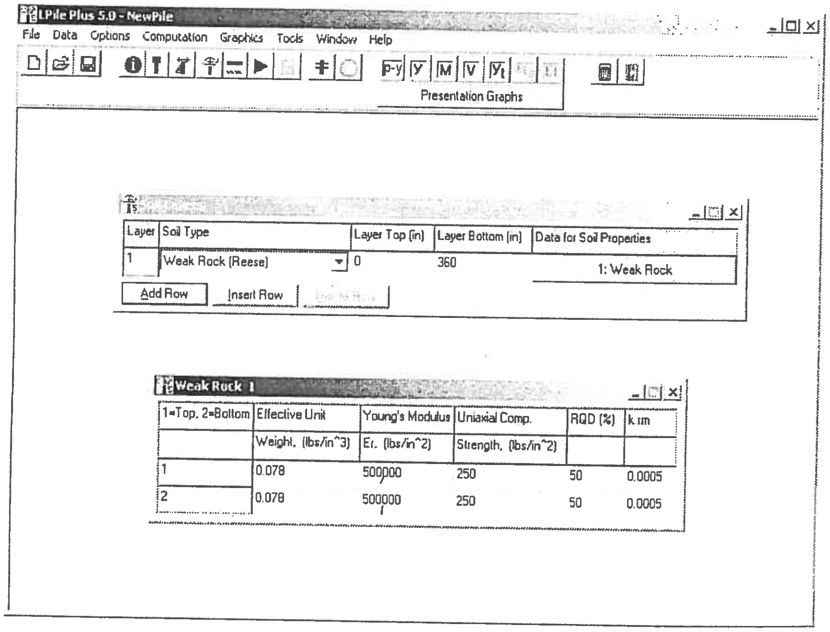
i. Parameter k_{rm} for Weak Rock

The parameter k_{rm} typically ranges between 0.0005 and 0.00005. The input window related to weak rock is shown in Fig. 3.14 for reference.

h. Input p-y Curves

This layer option allows the user to enter specific relationships of soil resistance (p) and lateral movement of the pile (y) at specified depths. These cases usually arise when local data for the soil response are available.

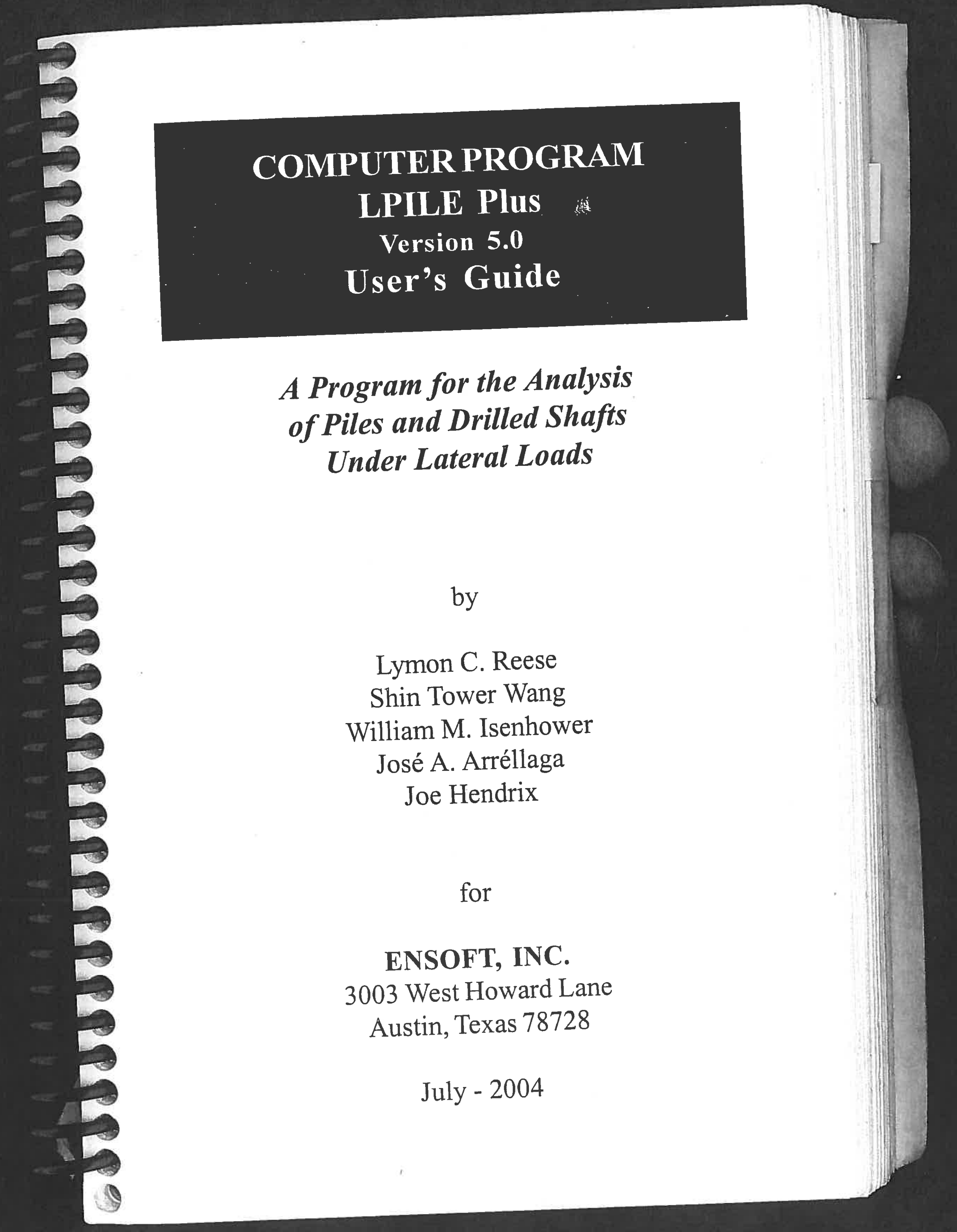
A general description for the data needed under each column for the



Layer	Soil Type	Layer Top (in)	Layer Bottom (in)	Data for Soil Properties
1	Weak Rock (Reese)	0	360	1: Weak Rock

1=Top, 2=Bottom	Effective Unit Weight, (lbs/in^3)	Young's Modulus Er, (lbs/in^2)	Uniaxial Comp. Strength, (lbs/in^2)	RQD (%)	k rm
1	0.078	500000	250	50	0.0005
2	0.078	500000	250	50	0.0005

Fig. 3.14 Window for sample Data - Weak Rock



COMPUTER PROGRAM
LPILE Plus
Version 5.0
User's Guide

A Program for the Analysis of Piles and Drilled Shafts Under Lateral Loads

by

Lymon C. Reese
Shin Tower Wang
William M. Isenhower
José A. Arréllaga
Joe Hendrix

for

ENSOFT, INC.
3003 West Howard Lane
Austin, Texas 78728

July - 2004

OHIO DEPARTMENT OF TRANSPORTATION

OFFICE OF GEOTECHNICAL ENGINEERING

PLAN SUBGRADES
Geotechnical Bulletin GB1

LUC-120-11.32
102940

Bridge Replacement: Central Avenue over Ottawa River, Toledo, Ohio

TTL Associates, Inc.

Prepared By: Christopher P. Iott, P.E.
Date prepared: Tuesday, June 23, 2020

Christopher P. Iott, P.E.
TTL Associates, Inc.
1915 N. 12th Street
Toledo, Ohio 43604
419-324-2222
ciott@ttlassoc.com

NO. OF BORINGS: 3

#	Boring ID	Alignment	Station	Offset	Dir	Drill Rig	ER	Boring EL.	Proposed Subgrade EL	Cut Fill
1	B-001-1-18	SR 120	598+82	24	Rt	CME 550X ATV	77	611.0	609.5	1.5 C
2	B-001-0-18	SR 120	598+82	24	Rt	CME 550X ATV	77	611.0	610.6	0.4 C
3	B-002-0-18	SR 120	601+16	21	Lt	CME 550X ATV	77	610.7	609.2	1.5 C

OHIO DEPARTMENT OF TRANSPORTATION

Subgrade Analysis

V. 14.5 1/18/2019

#	Boring	Sample	Sample Depth		Subgrade Depth		Standard Penetration		HP (tsf)	Physical Characteristics						Moisture		Ohio DOT		Sulfate Content (ppm)	Problem		Excavate and Replace (Item 204)		Recommendation (Enter depth in inches)
			From	To	From	To	N_{60}	N_{60L}		LL	PL	PI	% Silt	% Clay	P200	M_C	M_{OPT}	Class	GI		Unsuitable	Unstable	Unsuitable	Unstable	
1	B	1B	2.2	2.5	0.7	1.0	14		NP							9	10	A-2-6	4						
	001-1	2	2.5	4.0	1.0	2.5	41		3.5	24	16	8	35	8	43	14	11	A-4a	2	<100		Mc			
	18	3	4.0	6.0	2.5	4.5	8		0.63	27	16	11	22	41	63	17	14	A-6a	6						
		4	6.0	11.0	4.5	9.5	13	8	1.5							20	14	A-6a	10						
2	B	1A	1.3	2.2	0.9	1.8	14		NP							6	10	A-2-4	0						
	001-0	1B	2.2	2.5	1.8	2.1	14		NP							9	10	A-2-6	4						
	18	2	2.5	4.0	2.1	3.6	41		3.5	24	16	8	35	8	43	14	11	A-4a	2	<100					
		3	4.0	6.0	3.6	5.6	8	8	0.6	27	16	11	22	41	63	17	14	A-6a	6						
3	B	1B	1.7	3.7	0.2	2.2	14		NI	NP		NP	32	6	38	11	11	A-4a	1						
	002-0	2	3.7	4.0	2.2	2.5	27		NP	20	15	5	23	1	24	17	8	A-3a	0	<100					
	18	3	4.0	5.5	2.5	4.0	31		NP							9	10	A-2-4	0						
		4	5.5	8.0	4.0	6.5	14	14	NP							12	10	A-2-6	4						

OHIO DEPARTMENT OF TRANSPORTATION

Subgrade Analysis

V. 14.5 1/18/2019

PID: 102940

County-Route-Section: LUC-120-11.32
No. of Borings: 3

Geotechnical Consultant: TTL Associates, Inc.
Prepared By: Christopher P. Iott, P.E.
Date prepared: 6/23/2020

Chemical Stabilization Options

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

Excavate and Replace Stabilization Options

Global Geotextile Average(N60L): Average(HP):	 12" 12"
Global Geogrid Average(N60L): Average(HP):	 0" 0"

Design CBR	9

% Samples within 6 feet of subgrade

$N_{60} \le 5$	0%	$HP \le 0.5$	0%
$N_{60} < 12$	17%	$0.5 < HP \le 1$	17%
$12 \le N_{60} < 15$	50%	$1 < HP \le 2$	8%
$N_{60} \ge 20$	33%	$HP > 2$	17%
M+	8%		
Rock	0%		
Unsuitable	0%		

Excavate and Replace at Surface

Average	0"
Maximum	0"
Minimum	0"

% Proposed Subgrade Surface

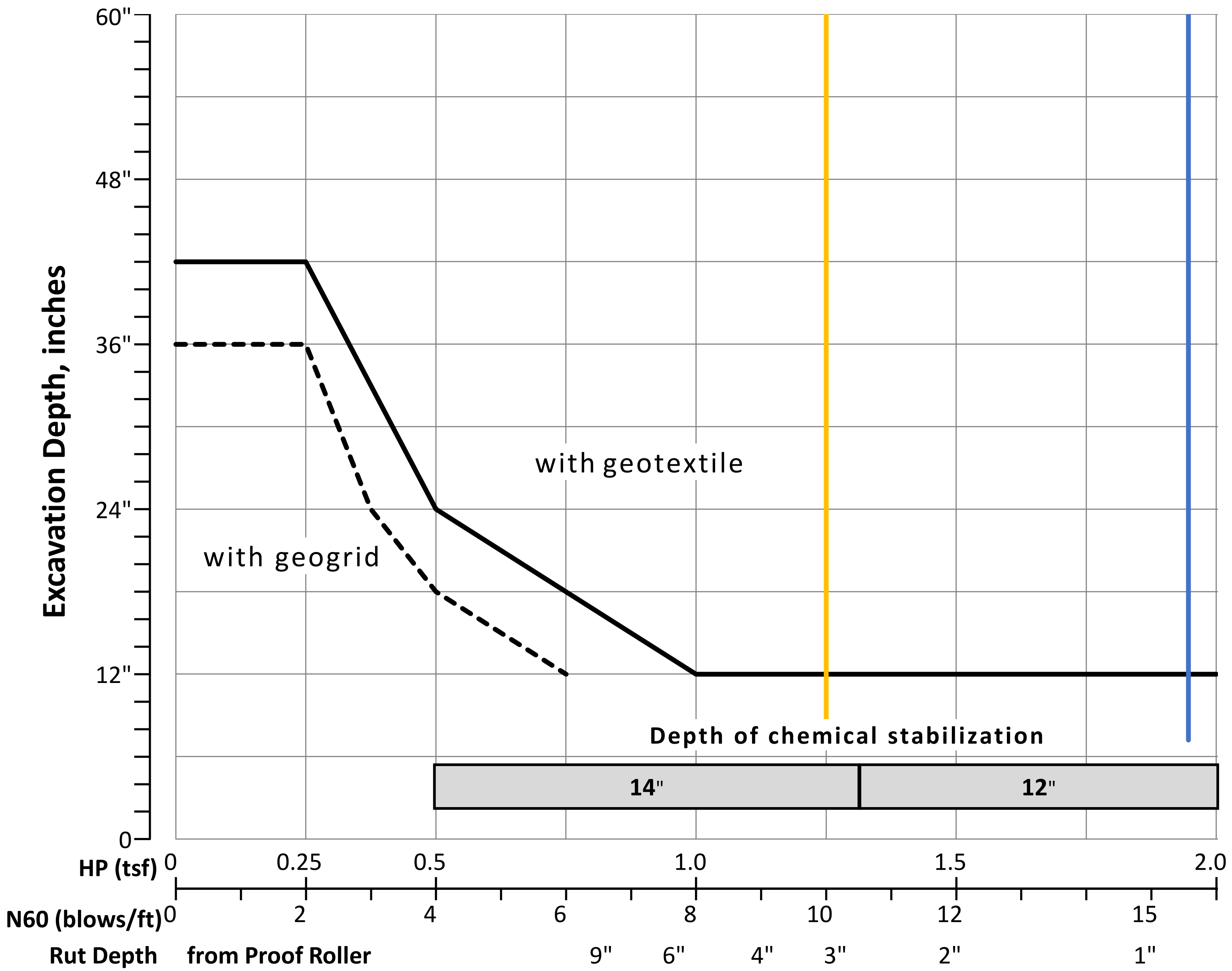
Unstable & Unsuitable	11%
Unstable	11%
Unsuitable	0%

	N_{60}	N_{60L}	HP	LL	PL	PI	Silt	Clay	P 200	M_C	M_{OPT}	GI
Average	20	10	1.95	24	16	9	28	18	46	13	11	3
Maximum	41	14	3.50	27	16	11	35	41	63	20	14	10
Minimum	8	8	0.60	20	15	5	22	1	24	6	8	0

Classification Counts by Sample

ODOT Class	Rock	A-1-a	A-1-b	A-2-4	A-2-5	A-2-6	A-2-7	A-3	A-3a	A-4a	A-4b	A-5	A-6a	A-6b	A-7-5	A-7-6	A-8a	A-8b	Totals
Count	0	0	0	2	0	3	0	0	1	3	0	0	3	0	0	0	0	0	12
Percent	0%	0%	0%	17%	0%	25%	0%	0%	8%	25%	0%	0%	25%	0%	0%	0%	0%	0%	100%
% Rock|Granular|Cohesive	0%	75%									25%								100%
Surface Class Count	0	0	0	2	0	2	0	0	1	3	0	0	1	0	0	0	0	0	9
Surface Class Percent	0%	0%	0%	22%	0%	22%	0%	0%	11%	33%	0%	0%	11%	0%	0%	0%	0%	0%	100%

GB1 Figure B – Subgrade Stabilization



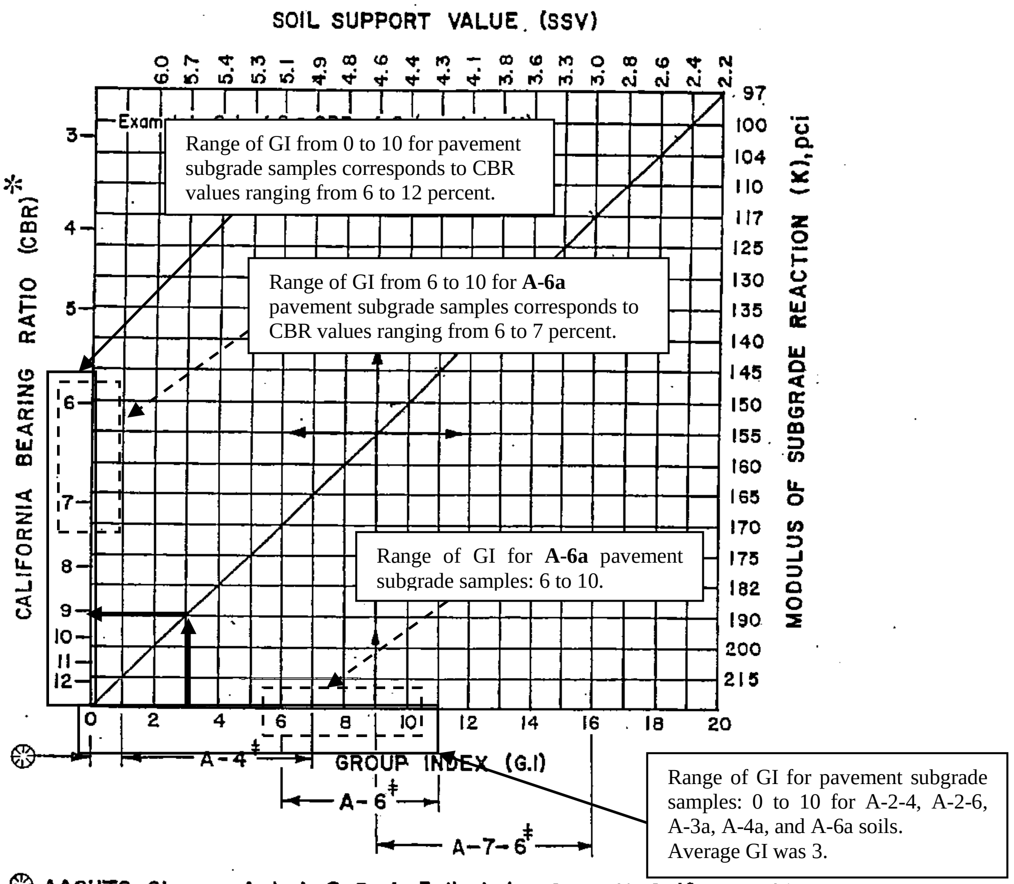
OVERRIDE TABLE

Calculated Average	New Values	Check to Override
1.95		☐ HP
10.00		☐ N60L

Average HP ▬

Average N_{60L} ▬

Fig.1301-3
Feb.1978



Range of GI from 0 to 10 for pavement subgrade samples corresponds to CBR values ranging from 6 to 12 percent.

Range of GI from 6 to 10 for **A-6a** pavement subgrade samples corresponds to CBR values ranging from 6 to 7 percent.

Range of GI for **A-6a** pavement subgrade samples: 6 to 10.

Range of GI for pavement subgrade samples: 0 to 10 for A-2-4, A-2-6, A-3a, A-4a, and A-6a soils. Average GI was 3.

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

‡ Usual range of AASHTO Classes.

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

CORRELATION CHART FOR
SUBGRADE STRENGTHS

ODOT GB-1 “Subgrade Analysis” worksheet resulted in a CBR value of 9 percent based on an average group index (GI) of 3. Group indices for the tested samples ranged from 0 to 10, which would correlate with a CBR value of 6 to 12 percent. However, it should be noted that, based on Boring B-001 located west of the bridge, ODOT A-6a soils were encountered at a depth of approximately 2½ feet below top of subgrade, when considering the beginning of the project where pavement grades will meet existing grades, and at a depth of approximately 3½ feet below top of subgrade for the portion of the project where grades will be raised approximately 1 foot. Group Indices associated with these soils tend to correlate with the lower CBR values of 6 to 7 percent compared to the GB-1 Design CBR value that was calculated based on the **average** Group Index value. These clays may govern the overall subgrade conditions. **As such, we recommend that the selected replacement pavement section incorporate a design CBR value of 6 percent**, or as an alternate, check that the pavement design is not sensitive to a variation in CBR value from 6 to 9 percent for the design traffic loading.

Appendix B:
Geotechnical Engineering Design Checklists

II. Reconnaissance and Planning Checklist

C-R-S: LUC-120-11.32	PID: 102940	Reviewer: CPI	Date: 08-21-19

Reconnaissance			
Y N **X**	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: ☐ Roadway plans ☐ Structures plans ☐ Geohazards plans	Plans prepared by others. Exploration performed at existing bridge structure location based on anticipated replacement structure at same location.
Y N X	2	Based on Section 302.2 in the SGE, has the Geotechnical Red Flag Summary, or in its absence, the resources listed in Section 202 of the SGE, been reviewed as part of the office reconnaissance?	Literature research was performed.
Y N X	3	Have all the features listed in Section 302.3 of the SGE been observed and evaluated during the field reconnaissance?	
Y N **X**	4	If notable features were discovered in the field reconnaissance, were the GPS coordinates of these features recorded?	

II. Reconnaissance and Planning Checklist

Planning - General			
Y N X	5	In planning the geotechnical exploration program for the project, have the specific geologic conditions, the proposed work, and existing subsurface exploration work been considered?	Moderately shallow bedrock was anticipated. Initially, footings on bedrock were anticipated. However, drilled shafts socketed into bedrock are currently planned.
Y N X	6	Have the borings been located to develop the maximum subsurface information while using a minimum number of borings?	
Y N X	7	Has 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 N X	8	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 N X	9	Have any previous geotechnical explorations been utilized to the fullest extent possible?	Shallow roadway borings (not extending to bedrock) were performed previously.
Y **N** X	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?	Text proposal was provided.
		The schedule of borings should present the following information for each boring:	
Y N X		☐ exploration identification number	
Y N X		☐ location by station and offset	
Y N X		☐ estimated amount of rock and soil, including the total for each for the entire program.	
Planning – Exploration Number			
Y N X	11	Have the coordinates, stations and offsets of all explorations (borings, probes, test pits, etc.) been identified?	
Y N X	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 N X	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?	

Notes:

II. Reconnaissance and Planning Checklist

Planning – Boring Types	
Y N X 14 Based on Sections 303.3 to 303.76 of the SGE, have the location, depth, and sampling requirements for the following boring types been determined for the project? Check all boring types utilized for this project: X 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) ☐ Karst (Type C6) ☐ Proposed Underground Utilities (Type D) ☐ Structure Borings (Type E) X Bridges (Type E1) ☐ Culverts (Type E2 a,b,c) ☐ Retaining Walls (Type E3 a,b,c) ☐ Noise Barrier (Type E4) ☐ High Mast Lighting Towers (Type E5) ☐ Buildings and Salt Domes (Type E6)	Type A performed in upper portion of Type E1 borings.

Notes:

III.C. Subgrade Checklist

C-R-S: LUC-120-11.32	PID: 102940	Reviewer: CPI	Date: 08-21-19

If you do not have any subgrade work on the project, you do not have to fill out this checklist.

Y N X	1	Has the subsurface investigation adequately characterized the soil or rock according to <u>Geotechnical Bulletin 1: Plan Subgrades (GB1)?</u>	
Y N **X**	2	If soils classified as A-2-5, A-4b, A-5, A-7-5, A-8a, or A-8b, or having a LL>65, are present at the proposed subgrade (soil profile), do the plans specify that these materials need to be removed and replaced or chemically stabilized?	Not encountered.
Y N X		a If these materials are to be removed and replaced, have the station limits, depth, and lateral limits for the planned removal been provided?	
Y N **X**	3	If there is any rock, shale, or coal present at the proposed subgrade (CMS 204.05), do the plans specify the removal of the material?	Not present at subgrade elevations.
Y N X		a If removal of any rock, shale, or coal is required, have the station limits, depth, and lateral limits for the planned removal of the material at proposed subgrade been provided?	
Y N X	4	In accordance with GB1, do the SPT values and existing moisture contents for the proposed subgrade soils indicate the need for subgrade stabilization?	
Y N **X**		a If removal and replacement is applicable, has the detail of subgrade removal been shown on the plans, including depth of removal, station limits, lateral extent, replacement material, and plan notes (Item 204 – Subgrade Compaction and Proof Rolling)?	Plans to be prepared by others. Discussion provided in report.
Y N **X**		b If chemical stabilization is applicable, has the detail of this treatment been shown on the plans, including depth, percentage of chemical, station limits, lateral extent, and plan notes?	Plans to be prepared by others. Discussion provided in report.
		Indicate type of subgrade treatment specified: X cement treatment ☐ lime treatment ☐ lime kiln dust ☐ other	Cement stabilization is indicated as an option, but is anticipated to be cost prohibitive compared to over-excavation and replacement.
Y N **X**	5	If drainage or groundwater is an issue with the proposed subgrade, has an appropriate drainage system (e.g., pipe, underdrains) been provided?	
Y N **X**	6	Has an appropriate quantity of Proof Rolling been included in the plans (CMS 204.06)?	Plans to be prepared by others.

III.C. Subgrade Checklist

Y N X 7 Has a design CBR value been provided?	

Notes:

Stage 1:

VI.D. Geotechnical Reports

C-R-S: LUC-120-11.32	PID: 102940	Reviewer: CPI	Date: 06-16-20

General	
[✓ Y] N X 1 Has the first complete version of a geotechnical report being submitted been labeled as ‘Draft’?	
[✓ Y] N X 2 Subsequent to ODOT’s review and approval, has the complete version of the revised geotechnical report being submitted been labeled ‘Final’?	This is the Final Report submittal.
[✓ Y] N X 3 Have all geotechnical reports being submitted been titled correctly as prescribed in Section 705.1 of the SGE?	

Report Body	
[✓ Y] N X 4 Do all geotechnical reports being submitted contain an Executive Summary as described in Section 705.2 of the SGE?	
[✓ Y] N X 5 Do all geotechnical reports being submitted contain an Introduction as described in Section 705.3 of the SGE?	
[✓ Y] N X 6 Do all geotechnical reports being submitted contain a section titled "Geology and Observations of the Project," as described in Section 705.4 of the SGE?	
[✓ Y] N X 7 Do all geotechnical reports being submitted contain a section titled "Exploration," as described in Section 705.5 of the SGE?	
[✓ Y] N X 8 Do all geotechnical reports being submitted contain a section titled "Findings," as described in Section 705.6 of the SGE?	
[✓ Y] N X 9 Do all geotechnical reports being submitted contain a section titled "Analyses and Recommendations," as described in Section 705.7 of the SGE?	

VI.D. Geotechnical Reports

Appendices		
Y N X 10	Do all geotechnical reports being submitted contain all applicable Appendices as described in Section 705.8 of the SGE?	
Y N X 11	Do the Appendices present a site Boring Plan showing all boring locations as described in Section 705.8.1 of the SGE?	
Y N X 12	Do the Appendices include boring logs as described in Section 705.8.2 of the SGE?	
Y N X 13	Do the Appendices present reports of undisturbed test data as described in Section 705.8.3 of the SGE?	
Y N X 14	Do the Appendices present calculations in a logical format to support recommendations as described in Section 705.8.4 of the SGE?	

Notes:

Appendix C:
Rock Core Photographic Logs

CORE PHOTO LOG - BORING B-001-0-18

GSE: 611.0

Project: LUC-120-11.32 Bridge Replacement
Project Location: Toledo, Lucas County, Ohio
TTL Project No.: 1771201
Core Date: April 4, 2019

Core Run	Depth (feet)	Elevation (feet)
RC-1	27.3 to 32.3	583.7 to 578.7
RC-2	32.3 to 37.3	578.7 to 573.7

Begin RC-1

Unconfined Compressive Strength
Test Specimen

End RC-1/Begin RC-2

Unconfined Compressive Strength
Test Specimen

Unconfined Compressive Strength
Test Specimen

End RC-2

TTL associates inc
Environmental, Geotechnical
Engineering & Testing

CORE PHOTO LOG - BORING B-002-0-18

GSE: 610.7

Project: LUC-120-11.32 Bridge Replacement
Project Location: Toledo, Lucas County, Ohio
TTL Project No.: 1771201
Core Date: April 3, 2019

Core Run	Depth (feet)	Elevation (feet)
RC-1	29.7 to 33.3	581.0 to 577.4
RC-2	33.3 to 34.7	577.4 to 576.0
RC-3	34.7 to 39.7	576.0 to 571.0

Begin RC-1

Unconfined Compressive Strength Test Specimen

Unconfined Compressive Strength Test Specimen

End RC-1/Begin RC-2

Unconfined Compressive Strength Test Specimen

End RC-2/Begin RC-3

End RC-3

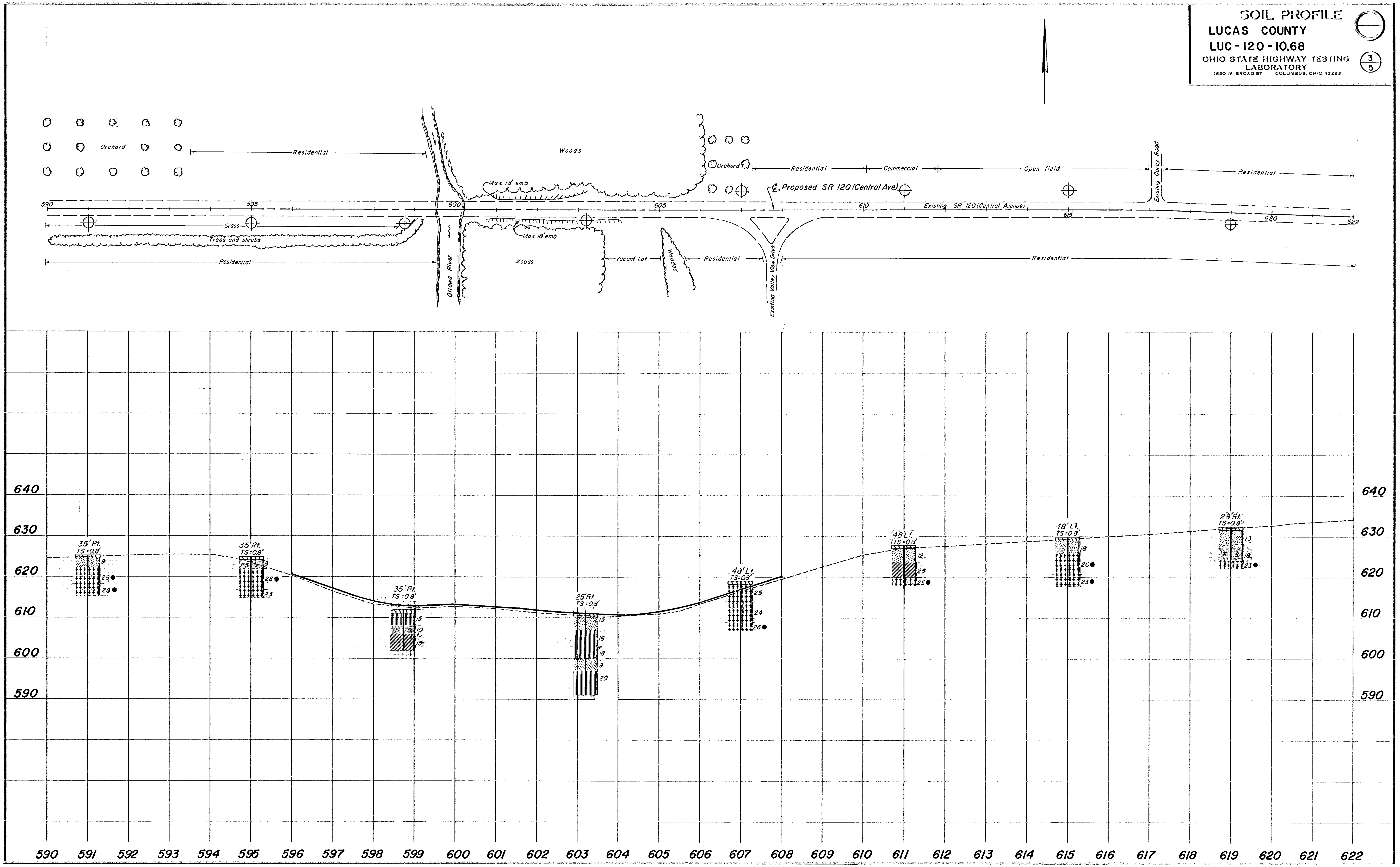
Appendix D:
Historic Borings

TE - 127
1500-12-66

SOIL PROFILE
LUCAS COUNTY
LUC - 120 - 10.68

OHIO STATE HIGHWAY TESTING
LABORATORY
1820 W. BROAD ST. COLUMBUS, OHIO 43223

3
5



FIELD BORING LOG

611.8'

County, Route No., Section LOC-120-10.68

Station 598+75 Offset 35RT Elev. 715

Date 4-2-68 Water Elev.

Crew SB-WJ KO Equipment TA

Drafting		

Depth Feet	Field Number	OC-01 Description
0.0 .8		TOPSOIL
4a	22/15	MOIST BR SANDY SILT
[illegible]	10	MOIST BR SAND
6a	15/24	MOIST BR SILTY CLAY w/ ROCK FRAG
10		COMPLETE 10
15		
20		
25		
30		

Use reverse side of this sheet for additional notes.

FIELD BORING LOG 611.0'

County, Route No., Section LUC-120-10.68

Station 603+20 Offset 25 RT Elev +8.2

Date 4-2-68 Water Elev.

Crew SB MJ KO Equipment T.A

Drafting		

Depth Feet	Field Number	OC-01 Description
0.0-0.2		TOP SOIL
3a	25 15	MOIST SANDY SILT W/ROCK FRAG
5 4a	16 26	MOIST SANDY SILT W/ROCK FRAG
4a 10	18 27	MOIST SANDY SILT W/ROCK FRAG
3a	9 28	MOIST BR SAND
15 4a	20 29	MOIST BR+GR SANDY SILT
20		COMPLETE 20
25		
30		

Use reverse side of this sheet for additional notes.