Resource International, Inc.

FRA-70-12.68 PROJECT 4R
FRA-70-1395C
S. FRONT STREET OVER I-70/71
PID NO. 105523
FRANKLIN COUNTY, OHIO

STRUCTURE FOUNDATION EXPLORATION REPORT

Prepared For:
GPD GROUP
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Columbus, OH 43215

Prepared By:
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Rii Project No. W-13-045

July 2018

RESOURCE INTERNATIONAL, INC.
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Planning

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April 3, 2015 (Revised July 11, 2018)

Mr. Christopher W. Luzier, P.E.
Project Manager
GPD GROUP
1801 Watermark Drive, Suite 210
Columbus, OH 43215

Re: Structure Foundation Exploration Report
FRA-70-12.68 Project 4R
FRA-70-1395C – S. Front Street over I-70/71
PID No. 105523
Rii Project No. W-13-045

Mr. Luzier:

Resource International, Inc. (Rii) is pleased to submit this structure foundation exploration report for the above referenced project. Engineering logs have been prepared and are attached to this report along with the results of laboratory testing. This report includes recommendations for the design and construction of the proposed FRA-70-1395C bridge structure carrying S. Front Street over I-70/71 as part of the FRA-70-12.68 Project 4R in Columbus, Ohio.

We sincerely appreciate the opportunity to be of service to you on this project. If you have any questions regarding the structure foundation exploration or this report, please contact us.

Sincerely,

RESOURCE INTERNATIONAL, INC.

Brian R. Trenner, P.E.
Director – Geotechnical Programming

Jonathan P. Sterenberg, P.E.
Director – Geotechnical Planning

Enclosure: Structure Foundation Exploration Report

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EXECUTIVE SUMMARY

Resource International, Inc. (Rii) has completed a structure foundation exploration for the design and construction of the proposed FRA-70-1395C bridge structures carrying S. Front Street and flanking cap structure to the east over I-70/71. It is understood that the existing structure will be removed and replaced with a two-span continuous composite steel plate girder structure with reinforced concrete deck structure and concrete substructures. The proposed I-70 roadway will require cuts up to 40 feet behind the existing abutments to maintain the proposed alignment and configuration. The two parallel structures will each react independently, with only a longitudinal expansion joint connecting them. The roadway profile along the I-70 eastbound beneath the structure will be cut approximately 5.0 feet below the existing roadway profile grade, and there will be no change in the profile grade of S. Front Street. The FRA-70-1390C Ramp C5 over I-70/71 bridge deck will also be integrated with the FRA-70-1395C bridge deck at the northwest corner of the structure, where the ramp will be aligned with Fulton Street.

Exploration and Findings

Between August 21 and 22, 2013, one (1) structural boring, designated as B-026-3-13, was drilled to a completion depth of 90.0 feet below the existing ground surface at the location shown on the boring plan provided in Appendix I of this report. In addition to the boring performed as part of the current exploration, two (2) historic borings, designated as B-002-F-59 and B-005-F-59, were obtained at the southwest and northeast corners of the existing bridge alignment, respectively. The historic borings were extended to a depth of 73.0 and 66.0 feet, respectively, below the ground surface at the time the borings were obtained.

Boring B-026-3-13 was performed within the existing sidewalk along the west side of S. Front Street, at the intersection with W. Fulton Street on the north side of the existing structure and encountered 6.0 inches of concrete overlying 6.0 inches of aggregate base at the existing ground surface. Beneath the surface materials, natural granular soils were encountered with intermittent layers of cohesive material. The granular soils were generally described as brown, gray and brownish gray gravel, gravel and sand, gravel with sand and silt and coarse and fine sand (ODOT A-1-a, A-1-b, A-2-4, A-3a). The cohesive soil seams encountered were generally described as brown, gray and brownish gray sandy silt, silt and clay and silty clay (ODOT A-4a, A-6a, A-6b).

Bedrock was not encountered in any of the borings performed during the historic or current explorations in the immediate vicinity of the S. Front Street structure. However, based on the subsurface conditions encountered in borings performed within the area of the adjacent FRA-70-1390C structure, shale bedrock is present at an approximate elevation of 630 feet msl.

In general, the historic borings encountered medium dense to very dense granular soils with intermittent layers of hard cohesive soils. The granular soils were generally described as brown and gray gravel, gravel and sand, gravel with sand and silt and coarse and fine sand (ODOT A-1-a, A-1-b, A-2-4, A-4a, A-4b), and the cohesive soils were generally described as gray sandy silt and silt and clay (ODOT A-4a, A-6a). Bedrock was not encountered in the historic borings prior to the termination depths. In general, the subsurface conditions encountered in the historic borings matched relatively closely with the subsurface conditions encountered in the current exploration boring

Analyses and Recommendations

Design details of the structure proposed were provided by GPD GROUP. Based on the information provided, it is understood that the existing structure will be removed and replaced with two parallel, two-span continuous composite steel plate girder structures with reinforced concrete deck structure and concrete substructures with a reinforced concrete pier supported on spread foundations and abutments supported on tangent drilled shafts.

Drilled Shaft Recommendations

It is understood that tangent drilled shaft foundations are being utilized to support the rear and forward abutment substructure units. It is recommended that the drilled shafts be designed using the axial design parameters provided in the following table. To achieve the most economical design, the drilled shafts should extend to bear in the very dense gravel or gravel and sand (ODOT A-1-a, A-1-b) at the corresponding elevations noted below in order to maximize the end bearing resistance. The drilled shafts should be proportioned for a nominal bearing resistance as follows:

Drilled Shaft Axial Design Parameters

Boring	Elevation [1] (feet msl)	Shaft Length (feet)	Soil Type	Nominal Resistance (ksf)		Resistance Factor	
				End	Side [2]	End	Side
B-026-3-13	725.8-719.9	0.0-5.9	A-4a	58	3.54	0.40	0.45
	719.9-704.9	5.9-20.9	A-1-b	60	2.86	0.50	0.55
	704.9-694.9	20.9-30.9	A-4a	72	3.60	0.40	0.45
	694.9-689.9	30.9-35.9	A-2-4	60	4.39	0.50	0.55
	689.9-669.9	35.9-55.9	A-1-b	60	3.89	0.50	0.55
	669.9-666.9	55.9-58.9	A-3a	60	3.11	0.50	0.55

Boring	Elevation [1] (feet msl)	Shaft Length (feet)	Soil Type	Nominal Resistance (ksf)		Resistance Factor	
				End	Side [2]	End	Side
B-002-F-59	725.8-712.1	0.0-13.7	A-1-b	60	2.91	0.50	0.55
	712.1-691.1	13.7-34.7	A-1-a	60	4.15	0.50	0.55
	691.1-681.1	34.7-44.7	A-3a	60	2.89	0.50	0.55
B-005-F-59	725.8-720.7	0.0-5.1	A-4a	57	3.60	0.40	0.45
	720.7-709.7	5.1-16.1	A-1-a	60	3.46	0.50	0.55
	709.7-699.7	16.1-26.1	A-1-b	60	4.02	0.50	0.55
	699.7-691.7	26.1-34.1	A-6a	72	3.60	0.40	0.45

1. *Top of shaft elevations based on structure information provided by GPD GROUP.*
2. *Side resistance should be neglected for the upper 5.0 feet of the shaft length where cohesive soils (ODOT A-4a, A-4b, A-6a, A-6b, A-7-6) are present below the top of shaft elevation.*

The axial resistance of a group of shafts may be less than the sum of the individual shaft resistance within a group of shafts. Per Section 10.8.3.6.3 of the AASHTO LRFD BDS, for soil profiles that consist of primarily granular soils, the individual nominal resistance of each drilled shaft shall be reduced by applying an adjustment factor, η, as defined in Table 10.8.3.6.1-1 of the AASHTO LRFD BDS. The following criteria are recommended for the group resistance of any shaft groups.

For a single row of drilled shafts:

- η = 0.9 for a center-to-center spacing of 2.0 diameters or less,
- η = 1.0 for a center-to-center spacing of 3.0 diameters or greater.

For intermediate spacing, the value of η may be determined by liner interpolation. Please note that the adjustment factor should be applied to the total individual nominal shaft resistance (including both end bearing side resistance along the shaft length).

Given that the drilled shafts at the abutments will be constructed tangent to each other, the shaft group capacity should also be checked using the block failure mechanism. Since the soil profile consists primarily of dense granular soils, the analysis should be performed considering the entire drilled shaft group as an equivalent strip footing with a length equal to the length of the tangent shaft wall and equivalent width equal to the total end area of the drilled shafts divided by the length of the drilled shaft wall. A resistance factor of φ_b = 0.45 should be utilized in calculating the factored bearing resistance for the this failure mode at the strength limit state.

The total group resistance shall be the lesser of the sum of the individual drilled shafts multiplied by the applicable group efficiency factor, η, or the resistance of the group in block failure mode.

Shallow Foundation Recommendations

It is understood that a shallow spread foundation will be utilized at the pier substructure unit. Based on plan information provided by GPD GROUP, the bottom of footing elevation at the pier substructure unit will bear at a minimum depth of 10.0 feet below the proposed finished grade, at the elevation noted Table 4 of the full report. At this elevation, the bearing soils are anticipated to consist of very dense gravel and gravel and sand (ODOT A-1-a, A-1-b). Shallow spread foundations bearing on these competent natural soils may be proportioned for a nominal bearing resistance as follows:

Spread Footing Design Parameters – Pier

Effective Footing Width (feet)	Service Limit Bearing Pressure (ksf) [1]			Nominal Bearing Resistance (ksf)	Factored Bearing Resistance [2] (ksf)
	1.0-inch	1.5-inch	2.0-inch		
10.0	2.50	5.33	13.61	121.68	54.76
12.0	2.35	4.81	11.85	131.26	59.07
14.0	2.25	4.44	10.62	140.90	63.40
16.0	2.17	4.16	9.72	150.56	67.75
18.0	2.11	3.95	9.04	160.22	72.10
20.0	2.06	3.78	8.51	169.86	76.44
22.0	2.02	3.65	8.09	179.45	80.75
24.0	1.99	3.54	7.70	188.99	85.04
26.0	1.96	3.44	7.29	198.46	89.31
28.0	1.94	3.36	6.93	207.85	93.53
30.0	1.92	3.30	6.63	217.17	97.73

1. *The service limit bearing pressure was calculated at total settlement values of 0.5, 1.0 and 2.0 inches.*
2. *A resistance factor of φ_b = 0.45 was utilized in calculating the factored nominal bearing resistance at the strength limit state.*

Based on the maximum service limit bearing pressure of 8.22 ksf, a total settlement of 1.55 inches is anticipated at the pier substructure unit. Additionally, the maximum factored bearing pressure of 10.65 ksf will not exceed the factored bearing resistance at the strength limit of 65.58 ksf at the pier substructure unit.

Please note that this executive summary does not contain all the information presented in the report. The unabridged subsurface exploration report should be read in its entirety to obtain a more complete understanding of the information presented.

1.0 INTRODUCTION

The overall purpose of this project is to provide detailed subsurface information and recommendations for the design and construction of the FRA-70-12.68/13.11/14.05C (Project 4R/4H/4A) projects in Columbus, Ohio. The projects represent the central portion of FRA-70-8.93 (PID 77369) I-70/71 south innerbelt improvements project. The FRA-70-12.68 (Project 4R) phase will consist of all work associated with the construction of Ramp C5, starting at the bridge over Souder Avenue and extending east to Front Street. The proposed Ramp C5 will be a two-lane to four-lane ramp that will collect and direct traffic from I-71 northbound and SR-315 southbound as well as I-70 eastbound to exit in downtown at the intersection of Front Street and W. Fulton Avenue. This project includes the construction of six (6) new bridge structures for the proposed Ramp C5 alignment and replacement of three (3) bridge structures, two along I-70 and the Front Street Structure over I-70, as well as the construction of fourteen (14) new retaining walls and a culvert structure to accommodate the new configuration.

This is a presentation of the structure foundation exploration performed for the design and construction of the proposed FRA-70-1395C bridge structures carrying S. Front Street and flanking cap structure to the east over I-70/71, as shown on the vicinity map and boring plan presented in Appendix I. The existing structure is a two-span bridge with a total length of approximately 137 feet. It is understood that the existing structure consists of a reinforced concrete deck on continuous steel beams, and will be removed and replaced with a two-span continuous composite steel plate girder structure with reinforced concrete deck and substructures. The proposed I-70 roadway will require cuts up to 40 feet behind the existing abutments to maintain the proposed alignment and configuration. The proposed structures will have an approximate length of 196 feet (center to center of bearings) and widths of 103 and 40 feet, representing the S. Front Street roadway and east cap structure, respectively. The two parallel structures will each react independently, with only a longitudinal expansion joint connecting them. The roadway profile along the I-70 eastbound beneath the structure will be cut approximately 5.0 feet below the existing roadway profile grade, and there will be no change in the profile grade of S. Front Street. The FRA-70-1390C Ramp C5 over I-70/71 bridge deck will also be integrated with the FRA-70-1395C bridge deck at the northwest corner of the structure, where the ramp will be aligned with Fulton Street.

2.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1 Site Geology

Both the Illinoian and Wisconsinan glaciers advanced over two-thirds of the State of Ohio, leaving behind glacial features such as moraines, kame deposits, lacustrine deposits and outwash terraces. The glacial and non-glacial regions comprise five physiographic sections based on geological age, depositional process and geomorphic occurrence (physical features or landforms). The project area lies within the Columbus Lowland District of the Till Plains Section. This area is characterized by flat to gently rolling ground moraine deposits from the Late Wisconsinan age. The site topography exhibits moderate to high relief. The ground moraine deposits are composed primarily of silty loam till (Darby, Bellefontaine, Centerburg, Grand Lake, Arcanum, Knightstown Tills), with smaller alluvium and outwash deposits bordering the Scioto River, its tributaries and floodplain areas. A ground moraine is the sheet of debris left after the steady retreat of glacial ice. The debris left behind ranges in composition from clay size particles to boulders (including silt, sand, and gravel). Outwash deposits consist of undifferentiated sand and gravel deposited by meltwater in front of glacial ice, and often occurs as valley terraces or low plains. Alluvium and alluvial terrace deposits range in composition from silty clay size particles to cobbles, usually deposited in present and former floodplain areas.

According to the bedrock geology and topography maps obtained from the Ohio Department of Natural Resources (ODNR), the underlying bedrock consists predominantly of the Middle to Lower Devonian-aged Columbus Limestone. This formation is further subdivided into two members in the central portion of the state, known as the Delhi and Bellepoint Members. The Delhi Member consists of light gray, finely to coarsely crystalline, irregularly bedded, fossiliferous limestone. The Bellepoint Member consists of variable brown, finely crystalline, massively bedded limy dolomite. Both of these members contain chert nodules. Just east of the Scioto River, the underlying bedrock consists of the Upper Devonian Ohio Shale Formation overlying the Middle Devonian-aged Delaware Limestone Formation. The Ohio Shale formation consists of brownish black to greenish gray, thinly bedded, fissile, carbonaceous shale. The Delaware Limestone consists of bluish gray, thin to medium bedded dolomitic limestone with nodules and layers of chert. Regionally, the bedrock surface forms a broad valley aligned roughly north-to-south beneath the Scioto River. According to bedrock topography mapping, the elevation of the bedrock surface ranges from approximately 600 feet mean sea level (msl) in the valley to approximately 625 feet msl near the project limits.

2.2 Existing Conditions

The proposed FRA-70-1395C structure is located at the existing S. Front Street over I-70/71 overpass, approximately 0.7 miles east of the Scioto River. The existing I-70/I-71 in the vicinity of the structure is a six-lane, bi-directional, composite asphalt and concrete paved roadway that is generally east-west aligned through downtown Columbus, Ohio. The existing S. Front Street crossing is a three-lane, asphalt paved roadway with northbound parking lane against the eastern curb. The existing I-70 profile is lowered from the surrounding terrain, as the existing corridor was cut approximately 20 to 25 below the existing grade of S. Front Street and the surrounding downtown area. Existing cast-in-place concrete wall-type abutments are present at both the rear and forward abutment, which extend east of the existing structure to the S. High Street crossing. Graded embankments are utilized along both sides of I-70/71 west of the structure abutments which are grass covered with patches of brush and other vegetation. The existing structure appears to be in poor condition, with concrete spalling and delamination evident on the columns, cracking in the curbs on the deck, failure of the end dam and longitudinal joint assemblies, and significant corrosion of the superstructure steel beams. This traffic volume along the project alignment is very high, and the alignment traverses primarily commercial and government properties. The surrounding terrain across the site is relatively flat-lying.

3.0 EXPLORATION

Between August 21 and 22, 2013, one (1) structural boring, designated as B-026-3-13, was drilled to a completion depth of 90.0 feet below the existing ground surface at the location shown on the boring plan provided in Appendix I of this report and summarized in Table 1 below. Borings B-027-0-08 and B-278-0-10 were also performed by DLZ in close proximity to the proposed structure as part of the FRA-70-08.93 and FRA-70-14.48 projects, respectively. However, these were borings drilled to a depth of 14.0 and 15.0 feet below the existing roadway grade for pavement subgrade analyses along I-70 westbound and W. Fulton Avenue, respectively. Given the shallow depth of these borings, it is considered that these borings do not provide sufficient subsurface information for analysis of the bridge structure foundations. As such, these borings are shown on the boring plan in Appendix I, but are not included in the commentary for the subsurface conditions and the boring logs are not included in this report.

Table 1. Test Boring Summary

Boring Number	Reference Alignment	Station	Offset	Latitude	Longitude	Ground Elevation (feet msl)	Boring Depth (feet)
B-026-3-13	BL Ramp C5	5091+04.93	11.5' Lt.	39.953296762	-83.000848553	756.9	90.0

The boring location was determined and located in the field by Rii representatives. Rii utilized a handheld GPS unit to obtain northing and easting coordinates of the boring location. The ground surface elevation at the boring location was interpolated using topographic mapping information provided by GPD GROUP.

The boring was drilled using an all-terrain vehicle (ATV) mounted rotary drilling machine, utilizing a 3.25-inch inside diameter, hollow-stem auger to advance the hole. Standard penetration test (SPT) and split spoon sampling were performed in the boring at 2.5-foot increments of depth to 30 feet and at 5.0-foot increments thereafter to the boring termination depth.

The SPT, per the American Society for Testing and Materials (ASTM) designation D1586, is conducted using a 140-pound hammer falling 30.0 inches to drive a 2.0-inch outside diameter split spoon sampler 18.0 inches. Rii utilized a calibrated automatic drop hammer to generate consistent energy transfer to the sampler. Driving resistance is recorded on the boring logs in terms of blow per 6.0-inch interval of the driving distance. The second and third intervals are added to obtain the number of blows per foot (N). Standard penetration blow counts aid in determining soil properties applicable in foundation system design. Measured blow count (N) values are corrected to an equivalent (60%) energy ratio, N_{60}, by the following equation. Both values are represented on boring logs in Appendix III.

$$N_{60} = N_m*(ER/60)$$

Where:
N_m = measured N value
ER = drill rod energy ratio, expressed as a percent, for the system used

The hammer for the ATV-mounted drill rig used for the current exploration was calibrated on April 26, 2013, and has a drill rod energy ratio of 82.6 percent.

During drilling for the borings, field logs were prepared by Rii personnel showing the encountered subsurface conditions. Soil samples obtained from the drilling operation were preserved and sealed in glass jars and delivered to the soil laboratory. In the laboratory, the soil samples were visually classified and select samples were tested, as noted in Table 2.

Table 2. Laboratory Test Schedule

Laboratory Test	Test Designation	Number of Tests Performed
Natural Moisture Content	ASTM D 2216	24
Plastic and Liquid Limits	AASHTO T89, T90	8
Gradation – Sieve/Hydrometer	AASHTO T88	8

The tests performed are necessary to classify existing soil according to the Ohio Department of Transportation (ODOT) classification system and to estimate engineering properties of importance in determining foundation design and construction recommendations. Results of the laboratory testing are presented on the boring logs in Appendix III. A description of the soil terms used throughout this report is presented in Appendix II.

Hand penetrometer readings, which provide a rough estimate of the unconfined compressive strength of the soil, were reported on the boring logs in units of tons per square foot (tsf) and were utilized to classify the consistency of the cohesive soil in each layer. An indirect estimate of the unconfined compressive strength of the cohesive split spoon samples can also be made from a correlation with the blow counts (N_{60}). Please note that split spoon samples are considered to be disturbed and the laboratory determination of their shear strengths may vary from undisturbed conditions.

In addition to the boring performed as part of the current exploration, historic borings performed in 1959 by the Department of Highways as part of the original FRA-40-12.82 project for the existing structure were obtained from the construction documents on record. Two (2) borings, designated as B-002-F-59 and B-005-F-59, were obtained at the southwest and northeast corners of the existing bridge alignment, respectively. Based on the elevations provided on the boring logs, it is anticipated that these borings were performed from the then-existing ground surface and that the profile for the then proposed US 40 (existing I-70/71) was lowered to provide sufficient clearance for the bridge to be constructed at the then-existing ground surface. The borings were extended to a depth of 73.0 and 66.0 feet, respectively, below the ground surface at the time the borings were obtained. Please note that the elevations provided on the historic boring logs were referenced to the North American Datum (NAD) 27. The current design survey is referenced to NAD 83. The NAD 27 datum is 0.6 feet lower than the NAD 83 datum. **Therefore, all elevations noted in this report with respect to the historic borings are adjusted to the current NAD 83 datum.** The historic boring locations are shown on the boring plan provided in Appendix I of this report and the historic boring logs are provided in Appendix IV.

4.0 FINDINGS

Interpreted engineering logs have been prepared based on the field logs, visual examination of samples and laboratory test results. Classification follows the respective version of the ODOT Specifications for Geotechnical Explorations (SGE) at the time the exploration borings were performed. The following is a summary of what was found in the test borings and what is represented on the boring logs.

4.1 Surface Materials

Boring B-026-3-13 was performed within the existing sidewalk along the west side of S. Front Street, at the intersection with W. Fulton Street on the north side of the existing structure and encountered 6.0 inches of concrete overlying 6.0 inches of aggregate base at the existing ground surface. Surface materials were not noted on the 1959 historic boring logs.

4.2 Subsurface Soils

Beneath the surface materials, natural granular soils were encountered with intermittent layers of cohesive material. The granular soils were generally described as brown, gray and brownish gray gravel, gravel and sand, gravel with sand and silt and coarse and fine sand (ODOT A-1-a, A-1-b, A-2-4, A-3a). The cohesive soil seams encountered were generally described as brown, gray and brownish gray sandy silt, silt and clay and silty clay (ODOT A-4a, A-6a, A-6b).

The relative density of granular soils is primarily derived from SPT blow counts (N_{60}). Based on the SPT blow counts obtained, the granular soil encountered ranged from loose ($5 \leq N_{60} \leq 10$ blows per foot [bpf]) to very dense ($N_{60} > 50$ bpf). Overall blow counts recorded from the SPT sampling ranged from 6 bpf to split spoon sampler refusal. The shear strength and consistency of the cohesive soils are primarily derived from the hand penetrometer values (HP). The cohesive soil encountered ranged from soft ($0.25 \leq HP \leq 0.5$ tsf) to hard ($HP > 4.0$ tsf). The unconfined compressive strength of the cohesive soil samples tested, obtained from the hand penetrometer, ranged from 0.5 to over 4.5 tsf (limit of instrument).

Natural moisture contents of the soil samples tested ranged from 6 to 23 percent. The natural moisture content of the cohesive soil samples tested for plasticity index ranged from 5 percent below to 4 percent above their corresponding plastic limits. In general, the soil exhibited natural moisture contents considered to be moderately below to moderately above optimum moisture levels.

4.3 Bedrock

Bedrock was not encountered in any of the borings performed during the historic or current explorations in the immediate vicinity of the S. Front Street structure. However, based on the subsurface conditions encountered in borings performed within the area of the adjacent FRA-70-1390C structure, shale bedrock is present at an approximate elevation of 630 feet msl.

4.4 Groundwater

Groundwater was encountered in the borings as presented in Table 3.

Table 3. Groundwater Levels

Boring Number	Ground Surface Elevation (feet msl)	Initial Groundwater		Upon Completion	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-026-3-13	756.9	36.0	720.9	N/A	N/A

Groundwater was encountered initially during the drilling process in boring B-026-3-13 at a depth of 36.0 feet below the existing ground surface, which corresponds to an elevation of 720.9 feet msl. The groundwater level at the completion of drilling was not obtained prior to backfilling the borehole. Please note that short-term water level readings, especially in cohesive soils, are not necessarily an accurate indication of the actual groundwater level. In addition, groundwater levels or the presence of groundwater are considered to be dependent on seasonal fluctuations in precipitation.

A more comprehensive description of what was encountered during the drilling process may be found on the boring log in Appendix III.

4.5 Historic Borings

In general, the historic borings encountered medium dense to very dense granular soils with intermittent layers of hard cohesive soils. The granular soils were generally described as brown and gray gravel, gravel and sand, gravel with sand and silt and coarse and fine sand (ODOT A-1-a, A-1-b, A-2-4, A-4a, A-4b), and the cohesive soils were generally described as gray sandy silt and silt and clay (ODOT A-4a, A-6a). Bedrock was not encountered in the historic borings prior to the termination depths. Groundwater levels were not noted in the borings performed during the 1959 exploration. In general, the subsurface conditions encountered in the historic borings matched relatively closely with the subsurface conditions encountered in the current exploration boring.

5.0 ANALYSES AND RECOMMENDATIONS

Data obtained from the historical and current subsurface explorations have been used to determine the foundation support capabilities and the settlement potential for the soil encountered at the site. These parameters have been used to provide guidelines for the design of foundation systems for the subject bridge, as well as the construction specifications related to the placement of foundation systems and general earthwork recommendations, which are discussed in the following paragraphs.

Design details of the structure proposed were provided by GPD GROUP. Based on the information provided, it is understood that the existing structure will be removed and replaced with two parallel, two-span continuous composite steel plate girder structures with reinforced concrete deck and a reinforced concrete pier supported on spread foundations and abutments supported on tangent drilled shafts. Proposed structural data was obtained from design details provided by GPD GROUP and are included in Table 4.

Table 4. Structure and Bridge Design Elevations

Substructure Unit	Structure Component [1]	Elevation [1] (feet msl)	Design Maximum Factored Load
Rear Abutment	Top of Shaft	Roadway: 725.8 East Cap: 725.8	273 kips/shaft
Pier	Bottom of Footing	Roadway: 718.5 East Cap: 718.5	8.22 ksf (Service) 10.65 ksf (Strength)
Forward Abutment	Top of Shaft	Roadway: 725.8 East Cap: 725.8	289 kips/shaft

1. *Proposed foundation elevations and structural loading based on structure information provided by GPD GROUP.*

5.1 Drilled Shaft Recommendations

It is understood that tangent drilled shaft foundations are being utilized to support the rear and forward abutment substructure units. It is recommended that the drilled shafts be designed using the axial design parameters provided in Table 5. To achieve the most economical design, the drilled shafts should extend to bear in the very dense gravel or gravel and sand (ODOT A-1-a, A-1-b) at the corresponding elevations noted below in order to maximize the end bearing resistance. The drilled shafts should be proportioned for a nominal bearing resistance as follows:

Table 5. Drilled Shaft Axial Design Parameters

Boring	Elevation [1] (feet msl)	Shaft Length (feet)	Soil Type	Nominal Resistance (ksf)		Resistance Factor	
				End	Side [2]	End	Side
B-026-3-13	725.8-719.9	0.0-5.9	A-4a	58	3.54	0.40	0.45
	719.9-704.9	5.9-20.9	A-1-b	60	2.86	0.50	0.55
	704.9-694.9	20.9-30.9	A-4a	72	3.60	0.40	0.45
	694.9-689.9	30.9-35.9	A-2-4	60	4.39	0.50	0.55
	689.9-669.9	35.9-55.9	A-1-b	60	3.89	0.50	0.55
	669.9-666.9	55.9-58.9	A-3a	60	3.11	0.50	0.55
B-002-F-59	725.8-712.1	0.0-13.7	A-1-b	60	2.91	0.50	0.55
	712.1-691.1	13.7-34.7	A-1-a	60	4.15	0.50	0.55
	691.1-681.1	34.7-44.7	A-3a	60	2.89	0.50	0.55
B-005-F-59	725.8-720.7	0.0-5.1	A-4a	57	3.60	0.40	0.45
	720.7-709.7	5.1-16.1	A-1-a	60	3.46	0.50	0.55
	709.7-699.7	16.1-26.1	A-1-b	60	4.02	0.50	0.55
	699.7-691.7	26.1-34.1	A-6a	72	3.60	0.40	0.45

1. *Top of shaft elevations based on structure information provided by GPD GROUP.*
2. *Side resistance should be neglected for the upper 5.0 feet of the shaft length where cohesive soils (ODOT A-4a, A-4b, A-6a, A-6b, A-7-6) are present below the top of shaft elevation.*

Drilled shaft lengths should measure a minimum of three (3) times the shaft diameter. Per Section 10.8.3.5.1b of the AASHTO LRFD BDS, side resistance should be neglected for the upper 5.0 feet of the shaft length where cohesive soils (ODOT A-4a, A-4b, A-6a, A-6b, A-7-6) are present below the bottom of footing/top of shaft elevation. Total settlement of the drilled shafts is estimated to be less than 1.0 inch for shafts bearing at or below elevation 705.0 feet msl.

Per Section 10.8.3.5.3 of the AASHTO LRFD BDS, where drilled shafts are extended to end bear in a strong soil layer overlying a weaker soil layer, the end bearing resistance shall be reduced if the tip elevation is within 1.5 times the diameter of the drilled shaft above the top of the weaker soil layer. A weighted average that varies linearly from the full end bearing resistance in the overlying strong soil layer at a distance of 1.5 times the diameter of the drilled shaft above the top of the weak soil layer to the end bearing resistance of the weak soil layer at the top of the weak soil layer should be used to determine the end bearing resistance utilized in the design. Therefore, the end bearing resistance utilized in the design will need to be adjusted accordingly if the tip elevation of the drilled shafts will be within 1.5 times the diameter of the drilled shaft above the underlying weaker soil layer. Drilled shaft calculations are provided in Appendix V.

5.1.1 Group Efficiency

The axial resistance of a group of shafts may be less than the sum of the individual shaft resistance within a group of shafts. Per Section 10.8.3.6.3 of the AASHTO LRFD BDS, for soil profiles that consist of primarily granular soils, the individual nominal resistance of each drilled shaft shall be reduced by applying an adjustment factor, η, as defined in Table 10.8.3.6.1-1 of the AASHTO LRFD BDS. The following criteria are recommended for the group resistance of any shaft groups.

For a single row of drilled shafts:

- η = 0.9 for a center-to-center spacing of 2.0 diameters or less,
- η = 1.0 for a center-to-center spacing of 3.0 diameters or greater.

For intermediate spacing, the value of η may be determined by liner interpolation. Please note that the adjustment factor should be applied to the total individual nominal shaft resistance (including both end bearing side resistance along the shaft length).

Given that the drilled shafts at the abutments will be constructed tangent to each other, the shaft group capacity should also be checked using the block failure mechanism. Since the soil profile consists primarily of dense granular soils, the analysis should be performed considering the entire drilled shaft group as an equivalent strip footing with a length equal to the length of the tangent shaft wall and equivalent width equal to the total end area of the drilled shafts divided by the length of the drilled shaft wall. A resistance factor of φ_b = 0.45 should be utilized in calculating the factored bearing resistance for the this failure mode at the strength limit state.

The total group resistance shall be the lesser of the sum of the individual drilled shafts multiplied by the applicable group efficiency factor, η, or the resistance of the group in block failure mode.

5.1.2 Lateral Design

If lateral load or moments are expected to be applied on the foundation elements, they should be analyzed to verify the shaft has enough lateral and bending resistance against these loads. A boring-by-boring tabulation of parameters that should be used for lateral loading design is provided in Appendix VI. In order to evaluate the lateral capacity, it is recommended that a derivation of COM624, such as LPILE, be utilized to determine the proper embedment depth and cross section required to resist the lateral load for a given end condition and deflection. Table 6 lists the eleven different soil types internal to the LPILE program. These strata were utilized to define the soil strata in the soil profile for each boring provided in Appendix VI.

Table 6. Subsurface Strata Description

Strata	Description
1	Soft Clay
2	Stiff Clay with Water
3	Stiff Clay without Free Water
4	Sand (Reese)
5	User Defined
6	Vuggy Limestone (Strong Rock)
7	Silt (with cohesion and internal friction angle)
8	API Sand
9	Weak Rock
10	Liquefiable Sand (Rollins)
11	Stiff Clay without free water with a specified initial K (Brown)

For the case of closely spaced drilled shafts, a pile group reduction factor will need to be applied to the p-y curves that are internally generated by the lateral analysis software. Reese, Isenhower, and Wang published an equation for the pile group p-reduction factor, otherwise known as p-multiplier (β_a), for a single row of piles placed side by side in the publication “Analysis and Design of Shallow and Deep Foundations” (2006), as follows:

$$\beta_a = 0.64(S/D)^{0.34}$$

In which:

$$1 \leq S/D < 3.75 \text{ and } 0.5 \leq \beta_a \leq 1.0$$

Where:
S = center to center spacing of the drilled shafts
D = diameter of drilled shafts

5.1.3 Drilled Shaft Considerations

The minimum requirements for proper inspection of drilled shaft construction are as follows:

- A qualified inspector should record the material types being removed from the hole as excavation proceeds.
- When the bearing material has been encountered and identified and/or the design tip elevation has been reached, the shaft walls and base should be observed for anomalies, unexpected soft soil conditions, obstructions or caving.
- Concrete placed freefall should not be allowed to hit the sidewalls of the excavation or the rebar cage and should not pass through any water.
- Structural stability of the rebar cage should be maintained during the concrete pour to prevent buckling.
- The volume of concrete should be checked to ensure voids did not result during extraction of the casing (if utilized).
- The placement of all concrete for the drilled shafts shall follow the American Concrete Institute's Design and Construction of Drilled Piers (ACI 336.3R-93).
- If concrete is placed by tremie method, it must be done so with an adequate head to displace water or slurry if groundwater has entered the caisson (all tremie procedures shall follow applicable ACI specifications).
- Pulling casing with insufficient concrete inside should be restricted.
- The bottom of drilled shaft excavation should be clean and free of loose material. Any loose material observed should be removed using a clean-out bucket (muck bucket).

The use of casing for drilled shafts is recommended under any of the following conditions:

- Caving material is encountered at any time during the drilling of the shaft.
- Groundwater is encountered at any time during the drilling of the shaft, or groundwater seepage occurs in the drilled shaft.
- Down hole inspection is planned (casing is required for this instance).

In addition, it is recommended that if casing is used, it be pulled immediately after the concrete is placed, allowing for re-use of the casing and eliminating reduction of side resistance (between soil and concrete).

It is anticipated that conventional drilled shaft equipment (with a standard soil bit) will be able to penetrate the surficial soils to the required tip elevation. Although not encountered in any of the borings performed for this structure, boulders were encountered in several of the borings performed in the area of this structure and should be anticipated to be encountered during installation of the drilled shafts. If boulders are encountered during installation of the drilled shafts, specialized drilling/coring equipment may be required to advance the drilled shaft excavation beyond the obstruction.

5.2 Shallow Foundation Recommendations

It is understood that a shallow spread foundation will be utilized at the pier substructure unit. Based on plan information provided by GPD GROUP, the bottom of footing elevation at the pier substructure unit will bear at a minimum depth of 10.0 feet below the proposed finished grade, at the elevation noted above in Table 4. At this elevation, the bearing soils are anticipated to consist of very dense gravel and gravel and sand (ODOT A-1-a, A-1-b). Shallow spread foundations bearing on these competent natural soils may be proportioned for a nominal bearing resistance as follows:

Table 7. Spread Footing Design Parameters – Pier

Effective Footing Width (feet)	Service Limit Bearing Pressure (ksf) [1]			Nominal Bearing Resistance (ksf)	Factored Bearing Resistance [2] (ksf)
	1.0-inch	1.5-inch	2.0-inch		
10.0	2.50	5.33	13.61	121.68	54.76
12.0	2.35	4.81	11.85	131.26	59.07
14.0	2.25	4.44	10.62	140.90	63.40
16.0	2.17	4.16	9.72	150.56	67.75
18.0	2.11	3.95	9.04	160.22	72.10
20.0	2.06	3.78	8.51	169.86	76.44
22.0	2.02	3.65	8.09	179.45	80.75
24.0	1.99	3.54	7.70	188.99	85.04
26.0	1.96	3.44	7.29	198.46	89.31
28.0	1.94	3.36	6.93	207.85	93.53
30.0	1.92	3.30	6.63	217.17	97.73

1. *The service limit bearing pressure was calculated at total settlement values of 0.5, 1.0 and 2.0 inches.*
2. *A resistance factor of $\varphi_b = 0.45$ was utilized in calculating the factored bearing resistance at the strength limit state.*

The nominal bearing resistance that results in a maximum total settlement of 0.5, 1.0 and 2.0 inches was calculated and presented in Table 7. A geotechnical resistance factor of φ_b = 0.45 has been considered in calculating the factored bearing resistance at the strength limit state. Based on the bearing pressures provided in Table 7 and applying the geotechnical resistance factor provided to the nominal bearing resistance at the strength limit state, the service limit state should control the minimum footing dimensions for all effective footing widths analyzed at 0.5, 1.0 and 2.0 inches of total settlement considered in the analysis. A graphical representation of the service limit bearing pressures and nominal and factored bearing resistance at the strength limit state is presented in Appendix VII.

Based on the maximum service limit bearing pressure of 8.22 ksf, a total settlement of 1.55 inches is anticipated at the pier substructure unit. Additionally, the maximum factored bearing pressure of 10.65 ksf will not exceed the factored bearing resistance at the strength limit of 65.58 ksf at the pier substructure unit. Calculations for settlement and nominal and factored bearing resistance for the shallow spread foundations are provided in Appendix VIII.

5.2.1 Sliding Resistance

The resistance of the footings to sliding will be dependent on the friction between the concrete footing and bearing surface. For concrete footing that rest on cohesionless soil, a coefficient "f" of 0.90 times the total vertical force on the base should be taken as the sliding resistance. A geotechnical resistance factor of φ_τ = 0.80 should be considered when calculating the factored shear resistance between the soil and foundation for sliding.

5.3 Lateral Earth Pressure

For the soil types encountered in the borings, the "in-situ" unit weight (γ), cohesion (c), effective angle of friction (φ'), and lateral earth pressure coefficients for at-rest conditions (k_o), active conditions (k_a), and passive conditions (k_p) have been estimated and are provided in Table 8 and Table 9.

Table 8. Estimated Undrained (Short-term) Soil Parameters for Design

Soil Type	γ (pcf) [1]	c (psf)	φ	k_a	k_o	k_p
Soft to Stiff Cohesive Soil	115	1,000	0°	N/A	N/A	N/A
Very Stiff to Hard Cohesive Soil	125	3,000	0°	N/A	N/A	N/A
Loose Granular Soil	120	0	28°	0.36	0.53	2.77
Medium Dense to Dense Granular Soil	130	0	32°	0.31	0.47	3.25
Very Dense Granular Soil	135	0	35°	0.27	0.43	3.69
Compacted Cohesive Engineered Fill	120	2,000	0°	N/A	N/A	N/A
Compacted Granular Engineered Fill	130	0	33°	0.30	0.46	3.39

1. *When below groundwater table, use effective unit weight, $\gamma' = \gamma$ - 62.4 pcf and add hydrostatic water pressure.*

Table 9. Estimated Drained (Long-term) Soil Parameters for Design

Soil Type	γ (pcf) [1]	c (psf)	φ'	k_a	k_o	k_p
Soft to Stiff Cohesive Soil	115	0	24°	0.42	0.59	2.37
Very Stiff to Hard Cohesive Soil	125	100	28°	0.36	0.53	2.77
Loose Granular Soil	120	0	28°	0.36	0.53	2.77
Medium Dense to Dense Granular Soil	130	0	32°	0.31	0.47	3.25
Very Dense Granular Soil	135	0	35°	0.27	0.43	3.69
Compacted Cohesive Engineered Fill	120	100	28°	0.36	0.53	2.77
Compacted Granular Engineered Fill	130	0	33°	0.30	0.46	3.39

1. *When below groundwater table, use effective unit weight, $\gamma' = \gamma$ - 62.4 pcf and add hydrostatic water pressure.*

These parameters are considered appropriate for the design of all subsurface structures and any excavation support systems. Subsurface structures (where the top of the structure is restrained from movement) should be designed based on at-rest conditions (k_o). For proposed temporary retaining structures (where the top of the structure is allowed to move), earth pressure distributions should be based on active (k_a) and passive (k_p) conditions. The values in this table have been estimated from correlation charts based on minimum standards specified for compacted engineered fill materials. These recommendations do not take into consideration the effect of any surcharge loading or a sloped ground surface (a flat surface is considered). Earth pressures on excavation support systems will be dependent on the type of sheeting and method of bracing or anchorage.

5.4 Construction Considerations

All site work shall conform to local codes and to the latest ODOT Construction and Materials Specifications (CMS), including that all excavation and embankment preparation and construction should follow ODOT Item 200 (Earthwork).

5.4.1 Excavation Considerations

All excavations should be shored / braced or laid back at a safe angle in accordance to Occupational Safety and Health Administration (OSHA) guidelines. During excavation, if slopes cannot be laid back to OSHA Standards due to adjacent structures or other obstructions, temporary shoring may be required. The following table should be utilized as a general guide for implementing OSHA guidelines when estimating excavation back slopes at the various boring locations. Actual excavation back slopes must be field verified by qualified personnel at the time of excavation in strict accordance with OSHA guidelines.

Table 10. Excavation Back Slopes

Soil	Maximum Back Slope	Notes
Soft to Medium Stiff Cohesive	1.5 : 1.0	Above Ground Water Table and No Seepage
Stiff Cohesive	1.0 : 1.0	Above Ground Water Table and No Seepage
Very Stiff to Hard Cohesive	0.75 : 1.0	Above Ground Water Table and No Seepage
All Granular & Cohesive Soil Below Ground Water Table or with Seepage	1.5 : 1.0	None

5.4.2 Groundwater Considerations

Based on the groundwater observations made during drilling, groundwater is anticipated during construction of the drilled shafts and may be encountered during excavation for the pier foundation. Where groundwater is encountered, proper groundwater control should be employed and maintained to prevent disturbance to excavation bottoms consisting of cohesive soil, and to prevent the possible development of a quick or "boiling" condition where soft silts and/or fine sands are encountered. It is preferable that the groundwater level, if encountered, be maintained at least 36 inches below the deepest excavation. In the case of drilled shafts, the utilization of casing will be required below the water table to maintain an open hole and prevent the sidewalls from collapse. In addition, concrete placed below the water table should be placed by tremie method using a rigid tremie pipe. Any seepage or groundwater encountered at this site should be able to be controlled by pumping from temporary sumps. Additional measures may be required depending on seasonal fluctuations of the groundwater level. Note that determining and maintaining actual groundwater levels during construction is the responsibility of the contractor.

6.0 LIMITATIONS OF STUDY

The above recommendations are predicated upon construction inspection by a qualified soil technician under the direct supervision of a professional geotechnical engineer. Adequate testing and inspection during construction are considered necessary to assure an adequate foundation system and are part of these recommendations.

The recommendations for this project were developed utilizing soil and bedrock information obtained from the test borings that were made at the proposed site for the current investigation. Resource International is not responsible for the data, conclusions, opinions or recommendations made by others during previous investigations at this site. At this time we would like to point out that soil borings only depict the soil and bedrock conditions at the specific locations and time at which they were made. The conditions at other locations on the site may differ from those occurring at the boring locations.

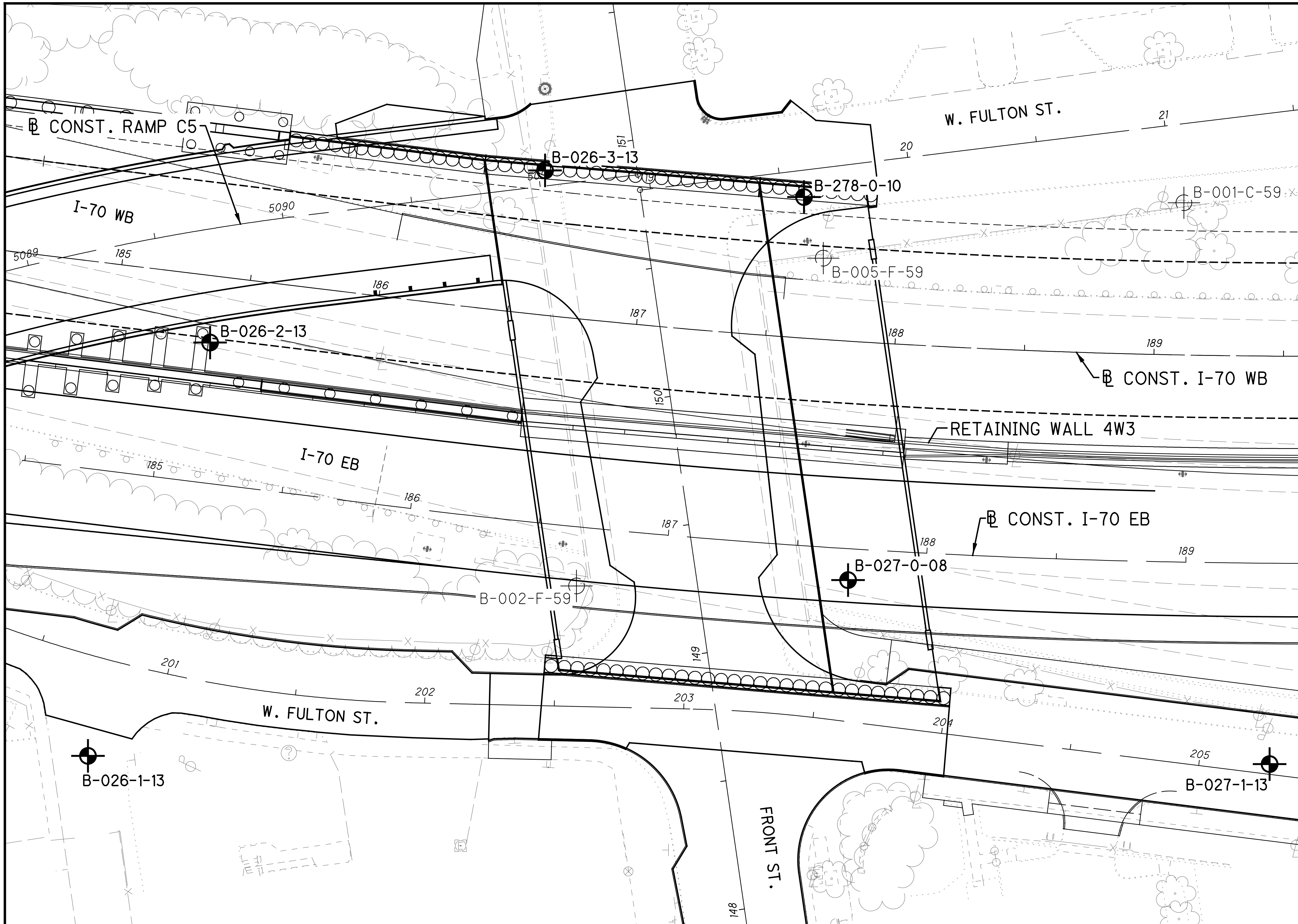
The conclusions and recommendations herein have been based upon the available soil and bedrock information and the design details furnished by a representative of the owner of the proposed project. Any revision in the plans for the proposed construction from those anticipated in this report should be brought to the attention of the geotechnical engineer to determine whether any changes in the foundation or earthwork recommendations are necessary. If deviations from the noted subsurface conditions are encountered during construction, they should also be brought to the attention of the geotechnical engineer.

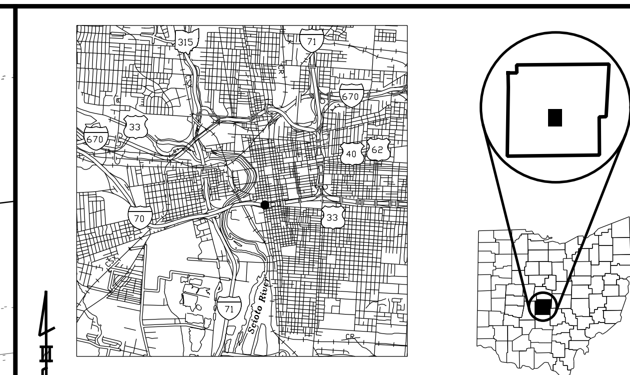
The scope of our services does not include any environmental assessment or investigation for the presence or absence of hazardous or toxic materials in the soil, groundwater or surface water within or beyond the site studied. Any statements in this report or on the test boring logs regarding odors, staining of soils or other unusual conditions observed are strictly for the information of our client.

Our professional services have been performed, our findings obtained and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices. Resource International is not responsible for the conclusions, opinions or recommendations made by others based upon the data included.

APPENDIX I

VICINITY MAP AND BORING PLAN





FRANKLIN COUNTY
VICINITY MAP

BORING PLAN
FRA-70-1395C
FRANKLIN COUNTY, OHIO

RII PROJECT NO. W-13-045		DRAWN RRM
SCALE: 1"=40' (0, 20, 40)	N	REVIEWED BRT
		DATE 7-3-18

RESOURCE INTERNATIONAL, INC.

APPENDIX II

DESCRIPTION OF SOIL TERMS

DESCRIPTION OF SOIL TERMS

The following terminology was used to describe soils throughout this report and is generally adapted from ASTM 2487/2488 and ODOT Specifications for Geotechnical Explorations.

Granular Soils – ODOT A-1, A-2, A-3, A-4 (non-plastic)
The relative compactness of granular soils is described as:

Description	Blows per foot – SPT (N_{60})		
Very Loose	Below		5
Loose	5	-	10
Medium Dense	11	-	30
Dense	31	-	50
Very Dense	Over		50

Cohesive Soils – ODOT A-4, A-5, A-6, A-7, A-8
The relative consistency of cohesive soils is described as:

Description	Unconfined Compression (tsf)		
Very Soft	Less than		0.25
Soft	0.25	-	0.5
Medium Stiff	0.5	-	1.0
Stiff	1.0	-	2.0
Very Stiff	2.0	-	4.0
Hard	Over		4.0

Gradation - The following size-related denominations are used to describe soils:

Soil Fraction		Size
Boulders		Larger than 12"
Cobbles		12" to 3"
Gravel	coarse	3" to ¾"
	fine	¾" to 2.0 mm (¾" to #10 Sieve)
Sand	coarse	2.0 mm to 0.42 mm (#10 to #40 Sieve)
	fine	0.42 mm to 0.074 mm (#40 to #200 Sieve)
Silt		0.074 mm to 0.005 mm (#200 to 0.005 mm)
Clay		Smaller than 0.005 mm

Modifiers of Components - The following modifiers indicate the range of percentages of the minor soil components:

Term	Range		
Trace	0%	-	10%
Little	10%	-	20%
Some	20%	-	35%
And	35%	-	50%

Moisture Table - The following moisture-related denominations are used to describe cohesive soils:

Term	Range - ODOT
Dry	Well below Plastic Limit
Damp	Below Plastic Limit
Moist	Above PL to 3% below LL
Wet	3% below LL to above LL

Organic Content – The following terms are used to describe organic soils:

Term	Organic Content (%)
Slightly organic	2-4
Moderately organic	4-10
Highly organic	>10

Bedrock – The following terms are used to describe the relative strength of bedrock:

Description	Field Parameter
Very Weak	Can be carved with knife and scratched by fingernail. Pieces 1 in. thick can be broken by finger pressure.
Weak	Can be grooved or gouged with knife readily. Small, thin pieces can be broken by finger pressure.
Slightly Strong	Can be grooved or gouged 0.05 in deep with knife. 1 in. size pieces from hard blows of geologist hammer.
Moderately Strong	Can be scratched with knife or pick. 1/4 in. size grooves or gouges from blows of geologist hammer.
Strong	Can be scratched with knife or pick with difficulty. Hard hammer blows to detach hand specimen.
Very Strong	Cannot be scratched by knife or pick. Hard repeated blows of geologist hammer to detach hand specimen.
Extremely Strong	Cannot be scratched by knife or pick. Hard repeated blows of geologist hammer to chip hand specimen.

CLASSIFICATION OF SOILS

Ohio Department of Transportation

(The classification of a soil is found by proceeding from top to bottom of the chart. The first classification that the test data fits is the correct classification.)

SYMBOL	DESCRIPTION	Classifcation AASHTO	Classifcation OHIO	$LL_O/LL \times 100$*	% Pass #40	% Pass #200	Liquid Limit (LL)	Plastic Index (PI)	Group Index Max.	REMARKS
	Gravel and/or Stone Fragments	A-1-a			30 Max.	15 Max.		6 Max.	0	Min. of 50% combined gravel, cobble and boulder sizes
	Gravel and/or Stone Fragments with Sand	A-1-b			50 Max.	25 Max.		6 Max.	0	
F S	Fine Sand	A-3			51 Min.	10 Max.	NON-PLASTIC		0	
	Coarse and Fine Sand	--	A-3a			35 Max.		6 Max.	0	Min. of 50% combined coarse and fine sand sizes
	Gravel and/or Stone Fragments with Sand and Silt	A-2-4				35 Max.	40 Max.	10 Max.	0	
		A-2-5					41 Min.			
	Gravel and/or Stone Fragments with Sand, Silt and Clay	A-2-6				35 Max.	40 Max.	11 Min.	4	
		A-2-7					41 Min.			
	Sandy Silt	A-4	A-4a	76 Min.		36 Min.	40 Max.	10 Max.	8	Less than 50% silt sizes
	Silt	A-4	A-4b	76 Min.		50 Min.	40 Max.	10 Max.	8	50% or more silt sizes
	Elastic Silt and Clay	A-5		76 Min.		36 Min.	41 Min.	10 Max.	12	
	Silt and Clay	A-6	A-6a	76 Min.		36 Min.	40 Max.	11 - 15	10	
	Silty Clay	A-6	A-6b	76 Min.		36 Min.	40 Max.	16 Min.	16	
	Elastic Clay	A-7-5		76 Min.		36 Min.	41 Min.	≦LL-30	20	
	Clay	A-7-6		76 Min.		36 Min.	41 Min.	>LL-30	20	
	Organic Silt	A-8	A-8a	75 Max.		36 Min.				W/o organics would classify as A-4a or A-4b
	Organic Clay	A-8	A-8b	75 Max.		36 Min.				W/o organics would classify as A-5, A-6a, A-6b, A-7-5 or A-7-6

MATERIAL CLASSIFIED BY VISUAL INSPECTION

- Sod and Topsoil
- Pavement or Base
- Uncontrolled Fill (Describe)
- Bouldery Zone
- P Peat

* Only perform the oven-dried liquid limit test and this calculation if organic material is present in the sample.

APPENDIX III

PROJECT BORING LOGS:

B-026-3-13

BORING LOGS

Definitions of Abbreviations

AS	=	Auger sample
GI	=	Group index as determined from the Ohio Department of Transportation classification system
HP	=	Unconfined compressive strength as determined by a hand penetrometer (tons per square foot)
LL_o	=	Oven-dried liquid limit as determined by ASTM D4318. Per ASTM D2487, if LL_o/LL is less than 75 percent, soil is classified as "organic".
LOI	=	Percent organic content (by weight) as determined by ASTM D2974 (loss on ignition test)
PID	=	Photo-ionization detector reading (parts per million)
QR	=	Unconfined compressive strength of intact rock core sample as determined by ASTM D2938 (pounds per square inch)
QU	=	Unconfined compressive strength of soil sample as determined by ASTM D2166 (pounds per square foot)
RC	=	Rock core sample
REC	=	Ratio of total length of recovered soil or rock to the total sample length, expressed as a percentage
RQD	=	Rock quality designation – estimate of the degree of jointing or fracture in a rock mass, expressed as a percentage:

$$\frac{\sum \text{segments equal to or longer than 4.0 inches}}{\text{core run length}} \times 100$$

S	=	Sulfate content (parts per million)
SPT	=	Standard penetration test blow counts, per ASTM D1586. Driving resistance recorded in terms of blows per 6-inch interval while letting a 140-pound hammer free fall 30 inches to drive a 2-inch outer diameter (O.D.) split spoon sampler a total of 18 inches. The second and third intervals are added to obtain the number of blows per foot (N_m).
N_{60}	=	Measured blow counts corrected to an equivalent (60 percent) energy ratio (ER) by the following equation: $N_{60} = N_m*(ER/60)$
SS	=	Split spoon sample
2S	=	For instances of no recovery from standard SS interval, a 2.5 inch O.D. split spoon is driven the full length of the standard SS interval plus an additional 6.0 inches to obtain a representative sample. Only the final 6.0 inches of sample is retained. Blow counts from 2S sampling are not correlated with N_{60} values.
3S	=	Same as 2S, but using a 3.0 inch O.D. split spoon sampler.
TR	=	Top of rock
W	=	Initial water level measured during drilling
▼	=	Water level measured at completion of drilling

Classification Test Data

Gradation (as defined on Description of Soil Terms):

GR	=	% Gravel
SA	=	% Sand
SI	=	% Silt
CL	=	% Clay

Atterberg Limits:

LL	=	Liquid limit
PL	=	Plastic limit
PI	=	Plasticity Index
WC	=	Water content (%)

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-12.68 - PHASE 4A	DRILLING FIRM / OPERATOR: RII / S.M.	DRILL RIG: CME-750 (SN 98048)	STATION / OFFSET: 5091+04.93 / 11.5' LT	EXPLORATION ID **B-026-3-13**
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / K.S.	HAMMER: CME AUTOMATIC	ALIGNMENT: BL RAMP C5	PAGE 1 OF 3
PID: 77372 BR ID: FRA-70-1390	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 4/26/13	ELEVATION: 756.9 (MSL) EOB: 90.0 ft.	
START: 8/21/13 END: 8/22/13	SAMPLING METHOD: SPT	ENERGY RATIO (%): 82.6	LAT / LONG: 39.953296762, -83.000848553	

MATERIAL DESCRIPTION AND NOTES	ELEV. 756.9	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
0.5' - CONCRETE (6.0")	756.4																	
0.5' - AGGREGATE BASE (6.0")	755.9																	
LOOSE, GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, DAMP.	753.9	1, 2, 3	3 / 2 / 2	6	33	SS-1	-	-	-	-	-	-	-	-	-	6	A-1-a (V)	
STIFF, BROWN **SILT AND CLAY**, SOME COARSE TO FINE SAND, LITTLE FINE GRAVEL, DAMP. -COBBLES PRESENT @ 5.0'	751.4	4, 5	4 / 9 / 8	23	39	SS-2	1.50	-	-	-	-	-	-	-	-	12	A-6a (V)	
LOOSE, GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, MOIST.	748.9	6, 7, 8	5 / 3 / 3	8	33	SS-3	-	-	-	-	-	-	-	-	-	8	A-1-a (V)	
SOFT, BROWN **SILTY CLAY**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST.	746.4	9, 10	WOH / 7 / 7	19	72	SS-4	0.50	8	7	10	46	29	36	19	17	23	A-6b (11)	
MEDIUM DENSE TO VERY DENSE, BROWN **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, DAMP TO MOIST.		11, 12, 13	5 / 7 / 8	21	67	SS-5	-	-	-	-	-	-	-	-	-	8	A-1-b (V)	
		14, 15	8 / 16 / 15	43	61	SS-6	-	32	39	11	15	3	19	17	2	7	A-1-b (0)	
		16, 17, 18	8 / 17 / 20	51	61	SS-7	-	-	-	-	-	-	-	-	-	6	A-1-b (V)	
		19, 20	18 / 16 / 13	40	72	SS-8	-	-	-	-	-	-	-	-	-	14	A-1-b (V)	
		21, 22, 23	18 / 12 / 14	36	83	SS-9	-	42	30	10	13	5	NP	NP	NP	8	A-1-b (0)	
		24, 25	6 / 10 / 13	32	72	SS-10	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	
		26, 27, 28	5 / 10 / 12	30	78	SS-11	-	-	-	-	-	-	-	-	-	7	A-1-b (V)	
-STONE FRAGMENTS PRESENT THROUGHOUT		29	8 / 11 / 16	37	67	SS-12	-	-	-	-	-	-	-	-	-	7	A-1-b (V)	

PID: 77372	BR ID: FRA-70-1390	PROJECT: FRA-70-12.68 - PHASE 4A	STATION / OFFSET: 5091+04.93 / 11.5 LT	START: 8/21/13	END: 8/22/13	PG 2 OF 3	B-026-3-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 726.9	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
MEDIUM DENSE TO VERY DENSE, BROWN **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, DAMP TO MOIST. *(same as above)*	724.9	31–32																
HARD, GRAY **SANDY SILT**, LITTLE CLAY, LITTLE FINE GRAVEL, DAMP.		34–35	10 22 24	63	83	SS-13	4.5+	15	11	17	38	19	21	14	7	9	A-4a (4)	
	719.9	W 36																
VERY DENSE, BROWN TO BROWNISH GRAY **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST.		39–40	11 26 26	72	83	SS-14	-	-	-	-	-	-	-	-	-	11	A-1-b (V)	
		44–45	8 29 50	109	83	SS-15	-	52	14	17	14	3	17	14	3	11	A-1-b (0)	
		49–50	10 20 28	66	56	SS-16	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	
	704.9	52																
HARD, GRAY **SANDY SILT**, LITTLE CLAY, LITTLE FINE GRAVEL, DAMP.		54–55	3 22 28	69	78	SS-17	4.5+	12	11	19	40	18	24	14	10	10	A-4a (5)	
		59–60	10 44 50/5"	-	88	SS-18	4.5+	-	-	-	-	-	-	-	-	8	A-4a (V)	
	694.9	61																

PID: 77372	BR ID: FRA-70-1390	PROJECT: FRA-70-12.68 - PHASE 4A	STATION / OFFSET: 5091+04.93 / 11.5 LT	START: 8/21/13	END: 8/22/13	PG 3 OF 3	B-026-3-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 694.8	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
VERY DENSE, GRAY **GRAVEL WITH SAND AND SILT**, TRACE CLAY, WET. *(same as above)*	689.9	64	WOH 45 50/3"	-	40	SS-19	-	-	-	-	-	-	-	-	-	20	A-2-4 (V)	
VERY DENSE, GRAY TO DARK GRAY **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, MOIST.		69	8 22 28	69	44	SS-20	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	
		74	12 23 28	70	67	SS-21	-	38	28	23	10	1	13	10	3	13	A-1-b (0)	
		79	37 50/3"	-	33	SS-22	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	
	669.9	84	10 19 28	65	56	SS-23	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	
VERY DENSE, GRAY **COARSE AND FINE SAND**, LITTLE FINE GRAVEL, LITTLE SILT, TRACE CLAY, WET.	666.9	89	12 28 50	107	67	SS-24	-	11	30	39	18	2	NP	NP	NP	10	A-3a (0)	
		EOB 90																

NOTES: GROUNDWATER INITIALLY ENCOUNTERED @ 36.0'

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PUMPED 376 LBS PORTLAND CEMENT / 100 LBS BENTONITE POWDER / 100 GAL WATER

APPENDIX IV

HISTORIC BORING LOGS:

B-002-F-59 and B-005-F-59

STATE OF OHIO
DEPARTMENT OF HIGHWAYS
TESTING LABORATORY

LOG OF BORING

CO., RT. NO., SEC. FRA-40-12.82 BRIDGE NO. FRA-40-1300
REAR ABUTMENT SOUTH INNERBELT UNDER FRONT STREET

LOCATION: T.H. 2B STA. 49+33 OFFSET 46'LT FED. NO.

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
754.7	0			
	2			
749.7	4			
	6	3/5	19858	Brown Silty Sandy Gravel
	8			
744.7	10			
	12	15/14	19859	Brown Silty Sandy Gravel
	14			
739.7	16	19/25	19860	Brown Silty Sandy Gravel
	18			
734.7	20			
	22	15/26	19861	Brown Silty Sandy Gravel
732.2	24	100/*	19862	Brown Sandy Gravel
729.7	26	18/21	19863	Brown Silty Sandy Gravel
727.2	28	38/59	19864	Brown Silty Sandy Gravel
724.7	30			
	32	34/31	19865	Brown Silty Sandy Gravel
722.2	34	42/*	19866	Brown Silty Sandy Gravel
719.7	36	21/57	19867	Gray Silty Sandy Gravel

*Refusal

BM
6.26.59

B-002-F-59

LOG OF BORING (CONTINUED) SHEET 5

BRIDGE NO. FRA-40-1300 T.H. 2 B

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
717.2	38			
		35/62	19868	Gray Sandy Gravel
714.7	40			
		60/128	19869	Gray Silty Sandy Gravel
	42			
712.2				
	44	47/70	19870	Gray Silty Sandy Gravel
709.7				
	46	45/52	19871	Gray Silty Sandy Gravel
	48			
704.7	50			
		75/108	19872	Gray Sandy Gravel
	52			
	54			
699.7	56	77/150	-----	Gray Silty Sandy Gravel
	58			
694.7	60			
		138/*	19873	Gray Silty Sandy Gravel
	62			
	64			
689.7	66	62/109	19874	Gray Silty Gravelly Sand
	68			
684.7	70			
		138/*	-----	Gray Silty Gravelly Sand
	72			
682.2				
681.7		70/*	19875	Gray Silty Sandy Gravel
				BOTTOM OF BORING
	74			
	76			
	78			*Refusal
	80			
	82			

B-005-F-59

SHEET 6

STATE OF OHIO
DEPARTMENT OF HIGHWAYS
TESTING LABORATORY

LOG OF BORING

CO., RT. NO., SEC. FRA-40-12.82 BRIDGE NO. FRA-40-1300
FORWARD ABUTMENT SOUTH INNERBELT UNDER FRONT STREET

LOCATION: T.H. 5B STA. 50+44 OFFSET 67'RT FED. NO.

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
758.3	0			
	2			
753.3	4			
	6	4/6	20608	Brown Sandy Silt
	8			
748.3	10	7/10	20609	Brown Silty Sandy Gravel
	12			
743.3	14			
	16	54/83	20610	Brown Sandy Gravel
	18			
738.3	20			
	22	58/46	20611	Brown Silty Sandy Gravel
	24			
733.3	26	44/58	20612	Brown and Gray Silty Sandy Gravel
	28			
728.3	30			
	32	48/40	20613	Gray Gravelly Sandy Silt
725.8	34	21/36	20614	Gray Sandy Silt
723.3	36	38/57	20615	Gray Sandy Silt

RM
6-26-59

B-005-F-59

SHEET 7

LOG OF BORING (CONTINUED)

BRIDGE NO. FRA-40-1300 T.H. 5 B

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
720.8	38	91/*	20616	Gray Silty Gravelly Sand
718.3	40	59/94	20617	Gray Sandy Gravel
715.8	42			
	44	100/*	20618	Gray Sandy Gravel
713.3	46	100/*	-----	Bouldery Gray Sandy Gravel
	48			
708.3	50			
		64/142	20619	Brown Silty Sandy Gravel
706.3	52	100/138	20620	Brown Silty Sandy Gravel
	54			
703.3	56	49/78	20621	Brown Silty Gravelly Sand
	58			
698.3	60			
	62	52/104	20622	Brownish-Gray Sandy Clay
	64			
693.3				
692.3	66	100/*	20623	Brown Gravelly Sandy Silt
				BOTTOM OF BORING
	68			
	70			
	72			*Refusal
	74			
	76			
	78			
	80			
	82			

APPENDIX V

DRILLED SHAFT CALCULATIONS

Boring	Proposed Top of Shaft Elevation (ft msl)	D_w (ft)	Shaft Diameter, D (ft)	Soil Class.	Material Type [1]	Stratum Depth, z (ft)	Stratum Thickness (ft)	Bottom Elevation (ft msl)	γ (pcf)	σ_v' (Midpoint) (psf)	σ_v (Bottom) (psf)	S_u [2] (psf)	N_c [3]	α [4]	N_{60} [5]	$(N_1)_{60}$ [6]	φ'_f [7]	σ_p' [8] (psf)	β [9]	Boring	Elevation (ft msl)	Shaft Length (ft)	Soil Class.	Nominal Tip Resistance, q_p [10,11] (ksf)	Nominal Side Resistance, q_s [12,13] (ksf)	φ_{qp} [14]	φ_{qs} [15]
B-026-3-13	725.8	4.9	5.0	A-4a	C	5.9	5.9	719.9	130	384	767	7,875	7.4	0.45						B-026-3-13	725.8-719.9	0.0-5.9	A-4a	58	3.54	0.40	0.45
				A-1-b	G	20.9	15.0	704.9	135	1,249	2,792				79	54	43	25,122	2.30		719.9-704.9	5.9-20.9	A-1-b	60	2.86	0.50	0.55
				A-4a	C	30.9	10.0	694.9	130	2,132	4,092	8,000	9.0	0.45							704.9-694.9	20.9-30.9	A-4a	72	3.60	0.40	0.45
				A-2-4	G	35.9	5.0	689.9	135	2,651	4,767				100	60	44	31,800	1.66		694.9-689.9	30.9-35.9	A-2-4	60	4.39	0.50	0.55
				A-1-b	G	55.9	20.0	669.9	135	3,559	7,467				76	43	43	24,168	1.10		689.9-669.9	35.9-55.9	A-1-b	60	3.89	0.50	0.55
				A-3a	G	58.9	3.0	666.9	135	4,393	7,872				100	52	43	15,792	0.71		669.9-666.9	55.9-58.9	A-3a	60	3.11	0.50	0.55
B-002-F-59	725.8	7.3	5.0	A-1-b	G	13.7	13.7	712.1	135	925	1,850				88	63	44	27,984	3.15	B-002-F-59	725.8-712.1	0.0-13.7	A-1-b	60	2.91	0.50	0.55
				A-1-a	G	34.7	21.0	691.1	135	2,212	4,685				100	64	44	31,800	1.88		712.1-691.1	13.7-34.7	A-1-a	60	4.15	0.50	0.55
				A-3a	G	44.7	10.0	681.1	135	3,338	6,035				100	58	44	15,792	0.87		691.1-681.1	34.7-44.7	A-3a	60	2.89	0.50	0.55
B-005-F-59	725.8	7.3	5.0	A-4a	C	5.1	5.1	720.7	135	344	689	8,000	7.2	0.45						B-005-F-59	725.8-720.7	0.0-5.1	A-4a	57	3.60	0.40	0.45
				A-1-a	G	16.1	11.0	709.7	135	1,225	2,174				100	67	44	31,800	2.83		720.7-709.7	5.1-16.1	A-1-a	60	3.46	0.50	0.55
				A-1-b	G	26.1	10.0	699.7	135	1,987	3,524				100	63	44	31,800	2.02		709.7-699.7	16.1-26.1	A-1-b	60	4.02	0.50	0.55
				A-6a	C	34.1	8.0	691.7	130	2,621	4,564	8,000	9.0	0.45							699.7-691.7	26.1-34.1	A-6a	72	3.60	0.40	0.45

1. C = cohesive soil stratum; G = granular soil stratum
2. S_u = average shear strength over stratum thickness (cohesive soil layers)
3. $N_C = 6[1+0.2(Z/D)] \le 9$; Ref. Section 10.8.3.5.1c, AASHTO LRFD BDS (cohesive soil layers)
4. $\alpha = 0.55$ for $S_u/P_a \le 1.5$; $\alpha = 0.55-0.1(S_u/P_a-1.5)$ for $1.5 \le S_u/P_a \le 2.5$, where P_a = 2.12 ksf = 2,120 psf; Ref. Section 10.8.3.5.1b AASHTO LRFD BDS (cohesive soil layers)
5. N_{60} = average energy corrected N-values over stratum thickness (granular soil layers)
6. $(N_1)_{60} = C_n N_{60}$, where $C_N = [0.77\log(40/\sigma_v')] \le 2.0$ ksf, where σ_v' = vetical effective stress at midpoint of soil layer with respect to the entire soil profile for the respective boring; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS (granular soil layers)
7. $\varphi'_f = 27.5+9.2\log[(N_1)_{60}]$; Ref. Section 10.8.3.5.2b, AASHTO LRFD BDS (granular soil layers)
8. $\sigma_p' = n(N_{60})^m(P_a)$, where n = 0.15 and m = 1.0 for A-1-a/1-b and A-2-4/2-6, n = 0.47 and m = 0.6 for A-3/3a, n = 0.47 and m = 0.8 for A-4a/4b soils, and P_a = 2.12 ksf = 2,120 psf; Ref. Section 10.8.3.5.2b, AASHTO LRFD BDS (granular soil layers)
9. $\beta = \tan\varphi'_f(1-\sin\varphi'_f)(\sigma_p'/\sigma_v')$^$(\sin\varphi'_f)$, where σ_v' = vetical effective stress at midpoint of soil layer; Ref. Section 10.8.3.5.2b, AASHTO LRFD BDS (granular soil layers)
10. $q_p = N_C S_u \le 80.0$ ksf; Ref. Section 10.8.3.5.1c, AASHTO LRFD BDS (cohesive soil layers)
11. $q_p = 1.2N_{60} \le 60$ ksf; Ref. Section 10.8.3.5.2c, AASHTO LRFD BDS (granular soil layers)
12. $q_s = \alpha S_u$; Ref. Section 10.8.3.5.1b, AASHTO LRFD BDS (cohesive soil layers)
13. $q_s = \beta\sigma_v'$, where σ_v' = vetical effective stress at midpoint of soil layer; Ref. Section 10.8.3.5.2b, AASHTO LRFD BDS (granular soil layers)
14. φ_{qp} = 0.50 for granular soils layers and 0.40 for cohesive soil layers; Ref. Table 10.5.5.2.4-1, AASHTO LRFD BDS
15. φ_{qs} = 0.55 for granular soils layers and 0.45 for cohesive soil layers; Ref. Table 10.5.5.2.4-1, AASHTO LRFD BDS

Calculated By: BRT Date: 8/20/2016
Checked By: JPS Date: 8/22/2016

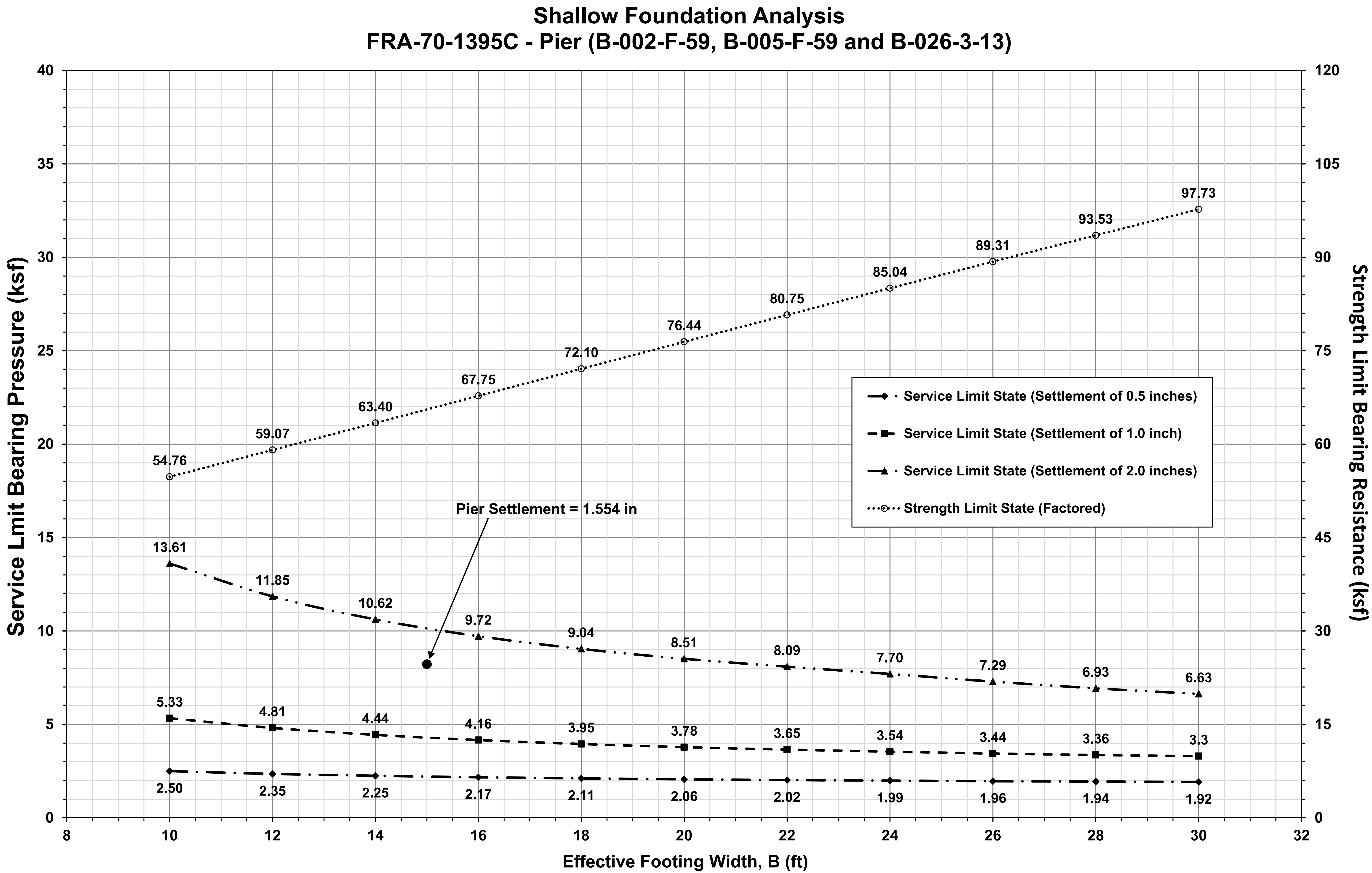
APPENDIX VI

LATERAL DESIGN PARAMETERS

Boring No.	Elevation (feet msl)	Soil Class.	Soil Type	Strata	N_{60}	$N1_{60}$	γ (pcf)	γ' (pcf)	Strength Parameter	k (soil) k_{rm} (rock)	ε_{50} (soil) E_r (rock)	RQD (rock)
B-026-3-13	756.9 to 746.4	A-6a	C	3	14	14	120 psf	120 psf	Su = 1,750 psf	585 pci	0.0067	-
	746.4 to 736.4	A-1-b	G	4	39	40	130 psf	130 psf	φ = 40°	280 pci	-	-
	736.4 to 724.9	A-1-b	G	4	34	28	130 psf	130 psf	φ = 39°	250 pci	-	-
	724.9 to 719.9	A-4a	C	3	63	63	130 psf	130 psf	Su = 7,875 psf	2,625 pci	0.0034	-
	719.9 to 704.9	A-1-b	G	4	79	54	135 psf	72.6 psf	φ = 42°	195 pci	-	-
	704.9 to 694.9	A-4a	C	2	84	84	130 psf	67.6 psf	Su = 8,000 psf	2,665 pci	0.0033	-
	694.9 to 689.9	A-2-4	G	4	100	60	135 psf	72.6 psf	φ = 41°	175 pci	-	-
	689.9 to 669.9	A-1-b	G	4	76	43	135 psf	72.6 psf	φ = 41°	175 pci	-	-
	669.9 to 666.9	A-3a	G	4	100	52	135 psf	72.6 psf	φ = 40°	155 pci	-	-
B-002-F-59	754.1 to 746.1	A-1-a	G	4	8	12	120 psf	120 psf	φ = 36°	160 pci	-	-
	746.1 to 732.1	A-1-b	G	4	38	39	130 psf	130 psf	φ = 40°	280 pci	-	-
	732.1 to 723.6	A-2-4	G	4	79	65	135 psf	135 psf	φ = 41°	315 pci	-	-
	723.6 to 712.1	A-1-b	G	4	88	63	135 psf	72.6 psf	φ = 42°	195 pci	-	-
	712.1 to 691.1	A-1-a	G	4	100	64	135 psf	72.6 psf	φ = 43°	215 pci	-	-
	691.1 to 681.1	A-3a	G	4	100	58	135 psf	72.6 psf	φ = 40°	155 pci	-	-
B-005-F-59	757.7 to 749.7	A-4a	C	3	10	10	115 psf	115 psf	Su = 1,250 psf	365 pci	0.0080	-
	749.7 to 744.7	A-1-b	G	4	17	20	125 psf	125 psf	φ = 37°	190 pci	-	-
	744.7 to 729.7	A-1-b	G	4	100	92	135 psf	135 psf	φ = 42°	355 pci	-	-
	729.7 to 720.7	A-4a	C	3	80	80	135 psf	135 psf	Su = 8,000 psf	2,665 pci	0.0033	-
	720.7 to 709.7	A-1-a	G	4	100	67	135 psf	72.6 psf	φ = 43°	215 pci	-	-
	709.7 to 699.7	A-1-b	G	4	100	63	135 psf	72.6 psf	φ = 42°	195 pci	-	-
	699.7 to 691.7	A-6a	C	2	100	100	130 psf	67.6 psf	Su = 8,000 psf	2,665 pci	0.0033	-

APPENDIX VII

BEARING RESISTANCE CHART



Shallow Foundation Analysis
FRA-70-1395C - Pier (B-002-F-59, B-005-F-59 and B-026-3-13)
Service Lmit Bearing Pressure (ksf)
Strength Limit Bearing Resistance (ksf)
Effective Footing Width, B (ft)
Pier Settlement = 1.554 in
Service Limit State (Settlement of 0.5 inches)
Service Limit State (Settlement of 1.0 inch)
Service Limit State (Settlement of 2.0 inches)
Strength Limit State (Factored)
54.76
59.07
63.40
67.75
72.10
76.44
80.75
85.04
89.31
93.53
97.73
13.61
11.85
10.62
9.72
9.04
8.51
8.09
7.70
7.29
6.93
6.63
5.33
4.81
4.44
4.16
3.95
3.78
3.65
3.54
3.44
3.36
3.3
2.50
2.35
2.25
2.17
2.11
2.06
2.02
1.99
1.96
1.94
1.92

APPENDIX VIII

SHALLOW FOUNDATION CALCULATIONS

W-13-045 - FRA-70-12.68 - FRA-70-1395C S. Front Street over I-70/71
Shallow Foundation Analysis - Pier

Calculated By: BRT Date: 7/11/2018
Checked By: JPS Date: 7/12/2018

Borings B-002-F-59, B-005-F-59 and B-026-3-13

B =	15.0	ft	Footing width
D_w =	0.0	ft	Depth below bottom of footing
q =	8,220	psf	Gross bearing pressure at bottom of wall
q_{net} =	7,020	psf	Net bearing pressure at bottom of wall (considers initial overburden stress of 1,200 psf from 10.0-foot cut to bottom of footing elevation)

Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
A-1-b	G	0.0	1.5	1.5	0.8	130	195	98	51	4,051					80	160	300	0.05	1.000	7,017	7,068	0.011	0.129
A-1-b	G	1.5	3.0	1.5	2.3	135	398	296	156	4,156					80	148	300	0.15	0.990	6,947	7,103	0.008	0.100
A-1-b	G	3.0	5.5	2.5	4.3	135	735	566	301	4,301					80	131	300	0.28	0.945	6,633	6,934	0.011	0.136
A-1-b	G	5.5	8.0	2.5	6.8	135	1,073	904	483	4,483					80	118	300	0.45	0.850	5,967	6,450	0.009	0.113
A-1-b	G	8.0	10.5	2.5	9.3	135	1,410	1,241	664	4,664					80	110	300	0.62	0.745	5,231	5,895	0.008	0.095
A-1-b	G	10.5	13.5	3.0	12.0	135	1,815	1,613	864	4,864					80	103	300	0.80	0.642	4,505	5,369	0.008	0.095
A-4a	C	13.5	18.5	5.0	16.0	130	2,465	2,140	1,142	5,142	24	0.126	0.013	0.460				1.07	0.524	3,676	4,818	0.027	0.324
A-4a	C	18.5	23.5	5.0	21.0	130	3,115	2,790	1,480	5,480	24	0.126	0.013	0.460				1.40	0.420	2,949	4,428	0.021	0.247
A-2-4	G	23.5	28.5	5.0	26.0	135	3,790	3,453	1,830	5,830					100	103	300	1.73	0.348	2,445	4,275	0.006	0.074
A-1-b	G	28.5	33.5	5.0	31.0	135	4,465	4,128	2,193	6,193					70	68	257	2.07	0.297	2,082	4,275	0.006	0.068
A-1-b	G	33.5	38.5	5.0	36.0	135	5,140	4,803	2,556	6,556					70	64	237	2.40	0.258	1,810	4,366	0.005	0.059
A-1-b	G	38.5	43.5	5.0	41.0	135	5,815	5,478	2,919	6,919					70	61	221	2.73	0.228	1,600	4,519	0.004	0.051
A-1-b	G	43.5	48.5	5.0	46.0	135	6,490	6,153	3,282	7,282					70	59	208	3.07	0.204	1,432	4,714	0.004	0.045
A-3a	G	48.5	51.5	3.0	50.0	135	6,895	6,693	3,573	7,573					100	81	257	3.33	0.188	1,321	4,894	0.002	0.019
																				Total Settlement:			1.554 in

1. $\sigma_p' = \sigma_{vo}' + \sigma_{m;}$ Estimate σ_m of 4,000 psf for moderately overconsolidated soil deposit; Ref. Table 11.2, Coduto 2003
2. $C_c = 0.009(LL-10)$; Ref. Table 6-9, FHWA GEC 5
3. $C_r = 0.15(Cc)$ for the existing fill and 0.10(Cc) for the natural soil deposits; Ref. Section 8.11, Holtz and Kovacs 1981
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_n N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index (limited to a value of 300); Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \leq \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (Cohesiv soil layers)
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)

B =	15.0	ft	
L =	148	ft	
c =	0	psf	
γ =	135	pcf	
D_f =	10.0	ft	
φ =	42	deg	
D_w =	6.5	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$$ = 145.72 ksf

$N_{cm} = N_c s_c i_c$ = 102.36

$N_{qm} = N_q s_q d_q i_q$ = 103.96

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 149.23

N_c =	93.71	s_c =	1+(15 ft/148 ft)(85.37/93.71) =	1.092	i_c =	1.000	d_q =	1+2tan(42°)[1-sin(42°)]²tan⁻¹(10 ft/15 ft) =	1.116
N_q =	85.37	s_q =	1+(15 ft/148 ft)tan(42°) =	1.091	i_q =	1.000	C_{wq} =	6.5 ft < 10.0 ft =	0.500
N_γ =	155.54	s_γ =	1-0.4(15 ft/148 ft) =	0.959	i_γ =	1.000	$C_{w\gamma}$ =	6.5 ft < 1.5(15 ft) + 10 ft =	0.500

$q_R = q_n \cdot \phi_b$ = 65.58 ksf

φ_b = 0.45