Resource International, Inc.

FRA-70-12.68 PROJECT 4R
RETAINING WALL 4W2
PID NO. 105523
FRANKLIN COUNTY, OHIO

STRUCTRURE FOUNDATION EXPLORATION REPORT

Prepared For:
GPD GROUP
1801 Watermark Drive, Suite 210
Columbus, Ohio 43215

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

January 2019

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July 13, 2018 (Revised January 30, 2019)

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
Retaining Wall 4W2
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. This report includes recommendations for the design and construction of the proposed Retaining Wall 4W2 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 preliminary structure foundation exploration or this report, please contact us.

Sincerely,

RESOURCE INTERNATIONAL, INC.

Peyman P. Majidi, E.I.
Staff Engineer

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

Enclosure: Structure Foundation Exploration Report

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EXECUTIVE SUMMARY

Resource International, Inc. (Rii) has completed a structure foundation exploration for retaining walls 4W2 as part of the FRA-70-12.68 project. Based on design information provided by GPD group, it is understood that a portion of the wall extending from bridge abutment of structure FRA-70-1395 at Sta. 187+89.65 to the angle point at Sta. 189+67.71 will consist of an 8-foot diameter tangent drilled shaft wall type. The remaining portion of the retaining wall from Sta. 189+67.71 to the bridge abutment of structure FRA-70-1405 at Sta.190+95.11 will be a cast-in-place (CIP) wall type supported on drilled shafts. It is planned to use cellular concrete as backfill between the existing wall and the new CIP wall along this portion of the wall. **Please note that the design of the drilled shaft retaining wall where it will support the north abutment of the proposed FRA-70-1395 and FRA-70-1405 structures will be governed by the recommendations in the respective bridge structure reports, which are presented under separate covers.**

Exploration and Findings

Historic borings performed in 1959 by the Department of Highways as part of the original FRA-40-12.82 project for the existing structures were obtained from the construction documents on record. Three (3) borings, designated as B-001-C-59, B-005-F-59 and B-011-0-59 were obtained in the vicinity of the existing bridge alignments, near the proposed northern abutments, and the existing retaining wall. Based on the elevation provided on the boring log, it is anticipated that the 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 ground surface at the time. The borings were extended to depths of 71.0 and 66.0 feet, respectively, below the ground surface at the time the borings were obtained.

Surface materials were not noted in the historic 1959 boring logs. In general, natural granular soils were encountered with intermittent seams of cohesive material. The granular soils were generally described as brown, brownish gray or gray gravel, silty sandy gravel, sandy gravel, sand, gravelly sand, silty gravelly sand, silty sand, sandy gravelly silt, gravelly sandy silt, and sandy silt. The cohesive soils were described as brown or brownish gray sandy gravelly clay and sandy clay. Soil described as bouldery gray sandy gravel was encountered in B-005-F-59 at about 46.0 feet below the ground surface.

Groundwater levels were not noted in the borings performed during the 1959 investigation. However, in borings B-026-3-13 and B-029-0-08, performed for the bridge structures FRA-70-1395 and FRA-70-1405, groundwater was encountered at elevations 720.9 and 721.3 feet msl, respectively.

Analyses and Recommendations

Design details of the proposed retaining wall were provided by GPD GROUP. Based on the information provided, it is understood that Retaining Wall 4W2 will consist of a tangent drilled shaft wall type between Sta. 187+89.65 and Sta. 189+67.71, and the remaining portion of the retaining wall from Sta. 189+67.71 to the bridge abutment of structure FRA-70-1405 at Sta.190+95.11 will be a cast-in-place (CIP) wall type supported on drilled shafts. It is planned to use cellular concrete as backfill between the existing wall and the new CIP wall along this portion of the wall.

It is understood that the drilled shafts in this project will be used to support cantilever retaining wall. Therefore, the lateral check of the drilled shafts will control the drilled shaft design. It is recommended that the drilled shafts be designed using the axial design parameters provided in the following table.

Drilled Shaft Axial Design Parameters

Boring	Elevation [1] (feet msl)	Shaft Length (feet)	Nominal Resistance		Resistance Factor	
			End (ksf)	Side (ksf)	End	Side
B-005-F-59	725.8 – 720.7	0.0 – 5.1	54	3.60	0.40	0.45
	720.7 – 709.7	5.1 – 16.1	60	3.20	0.50	0.55
	709.7 – 699.7	16.1 – 26.1	60	3.68	0.50	0.55
	699.7 – 691.7	26.1 – 34.1	72	3.60	0.40	0.45
B-001-C-59	725.8 – 723.4	0.0 – 2.4	60	1.00	0.50	0.55
	723.4 – 713.4	2.4 – 12.4	60	1.50	0.50	0.55
	713.4 – 705.4	12.4 – 20.4	60	2.73	0.50	0.55
	705.4 – 694.4	20.4 – 31.4	60	3.90	0.50	0.55
	694.4 – 691.4	31.4 – 34.4	60	4.23	0.50	0.55
B-011-0-59	729.2 – 721.9	0.0 – 7.3	62	3.60	0.40	0.45
	721.9 – 716.9	7.3 – 12.3	60	2.46	0.50	0.55
	716.9 – 698.9	12.3 – 30.3	60	3.94	0.50	0.55

1. *Top of shaft elevation based on structure information provided by GPD Group.*

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 report is a presentation of the structure foundation exploration performed for the design and construction of the proposed Retaining Wall 4W2 located along the north side of Interstate 70 WB, between Front Street over I-70 bridge (FRA-70-1395) and High street over I-70 bridge (FRA-70-1405), as shown on the vicinity map and boring plan presented in Appendix I. Based on design information provided by GPD group, it is understood that a portion of the wall extending from bridge abutment of structure FRA-70-1395 at Sta. 187+89.65 to the angle point at Sta. 189+67.71 will consist of an 8-foot diameter tangent drilled shaft wall type. The remaining portion of the retaining wall from Sta. 189+67.71 to the bridge abutment of structure FRA-70-1405 at Sta.190+95.11 will be a cast-in-place (CIP) wall type supported on drilled shafts. It is planned to use cellular concrete as backfill between the existing wall and the new CIP wall along this portion of the wall.

Please note that the design of the drilled shaft retaining wall where it will support the north abutment of the proposed FRA-70-1395 and FRA-70-1405 structures will be governed by the recommendations in the respective bridge structure reports, which are presented under separate covers.

Historic boring information from the 1959 investigation performed by the Ohio Department of Highways was obtained from the original construction records for the existing Front Street and High Street bridges. No additional borings were performed this structure for the current exploration. Therefore, all recommendations contained herein are based on historic information obtained from the original construction as well as preliminary studies performed within the project area by DLZ.

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 Retaining Wall 4W2 is located along the north side of I-70/71 between the S. Front Street and S. High Street overpasses, 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 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. An existing cast-in-place concrete retaining wall extends between the two bridge structures, which steps up a graded slope as it extends toward S. High Street. 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

Historic borings performed in 1959 by the Department of Highways as part of the original FRA-40-12.82 project for the existing structures were obtained from the construction documents on record. Three (3) borings, designated as B-001-C-59, B-005-F-59 and B-011-0-59 were obtained in the vicinity of the existing bridge alignments, near the proposed northern abutments, and the existing retaining wall. Based on the elevation provided on the boring log, it is anticipated that the 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 ground surface at the time. The borings were extended to depths of 71.0 and 66.0 feet, respectively, below the ground surface at the time the borings were obtained.

Rii has included a plan showing the historic soil borings performed in the project area in Appendix I.

4.0 FINDINGS

Surface materials were not noted in the historic 1959 boring logs. In general, natural granular soils were encountered with intermittent seams of cohesive material. The granular soils were generally described as brown, brownish gray or gray gravel, silty sandy gravel, sandy gravel, sand, gravelly sand, silty gravelly sand, silty sand, sandy gravelly silt, gravelly sandy silt, and sandy silt. The cohesive soils were described as brown or brownish gray sandy gravelly clay and sandy clay. Soil described as bouldery gray sandy gravel was encountered in B-005-F-59 at about 46.0 feet below the ground surface.

Groundwater levels were not noted in the borings performed during the 1959 investigation. However, in borings B-026-3-13 and B-029-0-08, performed for the bridge structures FRA-70-1395 and FRA-70-1405, groundwater was encountered at elevations 720.9 and 721.3 feet msl, respectively.

5.0 ANALYSES AND RECOMMENDATIONS

Data obtained from the review of existing geotechnical information has 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 retaining wall, 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 proposed retaining wall were provided by GPD GROUP. Based on the information provided, it is understood that Retaining Wall 4W2 will consist of a tangent drilled shaft wall type between Sta. 187+89.65 and Sta. 189+67.71, and the remaining portion of the retaining wall from Sta. 189+67.71 to the bridge abutment of structure FRA-70-1405 at Sta.190+95.11 will be a cast-in-place (CIP) wall type supported on drilled shafts. It is planned to use cellular concrete as backfill between the existing wall and the new CIP wall along this portion of the wall.

5.1 Drilled Shaft Recommendations

It is understood that the drilled shafts in this project will be used to support cantilever retaining wall. Therefore, the lateral check of the drilled shafts will control the drilled shaft design. It is recommended that the drilled shafts be designed using the axial design parameters provided in Table 1.

Table 1. Drilled Shaft Axial Design Parameters

Boring	Elevation [1] (feet msl)	Shaft Length (feet)	Nominal Resistance		Resistance Factor	
			End (ksf)	Side (ksf)	End	Side
B-005-F-59	725.8 – 720.7	0.0 – 5.1	54	3.60	0.40	0.45
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	716.9 – 698.9	12.3 – 30.3	60	3.94	0.50	0.55

1. *Top of shaft elevation based on structure information provided by GPD Group.*

Drilled shaft lengths should measure a minimum of three (3) times the shaft diameter. Per Section 10.8.3.5.3 of the 2017 AASHTO LRFD Bridge Design Specifications (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.

It is anticipated that 100 percent of the side friction resistance will be mobilized at a displacement of 1.0 percent of the diameter of the shaft, which is approximately 0.4 inches for a 3.5-foot diameter shaft. At this displacement, approximately 30 percent of the end bearing resistance will be mobilized. Therefore, if the drilled shafts are designed using a combination of side and end bearing resistance, the nominal end bearing resistance noted in Table 1 should be reduced to 30 percent of the values provided for the respective tip elevation in the determination of the design shaft resistance. Drilled shaft calculations are provided in Appendix IV.

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 2017 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 2017 AASHTO LRFD BDS. The following criteria are recommended for the group resistance of any shaft groups:

- η = 0.9 for a center-to-center spacing of 2.0 diameters,
- 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 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 factored 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 V. 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 2 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 V.

Table 2. 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:

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

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

It is understood that GPD GROUP has performed an analysis of the lateral loading on the drilled shaft elements, which were utilized to determine the shaft tip elevation provided in the Stage 2 design plans.

5.2 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 3 and Table 4.

Table 3. 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°	0.32	0.53	5.07
Medium Dense to Dense Granular Soil	130	0	32°	0.27	0.47	6.82
Very Dense Granular Soil	135	0	35°	0.24	0.43	8.56
Compacted Cohesive Engineered Fill	120	2,000	0°	N/A	N/A	N/A
Compacted Granular Engineered Fill	120	0	32°	0.27	0.47	6.82

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

Table 4. 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°	0.37	0.59	3.97
Very Stiff to Hard Cohesive Soil	125	0	28°	0.32	0.53	5.07
Loose Granular Soil	120	0	28°	0.32	0.53	5.07
Medium Dense to Dense Granular Soil	130	0	32°	0.27	0.47	6.82
Very Dense Granular Soil	135	0	35°	0.24	0.43	8.56
Compacted Cohesive Engineered Fill	120	0	30°	0.30	0.50	5.58
Compacted Granular Engineered Fill	120	0	32°	0.27	0.47	6.82

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.

5.3 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).

Fill soil placed for foundation and pavement support should be placed in loose lifts not to exceed 8.0 inches. Fill soil placed under pavement or structures shall be compacted to not less than 100 percent of maximum dry density obtained by the Standard Proctor Test (ASTM D698). Fill soil containing excess moisture shall be required to dry prior to or during compaction to a moisture content not greater than 3.0 percent above or below optimum. However, for material that displays pronounced elasticity or deformation under the action of loaded rubber tire construction equipment, the moisture content shall be reduced to optimum if necessary to secure stability. Drying of wet soil shall be expedited by the use of plows, discs, or by other approved methods when so ordered by the site geotechnical engineer.

Generally, materials utilized for engineered fill should free of waste construction debris and other deleterious materials and meet the following requirements:

- Maximum Dry Density per ASTM D698 — > 110 pcf
- Liquid Limit — < 40
- Plasticity Index — < 15
- Organic Matter — < 3 percent
- Maximum Particle Size — < 3 inches
- Silt Content (between 0.075 and 0.005 mm) — < 45 percent

Compacted granular fill shall meet the above specification and additionally shall have a maximum 35 percent passing the No. 200 sieve.

5.3.1 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, sheeting boxes 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 5. 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

5.3.2 Groundwater Considerations

Based on the groundwater observations made during drilling in borings performed nearby the proposed structure, groundwater may be encountered during construction of the drilled shafts. 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 preliminary recommendations in this report 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 our recommendations.

The recommendations for this project were developed utilizing soil and bedrock information obtained from historic and current test borings that were made at the proposed site. 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.

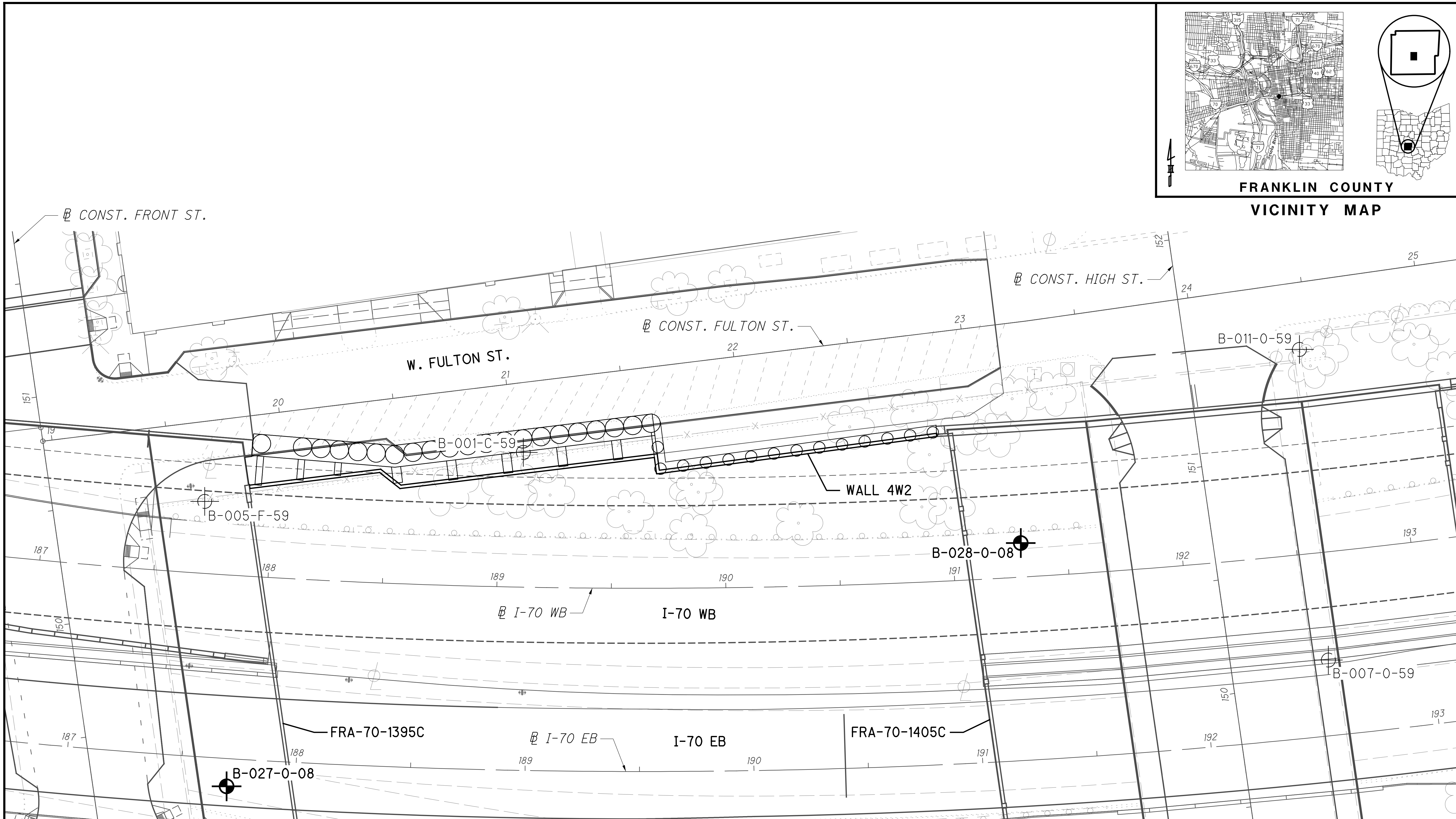
The conclusions and recommendations herein have been based upon the available soil and bedrock information and the preliminary design details furnished by a representative of the owner of the proposed project. Any revision in the plans for the proposed construction from those anticipated in this report 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



BORING PLAN

FRA-70-12.68 - RETAINING WALL 4W2

FRANKLIN COUNTY, OHIO

RII PROJECT NO. W-13-045	DRAWN RRM	RESOURCE INTERNATIONAL, INC.
SCALE: 1"=40' (0 20 40)	REVIEWED BRT	
	DATE 7-11-18	

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

HISTORIC BORING LOGS

B-001-C-59

STATE OF OHIO
DEPARTMENT OF HIGHWAYS
TESTING LABORATORY

SHEET 3

LOG OF BORING

CO., RT. NO., SEC. FRA-40-12.82 BRIDGE NO.
RETAINING WALL C RETAINING WALL C

LOCATION: T.H. 1B STA. 62+75 OFFSET 102'LT FED. NO.

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
763.0	0			
	2			
	4			
758.0	6	3/4	29107	Brown Sandy Gravelly Clay
	8			
753.0	10			
		26/32	29108	Brown Silty Sandy Gravel
	12			
	14			
748.0	16	-----		Brown Silty Sandy Gravel
	18			
743.0	20			
		96/90	29109	Brown Silty Sandy Gravel
	22			
	24			
738.0				
	26	33/47	29110	Gray Gravel
735.5	28			
		24/16		
733.0	30			
		37/36	29111	Brown Silty Sandy Gravel
	32			
	34			
728.0				
	36	27/18	29112	Brown Sandy Gravel

Glacial Boulders (igneous and limestone)

BM
8-24-59

LOG OF BORING (CONTINUED)

SHEET 4

BRIDGE NO. RETAINING WALL C T.H. 1B

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
725.5	38	34/34	29113	Brown Sandy Gravel
723.0	40	21/22	29114	Gray Silty Sand
	42			
720.5	44	40/76	29115	Gray Silty Sand
718.0	46	28/42	29116	Gray Silty Gravelly Sand
716.5	48	72/*	29117	Gray Silty Sand
713.0	50	59/36	29118	Gray Silty Sandy Gravel
	52			
711.5	54	24/39	29119	Gray Sandy Gravel
708.0	56	27/29	29120	Gray Sand
706.5	58	100*	29121	Brown Silty Sandy Gravel
703.0	60	43/53	-----	Gray Silty Sandy Gravel
	62			
	64			
698.0	66	33/*	29122	Gray Silty Gravelly Sand
	68			
693.0	70			
692.0		------	29123	Gray Sandy Gravelly Silt
	72			BOTTOM OF BORING
	74			*Refusal
	76			
	78			
	80			
	82			

Glacial Boulders (igneous and limestone)

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			
	4			
753.3				
	6	4/6	20608	Brown Sandy Silt
	8			
748.3	10	7/10	20609	Brown Silty Sandy Gravel
	12			
	14			
743.3				
	16	54/83	20610	Brown Sandy Gravel
	18			
738.3	20			
		58/46	20611	Brown Silty Sandy Gravel
	22			
	24			
733.3				
	26	44/58	20612	Brown and Gray Silty Sandy Gravel
	28			
728.3	30			
		48/40	20613	Gray Gravelly Sandy Silt
725.8	32			
	34	21/36	20614	Gray Sandy Silt
723.3				
	36	38/57	20615	Gray Sandy Silt

RM
6-26-59

B-005-F-59

LOG OF BORING (CONTINUED)

SHEET 7

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	52/104	20622	Brownish-Gray Sandy Clay
	62			
	64			
693.3				
692.3	66	100/*	20623	Brown Gravelly Sandy Silt
				BOTTOM OF BORING
	68			
	70			
	72			
	74			
	76			
	78			
	80			
	82			

*Refusal

B-011-0-59

STATE OF OHIO
DEPARTMENT OF HIGHWAYS
TESTING LABORATORY

LOG OF BORING

CO., RT. NO., SEC. FRA-40-12.82 BRIDGE NO. FRA-40-1310
FORWARD ABUTMENT SOUTH INNERBELT UNDER HIGH STREET

LOCATION: T.H. 11 STA. 51+45 OFFSET 50'RT FED. NO.

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
770.5	0			
	2			
	4			
765.5	6	7/9	20971	Brown Silty Sandy Gravel
	8			
760.5	10			
		15/30	20972	Brown Silty Gravelly Sand
758.5	12			
		7/10	20973	Brown Silty Sandy Gravel
	14			
755.5	16	11/11	20974	Gray Sandy Gravelly Silt
753.5	18	12/24	20975	Brown Silty Sandy Gravel
750.5	20			
		25/74	20976	Gray Sandy Gravel
748.5	22	29/49	20977	Brown Silty Sandy Gravel
	24			
745.5	26	15/29	20978	Brown Sandy Gravel
743.5	28	48/51	20979	Brown Silty Sandy Gravel
740.5	30			
		24/20	20980	Brown Gravelly Sand
738.5	32			
		31/73	20981	Gray Gravelly Sandy Silt
	34			
735.5	36	26/49	20982	Gray Gravelly Sandy Silt

JM
6·25·59

B-011-0-59

SHEET

LOG OF BORING (CONTINUED)

BRIDGE NO. FRA-40-1310 T.H. 11

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
	38			
730.5	40	36/75	20983	Gray Sandy Silt
	42			
	44			
725.5	46	38/76	20984	Gray Gravelly Sandy Silt
	48			
720.5	50	7/60	20985	Gray Sandy Gravel
	52			
	54			
715.5	56	88/122	20986	Gray Gravel
	58			
710.5	60	60/80	20987	Gray Gravelly Sand
	62			
	64			
705.5	66	75/159	20988	Gray Silty Sandy Gravel
	68			
700.5	70	80/*	20989	Gray Silty Sandy Gravel
699.5				BOTTOM OF BORING
	72			
	74			
	76			*Refusal
	78			
	80			
	82			

APPENDIX IV

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)	Nominal Tip Resistance, q_p [10,11] (ksf)	Nominal Side Resistance, q_s [12,13] (ksf)	φ_{qp} [14]	φ_{qs} [15]
B-005-F-59	725.8	4.8	8.0	A-4a	C	5.1	5.1	720.7	135	344	689	8,000	6.4	0.45						B-005-F-59	725.8-720.7	0.0-5.1	51	3.60	0.40	0.45
				A-1-a	G	16.1	11.0	709.7	135	1,069	2,173				100	68	43	31,800	3.00		720.7-709.7	5.1-16.1	60	3.20	0.50	0.55
				A-1-b	G	26.1	10.0	699.7	135	1,831	3,523				100	64	42	31,800	2.01		709.7-699.7	16.1-26.1	60	3.68	0.50	0.55
				A-6a	C	34.1	8.0	691.7	130	2,465	4,563	8,000	9.0	0.45							699.7-691.7	26.1-34.1	72	3.60	0.40	0.45
B-001-C-59	725.8	4.8	8.0	A-2-4	G	2.4	2.4	723.4	135	162	324				57	43	40	18,126	6.22	B-001-C-59	725.8-723.4	0.0-2.4	60	1.00	0.50	0.55
				A-3a	G	12.4	10.0	713.4	135	837	1,674				78	51	40	13,605	1.80		723.4-713.4	2.4-12.4	60	1.50	0.50	0.55
				A-1-a	G	20.4	8.0	705.4	135	1,490	2,754				71	44	42	22,578	1.84		713.4-705.4	12.4-20.4	60	2.73	0.50	0.55
				A-1-b	G	31.4	11.0	694.4	135	2,180	4,239				100	59	42	31,800	1.79		705.4-694.4	20.4-31.4	60	3.90	0.50	0.55
				A-4a	G	34.4	3.0	691.4	135	2,688	4,644				100	56	38	39,667	1.57		694.4-691.4	31.4-34.4	60	4.23	0.50	0.55
B-011-0-59	729.2	8.2	5.0	A-4a	C	7.3	7.3	721.9	135	493	986	8,000	6.9	0.45						B-011-0-59	729.2-721.9	0.0-7.3	55	3.60	0.40	0.45
				A-1-a	G	12.3	5.0	716.9	135	1,223	1,661				67	41	42	21,306	2.02		721.9-716.9	7.3-12.3	60	2.46	0.50	0.55
				A-1-a	G	30.3	18.0	698.9	135	2,058	4,091				100	57	43	31,800	1.92		716.9-698.9	12.3-30.3	60	3.94	0.50	0.55

1. C = cohesive soil stratum; G = granular soil stratum
2. $S_u = 125(N_{60}) \leq 8{,}000$ psf (cohesive soil layers)
3. $N_C = 6[1+0.2(Z/D)] \leq 9$; Ref. Section 10.8.3.5.1c, AASHTO LRFD BDS (cohesive soil layers)
4. $\alpha = 0.55$ for $S_u/P_a \leq 1.5$; $\alpha = 0.55-0.1(S_u/P_a-1.5)$ for $1.5 \leq S_u/P_a \leq 2.5$, 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 $C_N = [0.77\log(40/\sigma_v')] \leq 2.0$ 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. φ'_f estimated per Table 10.4.6.2.4-1; Ref. Section 10.4.6.2.4, 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. $q_p = N_C S_u \leq 80.0$ ksf; Ref. Section 10.8.3.5.1c, AASHTO LRFD BDS (cohesive soil layers)
11. $q_p = 1.2N_{60} \leq 60$ 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

APPENDIX V

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-005-F-59	757.7 to 749.7	A-4a	C	3	10	10	115	115	Su = 1,250 psf	365 pci	0.0080	-
	749.7 to 744.7	A-1-b	G	4	17	20	125	125	φ = 37°	190 pci	-	-
	744.7 to 729.7	A-1-b	G	4	100	92	135	135	φ = 42°	355 pci	-	-
	729.7 to 720.7	A-4a	C	3	80	80	135	135	Su = 8,000 psf	2,665 pci	0.0033	-
	720.7 to 709.7	A-1-a	G	4	100	68	135	72.6	φ = 43°	215 pci	-	-
	709.7 to 699.7	A-1-b	G	4	100	64	135	72.6	φ = 42°	195 pci	-	-
	699.7 to 691.7	A-6a	C	2	100	100	130	67.6	Su = 8,000 psf	2,665 pci	0.0033	-
B-001-C-59	762.4 to 752.4	A-6a	C	1	7	7	115	115	Su = 875 psf	165 pci	0.0095	-
	752.4 to 749.4	A-1-a	G	4	58	66	135	135	φ = 43°	395 pci	-	-
	749.4 to 735.4	A-1-a	G	4	93	86	135	135	φ = 43°	395 pci	-	-
	735.4 to 723.4	A-2-4	G	4	57	43	135	135	φ = 40°	280 pci	-	-
	723.4 to 713.4	A-3a	G	4	78	51	135	72.6	φ = 40°	155 pci	-	-
	713.4 to 705.4	A-1-a	G	4	71	44	135	72.6	φ = 42°	195 pci	-	-
	705.4 to 694.4	A-1-b	G	4	100	59	135	72.6	φ = 42°	195 pci	-	-
	694.4 to 691.4	A-4a	G	4	100	56	135	72.6	φ = 38°	125 pci	-	-
B-011-0-59	769.9 to 758.4	A-2-4	G	4	16	22	125	125	φ = 37°	190 pci	-	-
	758.4 to 753.4	A-4a	C	3	17	17	120	120	Su = 2,125 psf	710 pci	0.0062	-
	753.4 to 738.4	A-1-b	G	4	67	58	135	135	φ = 42°	355 pci	-	-
	738.4 to 721.9	A-4a	C	3	94	94	135	135	Su = 8,000 psf	2,665 pci	0.0033	-
	721.9 to 716.9	A-1-a	G	4	67	41	135	72.6	φ = 42°	195 pci	-	-
	716.9 to 698.9	A-1-a	G	4	100	57	135	72.6	φ = 43°	215 pci	-	-