Resource International, Inc.

FRA-70-14.05 PROJECT 4B
FRA-33-1747
S. THIRD STREET (US-33) OVER I-70/71
PID NO. 96053
FRANKLIN COUNTY, OHIO

STRUCTURE FOUNDATION EXPLORATION REPORT

Prepared For:
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Rii Project No. W-15-126

July 2022

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July 8, 2022

Mr. Christopher W. Luzier, P.E.
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1801 Watermark Drive, Suite 210
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Re: Structure Foundation Exploration Report
FRA-70-14.05 Project 4B
FRA-33-1747 – S. Third Street (US-33) over I-70/71
PID No. 96053
Rii Project No. W-15-126

Mr. Luzier:

Resource International, Inc. (Rii) is pleased to submit this structure foundation exploration report for the above referenced project. Engineering logs have been prepared and are attached to this report along with the results of laboratory testing. This report includes recommendations for the design and construction of the proposed FRA-33-1747 bridge structure carrying S. Third Street (US-33) over I-70/71 as part of the FRA-70-14.05 Project 4B 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

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EXECUTIVE SUMMARY

Resource International, Inc. (Rii) has completed a structure foundation exploration for the design and construction of the proposed FRA-33-1747 bridge structure carrying S. Third Street (US-33) and flanking cap structures over I-70/71. The existing structure is a two-span bridge with a total length of approximately 122 feet. It is understood that the existing structure consists of a reinforced concrete deck on continuous steel beams, and will be removed and replaced with a two-span continuous composite steel plate girder structure with reinforced concrete deck and concrete substructures.

Shallow Foundation Recommendations

It is understood that shallow spread foundations will be utilized at the rear abutment and pier substructure units. The bearing soils are anticipated to consist of dense to very dense gravel, gravel and sand and gravel with sand and silt (ODOT A-1-a, A-1-b, A-2-4) overlying hard sandy silt, silt and silt and clay (ODOT A-4a, A-4b, A-6a). Shallow spread foundations bearing on these competent natural soils may be proportioned for a nominal bearing resistance as presented in Table 5 for the rear abutment and Table 6 for the pier substructure in Section 5.1 of the full report.

Based on the service limit bearing pressures provided, total settlements of 1.53, 0.92 and 1.49 inches are anticipated for the west cap, bridge and east cap at the rear abutment, respectively, and 1.24, 0.96 and 1.59 inches are anticipated for the west cap, bridge and east cap at the pier, respectively. Differential settlement along the and pier substructures is anticipated to be less than 1/200. However, differential settlement between the bridge and the adjacent east and west caps may be less than 1/100 at the joints. Additionally, the maximum factored bearing pressure of 11.71 ksf at the rear abutment and 9.60 ksf at the pier substructure units will not exceed the factored bearing resistance at the strength limit of 16.76 and 20.88 ksf, respectively

For concrete footing that rest on cohesionless soil, a coefficient “f” of 0.78 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.

Drilled Shaft Recommendations

It is understood that a tangent drilled shaft foundation is 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 7 in Section 5.2 of the full report. Based on the subsurface conditions encountered, the embedded sections of the shafts will bear in dense to very dense gravel, gravel and sand, gravel with sand and silt and coarse and fine sand (ODOT A-1-a, A-1-b, A-2-4, A-3a). Given that the drilled shafts will be constructed tangent to each other, group efficiency of the foundation for axial resistance will also need to be considered, as outlined in Section 5.2.1 of the full report. Additionally, lateral design of the drilled shaft elements will likely control the required embedment depth. Therefore, lateral analysis of the shafts should be performed to determine the required embedment depth and cross section of the shafts as outlined in Section 5.2.2 of the full report.

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-70-14.05 Project 4B 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-70-14.05 Project 4B phase will consist of all work associated with the construction of the I-70/I-71 corridor from just east of S. High Street to just west of Grant Avenue, as well as a minimal amount of work Fulton Street and at the intersections of S. Third Street and S. Fourth Street with Livingston Avenue. This project includes the replacement of the FRA-33-1747 (S. Third Street) and FRA-23-1075 (S. Fourth Street) bridge structures over I-70/71, as well as the construction of three (3) new retaining walls along the north side and two (2) new retaining walls along the south side of I-70/71 to accommodate the new configuration.

This report is a presentation of the structure foundation exploration performed for the design and construction of the proposed FRA-33-1747 bridge structure carrying S. Third Street (US-33) 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 two-span bridge with a total length of approximately 122 feet. It is understood that the existing structure consists of a reinforced concrete deck on continuous steel beams, and will be removed and replaced with a two-span continuous composite steel plate girder structure with reinforced concrete deck and concrete substructures.

2.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1 Site Geology

Several episodes of ice advanced throughout Ohio during the Pleistocene Epoch. Both the Illinoian and Wisconsinan glaciers advanced over two-thirds of the state, leaving behind glacial features such as moraines, kame deposits, lacustrine deposits and outwash terraces. The glacial and non-glacial regions comprise five physiographic sections grouped by age, depositional process and geomorphic occurrence (physical features or landforms). The project area lies within the Columbus Lowland District of the Till Plains Section. The project area is characterized by flat to gently rolling ground moraine deposits of the Late Wisconsinan age with large alluvium and outwash deposits bordering the Scioto River, its tributaries and floodplain areas. Ground moraines are deposited during the retreat of a glacier, which results in an undifferentiated mixture of clay, silt, sand and gravel. Alluvium and alluvial terrace deposits range from silty clay to cobble sized deposits, usually deposited in present and former floodplain areas. Outwash deposits consist of undifferentiated sand and gravel deposited by meltwater in front of glacial ice.

Based on bedrock geology and topography maps obtained from Ohio Department of Natural Resources (ODNR), the bedrock beneath the project site consists of three formations. The project alignment extends east from the top of the eastern slope of a bedrock valley that generally follows the Scioto River valley, with the youngest formation at the top of the slope and the oldest formation within the bedrock valley. The youngest formation consists of the Upper Devonian-aged Ohio Shale Formation, which consists of three members, from youngest to oldest: the Cleveland, Chagrin, and Huron Members. These members consist of primarily shale with siltstone and very fine-grained sandstone, varying in color from brownish black to greenish gray. The bedding ranges from laminated to thinly bedded and the overall formation ranges between 250 to over 500 feet thick. The Middle Devonian-aged Delaware Limestone formation, which can be present along the slopes of the bedrock valley, consists of bluish-gray, dolomitic limestone, with thin to medium bedding, and contains nodules and layers of chert. The formation ranges between 0 to 45 feet thick and is not present south of Franklin County. The oldest unit, which present within the bedrock valley, is the Middle to Lower Devonian-aged Columbus Limestone Formation, which is further subdivided into four members, two of which are predominant 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, and the entire formation ranges between 0 to 105 feet thick.

The bedrock surface in the vicinity of the site forms a broad valley which roughly follows the present day Scioto River valley. The site lies on a slight plateaued area and slope along the east side of the valley where the underlying bedrock surface lies at an approximate elevation of 625 to 630 feet mean sea level and slopes down toward the west to an approximate elevation of 600 feet msl in the bedrock valley. According to bedrock topography mapping, the depth to the bedrock surface below the site ranges between approximately 105 to 135 feet below existing grade. Shale bedrock was encountered in several of the borings performed along the corridor at elevations ranging from 630 to 650 feet msl, increasing in elevation from west to east across the project alignment.

2.2 Existing Conditions

The proposed FRA-33-1747 structure is located at the existing S. Third Street (US-33) over I-70/71 overpass, approximately 0.87 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. There is an existing exit ramp from I-70 eastbound that connects with Livingston Avenue just west of the intersection with S. Third Street, and there are entrance ramps to both I-70 eastbound and westbound from S. Third Street. The existing S. Third Street crossing is a four-lane, one-way, asphalt paved roadway with two southbound exit lanes to I-70 eastbound, and a left turn lane and through lane at the intersection with Livingston Avenue. The existing I-70 profile is lowered from the surrounding terrain, as the existing corridor was cut approximately 25 feet below the existing grade on S. Third Street and the surrounding downtown area. Existing cast-in-place concrete wall type abutments and adjacent wingwalls are present at both rear and forward abutments. Graded slopes extend to the east and west of the abutments, which are grass covered along the north side of I-70/71 and covered with brush and other vegetation along the south side of I-70/71. The existing structure appears to be in poor condition, with concrete spalling and delamination evident on the interior 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

Between October 5, 2015, and January 7, 2016, four (4) structure borings, designated as B-032-3-15 through B-032-6-15, were drilled along the proposed bridge alignment at the locations shown on the boring plan provided in Appendix I of this report and summarized in Table 1. The borings were advanced to completion depths ranging from 75.0 to 80.0 feet below the existing ground surface at the respective boring locations.

Table 1. Test Boring Summary

Boring Number	Reference Alignment	Station	Offset	Latitude	Longitude	Ground Elevation (feet msl)	Boring Depth (feet)
B-032-3-15	BL I-70 EB	198+77.78	40.8' Rt.	39.953103	-82.996483	732.8	75.0
B-032-4-15	CL S. Third St.	1151+80.18	46.1' Lt.	39.953329	-82.996384	732.5	75.0
B-032-5-15	CL S. Third St.	1151+79.91	46.7' Rt.	39.953371	-82.996058	731.6	75.0
B-032-6-15	CL S. Third St.	1152+85.60	44.9' Rt.	39.953656	-82.996128	753.0	80.0

The boring locations were determined and located in the field by Rii representatives. Rii utilized a handheld GPS unit to obtain geographic latitude and longitude coordinates of the boring locations. Ground surface elevations at the boring locations were interpolated using topographic mapping information provided by GPD GROUP.

The borings were drilled using a truck or all-terrain vehicle (ATV) mounted rotary drilling machine, utilizing a 3.25-inch inside diameter hollow-stem auger to advance the holes. Standard penetration test (SPT) and split spoon sampling were performed in borings B-032-3-15 through B-032-5-15 at 2.5-foot intervals of depth to 30.0 feet and at 5.0-foot intervals thereafter to the boring termination depths. Boring B-032-6-15 was sampled at 5.0-foot intervals to a depth of 20.0 feet, 2.5-foot intervals to a depth of 50.0 feet and at 5.0-foot intervals thereafter to the boring termination depth. The SPT, per the American Society for Testing and Materials (ASTM) designation D1586, is conducted using a

140-pound hammer falling 30.0 inches to drive a 2.0-inch outside diameter split spoon sampler 18.0 inches. Rii utilized a calibrated automatic drop hammer to generate consistent energy transfer to the sampler. Driving resistance is recorded on the boring logs in terms of blows 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 truck and CME 750X ATV-mounted drill rigs used were calibrated on October 20, 2014, and have drill rod energy ratios of 92.0 and 85.7 percent, respectively.

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, Rii personnel prepared field logs 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	89
Plastic and Liquid Limits	AASHTO T89, T90	34
Gradation – Sieve/Hydrometer	AASHTO T88	34

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.

In addition to the borings performed as part of the current exploration, historic borings performed in 1959 by the Ohio 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. Two (2) borings, designated as B-001-3-59 and B-010-3-59, were obtained in the vicinity of the existing bridge abutments. 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 feet each 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 in the report to the current NAD 83 datum.** The historic boring locations are shown on the boring plan provided in Appendix I of this report and the historic boring logs are provided in Appendix IV.

4.0 FINDINGS

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

4.1 Surface Materials

Boring B-032-3-15 was performed at the toe of the slope in the grass berm adjacent to I-70 eastbound and encountered 3.0 inches of topsoil at the ground surface. Borings B-032-4-15 and B-034-5-15 were drilled within the existing outside shoulder of I-70 westbound and encountered 9.0 inches of asphalt overlying 9.0 and 21.0 inches of aggregate base at the ground surface, respectively. Boring B-032-6-15 was performed in the existing pavement of S. Third Street and encountered 4.0 inches of asphalt overlying 6.0 inches of concrete followed by 26.0 inches of aggregate base at the ground surface.

4.2 Subsurface Soils

Underlying the surficial materials, natural granular soils were encountered with intermittent seams and layers of cohesive material. The granular soils were described as medium dense to very dense, gray, brown, brownish gray dark gray and black gravel, gravel and sand, gravel with sand and silt, coarse and fine sand, sandy silt and silt (ODOT A-1-a, A-1-b, A-2-4, A-3a, A-4a, A-4b). The seams and layers of cohesive material were described as stiff to hard, gray, brown and brownish gray sandy silt and silt and clay (ODOT A-4a, A-6a). Cobbles were generally encountered within the very dense gravel and sand and gravel layers below approximate elevation 695 to 714 feet msl.

The relative density of granular soils is primarily derived from SPT blow counts (N_{60}). Based on the SPT blow counts obtained, the granular soil encountered ranged from medium dense ($11 \leq N_{60} < 30$ blows per foot [bpf]) to very dense ($N_{60} > 50$ bpf). Blow counts recorded from the SPT sampling within the granular soils ranged from 21 bpf to 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. 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 2.0 to over 4.5 tsf (limit of instrument).

Natural moisture contents of the cohesive soil samples ranged from 9 to 12 percent. The natural moisture content of the cohesive soil samples tested for plasticity index were 4 percent below their corresponding plastic limits. The seams of cohesive soil exhibited natural moisture contents considered to be moderately below optimum moisture levels. Natural moisture contents of the granular soil samples ranged from 4 to 23 percent, which were described as moist to wet.

4.3 Bedrock

Bedrock was not encountered in any of the borings performed for this exploration.

4.4 Groundwater

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

Table 3. Groundwater Levels

Boring Number	Ground Elevation (feet msl)	Initial Groundwater		Upon Completion	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-032-3-15	732.8	11.5	721.3	N/A [1]	-
B-032-4-15	732.5	11.5	721.0	N/A [1]	-
B-032-5-15	731.6	10.5	721.1	N/A [1]	-
B-032-6-15	753.0	31.0	722.0	N/A [1]	-

1. *The groundwater level at completion could not be obtained due to the addition of water or mud as a drilling fluid.*

Groundwater was encountered initially during the drilling process in borings B-032-3-15 through B-032-6-15 at a depths ranging from 10.5 to 31.0 feet below the existing ground surface, which corresponds to elevations ranging from 721.0 to 722.0 feet msl. The groundwater levels at the completion of drilling could not be measured due to the addition of mud or water to counteract heaving sands. 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-001-3-59 and B-010-3-59, encountered medium dense to very dense granular soils with a layer of very stiff cohesive soil within the top 7.5 feet of the soil profile in boring B-010-3-59. The granular soils were generally described as brown and gray gravel, gravel and sand, gravel with sand and silt, gravel with sand, silt and clay, fine sand and coarse and fine sand (ODOT A-1-a, A-1-b, A-2-4, A-2-6, A-3, A-3a), and the cohesive soil encountered in boring B-010-3-59 was described as brown sandy silt (ODOT A-4a). A bouldery zone was encountered in boring B-010-3-59 between elevations 726 and 741 feet msl. Bedrock was not encountered in the historic borings prior to the termination depths. Groundwater levels were not noted in the borings performed during the 1959 exploration. In general, the subsurface conditions encountered in the historic borings matched relatively closely with the subsurface conditions encountered in the current exploration borings.

5.0 ANALYSES AND RECOMMENDATIONS

Data obtained from the current and historic 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 the foundation systems for the subject bridge, as well as the construction specifications related to the placement of foundation systems and general earthwork recommendations, which are discussed in the following paragraphs.

Design details of the structure proposed were provided by GPD GROUP. Based on the information provided, it is understood that the existing structure will be removed and replaced with three parallel, two-span continuous composite steel plate girder structures with reinforced concrete 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. The proposed abutment structures will be shifted to approximately 10 feet and 50 feet outside of the existing rear and forward abutments, respectively. The proposed abutment structures will have an approximate length of 190 feet and widths of 60 feet, 64 feet and 60 feet, representing the west cap, S. High Street bridge, and east cap structures, respectively. The three parallel structures will each react independently, with only a longitudinal expansion joint connecting them.

Proposed structural data was obtained bridge cap structure from design details provided by GPD GROUP and are included in Table 4. At the time of this report, structure details and loading for the adjacent cap structures were not available.

Table 4. FRA-33-1747 Structure and Bridge Design Elevations

Substructure Unit (Borings)	Structure Component [1]	Elevation [1] (feet msl)	Design Maximum Factored Load	
			Service	Strength
Rear Abutment (B-001-3-59 / B-032-3-15)	Bottom of Footing	West Cap: 722.5 Bridge: 721.5 East Cap: 721.0	West Cap: 8.08 ksf Bridge: 6.00 ksf East Cap: 8.35 ksf	West Cap: 11.31 ksf Bridge: 8.67 ksf East Cap: 11.71 ksf
Pier (B-032-4-15 / B-032-5-15)	Bottom of Footing	West Cap: 722.5 Bridge: 722.0 East Cap: 721.5	West Cap: 5.99 ksf Bridge: 6.10 ksf East Cap: 5.99 ksf	West Cap: 8.29 ksf Bridge: 9.60 ksf East Cap: 8.29 ksf
Forward Abutment (B-010-3-59 / B-032-6-15)	Top of Embedded Shaft (Bottom of Wall)	725.0 Lt. 723.0 Rt.	West Cap: 309 kips/shaft Bridge: 172 kips/shaft East Cap: 309 kips/shaft	West Cap: 410 kips/shaft Bridge: 226 kips/shaft East Cap: 410 kips/shaft

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

The roadway profile grade along the proposed I-70 eastbound and a portion of I-70 westbound beneath the structure will be cut approximately 1.5 to 2.5 feet below the existing roadway profile grade, and there will be no change in the profile grade of S. Third Street. Where the proposed I-70 eastbound and westbound extends outside the limits of the existing I-70 roadway, the proposed profile grade will be cut up to 24.0 feet below the existing grade of S. Third Street and adjacent slopes.

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 4. At these elevations, the bearing soils are anticipated to consist of dense to very dense gravel, gravel and sand and gravel with sand and silt (ODOT A-1-a, A-1-b, A-2-4) overlying hard sandy silt, silt and silt and clay (ODOT A-4a, A-4b, A-6a). It should be noted that boring B-001-3-59 has a bottom of boring elevation of 694.6 feet msl, which is approximately 28.5 feet below the proposed bottom of footing elevation at the rear abutment. Shallow spread foundations bearing on these competent natural soils may be proportioned for a nominal bearing resistance as presented in Table 5 for the rear abutment and Table 6 for the pier substructure.

Table 5. FRA-33-1747 Rear Abutment 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 Bridge Cap (B-001-3-59)	5.0	4.88	8.28	15.53	24.31	13.37
	7.0	4.63	7.26	12.81	26.95	14.82
	9.0	4.49	6.70	11.32	29.66	16.31
	11.0	4.41	6.35	10.40	32.39	17.81
	13.0	4.35	6.12	9.78	35.11	19.31
	15.0	4.31	5.95	9.34	37.80	20.79
	17.0	4.27	5.83	9.01	40.45	22.25
	19.0	4.25	5.73	8.76	43.05	23.67
	21.0	4.23	5.66	8.57	45.58	25.07
	23.0	4.22	5.60	8.41	48.05	26.43
	25.0	4.21	5.55	8.29	50.46	27.75

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 East Cap (B-001-3-59)	5.0	2.57	5.98	13.22	24.31	13.37
	7.0	2.32	4.95	10.50	26.95	14.82
	9.0	2.19	4.39	9.02	29.66	16.31
	11.0	2.10	4.05	8.10	32.39	17.81
	13.0	2.04	3.81	7.48	35.11	19.31
	15.0	2.00	3.65	7.04	37.80	20.79
	17.0	1.97	3.52	6.71	40.45	22.25
	19.0	1.95	3.43	6.46	43.05	23.67
	21.0	1.93	3.35	6.26	45.58	25.07
	23.0	1.91	3.29	6.11	48.05	26.43
	25.0	1.90	3.25	5.98	50.46	27.75
Rear Abutment West Cap (B-032-3-15)	5.0	3.11	7.11	13.69	40.19	22.10
	7.0	2.71	5.70	10.56	40.43	22.24
	9.0	2.49	4.92	8.83	40.67	22.37
	11.0	2.35	4.42	7.73	40.92	22.50
	13.0	2.25	4.08	6.98	41.16	22.64
	15.0	2.18	3.83	6.43	41.40	22.77
	17.0	2.13	3.64	6.01	41.64	22.90
	19.0	2.08	3.50	5.68	41.89	23.04
	21.0	2.05	3.38	5.42	42.13	23.17
	23.0	2.02	3.28	5.21	42.37	23.30
	25.0	2.00	3.20	5.02	42.61	23.44

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 was utilized in calculating the factored bearing resistance at the strength limit state.*

Table 6. FRA-33-1747 Pier 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		
Pier Bridge and West Cap (B-032-4-15)	5.0	3.60	9.45	19.47	42.52	19.13
	7.0	3.09	7.45	14.84	48.90	22.01
	9.0	2.80	6.34	12.27	55.46	24.96
	11.0	2.62	5.64	10.64	62.09	27.94
	13.0	2.49	5.15	9.51	68.76	30.94
	15.0	2.40	4.79	8.68	75.43	33.94
	17.0	2.33	4.51	8.05	82.07	36.93
	19.0	2.27	4.30	7.55	88.69	39.91
	21.0	2.22	4.12	7.15	95.27	42.87
	23.0	2.18	3.98	6.82	101.80	45.81
	25.0	2.15	3.86	6.55	108.29	48.73
Pier Bridge and East Cap (B-032-5-15)	5.0	2.71	6.15	12.05	41.66	20.83
	7.0	2.39	4.98	9.36	41.75	20.87
	9.0	2.21	4.33	7.88	41.83	20.92
	11.0	2.10	3.92	6.95	41.92	20.96
	13.0	2.02	3.64	6.31	42.00	21.00
	15.0	1.96	3.43	5.84	42.09	21.05
	17.0	1.92	3.28	5.49	42.18	21.09
	19.0	1.88	3.16	5.21	42.26	21.13
	21.0	1.86	3.06	4.99	42.35	21.17
	23.0	1.83	2.98	4.81	42.43	21.22
	25.0	1.82	2.92	4.66	42.52	21.26

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 = 0.45 and 0.50 was utilized in calculating the factored bearing resistance at the strength limit state for B-032-4-15 and B-032-5-15, 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 5. A geotechnical resistance factor of φ_b = 0.55 for the rear abutment and φ_b = 0.45 to 0.50 at the pier substructure has been considered in calculating the factored bearing resistance at the strength limit state. Based on the bearing pressures provided in Table 5 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 all settlement values 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 4, total settlements of 1.53, 0.92 and 1.49 inches are anticipated for the west cap, bridge and east cap at the rear abutment, respectively, and 1.24, 0.96 and 1.59 inches are anticipated for the west cap, bridge and east cap at the pier, respectively. Differential settlement along the and pier substructures is anticipated to be less than 1/200. However, differential settlement between the bridge and the adjacent east and west caps is less than 1/100 at the joints. Additionally, the maximum factored bearing pressure of 11.71 ksf at the rear abutment and 9.60 ksf at the pier substructure units will not exceed the factored bearing resistance at the strength limit of 16.76 and 20.88 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 0.78 times the total vertical force on the base should be taken as the sliding resistance. A geotechnical resistance factor of φ_τ = 1.0 should be considered when calculating the factored shear resistance between the soil and foundation for sliding.

5.2 Drilled Shaft Recommendations

It is understood that a tangent drilled shaft foundation is 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 7. In the analysis, the top of shaft elevations for the embedded sections of the shafts were considered at the bottom of wall elevation. Based on the subsurface conditions encountered, the embedded sections of the shafts will bear in dense to very dense gravel, gravel and sand, gravel with sand and silt and coarse and fine sand (ODOT A-1-a, A-1-b, A-2-4 and A-3a). The drilled shafts should be proportioned for a nominal bearing resistance as presented in Table 7.

Table 7. FRA-33-1747 Drilled Shaft Axial Design Parameters

Boring	Elevation [1] (feet msl)	Shaft Length (feet)	Soil Type	Nominal Unit Resistance (ksf)		Resistance Factor	
				End	Side	End	Side
B-010-3-59	725.0-723.4	0-1.6	A-3	50	0.38	0.50	0.55
	723.4-714.9	1.6-10.1	A-3a	55	0.90	0.50	0.55
	714.9-709.9	10.1-15.1	A-3	60	1.47	0.50	0.55
	709.9-696.9	15.1-28.1	A-3a	60	1.84	0.50	0.55
B-032-6-15	723.0-712.5	0.0-10.5	A-1-a	60	1.55	0.50	0.55
	712.5-707.5	10.5-15.5	A-1-b	44	1.33	0.50	0.55
	707.5-701.0	15.5-22.0	A-1-a	46	1.64	0.50	0.55
	701.0-696.0	22.0-27.0	A-1-b	36	1.46	0.50	0.55
	696.0-686.0	27.0-37.0	A-1-a	60	2.71	0.50	0.55
	686.0-673.0	37.0-50.0	A-1-a	60	4.89	0.50	0.55

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

Drilled shaft lengths should measure a minimum of three (3) times the shaft diameter. Per Section 10.8.3.5.3 of the 2017 AASHTO LRFD Bridge Design Specifications (BDS), where drilled shafts are extended to end bear in a strong soil layer overlying a weaker soil layer, the end bearing resistance shall be reduced if the tip elevation is within 1.5 times the diameter of the drilled shaft above the top of the weaker soil layer. A weighted average that varies linearly from the full end bearing resistance in the overlying strong soil layer at a distance of 1.5 times the diameter of the drilled shaft above the top of the weak soil layer to the end bearing resistance of the weak soil layer at the top of the weak soil layer should be used to determine the end bearing resistance utilized in the design. Therefore, the end bearing resistance utilized in the design will need to be adjusted accordingly if the tip elevation of the drilled shafts will be within 1.5 times the diameter of the drilled shaft above the underlying weaker soil layer.

It is anticipated that 100 percent of the side friction resistance will be mobilized at a displacement of 1.0 percent of the diameter of the shaft, which is approximately 0.6 inches for a 5.0-foot diameter shaft. At this displacement, approximately 30 percent of the end bearing resistance will be mobilized. Therefore, the nominal end bearing resistance noted in Table 7 should be reduced to 30 percent of the values provided for the respective tip elevation in the determination of the design shaft resistance. Drilled shaft calculations are provided in Appendix 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 2017 AASHTO LRFD BDS, for soil profiles that consist of primarily granular soils, the individual nominal resistance of each drilled shaft shall be reduced by applying an adjustment factor, η, as defined in Table 10.8.3.6.1-1 of the 2017 AASHTO LRFD BDS. The following criteria are recommended for the group resistance of any shaft groups:

- η = 0.9 for a center-to-center spacing of 2.0 diameters,
- η = 1.0 for a center-to-center spacing of 3.0 diameters or greater,
- For intermediate spacing, the value of η may be determined by liner interpolation.

Please note that the adjustment factor should be applied to the total individual nominal shaft resistance (including both end bearing side resistance along the shaft length).

Given that the drilled shafts at the forward abutment 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 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 factored 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 VIII. 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 8 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 VIII.

Table 8. 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 for both foundation alternatives, which were utilized to determine the shaft tip elevation provided in the design plans.

5.2.3 Drilled Shaft Axial Resistance

The nominal and factored drilled shaft axial resistance has been calculated for the forward abutment, which is summarized in Table 9 below. A tip elevation of 678.3 feet msl was determined from the plan information provided 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. Based on the tip elevation provided, the drilled shafts will end bear within a layer of very dense gravel (ODOT A-1-a), which has a calculated nominal end bearing resistance of 60 ksf.

The bearing resistance for the block failure mode was also checked since the drilled shafts will be constructed tangent to each other. Based on the shaft tip elevation provided, the shafts will be bearing in very dense gravel (ODOT A-1-a). Using the friction angle for the very dense gravel (ODOT A-1-a), the resulting nominal unit bearing resistance is 429.3 ksf and the factored unit bearing resistance is 214.7 ksf, considering a resistance factor of 0.5.

Table 9. FRA-33-1747 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	5.0	723.0	678.3	44.7	5.0	1,060	N/A	1,060	530	N/A	530
Block	5.0	723.0	678.3	44.7	5.0	8,430	N/A	8,430	4,215	N/A	4,215

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 both the traditional drilled shaft analysis methodology and the block failure mode.*

The controlling resistance between the traditional drilled shaft analysis methodology and block failure mode is 530 kips per shaft. The maximum factored load per shaft is 226 and 410 kips/shaft for the bridge and east/west caps, 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.
- Due to the presence of granular soils with relatively high groundwater, it is recommend mud or slurry be utilized in the shaft excavation to counterbalance the hydrostatic head at the bottom of the excavation and minimize the potential for “heave” of the soils up and into the shaft excavation.
- Concrete placed freefall should not be allowed to hit the sidewalls of the excavation or the rebar cage and should not pass through any water.
- Structural stability of the rebar cage should be maintained during the concrete pour to prevent buckling.
- The volume of concrete should be checked to ensure voids did not result during extraction of the casing (if utilized).
- The placement of all concrete for the drilled shafts shall follow the American Concrete Institute’s Design and Construction of Drilled Piers (ACI 336.3R-93).
- If concrete is placed by tremie method, it must be done so with an adequate head to displace water or slurry if groundwater has entered the caisson (all tremie procedures shall follow applicable ACI specifications).
- Pulling casing with insufficient concrete inside should be restricted.
- The bottom of drilled shaft excavation should be clean and free of loose material. Any loose material observed should be removed using a clean-out bucket (muck bucket).

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

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

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

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

It is anticipated that conventional drilled shaft equipment (with a standard soil bit) will be able to penetrate the surficial soils to the required tip elevation. However, as noted in Section 4.2, cobbles were generally encountered within the very dense gravel and sand and gravel layers below approximate elevation 695 to 714 feet msl, and a bouldery zone was encountered in boring B-010-3-59 between elevations 726 and 741 feet msl. Therefore, difficult drilling conditions should be anticipated to be encountered during the excavation 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 10 and Table 11.

Table 10. 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	120	1,500	0°	N/A	N/A	N/A
Very Stiff to Hard Cohesive Soil	130	3,000	0°	N/A	N/A	N/A
Loose Granular Soil	125	0	28°	0.32	0.53	5.07
Medium Dense Granular Soil	130	0	32°	0.27	0.47	6.82
Dense to Very Dense Granular Soil	135	0	36°	0.23	0.41	9.09
Compacted Cohesive Engineered Fill	120	2,000	0°	N/A	N/A	N/A
Compacted Granular Engineered Fill	120	0	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 11. 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	120	0	26°	0.35	0.56	4.53
Very Stiff to Hard Cohesive Soil	130	0	28°	0.32	0.53	5.07
Loose Granular Soil	125	0	28°	0.32	0.53	5.07
Medium Dense Granular Soil	130	0	32°	0.27	0.47	6.82
Dense to Very Dense Granular Soil	135	0	36°	0.23	0.41	9.09
Compacted Cohesive Engineered Fill	120	0	30°	0.30	0.50	5.58
Compacted Granular Engineered Fill	120	0	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).

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 12. Excavation Back Slopes

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

5.4.2 Groundwater Considerations

Based on the groundwater observations made during drilling, groundwater is anticipated at or near the proposed bearing elevation for the shallow foundations at the rear abutment and pier and at the bottom of wall elevation (top of embedded shaft) at the forward abutment. Therefore, groundwater is also anticipated during construction of the drilled shafts at the forward abutment. 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. Given the granular nature of the soils, groundwater may not be able to be controlled by pumping from temporary sumps, and more significant dewatering efforts, such as deep well or well points system will likely be required. Note that determining and maintaining actual groundwater levels during construction of drilled shafts 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.

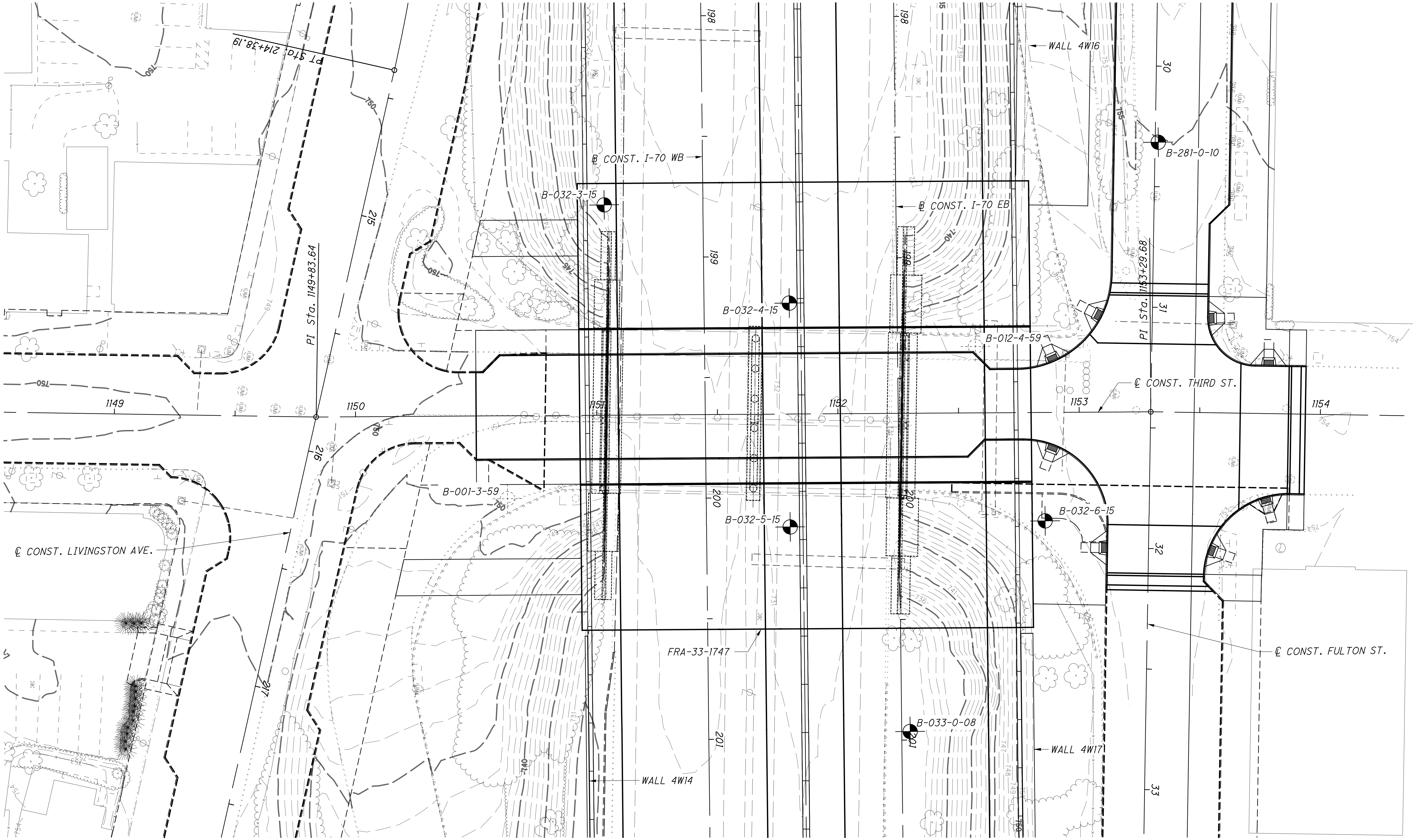
The conclusions and recommendations herein have been based upon the available soil and bedrock information and the design details furnished by a representative of the owner of the proposed project. Any revision in the plans for the proposed construction from those anticipated in this report should be 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 FRA-33-1747 S. THIRD ST. OVER I-70/71

FRA-70-14.05

FRANKLIN COUNTY, OHIO

RII PROJECT NO. W-15-126		DRAWN JAS
SCALE: 1"=20'		REVIEWED BRT
0 20 40		DATE 11-26-18

APPENDIX II

DESCRIPTION OF SOIL TERMS

DESCRIPTION OF SOIL TERMS

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

CLASSIFICATION OF SOILS

Ohio Department of Transportation

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

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

MATERIAL CLASSIFIED BY VISUAL INSPECTION

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

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

DESCRIPTION OF ROCK TERMS

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

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

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

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

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

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

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

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

Degree of Fracturing

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

Aperture Width

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

Surface Roughness

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

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

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

APPENDIX III

PROJECT BORING LOGS:

B-032-2-15 through B-032-6-15

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

<u>Classification Test Data</u>

Gradation (as defined on Description of Soil Terms):

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

Atterberg Limits:

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

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-14.05 PROJECT 4B	DRILLING FIRM / OPERATOR: RII / S.B.	DRILL RIG: CME 750X (SN 310218)	STATION / OFFSET: 197+39.71 / 39.1" RT
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: RII / CD/BW	HAMMER: CME AUTOMATIC	ALIGNMENT: BL CONST. I-70 EB
PID: 96053 BR ID: NA	DRILLING METHOD: 3.25" - HSA	CALIBRATION DATE: 10/20/14	ELEVATION: 733.1 (MSL) EOB: 60.0 ft.
START: 10/6/15 END: 10/6/15	SAMPLING METHOD: SPT	ENERGY RATIO (%): 85.7	COORD: 39.953042, -82.996969

EXPLORATION ID B-032-2-15

MATERIAL DESCRIPTION AND NOTES	ELEV. 733.1	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
0.3' - TOPSOIL (3.0")	732.8																	
HARD, BROWN TO GRAY **SANDY SILT**, SOME CLAY, LITTLE FINE GRAVEL, DAMP.		1–2	5 6 11	24	100	SS-1	4.5+	11	12	20	34	23	25	15	10	10	A-4a (4)	
		4–5	8 14 19	47	100	SS-2	4.5+	-	-	-	-	-	-	-	-	9	A-4a (V)	
-SS-3: SULFATE CONTENT = 907 PPM		6–7	9 15 19	49	100	SS-3	4.5+	-	-	-	-	-	-	-	-	8	A-4a (V)	
		9–10	10 17 21	54	100	SS-4	4.5+	-	-	-	-	-	-	-	-	8	A-4a (V)	
	720.1	11–12	9 10 18	40	100	SS-5	4.5+	-	-	-	-	-	-	-	-	9	A-4a (V)	
VERY DENSE, GRAY **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST.	717.6	14–15	7 18 32	71	100	SS-6	-	36	25	17	14	8	17	11	6	6	A-1-b (0)	
MEDIUM DENSE TO VERY DENSE, GRAY **COARSE AND FINE SAND**, LITTLE SILT, TRACE CLAY, TRACE FINE GRAVEL, MOIST TO WET.		W 16–17	2 3 12	21	100	SS-7	-	5	10	70	12	3	NP	NP	NP	19	A-3a (0)	
	712.6	19–20	9 22 38	86	100	SS-8	-	-	-	-	-	-	-	-	-	15	A-3a (V)	
HARD, GRAY **SANDY SILT**, LITTLE CLAY, LITTLE FINE GRAVEL, DAMP.		21–22	6 21 35	80	100	SS-9	-	12	16	24	35	13	18	12	6	10	A-4a (3)	
		24–25	11 26 48	106	100	SS-10	4.5+	-	-	-	-	-	-	-	-	10	A-4a (V)	
		29	12 47 50/3"	-	100	SS-11	4.5+	13	11	22	38	16	20	11	9	8	A-4a (4)	

PID: 96053	BR ID: NA	PROJECT: FRA-70-14.05 PROJECT 4B	STATION / OFFSET: 197+39.71 / 39.1' RT	START: 10/6/15	END: 10/6/15	PG 2 OF 2	B-032-2-15

MATERIAL DESCRIPTION AND NOTES	ELEV. 703.1	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
HARD, GRAY **SANDY SILT**, LITTLE CLAY, LITTLE FINE GRAVEL, DAMP. *(same as above)*	701.1	31 – 32																
MEDIUM DENSE TO VERY DENSE, GRAY **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, MOIST TO WET.		33																
		34 – 35	23 45 50/4"	-	100	SS-12	-	-	-	-	-	-	-	-	-	12	A-1-b (V)	
		36 – 38																
-HEAVING SANDS ENCOUNTERED @ 38.5' -WATER ADDED TO AUGERS @ 38.5'		39 – 40	17 41 50/5"	-	88	SS-13	-	-	-	-	-	-	-	-	-	8	A-1-b (V)	
		41 – 43																
		44 – 45	4 8 13	30	100	SS-14	-	21	55	17	5	2	NP	NP	NP	13	A-1-b (0)	
		46 – 48																
		49 – 50	9 18 33	73	100	SS-15	-	-	-	-	-	-	-	-	-	13	A-1-b (V)	
		51 – 53																
		54 – 55	11 42 50/4"	-	100	SS-16	-	11	47	30	8	4	NP	NP	NP	13	A-1-b (0)	
		56 – 58																
	673.1	59 – 60 EOB	16 44 47	130	67	SS-17	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 16.0'; CAVE-IN DEPTH @ 17.0'

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

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-14.05 PROJECT 4B	DRILLING FIRM / OPERATOR: RII / S.B.	DRILL RIG: CME 750X (SN 310218)	STATION / OFFSET: 198+77.78 / 40.8" RT
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: RII / C.D.	HAMMER: CME AUTOMATIC	ALIGNMENT: BL CONST. I-70 EB
PID: 96053 BR ID: FRA-33-1747	DRILLING METHOD: 3.25" - HSA	CALIBRATION DATE: 10/20/14	ELEVATION: 732.8 (MSL) EOB: 75.0 ft.
START: 10/7/15 END: 10/8/15	SAMPLING METHOD: SPT	ENERGY RATIO (%): 85.7	COORD: 39.953103, -82.996483

EXPLORATION ID B-032-3-15

MATERIAL DESCRIPTION AND NOTES	ELEV. 732.8
0.3' - TOPSOIL (3.0")	732.5
MEDIUM DENSE TO VERY DENSE, BROWN **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST. -ROCK FRAGMENTS PRESENT THROUGHOUT -COBBLES PRESENT @ 8.0'	722.3
DENSE TO VERY DENSE, BROWNISH GRAY TO GRAY **GRAVEL**, AND COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, WET. -MUD ADDED TO AUGERS @ 11.0' -ROCK FRAGMENTS PRESENT THROUGHOUT -HEAVING SANDS ENCOUNTERED @ 18.5' -COBBLES PRESENT FROM 18.5' TO 21.0'	708.3
HARD, GRAY **SANDY SILT**, LITTLE FINE GRAVEL, LITTLE CLAY, DAMP.	

DEPTHS: 1–29

W

SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
4 7 8	21	100	SS-1	-	-	-	-	-	-	-	-	-	6	A-1-b (V)	
8 13 13	37	100	SS-2	-	-	-	-	-	-	-	-	-	5	A-1-b (V)	
11 14 24	54	78	SS-3	-	57	21	6	13	3	NP	NP	NP	7	A-1-b (0)	
14 18 15	47	100	SS-4	-	-	-	-	-	-	-	-	-	7	A-1-b (V)	
9 12 78	129	83	SS-5	-	99	1	0	0	0	NP	NP	NP	10	A-1-a (0)	
26 20 20	57	89	SS-6	-	-	-	-	-	-	-	-	-	11	A-1-a (V)	
7 13 16	41	100	SS-7	-	61	21	8	7	3	NP	NP	NP	11	A-1-a (0)	
6 11 14	36	100	SS-8	-	-	-	-	-	-	-	-	-	9	A-1-a (V)	
9 10 14	34	100	SS-9	-	55	27	9	7	2	NP	NP	NP	12	A-1-a (0)	
17 11 21	46	100	SS-10A	-	-	-	-	-	-	-	-	-	8	A-1-a (V)	
			SS-10B	4.5+	-	-	-	-	-	-	-	-	11	A-4a (V)	
16 23 35	83	100	SS-11	4.5+	19	11	18	37	15	21	14	7	10	A-4a (3)	
8 15 23	54	100	SS-12	4.5+	-	-	-	-	-	-	-	-	12	A-4a (V)	

PID: 96053	BR ID: FRA-33-1747	PROJECT: FRA-70-14.05 PROJECT 4B	STATION / OFFSET: 198+77.78 / 40.8' RT	START: 10/7/15	END: 10/8/15	PG 2 OF 3	B-032-3-15

MATERIAL DESCRIPTION AND NOTES	ELEV. 702.8	DEPTHS
HARD, GRAY **SANDY SILT**, LITTLE FINE GRAVEL, LITTLE CLAY, DAMP. *(same as above)*	700.8	31 – 32
VERY DENSE, GRAY **SILT**, SOME COARSE TO FINE SAND, TRACE CLAY, TRACE FINE GRAVEL, MOIST.	695.8	32 – 37
VERY DENSE, DARK GRAY TO GRAY **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, WET. -COBBLES PRESENT @ 41.0' -HEAVING SANDS ENCOUNTERED @ 43.5' -HEAVING SANDS ENCOUNTERED @ 48.5' -COBBLES PRESENT @ 50.5'	670.8	37 – 62

DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)
34	28 48 50/4"	-	100	SS-13	1.50	5	5	22	61	7	NP	NP	NP	17	A-4b (7)
39	26 26 36	89	100	SS-14	-	-	-	-	-	-	-	-	-	14	A-1-b (V)
44	15 47 50/2"	-	100	SS-15	-	-	-	-	-	-	-	-	-	8	A-1-b (V)
49	8 23 38	87	100	SS-16	-	36	28	26	9	1	NP	NP	NP	11	A-1-b (0)
54	32 30 47	110	100	SS-17	-	44	27	18	9	2	NP	NP	NP	10	A-1-b (0)
59	10 40 46	123	100	SS-18	-	-	-	-	-	-	-	-	-	15	A-1-b (V)

PID: 96053	BR ID: FRA-33-1747	PROJECT: FRA-70-14.05 PROJECT 4B	STATION / OFFSET: 198+77.78 / 40.8' RT	START: 10/7/15	END: 10/8/15	PG 3 OF 3	B-032-3-15

MATERIAL DESCRIPTION AND NOTES	ELEV. 670.7	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
VERY DENSE, GRAY **SILT**, SOME COARSE TO FINE SAND, TRACE CLAY, WET. *(same as above)*		63																
		64	9 20 48	97	100	SS-19	-	0	1	20	74	5	NP	NP	NP	23	A-4b (8)	
-COBBLES PRESENT @ 66.0'	665.8	65–67																
MEDIUM DENSE TO DENSE, GRAY **COARSE AND FINE SAND**, TRACE SILT, TRACE FINE GRAVEL, TRACE CLAY, MOIST. -HEAVING SANDS ENCOUNTERED @ 68.5'		68																
		69	6 9 10	27	100	SS-20	-	5	40	42	10	3	NP	NP	NP	14	A-3a (0)	
		70–73																
	657.8	74	7 11 11	31	100	SS-21	-	-	-	-	-	-	-	-	-	12	A-3a (V)	
		EOB 75																

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 11.5'

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

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-14.05 PROJECT 4B	DRILLING FIRM / OPERATOR: RII / S.B.	DRILL RIG: CME 55 (SN 386345)	STATION / OFFSET: 1151+80.18 / 46.1" LT
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: RII / C.D.	HAMMER: CME AUTOMATIC	ALIGNMENT: CL CONST. THIRD ST.
PID: 96053 BR ID: FRA-33-1747	DRILLING METHOD: 3.25" - HSA	CALIBRATION DATE: 10/20/14	ELEVATION: 732.5 (MSL) EOB: 75.0 ft.
START: 1/6/16 END: 1/7/16	SAMPLING METHOD: SPT	ENERGY RATIO (%): 92	COORD: 39.953329, -82.996384

EXPLORATION ID B-032-4-15

MATERIAL DESCRIPTION AND NOTES	ELEV. 732.5
0.8' - ASPHALT (9.0")	731.7
0.7' - AGGREGATE BASE (9.0")	731.0
DENSE TO VERY DENSE, BROWN, GRAY AND BLACK TO BROWNISH GRAY **GRAVEL WITH SAND AND SILT**, TRACE CLAY, MOIST. -INTRODUCED MUD @ 11.5'	714.5
MEDIUM DENSE, BROWNISH GRAY **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, WET.	712.0
DENSE TO VERY DENSE, BROWNISH GRAY TO GRAY **GRAVEL**, LITTLE TO SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST.	

Water level mark: W at 11.5'

SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
8 / 7 / 11	27	78	SS-1	-	-	-	-	-	-	-	-	-	8	A-2-4 (V)
9 / 11 / 12	35	100	SS-2	-	-	-	-	-	-	-	-	-	7	A-2-4 (V)
14 / 29 / 19	72	61	SS-3	-	96	2	1	0	1	24	17	7	5	A-2-4 (0)
17 / 14 / 12	39	100	SS-4	-	-	-	-	-	-	-	-	-	6	A-2-4 (V)
20 / 26 / 24	75	0	SS-5	-	-	-	-	-	-	-	-	-	-	
46	-	100	SS-5A	-	60	20	6	9	5	24	17	7	10	A-2-4 (0)
14 / 15 / 16	47	100	SS-6	-	-	-	-	-	-	-	-	-	9	A-2-4 (V)
12 / 26 / 18	66	72	SS-7	-	-	-	-	-	-	-	-	-	10	A-2-4 (V)
11 / 13 / 12	38	100	SS-8	-	39	36	15	5	5	NP	NP	NP	19	A-1-b (0)
7 / 45 / 20	98	67	SS-9	-	-	-	-	-	-	-	-	-	11	A-1-a (V)
50/2"	-	100	SS-10	-	-	-	-	-	-	-	-	-	8	A-1-a (V)
7 / 14 / 19	50	83	SS-11	-	61	22	6	6	5	22	17	5	13	A-1-a (0)
15 / 18 / 22	60	94	SS-12	-	-	-	-	-	-	-	-	-	8	A-1-a (V)

PID: 96053	BR ID: FRA-33-1747	PROJECT: FRA-70-14.05 PROJECT 4B	STATION / OFFSET: 1151+80.18 / 46.1' LT	START: 1/6/16	END: 1/7/16	PG 2 OF 3	B-032-4-15

MATERIAL DESCRIPTION AND NOTES	ELEV. 702.5	DEPTHS
DENSE TO VERY DENSE, BROWNISH GRAY TO GRAY **GRAVEL**, LITTLE TO SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST. *(same as above)* -LIMESTONE FRAGMENTS PRESENT IN SS-15	685.5	30 – 47
HARD, GRAY **SILT AND CLAY**, SOME COARSE TO FINE SAND, SOME FINE GRAVEL, DAMP.	683.0	47 – 49.5
DENSE, GRAY **GRAVEL WITH SAND AND SILT**, TRACE CLAY, MOIST.	680.5	49.5 – 52
HARD, GRAY **SILT AND CLAY**, SOME FINE GRAVEL, SOME COARSE TO FINE SAND, MOIST.	678.5	52 – 54
VERY DENSE, GRAY **GRAVEL WITH SAND AND SILT**, TRACE CLAY, MOIST.	670.5	54 – 62

DEPTHS	SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
33.5 – 35	11 17 26	65	83	SS-13	-	-	-	-	-	-	-	-	-	8	A-1-a (V)
38.5 – 40	16 28 22	75	89	SS-14	-	76	14	4	3	3	20	15	5	5	A-1-a (0)
43.5 – 45	14 25 38	95	67	SS-15	-	-	-	-	-	-	-	-	-	7	A-1-a (V)
48.5 – 50	5 15 31	69	100	SS-16A	4.5+	-	-	-	-	-	-	-	-	12	A-6a (V)
				SS-16B	-	-	-	-	-	-	-	-	-	9	A-2-4 (V)
53.5 – 55	6 12 14	39	78	SS-17A	4.5+	33	14	13	23	17	25	14	11	10	A-6a (1)
				SS-17B	-	-	-	-	-	-	-	-	-	13	A-2-4 (V)
58.5 – 60	7 36 33	104	100	SS-18	-	-	-	-	-	-	-	-	-	9	A-2-4 (V)

BACK FILL

0-2018-ODOT BORING LOG RII - OH DOT.GDT - 11/26/18 12:23 - U:\GI8\PROJECTS\2015\W-15-126.GPJ

PID: 96053	BR ID: FRA-33-1747	PROJECT: FRA-70-14.05 PROJECT 4B	STATION / OFFSET: 1151+80.18 / 46.1' LT	START: 1/6/16	END: 1/7/16	PG 3 OF 3	B-032-4-15

MATERIAL DESCRIPTION AND NOTES	ELEV. 670.4	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 **SILT**, SOME COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST. *(same as above)* -ORGANICS AND WOOD FRAGMENTS PRESENT IN SS-19A	668.0	63 - 64	5 17 40	86	100	SS-19A	-	1	1	25	63	10	NP	NP	NP	17	A-4b (8)	
VERY DENSE, GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST. -ROCK FRAGMENTS PRESENT IN SS-19B	665.5	65 - 67				SS-19B	-	-	-	-	-	-	-	-	-	8	A-1-a (V)	
VERY DENSE, GRAY **COARSE AND FINE SAND**, SOME FINE GRAVEL, LITTLE SILT, TRACE CLAY, MOIST.		68 - 69	11 47 50/3"	-	100	SS-20	-	-	-	-	-	-	-	-	-	18	A-3a (V)	
	657.5	70 - 75 EOB	9 38 44	123	100	SS-21	-	-	-	-	-	-	-	-	-	8	A-3a (V)	

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 11.5'

ABANDONMENT METHODS, MATERIALS, QUANTITIES: COMPACTED WITH THE AUGER 150 LBS BENTONITE CHIPS AND SOIL CUTTINGS .

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-14.05 PROJECT 4B	DRILLING FIRM / OPERATOR: RII / S.B.	DRILL RIG: CME 55 (SN 386345)	STATION / OFFSET: 1151+79.91 / 46.7" RT	EXPLORATION ID B-032-5-15
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: RII / C.D.	HAMMER: CME AUTOMATIC	ALIGNMENT: CL CONST. THIRD ST.	
PID: 96053 BR ID: FRA-33-1747	DRILLING METHOD: 3.25" - HSA	CALIBRATION DATE: 10/20/14	ELEVATION: 731.6 (MSL) EOB: 75.0 ft.	PAGE 1 OF 3
START: 1/4/16 END: 1/5/16	SAMPLING METHOD: SPT	ENERGY RATIO (%): 92	COORD: 39.953371, -82.996058	

MATERIAL DESCRIPTION AND NOTES	ELEV. 731.6
0.8' - ASPHALT (9.0")	730.8
1.7' - AGGREGATE BASE (21.0")	729.1
DENSE, BROWN TO BROWNISH GRAY **GRAVEL**, SOME COARSE TO FINE SAND, LITTLE SILT, TRACE CLAY, DAMP TO MOIST. -INTRODUCED WATER @ 11.0'	718.6
DENSE TO VERY DENSE, BROWN, GRAY AND BLACK **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST.	712.1
HARD, BROWNISH GRAY **SANDY SILT**, LITTLE CLAY, LITTLE FINE GRAVEL, DAMP.	703.6
VERY DENSE, GRAY **COARSE AND FINE SAND**, LITTLE FINE GRAVEL, LITTLE SILT, TRACE CLAY, MOIST.	

DEPTHS: 1 – 29 (W at 10)

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
10 9 9	27	33	SS-1	-	49	24	12	11	4	17	15	2	5	A-1-b (0)	
7 11 16	41	0	SS-2	-	-	-	-	-	-	-	-	-	-		
18	-	100	3S-2A	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	
8 12 14	39	67	SS-3	-	-	-	-	-	-	-	-	-	7	A-1-a (V)	
8 10 11	32	89	SS-4	-	-	-	-	-	-	-	-	-	11	A-1-a (V)	
7 14 11	38	100	SS-5	-	54	24	7	11	4	23	17	6	13	A-1-a (0)	
7 13 14	41	100	SS-6	-	-	-	-	-	-	-	-	-	8	A-1-b (V)	
11 12 13	38	100	SS-7	-	-	-	-	-	-	-	-	-	12	A-1-b (V)	
39 16 22	57	100	SS-8A	-	44	29	12	11	4	NP	NP	NP	13	A-1-b (0)	
			SS-8B	4.5+	-	-	-	-	-	-	-	-	10	A-4a (V)	
35 46 50/5"	-	100	SS-9	4.5+	12	11	23	37	17	21	13	8	9	A-4a (4)	
9 32 43	113	100	SS-10	4.5+	-	-	-	-	-	-	-	-	9	A-4a (V)	
30 48 50/3"	-	100	SS-11	4.5+	-	-	-	-	-	-	-	-	10	A-4a (V)	
19 29 44	110	100	SS-12	-	-	-	-	-	-	-	-	-	21	A-3a (V)	

PID: 96053	BR ID: FRA-33-1747	PROJECT: FRA-70-14.05 PROJECT 4B	STATION / OFFSET: 1151+79.91 / 46.7' RT	START: 1/4/16	END: 1/5/16	PG 2 OF 3	B-032-5-15

MATERIAL DESCRIPTION AND NOTES	ELEV. 701.6	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 **COARSE AND FINE SAND**, LITTLE FINE GRAVEL, LITTLE SILT, TRACE CLAY, MOIST. *(same as above)*	699.6	31 – 32																
VERY DENSE, GRAY **SANDY SILT**, TRACE CLAY, TRACE FINE GRVAEL, MOIST. -HEAVING SANDS ENCOUNTERED @ 33.5'		34	11 44 46	135	100	SS-13	-	1	3	59	32	5	NP	NP	NP	14	A-4a (0)	
	694.6	35 – 37																
VERY DENSE, GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST. -HEAVING SANDS ENCOUNTERED @ 38.5'		39	12 45 50/4"	-	100	SS-14	-	-	-	-	-	-	-	-	-	7	A-1-a (V)	
-HEAVING SANDS ENCOUNTERED @ 43.5'		44	25 50/5"	-	100	SS-15	-	75	16	5	3	1	NP	NP	NP	7	A-1-a (0)	
	684.6	45 – 47																
VERY DENSE, BROWNISH GRAY **SANDY SILT**, LITTLE CLAY, TRACE FINE GRAVEL, DAMP.		49	50/1"	-	100	SS-16	-	-	-	-	-	-	-	-	-	7	A-4a (V)	
		54	50/1"	-	0	SS-17	-	-	-	-	-	-	-	-	-	-		
	674.6	55 – 57																
VERY DENSE, BROWNISH GRAY **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, MOIST.		59	40 38 28	99	44	SS-18	-	32	30	26	9	3	NP	NP	NP	11	A-1-b (0)	
		60 – 61																

PID: 96053	BR ID: FRA-33-1747	PROJECT: FRA-70-14.05 PROJECT 4B	STATION / OFFSET: 1151+79.91 / 46.7' RT	START: 1/4/16	END: 1/5/16	PG 3 OF 3	B-032-5-15

MATERIAL DESCRIPTION AND NOTES	ELEV. 669.5	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
VERY DENSE, BROWNISH GRAY **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, MOIST. *(same as above)*		63																
		64	28 30 41	107	67	SS-19	-	-	-	-	-	-	-	-	-	14	A-1-b (V)	
		65–68																
		69	30 50/5"	-	100	SS-20	-	41	27	24	6	2	NP	NP	NP	14	A-1-b (0)	
	659.6	70–71																
VERY DENSE, GRAY **COARSE AND FINE SAND**, TRACE SILT, TRACE FINE GRAVEL, DAMP.		72–73																
	656.6	EOB 74–75	12 28 40	102	100	SS-21	-	-	-	-	-	-	-	-	-	7	A-3a (V)	

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 10.5'

ABANDONMENT METHODS, MATERIALS, QUANTITIES: COMPACTED WITH THE AUGER SOIL CUTTINGS .

PROJECT: FRA-70-14.05 PROJECT 4B	DRILLING FIRM / OPERATOR: RII / S.B.	DRILL RIG: CME 750X (SN 310218)	STATION / OFFSET: 1152+85.60 / 44.9" RT
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: RII / C.D.	HAMMER: CME AUTOMATIC	ALIGNMENT: CL CONST. THIRD ST.
PID: 96053 BR ID: FRA-33-1747	DRILLING METHOD: 3.25" - HSA	CALIBRATION DATE: 10/20/14	ELEVATION: 753.0 (MSL) EOB: 80.0 ft.
START: 10/5/15 END: 10/5/15	SAMPLING METHOD: SPT	ENERGY RATIO (%): 85.7	COORD: 39.953656, -82.996128

EXPLORATION ID **B-032-6-15**

MATERIAL DESCRIPTION AND NOTES	ELEV. 753.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
0.3' - ASPHALT (4.0")	752.7																	
0.5' - CONCRETE (6.0")	752.2																	
2.2' - AGGREGATE BASE (26.0")	750.0																	
STIFF, BROWN **SILT AND CLAY**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP.	746.0	4	3 5 6	16	100	SS-1	2.00	-	-	-	-	-	-	-	-	9	A-6a (V)	
DENSE TO VERY DENSE, GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST.		9	4 11 22	47	67	SS-2	-	70	11	5	10	4	21	17	4	10	A-1-a (0)	
		14	17 22 16	54	89	SS-3	-	-	-	-	-	-	-	-	-	4	A-1-a (V)	
		19	2 11 14	36	78	SS-4	-	-	-	-	-	-	-	-	-	6	A-1-a (V)	
		21	9 14 13	39	100	SS-5	-	-	-	-	-	-	-	-	-	7	A-1-a (V)	
-ROCK FRAGMENTS PRESENT THROUGHOUT		24	12 18 22	57	72	SS-6	-	59	18	8	10	5	NP	NP	NP	6	A-1-a (0)	
		26	8 17 20	53	94	SS-7	-	-	-	-	-	-	-	-	-	6	A-1-a (V)	
		29	15 21 27	69	100	SS-8	-	62	18	7	8	5	NP	NP	NP	5	A-1-a (V)	

PID: 96053	BR ID: FRA-33-1747	PROJECT: FRA-70-14.05 PROJECT 4B	STATION / OFFSET: 1152+85.60 / 44.9' RT	START: 10/5/15	END: 10/5/15	PG 2 OF 3	B-032-6-15

MATERIAL DESCRIPTION AND NOTES	ELEV. 723.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
DENSE TO VERY DENSE, GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST. *(same as above)* -INTRODUCED MUD @ 31.0'		W 31 32	5 11 19	43	67	SS-9	-	-	-	-	-	-	-	-	-	9	A-1-a (V)	
		33 34 35	3 14 19	47	100	SS-10	-	60	23	7	7	3	NP	NP	NP	10	A-1-a (0)	
		36 37	9 23 31	77	78	SS-11	-	-	-	-	-	-	-	-	-	8	A-1-a (V)	
	712.5	38 39 40	4 26 28	77	100	SS-12	-	55	25	8	8	4	NP	NP	NP	9	A-1-a (0)	
DENSE, BROWN TO GRAY **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, MOIST.		41 42	3 9 16	36	100	SS-13	-	-	-	-	-	-	-	-	-	11	A-1-b (V)	
	707.5	43 44 45	10 13 14	39	100	SS-14	-	31	45	11	9	4	NP	NP	NP	14	A-1-b (0)	
DENSE, GRAY **GRAVEL**, LITTLE COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST.		46 47	5 13 18	44	100	SS-15	-	80	9	3	5	3	20	15	5	11	A-1-a (0)	
	701.0	48 49 50 51 52	5 12 12	34	100	SS-16	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	
DENSE, BROWN **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, MOIST.	696.0	53 54 55 56 57	4 10 11	30	100	SS-17	-	9	51	29	8	3	NP	NP	NP	15	A-1-b (0)	
VERY DENSE, BROWN TO BROWNISH GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST.		58 59 60 61	6 18 24	60	100	SS-18	-	-	-	-	-	-	-	-	-	7	A-1-a (V)	

PID: 96053	BR ID: FRA-33-1747	PROJECT: FRA-70-14.05 PROJECT 4B	STATION / OFFSET: 1152+85.60 / 44.9' RT	START: 10/5/15	END: 10/5/15	PG 3 OF 3	B-032-6-15

MATERIAL DESCRIPTION AND NOTES	ELEV. 690.9	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, BROWN TO BROWNISH GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST. *(same as above)*		63																
		64–65	5 19 20	56	100	SS-19	-	-	-	-	-	-	-	-	-	16	A-1-a (V)	
		66–68																
		69–70	12 36 34	100	100	SS-20	-	57	18	19	4	2	NP	NP	NP	16	A-1-a (0)	
		71–73																
-HEAVING SANDS ENCOUNTERED @ 73.5'		74–75	13 41 50/5"	-	100	SS-21	-	-	-	-	-	-	-	-	-	13	A-1-a (V)	
		76–78																
	673.0	EOB 79–80	11 37 43	114	100	SS-22	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 31.0'

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

APPENDIX IV

HISTORIC BORING LOGS:

B-001-3-59 and B-010-3-59

B-001-3-59

STATE OF OHIO
DEPARTMENT OF HIGHWAYS
TESTING LABORATORY

LOG OF BORING

CO., RT. NO., SEC. FRA-40- BRIDGE NO. FRA-40-
REAR ABUTMENT SOUTH INNERBELT UNDER S. THIRD ST.

LOCATION: T.H. 1 B STA. 50+63 OFFSET 33' RT. FED. NO.

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
751.2	0			
	2			
	4			
746.2	6	4/4	20641	Brown Clayey Sandy Gravel
	8			
741.2	10	9/5	-----	Brown Clayey Sandy Gravel
738.7	12	5/6	20642	Brown Clayey Sandy Gravel
736.2	14, 16	5/5	20643	Brown Silty Sandy Gravel
733.7	18	13/25	20644	Brown Silty Sandy Gravel
731.2	20	10/6	20645	Brown Silty Sandy Gravel
730.7	22	17/12	20646	Gray Silty Sandy Gravel
726.2	24, 26	10/13	20647	Gray Silty Sandy Gravel
723.7	28	12/12	20648	Gray Silty Sandy Gravel
721.2	30	18/18	20649	Gray Silty Sandy Gravel
	32			
	34			
716.2	36	16/19	20650	Gray Silty Sandy Gravel

BS
7·7·59

B-001-3-59

LOG OF BORING (CONTINUED)

SHEET 4

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

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
	38			
711.2	40			
		11/16	20651	Gray Silty Gravelly Sand
	42			
	44			
706.2	46	24/40	20652	Gray Silty Sandy Gravel
	48			
701.2	50			
		32/50	20653	Gray Silty Sandy Gravel
	52			
	54			
696.2				
695.2	56	33/55	20654	Gray Sandy Gravel
				BOTTOM OF BORING
	58			
	60			
	62			
	64			
	66			
	68			
	70			
	72			
	74			
	76			
	78			
	80			
	82			

B-010-3-59

STATE OF OHIO
DEPARTMENT OF HIGHWAYS
TESTING LABORATORY

LOG OF BORING

CO., RT. NO., SEC. FRA-40- BRIDGE NO. FRA-40-1325
FORWARD ABUTMENT SOUTH INNERBELT UNDER 3RD STREET

LOCATION: T.H. 10B STA. 52+56 OFFSET 33' LT FED. NO.

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
753.5	0			
	2			
	4			
748.5	6	5/18	21480	Brown Sandy Gravelly Silt
	8	80/*	21481	Brown Silty Sandy Gravel
743.5	10			
	12			
741.0	14	20/38	21482	Reddish Brown Bouldery Silty Sandy Gravel
738.5	16	60/*	21483	Reddish Brown Bouldery Silty Sandy Gravel
735.0	18			
733.5	20	32/40	21484	Gray Bouldery Silty Sandy Gravel
		35/47	21485	Gray Bouldery Silty Sandy Gravel
731.0	22			
	24	36/70	21486	Gray Bouldery Silty Sandy Gravel
728.5	26	20/30	21487	Gray Bouldery Silty Sand Gravel
726.0	28			
		15/27	21488	Gray Gravelly Sand
723.5	30	20/37	21489	Gray Silty Gravelly Sand
	32			
	34			
718.5	36	13/22	21490	Gray Sand

*Refusal

BS
7-7-59

B-010-3-59

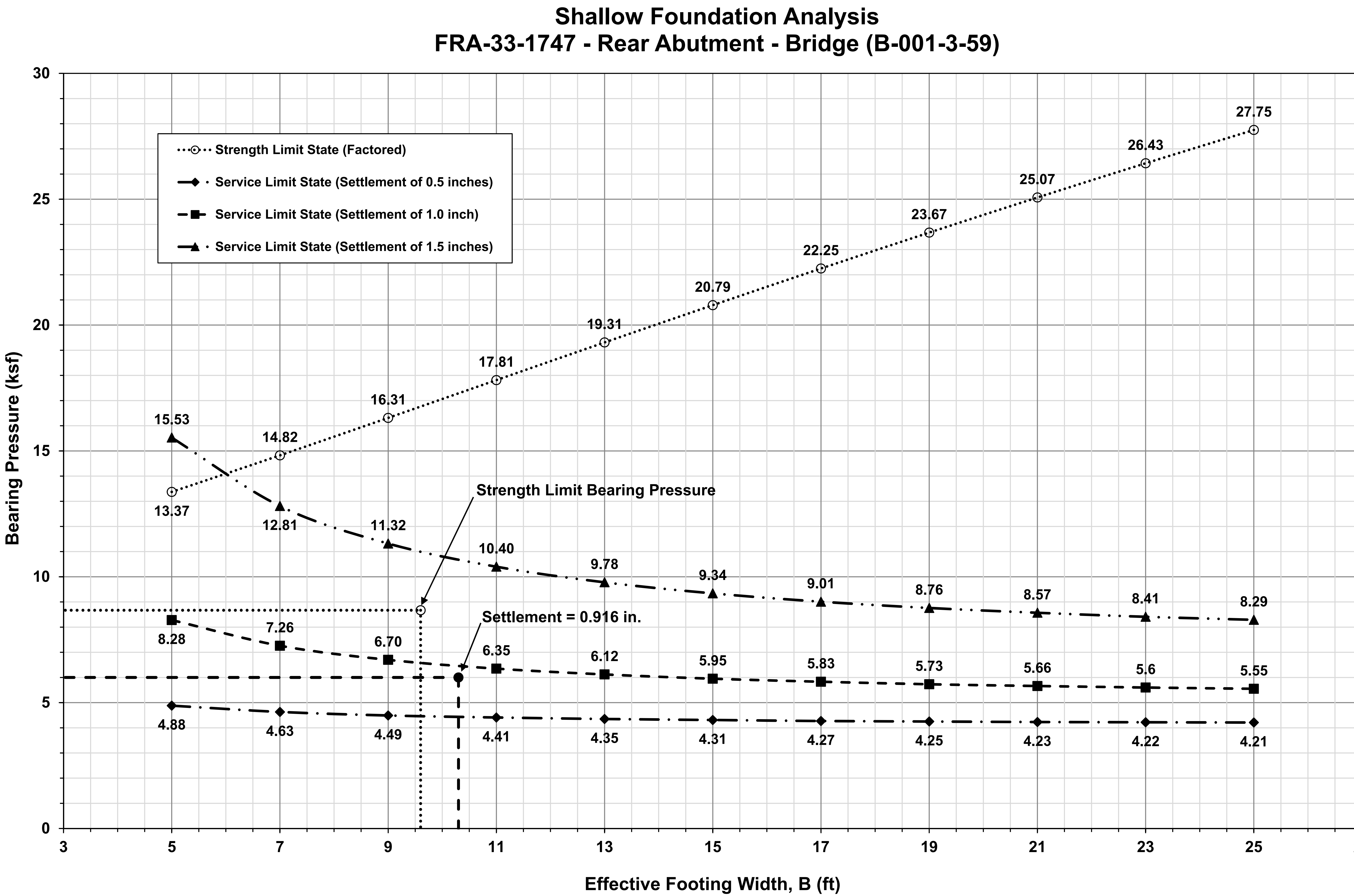
LOG OF BORING (CONTINUED)

SHEET 6

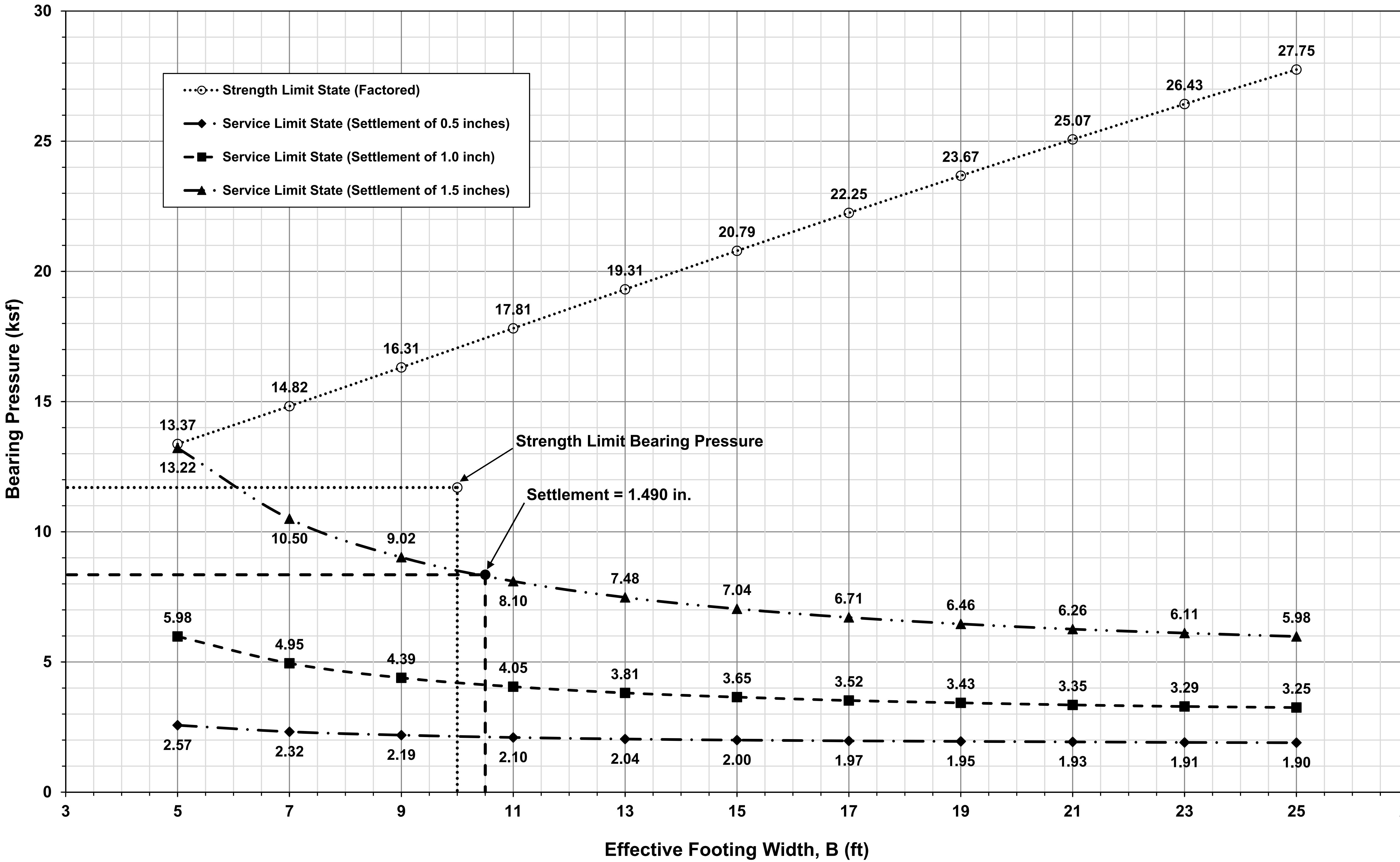
BRIDGE NO. FRA-40-1325 T.H. 10 B

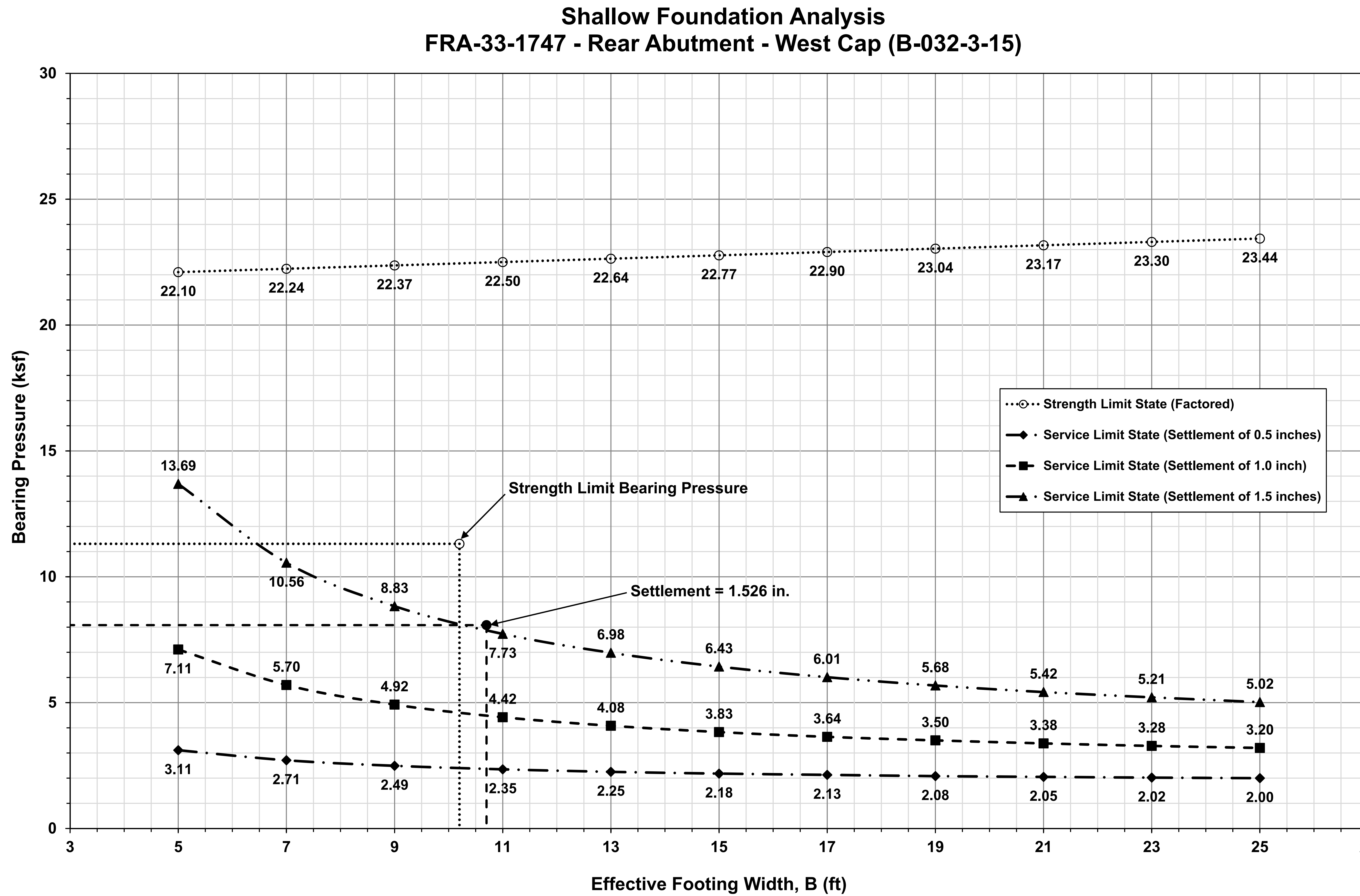
ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
	38			
713.5	40	25/53	21491	Gray Gravelly Sand
	42			
	44			
708.5	46	35/65	21492	Gray Silty Sand
	48			
703.5	50	25/45	21493	Gray Silty Gravelly Sand
	52			
	54			
698.5 697.5	56	20/30	21494	Gray Silty Sand BOTTOM OF BORING
	58			
	60			
	62			
	64			
	66			
	68			
	70			
	72			
	74			
	76			
	78			
	80			
	82			

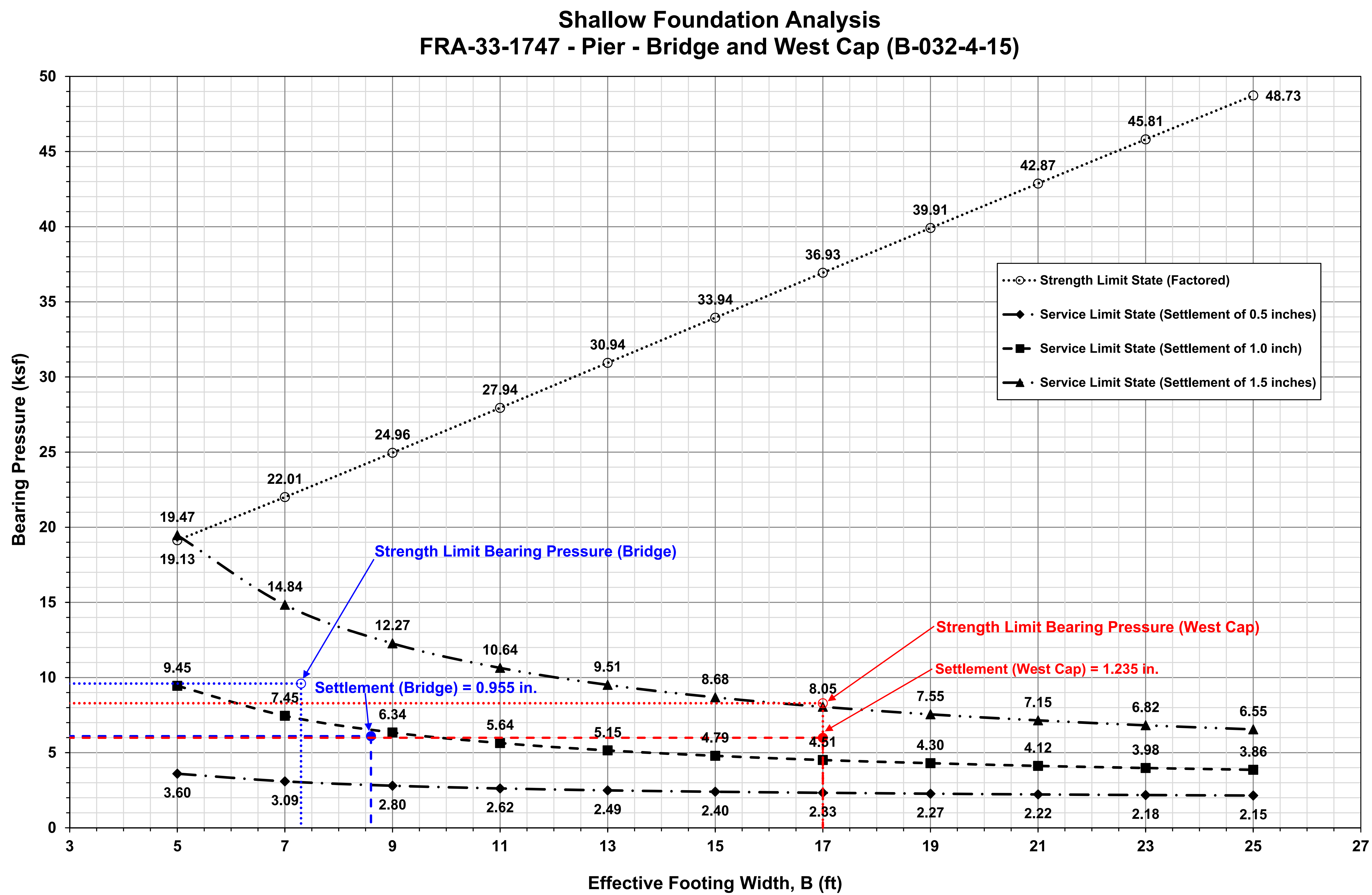
APPENDIX V
BEARING RESISTANCE CHARTS


Shallow Foundation Analysis
FRA-33-1747 - Rear Abutment - Bridge (B-001-3-59)
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)
Strength Limit Bearing Pressure
Settlement = 0.916 in.
Bearing Pressure (ksf)
Effective Footing Width, B (ft)
13.37
14.82
16.31
17.81
19.31
20.79
22.25
23.67
25.07
26.43
27.75
15.53
12.81
11.32
10.40
9.78
9.34
9.01
8.76
8.57
8.41
8.29
8.28
7.26
6.70
6.35
6.12
5.95
5.83
5.73
5.66
5.6
5.55
4.88
4.63
4.49
4.41
4.35
4.31
4.27
4.25
4.23
4.22
4.21
0
5
10
15
20
25
30
3
5
7
9
11
13
15
17
19
21
23
25
27

Shallow Foundation Analysis
FRA-33-1747 - Rear Abutment - East Cap (B-001-3-59)




Shallow Foundation Analysis
FRA-33-1747 - Rear Abutment - West Cap (B-032-3-15)
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)
Strength Limit Bearing Pressure
Settlement = 1.526 in.
22.10
22.24
22.37
22.50
22.64
22.77
22.90
23.04
23.17
23.30
23.44
13.69
10.56
8.83
7.73
6.98
6.43
6.01
5.68
5.42
5.21
5.02
7.11
5.70
4.92
4.42
4.08
3.83
3.64
3.50
3.38
3.28
3.20
3.11
2.71
2.49
2.35
2.25
2.18
2.13
2.08
2.05
2.02
2.00


Shallow Foundation Analysis
FRA-33-1747 - Pier - Bridge and West Cap (B-032-4-15)
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)
Strength Limit Bearing Pressure (Bridge)
Settlement (Bridge) = 0.955 in.
Strength Limit Bearing Pressure (West Cap)
Settlement (West Cap) = 1.235 in.
19.13
22.01
24.96
27.94
30.94
33.94
36.93
39.91
42.87
45.81
48.73
19.47
14.84
12.27
10.64
9.51
8.68
8.05
7.55
7.15
6.82
6.55
9.45
7.45
6.34
5.64
5.15
4.79
4.51
4.30
4.12
3.98
3.86
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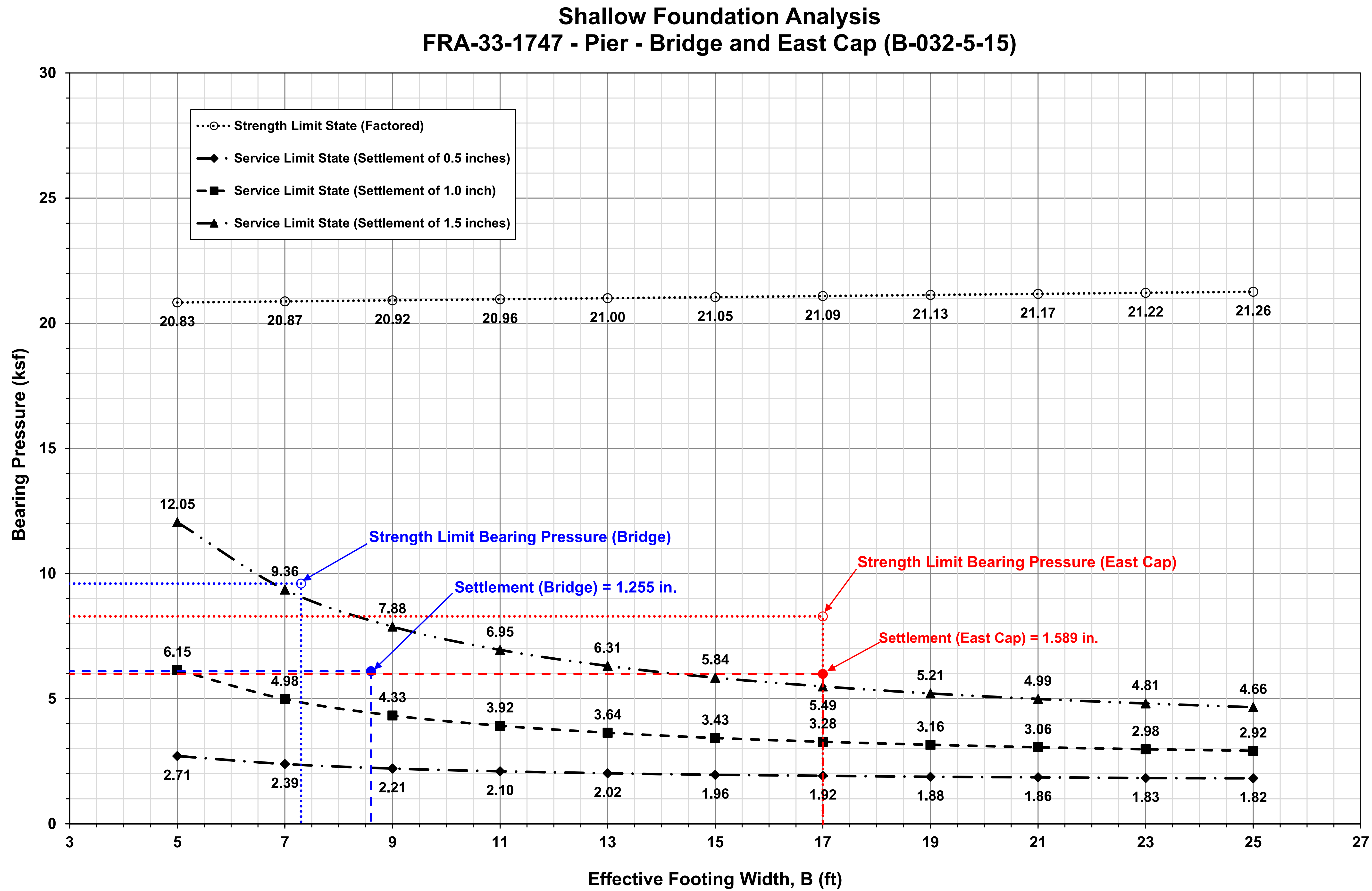
2.80
2.62
2.49
2.40
2.33
2.27
2.22
2.18
2.15


Shallow Foundation Analysis
FRA-33-1747 - Pier - Bridge and East Cap (B-032-5-15)
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)
20.83
20.87
20.92
20.96
21.00
21.05
21.09
21.13
21.17
21.22
21.26
12.05
9.36
7.88
6.95
6.31
5.84
5.49
5.21
4.99
4.81
4.66
6.15
4.98
4.33
3.92
3.64
3.43
3.28
3.16
3.06
2.98
2.92
2.71
2.39
2.21
2.10
2.02
1.96
1.92
1.88
1.86
1.83
1.82
Strength Limit Bearing Pressure (Bridge)
Settlement (Bridge) = 1.255 in.
Strength Limit Bearing Pressure (East Cap)
Settlement (East Cap) = 1.589 in.
Bearing Pressure (ksf)
Effective Footing Width, B (ft)

APPENDIX VI

SHALLOW FOUNDATION CALCULATIONS

W-15-126 - FRA-70-14.05 Project 4B - FRA-33-1747 S. Third Street over I-70/71
Shallow Foundation Analysis - Rear Abutment - Bridge - Settlement

Calculated By: BRT Date: 6/26/2022
Checked By: JPS Date: 6/27/2022

Boring B-001-3-59

B =	10.3	ft	Effective Footing width
D_w =	0.0	ft	Depth below bottom of footing
q =	6,000	psf	Service limit bearing pressure at bottom of wall
q_{net} =	2,376	psf	Net bearing pressure at bottom of footing (considers initial overburden stress of 3,624 psf from 30.2-foot cut to bottom of footing elevation)

Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	N_{60}	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_m (1) (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
A-1-b	G	0.0	1.9	1.9	1.0	31	130	247	124	64							62	225	0.09	0.997	2,370	2,434	0.013	0.160
A-1-b	G	1.9	3.9	2.0	2.9	31	130	507	377	196							55	192	0.28	0.946	2,247	2,443	0.011	0.137
A-1-b	G	3.9	6.4	2.5	5.2	31	130	832	670	348							49	166	0.50	0.818	1,944	2,292	0.012	0.148
A-1-b	G	6.4	8.9	2.5	7.7	31	130	1,157	995	517							45	149	0.74	0.672	1,597	2,114	0.010	0.123
A-3a	G	8.9	11.4	2.5	10.2	27	125	1,470	1,313	680							37	103	0.99	0.556	1,321	2,000	0.011	0.137
A-3a	G	11.4	13.9	2.5	12.7	27	130	1,795	1,632	843							35	98	1.23	0.468	1,113	1,956	0.009	0.112
A-1-b	G	13.9	16.4	2.5	15.2	64	140	2,145	1,970	1,024							78	300	1.47	0.403	957	1,981	0.002	0.029
A-1-b	G	16.4	18.9	2.5	17.7	64	140	2,495	2,320	1,218							75	295	1.71	0.352	836	2,054	0.002	0.023
A-2-4	G	18.9	21.4	2.5	20.2	82	140	2,845	2,670	1,412							92	300	1.96	0.312	741	2,154	0.002	0.018
A-2-4	G	21.4	23.9	2.5	22.7	82	140	3,195	3,020	1,606							88	300	2.20	0.280	665	2,271	0.001	0.015
A-1-a	G	23.9	26.9	3.0	25.4	88	140	3,615	3,405	1,820							91	300	2.47	0.251	597	2,417	0.001	0.015
																						Total Settlement:		0.916 in

1. $\sigma_p' = \sigma_{vo}'+\sigma_m$; Estimate σ_m of 4,000 psf for moderately overconsolidated soil deposit; Ref. Table 11.2, Coduto 2003
2. $C_c = 0.009(LL-10)$; Ref. Table 6-9, FHWA GEC 5
3. $C_r = 0.10(C_c)$; Ref. Chapter 8.11, Holtz and Kovacs 1981
4. $e_o = (C_c/0.54)+0.35$; Ref. Table 6-11, FHWA GEC 5
5. $(N1)_{60} = C_n N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ 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; $I = [\beta+\sin(\beta)\cos(\beta+2\delta)]/\pi$, where $\beta = \tan^{-1}[(x+B/2)/Z_f]-\delta$, $\delta = \tan^{-1}[(x-B/2)/Z_f]$ and x = horizontal distance from center of footing; Ref. Figure 6.13 and Equation 6.24, Das 2005
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ 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)

Boring B-001-3-59

B =	9.6	ft	
L =	64	ft	
c =	0	psf	
γ =	130	pcf	
D_f =	6.0	ft	
φ =	35	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$$ = 30.48 ksf

$$N_{cm} = N_c s_c i_c$$ = 51.12

$$N_{qm} = N_q s_q d_q i_q$$ = 42.03

$$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$$ = 45.15

N_c =	46.12	s_c =	1+(9.6 ft/64 ft)(33.3/46.12) =	1.108	i_c =	1.000	d_q =	1+2tan(35°)[1-sin(35°)]²tan⁻¹(6 ft/9.6 ft) =	1.142
N_q =	33.30	s_q =	1+(9.6 ft/64 ft)tan(35°) =	1.105	i_q =	1.000	C_{wq} =	0.0 ft < 6.0 ft =	0.500
N_γ =	48.03	s_γ =	1-0.4(9.6 ft/64 ft) =	0.940	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(9.6 ft) + 6 ft =	0.500

$$q_R = q_n \cdot \phi_b$$ = 16.76 ksf

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

W-15-126 - FRA-70-14.05 Project 4B - FRA-33-1747 S. Third Street over I-70/71

Shallow Foundation Analysis - Rear Abutment - East Cap - Settlement

Calculated By: BRT Date: 6/26/2022

Checked By: JPS Date: 6/27/2022

Boring B-001-3-59

B =	10.5	ft	Effective Footing width
D_w =	0.0	ft	Depth below bottom of footing
q =	8,350	psf	Service limit bearing pressure at bottom of wall
q_{net} =	7,030	psf	Net bearing pressure at bottom of footing (considers initial overburden stress of 1,320 psf from 11-foot cut to bottom of footing elevation)

Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	N_{60}	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_m (1) (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
A-1-b	G	0.0	1.4	1.4	0.7	31	130	182	91	47							62	225	0.07	0.999	7,023	7,070	0.014	0.162
A-1-b	G	1.4	3.4	2.0	2.4	31	130	442	312	162							57	201	0.23	0.968	6,803	6,966	0.016	0.195
A-1-b	G	3.4	5.9	2.5	4.7	31	130	767	605	314							50	170	0.44	0.855	6,007	6,322	0.019	0.230
A-1-b	G	5.9	8.4	2.5	7.2	31	130	1,092	930	483							46	152	0.68	0.707	4,970	5,453	0.017	0.208
A-3a	G	8.4	10.9	2.5	9.7	27	125	1,405	1,248	646							37	104	0.92	0.584	4,109	4,755	0.021	0.250
A-3a	G	10.9	13.4	2.5	12.2	27	130	1,730	1,567	809							35	99	1.16	0.491	3,455	4,264	0.018	0.219
A-1-b	G	13.4	15.9	2.5	14.7	64	140	2,080	1,905	990							79	300	1.40	0.421	2,961	3,952	0.005	0.060
A-1-b	G	15.9	18.4	2.5	17.2	64	140	2,430	2,255	1,184							75	299	1.63	0.367	2,582	3,766	0.004	0.050
A-2-4	G	18.4	20.9	2.5	19.7	82	140	2,780	2,605	1,378							92	300	1.87	0.325	2,284	3,663	0.004	0.042
A-2-4	G	20.9	23.4	2.5	22.2	82	140	3,130	2,955	1,572							89	300	2.11	0.291	2,046	3,618	0.003	0.036
A-1-a	G	23.4	26.4	3.0	24.9	88	140	3,550	3,340	1,786							91	300	2.37	0.261	1,833	3,619	0.003	0.037
																					Total Settlement:			1.490 in

1. $\sigma_p' = \sigma_{vo}' + \sigma_m$; Estimate σ_m of 4,000 psf for moderately overconsolidated soil deposit; Ref. Table 11.2, Coduto 2003
2. $C_c = 0.009(LL-10)$; Ref. Table 6-9, FHWA GEC 5
3. $C_r = 0.10(C_c)$; Ref. Chapter 8.11, Holtz and Kovacs 1981
4. $e_o = (C_c/0.54)+0.35$; Ref. Table 6-11, FHWA GEC 5
5. $(N1)_{60} = C_N N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ 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; $I = [\beta+\sin(\beta)\cos(\beta+2\delta)]/\pi$, where $\beta = \tan^{-1}[(x+B/2)/Z_f]-\delta$, $\delta = \tan^{-1}[(x-B/2)/Z_f]$ and x = horizontal distance from center of footing; Ref. Figure 6.13 and Equation 6.24, Das 2005
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ 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)

Boring B-001-3-59

B =	10.0	ft	
L =	64	ft	
c =	0	psf	
γ =	130	pcf	
D_f =	6.0	ft	
φ =	35	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma} = 31.02 \text{ ksf}$$

$$N_{cm} = N_c s_c i_c = 51.32 \qquad N_{qm} = N_q s_q d_q i_q = 42.03 \qquad N_{\gamma m} = N_\gamma s_\gamma i_\gamma = 45.03$$

N_c =	46.12	s_c =	1+(10 ft/64 ft)(33.3/46.12) =	1.113	i_c =	1.000	d_q =	1+2tan(35°)[1-sin(35°)]²tan⁻¹(6 ft/10 ft) =	1.138
N_q =	33.30	s_q =	1+(10 ft/64 ft)tan(35°) =	1.109	i_q =	1.000	C_{wq} =	0.0 ft < 6.0 ft =	0.500
N_γ =	48.03	s_γ =	1-0.4(10 ft/64 ft) =	0.938	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(10 ft) + 6 ft =	0.500

$$q_R = q_n \cdot \phi_b = 17.06 \text{ ksf}$$

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

W-15-126 - FRA-70-14.05 Project 4B - FRA-33-1747 S. Third Street over I-70/71 Calculated By: BRT Date: 6/26/2022
Shallow Foundation Analysis - Rear Abutment - West Cap - Settlement Checked By: JPS Date: 6/27/2022

Boring B-032-3-15

B =	10.7	ft	Effective Footing width
D_w =	0.0	ft	Depth below bottom of footing
q =	8,080	psf	Service limit bearing pressure at bottom of wall
q_{net} =	6,741	psf	Net bearing pressure at bottom of footing (considers initial overburden stress of 1,339 psf from 10.3-foot cut to bottom of footing elevation)

Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	N_{60}	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_m [1] (psf)	σ_p' [1] (psf)	LL	C_c [2]	C_r [3]	e_o [4]	$(N1)_{60}$ [5]	C' [6]	Z_f/B	I [7]	$\Delta\sigma_v$ [8] (psf)	σ_{vf}' Midpoint (psf)	S_c [9,10] (ft)	S_c (in)
A-1-b	G	0.0	0.2	0.2	0.1	40	130	26	13	7							80	300	0.01	1.000	6,741	6,748	0.002	0.024
A-1-a	G	0.2	2.7	2.5	1.5	93	135	364	195	104							185	300	0.14	0.992	6,689	6,793	0.015	0.181
A-1-a	G	2.7	5.2	2.5	4.0	93	135	701	532	286							154	300	0.37	0.899	6,062	6,348	0.011	0.135
A-1-a	G	5.2	8.2	3.0	6.7	37	130	1,091	896	478							55	190	0.63	0.739	4,984	5,462	0.017	0.200
A-1-a	G	8.2	11.2	3.0	9.7	37	130	1,481	1,286	681							50	171	0.91	0.590	3,978	4,659	0.015	0.176
A-1-a	G	11.2	14.2	3.0	12.7	37	135	1,886	1,684	891							47	157	1.19	0.482	3,246	4,137	0.013	0.153
A-4a	C	14.2	16.7	2.5	15.5	61	135	2,224	2,055	1,091	4,000	5,091	21	0.099	0.010	0.533			1.44	0.409	2,758	3,848	0.009	0.106
A-4a	C	16.7	19.2	2.5	18.0	61	135	2,561	2,392	1,272	4,000	5,272	21	0.099	0.010	0.533			1.68	0.359	2,418	3,690	0.007	0.090
A-4a	C	19.2	21.7	2.5	20.5	61	135	2,899	2,730	1,454	4,000	5,454	21	0.099	0.010	0.533			1.91	0.319	2,149	3,603	0.006	0.076
A-4b	G	21.7	24.2	2.5	23.0	120	140	3,249	3,074	1,641							128	201	2.14	0.287	1,932	3,573	0.004	0.050
A-4b	G	24.2	26.7	2.5	25.5	120	140	3,599	3,424	1,835							124	195	2.38	0.260	1,753	3,589	0.004	0.045
A-1-b	G	26.7	31.7	5.0	29.2	106	140	4,299	3,949	2,126							104	300	2.73	0.228	1,538	3,665	0.004	0.047
A-1-b	G	31.7	36.7	5.0	34.2	106	140	4,999	4,649	2,514							98	300	3.20	0.196	1,321	3,836	0.003	0.037
A-1-b	G	36.7	41.7	5.0	39.2	106	140	5,699	5,349	2,902							93	300	3.66	0.172	1,157	4,060	0.002	0.029
A-1-b	G	41.7	46.7	5.0	44.2	106	140	6,399	6,049	3,290							89	300	4.13	0.153	1,029	4,319	0.002	0.024
A-1-b	G	46.7	51.7	5.0	49.2	106	140	7,099	6,749	3,678							85	300	4.60	0.137	926	4,604	0.002	0.020
A-4b	G	51.7	56.7	5.0	54.2	97	140	7,799	7,449	4,066							74	121	5.07	0.125	842	4,908	0.003	0.041
A-3a	G	56.7	60.7	4.0	58.7	29	135	8,339	8,069	4,406							21	69	5.49	0.115	778	5,184	0.004	0.049
A-3a	G	60.7	64.7	4.0	62.7	29	135	8,879	8,609	4,696							21	68	5.86	0.108	729	5,425	0.004	0.044
																					Total Settlement:			1.526 in

1. $\sigma_p' = \sigma_{vo}' + \sigma_m$; Estimate σ_m of 4,000 psf for moderately overconsolidated soil deposit; Ref. Table 11.2, Coduto 2003
2. $C_c = 0.009(LL-10)$; Ref. Table 6-9, FHWA GEC 5
3. $C_r = 0.10(C_c)$; Ref. Chapter 8.11, Holtz and Kovacs 1981
4. $e_o = (C_c/0.54)+0.35$; Ref. Table 6-11, FHWA GEC 5
5. $(N1)_{60} = C_N N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ 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; $I = [\beta+\sin(\beta)\cos(\beta+2\delta)]/\pi$, where $\beta = \tan^{-1}[(x+B/2)/Z_f]-\delta$, $\delta = \tan^{-1}[(x-B/2)/Z_f]$ and x = horizontal distance from center of footing; Ref. Figure 6.13 and Equation 6.24, Das 2005
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ 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)

Calculated By:	BRT	Date:	6/26/2022
Checked By:	JPS	Date:	6/27/2022

Boring B-032-3-15

$B =$	10.2	ft	
$L =$	62.9	ft	
$c =$	7,625	psf	
$\gamma =$	130	pcf	
$D_f =$	6.0	ft	
$\varphi =$	0	deg	
$D_w =$	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma} = 40.82 \text{ ksf}$$

$N_{cm} = N_c s_c i_c =$ 5.30

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

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

$N_c =$	5.14	$s_c =$	1+(10.2 ft/62.9 ft)(1/5.14) =	1.032	$i_c =$	1.000	$d_q =$	1+2tan(0°)[1-sin(0°)]²tan⁻¹(6 ft/10.2 ft) =	1.000
$N_q =$	1.00	$s_q =$	1+(10.2 ft/62.9 ft)tan(0°) =	1.000	$i_q =$	1.000	$C_{wq} =$	0.0 ft < 6.0 ft =	0.500
$N_\gamma =$	0.00	$s_\gamma =$	1-0.4(10.2 ft/62.9 ft) =	0.935	$i_\gamma =$	1.000	$C_{w\gamma} =$	0.0 ft < 1.5(10.2 ft) + 6 ft =	0.500

$$q_R = q_n \cdot \phi_b = 22.45 \text{ ksf}$$

$\varphi_b =$ 0.55

Calculated By: BRT Date: 6/27/2022
Checked By: JPS Date: 6/27/2022

Boring B-032-4-15

B =	8.6	ft	Effective Footing width
D_w =	0.0	ft	Depth below bottom of footing
q =	6,100	psf	Service limit bearing pressure at bottom of wall
q_{net} =	4,800	psf	Net bearing pressure at bottom of footing (considers initial overburden stress of 1,300 psf from 10.0-foot cut to bottom of footing elevation)

Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	N_{60}	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_m (1) (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	$(N1)_{60}$ (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-2-4	G	0.0	0.5	0.5	0.3	34	130	65	33	17							68	257	0.03	1.000	4,800	4,817	0.005	0.057
A-2-4	G	0.5	3.0	2.5	1.8	63	135	403	234	125							122	300	0.20	0.976	4,686	4,810	0.013	0.159
A-2-4	G	3.0	5.5	2.5	4.3	63	135	740	571	306							103	300	0.49	0.822	3,946	4,252	0.010	0.114
A-2-4	G	5.5	8.0	2.5	6.8	63	135	1,078	909	488							93	300	0.78	0.650	3,118	3,606	0.007	0.087
A-1-b	G	8.0	10.5	2.5	9.3	38	130	1,403	1,240	663							52	178	1.08	0.520	2,498	3,161	0.010	0.114
A-1-a	G	10.5	13.0	2.5	11.8	109	140	1,753	1,578	844							141	300	1.37	0.429	2,058	2,903	0.004	0.054
A-1-a	G	13.0	15.5	2.5	14.3	109	140	2,103	1,928	1,038							133	300	1.66	0.363	1,741	2,779	0.004	0.043
A-1-a	G	15.5	18.0	2.5	16.8	58	140	2,453	2,278	1,232							67	254	1.95	0.313	1,504	2,736	0.003	0.041
A-1-a	G	18.0	21.0	3.0	19.5	58	140	2,873	2,663	1,446							64	237	2.27	0.272	1,306	2,752	0.004	0.042
A-1-a	G	21.0	24.0	3.0	22.5	58	140	3,293	3,083	1,679							62	223	2.62	0.238	1,140	2,819	0.003	0.036
A-1-a	G	24.0	27.0	3.0	25.5	58	140	3,713	3,503	1,911							59	210	2.97	0.211	1,012	2,923	0.003	0.032
A-1-a	G	27.0	32.0	5.0	29.5	85	140	4,413	4,063	2,222							82	300	3.43	0.183	878	3,100	0.002	0.029
A-1-a	G	32.0	37.0	5.0	34.5	85	140	5,113	4,763	2,610							78	300	4.01	0.157	754	3,364	0.002	0.022
A-6a	C	37.0	44.0	7.0	40.5	54	140	6,093	5,603	3,075	4,000	7,075	25	0.135	0.014	0.600			4.71	0.134	644	3,719	0.005	0.059
A-2-4	G	44.0	47.0	3.0	45.5	39	140	6,513	6,303	3,463							32	104	5.29	0.120	574	4,037	0.002	0.023
A-2-4	G	47.0	52.0	5.0	49.5	104	140	7,213	6,863	3,774							82	300	5.76	0.110	528	4,302	0.001	0.011
A-4b	G	52.0	54.5	2.5	53.3	86	140	7,563	7,388	4,065							66	108	6.19	0.102	491	4,556	0.001	0.014
A-1-a	G	54.5	57.0	2.5	55.8	86	140	7,913	7,738	4,259							64	238	6.48	0.098	470	4,728	0.000	0.006
A-3a	G	57.0	65.0	8.0	61.0	120	140	9,033	8,473	4,666							86	283	7.09	0.089	429	5,095	0.001	0.013
																					Total Settlement:			0.955 in

1. $\sigma_p' = \sigma_{vo}' + \sigma_m$; Estimate σ_m of 4,000 psf for moderately overconsolidated soil deposit; Ref. Table 11.2, Coduto 2003
2. $C_c = 0.009(LL-10)$; Ref. Table 6-9, FHWA GEC 5
3. $C_r = 0.10(C_c)$; Ref. Chapter 8.11, Holtz and Kovacs 1981
4. $e_o = (C_c/0.54)+0.35$; Ref. Table 6-11, FHWA GEC 5
5. $(N1)_{60} = C_n N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ 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; $I = [\beta+\sin(\beta)\cos(\beta+2\delta)]/\pi$, where $\beta = \tan^{-1}[(x+B/2)/Z_f]-\delta$, $\delta = \tan^{-1}[(x-B/2)/Z_f]$ and x = horizontal distance from center of footing; Ref. Figure 6.13 and Equation 6.24, Das 2005
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ 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)

Boring B-032-4-15

B =	7.3	ft	
L =	184	ft	
c =	0	psf	
γ =	130	pcf	
D_f =	5.0	ft	
φ =	40	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma} = 49.88 \text{ ksf}$$

$$N_{cm} = N_c s_c i_c = 77.86 \qquad N_{qm} = N_q s_q d_q i_q = 74.87 \qquad N_{\gamma m} = N_\gamma s_\gamma i_\gamma = 107.67$$

N_c =	75.31	s_c =	1+(7.3 ft/184 ft)(64.2/75.31) =	1.034	i_c =	1.000	d_q =	1+2tan(40°)[1-sin(40°)]²tan⁻¹(5 ft/7.3 ft) =	1.129
N_q =	64.20	s_q =	1+(7.3 ft/184 ft)tan(40°) =	1.033	i_q =	1.000	C_{wq} =	0.0 ft < 5.0 ft =	0.500
N_γ =	109.41	s_γ =	1-0.4(7.3 ft/184 ft) =	0.984	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(7.3 ft) + 5 ft =	0.500

$$q_R = q_n \cdot \phi_b = 22.44 \text{ ksf}$$

φ_b = 0.45 (Per Table 10.5.5.2.2-1, AASHTO LRFD BDS)

W-15-126 - FRA-70-14.05 Project 4B - FRA-33-1747 S. Third Street over I-70/71

Shallow Foundation Analysis - Pier - West Cap - Settlement

Calculated By: BRT Date: 6/27/2022

Checked By: JPS Date: 6/27/2022

Boring B-032-4-15

B =	17.0	ft	Effective Footing width
D_w =	0.0	ft	Depth below bottom of footing
q =	5,990	psf	Service limit bearing pressure at bottom of wall
q_{net} =	4,690	psf	Net bearing pressure at bottom of footing (considers initial overburden stress of 1,300 psf from 10.0-foot cut to bottom of footing elevation)

Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	N_{60}	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_m (1) (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	$(N1)_{60}$ (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-2-4	G	0.0	0.5	0.5	0.3	34	130	65	33	17							68	257	0.01	1.000	4,690	4,707	0.005	0.057
A-2-4	G	0.5	3.0	2.5	1.8	63	135	403	234	125							122	300	0.10	0.996	4,673	4,798	0.013	0.159
A-2-4	G	3.0	5.5	2.5	4.3	63	135	740	571	306							103	300	0.25	0.959	4,500	4,806	0.010	0.120
A-2-4	G	5.5	8.0	2.5	6.8	63	135	1,078	909	488							93	300	0.40	0.883	4,140	4,628	0.008	0.098
A-1-b	G	8.0	10.5	2.5	9.3	38	130	1,403	1,240	663							52	178	0.54	0.790	3,706	4,369	0.011	0.138
A-1-a	G	10.5	13.0	2.5	11.8	109	140	1,753	1,578	844							141	300	0.69	0.701	3,288	4,132	0.006	0.069
A-1-a	G	13.0	15.5	2.5	14.3	109	140	2,103	1,928	1,038							133	300	0.84	0.622	2,919	3,958	0.005	0.058
A-1-a	G	15.5	18.0	2.5	16.8	58	140	2,453	2,278	1,232							67	254	0.99	0.556	2,607	3,839	0.005	0.058
A-1-a	G	18.0	21.0	3.0	19.5	58	140	2,873	2,663	1,446							64	237	1.15	0.495	2,321	3,767	0.005	0.063
A-1-a	G	21.0	24.0	3.0	22.5	58	140	3,293	3,083	1,679							62	223	1.32	0.440	2,066	3,744	0.005	0.056
A-1-a	G	24.0	27.0	3.0	25.5	58	140	3,713	3,503	1,911							59	210	1.50	0.396	1,856	3,768	0.004	0.051
A-1-a	G	27.0	32.0	5.0	29.5	85	140	4,413	4,063	2,222							82	300	1.74	0.348	1,632	3,854	0.004	0.048
A-1-a	G	32.0	37.0	5.0	34.5	85	140	5,113	4,763	2,610							78	300	2.03	0.302	1,415	4,024	0.003	0.038
A-6a	C	37.0	44.0	7.0	40.5	54	140	6,093	5,603	3,075	4,000	7,075	25	0.135	0.014	0.600			2.38	0.260	1,218	4,293	0.009	0.103
A-2-4	G	44.0	47.0	3.0	45.5	39	140	6,513	6,303	3,463							32	104	2.68	0.232	1,090	4,554	0.003	0.041
A-2-4	G	47.0	52.0	5.0	49.5	104	140	7,213	6,863	3,774							82	300	2.91	0.214	1,006	4,779	0.002	0.021
A-4b	G	52.0	54.5	2.5	53.3	86	140	7,563	7,388	4,065							66	108	3.13	0.200	937	5,002	0.002	0.025
A-1-a	G	54.5	57.0	2.5	55.8	86	140	7,913	7,738	4,259							64	238	3.28	0.191	897	5,155	0.001	0.010
A-3a	G	57.0	65.0	8.0	61.0	120	140	9,033	8,473	4,666							86	283	3.59	0.175	822	5,488	0.002	0.024
																					Total Settlement:			1.235 in

1. $\sigma_p' = \sigma_{vo}' + \sigma_m$; Estimate σ_m of 4,000 psf for moderately overconsolidated soil deposit; Ref. Table 11.2, Coduto 2003
2. $C_c = 0.009(LL-10)$; Ref. Table 6-9, FHWA GEC 5
3. $C_r = 0.10(C_c)$; Ref. Chapter 8.11, Holtz and Kovacs 1981
4. $e_o = (C_c/0.54)+0.35$; Ref. Table 6-11, FHWA GEC 5
5. $(N1)_{60} = C_N N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \le 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing; $I = [\beta+\sin(\beta)\cos(\beta+2\delta)]/\pi$, where $\beta = \tan^{-1}[(x+B/2)/Z_f]-\delta$, $\delta = \tan^{-1}[(x-B/2)/Z_f]$ and x = horizontal distance from center of footing; Ref. Figure 6.13 and Equation 6.24, Das 2005
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \le \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \le \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (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)

Boring B-032-4-15

B =	17.0	ft	
L =	184	ft	
c =	0	psf	
γ =	130	pcf	
D_f =	5.0	ft	
φ =	40	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$$ = 82.07 ksf

$$N_{cm} = N_c s_c i_c$$ = 81.24

$$N_{qm} = N_q s_q d_q i_q$$ = 73.41

$$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$$ = 105.37

N_c =	75.31	s_c =	1+(17 ft/184 ft)(64.2/75.31) =	1.079	i_c =	1.000	d_q =	1+2tan(40°)[1-sin(40°)]²tan⁻¹(5 ft/17 ft) =	1.061
N_q =	64.20	s_q =	1+(17 ft/184 ft)tan(40°) =	1.078	i_q =	1.000	C_{wq} =	0.0 ft < 5.0 ft =	0.500
N_γ =	109.41	s_γ =	1-0.4(17 ft/184 ft) =	0.963	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(17 ft) + 5 ft =	0.500

$$q_R = q_n \cdot \phi_b$$ = 36.93 ksf

φ_b = 0.45 (Per Table 10.5.5.2.2-1, AASHTO LRFD BDS)

W-15-126 - FRA-70-14.05 Project 4B - FRA-33-1747 S. Third Street over I-70/71
Shallow Foundation Analysis - Pier - Bridge - Settlement

Calculated By: BRT Date: 6/27/2022
Checked By: JPS Date: 6/27/2022

Boring B-032-5-15

B =	8.6	ft	Effective Footing width
D_w =	0.0	ft	Depth below bottom of footing
q =	6,100	psf	Service limit bearing pressure at bottom of wall
q_{net} =	4,865	psf	Net bearing pressure at bottom of footing (considers initial overburden stress of 1,235 psf from 9.5-foot cut to bottom of footing elevation)

Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	N_{60}	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_m (1) (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	$(N1)_{60}$ (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	1.7	1.7	0.9	35	130	221	111	57							70	268	0.10	0.997	4,850	4,907	0.012	0.147
A-1-a	G	1.7	3.4	1.7	2.6	35	130	442	332	172							64	234	0.30	0.939	4,566	4,738	0.010	0.125
A-1-b	G	3.4	6.4	3.0	4.9	39	130	832	637	331							63	228	0.57	0.774	3,766	4,097	0.014	0.173
A-1-b	G	6.4	9.9	3.5	8.2	39	130	1,287	1,060	551							56	195	0.95	0.572	2,782	3,333	0.014	0.168
A-4a	C	9.9	12.4	2.5	11.2	103	135	1,625	1,456	760	4,000	4,760	25	0.135	0.014	0.600			1.30	0.448	2,180	2,940	0.012	0.149
A-4a	C	12.4	15.4	3.0	13.9	103	135	2,030	1,827	960	4,000	4,960	25	0.135	0.014	0.600			1.62	0.371	1,804	2,763	0.012	0.140
A-4a	C	15.4	18.4	3.0	16.9	103	135	2,435	2,232	1,177	4,000	5,177	25	0.135	0.014	0.600			1.97	0.311	1,512	2,689	0.009	0.109
A-3a	G	18.4	20.4	2.0	19.4	110	140	2,715	2,575	1,364							124	300	2.26	0.273	1,330	2,694	0.002	0.024
A-3a	G	20.4	22.4	2.0	21.4	110	140	2,995	2,855	1,519							120	300	2.49	0.249	1,212	2,731	0.002	0.020
A-4a	G	22.4	24.9	2.5	23.7	135	140	3,345	3,170	1,694							143	223	2.75	0.227	1,102	2,796	0.002	0.029
A-4a	G	24.9	27.4	2.5	26.2	135	140	3,695	3,520	1,888							138	216	3.04	0.206	1,001	2,888	0.002	0.026
A-1-a	G	27.4	32.4	5.0	29.9	120	140	4,395	4,045	2,179							117	300	3.48	0.181	879	3,058	0.002	0.029
A-1-a	G	32.4	37.4	5.0	34.9	120	140	5,095	4,745	2,567							110	300	4.06	0.155	756	3,322	0.002	0.022
A-4a	G	37.4	42.4	5.0	39.9	120	140	5,795	5,445	2,955							105	166	4.64	0.136	662	3,617	0.003	0.032
A-4a	G	42.4	47.4	5.0	44.9	120	140	6,495	6,145	3,343							100	159	5.22	0.121	590	3,932	0.002	0.027
A-1-b	G	47.4	52.4	5.0	49.9	109	140	7,195	6,845	3,731							86	300	5.80	0.109	531	4,262	0.001	0.012
A-1-b	G	52.4	57.4	5.0	54.9	109	140	7,895	7,545	4,119							83	300	6.38	0.099	483	4,602	0.001	0.010
A-1-b	G	57.4	62.4	5.0	59.9	109	140	8,595	8,245	4,507							80	300	6.97	0.091	443	4,950	0.001	0.008
A-3a	G	62.4	65.4	3.0	63.9	102	140	9,015	8,805	4,817							72	220	7.43	0.085	416	5,233	0.000	0.006

Total Settlement: 1.255 in

1. $\sigma_p' = \sigma_{vo}' + \sigma_m$; Estimate σ_m of 4,000 psf for moderately overconsolidated soil deposit; Ref. Table 11.2, Coduto 2003
2. $C_c = 0.009(LL-10)$; Ref. Table 6-9, FHWA GEC 5
3. $C_r = 0.10(C_c)$; Ref. Chapter 8.11, Holtz and Kovacs 1981
4. $e_o = (C_c/0.54)+0.35$; Ref. Table 6-11, FHWA GEC 5
5. $(N1)_{60} = C_N N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ 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; $I = [\beta+\sin(\beta)\cos(\beta+2\delta)]/\pi$, where $\beta = \tan^{-1}[(x+B/2)/Z_f]-\delta$, $\delta = \tan^{-1}[(x-B/2)/Z_f]$ and x = horizontal distance from center of footing; Ref. Figure 6.13 and Equation 6.24, Das 2005
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ 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)

Calculated By:	BRT	Date:	6/27/2022
Checked By:	JPS	Date:	6/27/2022

Boring B-032-5-15

B =	7.3	ft	
L =	186	ft	
c =	8,000	psf	
γ =	130	pcf	
D_f =	5.0	ft	
φ =	0	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma} = 41.76 \text{ ksf}$$

$$N_{cm} = N_c s_c i_c = 5.18 \qquad N_{qm} = N_q s_q d_q i_q = 1.00 \qquad N_{\gamma m} = N_\gamma s_\gamma i_\gamma = 0.00$$

N_c =	5.14	s_c =	1+(7.3 ft/186 ft)(1/5.14) =	1.008	i_c =	1.000	d_q =	1+2tan(0°)[1-sin(0°)]²tan⁻¹(5 ft/7.3 ft) =	1.000
N_q =	1.00	s_q =	1+(7.3 ft/186 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(7.3 ft/186 ft) =	0.984	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(7.3 ft) + 5 ft =	0.500

$$q_R = q_n \cdot \phi_b = 20.88 \text{ ksf}$$

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

W-15-126 - FRA-70-14.05 Project 4B - FRA-33-1747 S. Third Street over I-70/71 Calculated By: BRT Date: 6/27/2022

Shallow Foundation Analysis - Pier - East Cap - Settlement Checked By: JPS Date: 6/27/2022

Boring B-032-5-15

B =	17.0	ft	Effective Footing width
D_w =	0.0	ft	Depth below bottom of footing
q =	5,990	psf	Service limit bearing pressure at bottom of wall
q_{net} =	4,755	psf	Net bearing pressure at bottom of footing (considers initial overburden stress of 1,235 psf from 9.5-foot cut to bottom of footing elevation)

Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	N_{60}	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_m (1) (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	$(N1)_{60}$ (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	1.7	1.7	0.9	35	130	221	111	57							70	268	0.05	1.000	4,753	4,810	0.012	0.146
A-1-a	G	1.7	3.4	1.7	2.6	35	130	442	332	172							64	234	0.15	0.990	4,706	4,878	0.011	0.127
A-1-b	G	3.4	6.4	3.0	4.9	39	130	832	637	331							63	228	0.29	0.943	4,482	4,813	0.015	0.184
A-1-b	G	6.4	9.9	3.5	8.2	39	130	1,287	1,060	551							56	195	0.48	0.831	3,953	4,504	0.016	0.196
A-4a	C	9.9	12.4	2.5	11.2	103	135	1,625	1,456	760	4,000	4,760	25	0.135	0.014	0.600			0.66	0.722	3,431	4,191	0.016	0.188
A-4a	C	12.4	15.4	3.0	13.9	103	135	2,030	1,827	960	4,000	4,960	25	0.135	0.014	0.600			0.82	0.633	3,009	3,968	0.016	0.187
A-4a	C	15.4	18.4	3.0	16.9	103	135	2,435	2,232	1,177	4,000	5,177	25	0.135	0.014	0.600			0.99	0.552	2,626	3,803	0.013	0.155
A-3a	G	18.4	20.4	2.0	19.4	110	140	2,715	2,575	1,364							124	300	1.14	0.497	2,363	3,727	0.003	0.035
A-3a	G	20.4	22.4	2.0	21.4	110	140	2,995	2,855	1,519							120	300	1.26	0.459	2,183	3,702	0.003	0.031
A-4a	G	22.4	24.9	2.5	23.7	135	140	3,345	3,170	1,694							143	223	1.39	0.422	2,008	3,702	0.004	0.046
A-4a	G	24.9	27.4	2.5	26.2	135	140	3,695	3,520	1,888							138	216	1.54	0.387	1,841	3,729	0.003	0.041
A-1-a	G	27.4	32.4	5.0	29.9	120	140	4,395	4,045	2,179							117	300	1.76	0.344	1,635	3,813	0.004	0.049
A-1-a	G	32.4	37.4	5.0	34.9	120	140	5,095	4,745	2,567							110	300	2.05	0.298	1,419	3,986	0.003	0.038
A-4a	G	37.4	42.4	5.0	39.9	120	140	5,795	5,445	2,955							105	166	2.35	0.263	1,252	4,207	0.005	0.055
A-4a	G	42.4	47.4	5.0	44.9	120	140	6,495	6,145	3,343							100	159	2.64	0.235	1,120	4,462	0.004	0.047
A-1-b	G	47.4	52.4	5.0	49.9	109	140	7,195	6,845	3,731							86	300	2.94	0.213	1,012	4,743	0.002	0.021
A-1-b	G	52.4	57.4	5.0	54.9	109	140	7,895	7,545	4,119							83	300	3.23	0.194	923	5,041	0.001	0.018
A-1-b	G	57.4	62.4	5.0	59.9	109	140	8,595	8,245	4,507							80	300	3.52	0.178	848	5,355	0.001	0.015
A-3a	G	62.4	65.4	3.0	63.9	102	140	9,015	8,805	4,817							72	220	3.76	0.167	796	5,613	0.001	0.011
																						Total Settlement:		1.589 in

1. $\sigma_p' = \sigma_{vo}' + \sigma_m$; Estimate σ_m of 4,000 psf for moderately overconsolidated soil deposit; Ref. Table 11.2, Coduto 2003
2. $C_c = 0.009(LL-10)$; Ref. Table 6-9, FHWA GEC 5
3. $C_r = 0.10(C_c)$; Ref. Chapter 8.11, Holtz and Kovacs 1981
4. $e_o = (C_c/0.54)+0.35$; Ref. Table 6-11, FHWA GEC 5
5. $(N1)_{60} = C_N N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ 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; $I = [\beta+\sin(\beta)\cos(\beta+2\delta)]/\pi$, where $\beta = \tan^{-1}[(x+B/2)/Z_f]-\delta$, $\delta = \tan^{-1}[(x-B/2)/Z_f]$ and x = horizontal distance from center of footing; Ref. Figure 6.13 and Equation 6.24, Das 2005
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ 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)

W-15-126 - FRA-70-14.05 Project 4B - FRA-33-1747 S. Third Street over I-70/71
Shallow Foundation Analysis - Strength Limit State - Pier - East Cap

Calculated By: BRT Date: 6/27/2022
Checked By: JPS Date: 6/27/2022

Boring B-032-5-15

B =	17.0	ft	
L =	186	ft	
c =	8,000	psf	
γ =	130	pcf	
D_f =	5.0	ft	
φ =	0	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$$ = 42.18 ksf

$$N_{cm} = N_c s_c i_c$$ = 5.23

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

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

N_c =	5.14	s_c =	1+(17 ft/186 ft)(1/5.14) =	1.018	i_c =	1.000	d_q =	1+2tan(0°)[1-sin(0°)]²tan⁻¹(5 ft/17 ft) =	1.000
N_q =	1.00	s_q =	1+(17 ft/186 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 ft/186 ft) =	0.963	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(17 ft) + 5 ft =	0.500

$$q_R = q_n \cdot \phi_b$$ = 21.09 ksf

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

APPENDIX VII

DRILLED SHAFT CALCULATIONS

FRA-33-1747 Bridge Replacement
Drilled Shaft Resistance Calculations
Forward Abutment

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)	N_{60} [2]	γ (pcf)	σ_v' (Midpoint) (psf)	σ_v (Bottom) (psf)	S_u [3] (psf)	N_c [4]	α [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-010-3-59	725.0	0.0	5.0	A-3	G	1.6	1.6	723.4	42	130	54	208				42	38	9,384	7.18	B-010-3-59	725.0-723.4	0.0-1.6	50	0.38	0.50	0.55
				A-3a	G	10.1	8.5	714.9	46	130	395	1,313				43	39	9,910	2.28		723.4-714.9	1.6-10.1	55	0.90	0.50	0.55
				A-3	G	15.1	5.0	709.9	78	140	877	2,013				67	39	13,605	1.69		714.9-709.9	10.1-15.1	60	1.47	0.50	0.55
				A-3a	G	28.1	13.0	696.9	73	140	1,575	3,833				58	40	13,075	1.17		709.9-696.9	15.1-28.1	60	1.84	0.50	0.55
B-032-6-15	723.0	0.0	5.0	A-1-a	G	10.5	10.5	712.5	60	135	381	1,418				45	42	19,080	4.09	B-032-6-15	723.0-712.5	0.0-10.5	60	1.55	0.50	0.55
				A-1-b	G	15.5	5.0	707.5	37	135	944	2,093				25	38	11,766	1.42		712.5-707.5	10.5-15.5	44	1.33	0.50	0.55
				A-1-a	G	22.0	6.5	701.0	39	135	1,361	2,970				26	39	12,402	1.21		707.5-701.0	15.5-22.0	46	1.64	0.50	0.55
				A-1-b	G	27.0	5.0	696.0	30	135	1,779	3,645				19	37	9,540	0.82		701.0-696.0	22.0-27.0	36	1.46	0.50	0.55
				A-1-a	G	37.0	10.0	686.0	58	140	2,348	5,045				35	41	18,444	1.16		696.0-686.0	27.0-37.0	60	2.71	0.50	0.55
				A-1-a	G	50.0	13.0	673.0	111	140	3,241	6,865				62	43	35,298	1.51		686.0-673.0	37.0-50.0	60	4.89	0.50	0.55

1. C = cohesive soil stratum; G = granular soil stratum
2. N_{60} = average energy corrected N-values over stratum thickness
3. $S_u = 125(N_{60})$ (cohesive soil layers)
4. $N_C = 6[1+0.2(Z/D)] \leq 9$; Ref. Section 10.8.3.5.1c, AASHTO LRFD BDS (cohesive soil layers)
5. $\alpha = 0.55$ 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)
6. $(N_1)_{60} = C_n N_{60}$, where $C_N = [0.77\log(40/\sigma_v')] \leq 2.0$ ksf, where σ_v' = vetical effective stress at midpoint of soil layer with respect to the entire soil profile for the respective boring; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS (granular soil layers)
7. φ'_f estimated per Table 10.4.6.2.4-1; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS (granular soil layers)
8. $\sigma_p' = n(N_{60})^m(P_a)$, where n = 0.15 and m = 1.0 for A-1-a/1-b and A-2-4/2-6, n = 0.47 and m = 0.6 for A-3/3a, n = 0.47 and m = 0.8 for A-4a/4b soils, and P_a = 2.12 ksf = 2,120 psf; Ref. Section 10.8.3.5.2b, AASHTO LRFD BDS (granular soil layers)
9. $\beta = \tan\varphi'_f(1-\sin\varphi'_f)(\sigma_p'/\sigma_v')$^$(\sin\varphi'_f)$, where σ_v' = vetical effective stress at midpoint of soil layer; Ref. Section 10.8.3.5.2b, AASHTO LRFD BDS (granular soil layers)
10. $q_p = N_C S_u \leq 80.0$ ksf; Ref. Section 10.8.3.5.1c, AASHTO LRFD BDS (cohesive soil layers)
11. $q_p = 1.2N_{60} \leq 60$ ksf; Ref. Section 10.8.3.5.2c, AASHTO LRFD BDS (granular soil layers)
12. $q_s = \alpha S_u$; Ref. Section 10.8.3.5.1b, AASHTO LRFD BDS (cohesive soil layers)
13. $q_s = \beta\sigma_v'$, where σ_v' = vetical effective stress at midpoint of soil layer; Ref. Section 10.8.3.5.2b, AASHTO LRFD BDS (granular soil layers)
14. φ_{qp} = 0.50 for granular soils layers and 0.40 for cohesive soil layers; Ref. Table 10.5.5.2.4-1, AASHTO LRFD BDS
15. φ_{qs} = 0.55 for granular soils layers and 0.45 for cohesive soil layers; Ref. Table 10.5.5.2.4-1, AASHTO LRFD BDS

FRA-33-1747 Bridge Replacement
Drilled Shaft Resistance
Forward Abutment

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)	N_{60} [2]	γ (pcf)	σ_v' (Midpoint) (psf)	σ_v (Bottom) (psf)	S_u [3] (psf)	N_c [4]	α [5]	$(N_1)_{60}$ [6]	φ'_f [7]	σ_p' [8] (psf)	β [9]
B-032-6-15	723.0	0.0	5.0	A-1-a	G	10.5	10.5	712.5	60	135	381	1,418				45	42	19,080	4.09
				A-1-b	G	15.5	5.0	707.5	37	135	944	2,093				25	38	11,766	1.42
				A-1-a	G	22.0	6.5	701.0	39	135	1,361	2,970				26	39	12,402	1.21
				A-1-b	G	27.0	5.0	696.0	30	135	1,779	3,645				19	37	9,540	0.82
				A-1-a	G	37.0	10.0	686.0	58	140	2,348	5,045				35	41	18,444	1.16
				A-1-a	G	50.0	13.0	673.0	111	140	3,241	6,865				62	43	35,298	1.51

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-032-6-15	723.0-712.5	0.0-10.5	60	1.55	0.50	0.55
	712.5-707.5	10.5-15.5	44	1.33	0.50	0.55
	707.5-701.0	15.5-22.0	46	1.64	0.50	0.55
	701.0-696.0	22.0-27.0	36	1.46	0.50	0.55
	696.0-686.0	27.0-37.0	60	2.71	0.50	0.55
	686.0-673.0	37.0-50.0	60	4.89	0.50	0.55

Shaft Length (ft)	Shaft Tip Elevation (ft msl)	Nominal Tip Resistance, R_p (kips)	Nominal Side Resistance, R_s (kips)	Total Nominal Resistance, R_n (kips)	Factored Tip Resistance, $\varphi_{qp}R_p$ (kips)	Factored Side Resistance, $\varphi_{qs}R_s$ (kips)	Total Factored Resistance, R_R (kips)
44.7	678.3			1,060			530
		1,060			530		

Group Efficiency Factor, η = 0.9

1. C = cohesive soil stratum; G = granular soil stratum
2. N_{60} = average energy corrected N-values over stratum thickness
3. $S_u = 125(N_{60})$ (cohesive soil layers)
4. $N_C = 6[1+0.2(Z/D)] \le 9$; Ref. Section 10.8.3.5.1c, AASHTO LRFD BDS (cohesive soil layers)
5. $\alpha = 0.55$ 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)
6. $(N_1)_{60} = C_nN_{60}$, 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. φ'_f estimated per Table 10.4.6.2.4-1; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS (granular soil layers)
8. $\sigma_p' = n(N_{60})^m(P_a)$, where n = 0.15 and m = 1.0 for A-1-a/1-b and A-2-4/2-6, n = 0.47 and m = 0.6 for A-3/3a, n = 0.47 and m = 0.8 for A-4a/4b soils, and P_a = 2.12 ksf = 2,120 psf; Ref. Section 10.8.3.5.2b, AASHTO LRFD BDS (granular soil layers)
9. $\beta = \tan\varphi'_f(1-\sin\varphi'_f)(\sigma_p'/\sigma_v')$^$(\sin\varphi'_f)$, where σ_v' = vetical effective stress at midpoint of soil layer; Ref. Section 10.8.3.5.2b, AASHTO LRFD BDS (granular soil layers)
10. $q_p = N_CS_u \le 80.0$ 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

W-15-126 - FRA-70-14.05 Project 4B - FRA-33-1747 S. Third Street over I-70/71
Tangent Shafts - Block Failure Mode - Forward Abutment

Calculated By: BRT Date: 6/27/2022
Checked By: JPS Date: 6/27/2022

Boring B-032-6-15

D =	5.0	ft	Diameter of individual drilled shafts
B' =	3.9	ft	Equivalent footing width based on overall end bearing area of drilled shafts
L =	190.0	ft	38 drilled shafts @ 5.0 ft diameter each bridge and adjacent wingwalls
c =	0	psf	
γ =	140	pcf	
D_f =	44.7	ft	
φ =	43	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$$ = 429.34 ksf

$$N_{cm} = N_c s_c i_c$$ = 107.14

$$N_{qm} = N_q s_q d_q i_q$$ = 129.14

$$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$$ = 185.00

N_c =	105.11	s_c =	1+(3.9 ft/190 ft)(99.01/105.11) =	1.019	i_c =	1.000	d_q =	1+2tan(43°)[1-sin(43°)]²tan⁻¹(44.7 ft/3.9 ft) =	1.280
N_q =	99.01	s_q =	1+(3.9 ft/190 ft)tan(43°) =	1.019	i_q =	1.000	C_{wq} =	0.0 ft < 44.7 ft =	0.500
N_γ =	186.53	s_γ =	1-0.4(3.9 ft/190 ft) =	0.992	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(3.9 ft) + 44.7 ft =	0.500

$$q_R = q_n \cdot \phi_b$$ = 214.67 ksf

φ_b = 0.5

$$R_R = q_R \cdot A_p$$ = 4,215 kips

8430.056

APPENDIX VIII

LATERAL DESIGN PARAMETERS

FRA-33-1747 Bridge Replacement
Lateral Design Parameters
Forward Abutment

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-001-3-59	750.6 to 736.1	A-2-6	G	4	11	14	125	125	φ = 35°	135 pci	-	-
	736.1 to 732.6	A-1-b	G	4	10	10	125	125	φ = 34°	115 pci	-	-
	732.6 to 726.1	A-2-4	G	4	27	24	135	135	φ = 37°	190 pci	-	-
	726.1 to 723.6	A-1-a	G	4	23	19	135	135	φ = 38°	215 pci	-	-
	723.6 to 712.6	A-1-b	G	4	31	24	140	78	φ = 38°	125 pci	-	-
	712.6 to 707.6	A-3a	G	4	27	19	135	73	φ = 35°	85 pci	-	-
	707.6 to 702.6	A-1-b	G	4	64	44	140	78	φ = 41°	175 pci	-	-
	702.6 to 697.6	A-2-4	G	4	82	55	140	78	φ = 41°	175 pci	-	-
	697.6 to 694.6	A-1-a	G	4	88	57	140	78	φ = 43°	215 pci	-	-
B-010-3-59	752.9 to 745.4	A-4a	C	3	23	23	125	125	Su = 2,875 psf	960 pci	0.0052	-
	745.4 to 741.9	A-1-a	G	4	120	142	135	135	φ = 43°	395 pci	-	-
	741.9 to 738.9	A-2-4	G	4	58	75	140	140	φ = 41°	315 pci	-	-
	738.9 to 725.9	A-1-b	G	4	86	95	140	140	φ = 42°	355 pci	-	-
	725.9 to 723.4	A-3	G	4	42	42	140	140	φ = 38°	215 pci	-	-
	723.4 to 714.9	A-3a	G	4	46	43	140	78	φ = 39°	140 pci	-	-
	714.9 to 709.9	A-3	G	4	78	67	140	78	φ = 39°	140 pci	-	-
	709.9 to 696.9	A-3a	G	4	73	58	140	78	φ = 40°	155 pci	-	-
B-032-3-15	732.8 to 722.3	A-1-b	G	4	40	54	130	130	φ = 42°	355 pci	-	-
	722.3 to 717.3	A-1-a	G	4	93	100	140	78	φ = 43°	215 pci	-	-
	717.3 to 708.3	A-1-a	G	4	37	36	135	73	φ = 41°	175 pci	-	-
	708.3 to 700.8	A-4a	C	2	61	61	140	78	Su = 7,625 psf	2,540 pci	0.0035	-
	700.8 to 695.8	A-4b	G	4	120	101	140	78	φ = 38°	125 pci	-	-
	695.8 to 670.8	A-1-b	G	4	106	78	140	78	φ = 42°	195 pci	-	-
	670.8 to 665.8	A-4b	G	4	97	64	140	78	φ = 38°	125 pci	-	-
	665.8 to 657.8	A-3a	G	4	29	18	135	73	φ = 35°	85 pci	-	-
B-032-4-15	732.5 to 722.0	A-2-4	G	4	34	46	130	130	φ = 40°	280 pci	-	-
	722.0 to 714.5	A-2-4	G	4	63	66	140	78	φ = 41°	175 pci	-	-
	714.5 to 712.0	A-1-b	G	4	38	37	135	73	φ = 40°	155 pci	-	-
	712.0 to 707.0	A-1-a	G	4	109	103	140	78	φ = 43°	215 pci	-	-
	707.0 to 695.5	A-1-a	G	4	58	50	140	78	φ = 43°	215 pci	-	-
	695.5 to 685.5	A-1-a	G	4	85	66	140	78	φ = 43°	215 pci	-	-
	685.5 to 678.5	A-6a	C	2	54	54	140	78	Su = 6,750 psf	2,250 pci	0.0038	-
	678.5 to 675.5	A-2-4	G	4	39	27	140	78	φ = 38°	125 pci	-	-
	675.5 to 670.5	A-2-4	G	4	104	71	140	78	φ = 41°	175 pci	-	-
	670.5 to 668.0	A-4b	G	4	86	57	140	78	φ = 38°	125 pci	-	-
	668.0 to 665.5	A-1-a	G	4	86	56	140	78	φ = 43°	215 pci	-	-
	665.5 to 657.5	A-3a	G	4	120	75	140	78	φ = 40°	155 pci	-	-
B-032-5-15	731.6 to 718.6	A-1-a	G	4	35	45	130	130	φ = 42°	355 pci	-	-
	718.6 to 712.1	A-1-b	G	4	39	41	135	73	φ = 41°	175 pci	-	-
	712.1 to 703.6	A-4a	C	2	103	103	140	78	Su = 8,000 psf	2,665 pci	0.0033	-
	703.6 to 699.6	A-3a	G	4	110	98	140	78	φ = 40°	155 pci	-	-
	699.6 to 694.6	A-4a	G	4	120	102	140	78	φ = 38°	125 pci	-	-
	694.6 to 684.6	A-1-a	G	4	120	95	140	78	φ = 43°	215 pci	-	-
	684.6 to 674.6	A-4a	G	4	120	87	140	78	φ = 38°	125 pci	-	-
	674.6 to 659.6	A-1-b	G	4	109	72	140	78	φ = 42°	195 pci	-	-
	659.6 to 656.6	A-3a	G	4	102	64	140	78	φ = 40°	155 pci	-	-
B-032-6-15	753.0 to 746.0	A-6a	C	3	16	16	125	125	Su = 2,000 psf	665 pci	0.0063	-
	746.0 to 736.0	A-1-a	G	4	51	55	140	140	φ = 43°	395 pci	-	-
	736.0 to 730.0	A-1-a	G	4	37	33	135	135	φ = 41°	315 pci	-	-
	730.0 to 712.5	A-1-a	G	4	60	45	140	78	φ = 42°	195 pci	-	-
	712.5 to 707.5	A-1-b	G	4	37	25	140	78	φ = 38°	125 pci	-	-
	707.5 to 701.0	A-1-a	G	4	39	26	140	78	φ = 39°	140 pci	-	-
	701.0 to 696.0	A-1-b	G	4	30	19	135	73	φ = 37°	110 pci	-	-
	696.0 to 686.0	A-1-a	G	4	58	35	140	78	φ = 41°	175 pci	-	-
	686.0 to 673.0	A-1-a	G	4	111	62	140	78	φ = 43°	215 pci	-	-