Resource International, Inc.

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FRA-70-14.05C PROJECT 4H
FRA-70-1405C
S. HIGH STREET (US-23D) OVER I-70/71
PID NO. 105596
FRANKLIN COUNTY, OHIO

STRUCTURE FOUNDATION EXPLORATION REPORT (REV. 3)

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

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September 16, 2015 (Revised July 8, 2022)

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

Re: Structure Foundation Exploration Report (Rev. 3)
FRA-70-14.05C Project 4H
FRA-70-1405C – S. High Street (US-23D) over I-70/71
PID No. 105596
Rii Project No. W-13-045

Mr. Luzier:

Resource International, Inc. (Rii) is pleased to submit this revised structure foundation exploration report for the above referenced project. Engineering logs have been prepared and are attached to this report along with the results of laboratory testing. This report includes recommendations for the design and construction of the proposed FRA-70-1405C bridge structure carrying S. High Street (US-23D) over I-70/71 as part of the FRA-70-14.05C Project 4H in Columbus, Ohio.

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

Sincerely,

RESOURCE INTERNATIONAL, INC.

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

Jonathan P. Sterenberg, P.E.
Vice President – Geotechnical Services

Enclosure: Structure Foundation Exploration Report (Rev. 3)

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TABLE OF CONTENTS

APPENDICIES

EXECUTIVE SUMMARY

Resource International, Inc. (Rii) has completed a structure foundation exploration for the design and construction of the proposed FRA-70-1405C bridge structure carrying S. High Street (US-23D) and flanking cap structures over I-70/71. It is understood that the existing structure will be removed and replaced with a two-span continuous composite steel plate girder structure with a composite deck. The existing ramp from I-70 eastbound to 3rd Street will be eliminated and the proposed structure abutments will be shifted to the inside of the existing abutments. The three parallel structures will each react independently, with only a longitudinal expansion joint connecting them. The roadway profile grade along the I-70 eastbound beneath the structure will be cut approximately 2.0 feet below the existing roadway profile grade, and there will be no change in the profile grade of S. High Street.

Exploration and Findings

On December 2 and 3, 2015, one (1) structure boring, designated as B-030-1-15, was drilled to a completion depth of 59.4 feet below the existing ground surface on the east side of the existing structure. In addition to the borings performed by Rii as part of the current exploration, between July 9 and 24, 2008, three (3) borings, designated as B-027-0-08, B-029-0-08 and B-030-0-08, were advanced to a completion depth of 10.0, 136.5 and 111.0 feet below the existing ground surface, respectively, on the west and east sides of the existing structure by DLZ. Additionally, three (3) historic borings, designated as B-001-0-59, B-007-0-59 and B-011-0-59, were obtained along the east side of the existing bridge alignment, which were extended to a depth of 56.0, 65.2 and 71.0 feet, respectively, below the ground surface at the time the borings were obtained.

Borings B-028-0-08 was drilled in the existing shoulder of I-70 westbound and encountered 12.0 inches of concrete overlying 6.0 inches of aggregate base at the ground surface. Boring B-029-0-08 was performed in the existing I-70 eastbound ramp to Fourth Street and encountered 7.0 inches of asphalt overlying 9.0 inches of concrete and 11.0 inches of aggregate base at the ground surface. Boring B-030-0-08 was drilled within the existing Third Street entrance ramp to I-70 westbound and encountered 11.0 inches of asphalt overlying 6.0 inches of aggregate base. Boring B-030-1-15 was drilled within the existing I-70 eastbound exit ramp to Third Street and encountered 6.0 inches of asphalt overlying 12.0 inches of concrete followed by 6.0 inches of aggregate base.

Beneath the pavement materials in borings B-028-0-08 and B-029-0-08, material identified as existing fill or possible fill was encountered extending to a depth of 10.0 (bottom of boring) and 6.0 feet, respectively, below exiting grade, which corresponds to an elevation of 721.7 and 736.3 feet msl. The fill material was described as brown and gray gravel and gravel with sand (ODOT A-1-a, A-1-b) and contained brick fragments in one of the samples, as noted on the visual descriptions provided on the boring logs.

Underlying the surficial materials and existing fill, where encountered, natural granular soils were encountered with intermittent seams of cohesive material. The granular soils were generally described as gray, brown and brownish gray gravel, gravel with 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 gray and brown sandy silt, silt, silt and clay and silty clay (ODOT A-4a, A-4b, A-6a, A-6b).

Top of bedrock was encountered in borings B-029-0-08 and B-030-0-08 at a depth of 113.5 and 103.5 feet below existing grade, respectively, which corresponds to an elevation of 628.8 and 633.2feet msl. The upper portion of the bedrock consisted of gray, severely weathered shale overlying dark gray, moderately to highly weathered shale overlying competent limestone bedrock, which was encountered at an elevation of 621.7 feet msl in boring B-029-0-08.

In general, the historic borings, designated B-001-0-59, B-007-0-59 and B-011-0-59, encountered medium dense to very dense granular soils with intermittent seams of very stiff to hard cohesive soils. The granular soils were generally described as brown and gray gravel, gravel and sand, gravel with sand and silt, sandy silt and silt (ODOT A-1-a, A-1-b, A-2-4, A-4a, A-4b), and the cohesive soils were generally described as gray and brown sandy silt and silt and clay (ODOT A-4a, A-6a). A boulder zone was encountered in boring B-001-0-59 between elevations 740.2 and 750.2 feet msl, and boulders were encountered for the entire depth of boring B-007-0-59, to elevation 703.1 feet msl. Boulders were not noted on the log for boring B-011-0-59. In general, the subsurface conditions encountered in the historic borings matched relatively closely with the subsurface conditions encountered in the current exploration boring.

Analyses and Recommendations

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

Shallow Foundation Recommendations

It is understood that shallow spread foundations will be utilized at the rear abutment and pier substructure units. Based on plan information provided by GPD GROUP, the bottom of footing elevation at both substructure units will bear at a minimum depth of 5.0 feet below the proposed finished grade. At these elevations, the bearing soils are anticipated to consist of very dense gravel, gravel and sand and sandy silt (ODOT A-1-a, A-1-b, A-4a). Boring B-007-0-59, which is the only boring that is located at the pier substructure, has a bottom of boring elevation of 701.6 feet msl, which is approximately 25 feet below the proposed bottom of footing elevation at the pier. Given the limited depth of subsurface information below the proposed bottom of footing elevation considering boring B-007-0-59, the subsurface conditions from boring B-029-0-08 was utilized below this elevation in the settlement analysis. Shallow spread foundations bearing on these competent natural soils may be proportioned for a nominal bearing resistance as presented in the following table for the rear abutment and pier substructures.

Spread Footing Design Parameters

Substructure Unit (Borings)	Effective Footing Width (feet)	Service Limit Bearing Pressure (ksf) [1]			Nominal Bearing Resistance (ksf)	Factored Bearing Resistance [2] (ksf)
		1.0-inch	1.5-inch	1.5-inch		
Rear Abutment (B-001-0-59 / B-029-0-09 / B-030-1-15)	10.0	2.92	5.26	8.75	32.73	18.00
	12.0	2.77	4.79	7.79	32.78	18.03
	14.0	2.67	4.46	7.10	32.84	18.06
	16.0	2.59	4.21	6.58	32.89	18.09
	18.0	2.53	4.02	6.18	32.95	18.12
	20.0	2.48	3.87	5.86	33.01	18.15
	22.0	2.44	3.74	5.60	33.06	18.18
	24.0	2.41	3.64	5.39	33.12	18.21
	26.0	2.38	3.55	5.21	33.17	18.24
	28.0	2.36	3.47	5.05	33.23	18.28
	30.0	2.34	3.41	4.92	33.28	18.31
Pier (B-007-0-59 / B-029-0-09)	10.0	2.34	4.93	8.94	32.73	16.36
	12.0	2.19	4.43	7.87	32.78	16.39
	14.0	2.08	4.08	7.11	32.84	16.42
	16.0	2.00	3.81	6.54	32.89	16.45
	18.0	1.94	3.61	6.10	32.95	16.48
	20.0	1.89	3.44	5.75	33.01	16.50
	22.0	1.85	3.31	5.46	33.06	16.53
	24.0	1.82	3.19	5.22	33.12	16.56
	26.0	1.79	3.10	5.02	33.17	16.59
	28.0	1.76	3.02	4.85	33.23	16.61
	30.0	1.74	2.95	4.70	33.28	16.64

1. *The service limit bearing pressure was calculated at total settlement values of 0.5, 1.0 and 1.5 inches.*
2. *A resistance factor of φ_b = 0.55 and 0.50 was utilized in calculating the factored bearing resistance at the strength limit state for the rear abutment and pier substructures, respectively.*

Based on the service limit bearing pressures provided in Table 6 of the full report, total settlements of 1.34, 1.27 and 1.24 inches are anticipated at the west cap, roadway cap and east cap of the rear abutment, and 1.49, 0.99 and 1.47 inches are anticipated at the west cap, roadway cap and east cap of the pier. Differential settlement along the rear abutment substructure is anticipated to be less than 1/1000; however, differential settlement between the roadway cap and the adjacent east and west caps at the pier substructure may be less than 1/100 at the joints. Additionally, the maximum factored bearing pressure of 6.80 to 7.71 ksf at the rear abutment and 5.26 to 9.26 ksf at the pier substructure units will not exceed the factored bearing resistance at the strength limit of 18.09 and 16.42 ksf, respectively.

For concrete footing that rest on cohesionless soil, a coefficient “f” of 0.93 times the total vertical force on the base should be taken as the sliding resistance. A geotechnical resistance factor of $\varphi_\tau = 1.0$ should be considered when calculating the factored shear resistance between the soil and foundation for sliding.

Based on the rear abutment foundation dimensions provided in the proposed design documents, the resulting factor of safety under drained conditions (long-term stability) for the rear abutment was greater than 1.5.

Drilled Shaft Recommendations

It is understood that drilled shaft foundations are being utilized to support the forward abutment substructure unit. It is recommended that the drilled shafts be designed using the axial design parameters provided in the following table. Boring B-011-0-59, which is the only boring that is located at the forward abutment substructure, has a bottom of boring elevation of 698.9 feet msl, which is 30.0 feet below the proposed bottom of footing elevation at the forward abutment. Therefore, if the required bearing resistance cannot be achieved at the bottom of boring elevation for B-011-0-59, then the subsurface conditions and corresponding bearing resistance from boring B-029-0-08 should be utilized below this elevation. To achieve the most economical design, the drilled shafts should extend to bear in the very dense gravel or gravel and sand (ODOT A-1-a, A-1-b) at the corresponding elevations noted below in order to maximize the end bearing resistance.

Drilled Shaft Axial Design Parameters

Boring	Elevation [1] (feet msl)	Shaft Length (feet)	Soil Type	Nominal Resistance (ksf)		Resistance Factor	
				End	Side [2]	End	Side
B-001-1-59	728.9-727.5	0.0-1.4	A-6a	27	2.19	0.40	0.45
	727.5-725.0	1.4-3.9	A-4a	52	3.60	0.40	0.45
	725.0-721.0	3.9-7.9	A-1-a	60	1.98	0.50	0.55
	721.0-711.0	7.9-17.9	A-4b	60	3.97	0.50	0.55
	711.0-708.0	17.9-20.9	A-1-b	60	3.72	0.50	0.55

Boring	Elevation [1] (feet msl)	Shaft Length (feet)	Soil Type	Nominal Resistance (ksf)		Resistance Factor	
				End	Side [2]	End	Side
B-007-0-59	728.9-727.6	0.0-1.3	A-4a	49	3.60	0.40	0.45
	727.6-720.1	1.3-8.8	A-1-a	60	2.89	0.50	0.55
	720.1-715.1	8.8-13.8	A-4b	60	3.49	0.50	0.55
	715.1-709.1	13.8-19.8	A-4a	71	3.60	0.40	0.45
	709.1-704.1	19.8-24.8	A-1-b	60	2.87	0.50	0.55
	704.1-701.6	24.8-27.3	A-1-a	60	4.23	0.50	0.55
B-011-0-59	728.9-721.9	0.0-7.0	A-4a	56	3.60	0.40	0.45
	721.9-716.9	7.0-12.0	A-1-a	60	2.43	0.50	0.55
	716.9-698.9	12.0-30.0	A-1-a	60	4.03	0.50	0.55
B-029-0-08	728.9-727.3	0.0-1.6	A-4a	24	2.00	0.40	0.45
	727.3-721.3	1.6-7.6	A-1-a	60	3.06	0.50	0.55
	721.3-717.3	7.6-11.6	A-2-4	60	2.07	0.50	0.55
	717.3-700.3	11.6-28.6	A-4a	56	2.81	0.40	0.45
	700.3-695.3	28.6-33.6	A-4a	60	3.35	0.50	0.55
	695.3-685.3	33.6-43.6	A-1-b	60	4.67	0.50	0.55
	685.3-668.5	43.6-60.4	A-3a	60	2.97	0.50	0.55
	668.5-658.3	60.4-70.6	A-6a	72	3.60	0.40	0.45
	658.3-642.3	70.6-86.6	A-1-b	60	5.02	0.50	0.55
	642.3-628.8	86.6-100.1	A-3	60	3.32	0.50	0.55
B-030-0-08	730.3-728.7	0.0-1.6	A-6b	28	2.19	0.40	0.45
	728.7-725.7	1.6-4.6	A-a	35	2.37	0.40	0.45
	725.7-721.2	4.6-9.1	A-1-a	60	2.95	0.50	0.55
	721.2-713.2	9.1-17.1	A-1-a	58	2.15	0.50	0.55
	713.2-674.7	17.1-55.6	A-3a	60	2.69	0.50	0.55
	674.7-669.7	55.6-60.6	A-3	60	2.71	0.50	0.55
	669.7-633.2	60.6-97.1	A-1-b	60	5.96	0.50	0.55

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

For a single row of drilled shafts in group configuration in a predominately cohesionless profile, the following criteria are recommended for the group reduction factors for bearing resistance.

- 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 reduction factors should be applied to the total individual nominal shaft resistance (including both end bearing side resistance along the shaft length).

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

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

At the request of GPD GROUP, the nominal and factored drilled shaft axial resistance has been calculated for the forward abutment, which is summarized in the following table. A tip elevation of 671.3 feet msl was determined from the results of the lateral analysis, which was performed by GPD GROUP, and was utilized in the axial resistance calculations. For the traditional drilled shaft analysis, only end bearing resistance was accounted for in the determination of the nominal and factored axial resistance. A group reduction factor of 0.9 was utilized based on the center to center spacing of the shafts. The factored resistance using traditional drilled shaft analysis methodology is 1,357 kips per shaft. The bearing resistance for the block failure mode was also checked since the drilled shaft will be constructed tangent to each other. The resulting factored resistance per shaft for the block failure mode is 1,023 kips.

The controlling resistance between the traditional drilled shaft analysis methodology and block failure mode is 1,023 kips per shaft. The maximum factored load per shaft is 620, 724 and 718 kips/shaft for the bridge, west cap and east cap, respectively, based on the structural loading 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-70-14.05C (Project 4H) phase will consist of all work associated with the construction of the FRA-70-1405C bridge structure carrying S. High Street over I-70/I-71. This phase will also include a minimal amount of work along the side streets at the intersections with Livingston Avenue and Fulton Street.

This report is a presentation of the structure foundation exploration performed for the design and construction of the proposed FRA-70-1405C bridge structure carrying S. High Street (US-23D) and flanking cap structures over I-70/71, as shown on the vicinity map and boring plan presented in Appendix I. The existing structure is a four-span bridge with a total length of approximately 237 feet. It is understood that the existing structure consists of a reinforced concrete deck on continuous steel beams, and will be removed and replaced with a two-span continuous composite steel plate girder structure with a composite deck. The existing ramp from I-70 eastbound to 3rd Street will be eliminated and the proposed structure abutments will be shifted to the inside of the existing abutments. The proposed structures will have an approximate length of 202 feet and widths of 60 feet, 94 feet and 60 feet, representing the west cap, S. High Street roadway, and east cap structures, respectively. The three parallel structures will each react independently, with only a longitudinal expansion joint connecting them. The roadway profile grade along the I-70 eastbound beneath the structure will be cut approximately 2.0 feet below the existing roadway profile grade, and there will be no change in the profile grade of S. High Street.

2.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1 Site Geology

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

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

2.2 Existing Conditions

The proposed FRA-70-1405C structure is located at the existing S. High Street (US-23D) over I-70/71 overpass, approximately 0.7 miles east of the Scioto River. The existing I-70/I-71 in the vicinity of the structure is a six-lane, bi-directional, composite asphalt and concrete paved roadway that is generally east-west aligned through downtown Columbus, Ohio. The Third Street entrance ramp converges with I-70 westbound at the structure crossing, creating a fourth lane beneath the structure, and the existing Fourth Street exit ramp from I-70 eastbound is aligned just south of I-70. The existing S. High Street crossing is a four-lane, bi-directional, asphalt paved roadway with a southbound left turn lane, and both a northbound and southbound parking lane against the curbs. The existing I-70 profile is lowered from the surrounding terrain, as the existing corridor was cut approximately 25 to 35 below the existing grade of S. High Street and the surrounding downtown area. An existing cast-in-place concrete wall type abutment is present at the rear abutment, where the ramp from I-70 eastbound to Fourth Street crosses the structure, and graded embankment with a concrete spill through slope is utilized in front of the existing forward abutment. The embankment to the east and west of the structure at the forward abutment is grass covered with patches of brush and other vegetation, and the wall at the rear abutment continues west to the Front Street crossing. The existing structure appears to be in poor condition, with concrete spalling and delamination evident on the columns with exposed corroded reinforcing steel bars, and significant corrosion of the superstructure fascia steel beams. The 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

On December 2 and 3, 2015, one (1) structure boring, designated as B-030-1-15, was drilled to a completion depth of 59.4 feet below the existing ground surface on the east side of the existing structure. In addition to the borings performed by Rii as part of the current exploration, between July 9 and 24, 2008, three (3) borings, designated as B-027-0-08, B-029-0-08 and B-030-0-08, were performed by DLZ as part of the FRA-70-8.93 preliminary exploration and their findings were published in a report dated September 24, 2009. The borings were advanced to a completion depth of 10.0, 136.5 and 111.0 feet below the existing ground surface, respectively, on the west and east sides of the existing structure. The current project boring locations are shown on the boring plan provided in Appendix I of this report and summarized in Table 1 below.

Table 1. Test Boring Summary

Boring Number	Reference Alignment	Station	Offset	Latitude	Longitude	Ground Elevation (feet msl)	Boring Depth (feet)
B-028-0-08	BL I-70 WB	191+29.78	14.9’ Lt.	39.953161	-82.999193	731.7	10.0
B-029-0-08	BL I-70 EB	191+53.21	46.3’ Rt.	39.952780	-82.999049	742.3	136.5
B-030-0-08	BL I-70 EB	194+15.01	104.2' Lt.	39.953275	-82.998195	736.7	111.0
B-030-1-15	BL I-70 EB	194+37.05	70.0' Rt.	39.952814	-82.998014	748.9	59.4

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

The boring performed by Rii for the current exploration was drilled using a truck-mounted rotary drilling machine, utilizing a 3.25-inch inside diameter, hollow-stem augers to advance the hole. The borings performed by DLZ were drilled using a truck-mounted rotary drilling machine, utilizing a 3.25-inch inside diameter, hollow-stem auger to advance the holes.

Standard penetration testing (SPT) and split spoon sampling were performed in the borings at 2.5-foot increments of depth to 10.0 to 35 feet, and at 5.0-foot increments thereafter to the boring termination depth or top of bedrock. An automatic drop hammer was utilized by Rii and DLZ 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 CME 55 drill rig used by Rii was calibrated on October 20, 2014, and has a drill rod energy ratio 92.0 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.

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.

During drilling for the boring performed by Rii, 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	17
Plastic and Liquid Limits	AASHTO T89, T90	9
Gradation – Sieve/Hydrometer	AASHTO T88	9

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.

The depth to bedrock was determined by split spoon sampler refusal. Split spoon sampler refusal is defined as exceeding 50 blows from the hammer with less than 6.0 inches of penetration by the split spoon sampler.

Where borings were extended into the competent bedrock (after encountering auger refusal), an NQ double-tube diamond bit core barrel (utilizing wire line equipment) was used to core the bedrock. Coring produced 1.85 inch diameter cores from which the type of rock and its geological characteristics were determined.

Rock cores were analyzed to identify the type of rock, color, mineral content, bedding planes and other geological and mechanical features of interest in this project. The Rock Quality Designation (RQD) for each rock core run was calculated according to the following equation:

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

In addition to the borings performed as part of the preliminary engineering exploration, historic borings performed in 1959 by the Department of Highways as part of the original FRA-40-12.82 project for the existing structure were also obtained from the construction documents on record. Three (3) borings, designated as B-001-0-59, B-007-0-59 and B-011-0-59, were obtained along the east side of the existing bridge alignment. Based on the elevations provided on the boring logs, it is anticipated that these borings were performed from the then-existing ground surface and that the profile for the then-proposed US 40 (existing I-70/71) was lowered to provide sufficient clearance for the bridge to be constructed at the then-existing ground surface. The borings were extended to a depth of 56.0, 65.2 and 71.0 feet, respectively, below the ground surface at the time the borings were obtained. Please note that the elevations provided on the historic boring logs were referenced to the North American Datum (NAD) 27. The current design survey is referenced to NAD 83. The NAD 27 datum is 0.6 feet lower than the NAD 83 datum. **Therefore, all elevations noted in this report with respect to the historic borings are adjusted to the current NAD 83 datum.** The historic boring locations are shown on the boring plan provided in Appendix I of this report and the historic boring logs are provided in Appendix IV.

4.0 FINDINGS

Interpreted engineering logs have been prepared by Rii and DLZ 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 performed as part of the preliminary engineering phase and current exploration and what is represented on the boring logs.

4.1 Surface Materials

Borings B-028-0-08 was drilled in the existing shoulder of I-70 westbound and encountered 12.0 inches of concrete overlying 6.0 inches of aggregate base at the ground surface. Boring B-029-0-08 was performed in the existing I-70 eastbound ramp to Fourth Street and encountered 7.0 inches of asphalt overlying 9.0 inches of concrete and 11.0 inches of aggregate base at the ground surface. Boring B-030-0-08 was drilled within the existing Third Street entrance ramp to I-70 westbound and encountered 11.0 inches of asphalt overlying 6.0 inches of aggregate base. Boring B-030-1-15 was drilled within the existing I-70 eastbound exit ramp to Third Street and encountered 6.0 inches of asphalt overlying 12.0 inches of concrete followed by 6.0 inches of aggregate base. Surface materials were not noted on the 1959 historic boring logs.

4.2 Subsurface Soils

Beneath the pavement materials in borings B-028-0-08 and B-029-0-08, material identified as existing fill or possible fill was encountered extending to a depth of 10.0 (bottom of boring) and 6.0 feet, respectively, below exiting grade, which corresponds to an elevation of 721.7 and 736.3 feet msl. The fill material was described as brown and gray gravel and gravel with sand (ODOT A-1-a, A-1-b) and contained brick fragments in one of the samples, as noted on the visual descriptions provided on the boring logs.

Underlying the surficial materials and existing fill, where encountered, natural granular soils were encountered with intermittent seams of cohesive material. The granular soils were generally described as gray, brown and brownish gray gravel, gravel with 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 gray and brown sandy silt, silt, silt and clay and silty clay (ODOT A-4a, A-4b, A-6a, A-6b).

The relative density of granular soils is primarily derived from SPT blow counts (N_{60}). Based on the SPT blow counts obtained, the granular soil encountered ranged from very loose ($5 < N_{60}$ blows per foot [bpf]) to very dense ($N_{60} > 50$ bpf). Overall blow counts recorded from the SPT sampling ranged from 4 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 stiff ($1.0 < HP \leq 2.0$ 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 1 to 19 percent. The natural moisture content of the cohesive soil samples tested for plasticity index ranged from 6 percent below to 1 percent above their corresponding plastic limits. In general, the soil exhibited natural moisture contents considered to be significantly below to near optimum moisture levels.

4.3 Bedrock

Bedrock was encountered in borings B-029-0-08 and B-030-0-08 as presented in Table 3.

Table 3. Top of Bedrock Elevations

Boring Number	Ground Surface Elevation (feet msl)	Top of Bedrock		Top of Bedrock Core	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-029-0-08	742.3	113.5	628.8	116.5	625.8
B-030-0-08	736.7	103.5	633.2	105.0	631.7

Top of bedrock was encountered in borings B-029-0-08 and B-030-0-08 at a depth of 113.5 and 103.5 feet below existing grade, respectively, which corresponds to an elevation of 628.8 and 633.2feet msl. The upper portion of the bedrock consisted of gray, severely weathered shale overlying dark gray, moderately to highly weathered shale overlying competent limestone bedrock, which was encountered at an elevation of 621.7 feet msl in boring B-029-0-08. The cored shale is described as dark gray, moderately to highly weathered, weak, thinly laminated to laminated, calcareous, pyritic, fissile, friable, jointed and fractured to highly fractured with tight, slightly rough apertures. The limestone encountered in boring B-029-0-08 is described as brownish gray to light gray, slightly to moderately weathered, moderately strong to strong, very thin to thin bedded, fossiliferous, stylolitic, cherty, pyritic and slightly to moderately fractured with tight to open, slightly rough apertures.

The percent recovery and RQD values from the bedrock core runs in boring B-029-0-08 are summarized in Table 4.

Table 4. Rock Core Summary

Boring	Core No.	Depth (feet)	Recovery (%)	RQD (%)
B-029-0-08	R-1	116.5 to 120.0	90	0
	R-2	120.0 to 125.0	100	88
	R-3	125.0 to 130.0	100	85
	R-4	130.0 to 135.0	90	76
	R-5	135.0 to 136.5	100	100
B-030-0-08	R-1	105.0 to 110.0	95	37
	R-2	110.0 to 111.0	100	42

It should be noted that bedrock can experience mechanical breaks during the drilling and coring processes. It is anticipated that DLZ attempted to account for fresh, manmade breaks during tabulation of the RQD analysis, per ODOT SGE specifications. The quality of the shale bedrock, according to the RQD values, was very poor (RQD ≤ 25%) to poor (25 < RQD ≤ 50%), and the quality of the limestone bedrock was good (75 < RQD ≤ 90%) 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 Surface Elevation (feet msl)	Initial Groundwater		Upon Completion	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-028-0-08	731.7	Dry	Dry	Dry	Dry
B-029-0-08	742.3	21.0	721.3	20.5 [1]	721.8
B-030-0-08	736.7	16.0	720.7	13.6	723.1
B-030-1-15	748.9	28.5	720.4	28.5	720.4

1. *The groundwater level at completion was measured after the rock coring process, which included the addition of water for coring.*

Groundwater was not encountered during or at the completion of drilling in boring B-028-0-08. Groundwater was encountered initially during the drilling process in the remaining borings at a depths ranging from 16.0 to 28.5 feet below the existing ground surface, which corresponds to an elevations ranging from 720.4 to 723.1 feet msl. The groundwater level at the completion of drilling in boring B-029-0-08 was 20.5 feet below existing grade following the rock coring process. At the completion of drilling in borings B-030-0-08 and B-030-1-15 was encountered at a depth of 13.6 and 28.5 feet below grade, respectively, which corresponds to an elevation of 723.1 and 720.4 feet msl. Additionally, DLZ noted that they frequently added water to the borehole to clean out the augers after encountering sand heave of varying amounts at various depths.

Please note that short-term water level readings, especially in cohesive soils, are not necessarily an accurate indication of the actual groundwater level. In addition, groundwater levels or the presence of groundwater are considered to be dependent on seasonal fluctuations in precipitation.

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

4.5 Historic Borings

In general, the historic borings, designated B-001-0-59, B-007-0-59 and B-011-0-59, encountered medium dense to very dense granular soils with intermittent seams of very stiff to hard cohesive soils. The granular soils were generally described as brown and gray gravel, gravel and sand, gravel with sand and silt, sandy silt and silt (ODOT A-1-a, A-1-b, A-2-4, A-4a, A-4b), and the cohesive soils were generally described as gray and brown sandy silt and silt and clay (ODOT A-4a, A-6a). A boulder zone was encountered in boring B-001-0-59 between elevations 740.2 and 750.2 feet msl, and boulders were encountered for the entire depth of boring B-007-0-59, to elevation 703.1 feet msl. Boulders were not noted on the log for boring B-011-0-59. Bedrock was not encountered in the historic borings prior to the termination depths. Groundwater levels were not noted in the borings performed during the 1959 exploration. In general, the subsurface conditions encountered in the historic borings matched relatively closely with the subsurface conditions encountered in the preliminary engineering exploration borings.

5.0 ANALYSES AND RECOMMENDATIONS

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

Design details of the structures proposed were provided by GPD GROUP. Based on the information provided, it is understood that the existing structure will be removed and replaced with three parallel, two-span continuous composite steel plate girder structures with a composite deck with a full-height, wall-type rear abutment and reinforced concrete pier supported on spread foundations and forward abutment supported on tangent drilled shafts. Proposed structural data was obtained from design details provided by GPD GROUP and are included in Table 6.

Table 6. FRA-70-1405C Structure and Bridge Design Elevations

Substructure Unit (Borings)	Structure Component [1]	Elevation [1] (feet msl)	Design Maximum Factored Load	
			Service	Strength
Rear Abutment (B-001-0-59 / B-029-0-09 / B-030-1-15)	Bottom of Footing	West Cap: 728.5 Bridge:729.9 East Cap: 731.2	West Cap: 5.57 ksf Bridge: 5.24 ksf East Cap: 4.97 ksf	West Cap: 7.71 ksf Bridge: 7.41 ksf East Cap: 6.80 ksf
Pier (B-007-0-59 / B-029-0-09)	Bottom of Footing	West Cap: 725.4 Bridge: 726.9 East Cap: 728.4	West Cap: 6.78 ksf Bridge: 3.93 ksf East Cap: 6.66 ksf	West Cap: 9.26 ksf Bridge: 5.26 ksf East Cap: 9.10 ksf
Forward Abutment (B-011-0-59 / B-029-0-09 / B-030-0-08)	Top of Embedded Shafts (Bottom of Wall Panels)	West Cap: 727.5 Bridge: 728.9 East Cap: 730.3	West Cap: 564 kips/shaft Bridge: 480 kips/shaft East Cap: 557 kips/shaft	West Cap: 724 kips/shaft Bridge: 620 kips/shaft East Cap: 718 kips/shaft

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

Boring B-028-0-08 was not used for analysis at any of the substructure units due to the shallow depth of the boring.

5.1 Shallow Foundation Recommendations

It is understood that shallow spread foundations will be utilized at the rear abutment and pier substructure units. Based on plan information provided by GPD GROUP, the bottom of footing elevation at both substructure units will bear at a minimum depth of 5.0 feet below the proposed finished grade, at elevations noted above in Table 6. At these elevations, the bearing soils are anticipated to consist of very dense gravel, gravel and sand and sandy silt (ODOT A-1-a, A-1-b, A-4a). Boring B-007-0-59, which is the only boring that is located at the pier substructure, has a bottom of boring elevation of 701.6 feet msl, which is approximately 25 feet below the proposed bottom of footing elevation at the pier. Given the limited depth of subsurface information below the proposed bottom of footing elevation considering boring B-007-0-59, the subsurface conditions from boring B-029-0-08 was utilized below this elevation in the settlement analysis. Shallow spread foundations bearing on these competent natural soils may be proportioned for a nominal bearing resistance as presented in Table 7 for the rear abutment and pier substructures.

Table 7. Spread Footing Design Parameters

Substructure Unit (Borings)	Effective Footing Width (feet)	Service Limit Bearing Pressure (ksf) [1]			Nominal Bearing Resistance (ksf)	Factored Bearing Resistance [2] (ksf)
		0.5-inch	1.0-inch	1.5-inch		
Rear Abutment (B-001-0-59 / B-029-0-09 / B-030-1-15)	10.0	2.92	5.26	8.75	32.73	18.00
	12.0	2.77	4.79	7.79	32.78	18.03
	14.0	2.67	4.46	7.10	32.84	18.06
	16.0	2.59	4.21	6.58	32.89	18.09
	18.0	2.53	4.02	6.18	32.95	18.12
	20.0	2.48	3.87	5.86	33.01	18.15
	22.0	2.44	3.74	5.60	33.06	18.18
	24.0	2.41	3.64	5.39	33.12	18.21
	26.0	2.38	3.55	5.21	33.17	18.24
	28.0	2.36	3.47	5.05	33.23	18.28
	30.0	2.34	3.41	4.92	33.28	18.31
Pier (B-007-0-59 / B-029-0-09)	10.0	2.34	4.93	8.94	32.73	16.36
	12.0	2.19	4.43	7.87	32.78	16.39
	14.0	2.08	4.08	7.11	32.84	16.42
	16.0	2.00	3.81	6.54	32.89	16.45
	18.0	1.94	3.61	6.10	32.95	16.48
	20.0	1.89	3.44	5.75	33.01	16.50
	22.0	1.85	3.31	5.46	33.06	16.53
	24.0	1.82	3.19	5.22	33.12	16.56
	26.0	1.79	3.10	5.02	33.17	16.59
	28.0	1.76	3.02	4.85	33.23	16.61
	30.0	1.74	2.95	4.70	33.28	16.64

1. *The service limit bearing pressure was calculated at total settlement values of 0.5, 1.0 and 1.5 inches.*
2. *A resistance factor of φ_b = 0.55 and 0.50 was utilized in calculating the factored bearing resistance at the strength limit state for the rear abutment and pier substructures, respectively.*

The service limit bearing pressure that results in a maximum total settlement of 0.5, 1.0 and 1.5 inches was calculated and presented in Table 7. A geotechnical resistance factor of φ_b = 0.55 and 0.50 has been considered in calculating the factored bearing resistance at the strength limit state for the rear abutment and pier substructures, respectively. Based on the bearing pressures provided in Table 7 and applying the geotechnical resistance factor provided to the nominal bearing resistance at the strength limit state, the service limit state should control the minimum footing dimensions for all effective footing widths analyzed at 0.5, 1.0 and 1.5 inches of total settlement considered in the analysis. A graphical representation of the service limit bearing pressures and nominal and factored bearing resistance at the strength limit state for the rear abutment and pier substructures is presented in Appendix V.

Based on the service limit bearing pressures provided in Table 6, total settlements of 1.34, 1.27 and 1.24 inches are anticipated at the west cap, bridge and east cap of the rear abutment, and 1.49, 0.99 and 1.47 inches are anticipated at the west cap, bridge and east cap of the pier. Differential settlement along the rear abutment substructure is anticipated to be less than 1/1000; however, differential settlement between the bridge and the adjacent east and west caps at the pier substructure may be less than 1/100 at the joints. Additionally, the maximum factored bearing pressure of 6.80 to 7.71 ksf at the rear abutment and 5.26 to 9.26 ksf at the pier substructure units will not exceed the factored bearing resistance at the strength limit of 18.09 and 16.42 ksf, respectively.

Calculations for settlement of the west, bridge and east caps and nominal and factored bearing resistance for the shallow spread foundations are provided in Appendix VI.

5.1.1 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 0.93 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.

5.1.2 Overall (Global) Stability

A slope stability analysis was performed to check the global stability at the rear abutment. 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 resisting soil mass above the concrete or through the concrete footing itself.

Table 8. Shear Strength Parameters Utilized in Stability Analyses

Material Type	γ (pcf)	φ' (1) (°)	c' (2) (psf)	S_u (3) (psf)
Item 203 Granular Embankment (Backfill between Ex. and Pr. Foundations)	125	32	0	N/A
Very Stiff to Hard Sandy Silt (ODOT A-4a)	120 to 130	29 to 32	0 to 50	2,250 to 6,250
Hard Silt and Clay (ODOT A-6a)	125	28	25	4,500
Very Dense Gravel with Sand and Silt, Coarse and Fine Sand, Sandy Silt and Silt (ODOT A-2-4, A-3a, A-4a, A-4b)	135	36 to 38	0	N/A
Dense to Very Dense Gravel and Gravel with Sand (ODOT A-1-a, A-1-b)	130 to 135	42 to 43	0	N/A

1. *Per Figure 7-45, Section 7.6.9 of FHWA GEC 5 for cohesive soils and Table 10.4.6.2.4-1 of the AASHTO LRFS BDS for granular soils.*
2. *Estimated based on overconsolidated nature of soil.*
3. *S_u = 125(N_{60}), Terzaghi and Peck (1967).*

Per Section 11.6.2.3 of the AASHTO LRFD BDS, overall (global) stability for CIP walls that are integrated with or supporting structural foundations or elements is satisfied if the product of the factor of safety from the slope stability output multiplied by the resistance factor φ=0.65 is greater than 1.0. Therefore, global stability is satisfied when a minimum factor of safety of 1.5 is obtained. Based on the rear abutment foundation dimensions provided in the proposed design documents, the resulting factor of safety under drained conditions (long-term stability) for the rear abutment was greater than 1.5. Given the very stiff to hard nature of the cohesive soils, slope stability under undrained conditions (short-term stability) was not checked, as drained conditions will control the design. Calculations for overall (global) stability of the rear abutment foundation is provided in Appendix VII.

5.2 Drilled Shaft Recommendations

It is understood that tangent drilled shaft foundations are being utilized to support the forward abutment substructure unit. It is recommended that the drilled shafts be designed using the axial design parameters provided in Table 9. Boring B-011-0-59, which is the only boring that is located within the limits of the forward abutment substructure, has a bottom of boring elevation of 698.9 feet msl, which is 30.0 feet below the proposed bottom of footing elevation at the forward abutment. Therefore, if the required bearing resistance cannot be achieved at the bottom of boring elevation for B-011-0-59, then the subsurface conditions and corresponding bearing resistance from boring B-029-0-08 or B-030-0-08 should be utilized below this elevation. To achieve the most economical design, the drilled shafts should extend to bear in the very dense gravel or gravel with sand (ODOT A-1-a, A-1-b) at the corresponding elevations noted below in order to maximize the end bearing resistance. The drilled shafts should be proportioned for a nominal bearing resistance as presented in Table 9.

Table 9. Drilled Shaft Axial Design Parameters

Boring	Elevation [1] (feet msl)	Shaft Length (feet)	Soil Type	Nominal Resistance (ksf)		Resistance Factor	
				End	Side [2]	End	Side
B-001-1-59	728.9-727.5	0.0-1.4	A-6a	27	2.19	0.40	0.45
	727.5-725.0	1.4-3.9	A-4a	52	3.60	0.40	0.45
	725.0-721.0	3.9-7.9	A-1-a	60	1.98	0.50	0.55
	721.0-711.0	7.9-17.9	A-4b	60	3.97	0.50	0.55
	711.0-708.0	17.9-20.9	A-1-b	60	3.72	0.50	0.55
B-007-0-59	728.9-727.6	0.0-1.3	A-4a	49	3.60	0.40	0.45
	727.6-720.1	1.3-8.8	A-1-a	60	2.89	0.50	0.55
	720.1-715.1	8.8-13.8	A-4b	60	3.49	0.50	0.55
	715.1-709.1	13.8-19.8	A-4a	71	3.60	0.40	0.45
	709.1-704.1	19.8-24.8	A-1-b	60	2.87	0.50	0.55
	704.1-701.6	24.8-27.3	A-1-a	60	4.23	0.50	0.55
B-011-0-59	728.9-721.9	0.0-7.0	A-4a	56	3.60	0.40	0.45
	721.9-716.9	7.0-12.0	A-1-a	60	2.43	0.50	0.55
	716.9-698.9	12.0-30.0	A-1-a	60	4.03	0.50	0.55
B-029-0-08	728.9-727.3	0.0-1.6	A-4a	24	2.00	0.40	0.45
	727.3-721.3	1.6-7.6	A-1-a	60	3.06	0.50	0.55
	721.3-717.3	7.6-11.6	A-2-4	60	2.07	0.50	0.55
	717.3-700.3	11.6-28.6	A-4a	56	2.81	0.40	0.45
	700.3-695.3	28.6-33.6	A-4a	60	3.35	0.50	0.55
	695.3-685.3	33.6-43.6	A-1-b	60	4.67	0.50	0.55
	685.3-668.5	43.6-60.4	A-3a	60	2.97	0.50	0.55
	668.5-658.3	60.4-70.6	A-6a	72	3.60	0.40	0.45
	658.3-642.3	70.6-86.6	A-1-b	60	5.02	0.50	0.55
	642.3-628.8	86.6-100.1	A-3	60	3.32	0.50	0.55
B-030-0-08	730.3-728.7	0.0-1.6	A-6b	28	2.19	0.40	0.45
	728.7-725.7	1.6-4.6	A-a	35	2.37	0.40	0.45
	725.7-721.2	4.6-9.1	A-1-a	60	2.95	0.50	0.55
	721.2-713.2	9.1-17.1	A-1-a	58	2.15	0.50	0.55
	713.2-674.7	17.1-55.6	A-3a	60	2.69	0.50	0.55
	674.7-669.7	55.6-60.6	A-3	60	2.71	0.50	0.55
	669.7-633.2	60.6-97.1	A-1-b	60	5.96	0.50	0.55

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

Drilled shaft lengths should measure a minimum of three (3) times the shaft diameter. Per Section 10.8.3.5.1b of the AASHTO LRFD BDS, side resistance should be neglected for the upper 5.0 feet of the shaft length where cohesive soils (ODOT A-4a, A-4b, A-6a, A-6b, A-7-6) are present below the bottom of footing/top of shaft elevation. Total settlement of the drilled shafts is estimated to be less than 1.0 inch for shafts bearing at or below elevation 695.0 feet msl.

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

5.2.1 Group Efficiency

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

- 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 reduction factors should be applied to the total individual nominal shaft resistance (including both end bearing side resistance along the shaft length).

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

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

5.2.2 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 IX. 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 10 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 IX.

Table 10. 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 foundation elements at the forward abutment, which was utilized to determine the shaft tip elevation provided in the design plans.

5.2.3 Drilled Shaft Axial Resistance

At the request of GPD GROUP, the nominal and factored drilled shaft axial resistance has been calculated for the forward abutment, which is summarized in Table 11 below. A tip elevation of 671.3 feet msl was determined from the results of the lateral analysis, which was performed by GPD GROUP, and was utilized in the axial resistance calculations. For the traditional drilled shaft analysis, only end bearing resistance was accounted for in the determination of the nominal and factored axial resistance. A group reduction factor of 0.9 was utilized based on the center to center spacing of the shafts.

The bearing resistance for the block failure mode was also checked since the drilled shaft will be constructed tangent to each other. Based on the shaft tip elevation provided, the shafts will be bearing in very dense sand and gravel (ODOT A-1-b) overlying hard silt and clay (ODOT A-6a). The analysis was performed considering both undrained and drained shear strength parameters for both material types and the limiting resistance was considered in the design calculations. Using the undrained shear strength from the hard silt and clay (ODOT A-6a), the resulting nominal unit bearing resistance is 45.2 ksf and the factored unit bearing resistance is 20.4 ksf, considering a resistance factor of 0.45.

Table 11. FRA-70-14.05C Drilled Shaft Recommendations – Forward Abutment

Drilled Shaft Analysis Methodology	Shaft Diameter (feet)	Shaft Elevation (feet msl)		Shaft Length (feet)	C-C Shaft Spacing (feet)	Nominal Resistance [1] (kips)			Factored Resistance (kips)		
		Top [2]	Tip			End	Side	Total	End [3]	Side	Total
Traditional	8.0	728.9	671.3	57.6	8.0	2,714	N/A	2,714	1,357	N/A	1,357
Block	8.0	728.9	671.3	57.6	8.0	2,273	N/A	2,273	1,023	N/A	1,023

1. *A group reduction factor of 0.9 was utilized based on the center-to-center spacing of the shafts for the traditional analysis methodology.*
2. *Top of shaft elevation corresponds to the bottom of wall elevation.*
3. *A resistance factor of 0.5 was utilized for the traditional drilled shaft analysis methodology, and 0.45 was utilized for the block failure mode.*

The controlling resistance between the traditional drilled shaft analysis methodology and block failure mode is 1,023 kips per shaft. The maximum factored load per shaft is 620, 724 and 718 kips/shaft for the bridge, west cap and east cap, respectively, based on the structural loading information provided by GPD GROUP. Calculations for the drilled shaft axial resistance are provided in Appendix VII.

5.2.4 Drilled Shaft Considerations

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

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

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

- Caving material is encountered at any time during the drilling of the shaft.

- Groundwater is encountered at any time during the drilling of the shaft, or groundwater seepage occurs in the drilled shaft.

- Down hole inspection is planned (casing is required for this instance).

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

It is anticipated that conventional drilled shaft equipment (with a standard soil bit) will be able to penetrate the surficial soils to the required tip elevation. However, as noted in Section 4.5, large boulders were encountered in borings B-001-0-59 and B-011-0-59 at within the embedment zone for the drilled shafts. Therefore, 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.

5.3 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 12 and Table 13.

Table 12. 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	130	0	33°	0.26	0.46	7.41

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

Table 13. 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	130	0	33°	0.26	0.46	7.41

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).

Given the proximity of the drilled shafts for both foundation alternatives at the forward abutment to the existing 96-inch storm sewer, the influence due to lateral loading from the drilled shafts on the sewer line should be evaluated. Based on the plan information provided, it is understood that the shafts will have approximately 7.0 feet of clear distance between the edge of the shafts and edge of the sewer.

5.4.1 Excavation Considerations

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

Table 14. 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

Based on plan information provided, it is understood that temporary shoring, consisting of soldier pile and lagging wall types, will be utilized for excavation and construction of the foundation elevations at the rear abutment and pier substructure locations. The temporary shoring should be designed using the lateral design parameters in Appendix IX and the lateral loading from the retained soil should be calculated using the lateral earth pressure coefficients in Section 5.3. It is understood that GPD GROUP will be providing all calculations for temporary shoring.

5.4.2 Groundwater Considerations

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

6.0 LIMITATIONS OF STUDY

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

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

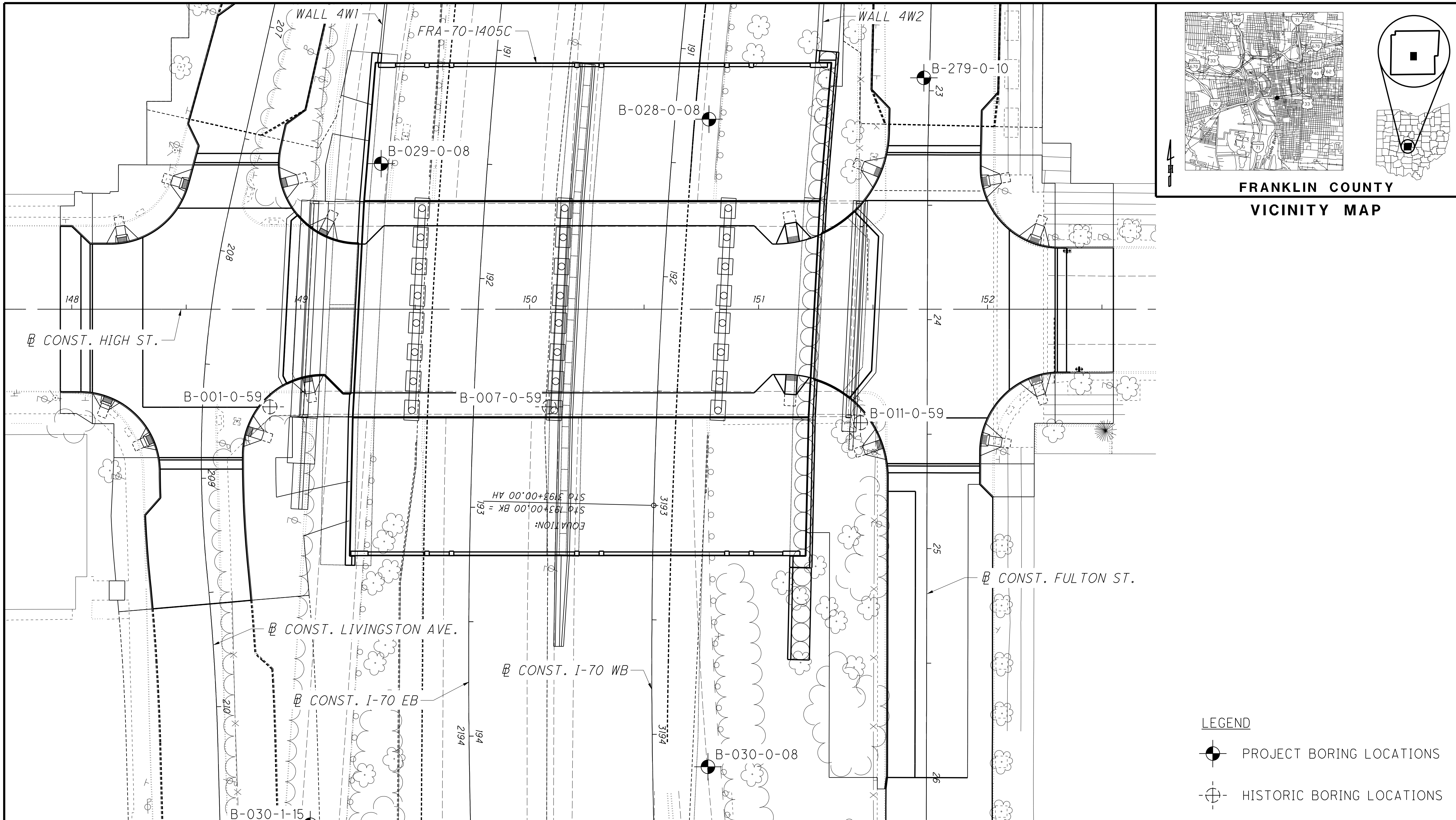
The conclusions and recommendations herein have been based upon the available soil and bedrock information and the design details furnished by a representative of the owner of the proposed project. Any revision in the plans for the proposed construction from those anticipated in this report should be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

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

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

APPENDIX I

VICINITY MAP AND BORING PLAN



BORING PLAN

BRIDGE NO. FRA-70-1405C

FRANKLIN COUNTY, OHIO

PROJECT NO. Rii W-13-045		DRAWN RRM
SCALE: 1"=40' 0 20 40	N	REVIEWED BRT
		DATE 6/11/2020

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 A-2-5				35 Max.	40 Max. 41 Min.	10 Max.	0	
	Gravel and/or Stone Fragments with Sand, Silt and Clay	A-2-6 A-2-7				35 Max.	40 Max. 41 Min.	11 Min.	4	
	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-028-0-08, B-029-0-08, B-030-0-08 and B-030-1-15

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 (%)

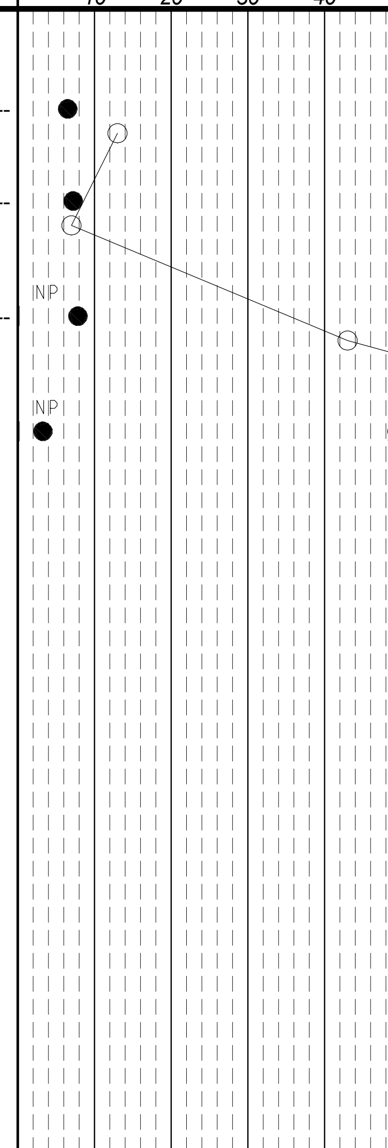
Client: ms consultants	Project: FRA-70-8.93	Job No. 0221-1004.01
LOG OF: Boring B-028-0-08	Location: Sta. 191+29.78, 14.9' LT., BL I-70 WB	Date Drilled: 7/24/2008

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

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

Depth (ft)	Elev. (ft) 731.7	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
1.5	730.2						Asphalt Concrete - 12" Aggregate Base - 6"							
		8 6 7	8	1			FILL: Loose to medium dense brown GRAVEL (A-1-a), little to some fine to coarse sand, little silty clay; moist.		65	16	---	6	--13--	
6.0	725.7	4 5 2	7	2					70	14	---	5	--11--	
8.5	723.2	10 20 22	16	3			FILL: Dense brown GRAVEL WITH SAND (A-1-b), little silt; contains brick fragments and silty clay seams; damp.		34	35	---	14	--17--	
10.0	721.7	47 49 46	12	4			FILL: Very dense gray GRAVEL (A-1-a), trace fine to coarse sand; contains few silty clay seams; damp.		84	5	---	3	--8--	
							Bottom of Boring - 10.0'							

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

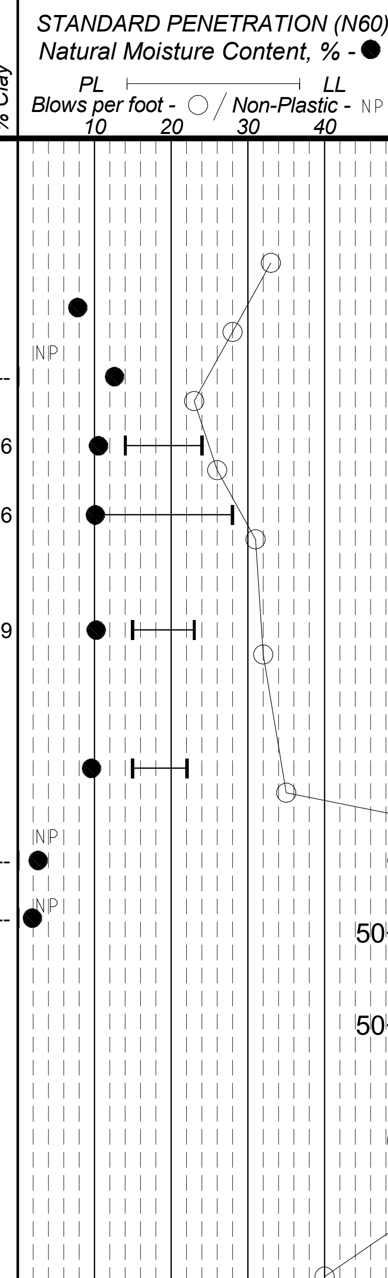


Client: ms consultants	*Project:* FRA-70-8.93	*Job No.* 0221-1004.01
LOG OF: Boring B-029-0-08	*Location:* Sta. 191+53.21, 46.3' RT., BL I-70 EB	*Date Drilled:* 7/9/2008 to 7/14/2008

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)	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
	742.3						Asphalt Concrete Pavement - 7" Portland Cement Concrete - 9" Aggregate Base - 11"						
2.3	740.0	5 15 17	7	1									
							POSSIBLE FILL: Medium dense to dense brown GRAVEL WITH SAND (A-1-b), little to some silt; damp.						
		13 15 12	10	2									
5		11 13 10	6	3				55	17	---	9	--19--	
6.0	736.3												
		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
7.5	734.8												
		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
10.0	732.3												
		10 14 17	18	6		4.5	Hard gray SANDY SILT (A-4a), some fine to coarse sand, trace gravel; damp.	8	8	---	15	40	29
							@ 11.5'-21.0', difficult drilling.						
		8 15 19	18	7		4.5							
15.0	727.3												
		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--	
		50/5	1	10			@ 18.5'-18.9', rock fragments; possible cobble blocking shoe.						
20													
21.0	721.3												
		21 28 33	6	11			Very dense gray GRAVEL WITH SAND AND SILT (A-2-4), some silt; wet.						
23.5	718.8												
		11 18 21	16	12			Dense gray SILT (A-4b), little fine sand; contains interbedded sand seams; wet.						
25	717.3												



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.

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

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	Standard Penetration (N60)
		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.							55
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.							58
		14 23 33	14	17		4.5+		28	15	---	19	25	13	57
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.							
47.0	695.3	19 37 50/3	15	19			Very dense gray GRAVEL WITH SAND (A-1-b), trace silt; wet. @ 48.5', 6 inches sand heave.	21	48	---	27	--4--		NP 50+
50	692.3													

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													

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)	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetro-meter (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N60)
	667.3						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+
80														
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+
85														
		15 50/4	10	27										50+
90														
		13 26 30	18	28				31	48	---	18	--3--		NP 57
95														
		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 | 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, % / PL–LL
							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', 22 feet sand heave; washed out.							
113.5	628.8	50/2	2	32			Severely weathered gray SHALE.							50+
115														
116.5	625.8	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														
120.6	621.7	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							
125	617.3													

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) 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

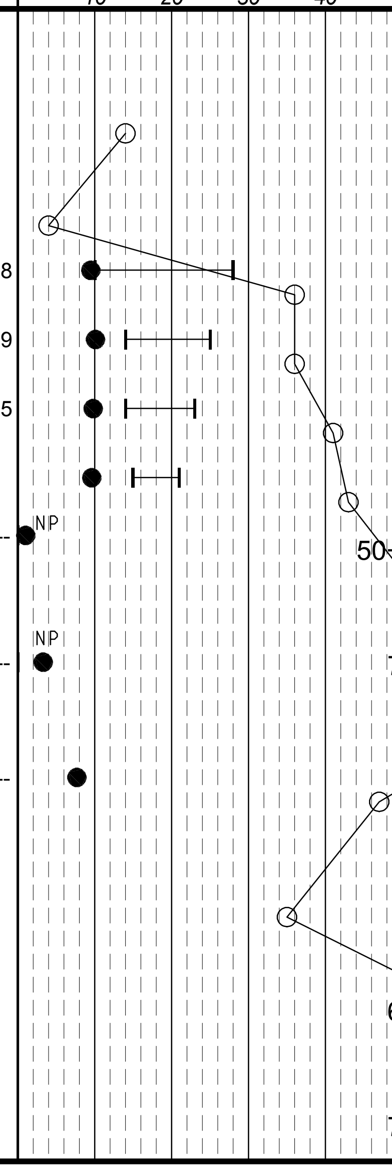
Client: ms consultants	Project: FRA-70-8.93	Job No. 0221-1004.01
LOG OF: Boring B-030-0-08	Location: Sta. 194+14.54, 24.17' LT., BL I-70 WB	Date Drilled: 7/20/2008 to 7/23/2008

WATER OBSERVATIONS:
Water seepage at: 16.0'
Water level at completion: 13.6' (includes drilling water)

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

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
	736.7						Asphalt Concrete - 11" Aggregate Base - 6"						
1.4	735.3												
		4 6 8	11	1			Loose to medium dense brown GRAVEL WITH SAND (A-1-b), some fine to coarse sand, little silt; damp.						
5.0	731.7	3 3 1	8	2									
6.5	730.2	4 14 21	18	3		4.5+	Hard gray SILTY CLAY (A-6b), some fine to coarse sand, trace gravel; contains few rock fragments; damp to moist.	9	9	---	14	40	28
8.0	728.7	10 16 19	18	4		4.5+	Hard gray SILT AND CLAY (A-6a), some fine to coarse sand, trace gravel; damp.	6	10	---	15	40	29
		13 19 21	18	5		4.5+	Hard gray SANDY SILT (A-4a), some fine to coarse sand, trace gravel; damp.	5	11	---	18	41	25
11.0	725.7	11 15 27	18	6		4.5+							
		16 50/5	8	7			Dense to very dense gray GRAVEL (A-1-a), little to some fine to coarse sand, trace to little silty clay; damp.	71	12	---	5	--12--	
		11 33 38	13	8				59	21	---	8	--12--	
		18 23 23	13	9			@ 16.0', becomes wet.	61	20	---	8	--11--	
		12 13 21	12	10									
23.5	713.2	13 28 36	12	11									
25	711.7	17 34 40	14	12			Very dense gray COARSE AND FINE SAND (A-3a), some silty clay; wet.						

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-030-0-08 | *Location:* Sta. 194+14.54, 24.17' LT., BL I-70 WB | *Date Drilled:* 7/20/2008 to 7/23/2008

WATER OBSERVATIONS:
Water seepage at: 16.0'
Water level at completion: 13.6' (includes drilling water)

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

Depth (ft)	Elev. (ft) 711.7	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 COARSE AND FINE SAND (A-3a), some silty clay; wet.							
		23 45 50/4	15	13			@ 26.0', 0.4 feet sand heave.							50+
30		9 21 31	16	14										53
35		14 30 45	16	15				0	8	---	69	--23--		77
40		50/4	4	16			@ 38.5'-38.8', contains sandy silt seams.							50+
							@ 38.5'-43.5', difficult drilling.							
45		11 32 50/5	17	17			@ 43.5', 1.9 feet sand heave.							50+
47.0	689.7						Very dense gray GRAVEL WITH SAND (A-1-b), trace silt; wet.							
		12 50/5	10	18				15	54	---	23	--8--		50+
50	686.7													

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-030-0-08	Location: Sta. 194+14.54, 24.17' LT., BL I-70 WB	Date Drilled: 7/20/2008 to 7/23/2008

WATER OBSERVATIONS:
Water seepage at: 16.0'
Water level at completion: 13.6' (includes drilling water)

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

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

Depth (ft)	Elev. (ft) 686.7	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 COARSE AND FINE SAND (A-3a), little gravel, little to some silty clay; wet.							
55		41 50/6	10	19			@ 53.5', 0.7 feet sand heave.							50+
60		24 39 45	13	20										86
62.0	674.7						Very dense gray FINE SAND (A-3), trace silt, trace gravel; wet.							
65		16 31 41	15	21										73
67.0	669.7						Very dense gray GRAVEL WITH SAND (A-1-b), trace to little silty clay; wet.							
70		33 50/5	9	22				49	29	---	13	--9--		50+
75	661.7	43 50/3	9	23										50+

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

LOG OF: Boring B-030-0-08 | Location: Sta. 194+14.54, 24.17' LT., BL I-70 WB | Date Drilled: 7/20/2008 to 7/23/2008

WATER OBSERVATIONS:
Water seepage at: 16.0'
Water level at completion: 13.6' (includes drilling water)

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

Depth (ft)	Elev. (ft) 661.7	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 to little silty clay; wet.							
80		35 50/5	10	24			@ 78.5', 1.0 foot sand heave.							50+
85		21 47 47	13	25			@ 83.5', 4.6 feet sand heave.	15	39	---	33	--13--		96
90		15 23 46	9	26										70
95		37 50/3	9	27										50+
100	636.7	38 50/5	7	28										50+

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-030-0-08 | *Location:* Sta. 194+14.54, 24.17' LT., BL I-70 WB | *Date Drilled:* 7/20/2008 to 7/23/2008

WATER OBSERVATIONS:
Water seepage at: 16.0'
Water level at completion: 13.6' (includes drilling water)

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

Depth (ft)	Elev. (ft) 636.7	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, %
103.5	633.2						Very dense gray GRAVEL WITH SAND (A-1-b), trace to little silty clay; wet. @ 100.0'-103.5', 25.0 to 27.3 feet sand heave; washed out twice.								
105.0	631.7	50/3	3	29			Severely weathered gray SHALE.								50+
		Core 60"	Rec 57"	RQD 37%	R1		Shale, dark gray, highly weathered, weak, laminated, slightly calcareous, contains abundant pyritic inclusions, fissile, laminated, fractured to highly fractured, RQD 38%, Loss 4%. @ 107.7'-107.8', encounterd large pyritic inclusion. @ 108.1'-109.1', moderately weathered, moderately strong.								
111.0	625.7	Core 12"	Rec 12"	RQD 42%	R2		@ 111.0', core steel damaged. Abandoned boring at 111.0'.								
							Bottom of Boring - 111.0'								

PL ⊢⊣ LL
Blows per foot - ○ / Non-Plastic - NP
10 20 30 40

105 110 115 120 125

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-14.05 PROJECT 4B	DRILLING FIRM / OPERATOR: RII / S.B.	DRILL RIG: CME 55 (SN 386345)	STATION / OFFSET: 194+37.05 / 70' RT
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: RII / C.D.	HAMMER: CME AUTOMATIC	ALIGNMENT: I-70 EB
PID: 96053 BR ID: NA	DRILLING METHOD: 3.25" - HSA	CALIBRATION DATE: 10/20/14	ELEVATION: 748.9 (MSL) EOB: 59.4 ft.
START: 12/2/15 END: 12/3/15	SAMPLING METHOD: SPT	ENERGY RATIO (%): 92	COORD: 39.952814, -82.998014

EXPLORATION ID **B-030-1-15**

MATERIAL DESCRIPTION AND NOTES	ELEV. 748.9
0.5' - ASPHALT (6.0")	748.4
1.0' - CONCRETE (12.0")	747.4
0.5' - AGGREGATE BASE (6.0")	746.9
VERY STIFF, BROWN **SANDY SILT**, SOME FINE GRAVEL, LITTLE CLAY, DAMP.	745.4
DENSE TO VERY DENSE, BROWN TO BROWNISH GRAY **GRAVEL WITH SAND AND SILT**, TRACE CLAY, MOIST. -COBBLES PRESENT THROUGHOUT	738.4
HARD, GRAY **SILT AND CLAY**, SOME COARSE TO FINE SAND, LITTLE TO SOME FINE GRAVEL, DAMP.	729.2
VERY DENSE, GRAY TO BROWN **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, DAMP TO MOIST. -COBBLES PRESENT @ 22.0' -WATER ADDED TO AUGERS @ 28.5'	

DEPTHS: 1–29 (W @ 28.5')

SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
7 8 11	29	100	SS-1	4.00	25	16	12	27	20	24	15	9	12	A-4a (2)	
8 12 9	32	67	SS-2	-	-	-	-	-	-	-	-	-	7	A-2-4 (V)	
9 15 16	48	100	SS-3	-	49	21	9	14	7	24	17	7	8	A-2-4 (0)	
19 20 16	55	100	SS-4	-	-	-	-	-	-	-	-	-	8	A-2-4 (V)	
26 42 37	121	33	SS-5	4.5+	22	9	12	31	26	26	13	13	9	A-6a (6)	
10 19 24	66	0	SS-6	-	-	-	-	-	-	-	-	-	-	A-6a (V)	
45	-	100	3S-6A	4.5+	-	-	-	-	-	-	-	-	10	A-6a (V)	
11 14 17	48	100	SS-7	4.5+	-	-	-	-	-	-	-	-	10	A-6a (V)	
6 16 50/3"	-	100	SS-8	4.5+	12	9	13	38	28	25	13	12	10	A-6a (7)	
				-	-	-	-	-	-	-	-	-	5	A-1-b (V)	
22 32 37	106	100	SS-9	-	23	34	29	9	5	NP	NP	NP	4	A-1-b (0)	
24 31 29	92	100	SS-10	-	-	-	-	-	-	-	-	-	12	A-1-b (V)	

PID: 96053	BR ID: NA	PROJECT: FRA-70-14.05 PROJECT 4B	STATION / OFFSET: 194+37.05 / 70 RT	START: 12/2/15	END: 12/3/15	PG 2 OF 2	B-030-1-15

MATERIAL DESCRIPTION AND NOTES	ELEV. 718.9	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
VERY DENSE, GRAY TO BROWN **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, DAMP TO MOIST. *(same as above)*	711.9	31–37	5 19 30	75	100	SS-11	-	43	26	17	10	4	NP	NP	NP	10	A-1-b (0)	
VERY DENSE, GRAY **SILT**, "AND" COARSE TO FINE SAND, TRACE CLAY, TRACE FINE GRAVEL, MOIST.	706.9	37–42	5 25 33	89	100	SS-12	-	1	0	38	54	7	NP	NP	NP	17	A-4b (5)	
HARD, GRAY **SANDY SILT**, LITTLE FINE GRAVEL, LITTLE CLAY, MOIST.		42–52	36 48 50/4"	-	100	SS-13	4.5+	-	-	-	-	-	-	-	-	9	A-4a (V)	
	696.9		24 50/5"	-	100	SS-14	4.5+	16	13	23	37	11	18	13	5	10	A-4a (3)	
MEDIUM DENSE TO VERY DENSE, GRAY **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, MOIST TO WET. -HEAVING SANDS ENCOUNTERED @ 53.5'		52–59	1 2 8	15	78	SS-15	-	-	-	-	-	-	-	-	-	17	A-1-b (V)	
	689.5	EOB	14 50/5"	-	100	SS-16	-	33	47	12	6	2	NP	NP	NP	11	A-1-b (0)	

NOTES: GROUNDWATER INITIALLY ENCOUNTERED 28.5'

ABANDONMENT METHODS, MATERIALS, QUANTITIES: COMPACTED WITH THE AUGER 200 LBS BENTONITE CHIPS AND SOIL CUTTINGS

APPENDIX IV

HISTORIC BORING LOGS:

B-001-0-59, B-007-0-59 and B-011-0-59

B-001-0-59

SHEET 4

STATE OF OHIO
DEPARTMENT OF HIGHWAYS
TESTING LABORATORY

LOG OF BORING

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

LOCATION: T.H. 1 STA. 48+87 OFFSET 43' Rt. FED. NO.

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
764.6	0			
	2			
	4			
759.6	6	8/10	20655	Brown Silty Sandy Gravel
	8			
754.6	10	16/18	20656	Brown Silty Sandy Gravel
752.1	12	8/10	20657	Brown Gravelly Sandy Silt
749.6	14	16/15	20658	Brown Silty Gravelly Sand W/Boulders
747.1	16	23/28	20659	Brown Silty Sandy Gravel
744.6	18, 20	75/38*	20660	Brown Silty Sandy Gravel W/Boulders
742.1	22	45/50	20661	Brown Silty Sandy Gravel W/Boulders
739.6	24, 26	9/7	20662	Gray Gravel
737.1	28	14/16	20663	Gray Gravelly Sandy Silt
734.6	30	11/16	20664	Gray Gravelly Sandy Silt
732.1	32, 34	13/23	20665	Gray Clayey Gravelly Sand
729.6	36	16/21	-----	Gray Clayey Gravelly Sand

BM
6-25-59

LOG OF BORING (CONTINUED)

BRIDGE NO. FRA-40-1310 T.H. 1

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
727.1	38	50/*	20666	Gray Silty Gravelly Sand
724.6	40	20/41	20667	Gray Sandy Gravel W/Boulders
	42			
	44			
719.6	46	33/64	20668	Gray Sandy Silt
	48			
714.6	50	33/50	20669	Gray Silty Gravelly Sand
	52			
	54			
709.6				
708.6	56	27/55	20670	Gray Silty Sandy Gravel
	58			BOTTOM OF BORING
	60			
	62			* Refusal
	64			
	66			
	68			
	70			
	72			
	74			
	76			
	78			
	80			
	82			

SHEET 6

STATE OF OHIO
DEPARTMENT OF HIGHWAYS
TESTING LABORATORY

LOG OF BORING

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

LOCATION: T.H. 7 STA. 50+09 OFFSET 43' RT FED. NO.

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
767.7	0			
	2			
	4			
762.7	6	11/6	20624	Brown Sandy Gravelly Silt
	8			
757.7	10			
	12	9/14	20625	Brown Sandy Gravel
	14			
752.7	16	12/12	20626	Brown Sandy Gravel W/Boulders
	18			
747.7	20			
	22	21/26	20627	Brown Silty Sandy Gravel W/Boulders
	24			
742.7	26	23/30	20628	Brownish-Gray Gravel W/Boulders
	28			
737.7	30			
	32	32/37	20629	Gray Silty Gravel W/Boulders
	34			
732.7	36	30/50	20630	Gray Sandy Silt

BM
6.25-59

B-007-0-59

LOG OF BORING (CONTINUED)

SHEET 7

BRIDGE NO. FRA-40-1310 T.H. 7

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
730.2	38			
		23/37	20631	Gray Gravelly Sandy Silt
727.7	40			
		33/*	20632	Gray Silty Sandy Gravel W/Boulders
	42			
725.2		75/*	20633	Brownish-Gray Sandy Gravel W/Boulders
	44			
722.7	46	125/*	20634	Gray Gravel W/Boulders
720.2	48	23/35	20635	Gray Sandy Silt W/Boulders
717.7	50			
		50/85	20636	Gray Sandy Silt W/Boulders
	52			
715.2		35/50	20637	Gray Gravelly Sandy Silt W/Boulders
	54			
712.7	56	40/60	20638	Gray Gravelly Sandy Silt W/Boulders
	58			
707.7	60			
		20/45	20639	Gray Silty Sandy Gravel W/Boulders
	62			
	64			
702.7 702.5	66	100/*	20640	Gray Gravel W/Boulders
				BOTTOM OF BORING
	68			
	70			*Refusal
	72			
	74			
	76			
	78			
	80			
	82			

B-011-0-59

SHEET 8

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

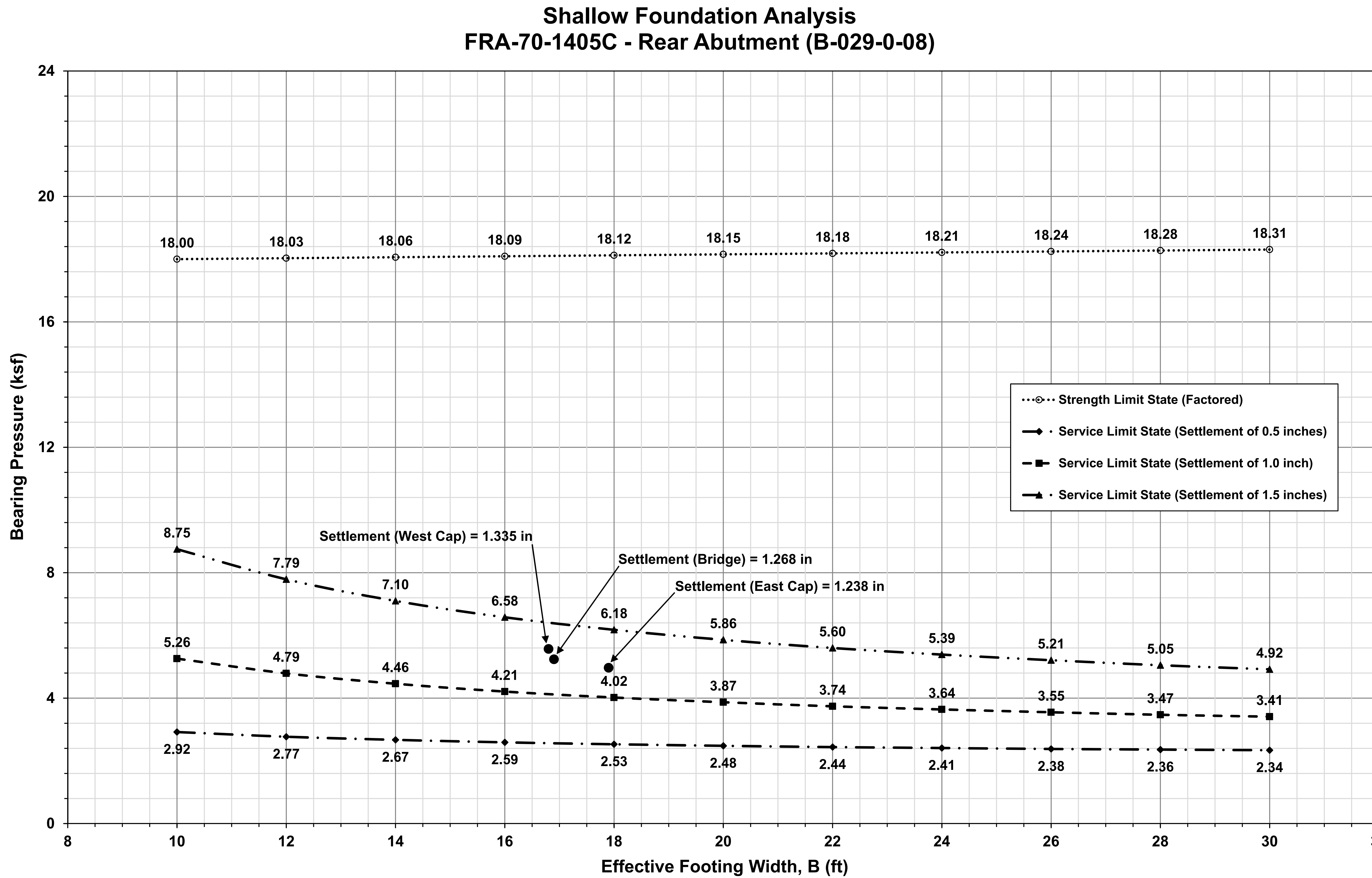
LOG OF BORING (CONTINUED)

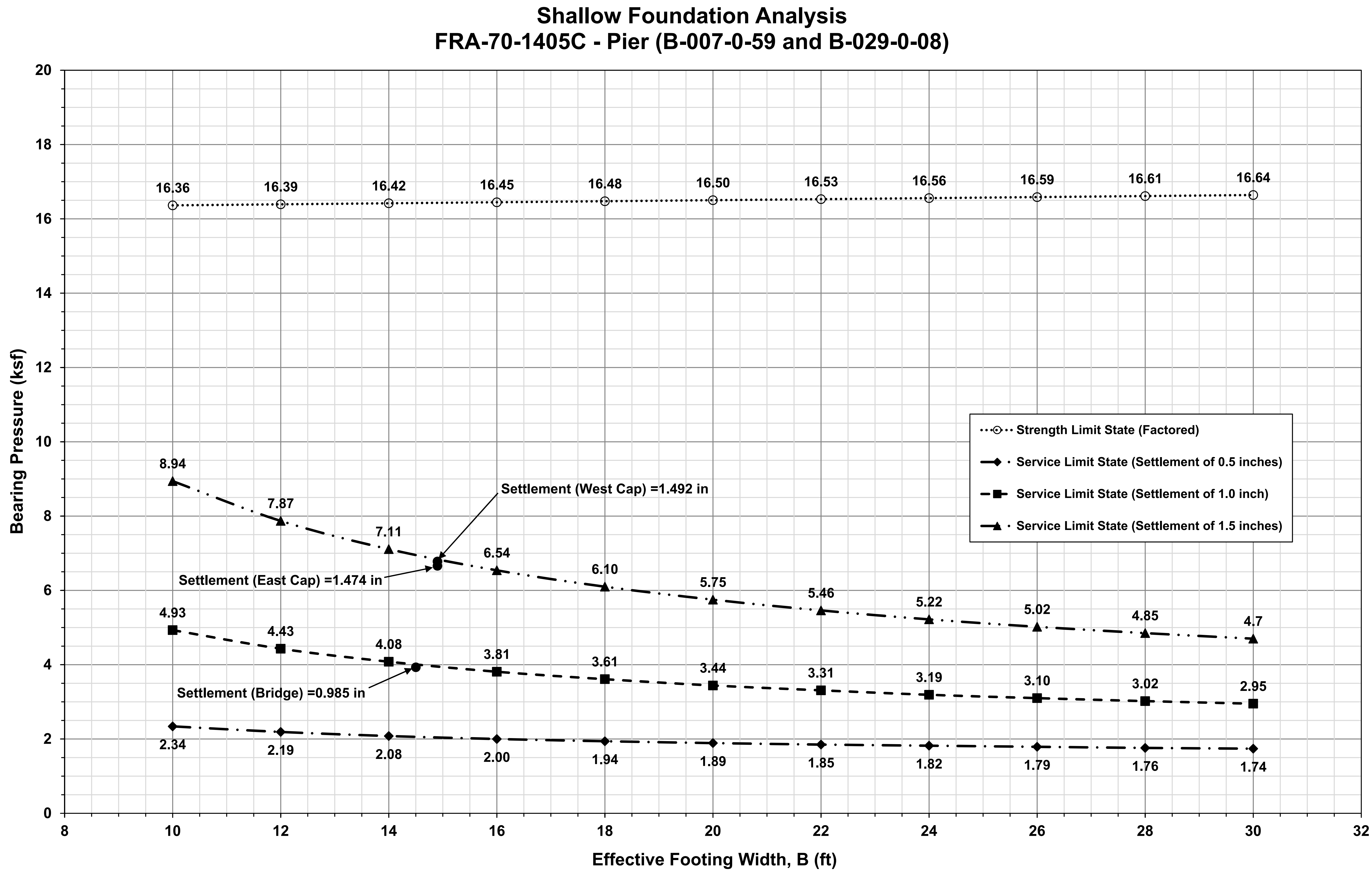
BRIDGE NO. FRA-40-1310 T.H. 11

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 V

BEARING RESISTANCE CHART


Shallow Foundation Analysis
FRA-70-1405C - Rear Abutment (B-029-0-08)
Bearing Pressure (ksf)
Effective Footing Width, B (ft)
Strength Limit State (Factored)
Service Limit State (Settlement of 0.5 inches)
Service Limit State (Settlement of 1.0 inch)
Service Limit State (Settlement of 1.5 inches)
Settlement (West Cap) = 1.335 in
Settlement (Bridge) = 1.268 in
Settlement (East Cap) = 1.238 in
18.00
18.03
18.06
18.09
18.12
18.15
18.18
18.21
18.24
18.28
18.31
8.75
7.79
7.10
6.58
6.18
5.86
5.60
5.39
5.21
5.05
4.92
5.26
4.79
4.46
4.21
4.02
3.87
3.74
3.64
3.55
3.47
3.41
2.92
2.77
2.67
2.59
2.53
2.48
2.44
2.41
2.38
2.36
2.34



Shallow Foundation Analysis
FRA-70-1405C - Pier (B-007-0-59 and B-029-0-08)
Bearing Pressure (ksf)
Effective Footing Width, B (ft)
0
2
4
6
8
10
12
14
16
18
20
8
10
12
14
16
18
20
22
24
26
28
30
32
Strength Limit State (Factored)
Service Limit State (Settlement of 0.5 inches)
Service Limit State (Settlement of 1.0 inch)
Service Limit State (Settlement of 1.5 inches)
16.36
16.39
16.42
16.45
16.48
16.50
16.53
16.56
16.59
16.61
16.64
8.94
7.87
7.11
6.54
6.10
5.75
5.46
5.22
5.02
4.85
4.7
4.93
4.43
4.08
3.81
3.61
3.44
3.31
3.19
3.10
3.02
2.95
2.34
2.19
2.08
2.00
1.94
1.89
1.85
1.82
1.79
1.76
1.74
Settlement (West Cap) =1.492 in
Settlement (East Cap) =1.474 in
Settlement (Bridge) =0.985 in

APPENDIX VI

SHALLOW FOUNDATION CALCULATIONS

W-13-045 - FRA-70-14.05C Project 4H - FRA-70-1405C S. High Street over I-70/71
Shallow Foundation Analysis - Rear Abutment - West Cap

Calculated By: BRT Date: 7/3/2022
Checked By: JPS Date: 7/5/2022

Boring B-029-0-08

B =	16.8	ft	Effective Footing width
D_w =	0.0	ft	Depth below bottom of footing
q =	5,570	psf	Service limit bearing pressure at bottom of wall
q_{net} =	3,890	psf	Net bearing pressure at bottom of wall (considers initial overburden stress of 1,680 psf from 14.0-foot cut to bottom of footing elevation)

Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
A-1-a	G	0.0	5.0	5.0	2.5	135	675	338	182	4,182					90	162	300	0.15	0.990	3,851	4,032	0.022	0.269
A-1-a	G	5.0	10.0	5.0	7.5	135	1,350	1,013	545	4,545					90	129	300	0.45	0.852	3,315	3,860	0.014	0.170
A-4a	C	10.0	15.0	5.0	12.5	130	2,000	1,675	895	4,895	19	0.081	0.008	0.420				0.74	0.671	2,612	3,507	0.017	0.203
A-3a	G	15.0	18.0	3.0	16.5	135	2,405	2,203	1,173	5,173					55	65	191	0.98	0.557	2,167	3,340	0.007	0.085
A-4a	C	18.0	23.0	5.0	20.5	130	3,055	2,730	1,451	5,451	22	0.108	0.011	0.444				1.22	0.471	1,832	3,283	0.013	0.159
A-4a	C	23.0	28.5	5.5	25.8	130	3,770	3,413	1,806	5,806	22	0.108	0.011	0.444				1.53	0.388	1,511	3,317	0.011	0.130
A-4a	G	28.5	33.5	5.0	31.0	135	4,445	4,108	2,173	6,173					100	97	155	1.85	0.329	1,280	3,454	0.006	0.078
A-1-b	G	33.5	38.5	5.0	36.0	135	5,120	4,783	2,536	6,536					100	92	300	2.14	0.287	1,116	3,652	0.003	0.032
A-1-b	G	38.5	43.5	5.0	41.0	135	5,795	5,458	2,899	6,899					100	88	300	2.44	0.254	987	3,886	0.002	0.025
A-3a	G	43.5	51.5	8.0	47.5	135	6,875	6,335	3,371	7,371					100	83	266	2.83	0.221	858	4,229	0.003	0.036
A-3a	G	51.5	60.0	8.5	55.8	135	8,023	7,449	3,970	7,970					100	77	242	3.32	0.189	735	4,705	0.003	0.031
A-6a	C	60.0	65.0	5.0	62.5	130	8,673	8,348	4,448	8,448	22	0.108	0.011	0.444				3.72	0.169	658	5,105	0.002	0.027
A-6a	C	65.0	70.5	5.5	67.8	130	9,388	9,030	4,802	8,802	22	0.108	0.011	0.444				4.03	0.156	608	5,410	0.002	0.026
A-1-b	G	70.5	78.5	8.0	74.5	135	10,468	9,928	5,279	9,279					100	68	255	4.43	0.142	554	5,832	0.001	0.016
A-1-b	G	78.5	86.5	8.0	82.5	135	11,548	11,008	5,860	9,860					100	64	237	4.91	0.129	501	6,360	0.001	0.014
A-3	G	86.5	93.0	6.5	89.8	135	12,425	11,986	6,386	10,386					100	61	143	5.34	0.118	461	6,847	0.001	0.017
A-3	G	93.0	100.0	7.0	96.5	135	13,370	12,898	6,876	10,876					100	59	137	5.74	0.110	429	7,305	0.001	0.016
																					Total Settlement:		1.335 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 26, FHWA GEC 5
3. C_r = 0.10(C_c); Ref. Section 5.4.2.5 of FHWA GEC 5
4. e_o = (C_c/1.15)+0.35; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_N N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \leq \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (Cohesiv soil layers)
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)

B =	16.1	ft	
L =	225	ft	
c =	6,250	psf	
γ =	130	pcf	
D_f =	5.0	ft	
φ =	0	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$$ = 32.90 ksf

$$N_{cm} = N_c s_c i_c$$ = 5.21

$$N_{qm} = N_q s_q d_q i_q$$ = 1.00

$$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$$ = 0.00

N_c =	5.14	s_c =	1+(16.1 ft/225 ft)(1/5.14) =	1.014	i_c =	1.000	d_q =	1+2tan(0°)[1-sin(0°)]²tan⁻¹(5 ft/16.1 ft) =	1.000
N_q =	1.00	s_q =	1+(16.1 ft/225 ft)tan(0°) =	1.000	i_q =	1.000	C_{wq} =	0.0 ft < 5.0 ft =	0.500
N_γ =	0.00	s_γ =	1-0.4(16.1 ft/225 ft) =	0.971	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(16.1 ft) + 5 ft =	0.500

$$q_R = q_n \cdot \phi_b$$ = 18.09 ksf

φ_b = 0.55 (Per Table 11.5.7-1, AASHTO LRFD BDS)

W-13-045 - FRA-70-14.05C Project 4H - FRA-70-1405C S. High Street over I-70/71

Shallow Foundation Analysis - Rear Abutment - Bridge Cap

Calculated By: BRT Date: 7/3/2022

Checked By: JPS Date: 7/5/2022

Boring B-029-0-08

B =	16.9	ft	Effective Footing width
D_w =	0.0	ft	Depth below bottom of footing
q =	5,240	psf	Service limit bearing pressure at bottom of wall
q_{net} =	3,560	psf	Net bearing pressure at bottom of wall (considers initial overburden stress of 1,680 psf from 14.0-foot cut to bottom of footing elevation)

Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' [1] (psf)	LL	C_c [2]	C_r [3]	e_o [4]	N_{60}	$(N1)_{60}$ [5]	C' [6]	Z_f/B	I [7]	$\Delta\sigma_v$ [8] (psf)	σ_{vf}' Midpoint (psf)	S_c [9,10] (ft)	S_c (in)
A-1-a	G	0.0	5.0	5.0	2.5	135	675	338	182	4,182					90	162	300	0.15	0.990	3,525	3,706	0.022	0.262
A-1-a	G	5.0	10.0	5.0	7.5	135	1,350	1,013	545	4,545					90	129	300	0.44	0.854	3,040	3,584	0.014	0.164
A-4a	C	10.0	15.0	5.0	12.5	130	2,000	1,675	895	4,895	19	0.081	0.008	0.420				0.74	0.674	2,399	3,294	0.016	0.194
A-3a	G	15.0	18.0	3.0	16.5	135	2,405	2,203	1,173	5,173					55	65	191	0.98	0.560	1,992	3,165	0.007	0.081
A-4a	C	18.0	23.0	5.0	20.5	130	3,055	2,730	1,451	5,451	22	0.108	0.011	0.444				1.21	0.473	1,685	3,135	0.013	0.150
A-4a	C	23.0	28.5	5.5	25.8	130	3,770	3,413	1,806	5,806	22	0.108	0.011	0.444				1.52	0.390	1,390	3,196	0.010	0.122
A-4a	G	28.5	33.5	5.0	31.0	135	4,445	4,108	2,173	6,173					100	97	155	1.83	0.331	1,178	3,351	0.006	0.073
A-1-b	G	33.5	38.5	5.0	36.0	135	5,120	4,783	2,536	6,536					100	92	300	2.13	0.288	1,027	3,563	0.002	0.030
A-1-b	G	38.5	43.5	5.0	41.0	135	5,795	5,458	2,899	6,899					100	88	300	2.43	0.255	909	3,808	0.002	0.024
A-3a	G	43.5	51.5	8.0	47.5	135	6,875	6,335	3,371	7,371					100	83	266	2.81	0.222	790	4,161	0.003	0.033
A-3a	G	51.5	60.0	8.5	55.8	135	8,023	7,449	3,970	7,970					100	77	242	3.30	0.190	677	4,647	0.002	0.029
A-6a	C	60.0	65.0	5.0	62.5	130	8,673	8,348	4,448	8,448	22	0.108	0.011	0.444				3.70	0.170	605	5,053	0.002	0.025
A-6a	C	65.0	70.5	5.5	67.8	130	9,388	9,030	4,802	8,802	22	0.108	0.011	0.444				4.01	0.157	560	5,362	0.002	0.024
A-1-b	G	70.5	78.5	8.0	74.5	135	10,468	9,928	5,279	9,279					100	68	255	4.41	0.143	510	5,788	0.001	0.015
A-1-b	G	78.5	86.5	8.0	82.5	135	11,548	11,008	5,860	9,860					100	64	237	4.88	0.130	461	6,321	0.001	0.013
A-3	G	86.5	93.0	6.5	89.8	135	12,425	11,986	6,386	10,386					100	61	143	5.31	0.119	424	6,810	0.001	0.015
A-3	G	93.0	100.0	7.0	96.5	135	13,370	12,898	6,876	10,876					100	59	137	5.71	0.111	395	7,271	0.001	0.015
																					Total Settlement:		1.268 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 26, FHWA GEC 5
3. $C_r = 0.10(C_c)$; Ref. Section 5.4.2.5 of FHWA GEC 5
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_N N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \leq \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (Cohesiv soil layers)
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)

B =	16.1	ft	
L =	225	ft	
c =	6,250	psf	
γ =	130	pcf	
D_f =	5.0	ft	
φ =	0	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma} = 32.90 \text{ ksf}$$

$$N_{cm} = N_c s_c i_c = 5.21 \qquad N_{qm} = N_q s_q d_q i_q = 1.00 \qquad N_{\gamma m} = N_\gamma s_\gamma i_\gamma = 0.00$$

N_c =	5.14	s_c =	1+(16.1 ft/225 ft)(1/5.14) =	1.014	i_c =	1.000	d_q =	1+2tan(0°)[1-sin(0°)]²tan⁻¹(5 ft/16.1 ft) =	1.000
N_q =	1.00	s_q =	1+(16.1 ft/225 ft)tan(0°) =	1.000	i_q =	1.000	C_{wq} =	0.0 ft < 5.0 ft =	0.500
N_γ =	0.00	s_γ =	1-0.4(16.1 ft/225 ft) =	0.971	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(16.1 ft) + 5 ft =	0.500

$$q_R = q_n \cdot \phi_b = 18.09 \text{ ksf}$$

φ_b = 0.55 (Per Table 11.5.7-1, AASHTO LRFD BDS)

W-13-045 - FRA-70-14.05C Project 4H - FRA-70-1405C S. High Street over I-70/71

Shallow Foundation Analysis - Rear Abutment - East Cap

Calculated By: BRT Date: 7/3/2022

Checked By: JPS Date: 7/5/2022

Boring B-029-0-08

B =	17.9	ft	Effective Footing width
D_w =	0.0	ft	Depth below bottom of footing
q =	4,970	psf	Service limit bearing pressure at bottom of wall
q_{net} =	3,290	psf	Net bearing pressure at bottom of wall (considers initial overburden stress of 1,680 psf from 14.0-foot cut to bottom of footing elevation)

Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' [1] (psf)	LL	C_c [2]	C_r [3]	e_o [4]	N_{60}	$(N1)_{60}$ [5]	C' [6]	Z_f/B	I [7]	$\Delta\sigma_v$ [8] (psf)	σ_{vf}' Midpoint (psf)	S_c [9,10] (ft)	S_c (in)
A-1-a	G	0.0	5.0	5.0	2.5	135	675	338	182	4,182					90	162	300	0.14	0.992	3,262	3,444	0.021	0.256
A-1-a	G	5.0	10.0	5.0	7.5	135	1,350	1,013	545	4,545					90	129	300	0.42	0.869	2,860	3,405	0.013	0.159
A-4a	C	10.0	15.0	5.0	12.5	130	2,000	1,675	895	4,895	19	0.081	0.008	0.420				0.70	0.697	2,293	3,188	0.016	0.189
A-3a	G	15.0	18.0	3.0	16.5	135	2,405	2,203	1,173	5,173					55	65	191	0.92	0.583	1,919	3,092	0.007	0.079
A-4a	C	18.0	23.0	5.0	20.5	130	3,055	2,730	1,451	5,451	22	0.108	0.011	0.444				1.15	0.496	1,630	3,081	0.012	0.147
A-4a	C	23.0	28.5	5.5	25.8	130	3,770	3,413	1,806	5,806	22	0.108	0.011	0.444				1.44	0.410	1,350	3,156	0.010	0.120
A-4a	G	28.5	33.5	5.0	31.0	135	4,445	4,108	2,173	6,173					100	97	155	1.73	0.349	1,147	3,320	0.006	0.071
A-1-b	G	33.5	38.5	5.0	36.0	135	5,120	4,783	2,536	6,536					100	92	300	2.01	0.304	1,001	3,537	0.002	0.029
A-1-b	G	38.5	43.5	5.0	41.0	135	5,795	5,458	2,899	6,899					100	88	300	2.29	0.269	887	3,786	0.002	0.023
A-3a	G	43.5	51.5	8.0	47.5	135	6,875	6,335	3,371	7,371					100	83	266	2.65	0.234	771	4,142	0.003	0.032
A-3a	G	51.5	60.0	8.5	55.8	135	8,023	7,449	3,970	7,970					100	77	242	3.11	0.201	661	4,631	0.002	0.028
A-6a	C	60.0	65.0	5.0	62.5	130	8,673	8,348	4,448	8,448	22	0.108	0.011	0.444				3.49	0.180	592	5,039	0.002	0.024
A-6a	C	65.0	70.5	5.5	67.8	130	9,388	9,030	4,802	8,802	22	0.108	0.011	0.444				3.78	0.166	547	5,349	0.002	0.023
A-1-b	G	70.5	78.5	8.0	74.5	135	10,468	9,928	5,279	9,279					100	68	255	4.16	0.152	498	5,777	0.001	0.015
A-1-b	G	78.5	86.5	8.0	82.5	135	11,548	11,008	5,860	9,860					100	64	237	4.61	0.137	451	6,310	0.001	0.013
A-3	G	86.5	93.0	6.5	89.8	135	12,425	11,986	6,386	10,386					100	61	143	5.01	0.126	415	6,801	0.001	0.015
A-3	G	93.0	100.0	7.0	96.5	135	13,370	12,898	6,876	10,876					100	59	137	5.39	0.117	386	7,262	0.001	0.015
																				Total Settlement:			1.238 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 26, FHWA GEC 5
3. $C_r = 0.10(C_c)$; Ref. Section 5.4.2.5 of FHWA GEC 5
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_n N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \leq \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (Cohesiv soil layers)
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)

B =	17.4	ft	
L =	225	ft	
c =	6,250	psf	
γ =	130	pcf	
D_f =	5.0	ft	
φ =	0	deg	
D_w =	0.0	ft	Below ground surface

$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \frac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$ = 32.93 ksf

$N_{cm} = N_c s_c i_c$ = 5.22

$N_{qm} = N_q s_q d_q i_q$ = 1.00

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 0.00

N_c =	5.14	s_c =	1+(17.4 ft/225 ft)(1/5.14) =	1.015	i_c =	1.000	d_q =	1+2tan(0°)[1-sin(0°)]²tan⁻¹(5 ft/17.4 ft) =	1.000
N_q =	1.00	s_q =	1+(17.4 ft/225 ft)tan(0°) =	1.000	i_q =	1.000	C_{wq} =	0.0 ft < 5.0 ft =	0.500
N_γ =	0.00	s_γ =	1-0.4(17.4 ft/225 ft) =	0.969	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(17.4 ft) + 5 ft =	0.500

$q_R = q_n \cdot \phi_b$ = 18.11 ksf

φ_b = 0.55 (Per Table 11.5.7-1, AASHTO LRFD BDS)

W-13-045 - FRA-70-14.05C Project 4H - FRA-70-1405C S. High Street over I-70/71
Shallow Foundation Analysis - Pier - West Cap

Calculated By: BRT Date: 7/3/2022
Checked By: JPS Date: 7/5/2022

Borings B-007-0-59 and B-029-0-08

B =	14.9	ft	Effective Footing width
D_w =	0.0	ft	Depth below bottom of footing
q =	6,780	psf	Service limit bearing pressure at bottom of wall
q_{net} =	5,760	psf	Net bearing pressure at bottom of wall (considers initial overburden stress of 1,020 psf from 8.5-foot cut to bottom of footing elevation)

Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
A-1-a	G	0.0	7.0	7.0	3.5	135	945	473	254	4,254					100	169	300	0.23	0.965	5,561	5,815	0.032	0.381
A-4b	G	7.0	11.0	4.0	9.0	135	1,485	1,215	653	4,653					96	132	207	0.60	0.753	4,337	4,990	0.017	0.205
A-4a	C	11.0	17.0	6.0	14.0	130	2,265	1,875	1,001	5,001	20	0.090	0.009	0.428				0.94	0.575	3,314	4,315	0.024	0.288
A-1-b	G	17.0	22.0	5.0	19.5	135	2,940	2,603	1,386	5,386					65	73	286	1.31	0.445	2,561	3,946	0.008	0.095
A-1-a	G	22.0	24.5	2.5	23.3	130	3,265	3,103	1,652	5,652					100	107	300	1.56	0.382	2,203	3,854	0.003	0.037
A-4a	C	24.5	26.5	2.0	25.5	130	3,525	3,395	1,804	5,804	22	0.108	0.011	0.444				1.71	0.352	2,029	3,833	0.005	0.059
A-4a	G	26.5	31.5	5.0	29.0	135	4,200	3,863	2,053	6,053					100	99	158	1.95	0.314	1,806	3,859	0.009	0.104
A-1-b	G	31.5	36.5	5.0	34.0	135	4,875	4,538	2,416	6,416					100	94	300	2.28	0.270	1,558	3,974	0.004	0.043
A-1-b	G	36.5	41.5	5.0	39.0	135	5,550	5,213	2,779	6,779					100	89	300	2.62	0.237	1,368	4,147	0.003	0.035
A-3a	G	41.5	49.5	8.0	45.5	135	6,630	6,090	3,251	7,251					100	84	272	3.05	0.205	1,180	4,431	0.004	0.047
A-3a	G	49.5	58.0	8.5	53.8	135	7,778	7,204	3,850	7,850					100	78	246	3.61	0.174	1,004	4,853	0.003	0.042
A-6a	C	58.0	63.0	5.0	60.5	130	8,428	8,103	4,327	8,327	22	0.108	0.011	0.444				4.06	0.155	894	5,221	0.003	0.037
A-6a	C	63.0	68.5	5.5	65.8	130	9,143	8,785	4,682	8,682	22	0.108	0.011	0.444				4.41	0.143	824	5,506	0.003	0.035
A-1-b	G	68.5	76.5	8.0	72.5	135	10,223	9,683	5,159	9,159					100	68	259	4.87	0.130	748	5,907	0.002	0.022
A-1-b	G	76.5	84.5	8.0	80.5	135	11,303	10,763	5,739	9,739					100	65	240	5.40	0.117	675	6,414	0.002	0.019
A-3	G	84.5	91.0	6.5	87.8	135	12,180	11,741	6,266	10,266					100	62	144	5.89	0.108	620	6,885	0.002	0.022
A-3	G	91.0	98.0	7.0	94.5	135	13,125	12,653	6,756	10,756					100	59	138	6.34	0.100	576	7,331	0.002	0.022
																					Total Settlement:		1.492 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 26, FHWA GEC 5
3. $C_r = 0.10(C_c)$; Ref. Section 5.4.2.5 of FHWA GEC 5
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_N N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \le 2.0$ 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
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 (Cohesiv soil layers)
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)

B =	14.9	ft	
L =	225	ft	
c =	6,250	psf	
γ =	130	pcf	
D_f =	5.0	ft	
φ =	0	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$$ = 32.86 ksf

$$N_{cm} = N_c s_c i_c$$ = 5.21

$$N_{qm} = N_q s_q d_q i_q$$ = 1.00

$$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$$ = 0.00

N_c =	5.14	s_c =	1+(14.9 ft/225 ft)(1/5.14) =	1.013	i_c =	1.000	d_q =	1+2tan(0°)[1-sin(0°)]²tan⁻¹(5 ft/14.9 ft) =	1.000
N_q =	1.00	s_q =	1+(14.9 ft/225 ft)tan(0°) =	1.000	i_q =	1.000	C_{wq} =	0.0 ft < 5.0 ft =	0.500
N_γ =	0.00	s_γ =	1-0.4(14.9 ft/225 ft) =	0.974	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(14.9 ft) + 5 ft =	0.500

$$q_R = q_n \cdot \phi_b$$ = 16.43 ksf

φ_b = 0.5 (Per Table 10.5.5.2.2-1, AASHTO LRFD BDS)

W-13-045 - FRA-70-14.05C Project 4H - FRA-70-1405C S. High Street over I-70/71
Shallow Foundation Analysis - Pier - Bridge

Calculated By: BRT Date: 7/3/2022
Checked By: JPS Date: 7/5/2022

Borings B-007-0-59 and B-029-0-08

B =	14.5	ft	Effective Footing width
D_w =	0.0	ft	Depth below bottom of footing
q =	3,930	psf	Service limit bearing pressure at bottom of wall
q_{net} =	2,910	psf	Net bearing pressure at bottom of wall (considers initial overburden stress of 1,020 psf from 8.5-foot cut to bottom of footing elevation)

Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
A-1-a	G	0.0	7.0	7.0	3.5	135	945	473	254	4,254					100	169	300	0.24	0.963	2,802	3,056	0.025	0.302
A-4b	G	7.0	11.0	4.0	9.0	135	1,485	1,215	653	4,653					96	132	207	0.62	0.743	2,161	2,815	0.012	0.147
A-4a	C	11.0	17.0	6.0	14.0	130	2,265	1,875	1,001	5,001	20	0.090	0.009	0.428				0.97	0.564	1,642	2,643	0.016	0.191
A-1-b	G	17.0	22.0	5.0	19.5	135	2,940	2,603	1,386	5,386					65	73	286	1.34	0.435	1,265	2,650	0.005	0.059
A-1-a	G	22.0	24.5	2.5	23.3	130	3,265	3,103	1,652	5,652					100	107	300	1.60	0.373	1,086	2,738	0.002	0.022
A-4a	C	24.5	26.5	2.0	25.5	130	3,525	3,395	1,804	5,804	22	0.108	0.011	0.444				1.76	0.344	1,000	2,804	0.003	0.034
A-4a	G	26.5	31.5	5.0	29.0	135	4,200	3,863	2,053	6,053					100	99	158	2.00	0.306	890	2,943	0.005	0.059
A-1-b	G	31.5	36.5	5.0	34.0	135	4,875	4,538	2,416	6,416					100	94	300	2.34	0.264	767	3,183	0.002	0.024
A-1-b	G	36.5	41.5	5.0	39.0	135	5,550	5,213	2,779	6,779					100	89	300	2.69	0.231	673	3,452	0.002	0.019
A-3a	G	41.5	49.5	8.0	45.5	135	6,630	6,090	3,251	7,251					100	84	272	3.14	0.200	581	3,831	0.002	0.025
A-3a	G	49.5	58.0	8.5	53.8	135	7,778	7,204	3,850	7,850					100	78	246	3.71	0.170	494	4,344	0.002	0.022
A-6a	C	58.0	63.0	5.0	60.5	130	8,428	8,103	4,327	8,327	22	0.108	0.011	0.444				4.17	0.151	440	4,767	0.002	0.019
A-6a	C	63.0	68.5	5.5	65.8	130	9,143	8,785	4,682	8,682	22	0.108	0.011	0.444				4.53	0.139	405	5,087	0.001	0.018
A-1-b	G	68.5	76.5	8.0	72.5	135	10,223	9,683	5,159	9,159					100	68	259	5.00	0.126	368	5,527	0.001	0.011
A-1-b	G	76.5	84.5	8.0	80.5	135	11,303	10,763	5,739	9,739					100	65	240	5.55	0.114	332	6,071	0.001	0.010
A-3	G	84.5	91.0	6.5	87.8	135	12,180	11,741	6,266	10,266					100	62	144	6.05	0.105	305	6,570	0.001	0.011
A-3	G	91.0	98.0	7.0	94.5	135	13,125	12,653	6,756	10,756					100	59	138	6.52	0.097	283	7,039	0.001	0.011
																				Total Settlement:			0.985 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 26, FHWA GEC 5
3. $C_r = 0.10(C_c)$; Ref. Section 5.4.2.5 of FHWA GEC 5
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_N N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \leq \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (Cohesiv soil layers)
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)

B =	14.2	ft	
L =	225	ft	
c =	6,250	psf	
γ =	130	pcf	
D_f =	5.0	ft	
φ =	0	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma} = 32.84 \text{ ksf}$$

$$N_{cm} = N_c s_c i_c = 5.20 \qquad N_{qm} = N_q s_q d_q i_q = 1.00 \qquad N_{\gamma m} = N_\gamma s_\gamma i_\gamma = 0.00$$

N_c =	5.14	s_c =	1+(14.2 ft/225 ft)(1/5.14) =	1.012	i_c =	1.000	d_q =	1+2tan(0°)[1-sin(0°)]²tan⁻¹(5 ft/14.2 ft) =	1.000
N_q =	1.00	s_q =	1+(14.2 ft/225 ft)tan(0°) =	1.000	i_q =	1.000	C_{wq} =	0.0 ft < 5.0 ft =	0.500
N_γ =	0.00	s_γ =	1-0.4(14.2 ft/225 ft) =	0.975	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(14.2 ft) + 5 ft =	0.500

$$q_R = q_n \cdot \phi_b = 16.42 \text{ ksf}$$

φ_b = 0.5 (Per Table 10.5.5.2.2-1, AASHTO LRFD BDS)

W-13-045 - FRA-70-14.05C Project 4H - FRA-70-1405C S. High Street over I-70/71
Shallow Foundation Analysis - Pier - East Cap

Calculated By: BRT Date: 7/3/2022
Checked By: JPS Date: 7/5/2022

Borings B-007-0-59 and B-029-0-08

B =	14.9	ft	Effective Footing width
D_w =	0.0	ft	Depth below bottom of footing
q =	6,660	psf	Service limit bearing pressure at bottom of wall
q_{net} =	5,640	psf	Net bearing pressure at bottom of wall (considers initial overburden stress of 1,020 psf from 8.5-foot cut to bottom of footing elevation)

Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
A-1-a	G	0.0	7.0	7.0	3.5	135	945	473	254	4,254					100	169	300	0.23	0.965	5,445	5,699	0.032	0.378
A-4b	G	7.0	11.0	4.0	9.0	135	1,485	1,215	653	4,653					96	132	207	0.60	0.753	4,246	4,900	0.017	0.203
A-4a	C	11.0	17.0	6.0	14.0	130	2,265	1,875	1,001	5,001	20	0.090	0.009	0.428				0.94	0.575	3,245	4,246	0.024	0.285
A-1-b	G	17.0	22.0	5.0	19.5	135	2,940	2,603	1,386	5,386					65	73	286	1.31	0.445	2,507	3,893	0.008	0.094
A-1-a	G	22.0	24.5	2.5	23.3	130	3,265	3,103	1,652	5,652					100	107	300	1.56	0.382	2,157	3,808	0.003	0.036
A-4a	C	24.5	26.5	2.0	25.5	130	3,525	3,395	1,804	5,804	22	0.108	0.011	0.444				1.71	0.352	1,987	3,791	0.005	0.058
A-4a	G	26.5	31.5	5.0	29.0	135	4,200	3,863	2,053	6,053					100	99	158	1.95	0.314	1,768	3,821	0.009	0.102
A-1-b	G	31.5	36.5	5.0	34.0	135	4,875	4,538	2,416	6,416					100	94	300	2.28	0.270	1,525	3,941	0.004	0.043
A-1-b	G	36.5	41.5	5.0	39.0	135	5,550	5,213	2,779	6,779					100	89	300	2.62	0.237	1,339	4,118	0.003	0.034
A-3a	G	41.5	49.5	8.0	45.5	135	6,630	6,090	3,251	7,251					100	84	272	3.05	0.205	1,155	4,406	0.004	0.047
A-3a	G	49.5	58.0	8.5	53.8	135	7,778	7,204	3,850	7,850					100	78	246	3.61	0.174	983	4,833	0.003	0.041
A-6a	C	58.0	63.0	5.0	60.5	130	8,428	8,103	4,327	8,327	22	0.108	0.011	0.444				4.06	0.155	875	5,203	0.003	0.036
A-6a	C	63.0	68.5	5.5	65.8	130	9,143	8,785	4,682	8,682	22	0.108	0.011	0.444				4.41	0.143	807	5,489	0.003	0.034
A-1-b	G	68.5	76.5	8.0	72.5	135	10,223	9,683	5,159	9,159					100	68	259	4.87	0.130	733	5,891	0.002	0.021
A-1-b	G	76.5	84.5	8.0	80.5	135	11,303	10,763	5,739	9,739					100	65	240	5.40	0.117	661	6,400	0.002	0.019
A-3	G	84.5	91.0	6.5	87.8	135	12,180	11,741	6,266	10,266					100	62	144	5.89	0.108	607	6,872	0.002	0.022
A-3	G	91.0	98.0	7.0	94.5	135	13,125	12,653	6,756	10,756					100	59	138	6.34	0.100	564	7,319	0.002	0.021
																				Total Settlement:			1.474 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 26, FHWA GEC 5
3. $C_r = 0.10(C_c)$; Ref. Section 5.4.2.5 of FHWA GEC 5
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_N N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \leq \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (Cohesiv soil layers)
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)

B =	14.9	ft	
L =	225	ft	
c =	6,250	psf	
γ =	130	pcf	
D_f =	5.0	ft	
φ =	0	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$$ = 32.86 ksf

$$N_{cm} = N_c s_c i_c$$ = 5.21

$$N_{qm} = N_q s_q d_q i_q$$ = 1.00

$$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$$ = 0.00

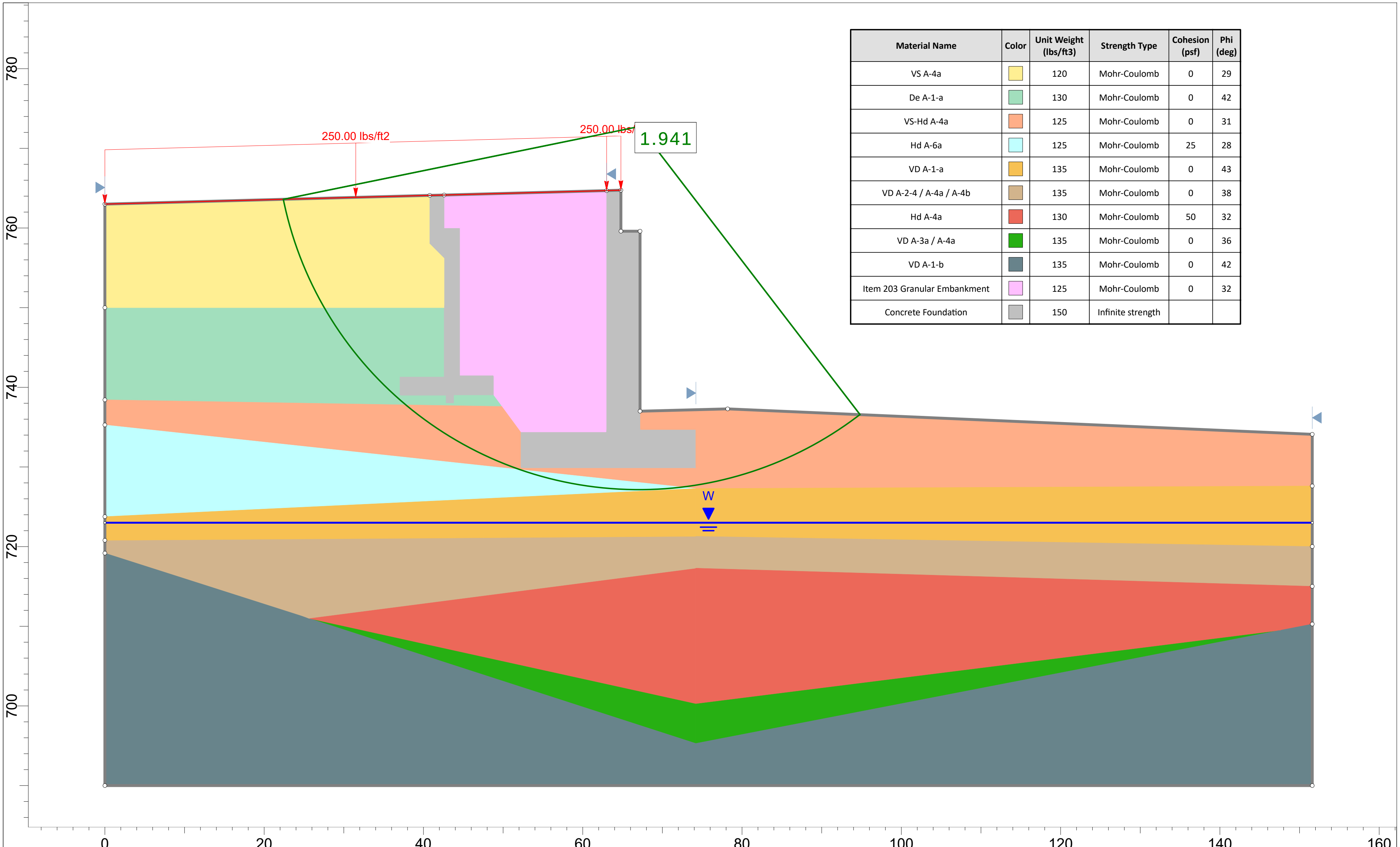
N_c =	5.14	s_c =	1+(14.9 ft/225 ft)(1/5.14) =	1.013	i_c =	1.000	d_q =	1+2tan(0°)[1-sin(0°)]²tan⁻¹(5 ft/14.9 ft) =	1.000
N_q =	1.00	s_q =	1+(14.9 ft/225 ft)tan(0°) =	1.000	i_q =	1.000	C_{wq} =	0.0 ft < 5.0 ft =	0.500
N_γ =	0.00	s_γ =	1-0.4(14.9 ft/225 ft) =	0.974	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(14.9 ft) + 5 ft =	0.500

$$q_R = q_n \cdot \phi_b$$ = 16.43 ksf

φ_b = 0.5 (Per Table 10.5.5.2.2-1, AASHTO LRFD BDS)

APPENDIX VI

GLOBAL (SLOPE) STABILITY ANALYSIS OUTPUT



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
VS A-4a		120	Mohr-Coulomb	0	29
De A-1-a		130	Mohr-Coulomb	0	42
VS-Hd A-4a		125	Mohr-Coulomb	0	31
Hd A-6a		125	Mohr-Coulomb	25	28
VD A-1-a		135	Mohr-Coulomb	0	43
VD A-2-4 / A-4a / A-4b		135	Mohr-Coulomb	0	38
Hd A-4a		130	Mohr-Coulomb	50	32
VD A-3a / A-4a		135	Mohr-Coulomb	0	36
VD A-1-b		135	Mohr-Coulomb	0	42
Item 203 Granular Embankment		125	Mohr-Coulomb	0	32
Concrete Foundation		150	Infinite strength		

Resource International, Inc.
Planning | Engineering | Construction Management | Technology

Project	FRA-70-14.05C Project 4H - FRA-70-1405C - Rear Abutment - Global Stability		
Analysis Description	Rear Abutment - Borings B-001-0-59, B-007-0-59 and B-029-0-08 - Drained - Spencer's		
Drawn By BRT	*Scale* 1:200	*Company*	Resource International, Inc.
Date 6/9/2020		*File Name*	FRA-70-1405C - Rear Abutment - Global Stability.slmd

SLIDEINTERPRET 8.020

APPENDIX VIII

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 Unit Tip Resistance, q_p [10,11] (ksf)	Nominal Unit Side Resistance, q_s [12,13] (ksf)	φ_{qp} [14]	φ_{qs} [15]
B-001-1-59	728.9	7.9	8.0	A-6a	C	1.4	1.4	727.5	125	87	175	4,500	6.2	0.49						B-001-1-59	728.9-727.5	0.0-1.4	27	2.19	0.40	0.45
				A-4a	C	3.9	2.5	725.0	130	337	500	8,000	6.6	0.45							727.5-725.0	1.4-3.9	52	3.60	0.40	0.45
				A-1-a	G	7.9	4.0	721.0	135	770	1,040				61	42	42	19,398	2.58		725.0-721.0	3.9-7.9	60	1.98	0.50	0.55
				A-4b	G	17.9	10.0	711.0	135	1,403	2,390				90	58	44	36,461	2.83		721.0-711.0	7.9-17.9	60	3.97	0.50	0.55
				A-1-b	G	20.9	3.0	708.0	135	1,875	2,795				92	57	44	29,256	1.99		711.0-708.0	17.9-20.9	60	3.72	0.50	0.55
B-007-0-59	728.9	7.9	8.0	A-4a	C	1.3	1.3	727.6	130	84	169	8,000	6.2	0.45						B-007-0-59	728.9-727.6	0.0-1.3	49	3.60	0.40	0.45
				A-1-a	G	8.8	7.5	720.1	135	675	1,181				100	66	44	31,800	4.28		727.6-720.1	1.3-8.8	60	2.89	0.50	0.55
				A-4b	G	13.8	5.0	715.1	135	1,307	1,856				79	49	43	32,850	2.67		720.1-715.1	8.8-13.8	60	3.49	0.50	0.55
				A-4a	C	19.8	6.0	709.1	130	1,691	2,636	8,000	9.0	0.45							715.1-709.1	13.8-19.8	71	3.60	0.40	0.45
				A-1-b	G	24.8	5.0	704.1	135	2,075	3,311				65	38	42	20,670	1.39		709.1-704.1	19.8-24.8	60	2.87	0.50	0.55
				A-1-a	G	27.3	2.5	701.6	135	2,348	3,649				100	57	44	31,800	1.80		704.1-701.6	24.8-27.3	60	4.23	0.50	0.55
B-011-0-59	728.9	7.9	8.0	A-4a	C	7.0	7.0	721.9	135	473	945	8,000	7.1	0.45						B-011-0-59	728.9-721.9	0.0-7.0	56	3.60	0.40	0.45
				A-1-a	G	12.0	5.0	716.9	135	1,183	1,620				67	41	42	21,306	2.06		721.9-716.9	7.0-12.0	60	2.43	0.50	0.55
				A-1-a	G	30.0	18.0	698.9	135	2,018	4,050				100	57	44	31,800	2.00		716.9-698.9	12.0-30.0	60	4.03	0.50	0.55
B-029-0-08	728.9	7.6	8.0	A-4a	C	1.6	1.6	727.3	135	108	216	3,875	6.2	0.52						B-029-0-08	728.9-727.3	0.0-1.6	24	2.00	0.40	0.45
				A-1-a	G	7.6	6.0	721.3	135	621	1,026				100	95	46	31,800	4.93		727.3-721.3	1.6-7.6	60	3.06	0.50	0.55
				A-2-4	G	11.6	4.0	717.3	130	1,161	1,546				51	45	43	16,218	1.79		721.3-717.3	7.6-11.6	60	2.07	0.50	0.55
				A-4a	C	28.6	17.0	700.3	135	1,914	3,841	6,250	9.0	0.45							717.3-700.3	11.6-28.6	56	2.81	0.40	0.45
				A-4a	G	33.6	5.0	695.3	135	2,712	4,516				50	37	42	22,783	1.24		700.3-695.3	28.6-33.6	60	3.35	0.50	0.55
				A-1-b	G	43.6	10.0	685.3	135	3,257	5,866				100	70	44	31,800	1.44		695.3-685.3	33.6-43.6	60	4.67	0.50	0.55
				A-3a	G	60.4	16.8	668.5	130	4,187	8,050				90	58	44	14,824	0.71		685.3-668.5	43.6-60.4	60	2.97	0.50	0.55
				A-6a	C	70.6	10.2	658.3	135	5,126	9,427	8,000	9.0	0.45							668.5-658.3	60.4-70.6	72	3.60	0.40	0.45
				A-1-b	G	86.6	16.0	642.3	135	6,077	11,587				86	47	43	27,348	0.83		658.3-642.3	70.6-86.6	60	5.02	0.50	0.55
				A-3	G	100.1	13.5	628.8	135	7,147	13,410				81	41	42	13,916	0.47		642.3-628.8	86.6-100.1	60	3.32	0.50	0.55
B-030-0-08	730.3	9.6	8.0	A-6b	C	1.6	1.6	728.7	130	104	208	4,500	6.2	0.49						B-030-0-08	730.3-728.7	0.0-1.6	28	2.19	0.40	0.45
				A-a	C	4.6	3.0	725.7	130	403	598	5,250	6.7	0.45							728.7-725.7	1.6-4.6	35	2.37	0.40	0.45
				A-1-a	G	9.1	4.5	721.2	135	902	1,205				96	101	43	30,528	3.28		725.7-721.2	4.6-9.1	60	2.95	0.50	0.55
				A-1-a	G	17.1	8.0	713.2	135	1,527	2,285				49	47	42	15,582	1.41		721.2-713.2	9.1-17.1	58	2.15	0.50	0.55
				A-3a	G	55.6	38.5	674.7	140	3,311	7,675				99	75	40	15,697	0.81		713.2-674.7	17.1-55.6	60	2.69	0.50	0.55
				A-3	G	60.6	5.0	669.7	140	4,999	8,375				73	47	38	13,075	0.54		674.7-669.7	55.6-60.6	60	2.71	0.50	0.55
				A-1-b	G	97.1	36.5	633.2	140	6,609	13,486				109	61	42	34,662	0.90		669.7-633.2	60.6-97.1	60	5.96	0.50	0.55

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

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]	Nominal Unit Tip Resistance, q_p [10,11] (ksf)	Nominal Unit Side Resistance, q_s [12,13] (ksf)	φ_{qp} [14]	φ_{qs} [15]
B-011-0-59 and B-029-0-08	728.9	7.6	8.0	A-4a	C	7.0	7.0	721.9	135	473	945	8,000	7.1	0.45						56	3.60	0.40	0.45
				A-1-a	G	12.0	5.0	716.9	135	1,164	1,620				67	41	42	21,306	2.08	60	2.42	0.50	0.55
				A-1-a	G	30.0	18.0	698.9	135	1,999	4,050				100	57	44	31,800	2.02	60	4.02	0.50	0.55
				A-4a	G	33.6	3.6	695.3	135	2,783	4,536				50	37	42	22,783	1.22	60	3.38	0.50	0.55
				A-1-b	G	43.6	10.0	685.3	135	3,277	5,886				100	70	44	31,800	1.43	60	4.68	0.50	0.55
				A-3a	G	60.4	16.8	668.5	130	4,207	8,070				90	58	44	14,824	0.71	60	2.97	0.50	0.55
				A-6a	C	70.6	10.2	658.3	135	5,146	9,447	8,000	9.0	0.45						72	3.60	0.40	0.45
				A-1-b	G	86.6	16.0	642.3	135	6,097	11,607				86	47	43	27,348	0.83	60	5.03	0.50	0.55
				A-3	G	100.1	13.5	628.8	135	7,167	13,430				81	41	42	13,916	0.46	60	3.32	0.50	0.55

1. C = cohesive soil stratum; G = granular soil stratum
2. S_u = average shear strength over stratum thickness (cohesive soil layers)
3. $N_C = 6[1+0.2(Z/D)] \leq 9$; 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. $(N_1)_{60} = C_nN_{60}$, 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. $\varphi'_f = 27.5+9.2\log[(N_1)_{60}]$; Ref. Section 10.8.3.5.2b, AASHTO LRFD BDS (granular soil layers)
8. $\sigma_p' = n(N_{60})^m(P_a)$, where n = 0.15 and m = 1.0 for A-1-a/1-b and A-2-4/2-6, n = 0.47 and m = 0.6 for A-3/3a, n = 0.47 and m = 0.8 for A-4a/4b soils, and P_a = 2.12 ksf = 2,120 psf; Ref. Section 10.8.3.5.2b, AASHTO LRFD BDS (granular soil layers)
9. $\beta = \tan\varphi'_f(1-\sin\varphi'_f)(\sigma_p'/\sigma_v')$^$(\sin\varphi'_f)$, where σ_v' = vetical effective stress at midpoint of soil layer; Ref. Section 10.8.3.5.2b, AASHTO LRFD BDS (granular soil layers)
10. $q_p = N_CS_u \leq 80.0$ 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

Shaft Length (ft)	Shaft Tip Elevation (ft msl)	Nominal Tip Resistance, ηR_p (kips)	Nominal Side Resistance, ηR_s (kips)	Total Nominal Resistance, ηR_n (kips)	Factored Tip Resistance, $\varphi_{qp}R_p$ (kips)	Factored Side Resistance, $\varphi_{qs}R_s$ (kips)	Total Factored Resistance, R_R (kips)
57.6	671.3			2,714			1,357
		2,714			1,357		

Group Efficiency Factor, η = 0.9

W-13-045 - FRA-70-14.05C Project 4H - FRA-70-1405C Forward Abutment

Tangent Shaft Alternative - Block Failure Mode

Calculated By: BRT Date: 6/8/2020

Checked By: JPS Date: 6/8/2020

Borings B-011-0-59 and B-029-0-09

D =	8.0	ft	Diameter of individual drilled shafts
B' =	6.3	ft	Equivalent footing width based on overall end bearing area of drilled shafts
L =	216.0	ft	27 drilled shafts @ 8.0 ft diameter each along west, roadway and east caps
c =	8,000	psf	
γ =	135	pcf	
D_f =	57.6	ft	
φ =	0	deg	
D_w =	7.6	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma} = 45.24 \text{ ksf}$$

$N_{cm} = N_c s_c i_c$ = 5.17 $\quad N_{qm} = N_q s_q d_q i_q$ = 1.00 $\quad N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 0.00

N_c =	5.14	s_c =	1+(6.3 ft/216 ft)(1/5.14) =	1.006	i_c =	1.000	d_q =	1+2tan(0°)[1-sin(0°)]²tan⁻¹(57.6 ft/6.3 ft) =	1.000
N_q =	1.00	s_q =	1+(6.3 ft/216 ft)tan(0°) =	1.000	i_q =	1.000	C_{wq} =	7.6 ft < 57.6 ft =	0.500
N_γ =	0.00	s_γ =	1-0.4(6.3 ft/216 ft) =	0.988	i_γ =	1.000	$C_{w\gamma}$ =	7.6 ft < 1.5(6.3 ft) + 57.6 ft =	0.500

$q_R = q_n \cdot \phi_b$ = 20.36 ksf

φ_b = 0.45

$\boldsymbol{R_R = q_R \cdot A_p}$ = 1,023 kips

APPENDIX IX

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-001-0-59	764.0 to 756.0	A-4a	C	3	18	18	120 psf	120 psf	Su = 2,250 psf	750 pci	0.0060	-
	756.0 to 752.5	A-2-4	G	4	34	40	130 psf	130 psf	φ = 39°	250 pci	-	-
	752.5 to 750.0	A-4a	C	3	18	18	120 psf	120 psf	Su = 2,250 psf	750 pci	0.0060	-
	750.0 to 745.0	A-1-a	G	4	41	41	130 psf	130 psf	φ = 42°	355 pci	-	-
	745.0 to 740.0	A-1-a	G	4	100	90	135 psf	135 psf	φ = 43°	395 pci	-	-
	740.0 to 738.0	A-1-a	G	4	16	14	125 psf	125 psf	φ = 37°	190 pci	-	-
	738.0 to 732.5	A-4a	C	3	28	28	125 psf	125 psf	Su = 3,500 psf	1,165 pci	0.0048	-
	732.5 to 727.5	A-6a	C	3	36	36	125 psf	125 psf	Su = 4,500 psf	1,500 pci	0.0045	-
	727.5 to 725.0	A-4a	C	3	100	100	130 psf	130 psf	Su = 8,000 psf	2,665 pci	0.0033	-
	725.0 to 721.0	A-1-a	G	4	61	42	135 psf	135 psf	φ = 42°	355 pci	-	-
	721.0 to 711.0	A-4b	G	4	90	58	135 psf	72.6 psf	φ = 38°	125 pci	-	-
	711.0 to 708.0	A-1-b	G	4	92	57	135 psf	72.6 psf	φ = 42°	195 pci	-	-
B-007-0-59	767.1 to 759.1	A-4a	C	3	17	17	120 psf	120 psf	Su = 2,125 psf	710 pci	0.0062	-
	759.1 to 749.1	A-1-a	G	4	23	25	125 psf	125 psf	φ = 39°	250 pci	-	-
	749.1 to 734.1	A-1-b	G	4	56	47	135 psf	135 psf	φ = 41°	315 pci	-	-
	734.1 to 727.6	A-4a	C	3	70	70	130 psf	130 psf	Su = 8,000 psf	2,665 pci	0.0033	-
	727.6 to 720.1	A-1-a	G	4	100	66	135 psf	135 psf	φ = 43°	395 pci	-	-
	720.1 to 715.1	A-4b	G	4	79	49	135 psf	72.6 psf	φ = 37°	110 pci	-	-
	715.1 to 709.1	A-4a	C	2	92	92	130 psf	67.6 psf	Su = 8,000 psf	2,665 pci	0.0033	-
	709.1 to 704.1	A-1-b	G	4	65	38	135 psf	72.6 psf	φ = 40°	155 pci	-	-
	704.1 to 701.6	A-1-a	G	4	100	57	135 psf	72.6 psf	φ = 43°	215 pci	-	-
B-011-0-59	769.9 to 758.4	A-2-4	G	4	16	22	125 psf	125 psf	φ = 37°	190 pci	-	-
	758.4 to 753.4	A-4a	C	3	17	17	120 psf	120 psf	Su = 2,125 psf	710 pci	0.0062	-
	753.4 to 738.4	A-1-b	G	4	67	58	135 psf	135 psf	φ = 42°	355 pci	-	-
	738.4 to 721.9	A-4a	C	3	94	94	135 psf	135 psf	Su = 8,000 psf	2,665 pci	0.0033	-
	721.9 to 716.9	A-1-a	G	4	67	41	135 psf	72.6 psf	φ = 42°	195 pci	-	-
	716.9 to 698.9	A-1-a	G	4	100	57	135 psf	72.6 psf	φ = 43°	215 pci	-	-
B-029-0-08	742.3 to 736.3	A-1-b	G	4	28	43	130 psf	130 psf	φ = 41°	315 pci	-	-
	736.3 to 727.3	A-4a	C	3	31	31	125 psf	125 psf	Su = 3,875 psf	1,290 pci	0.0047	-
	727.3 to 721.3	A-1-a	G	4	100	95	135 psf	135 psf	φ = 43°	395 pci	-	-
	721.3 to 717.3	A-2-4	G	4	51	45	135 psf	72.6 psf	φ = 40°	155 pci	-	-
	717.3 to 700.3	A-4a	C	2	50	50	130 psf	67.6 psf	Su = 6,250 psf	2,085 pci	0.0039	-
	700.3 to 695.3	A-4a	G	4	50	37	135 psf	72.6 psf	φ = 36°	95 pci	-	-
	695.3 to 685.3	A-1-b	G	4	100	70	135 psf	72.6 psf	φ = 42°	195 pci	-	-
	685.3 to 668.5	A-3a	G	4	90	58	135 psf	72.6 psf	φ = 40°	155 pci	-	-
	668.5 to 658.3	A-6a	C	2	100	100	130 psf	67.6 psf	Su = 8,000 psf	2,665 pci	0.0033	-
	658.3 to 642.3	A-1-b	G	4	86	47	135 psf	72.6 psf	φ = 41°	175 pci	-	-
	642.3 to 628.8	A-3	G	4	81	41	135 psf	72.6 psf	φ = 38°	125 pci	-	-
	628.8 to 625.8	Shale	R	9	-	-	150 psf	87.6 psf	Qu = 200 psi	0.0005	20,000 psi	0
	625.8 to 621.7	Shale	R	9	-	-	150 psf	87.6 psf	Qu = 360 psi	0.0005	32,000 psi	0
	621.7 to 605.8	Limestone	R	9	-	-	165 psf	102.6 psf	Qu = 10,000 psi	0.00005	1,000,000 psi	84
B-030-0-08	736.7 to 731.7	A-1-b	G	4	9	15	120	120	φ = 36°	160 pci	-	-
	731.7 to 728.7	A-6b	C	3	36	36	130	130	Su = 4,500 psf	1,500 pci	0.0045	-
	728.7 to 725.7	A-4a	C	3	42	42	130	130	Su = 5,250 psf	1,750 pci	0.0043	-
	725.7 to 721.2	A-1-a	G	4	96	101	140	140	φ = 43°	395 pci	-	-
	721.2 to 713.2	A-1-a	G	4	49	47	135	73	φ = 42°	195 pci	-	-
	713.2 to 674.7	A-3a	G	4	99	75	140	78	φ = 40°	155 pci	-	-
	674.7 to 669.7	A-3	G	4	73	47	140	78	φ = 38°	125 pci	-	-
	669.7 to 633.2	A-1-b	G	4	109	61	140	78	φ = 42°	195 pci	-	-
	633.2 to 625.7	Shale	R	9	-	-	150	88	Qu = 750 psi	0.00025	68,000 psi	38
B-030-1-15	748.9 to 745.4	A-4a	C	3	29	29	125 psf	125 psf	Su = 3,625 psf	1,210 pci	0.0048	-
	745.4 to 738.4	A-2-4	G	4	45	57	130 psf	130 psf	φ = 41°	315 pci	-	-
	738.4 to 730.9	A-6a	C	3	50	50	130 psf	130 psf	Su = 6,250 psf	2,085 pci	0.0039	-
	730.9 to 729.2	A-6a	C	3	60	60	130 psf	130 psf	Su = 7,500 psf	2,500 pci	0.0035	-
	729.2 to 720.4	A-1-b	G	4	106	90	135 psf	135 psf	φ = 42°	355 pci	-	-
	720.4 to 711.9	A-1-b	G	4	75	58	135 psf	72.6 psf	φ = 42°	195 pci	-	-
	711.9 to 706.9	A-4b	G	4	89	65	135 psf	72.6 psf	φ = 38°	125 pci	-	-
	706.9 to 696.9	A-4a	C	2	50	50	130 psf	67.6 psf	Su = 6,250 psf	2,085 pci	0.0039	-
	696.9 to 689.5	A-1-b	G	4	33	22	130 psf	67.6 psf	φ = 38°	125 pci	-	-