Resource International, Inc.

FRA-70-12.68 PROJECT 4R
RETAINING WALL 4W1
PID NO. 105523
FRANKLIN COUNTY, OHIO

STRUCTURE FOUNDATION EXPLORATION REPORT

Prepared For:
GPD GROUP
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Prepared By:
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Rii Project No. W-13-045

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RESOURCE INTERNATIONAL, INC.
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Planning

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July 16, 2018 (Revised January 30, 2019)

Mr. Christopher W. Luzier, P.E.
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Columbus, OH 43215

Re: Structure Foundation Exploration Report
FRA-70-12.68 Project 4R
Retaining Wall 4W1
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 4W1 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

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Enclosure: Structure Foundation Exploration Report

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EXECUTIVE SUMMARY

Resource International, Inc. (Rii) has completed a structure foundation exploration for retaining walls 4W1 as part of the FRA-70-12.68 project. The proposed retaining wall will extend under the southern end of the bridge structure, along I-70/71 eastbound, to the proposed FRA-70-1405 bridge structure. It is also understood that the south abutment of the proposed FRA-70-1395 and FRA-70-1405 bridge structures will be supported on this retaining wall. The wall begins at Sta. 188+11.74 supported on tangent drilled shaft and continues east to angle point at Sta. 189+27.40, at which point the wall turn north and turns again to east supported on drilled shaft to angle point at Sta. 190+49.23. At this point the wall is founded on shallow foundation to the end of the wall designated at Sta. 191+05.91.

Exploration and Findings

On August 7, 2013, one (1) structural boring, designated as B-027-1-13, was drilled as part of the current investigation, to completion depth of 49.3 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. A preliminary geotechnical exploration was also performed within this project study area by DLZ for the FRA-70-8.93 project for the proposed trench retaining walls, and their findings were published in a report dated September 24, 2009. Two (2) borings, designated as B-027-0-08 and B-029-0-08, were performed along the east side of the bridge structure near the proposed southern abutment location. The borings were advanced to completion depths of 14.0 and 136.5 feet below the existing ground surface, respectively, and SPT sampling was performed at a maximum of 5.0 foot intervals to obtain representative soil samples for laboratory classification testing.

Boring B-027-1-13 was performed within the existing roadway of W. Fulton Street on the south side of the proposed retaining wall location, and encountered 2.0 inches of asphalt overlying 8.0 inches of concrete at the existing ground surface. Boring B-029-0-08 was performed within the existing pavement of I-70/71 and encountered 7.0 inches of asphalt overlying 9.0 inches of concrete and 11.0 inches of aggregate base. Surface materials were not noted on the 1959 historic boring logs.

Possible fill material was encountered in boring B-029-0-08, extending to a depth of 6.0 feet below the ground surface. The fill materials encountered in this boring was described as brown gravel with sand (ODOT A-1-b).

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

(ODOT A-4a, A-4b, A-6a, A-6b). Cobbles were noted as present in B-027-1-13 at 8.0 feet and 41.0 feet below the ground surface and limestone fragments were noted in SS-4. Cobbles or difficult drilling were noted in B-029-0-08 between 11.5 feet and 21.0 feet, 31.0 feet and 43.5 feet below the ground surface. Heaving sands were also noted in this boring at 33.5 feet, 48.5 feet, 53.5 feet, 58.5 feet to 68.5 feet, 98.5 feet, 103.5 feet, and 113.5 feet.

Bedrock was encountered in boring B-029-0-08 at a depth of 113.5 feet below the ground surface. The bedrock consisted of dark gray, severely weathered shale overlying brownish-gray, slightly to moderately weathered limestone.

Analyses and Recommendations

Design details of the proposed retaining wall were provided by GPD GROUP. Based on the information provided, it is understood that portion of Retaining Wall 4W1 will be a tangent drilled shaft wall type, while the remaining portion will be drilled shaft and shallow foundation.

Drilled Shaft Recommendations

It is understood that the drilled shafts used in this project are to support cantilever retaining walls, therefore, the bearing depth of the shafts are controlled by the lateral analysis of the shafts. The following table should be used for calculation of the bearing capacity of the shafts.

Drilled Shaft Axial Design Parameters

Boring	Elevation [1] (feet msl)	Shaft Length (feet)	Nominal Resistance		Resistance Factor	
			End (ksf)	Side (ksf)	End	Side
B-002-F-59	727.2 – 723.6	0.0 – 3.6	60	1.52	0.50	0.55
	723.6 – 712.1	3.6 – 15.1	60	2.82	0.50	0.55
	712.1 – 691.1	15.1 – 36.1	60	4.05	0.50	0.55
	691.1 – 681.1	36.1 – 46.1	60	2.72	0.50	0.55
B-027-1-13	730.5 – 728.5	0.0 – 2.0	21	0.35	0.50	0.55
	728.5 – 723.5	2.0 – 7.0	60	2.01	0.50	0.55
	723.5 – 713.5	7.0 – 17.0	56	2.01	0.50	0.55
	713.5 – 706.2	17.0 – 24.3	60	3.93	0.50	0.55

Boring	Elevation [1] (feet msl)	Shaft Length (feet)	Nominal Resistance		Resistance Factor	
			End (ksf)	Side (ksf)	End	Side
B-029-0-08	731.4 – 727.3	0.0 – 4.1	26	2.00	0.40	0.45
	727.3 – 721.3	4.1 – 10.1	60	3.05	0.50	0.55
	721.3 – 717.3	10.1 – 14.1	60	2.06	0.50	0.55
	717.3 – 700.3	14.1 – 31.1	56	2.81	0.40	0.45
	700.3 – 695.3	31.1 – 36.1	60	2.93	0.50	0.55
	695.3 – 685.3	36.1 – 46.1	60	4.55	0.50	0.55
	685.3 – 668.5	46.1 – 62.9	60	2.89	0.50	0.55
	668.5 – 658.3	62.9 – 73.1	72	3.60	0.40	0.45
	658.3 – 642.3	73.1 – 89.1	60	4.94	0.50	0.55
	642.3 – 628.8	89.1 – 102.6	60	3.28	0.50	0.55

1. Top of shaft elevation estimated from design plans provided by GPD GROUP.

Shallow Foundation Recommendations

Based on design information provided by GPD GROUP, approximately 75 feet of the wall will be on shallow foundation starting from Sta. 190+49.23 to Sta. 191+05.90. Based on the soil conditions encountered in boring B-029-0-08, the bottom of the footing will be at 727.7 ft. msl. At this elevation, the bearing soils are anticipated to consist of hard sandy silt and very dense gravel (ODOT A-4a, A-1-a). Shallow spread foundations bearing on these competent natural soils may be proportioned for a nominal bearing resistance as follows:

- Nominal bearing resistance of q_n = 91.7 ksf at the strength limit state.
- LRFD Bearing Resistance Factor of φ = 0.55 at the strength limit state.

Proposed structural loading was provided by GPD GROUP. Based on the maximum service limit bearing pressure of 3.03 ksf, a total settlement of 0.84 inches is anticipated along the wall alignment. Additionally, the maximum factored bearing pressure of 4.47 ksf will not exceed the factored bearing resistance at the strength limit of 50.4 ksf.

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 4W1, which will extend along the south edge of I-70/71 eastbound between the FRA-70-1395 Front Street bridge and the FRA-70-1405 High Street bridge as shown on the vicinity map and boring plan presented in Appendix I. Based on information provided by GPD GROUP, it is understood that the proposed wall will consist of a tangent drilled shaft wall on the west half of the alignment, and a cast in place (CIP) wall supported on drilled shafts on the east half. It is also understood that the south abutment of the proposed FRA-70-1395 and FRA-70-1405 bridge structures will be supported on this retaining wall. The wall begins at Sta. 188+11.74 and continues east to angle point at Sta. 189+27.40, at which point the wall turns north and again to the east and transitions from the tangent drilled shafts to the CIP wall to angle point at Sta. 190+49.23. At this point the wall is founded on shallow foundation to the end of the wall designated at Sta. 191+05.91.

A preliminary structure foundation exploration was performed by DLZ for the proposed retaining walls as part of the FRA-70-8.93 Preliminary Engineering project (PID No. 77369) and their findings are presented in the report dated September 24, 2009. Historic boring information from the 1959 investigation performed by the Ohio Department of Highways was also obtained from the original construction records for the existing Front Street and High Street bridges. These preliminary engineering and historic borings were used to supplement the information obtained by Rii during the current investigation.

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. Shale bedrock over limestone was encountered in borings B-029-0-08 at an elevation of 628.8 feet msl.

2.2. Existing Conditions

The project alignment is along the I-70/71 south innerbelt, primarily along I-70 eastbound between Souder Avenue and High Street. The I-71, SR-315 and I-70 interchange is a major interchange with many entrance and exit ramps that connect the various alignments. I-70 crosses over the Scioto River just east of the I-71 and SR-315 interchange, with three existing bridges that cross the river and converge at the eastern bank into an eight-lane roadway. The roadway then reduces to a six-lane expressway which continues into downtown Columbus and crosses under Front Street and High Street. The existing I-70 is elevated from the surrounding terrain from east of the Scioto River to just west of Front Street and there are existing overpass bridges where the roadway crosses the existing CSX and Norfolk Southern Railroads and Short Street. The roadway profile is lowered from the surrounding terrain where the alignment enters into downtown from just west of Front Street to the end of the project alignment. There is also an entrance ramp from Mound Street to I-70 westbound and an exit ramp from I-70 eastbound to Fulton Street and Livingston Avenue, which is where the existing eight-lane alignment transitions to six lanes. The daily traffic volume along the project alignment is very high. The alignment traverses primarily commercial and government properties. The surrounding terrain across the site is relatively flat-lying, with general slope toward the Scioto River.

3.0 EXPLORATION

On August 7, 2013, one (1) structural boring, designated as B-027-1-13, was drilled as part of the current investigation, to completion depth of 49.3 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. A preliminary geotechnical exploration was also performed within this project study area by DLZ for the FRA-70-8.93 project for the proposed trench retaining walls, and their findings were published in a report dated September 24, 2009. Two (2) borings, designated as B-027-0-08 and B-029-0-08, were performed along the east side of the bridge structure near the proposed southern abutment location. The borings were advanced to completion depths of 14.0 and 136.5 feet below the existing ground surface, respectively, and SPT sampling was performed at a maximum of 5.0-foot intervals to obtain representative soil samples for laboratory classification testing.

Table 1. Test Boring Summary

Boring Number	Reference Alignment	Station	Offset	Latitude	Longitude	Ground Elevation (feet msl)	Boring Depth (feet)
B-027-0-08	BL I-70 EB	187+70.27	12.1' Rt.	39.952864	-83.000428	735.9	14.0
B-027-1-13	BL I-70 EB	189+32.64	78.7' Rt.	39.952672	-82.999847	755.5	49.3
B-029-0-08	BL I-70 EB	191+53.21	46.3' Rt.	39.952780	-82.999049	742.3	136.5

The location of boring B-027-1-13 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.

Boring B-027-1-13 was performed with an all-terrain-vehicle (ATV) mounted rotary drilling machine utilizing a 3.25-inch ID, continuous HSA to advance the hole. Standard penetration testing (SPT) and split-spoon sampling were performed in boring B-027-1-13 was sampled at 2.5-foot increments of depth to 20 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. The hammer for the CME drill rig used by DLZ has a drill rod energy ratio of 61.2 percent. No calibration date was provided on the boring logs. No calibration factor was applied to the blow counts presented on the historic boring logs, as these were performed using a manual hammer.

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	14
Plastic and Liquid Limits	AASHTO T89, T90	4
Gradation – Sieve/Hydrometer	AASHTO T88	4

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 borings 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 Front Street bridge structure were obtained from the construction documents on record. One (1) boring, designated as B-002-F-59, was obtained along the west side of the existing bridge alignment, near the west end of the Retaining Wall 4W1 alignment. Based on the elevation provided on the boring log, it is anticipated that the boring was 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 boring was extended to a depth of 73.0 feet below the ground surface at the time the boring was obtained.

Rii has included a plan showing the current, historic and preliminary engineering soil borings performed in the project area in Appendix I.

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-027-1-13 was performed within the existing roadway of W. Fulton Street on the south side of the proposed retaining wall location, and encountered 2.0 inches of asphalt overlying 8.0 inches of concrete at the existing ground surface. Borings B-027-0-08 and B-029-0-08 were performed within the existing pavement of I-70/71 and encountered 7.0 inches of asphalt overlying 11.0 and 9.0 inches of concrete, respectively, followed by 11.0 inches of aggregate base. Surface materials were not noted on the 1959 historic boring logs.

4.2. Subsurface Soils

Possible existing fill material was encountered in boring B-029-0-08, extending to a depth of 6.0 feet below the ground surface. The fill materials encountered in this boring was described as brown gravel with sand (ODOT A-1-b). in addition, a thin (0.6-foot thick) layer of brock fragments was encountered below the pavement section in boring B-027-0-08.

Beneath the surficial materials and/or fill, natural granular soils were encountered with intermittent seams of cohesive material. The granular soils were generally described as brown and gray gravel, gravel and sand, gravel with sand and silt, fine sand, coarse and fine sand, sandy silt and silt (ODOT A-1-a, A-1-b, A-2-4, A-3, A-3a, A-4a, A-4b). The cohesive materials were described as brown and gray sandy silt, silt and clay, silty clay (ODOT A-4a, A-4b, A-6a, A-6b). Cobbles were noted as present in B-027-1-13 at 8.0 feet and 41.0 feet below the ground surface and limestone fragments were noted in SS-4. Cobbles or difficult drilling were noted in B-029-0-08 between 11.5 feet and 21.0 feet, 31.0 feet and 43.5 feet below the ground surface. Heaving sands were also noted in this boring at 33.5 feet, 48.5 feet, 53.5 feet, 58.5 feet to 68.5 feet, 98.5 feet, 103.5 feet, and 113.5 feet.

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 medium dense ($11 \leq N_{60} \leq 30$ blows per foot [bpf]) to very dense ($N_{60} > 50$ bpf). Overall blow counts recorded from the SPT sampling ranged from 5 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 medium stiff ($0.5 \leq HP \leq 1.0$ tsf) to hard ($HP > 4.0$ tsf). The unconfined compressive strength of the cohesive soil samples tested, obtained from the hand penetrometer, ranged from 1.5 to over 4.5 tsf (limit of instrument).

Natural moisture contents of the soil samples tested ranged from 6 to 18 percent. The natural moisture content of the cohesive soil samples tested for plasticity index ranged from 12 percent below to 5 percent below their corresponding plastic limits. In general, the soil exhibited natural moisture contents considered to be significantly below to moderately below optimum moisture levels.

4.3. Bedrock

Bedrock was encountered in boring B-029-0-08 at a depth of 113.5 feet below the ground surface. The bedrock consisted of dark gray, severely weathered shale overlying brownish-gray, slightly to moderately weathered limestone and is presented in Table 3.

Table 3. Top of Bedrock Elevations

Boring Number	Ground Surface Elevation (feet msl)	Top of Bedrock		Top of Bedrock Core (Auger Refusal)	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-029-0-08	742.3	113.5	628.8	113.5	628.8

The cored shale bedrock was described as dark gray, moderately to highly weathered, weak, thinly laminated, calcareous, pyritic, fissile, friable, jointed, fractured, tight, and slightly rough. The cored limestone bedrock was described as brownish-gray, moderately weathered, moderately strong to strong, very thinly bedded, pyritic, cherty, moderately fractured, tight, and slightly rough.

One (1) unconfined compressive strength test was performed on the recovered bedrock core runs from this boring, with an unconfined compressive strength test result of 17,137 psi at depth from 121.2 feet to 121.5 feet beneath the existing ground surface.

Table 4. Rock Core Summary

Boring	Core No.	Depth (feet)	Recovery (%)	RQD (%)	Unconfined Compressive Strength
B-029-0-08	RC-1	116.5 to 120.5	90	0	N/A
	RC-2	120.5 to 125.2	100	88	q_r @ 121.1 = 17,137 psi
	RC-3	125.2 to 130.2	100	85	N/A
	RC-4	130.2 to 135.2	90	76	N/A
	RC-5	135.2 to 136.2	100	100	N/A

It should be noted that bedrock experiences mechanical breaks during the drilling and coring processes. Rii attempted to account for fresh, manmade breaks during tabulation of the RQD analysis. Percent recoveries of the rock cores ranged from 90% to 100%, while RQD values ranged from 0% to 100%. The rock mass quality, according to the RQD values, ranged from very poor (RQD ≤ 25%) to excellent (RQD < 90%).

4.4. Groundwater

Groundwater was encountered in the borings as presented in Table 5.

Table 5. Groundwater Levels

Boring Number	Ground Elevation (feet msl)	Initial Groundwater		Upon Completion	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-027-0-08	735.9	5.0	730.9	Dry	-
B-027-1-13	755.5	37.0	718.5	N/A [1]	-
B-029-0-08	742.3	21.0	721.3	20.5 [2]	721.8

1. *The groundwater level at completion could was not obtained.*
2. *The groundwater level at completion was measured after the rock coring process, which included the addition of water for coring.*

Groundwater was encountered initially during drilling in borings B-027-0-08, B-027-1-13 and B-029-0-08 at a depth of 5.0, 37.0 and 21.0 feet below existing grade, respectively. At the completion of drilling in boring B-027-0-08, the boring was observed to be dry. No final groundwater reading was obtained in boring B-027-1-13. A final groundwater level of 20.5 feet below existing grade was measured at the completion of drilling in b boring B-029-0-08, but it should be noted that this includes water introduced during the coring process. Groundwater levels were not noted in the borings performed during the 1959 investigation.

Please note that short-term water level readings, especially in cohesive materials, are not necessarily an accurate indication of the actual groundwater level. In addition, groundwater levels and 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 logs in Appendix III.

4.5. Historic Borings

As previously indicated, a subsurface investigation was performed in 1959 as part of the Department of as part of the original FRA-40-12.82 project for the existing Front Street bridge structure were obtained from the construction documents on record. One (1) boring, designated as B-002-F-59, was obtained along the west side of the existing bridge alignment, near the west end of the Retaining Wall 4W1 alignment. One boring, identified as B-002-F-59 from this investigation was reviewed and are referenced in this report to supplement the subsurface information obtained as part of the current investigation. The subsurface soils encountered in the borings generally consisted of granular soils comprised of loose to very dense sandy gravel and silty sandy gravel from the ground surface at approximately 754.7 feet to the termination depth at 681.8 feet below the existing ground surface. Groundwater elevations in the boreholes were not provided on the historic logs. In general, the soil strata encountered in the historic borings matched relatively closely with those encountered in the soil borings for the current investigation. A copy of the historic boring logs is provided in Appendix IV, and the historic boring locations are shown on the boring plan in Appendix I.

5.0 ANALYSES AND RECOMMENDATIONS

Data obtained from the review of existing geotechnical information has been used in conjunction with data obtained during the current investigation 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 portion of Retaining Wall 4W1 will be a tangent drilled shaft wall type, while the remaining portion will be drilled shaft and shallow foundation. Given the limited depth of boring B-027-0-08 below the bottom of the wall, this boring was not utilized in the foundation analysis for this wall.

5.1. Drilled Shaft Recommendations

It is understood that the drilled shafts used in this project are to support cantilever retaining walls, therefore, the bearing depth of the shafts are controlled by the lateral analysis of the shafts. Table 6 should be used for calculation of the bearing capacity of the shafts.

Table 6. Drilled Shaft Axial Design Parameters

Boring	Elevation [1] (feet msl)	Shaft Length (feet)	Nominal Resistance		Resistance Factor	
			End (ksf)	Side (ksf)	End	Side
B-002-F-59	727.2 – 723.6	0.0 – 3.6	60	1.52	0.50	0.55
	723.6 – 712.1	3.6 – 15.1	60	2.82	0.50	0.55
	712.1 – 691.1	15.1 – 36.1	60	4.05	0.50	0.55
	691.1 – 681.1	36.1 – 46.1	60	2.72	0.50	0.55

Boring	Elevation [1] (feet msl)	Shaft Length (feet)	Nominal Resistance		Resistance Factor	
			End (ksf)	Side (ksf)	End	Side
B-027-1-13	730.5 – 728.5	0.0 – 2.0	21	0.35	0.50	0.55
	728.5 – 723.5	2.0 – 7.0	60	2.01	0.50	0.55
	723.5 – 713.5	7.0 – 17.0	56	2.01	0.50	0.55
	713.5 – 706.2	17.0 – 24.3	60	3.93	0.50	0.55
B-029-0-08	731.4 – 727.3	0.0 – 4.1	26	2.00	0.40	0.45
	727.3 – 721.3	4.1 – 10.1	60	3.05	0.50	0.55
	721.3 – 717.3	10.1 – 14.1	60	2.06	0.50	0.55
	717.3 – 700.3	14.1 – 31.1	56	2.81	0.40	0.45
	700.3 – 695.3	31.1 – 36.1	60	2.93	0.50	0.55
	695.3 – 685.3	36.1 – 46.1	60	4.55	0.50	0.55
	685.3 – 668.5	46.1 – 62.9	60	2.89	0.50	0.55
	668.5 – 658.3	62.9 – 73.1	72	3.60	0.40	0.45
	658.3 – 642.3	73.1 – 89.1	60	4.94	0.50	0.55
	642.3 – 628.8	89.1 – 102.6	60	3.28	0.50	0.55

1. *Top of shaft elevation estimated from design plans 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 6 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 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 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 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 7 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 7. 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

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.

5.1.3. 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.
- Due to the presence of granular soils with relatively high groundwater, it is recommend mud or slurry be utilized in the shaft excavation to counterbalance the hydrostatic head at the bottom of the excavation and minimize the potential for “heave” of the soils up and into the shaft excavation.
- 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. However, based on conditions encountered in other borings performed within the corridor, cobbles and boulders were encountered throughout the very dense sand and gravel deposits. Therefore, difficult drilling conditions or boulders 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

Based on design information provided by GPD GROUP, approximately 57 feet of the wall will be on shallow foundation starting from Sta. 190+49.23 to Sta. 191+05.90. Based on the soil conditions encountered in boring B-029-0-08, the bottom of the footing will be at 727.7 ft. msl. At this elevation, the bearing soils are anticipated to consist of hard sandy silt and very dense gravel (ODOT A-4a, A-1-a). Shallow spread foundations bearing on these competent natural soils may be proportioned for a nominal bearing resistance as follows:

- Nominal bearing resistance of q_n = 91.7 ksf at the strength limit state.
- LRFD Bearing Resistance Factor of φ = 0.55 at the strength limit state.

Proposed structural loading was provided by GPD GROUP. Based on the maximum service limit bearing pressure of 3.03 ksf, a total settlement of 0.84 inches is anticipated along the wall alignment. Additionally, the maximum factored bearing pressure of 4.47 ksf will not exceed the factored bearing resistance at the strength limit of 50.4 ksf. Calculations for settlement and nominal and factored bearing resistance for the shallow spread foundation are provided in Appendix VII.

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 0.84 times the total vertical force on the base should be taken as the sliding resistance. A geotechnical resistance factor of φ_τ = 1.0 should be considered when calculating the factored shear resistance between the soil and foundation for sliding.

5.2.2. Overall (Global) Stability

A slope stability analysis was performed to check the global stability of the portion of the wall that is supported on shallow foundation at approximately Sta. 191+00. As per AASHTO LRFD BDS, safety against global stability failure shall be evaluated at the service limit state. Soil parameters utilized in external stability analyses are presented in Table 8. For the global stability condition, it was considered that the failure plane will not cross through any portion of the supported soil mass above the concrete or through the concrete footing itself.

Table 8. Shear Strength Parameters Utilized In Stability Analyses

Material Type	**Unit Weight, γ (pcf)**	**Effective Friction Angle, φ' (°)**	**Effective Cohesion, c' (psf)**	**Undrained Shear Strength, S_u (psf)**
Item 203 Granular Embankment	120	32	0	N/A
Stiff to Hard Cohesive Soils	115 to 125	26 to 32	0	1,250 to 3,875
Medium Dense to Very Dense Granular Soils	125 to 135	36 to 43	0	N/A

Per Section 11.6.2.3 of the 2017 AASHTO LRFD BDS, overall (global) stability for CIP walls not supporting structural foundations on spread footings is satisfied if the product of the factor of safety from the slope stability output multiplied by the resistance factor φ=0.75 is greater than 1.0. Therefore, global stability is satisfied when a minimum factor of safety of 1.33 is obtained. Based on the footing dimensions provided in the proposed design documents, the resulting factor of safety under drained conditions (long-term stability) was greater than 1.33. An output of the overall (global) stability of the wall is provided in Appendix VIII.

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 9 and Table 10.

Table 9. 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	1,500	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 Granular Soil	125	0	32°	0.27	0.47	6.82
Dense to Very Dense Granular Soil	130	0	36°	0.23	0.41	9.09
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 10. 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	26°	0.35	0.56	4.53
Very Stiff to Hard Cohesive Soil	125	50	28°	0.32	0.53	5.07
Loose Granular Soil	120	0	28°	0.32	0.53	5.07
Medium Dense Granular Soil	125	0	32°	0.27	0.47	6.82
Dense to Very Dense Granular Soil	130	0	36°	0.23	0.41	9.09
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.

5.4. 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).

5.1.1. 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 11. 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
Rock to 3.0' +/- below Auger Refusal	0.75 : 1.0	Above Ground Water Table and No Seepage
Stable Rock	Vertical	Above Ground Water Table and No Seepage

5.5. Groundwater Considerations

Based on the groundwater observations made during drilling, 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.

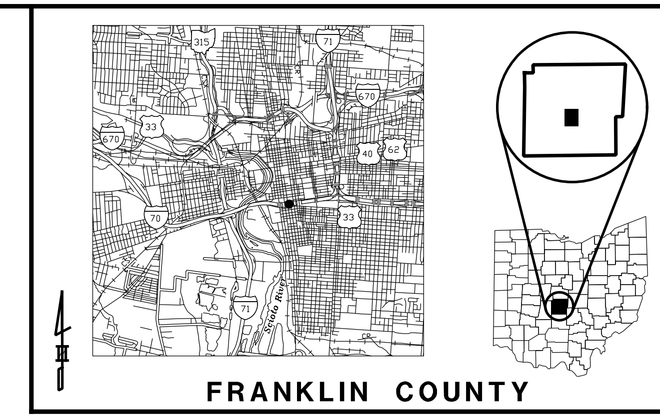
The conclusions and recommendations herein have been based upon the available soil and bedrock information and the preliminary design details furnished by a representative of the owner of the proposed project. Any revision in the plans for the proposed construction from those anticipated in this report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

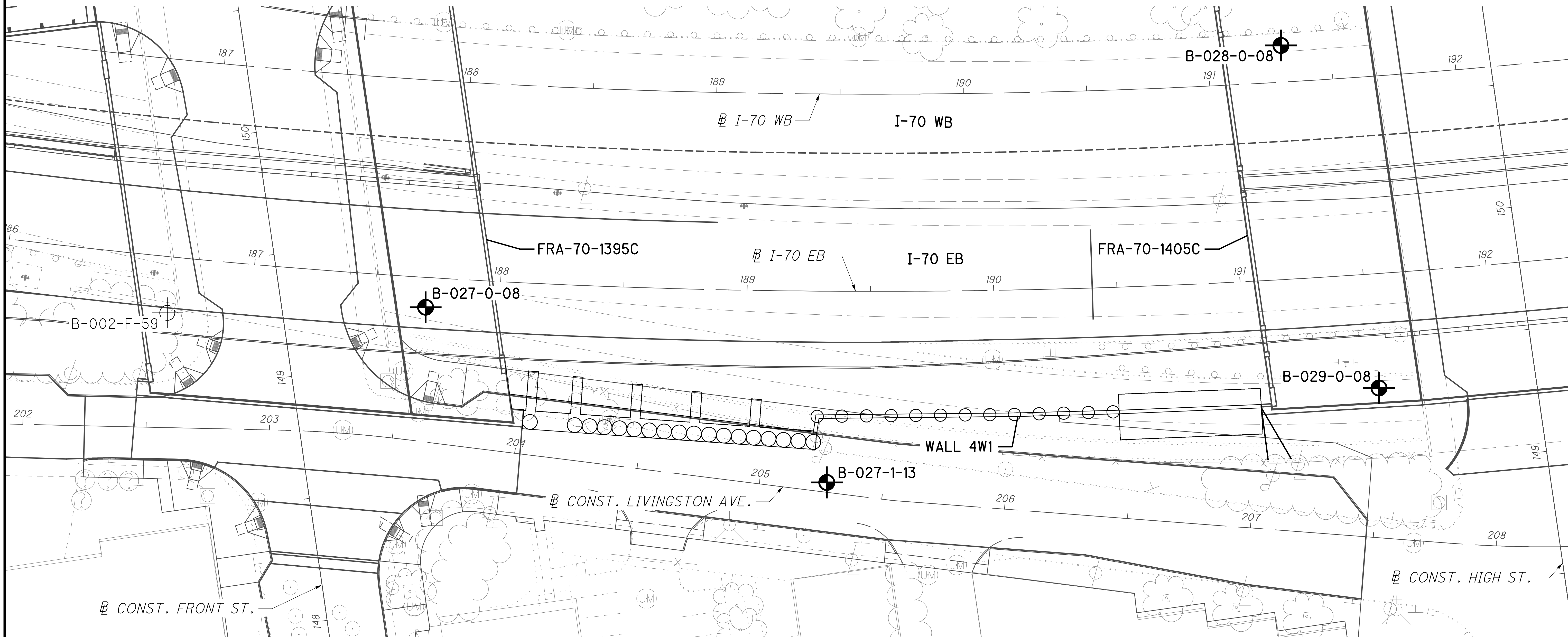
The scope of our services does not include any environmental assessment or investigation for the presence or absence of hazardous or toxic materials in the soil, groundwater or surface water within or beyond the site studied. Any statements in this report or on the test boring logs regarding odors, staining 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 4W1

FRANKLIN COUNTY, OHIO

RII PROJECT NO. W-13-045

SCALE: 1"=40'

0 20 40

DRAWN RRM

REVIEWED BRT

DATE 7-11-18

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 × 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.

DESCRIPTION OF ROCK TERMS

The following terminology was used to describe the rock throughout this report and is generally adapted from ASTM D5878 and the ODOT Specifications for Geotechnical Explorations.

Weathering – Describes the degree of weathering of the rock mass:

Description	Field Parameter
Unweathered	No evidence of any chemical or mechanical alteration of the rock mass. Mineral crystals have a right appearance with no discoloration. Fractures show little or not staining on surfaces.
Slightly Weathered	Slight discoloration of the rock surface with minor alterations along discontinuities. Less than 10% of the rock volume presents alteration.
Moderately Weathered	Portions of the rock mass are discolored as evident by a dull appearance. Surfaces may have a pitted appearance with weathering "halos" evident. Isolated zones of varying rock strengths due to alteration may be present. 10 to 15% of the rock volume presents alterations.
Highly Weathered	Entire rock mass appears discolored and dull. Some pockets of slightly to moderately weathered rock may be present and some areas of severely weathered materials may be present.
Severely Weathered	Majority of the rock mass reduced to a soil-like state with relic rock structure discernable. Zones of more resistant rock may be present but the material can generally be molded and crumbled by hand pressures.

Strength of 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.

Bedding Thickness – Description of bedding thickness as the average perpendicular distances between bedding surfaces:

Description	Thickness
Very Thick	Greater than 36 inches
Thick	18 to 36 inches
Medium	10 to 18 inches
Thin	2 to 10 inches
Very Thin	0.4 to 2 inches
Laminated	0.1 to 0.4 inches
Thinly Laminated	Less than 0.1 inches

Fracturing – Describes the degree and condition of fracturing (fault, joint, or shear):

Degree of Fracturing

Description	Spacing
Unfractured	Greater than 10 feet
Intact	3 to 10 feet
Slightly Fractured	1 to 3 feet
Moderately Fractured	

Aperture Width

Description	Width
Open	Greater than 0.2 inches
Narrow	0.05 to 0.2 inches
Tight	Less than 0.05 inches

Surface Roughness

Description	Criteria
Very Rough	Near vertical steps and ridges occur on surface
Slightly Rough	Asperities on the surfaces distinguishable
Slickensided	Surface has smooth, glassy finish, evidence of Striations

RQD – Rock Quality Designation (calculation shown in report) and Rock Quality (ODOT, GB 3, January 13, 2006):

RQD %	Rock Index Property Classification (based on RQD, not slake durability index)
0 – 25%	Very Poor
26 – 50%	Poor
51 – 70%	Fair
71 – 85%	Good
86 – 100%	Very Good

APPENDIX III

PROJECT BORING LOGS

B-027-0-08, B-027-1-13
and B-029-0-08

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 (%)

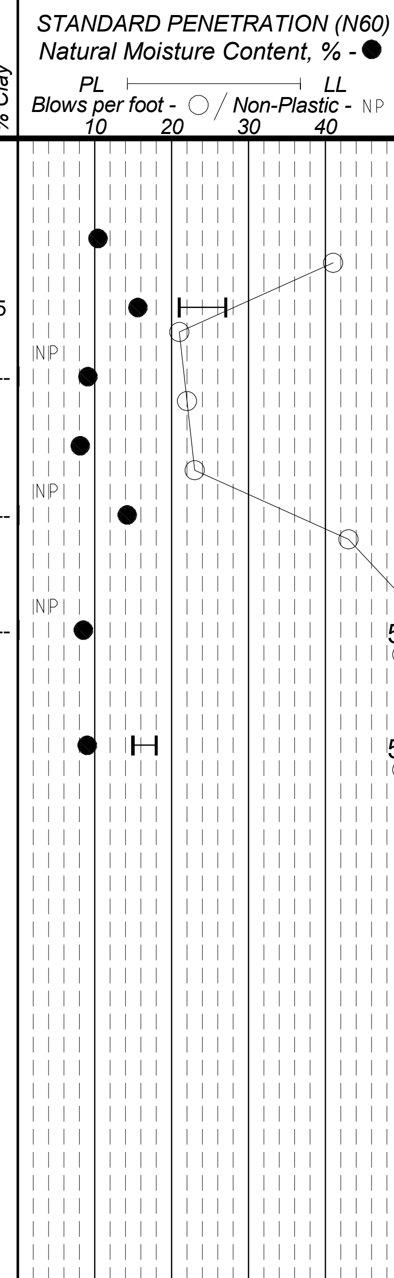
Client: ms consultants | *Project:* FRA-70-8.93 | *Job No.* 0221-1004.01

LOG OF: Boring B-027-0-08 | *Location:* Sta. 187+70.27, 12.1' RT., BL I-70 EB | *Date Drilled:* 7/8/2008

WATER OBSERVATIONS:
Water seepage at: 5.0'
Water level at completion: None

FIELD NOTES:
Advanced boring using 3.25" diameter hollowstem augers.

Depth (ft)	Elev. (ft) 735.9	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
							Asphalt Concrete Pavement - 7" Portland Cement Concrete - 11" Aggregate Base - 11"						
2.4	733.5	16 23 17	18	1									
3.0	732.9						FILL: Brick fragments, little fine to coarse sand; damp.						
		3 8 12	18	2			Medium dense to dense brown GRAVEL WITH SAND (A-1-b), little to some silt; damp.	19	34	---	25	17	5
		13 12 9	18	3				39	32	---	10	--19--	
		8 6 16	14	4									
		9 14 28	15	5			@ 7.5'-9.0', contains rust stains.	28	34	---	15	--23--	
10.0	725.9												
		13 26 29	13	6			Dense to very dense brown SANDY SILT (A-4a), trace to little gravel; moist.	17	20	---	25	--38--	
14.0	721.9	9 27 22	12	7									
							Bottom of Boring - 14.0'						



RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-12.68 - PHASE 4A	DRILLING FIRM / OPERATOR: RII / S.M.	DRILL RIG: CME-750 (SN 98048)	STATION / OFFSET: 189+32.64 / 78.7' RT		EXPLORATION ID B-027-1-13
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / K.S.	HAMMER: CME AUTOMATIC	ALIGNMENT: BL I-70 EB		
PID: 77372 BR ID: N/A	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 4/26/13	ELEVATION: 755.5 (MSL)	EOB: 49.3 ft.	PAGE 1 OF 2
START: 8/7/13 END: 8/7/13	SAMPLING METHOD: SPT	ENERGY RATIO (%): 82.6	LAT / LONG: 39.952671524, -82.999846757		

MATERIAL DESCRIPTION AND NOTES	ELEV. 755.5	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
0.2' - ASPHALT (2.0")	755.3																	
0.7' - CONCRETE (8.0")	754.7																	
VERY DENSE, BROWN **SANDY SILT**, SOME FINE GRAVEL, TRACE CLAY, DAMP.	752.5	1 – 2	12 36 18	74	33	SS-1	-	-	-	-	-	-	-	-	-	5	A-4a (V)	
MEDIUM STIFF TO STIFF, BROWN **SILT AND CLAY**, SOME COARSE TO FINE SAND, LITTLE FINE GRAVEL, DAMP TO MOIST.		4 – 5	4 2 3	7	72	SS-2	1.75	18	19	15	19	29	33	18	15	13	A-6a (4)	
-COBBLES PRESENT @ 8.0'	747.5	6 – 7	4 4 6	14	44	SS-3	1.50	-	-	-	-	-	-	-	-	17	A-6a (V)	
MEDIUM DENSE TO VERY DENSE, BROWN **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, DAMP TO MOIST. -LIMESTONE FRAGMENTS PRESENT IN SS-4		9 – 10	15 11 18	40	44	SS-4	-	-	-	-	-	-	-	-	-	6	A-1-b (V)	
		11 – 12	8 7 5	17	69	SS-5	-	-	-	-	-	-	-	-	-	14	A-1-b (V)	
		14 – 15	8 9 10	26	72	SS-6	-	52	23	9	8	8	21	18	3	7	A-1-b (0)	
		16 – 17	8 32 28	83	44	SS-7	-	-	-	-	-	-	-	-	-	6	A-1-b (V)	
		19 – 20	14 13 10	32	72	SS-8	-	-	-	-	-	-	-	-	-	7	A-1-b (V)	
		24 – 25	5 5 8	18	67	SS-9	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
	728.5																	
VERY DENSE, GRAY **GRAVEL WITH SAND AND SILT**, LITTLE CLAY, DAMP.		29	6 23 32	76	100	SS-10	-	24	25	16	16	19	20	13	7	8	A-2-4 (0)	

PID: 77372	BR ID: N/A	PROJECT: FRA-70-12.68 - PHASE 4A	STATION / OFFSET: 189+32.64 / 78.7 RT	START: 8/7/13	END: 8/7/13	PG 2 OF 2	B-027-1-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 725.5	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
VERY DENSE, GRAY **GRAVEL WITH SAND AND SILT**, LITTLE CLAY, DAMP. *(same as above)*	723.5	31–32																
DENSE TO VERY DENSE, GRAY **GRAVEL AND SAND**, TRACE CLAY, TRACE SILT, MOIST.		33																
		34–35	7 18 15	45	100	SS-11	-	-	-	-	-	-	-	-	-	13	A-1-b (V)	
		36–37 W																
		39–40	7 18 18	50	100	SS-12	-	15	35	37	6	7	NP	NP	NP	16	A-1-b (0)	
-COBBLES PRESENT @ 41.0'		41–43																
		44–45	8 49 50/5"	-	88	SS-13	-	-	-	-	-	-	-	-	-	8	A-1-b (V)	
	706.2	49 EOB	47 50/3"	-	100	SS-14	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	

NOTES: GROUNDWATER INITIALLY ENCOUNTERED @ 37.0'

ABANDONMENT METHODS, MATERIALS, QUANTITIES: COMPACTED WITH THE AUGER 100 LBS BENTONITE CHIPS AND SOIL CUTTINGS

2014 ODOT BORING LOG-RII NE BRIDGE ID - OH DOT.GDT - 3/14/15 17:35 - U:\GI8\PROJECTS\2013\W-13-045.GPJ

Client: ms consultants | **Project:** FRA-70-8.93 | **Job No.** 0221-1004.01

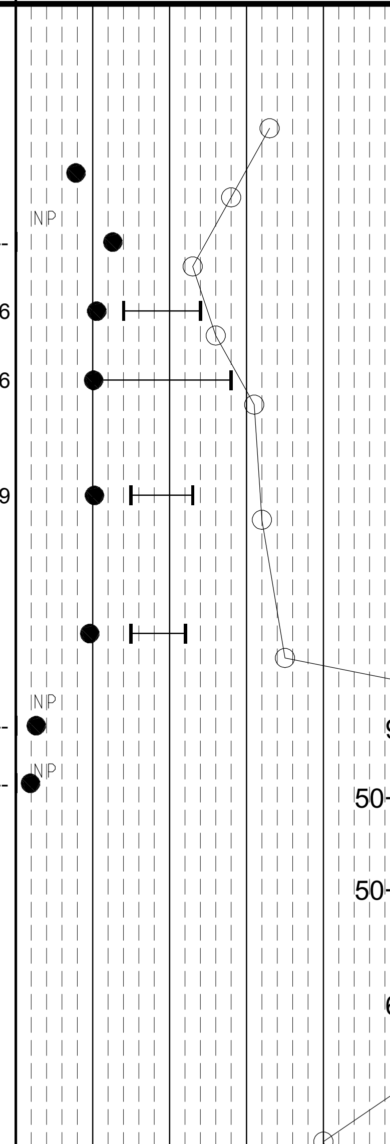
LOG OF: Boring B-029-0-08 | **Location:** Sta. 191+53.21, 46.3' RT., BL I-70 EB | **Date Drilled:** 7/9/2008 to 7/14/2008

WATER OBSERVATIONS:
Water seepage at: 21.0'-25.0',30.0'-32.0',40.0'-110.0'
Water level at completion: 29.4' (beginning of shift, 7/10/08)
20.5' (includes drilling water)

FIELD NOTES:
Advanced boring using 3.25" diameter hollowstem augers.

Depth (ft)	Elev. (ft) 742.3	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	Graphic Log	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
2.3	740.0	5 15 17	7	1			Asphalt Concrete Pavement - 7" Portland Cement Concrete - 9" Aggregate Base - 11"							
		13 15 12	10	2			POSSIBLE FILL: Medium dense to dense brown GRAVEL WITH SAND (A-1-b), little to some silt; damp.							
6.0	736.3	11 13 10	6	3					55	17	---	9	--19--	
7.5	734.8	4 10 15	14	4		4.5	Hard gray SANDY SILT (A-4a), some fine to coarse sand, little gravel; damp.		12	11	---	16	35	26
10.0	732.3	12 15 15	12	5		4.5	Hard gray SILTY CLAY (A-6b), some fine to coarse sand, trace gravel; damp to moist.		9	9	---	15	41	26
		10 14 17	18	6		4.5	Hard gray SANDY SILT (A-4a), some fine to coarse sand, trace gravel; damp. @ 11.5'-21.0', difficult drilling.		8	8	---	15	40	29
15.0	727.3	8 15 19	18	7		4.5								
		15 48 45	12	8			Very dense gray GRAVEL (A-1-a), some fine to coarse sand, trace to little silt; damp.		55	24	---	10	--11--	
		29 50/3	8	9					61	16	---	12	--11--	
21.0	721.3	50/5	1	10			@ 18.5'-18.9', rock fragments; possible cobble blocking shoe.							
23.5	718.8	21 28 33	6	11			Very dense gray GRAVEL WITH SAND AND SILT (A-2-4), some silt; wet.							
25	717.3	11 18 21	16	12			Dense gray SILT (A-4b), little fine sand; contains interbedded sand seams; wet.							

STANDARD PENETRATION (N60)
Natural Moisture Content, % - ●
PL ⊢—⊣ LL
Blows per foot - ○ / Non-Plastic - NP
10 20 30 40



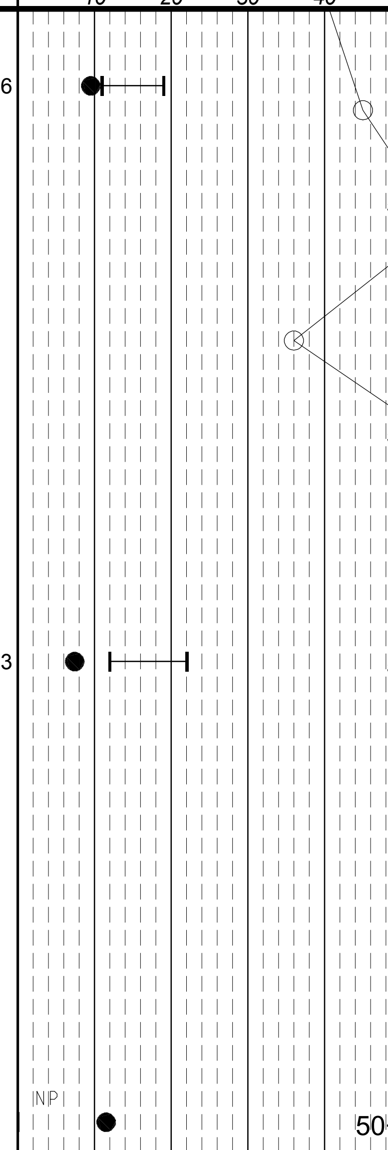
Client: ms consultants	Project: FRA-70-8.93	Job No. 0221-1004.01
LOG OF: Boring B-029-0-08	Location: Sta. 191+53.21, 46.3' RT., BL I-70 EB	Date Drilled: 7/9/2008 to 7/14/2008

WATER OBSERVATIONS:
Water seepage at: 21.0'-25.0',30.0'-32.0',40.0'-110.0'
Water level at completion: 29.4' (beginning of shift, 7/10/08)
20.5' (includes drilling water)

FIELD NOTES:
Advanced boring using 3.25" diameter hollowstem augers.

Depth (ft)	Elev. (ft) 717.3	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
		10 19 25	15	13		4.5+	Hard gray SANDY SILT (A-4a), "and" fine to coarse sand, trace gravel; contains interbedded sand seams; damp.	9	8	---	33	34	16
28.5	713.8	13 23 31	13	14			Dense to very dense gray COARSE AND FINE SAND (A-3a), some silt; moist.						
30													
31.8	710.5	3 6 29	14	15A 15B		4.5+	Hard gray SANDY SILT (A-4a), some gravel, little to some fine to coarse sand; damp. @ 31.0'-43.5', difficult drilling.						
		13 26 31	9	16		--	@ 33.5', 5 inches sand heave.						
35													
		14 23 33	14	17		4.5+		28	15	---	19	25	13
40													
42.0	700.3						Dense gray COARSE AND FINE SAND (A-3a), little silt; contains silty clay seams; wet.						
44.2	698.1	10 20 29	18	18A 18B			Dense gray SANDY SILT (A-4a), some fine to coarse sand, trace gravel; moist.						
45													
47.0	695.3						Very dense gray GRAVEL WITH SAND (A-1-b), trace silt; wet.						
		19 37 50/3	15	19			@ 48.5', 6 inches sand heave.	21	48	---	27	--4--	
50	692.3												

STANDARD PENETRATION (N60)
Natural Moisture Content, % - ●
PL ├──┤ LL
Blows per foot - ○ / Non-Plastic - NP



Client: ms consultants | *Project:* FRA-70-8.93 | *Job No.* 0221-1004.01

LOG OF: Boring B-029-0-08 | *Location:* Sta. 191+53.21, 46.3' RT., BL I-70 EB | *Date Drilled:* 7/9/2008 to 7/14/2008

WATER OBSERVATIONS:
Water seepage at: 21.0'-25.0',30.0'-32.0',40.0'-110.0'
Water level at completion: 29.4' (beginning of shift, 7/10/08) 20.5' (includes drilling water)

FIELD NOTES:
Advanced boring using 3.25" diameter hollowstem augers.

Depth (ft)	Elev. (ft) 692.3	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N60)
							Very dense gray GRAVEL WITH SAND (A-1-b), trace silt; wet.							
55		10, 16, 50/5	13	20			@ 53.5', 1.7 feet sand heave; washed out.							50+
57.0	685.3						Very dense gray COARSE AND FINE SAND (A-3a), little to some silt, trace to little gravel; wet.							
60		31, 50/5	11	21			@ 58.5'-68.5', three to six inches sand heave.							50+
65		21, 50/5	11	22				18	13	---	52	--17--		NP; 50+
70		13, 29, 41	14	23										71
73.8	668.5	25, 38, 50/6	18	24A, 24B		3.75	Very stiff gray SILT (A-4b), little fine sand; moist.							50+
75	667.3													

PL — LL; Blows per foot - ○ / Non-Plastic - NP; Natural Moisture Content, % - ●; 10 20 30 40

Client: ms consultants	*Project:* FRA-70-8.93	*Job No.* 0221-1004.01
LOG OF: Boring B-029-0-08	*Location:* Sta. 191+53.21, 46.3' RT., BL I-70 EB	*Date Drilled:* 7/9/2008 to 7/14/2008

WATER OBSERVATIONS:
Water seepage at: 21.0'-25.0',30.0'-32.0',40.0'-110.0'
Water level at completion: 29.4' (beginning of shift, 7/10/08)
20.5' (includes drilling water)

FIELD NOTES:
Advanced boring using 3.25" diameter hollowstem augers.

Depth (ft)	Elev. (ft) 667.3	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N60)
							Stiff gray SILT AND CLAY (A-6a), some to "and" fine to coarse sand, trace to little gravel; damp to moist.							
		50/4	4	25		1.5		25	20	---	18	26	11	50+
84.0	658.3	31 31 50/3	15	26A 26B		4.5+	Very dense gray GRAVEL WITH SAND (A-1-b), trace silt; wet.							50+
		15 50/4	10	27										50+
		13 26 30	18	28				31	48	---	18	--3--		57 (NP)
		41 50/2	8	29			@ 98.5', 1.2 feet sand heave.							50+
100	642.3													

STANDARD PENETRATION (N60) — Natural Moisture Content, % - ●; PL ⊢⊣ LL; Blows per foot - ○ / Non-Plastic - NP; scale 10, 20, 30, 40

Client: ms consultants | Project: FRA-70-8.93 | Job No. 0221-1004.01

LOG OF: Boring B-029-0-08 | Location: Sta. 191+53.21, 46.3' RT., BL I-70 EB | Date Drilled: 7/9/2008 to 7/14/2008

WATER OBSERVATIONS:
Water seepage at: 21.0'-25.0',30.0'-32.0',40.0'-110.0'
Water level at completion: 29.4' (beginning of shift, 7/10/08)
20.5' (includes drilling water)

FIELD NOTES:
Advanced boring using 3.25" diameter hollowstem augers.

Depth (ft)	Elev. (ft) 642.3	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N60) / Natural Moisture Content, %
							Very dense gray FINE SAND (A-3), little coarse sand, trace gravel; wet.							
105		2 12 49	18	30			@ 103.5', 2.3 feet sand heave.	3	13	---	76	--8--		NP; 62
110		41 50/0	6	31										50+
113.5	628.8						@ 113.5', 22 feet sand heave; washed out.							
115		50/2	2	32			Severely weathered gray SHALE.							50+
116.5	625.8													
120		Core 42"	Rec 38"	RQD 0%	R-1		Shale, dark gray, moderately to highly weathered, weak, thinly laminated, calcareous, pyritic, fissile, friable, jointed, fractured, tight, slightly rough; RQD 0%, Loss 10%.							
120.6	621.7													
125	617.3	Core 60"	Rec 60"	RQD 88%	R-2		Limestone, brownish-gray, moderately weathered, moderately strong to strong, very thinly bedded, pyritic, cherty, moderately fractured, tight, slightly rough, RQD 86%, Loss 0%. [Delaware Limestone] @ 121.2' - 121.5', qu = 14,137 psi							

PL —— LL; Blows per foot - ○ / Non-Plastic - NP; 10 20 30 40

Client: ms consultants	Project: FRA-70-8.93	Job No. 0221-1004.01
LOG OF: Boring B-029-0-08	Location: Sta. 191+53.21, 46.3' RT., BL I-70 EB	Date Drilled: 7/9/2008 to 7/14/2008

WATER OBSERVATIONS:
Water seepage at: 21.0'-25.0',30.0'-32.0',40.0'-110.0'
Water level at completion: 29.4' (beginning of shift, 7/10/08) 20.5' (includes drilling water)

FIELD NOTES:
Advanced boring using 3.25" diameter hollowstem augers.

Depth (ft)	Elev. (ft) 617.3	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	Graphic Log	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N60) / Natural Moisture Content, % - ● / PL — LL / Blows per foot - ○ / Non-Plastic - NP (10, 20, 30, 40)
		Core 60"	Rec 60"	RQD 85%	R-3		Limestone, brownish-gray, moderately weathered, moderately strong to strong, very thinly bedded, pyritic, cherty, moderately fractured, tight, slightly rough, RQD 86%, Loss 0%. [Delaware Limestone]								
130.2	612.1	Core 60"	Rec 54"	RQD 76%	R-4		Limestone, light gray, slightly weathered, strong, thinly bedded, slightly fossiliferous, stylolitic, slightly fractured, tight to narrow, slightly rough, RQD 82%, Loss 8%. [Columbus Limestone]								
		Core 18"	Rec 18"	RQD 100%	R-5										
136.5	605.8						Bottom of Boring - 136.5'								

Depth scale: 130, 135, 140, 145, 150

APPENDIX IV

HISTORIC BORING LOGS

B-002-F-59

B-002-F-59

SHEET 4

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			
	4			
749.7	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
712.2	42, 44	47/70	19870	Gray Silty Sandy Gravel
709.7	46	45/52	19871	Gray Silty Sandy Gravel
	48			
704.7	50, 52	75/108	19872	Gray Sandy Gravel
	54			
699.7	56	77/150	------	Gray Silty Sandy Gravel
	58			
694.7	60, 62	138/*	19873	Gray Silty Sandy Gravel
	64			
689.7	66	62/109	19874	Gray Silty Gravelly Sand
	68			
684.7	70, 72	138/*	-----	Gray Silty Gravelly Sand
682.2		70/*	19875	Gray Silty Sandy Gravel
681.7	74			BOTTOM OF BORING
	76			
	78			*Refusal
	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)	Nominal Tip Resistance, q_p [10,11] (ksf)	Nominal Side Resistance, q_s [12,13] (ksf)	φ_{qp} [14]	φ_{qs} [15]
B-002-F-59	727.2	6.2	5.0	A-2-4	G	3.6	3.6	723.6	135	243	486				79	65	41	25,122	6.27	B-002-F-59	727.2-723.6	0.0-3.6	60	1.52	0.50	0.55
				A-1-b	G	15.1	11.5	712.1	135	1,066	2,039				88	64	42	27,984	2.65		723.6-712.1	3.6-15.1	60	2.82	0.50	0.55
				A-1-a	G	36.1	21.0	691.1	135	2,245	4,874				100	65	43	31,800	1.81		712.1-691.1	15.1-36.1	60	4.05	0.50	0.55
				A-3a	G	46.1	10.0	681.1	135	3,371	6,224				100	59	40	15,792	0.81		691.1-681.1	36.1-46.1	60	2.72	0.50	0.55
B-027-1-13	730.5	12.0	5.0	A-1-b	G	2.0	2.0	728.5	125	125	250				18	15	36	5,724	2.84	B-027-1-13	730.5-728.5	0.0-2.0	21	0.35	0.50	0.55
				A-2-4	G	7.0	5.0	723.5	135	588	925				76	60	41	24,168	3.43		728.5-723.5	2.0-7.0	60	2.01	0.50	0.55
				A-1-b	G	17.0	10.0	713.5	135	1,600	2,275				47	34	40	14,946	1.26		723.5-713.5	7.0-17.0	56	2.01	0.50	0.55
				A-1-b	G	24.3	7.3	706.2	135	2,228	3,261				100	67	42	31,800	1.76		713.5-706.2	17.0-24.3	60	3.93	0.50	0.55
B-029-0-08	731.4	10.1	6.0	A-4a	C	4.1	4.1	727.3	125	256	513	3,875	6.8	0.52						B-029-0-08	731.4-727.3	0.0-4.1	26	2.00	0.40	0.45
				A-1-a	G	10.1	6.0	721.3	135	918	1,323				100	95	43	31,800	3.33		727.3-721.3	4.1-10.1	60	3.05	0.50	0.55
				A-2-4	G	14.1	4.0	717.3	135	1,468	1,863				51	45	40	16,218	1.40		721.3-717.3	10.1-14.1	60	2.06	0.50	0.55
				A-4a	C	31.1	17.0	700.3	130	2,188	4,073	6,250	9.0	0.45							717.3-700.3	14.1-31.1	56	2.81	0.40	0.45
				A-4a	G	36.1	5.0	695.3	135	2,944	4,748				50	37	36	22,783	1.00		700.3-695.3	31.1-36.1	60	2.93	0.50	0.55
				A-1-b	G	46.1	10.0	685.3	135	3,488	6,098				100	70	42	31,800	1.31		695.3-685.3	36.1-46.1	60	4.55	0.50	0.55
				A-3a	G	62.9	16.8	668.5	135	4,461	8,366				90	58	40	14,824	0.65		685.3-668.5	46.1-62.9	60	2.89	0.50	0.55
				A-6a	C	73.1	10.2	658.3	130	5,416	9,692	8,000	9.0	0.45							668.5-658.3	62.9-73.1	72	3.60	0.40	0.45
				A-1-b	G	89.1	16.0	642.3	135	6,341	11,852				86	47	41	27,348	0.78		658.3-642.3	73.1-89.1	60	4.94	0.50	0.55
				A-3	G	102.6	13.5	628.8	135	7,412	13,674				81	41	38	13,916	0.44		642.3-628.8	89.1-102.6	60	3.28	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 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-002-F-59	754.1 to 746.1	A-1-a	G	4	8	12	120	120	φ = 36°	160 pci	-	-
	746.1 to 732.1	A-1-b	G	4	38	39	130	130	φ = 40°	280 pci	-	-
	732.1 to 723.6	A-2-4	G	4	79	65	135	135	φ = 41°	315 pci	-	-
	723.6 to 712.1	A-1-b	G	4	88	64	135	72.6	φ = 42°	195 pci	-	-
	712.1 to 691.1	A-1-a	G	4	100	65	135	72.6	φ = 43°	215 pci	-	-
	691.1 to 681.1	A-3a	G	4	100	59	135	72.6	φ = 40°	155 pci	-	-
B-027-1-13	755.5 to 747.5	A-6a	C	3	10	10	115	115	Su = 1,250 psf	365 pci	0.0080	-
	747.5 to 733.5	A-1-b	G	4	29	30	130	130	φ = 39°	250 pci	-	-
	733.5 to 728.5	A-1-b	G	4	18	15	125	125	φ = 36°	160 pci	-	-
	728.5 to 723.5	A-2-4	G	4	76	60	135	135	φ = 41°	315 pci	-	-
	723.5 to 713.5	A-1-b	G	4	47	34	135	72.6	φ = 40°	155 pci	-	-
	713.5 to 706.2	A-1-b	G	4	100	67	135	72.6	φ = 42°	195 pci	-	-
B-029-0-08	742.3 to 736.3	A-1-b	G	4	28	43	130	130	φ = 41°	315 pci	-	-
	736.3 to 727.3	A-4a	C	3	31	31	125	125	Su = 3,875 psf	1,290 pci	0.0047	-
	727.3 to 721.3	A-1-a	G	4	100	95	135	135	φ = 43°	395 pci	-	-
	721.3 to 717.3	A-2-4	G	4	51	45	135	72.6	φ = 40°	155 pci	-	-
	717.3 to 700.3	A-4a	C	2	50	50	130	67.6	Su = 6,250 psf	2,085 pci	0.0039	-
	700.3 to 695.3	A-4a	G	4	50	37	135	72.6	φ = 36°	95 pci	-	-
	695.3 to 685.3	A-1-b	G	4	100	70	135	72.6	φ = 42°	195 pci	-	-
	685.3 to 668.5	A-3a	G	4	90	58	135	72.6	φ = 40°	155 pci	-	-
	668.5 to 658.3	A-6a	C	2	100	100	130	67.6	Su = 8,000 psf	2,665 pci	0.0033	-
	658.3 to 642.3	A-1-b	G	4	86	47	135	72.6	φ = 41°	175 pci	-	-
	642.3 to 628.8	A-3	G	4	81	41	135	72.6	φ = 38°	125 pci	-	-
	628.8 to 625.8	Shale	R	9	-	-	150	87.6	Qu = 200 psi	0.0005	20,000 psi	0
	625.8 to 621.7	Shale	R	9	-	-	150	87.6	Qu = 360 psi	0.0005	32,000 psi	0
	621.7 to 605.8	Limestone	R	9	-	-	165	102.6	Qu = 10,000 psi	0.00005	1,000,000 psi	84

APPENDIX VII

SHALLOW FOUNDATION CALCULATIONS

W-13-045 FRA-70-12.68
Settlement - Shallow Foundation Bearing Resistance for Retaining Wall 4W1
Boring B-029-0-08

Calculated By: PM Date: 7/13/2018
Checked By: BRT Date: 7/15/2018

B =	9.9	ft
D_w =	7.0	ft
q =	3,030	psf

Layer	Soil Class.	Soil Type	Elevation (ft. msl)		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_m [1] (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)
1	A-4a	C	727.7	726.7	0.0	1.0	1.0	0.5	125	125	63	63	4,000	4,063	23	0.117	0.012	0.567				0.05	1.000	3,029	3,091	0.013	0.152
2	A-1-a	G	726.7	724.2	1.0	3.5	2.5	2.3	135	463	294	294							100	164	300	0.23	0.968	2,934	3,227	0.009	0.104
	A-1-a	G	724.2	721.7	3.5	6.0	2.5	4.8	135	800	631	631							100	139	300	0.48	0.831	2,518	3,150	0.006	0.070
3	A-2-4	G	721.7	718.2	6.0	9.5	3.5	7.8	135	1,273	1,036	989							51	63	231	0.78	0.651	1,971	2,961	0.007	0.087
3	A-4b	C	718.2	713.7	9.5	14.0	4.5	11.8	130	1,858	1,565	1,269	4,000	5,269	19	0.081	0.008	0.500				1.19	0.482	1,459	2,728	0.008	0.097
4	A-3a	G	713.7	710.2	14.0	17.5	3.5	15.8	135	2,330	2,094	1,548							55	60	173	1.59	0.376	1,139	2,687	0.005	0.058
5	A-4a	C	710.2	705.7	17.5	22.0	4.5	19.8	130	2,915	2,623	1,827	4,000	5,827	22	0.108	0.011	0.550				1.99	0.306	929	2,755	0.006	0.067
	A-4a	C	705.7	700.7	22.0	27.0	5.0	24.5	130	3,565	3,240	2,148	4,000	6,148	22	0.108	0.011	0.550				2.47	0.250	759	2,907	0.005	0.055
6	A-3a	G	700.7	698.2	27.0	29.5	2.5	28.3	135	3,903	3,734	2,408							50	47	131	2.85	0.219	663	3,070	0.002	0.024
7	A-4a	G	698.2	695.4	29.5	32.3	2.8	30.9	135	4,281	4,092	2,600							50	46	78	3.12	0.201	608	3,208	0.003	0.039
8	A-1-b	G	695.4	690.4	32.3	37.3	5.0	34.8	135	4,956	4,618	2,883							100	88	300	3.52	0.179	541	3,425	0.001	0.015
	A-1-b	G	690.4	685.4	37.3	42.3	5.0	39.8	135	5,631	5,293	3,246							100	84	300	4.02	0.157	475	3,721	0.001	0.012
9	A-3a	G	685.4	677.0	42.3	50.7	8.4	46.5	135	6,765	6,198	3,733							90	71	217	4.70	0.135	408	4,140	0.002	0.021
	A-3a	G	677.0	668.6	50.7	59.1	8.4	54.9	135	7,899	7,332	4,343							90	67	199	5.55	0.114	346	4,689	0.001	0.017
10	A-6a	C	668.6	663.5	59.1	64.2	5.1	61.7	130	8,562	8,230	4,820	4,000	8,820	22	0.108	0.011	0.550				6.23	0.102	308	5,128	0.001	0.011
	A-6a	C	663.5	658.4	64.2	69.3	5.1	66.8	130	9,225	8,893	5,165	4,000	9,165	22	0.108	0.011	0.550				6.74	0.094	285	5,450	0.001	0.010
11	A-1-b	G	658.4	650.4	69.3	77.3	8.0	73.3	135	10,305	9,765	5,627							86	56	198	7.40	0.086	260	5,887	0.001	0.010
	A-1-b	G	650.4	642.4	77.3	85.3	8.0	81.3	135	11,385	10,845	6,208							86	54	185	8.21	0.077	234	6,442	0.001	0.008
12	A-3	G	642.4	635.7	85.3	92.0	6.7	88.7	135	12,289	11,837	6,742							81	48	112	8.95	0.071	215	6,957	0.001	0.010
	A-3	G	635.7	629.0	92.0	98.7	6.7	95.4	135	13,194	12,741	7,228							81	46	108	9.63	0.066	200	7,428	0.001	0.009
																							Total Settlement:				0.839 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 = 0.10(Cc); Ref. Section 8.11, Holtz and Kovacs 1981
4. $e_o = (C_c/0.54)+0.35$; Ref. Table 6-11, FHWA GEC 5
5. $(N1)_{60} = C_N N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \le 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing; $I = [\beta+\sin(\beta)\cos(\beta+2\delta)]/\pi$, where $\beta = \tan^{-1}[(x+B/2)/Z_f]-\delta$, $\delta = \tan^{-1}[(x-B/2)/Z_f]$ and x = horizontal distance from center of footing; Ref. Figure 6.13 and Equation 6.24, Das 2005
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \le \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \le \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (Cohesive 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)

W-13-045 - FRA-70-12.68	Calculated By: PM	Date: 7/13/2018
Shallow Foundation Bearing Resistance for Retaining Wall 4W1	Checked By: BRT	Date: 7/15/2018

Limits of the wall on shallow foundation: Sta.190+49.23 to Sta. 191+05.90 / Boring B-029-0-08

B =	8.8	ft	
L =	57	ft	
c =	0	psf	
γ_{BF} =	120	pcf	Unit weight of backfill material
γ_{BS} =	135	pcf	Unit weight of foundation soil
D_f =	6.0	ft	Bottom of footing elevation at 727.7 ft. msl.
φ =	40	deg	
D_w =	7.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma} = 91.71 \text{ ksf}$$

$N_{cm} = N_c s_c i_c$ = 85.22 $\quad$ $N_{qm} = N_q s_q d_q i_q$ = 81.81 $\quad$ $N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 102.65

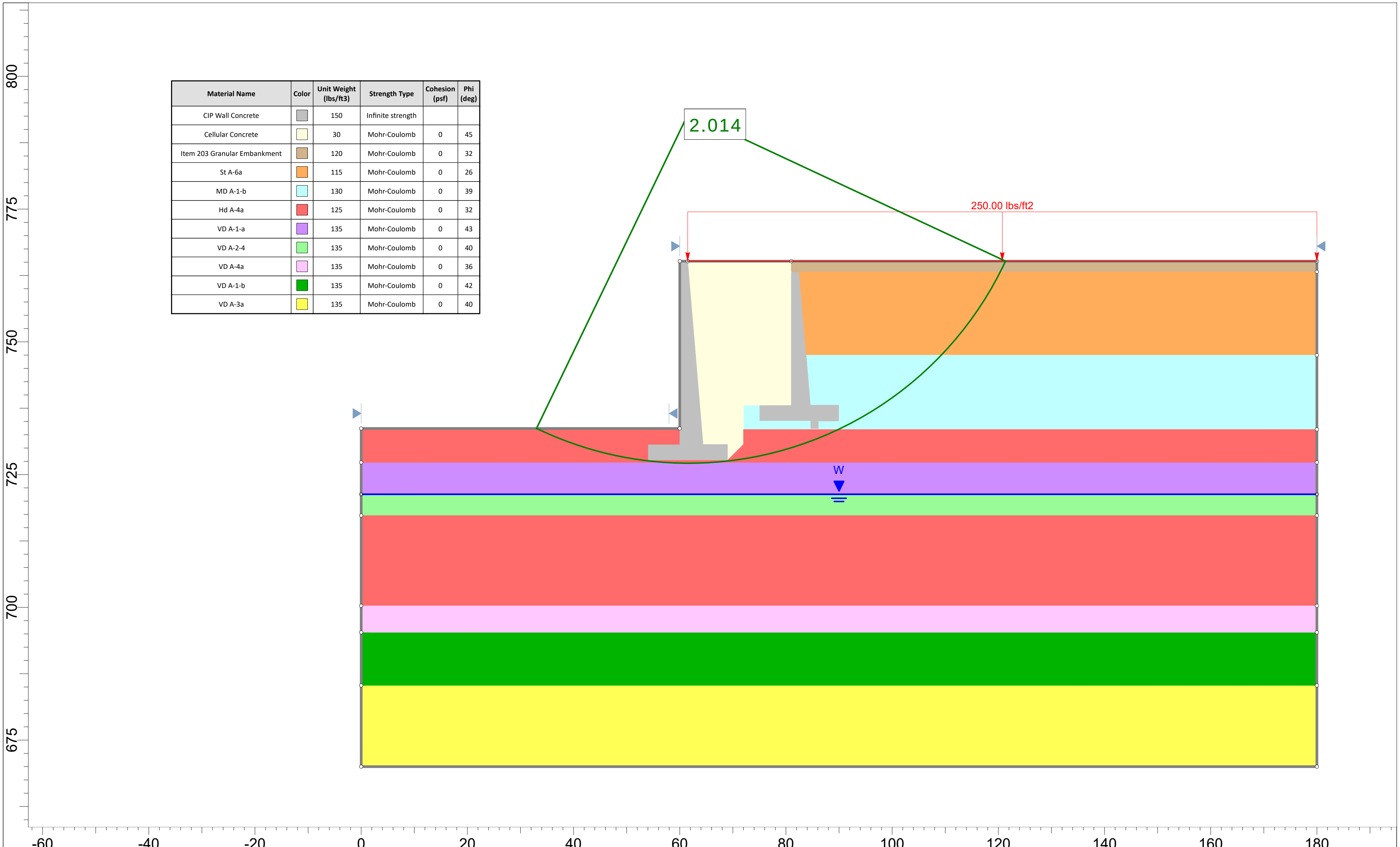
N_c =	75.31	s_c =	1.132	i_c =	1.000	d_q =	1.128
N_q =	64.20	s_q =	1.130	i_q =	1.000	C_{wq} =	1.000
N_γ =	109.41	s_γ =	0.938	i_γ =	1.000	$C_{w\gamma}$ =	0.538

$$q_R = q_n \cdot \phi_b = 50.44 \text{ ksf}$$

φ_b = 0.55

APPENDIX VIII

GLOBAL STABILITY ANALYSIS OUTPUT



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
CIP Wall Concrete		150	Infinite strength		
Cellular Concrete		30	Mohr-Coulomb	0	45
Item 203 Granular Embankment		120	Mohr-Coulomb	0	32
St A-6a		115	Mohr-Coulomb	0	26
MD A-1-b		130	Mohr-Coulomb	0	39
Hd A-4a		125	Mohr-Coulomb	0	32
VD A-1-a		135	Mohr-Coulomb	0	43
VD A-2-4		135	Mohr-Coulomb	0	40
VD A-4a		135	Mohr-Coulomb	0	36
VD A-1-b		135	Mohr-Coulomb	0	42
VD A-3a		135	Mohr-Coulomb	0	40

rocscience

Project	FRA-70-12.68 - Retaining Wall 4W1 - Global Stability		
Analysis Description	Sta. 191+00 - B-027-1-13 and B-029-0-08 - CIP Wall Type - Drained - Circular - Spencer's		
Drawn By	BRT	*Scale*	1:300
Company	Resource International, Inc.		
Date	01/31/2019	*File Name*	Retaining Wall 4W1 - Global Stability.slmd

SLIDEINTERPRET 8.020