Resource International, Inc.

FRA-70-14.05 PROJECT 4B
FRA-23-1075
S. FOURTH STREET (US-23) 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

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Re: Structure Foundation Exploration Report
FRA-70-14.05 Project 4B
FRA-23-1075 – S. Fourth Street (US-23) 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-23-1075 bridge structure carrying S. Fourth Street (US-23) 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 Trenner

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

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

Enclosure: Structure Foundation Exploration Report

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EXECUTIVE SUMMARY

Resource International, Inc. (Rii) has completed a structure foundation exploration for the design and construction of the proposed FRA-23-1075 bridge structure carrying S. Fourth Street (US-23) over I-70/71. The existing structure is a three-span bridge with a total length of approximately 210 feet and width of 60 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 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 very dense gravel, gravel and sand and coarse and fine sand (ODOT A-1-a, A-1-b, A-3a). Shallow spread foundations bearing on these competent natural soils may be proportioned for a nominal bearing resistance as presented in Table 7 for the rear abutment and Table 8 for the pier substructure in Section 5.1 of the full report.

Based on the service limit bearing pressures provided, total settlements of 0.81 to 1.10 inches are anticipated at the rear abutment, and 0.21 to 0.38 inches are anticipated at the pier. Differential settlement along the and pier substructures is anticipated to be less than 1 in. / 500 ft. Additionally, the maximum factored bearing pressure of 8.46 ksf at the rear abutment and 2.60 ksf at the pier substructure units will not exceed the factored bearing resistance at the strength limit of 12.66 and 16.47 ksf, respectively.

For concrete footing that rest on cohesionless soil, a coefficient “f” of 0.84 times the total vertical force on the base should be taken as the sliding resistance. A geotechnical resistance factor of $\varphi_\tau = 1.0$ 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 9 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, fine sand, sandy silt and silt (ODOT A-1-a, A-1-b, A-2-4, A-3, A-4a, A-4b) with seams of hard sandy silt and silt and clay (ODOT A-4a, A-6a). 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 project represents 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-23-1075 bridge structure carrying S. Fourth Street (US-23) over I-70/71, as shown on the vicinity map and boring plan presented in Appendix I. The existing structure is a three-span bridge with a total length of approximately 210 feet and width of 60 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 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

As stated, the proposed FRA-23-1075 structure is located at the existing S. Fourth Street (US-23) over I-70/71 overpass, approximately 1.0 mile east of the Scioto River. The existing I-70/I-71 in the vicinity of the structure is a six-lane, bi-directional, composite asphalt and concrete paved roadway that is generally east-west aligned through downtown Columbus, Ohio. The existing entrance ramp from S. Third Street to I-70 eastbound crosses under the southern end of the central span. The existing S. Fourth Street crossing is a three-lane, one-way, asphalt paved roadway with sidewalks aligning both sides of the road. The existing I-70 profile is lowered from the surrounding terrain, as the existing corridor was cut approximately 25 feet below the existing grade of S. Fourth Street and the surrounding downtown area. An existing cast-in-place concrete wall type abutment is present at the existing forward abutment, and a graded embankment with a concrete spill through slope is utilized in front of the existing rear abutment. The existing structure appears to be in poor condition, with concrete spalling and delamination evident on the columns with exposed corroded reinforcing steel bars, and significant corrosion of the superstructure fascia steel beams. Graded slopes extend to the east and west of the rear abutment and west of the forward abutment, 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. Construction is currently under way on the north side of I-70/71 for the adjacent FRA-70-14.48 project to the east of S. Fourth Street, which includes the replacement of the Grant Avenue bridge structure over I-71/71. The traffic volume along the project alignment is very high, and the alignment traverses primarily commercial and government properties. The regional topography generally slopes downward to the west and south toward the Scioto River.

3.0 EXPLORATION

On October 7, 2015, one (1) structure boring, designated as B-033-3-15, was advanced to a completion depth of 69.0 feet below the existing ground surface. In addition to the boring performed by Rii as part of the current exploration, two (2) borings, designated as B-034-0-08 and B-035-0-08, were performed DLZ in the vicinity of the bridge structure as part of the FRA-70-8.93 preliminary exploration (PID 77369), and their findings were published in a report dated September 24, 2009. The borings were performed between July 7 and August 14, 2008, and were advanced to a completion depth of 135.5 and 15.0 feet below the existing ground surface, respectively. The current project boring locations are shown on the boring plan provided in Appendix I of this report and summarized in Table 1 below.

Table 1. Test Boring Summary

Boring Number	Reference Alignment	Station	Offset	Latitude	Longitude	Ground Elevation (feet msl)	Boring Depth (feet)
B-033-3-15	BL I-70 EB	203+36.15	47.7' Rt.	39.953302	-82.994869	731.2 [1]	69.0
B-034-0-08	BL I-70 WB	203+85.26	56.6' Lt.	39.953825	-82.994806	751.5	135.5
B-035-0-08	BL I-70 EB	203+82.19	32.7' Rt.	39.953365	-82.994716	732.3	15.0

1. *Ground surface elevations at the current exploration boring location was interpolated using topographic mapping information provided by GPD GROUP.*

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

The boring performed by Rii for the current exploration was drilled using a truck mounted rotary drilling machine, utilizing a 3.25-inch inside diameter hollow-stem auger to advance the hole. The borings performed by DLZ were drilled using a truck or an all-terrain vehicle (ATV) mounted rotary drilling machine, utilizing either a 3.25-inch inside diameter hollow stem auger or a 4.0-inch flush joint casing to advance the holes.

Standard penetration test (SPT) and split spoon sampling were performed in boring B-033-3-15 and B-034-0-08 at 2.5-foot intervals of depth to 20.0 and 45.0 feet, respectively, and at 5.0-foot intervals thereafter to the boring termination depths. Boring B-035-0-08 was sampled continuously below the pavement section and at 2.5-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. A calibrated automatic drop hammer was utilized by Rii and DLZ to generate consistent energy transfer to the sampler. Driving resistance is recorded on the boring logs in terms of 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 mounted drill rig operated by Rii was calibrated on October 20, 2014, and has a drill rod energy ratio of 92.0 percent. The hammer for the CME 750X drill rig operated by DLZ was calibrated on February 15, 2008, and has a drill rod energy ratio 63.1 percent. The hammer for the CME 75 drill rig operated by DLZ was calibrated on February 11, 2009, and has a drill rod energy ratio 62.1 percent.

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

During drilling for the borings performed by Rii, field logs were prepared by Rii personnel showing the encountered subsurface conditions. Soil samples obtained from the drilling operation were preserved and sealed in glass jars and delivered to the soil laboratory. In the laboratory, the soil samples obtained by Rii 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	19
Plastic and Liquid Limits	AASHTO T89, T90	7
Gradation – Sieve/Hydrometer	AASHTO T88	7
Sulfate Content – Colorimetric Method	TEX-145-E	1

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.

Where boring B-034-0-08 was extended into the underlying bedrock by DLZ, an NMX or NQ-sized double-tube diamond bit core barrel (utilizing wire line equipment) was used to core the bedrock. The Rock Quality Designation (RQD) for each rock core run was provided on the boring log.

In addition to the borings performed as part of the preliminary and current explorations, historic borings performed in 1959 by the Department of Highways as part of the original FRA-40-12.82 project for the existing structure were also obtained from the construction documents on record. Three (3) borings, designated as B-001-4-59, B-008-4-59, B-012-4-59, were obtained along the alignment of the existing adjacent S. Fourth Street bridge structure over I-70/I-71 (FRA-40-1334). 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 depths ranging from 51.0 to 56.0 feet 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 higher than the NAD 83 datum. **Therefore, all elevations noted in this report with respect to the historic borings are adjusted to the current NAD 83 datum.** The historic boring locations are shown on the boring plan provided in Appendix I of this report and the historic boring logs are provided in Appendix IV.

4.0 FINDINGS

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

4.1 Surface Materials

Boring B-033-3-15 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. Boring B-034-0-08 was performed at the top of the slope in the grass berm on the south side of Fulton Street, west of S. Fourth Street, and encountered 3.0 inches of topsoil at the ground surface. Boring B-035-0-08 was performed in the existing pavement of I-70 eastbound and encountered 8.0 inches of asphalt overlying 10.0 inches of concrete followed by 3.0 inches of aggregate base at the ground surface.

4.2 Subsurface Soils

Beneath the topsoil in boring B-034-0-08, material identified as existing fill was encountered extending to a depth of 18.5 feet below exiting grade, which corresponds to an elevation of 733.0 feet msl. The fill material was described as loose to medium dense, brown gravel and sand (ODOT A-1-b) and stiff, brown sandy silt (ODOT A-4a) and contained brick fragments throughout, as noted on the visual descriptions provided on the boring logs. Two thick layers of brick fragments were also present within the fill material.

Underlying the surficial materials and existing fill, natural granular soils were encountered with intermittent seams and layers of cohesive material. The granular soils were described as loose to very dense, gray, brown and brownish gray gravel, gravel and sand, fine sand, coarse and fine sand, sandy silt and silt (ODOT A-1-a, A-1-b, A-3, A-3a, A-4a, A-4b). The seams and layers of cohesive material were described as very soft to hard, gray and brown sandy silt, silt and silt and clay (ODOT A-4a, A-4b, A-6a). A layer of cobbles and boulders was encountered in boring B-034-0-08 at approximate elevation 684.5 to 689.5 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 loose ($5 \leq N_{60} \leq 10$ blows per foot [bpf]) to very dense ($N_{60} > 50$ bpf). Blow counts recorded from the SPT sampling within the granular soils ranged from 8 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 very soft (HP ≤ 0.25 tsf) to hard (HP > 4.0 tsf). The unconfined compressive strength of the cohesive soil samples tested, obtained from the hand penetrometer, ranged from 0.25 to over 4.5 tsf (limit of instrument).

Natural moisture contents of the cohesive soil samples ranged from 8 to 13 percent. The natural moisture content of the cohesive soil samples tested for plasticity index ranged from 3 percent below to 2 percent above their corresponding plastic limits. The seams of cohesive soil exhibited natural moisture contents considered to be slightly below to slightly above optimum moisture levels. Natural moisture contents of the granular soil samples ranged from 4 to 16 percent, which were described as moist to wet.

4.3 Bedrock

Bedrock was encountered in boring B-034-0-08 as presented in Table 3.

Table 3. Top of Bedrock Elevations

Boring Number	Ground Surface Elevation (feet msl)	Top of Bedrock		Top of Bedrock Core	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-034-0-08	751.5	115.0	636.5	115.5	636.0

Top of bedrock was encountered in boring B-034-0-08 at a depth of 115.0 feet below existing grade, which corresponds to an elevation of 636.5 feet msl. The bedrock consisted of gray, severely weathered shale overlying dark gray, highly to severely weathered shale. The cored shale bedrock is described as dark gray, highly to severely weathered, weak, thinly laminated, calcareous, pyritic, fissile, jointed and fractured with tight, slightly rough apertures.

The percent recovery and RQD values from the bedrock core runs in boring B-034-0-08 are summarized in Table 4.

Table 4. Rock Core Summary

Boring	Core No.	Depth (feet)	Recovery (%)	RQD (%)
B-034-0-08	R-1	115.5 to 120.5	65	55
	R-2	120.5 to 125.5	88	77
	R-3	125.5 to 130.5	100	57
	R-5	130.5 to 135.5	100	85

It should be noted that bedrock can experience mechanical breaks during the drilling and coring processes. It is anticipated that DLZ attempted to account for fresh, manmade breaks during tabulation of the RQD analysis, per ODOT SGE specifications. The quality of the shale bedrock, according to the RQD values, was fair (50 < RQD ≤ 75%) to good (75 < RQD ≤ 90%).

4.4 Groundwater

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

Table 5. Groundwater Levels

Boring Number	Ground Elevation (feet msl)	Initial Groundwater		Upon Completion	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-033-3-15	731.2	11.0	720.2	N/A [1]	-
B-034-0-08	751.5	28.5	723.0	43.3 [2]	708.2
B-035-0-08	732.3	9.2	723.1	9.2	723.1

2. *The groundwater level at completion could not be obtained due to the addition of mud as a drilling fluid.*
3. *Groundwater level at completion of drilling was measured inside the casing, and may be influenced by the introduction of water during the coring process.*

Groundwater was encountered initially during the drilling process in borings B-033-3-15, B-034-0-08 and B-035-0-08 at a depths ranging from 9.2 to 28.5 feet below the existing ground surface, which corresponds to elevations ranging from 720.2 to 723.1 feet msl. At the completion of drilling, groundwater was encountered in borings B-034-0-08 and B-035-0-08 at a depth of 43.3 and 9.2 feet below grade, respectively, which corresponds to elevations of 708.2 and 723.1 feet msl. It should be noted that the groundwater level at completion of drilling in boring B-034-0-08 was measured inside the casing, and may be influenced by the introduction of water during the coring process. The groundwater level at the completion of drilling in boring B-033-3-15 could not be measured due to the addition of mud 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-001-4-59, B-008-4-59 and B-012-4-59, encountered loose to very dense granular soils with intermittent seams hard cohesive soil. 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, coarse and fine sand, sandy silty and silt (ODOT A-1-a, A-1-b, A-2-4, A-2-6, A-3, A-3a, A-4a, A-4b), and the cohesive soil seams encountered were described as brown and gray sandy silt, silt, silt and clay and silty clay (ODOT A-4a, A-4b, A-6a, A-6b). Existing fill consisting of hard sandy silt with brick fragments was encountered in boring B-012-4-59 extending to a depth of 12.0 feet below grade. 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 a two-span continuous steel plate girder structure 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 existing ramp from S. Third Street to I-70 eastbound will be eliminated and the proposed rear abutment will be shifted approximately 95 feet north of the existing rear abutment, and the proposed forward abutment will be shifted to approximately 70 feet north of the existing forward abutment. The proposed structure will have an approximate length of 190 feet and width of 86 feet.

Proposed structural data was obtained from design details provided by GPD GROUP and are included in Table 6.

Table 6. FRA-23-1075 Structure and Bridge Design Elevations

Substructure Unit (Borings)	Structure Component [1]	Elevation [1] (feet msl)	Design Maximum Factored Load	
			Service	Strength
Rear Abutment (B-001-4-59 / B-033-3-15)	Bridge Bottom of Footing	722.5	5.90 ksf	8.46 ksf
	Wingwall Bottom of Footing	West: 721.5 East: 723.5	4.35 ksf	6.21 ksf
Pier (B-008-4-59 / B-012-4-59)	Bottom of Footing	722.5	1.75 ksf	2.60 ksf
Forward Abutment (B-012-4-59 / B-034-0-08)	Top of Embedded Shaft (Bottom of Wall)	722.0 Lt. 724.5 Rt.	164 kips	214 kips

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 2.5 feet below the existing roadway profile grade, and there will be no change in the profile grade of S. Fourth Street. Where the proposed I-70 westbound extends outside the limits of the existing I-70 roadway, the proposed profile grade will be cut up to 19.5 feet below the existing grade of S. Fourth Street and adjacent slopes. Embankment fill will be placed behind the proposed rear abutment to achieve the final design grade above the existing ramp and spill through slope in front of the existing abutment, with a maximum fill height of approximately 21 feet.

5.1 Shallow Foundation Recommendations

It is understood that shallow spread foundations will be utilized at the rear abutment and pier substructure units. Based on plan information provided by GPD GROUP, the bottom of footing elevation at both substructure units will bear at a minimum depth of 5.0 feet below the proposed finished grade, at elevations noted above in Table 6. At these elevations, the bearing soils are anticipated to consist of dense to very dense gravel, gravel with sand and coarse and fine sand (ODOT A-1-a, A-1-b, A-3a) overlying hard silt, silt and clay and silty clay (ODOT A-4b, A-6a, A-6b). It should be noted that borings B-008-4-59 and B-012-4-59, which are the only borings that are located at the pier substructure, have a bottom of boring elevation of 695.3 and 699.6 feet msl, respectively, which are approximately 27 and 22 feet below the proposed bottom of footing elevation at the pier. Shallow spread foundations bearing on these competent natural soils may be proportioned for a nominal bearing resistance as presented in Table 7 for the rear abutment and Table 8 for the pier substructure.

Table 7. FRA-23-1075 Rear Abutment Spread Footing Design Parameters

Boring No.	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		
B-001-4-59	5.0	3.23	8.03	15.12	22.92	12.60
	7.0	2.80	6.42	11.64	22.96	12.63
	9.0	2.57	5.54	9.78	23.00	12.65
	11.0	2.42	4.99	8.64	23.04	12.67
	13.0	2.32	4.62	7.89	23.08	12.70
	15.0	2.25	4.35	7.37	23.13	12.72
	17.0	2.19	4.15	6.99	23.17	12.74
	19.0	2.15	4.00	6.71	23.21	12.76
	21.0	2.12	3.88	6.49	23.25	12.79
	23.0	2.09	3.79	6.31	23.29	12.81
	25.0	2.07	3.71	6.17	23.33	12.83
B-033-3-15	5.0	4.04	11.33	24.36	57.76	31.77
	7.0	3.44	8.87	18.48	66.99	36.84
	9.0	3.10	7.51	15.22	76.41	42.03
	11.0	2.88	6.64	13.15	85.94	47.27
	13.0	2.73	6.03	11.72	95.50	52.53
	15.0	2.62	5.59	10.67	105.06	57.78
	17.0	2.53	5.25	9.87	114.60	63.03
	19.0	2.46	4.99	9.23	124.10	68.25
	21.0	2.41	4.77	8.73	133.55	73.45
	23.0	2.36	4.59	8.31	142.94	78.62
	25.0	2.32	4.44	7.96	152.27	83.75

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 8. FRA-23-1075 Pier Spread Footing Design Parameters

Boring No.	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		
B-008-4-59	5.0	2.51	4.90	9.11	32.17	16.09
	7.0	2.26	4.07	7.24	32.32	16.16
	9.0	2.12	3.63	6.24	32.46	16.23
	11.0	2.04	3.35	5.62	32.61	16.30
	13.0	1.98	3.17	5.20	32.76	16.38
	15.0	1.94	3.04	4.91	32.90	16.45
	17.0	1.91	2.94	4.69	33.05	16.52
	19.0	1.89	2.87	4.53	33.19	16.60
	21.0	1.88	2.81	4.40	33.34	16.67
	23.0	1.86	2.77	4.30	33.48	16.74
	25.0	1.85	2.73	4.21	33.63	16.82
B-012-4-59	5.0	4.56	15.89	46.73	27.81	12.51
	7.0	3.88	12.51	35.37	31.58	14.21
	9.0	3.51	10.66	28.44	35.43	15.94
	11.0	3.27	9.49	24.03	39.31	17.69
	13.0	3.10	8.69	21.00	43.18	19.43
	15.0	2.99	8.12	18.78	47.01	21.15
	17.0	2.90	7.69	17.10	50.78	22.85
	19.0	2.83	7.36	15.78	54.50	24.53
	21.0	2.77	7.09	14.71	58.16	26.17
	23.0	2.73	6.88	13.84	61.75	27.79
	25.0	2.69	6.71	13.11	65.26	29.37

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 = 0.50 and 0.45 was utilized in calculating the factored bearing resistance at the strength limit state for B-008-4-59 and B-012-4-59, respectively.*

The service limit bearing pressure that results in a maximum total settlement of 0.5, 1.0 and 1.5 inches was calculated and presented in Table 7. A geotechnical resistance factor of φ_b = 0.55 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 7 and applying the geotechnical resistance factor provided to the nominal bearing resistance at the strength limit state, the service limit state should control the minimum footing dimensions for all effective footing widths analyzed at all settlement values considered in the analysis, with the exception of boring B-012-4-59 at the pier substructure. For this boring, the factored bearing resistance will control the design below a foundation width of 11.0 feet. A graphical representation of the service limit bearing pressures and nominal and factored bearing resistance at the strength limit state for the rear abutment and pier substructures is presented in Appendix V.

Based on the service limit bearing pressures provided in Table 6, total settlements of 0.81 to 1.10 inches are anticipated at the rear abutment, and 0.21 to 0.38 inches are anticipated at the pier. Differential settlement along the and pier substructures is anticipated to be less than 1 in. / 500 ft. Additionally, the maximum factored bearing pressure of 8.46 ksf at the rear abutment and 2.60 ksf at the pier substructure units will not exceed the factored bearing resistance at the strength limit of 12.66 and 16.47 ksf, respectively.

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

5.2 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 9. 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, fine sand, sandy silt and silt (ODOT A-1-a, A-1-b, A-2-4, A-3, A-4a, A-4b) with seams of hard sandy silt and silt and clay (ODOT A-4a, A-6a). The drilled shafts should be proportioned for a nominal bearing resistance as presented in Table 9.

Table 9. FRA-23-1075 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-012-4-59	724.5-723.6	0.0-0.9	A-2-4	60	0.68	0.50	0.55
	723.6-713.6	0.9-10.9	A-1-b	60	1.84	0.50	0.55
	713.6-708.6	10.9-15.9	A-1-a	60	3.53	0.50	0.55
	708.6-706.6	15.9-17.9	A-3	60	1.34	0.50	0.55
	706.6-703.6	17.9-20.9	A-2-6	60	3.69	0.50	0.55
	703.6-701.1	20.9-23.4	A-3	60	1.50	0.50	0.55
	701.1-699.6	23.4-24.9	A-1-b	60	4.14	0.50	0.55
B-034-0-08	722.0-718.0	0.0-4.0	A-1-a	20	0.36	0.50	0.55
	718.0-713.0	4.0-9.0	A-1-b	58	1.28	0.50	0.55
	713.0-710.5	9.0-11.5	A-4b	60	2.23	0.50	0.55
	710.5-707.5	11.5-14.5	A-1-a	21	0.81	0.50	0.55
	707.5-704.5	14.5-17.5	A-6a	65	3.26	0.40	0.45
	704.5-701.5	17.5-20.5	A-1-a	60	3.86	0.50	0.55
	701.5-689.5	20.5-32.5	A-4a	42	2.26	0.40	0.45
	689.5-684.5	32.5-37.5	A-1-a	60	4.78	0.50	0.55
	684.5-664.5	37.5-57.5	A-1-b	60	4.90	0.50	0.55
	664.5-659.5	57.5-62.5	A-4a	60	5.63	0.50	0.55
	659.5-636.5	62.5-85.5	A-1-b	60	5.57	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 9 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 primarily of dense granular soils, the analysis should be performed considering the entire drilled shaft group as an equivalent strip footing with a length equal to the length of the tangent shaft wall and equivalent width equal to the total end area of the drilled shafts divided by the length of the drilled shaft wall. A resistance factor of φ_b = 0.45 should be utilized in calculating the factored bearing resistance for this failure mode at the strength limit state.

The total group resistance shall be the lesser of the sum of the individual drilled shafts multiplied by the applicable group efficiency factor, η, or the 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 10 lists the eleven different soil types internal to the LPILE program. These strata were utilized to define the soil strata in the soil profile for each boring provided in Appendix VIII.

Table 10. Subsurface Strata Description

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

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

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

In which:

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

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

It is understood that GPD GROUP has performed an analysis of the lateral loading on the foundation elements at the forward abutment 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 11 below. A tip elevation of 681.0 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 with sand (ODOT A-1-b), 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 with sand (ODOT A-1-b). Using the friction angle for the very dense gravel with sand (ODOT A-1-b), the resulting nominal unit bearing resistance is 330.3 ksf and the factored unit bearing resistance is 165.2 ksf, considering a resistance factor of 0.5.

Table 11. FRA-23-1075 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	722.0	681.0	41.0	5.0	1,060	N/A	1,060	530	N/A	530
Block	5.0	722.0	681.0	41.0	5.0	6,486	N/A	6,486	3,243	N/A	3,243

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 214 kips 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, based on conditions encountered in other borings performed through the corridor, cobbles and boulders were encountered throughout the very dense sand and gravel deposits. Therefore, difficult drilling conditions or boulders should be anticipated to be encountered during installation of the drilled shafts. If boulders are encountered during installation of the drilled shafts, specialized drilling/coring equipment may be required to advance the drilled shaft excavation beyond the obstruction.

5.3 Lateral Earth Pressure

For the soil types encountered in the borings, the "in-situ" unit weight (γ), cohesion (c), effective angle of friction (φ'), and lateral earth pressure coefficients for at-rest conditions (k_o), active conditions (k_a), and passive conditions (k_p) have been estimated and are provided in Table 12 and Table 13.

Table 12. Estimated Undrained (Short-term) Soil Parameters for Design

Soil Type	γ (pcf) [1]	c (psf)	φ	k_a	k_o	k_p
Soft to Stiff Cohesive Soil	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	32°	0.27	0.47	6.82

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

Table 13. Estimated Drained (Long-term) Soil Parameters for Design

Soil Type	γ (pcf) [1]	c (psf)	φ'	k_a	k_o	k_p
Soft to Stiff Cohesive Soil	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	32°	0.27	0.47	6.82

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

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

5.4 Construction Considerations

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

5.4.1 Excavation Considerations

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

Table 14. Excavation Back Slopes

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

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.

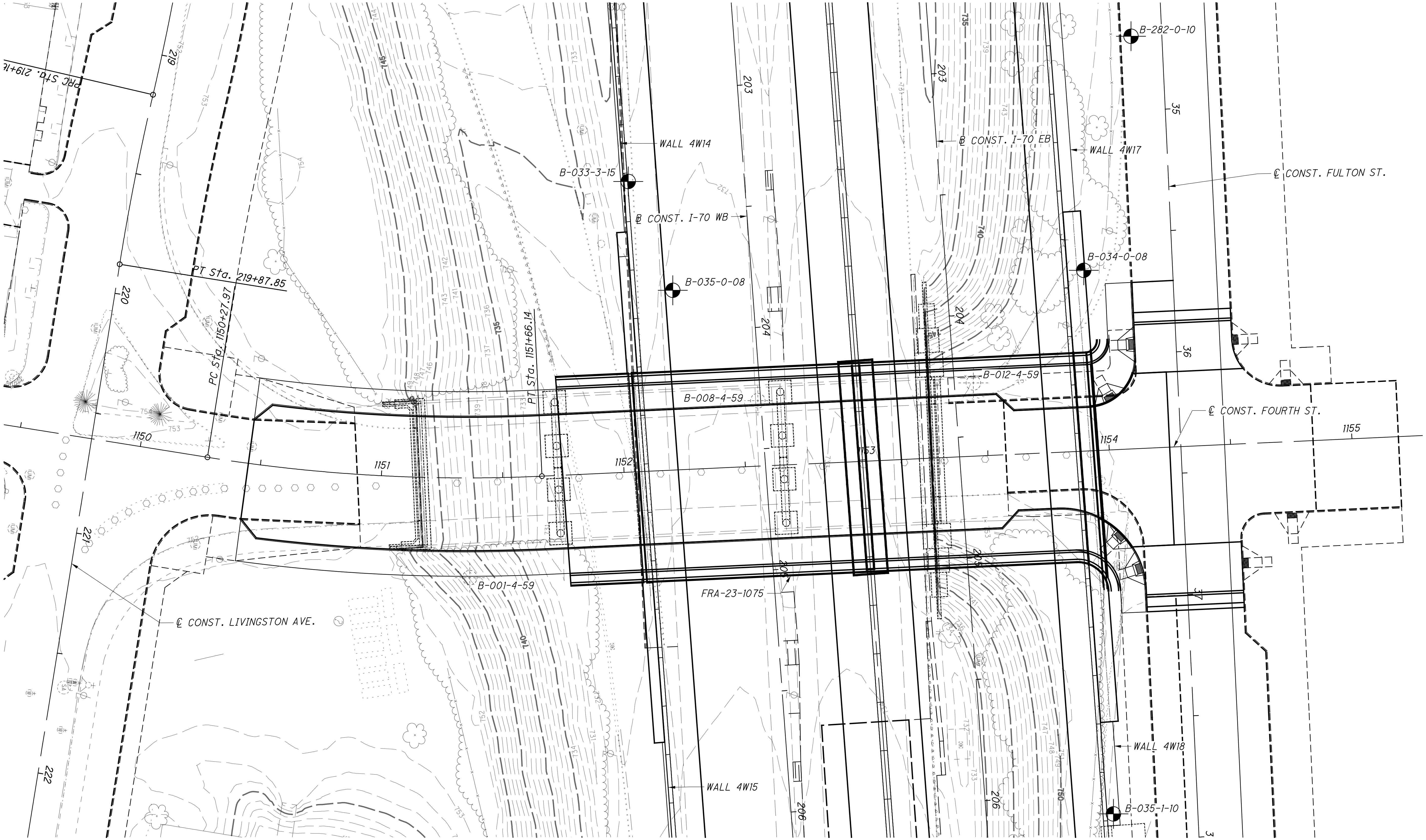
The conclusions and recommendations herein have been based upon the available soil and bedrock information and the design details furnished by a representative of the owner of the proposed project. Any revision in the plans for the proposed construction from those anticipated in this report should be 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-23-1075 S. FOURTH ST. OVER I-70/71

FRA-70-14.05

FRANKLIN COUNTY, OHIO

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

RESOURCE INTERNATIONAL Rii

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-033-3-15, B-034-0-08 and B-035-0-08

BORING LOGS

Definitions of Abbreviations

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

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

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

Classification Test Data

Gradation (as defined on Description of Soil Terms):

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

Atterberg Limits:

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

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-14.05 PROJECT 4B	DRILLING FIRM / OPERATOR: RII / M.W.	DRILL RIG: CME 55 (SN 386345)	STATION / OFFSET: 203+36.15 / 47.7" RT	EXPLORATION ID B-033-3-15
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: RII / N.A.	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: 731.2 (MSL) EOB: 69.0 ft.	PAGE 1 OF 3
START: 10/7/15 END: 10/7/15	SAMPLING METHOD: SPT	ENERGY RATIO (%): 92	COORD: 39.953302, -82.994869	

MATERIAL DESCRIPTION AND NOTES	ELEV. 731.2	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")	730.9																	
VERY STIFF, BROWN **SANDY SILT**, SOME CLAY, LITTLE FINE GRAVEL, DAMP. -ORGANICS PRESENT IN SS-1	728.2	1–2	6 10 6	24	94	SS-1	4.00	-	-	-	-	-	-	-	-	10	A-4a (V)	
DENSE, BROWN **COARSE AND FINE SAND**, LITTLE FINE GRAVEL, TRACE SILT, TRACE CLAY, DAMP.	725.7	3.5–5	11 15 16	47	78	SS-2	-	11	29	48	9	3	NP	NP	NP	4	A-3a (0)	
DENSE TO VERY DENSE, BROWN AND BROWNISH GRAY **GRAVEL AND SAND**, TRACE TO LITTLE SILT, TRACE CLAY, MOIST. -SS-3 SULFATE CONTENT = 87 PPM		6–7.5	11 15 19	51	100	SS-3	-	-	-	-	-	-	-	-	-	7	A-1-b (V)	
		8.5–10	6 17 17	51	100	SS-4	-	34	30	26	7	3	NP	NP	NP	7	A-1-b (0)	
		W 11–12.5	3 17 20	56	100	SS-5	-	-	-	-	-	-	-	-	-	13	A-1-b (V)	
		13.5–15	12 15 18	50	100	SS-6	-	-	-	-	-	-	-	-	-	13	A-1-b (V)	
		16–17.5	11 17 21	57	100	SS-7	-	35	33	19	11	2	NP	NP	NP	14	A-1-b (0)	
-HEAVING SANDS ENCOUNTERED @ 18.5' -INTRODUCED MUD @ 18.5'		18.5–20	19 20 29	74	100	SS-8	-	-	-	-	-	-	-	-	-	12	A-1-b (V)	
		23.5–25	19 23 28	77	100	SS-9	-	23	29	33	13	2	NP	NP	NP	12	A-1-b (0)	
	701.5	28.5–30	21 28 27	83	78	SS-10A	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	

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PID: 96053	BR ID: NA	PROJECT: FRA-70-14.05 PROJECT 4B	STATION / OFFSET: 203+36.15 / 47.7' RT	START: 10/7/15	END: 10/7/15	PG 2 OF 3	B-033-3-15

MATERIAL DESCRIPTION AND NOTES	ELEV. 701.2	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 **SILT**, SOME COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP TO MOIST. *(same as above)*						SS-10B	4.5+	-	-	-	-	-	-	-	-	10	A-4b (V)	
		34	29 30 41	107	100	SS-11	4.50	3	5	22	59	11	19	11	8	13	A-4b (7)	
	694.2																	
VERY DENSE, GRAY **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, MOIST.																		
		39	13 39 43	123	100	SS-12	-	-	-	-	-	-	-	-	-	16	A-1-b (V)	
		44	20 34 35	104	83	SS-13	-	42	39	12	6	1	NP	NP	NP	11	A-1-b (0)	
-HEAVING SANDS ENCOUNTERED @ 48.5'		49	43 50/4"	-	100	SS-14	-	-	-	-	-	-	-	-	-	8	A-1-b (V)	
		54	50/5"	-	100	SS-15	-	-	-	-	-	-	-	-	-	13	A-1-b (V)	
	674.2																	
HARD, GRAY **SILT AND CLAY**, "AND" COARSE TO FINE SAND, LITTLE FINE GRAVEL, DAMP.																		
		59	37 50/1"	-	100	SS-16	4.5+	17	15	22	30	16	24	13	11	10	A-6a (3)	
	669.2																	

MATERIAL DESCRIPTION AND NOTES	ELEV. 669.1	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
VERY DENSE, GRAY **SANDY SILT**, LITTLE FINE GRAVEL, TRACE CLAY, MOIST. *(same as above)*		63																
		64 – 65	20 35 40	113	100	SS-17	-	-	-	-	-	-	-	-	-	11	A-4a (V)	
	664.2	66 – 67																
VERY DENSE, GRAY **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, MOIST.	662.2	68 – 69 EOB	50/6"	-	100	SS-18	-	-	-	-	-	-	-	-	-	13	A-1-b (V)	

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 11.0'

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

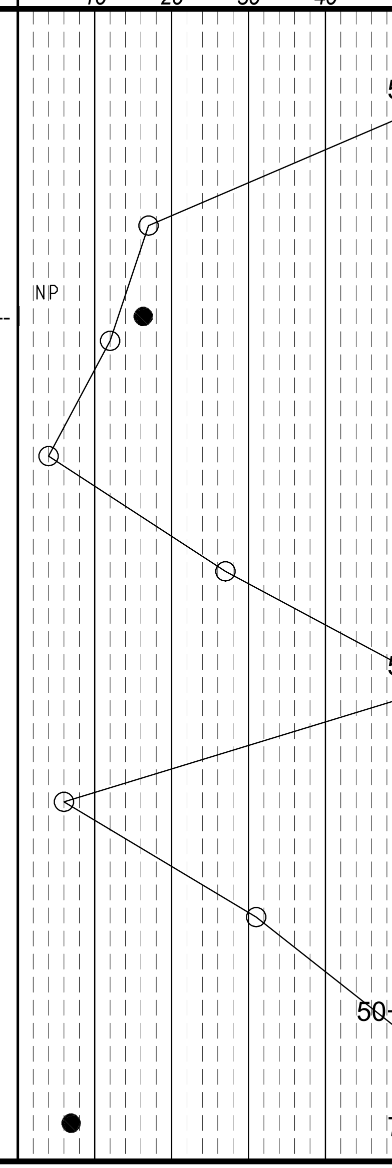
Client: ms consultants	Project: FRA-70-8.93	Job No. 0221-1004.01
LOG OF: Boring B-034-0-08	Location: Sta. 203+85.26, 56.62' LT., BL I-70 WB	Date Drilled: 8/7/2008 to 8/14/2008

WATER OBSERVATIONS:
Water seepage at: 5.5', 10.5', 20.5', 25.5', 62.0'
Water level at completion: 43.3' (measured inside casing)
At beginning of shift: 8/8 28.5, BOH 45.0'

FIELD NOTES:
Advanced boring using 4.0" diameter flush joint casing.

Depth (ft)	Elev. (ft) 751.5	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	Graphic Log	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0.3	751.2						Topsoil - 3"							
3.5	748.0	19 35 14	8	1			Brick Fragments							
6.0	745.5	5 7 9	3	2		1.5	FILL: Stiff brown SANDY SILT (A-4a), little fine to coarse sand, little gravel; contains numerous 1/8"-1/4" size brick fragments; moist.							
8.5	743.0	2 5 6	16	3			FILL: Medium dense brown GRAVEL WITH SAND (A-1-b), little silt; contains numerous 1/8"-1/4" size brick fragments; wet.		41	32	---	12	--15--	
11.0	740.5	2 4 WOH	8	4			Brick Fragments							
		4 6 20	10	5		1.75	FILL: Stiff brown and gray SANDY SILT (A-4a), some fine to coarse sand, some gravel; contains brick and rock fragments; moist.							
16.0	735.5	15 25 27	18	6		--								
18.5	733.0	6 2 4	6	7			FILL: Loose brown GRAVEL WITH SAND (A-1-b), little silt; contains few 1/8'-1/2' size brick fragments; wet.							
21.0	730.5	11 13 16	13	8		4.5+	Hard gray SILT AND CLAY (A-6a), some fine to coarse sand, trace gravel; contains few 1/8'-1/2' size brick fragments; damp.							
23.5	728.0	28 45 50/5	2	9			Very dense gray GRAVEL WITH SAND (A-1-b), trace silt; possible cobbles; wet.							
25	726.5	13 23 49	2	10		4.5+	Hard gray SANDY SILT (A-4a), some fine to coarse sand, little gravel; damp.							

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

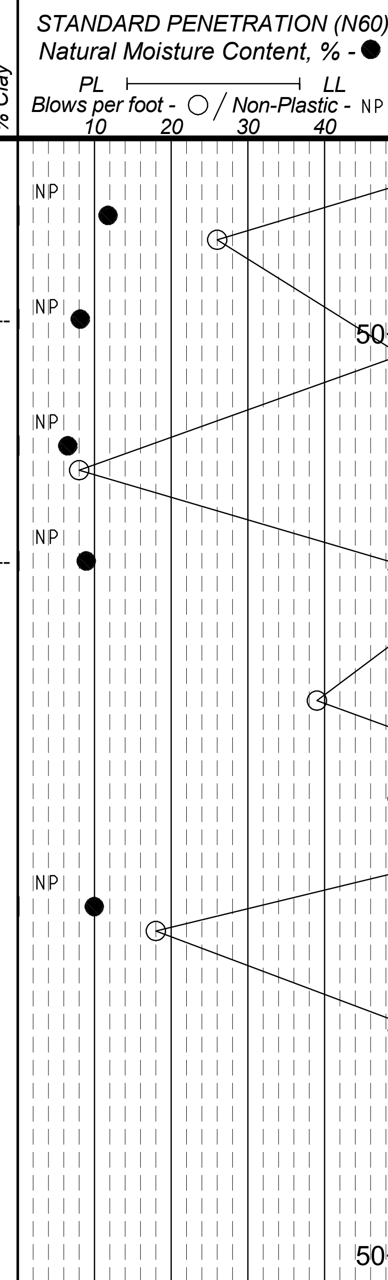


Client: ms consultants	*Project:* FRA-70-8.93	*Job No.* 0221-1004.01
LOG OF: Boring B-034-0-08	*Location:* Sta. 203+85.26, 56.62' LT., BL I-70 WB	*Date Drilled:* 8/7/2008 to 8/14/2008

WATER OBSERVATIONS:
Water seepage at: 5.5', 10.5', 20.5', 25.5', 62.0'
Water level at completion: 43.3' (measured inside casing)
At beginning of shift: 8/8 28.5, BOH 45.0'

FIELD NOTES:
Advanced boring using 4.0" diameter flush joint casing.

Depth (ft)	Elev. (ft) 726.5	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	Graphic Log	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
		6 11 14	12	11			Medium dense gray GRAVEL (A-1-a), little to some fine to coarse sand, trace silt; contains few brick fragments (likely picked up at higher elevation); wet.		74	15	---	6	--5--	
28.5	723.0	19 50/5	6	12			Very dense gray GRAVEL WITH SAND (A-1-b), little silt; wet.		36	34	---	15	--15--	
31.0	720.5	6 4 4	18	13			Very loose to loose gray GRAVEL (A-1-a), little fine to coarse sand, trace silt; contains few brick fragments (likely picked up at higher elevation); wet.		81	12	---	4	--3--	
33.5	718.0	19 30 26	18	14			Very dense gray GRAVEL WITH SAND (A-1-b), trace to little silt; wet.		33	30	---	21	--16--	
		8 11 26	14	15			@ 36.0'-37.5', contains 3/4"x1-1/2" steel fragment - possible conduit fragment (likely picked up at higher elevation); wet.							
38.5	713.0	16 35 41	18	16			Very dense gray SILT (A-4b), trace silt, trace clay; contains occasional sand seams; wet.							
41.0	710.5	8 8 9	18	17			Medium dense gray GRAVEL (A-1-a), little fine to coarse sand; wet. @ 43.5'-44.0', fine sand seam.		87	9	---	3	--1--	
44.0	707.5	18 25 30	18	18		<0.25	Very soft gray SILT AND CLAY (A-6a), some fine to coarse sand, trace gravel; wet.							
47.0	704.5						Very dense gray GRAVEL (A-1-a), some fine to coarse sand; wet.							
50	701.5	8 28 50/5	8	19										



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

LOG OF: Boring B-034-0-08 | Location: Sta. 203+85.26, 56.62' LT., BL I-70 WB | Date Drilled: 8/7/2008 to 8/14/2008

WATER OBSERVATIONS:
Water seepage at: 5.5', 10.5', 20.5', 25.5', 62.0'
Water level at completion: 43.3' (measured inside casing)
At beginning of shift: 8/8 28.5, BOH 45.0'

FIELD NOTES:
Advanced boring using 4.0" diameter flush joint casing.

Depth (ft)	Elev. (ft) 701.5	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	Graphic Log	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N60)
							Very stiff to hard gray SANDY SILT (A-4a), some to "and" fine to coarse sand, trace gravel; damp.								
55		20 18 18	8	20		3.75			5	14	---	21	46	14	
60		50/2	2	21		4.5+	@ 58.5'-62.0', possible cobbles.								50+
62.0	689.5						Boulder/cobble fragments.								
65		50/5	3	22											50+
67.0	684.5						Very dense gray GRAVEL WITH SAND (A-1-b), some fine to coarse sand, little silt; possible cobbles; wet.								
70		50/2	2	23											50+
72.0	679.5						Very dense gray COARSE AND FINE SAND (A-3a), little gravel, trace silt; possible cobbles; wet.								
75	676.5	37 50/5	9	24											50+

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

Client: ms consultants	Project: FRA-70-8.93	Job No. 0221-1004.01
LOG OF: Boring B-034-0-08	Location: Sta. 203+85.26, 56.62' LT., BL I-70 WB	Date Drilled: 8/7/2008 to 8/14/2008

WATER OBSERVATIONS:
Water seepage at: 5.5', 10.5', 20.5', 25.5', 62.0'
Water level at completion: 43.3' (measured inside casing)
At beginning of shift: 8/8 28.5, BOH 45.0'

FIELD NOTES:
Advanced boring using 4.0" diameter flush joint casing.

Depth (ft)	Elev. (ft) 676.5	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	Standard Penetration (N60) / Natural Moisture Content, %
							Very dense gray GRAVEL WITH SAND (A-1-b), some silt; moist.							
80		25 38 50/5	15	25				28	31	---	9	--32--		NP; 50+
85		21 32 47	18	26										83
87.0	664.5						Very dense gray SANDY SILT (A-4a), some gravel; moist.							
90		22 35 50/5	17	27										50+
92.0	659.5						Very dense gray GRAVEL WITH SAND (A-1-b), "and" fine to coarse sand, little silt; wet.							
95		35 50/5	9	28										50+
97.0	654.5						Very stiff gray SANDY SILT (A-4a), some fine to coarse sand, some gravel; possible cobbles; moist.							
100	651.5	50/5	5	29		3.25								50+

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

LOG OF: Boring B-034-0-08 | **Location:** Sta. 203+85.26, 56.62' LT., BL I-70 WB | **Date Drilled:** 8/7/2008 to 8/14/2008

WATER OBSERVATIONS:
Water seepage at: 5.5', 10.5', 20.5', 25.5', 62.0'
Water level at completion: 43.3' (measured inside casing)
At beginning of shift: 8/8 28.5, BOH 45.0'

FIELD NOTES:
Advanced boring using 4.0" diameter flush joint casing.

Depth (ft)	Elev. (ft) 651.5	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N60) / Natural Moisture Content, % / PL–LL / Blows per foot / Non-Plastic - NP
							Very dense gray GRAVEL WITH SAND (A-1-b), little to some silt; wet. @ 100.0'-105.0', possible cobbles.							
105		37 50/5	6	30										50+
110		30 33 33	18	31				25	47	---	16	--12--		NP; 69
115.0 (115)	636.5	23 42 50/3	12	32										50+
115.5	636.0	50/3	3	33			Severely weathered gray SHALE.							50+
		Core 60"	Rec 39"	RQD 55%	R-1		Shale, dark gray, highly to severely weathered, weak, thinly laminated, fissile, calcareous, pyritic, jointed, fractured, tight, slightly rough, RQD 69%, Loss 12%. Contains occasional large (up to 1" diameter) pyritic inclusion. @ 116.6'-117.0', encountered thin limestone bed. @ 117.5'-119.2', 121.1'-121.7', possible lost recovery.							
120														
		Core 60"	Rec 53"	RQD 77%	R-2		@ 121.7', moderately fractured, highly weathered.							
125	626.5													

SPT plot scale: 10, 20, 30, 40

Client: ms consultants	Project: FRA-70-8.93	Job No. 0221-1004.01
LOG OF: Boring B-034-0-08	Location: Sta. 203+85.26, 56.62' LT., BL I-70 WB	Date Drilled: 8/7/2008 to 8/14/2008

WATER OBSERVATIONS:
Water seepage at: 5.5', 10.5', 20.5', 25.5', 62.0'
Water level at completion: 43.3' (measured inside casing)
At beginning of shift: 8/8 28.5, BOH 45.0'

FIELD NOTES:
Advanced boring using 4.0" diameter flush joint casing.

Depth (ft)	Elev. (ft) 626.5	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	Graphic Log	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N60) Natural Moisture Content, % - ● PL LL Blows per foot - ○ / Non-Plastic - NP 10 20 30 40
		Core 60"	Rec 60"	RQD 57%	R-3		Shale, dark gray, highly to severely weathered, weak, thinly laminated, fissile, calcareous, pyritic, jointed, fractured, tight, slightly rough, RQD 69%, Loss 12%. Contains occasional large (up to 1" diameter) pyritic inclusion. @ 126.7'-127.2', 129.8'-130.4', 131.1'-131.5', high angle fractures.								
130							@ 130.0' -130.3', qu = 3978 psi								
		Core 60"	Rec 60"	RQD 85%	R-4		@ 133.5'-133.7', encountered thin limestone bed.								
135															
135.5	616.0						Bottom of Boring - 135.5'								
140															
145															
150															

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

LOG OF: Boring B-035-0-08 | Location: Sta. 203+82.19, 32.74' RT., BL I-70 EB | Date Drilled: 7/7/2008

WATER OBSERVATIONS:
Water seepage at: 9.2'-15.0'
Water level at completion: 9.2'

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

Depth (ft)	Elev. (ft) 732.3	Blows per 6"	Recovery	Sample No. Drive	Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	Natural Moisture Content / Blows per foot (N60)
1.8	730.5						Asphalt Concrete Pavement - 8" Portland Cement Concrete - 10" Aggregate Base - 3"							
3.0	729.3	7 15 26	15	1			Dense brown FINE SAND (A-3), trace silt, trace gravel; contains silty clay seams; damp.							
		9 25 26	15	2			Very dense brown GRAVEL WITH SAND (A-1-b), trace to little silt; damp.							52
6.0	726.3	10 25 27	15	3				37	36	---	17	--10--		NP; 53
7.5	724.8	5 14 18	13	4			Dense to very dense brown FINE SAND (A-3), trace to little gravel; damp.	12	20	---	60	--8--		
		10 24 27	13	5			Dense to very dense brown GRAVEL WITH SAND (A-1-b), trace to little silt; wet.	49	28	---	15	--8--		NP; 52
		11 22 25	13	6				34	36	---	20	--10--		NP
		12 23 27	15	7										51
15.0	717.3	10 22 34	13	8										57
							Bottom of Boring - 15.0'							

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

APPENDIX IV

HISTORIC BORING LOGS:

B-001-4-59, B-008-4-59 and B-012-4-59

B-001-4-59

STATE OF OHIO
DEPARTMENT OF HIGHWAYS
TESTING LABORATORY

LOG OF BORING

CO., RT. NO., SEC. FRA-40-12.82 BRIDGE NO. FRA-40-1334
REAR ABUTMENT SOUTH INNERBELT UNDER FOURTH STREET

LOCATION: T.H. 1B STA. 51+36 OFFSET 37'RT FED. NO.

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
752.9	0			
	2			
	4			
747.9	6	17/42	21530	Brown Silty Sandy Gravel W/Stone Fragts.
	8			
742.9	10			
		25/35	21531	Gray Silty Sandy Gravel
740.4	12			
	14	20/36	21532	Gray Gravelly Sandy Silt
737.9				
	16	24/36	21533	Gray Gravelly Sandy Silt
735.4	18	24/49	21534	Gray Gravelly Sandy Silt
732.9	20			
	22	36/46	21535	Gray Sandy Gravel
730.4				
	24	9/14	21536	Gray Sandy Gravel
727.9				
	26	43/60	21537	Gray Sandy Gravel
725.4	28			
		39/56	21538	Gray Gravelly Sand
722.9	30			
		43/70	21539	Gray Gravelly Sand
	32			
	34			
717.9				
	36	50/65	21540	Gray Sandy Gravel

BM
7-7-59

B-001-4-59

SHEET 5

LOG OF BORING (CONTINUED)

BRIDGE NO. FRA-40-1334 T.H. 1B

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
	38			
712.9	40	10/25	21541	Gray Silt
	42			
	44			
707.9	46	22/40	21542	Gray Silt and Clay
	48			
702.9	50	50/50	21543	Gray Gravelly Sandy Silt
	52			
	54			
697.9		82/100	21544	Gray Sandy Silt
696.9	56			BOTTOM OF BORING
	58			
	60			
	62			
	64			
	66			
	68			
	70			
	72			
	74			
	76			
	78			
	80			
	82			

B-008-4-59

SHEET 6

STATE OF OHIO
DEPARTMENT OF HIGHWAYS
TESTING LABORATORY

LOG OF BORING

CO., RT. NO., SEC. FRA-40-12.82 BRIDGE NO. FRA-40-1334
THIRD PIER S. INNERBELT UNDER FOURTH ST.

LOCATION: T.H. 8 B STA. 52+58 OFFSET 30'LT. FED. NO.

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
751.9	0			Sidewalk
	2			
	4			
746.9	6	3/4	22544	Brown Silty Gravel
	8			
741.9	10			
	12	18/24	22545	Brown Silty Sand
	14			
736.9	16	22/34	22546	Gray Sandy Silt
	18			
731.9	20			
	22	37/70	22547	Gray Gravelly Sand
729.4	24	23/45	22548	Gray Silty Gravelly Sand
726.9	26	15/50	22549	Gray Silty Gravelly Sand
724.4	28	57/66	22550	Gray Silty Gravelly Sand
721.9	30			
	32	18/80	22551	Gray Silty Gravelly Sand
719.4	34	54/50	22552	Gray Silty Gravelly Sand
716.9	36	18/28	22553	Gray Silt and Clay

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B-008-4-59

LOG OF BORING (CONTINUED)

SHEET 7

BRIDGE NO. FRA-40-1334 T.H. 8B

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
714.4	38			
		19/32	22554	Gray Silt and Clay
711.9	40			
		16/80	22555	Gray Silty Clay
	42			
708.9				
	44		22556	Gray Sand (Wash Sample)
606.9				
	46	50/100	22557	Gray Silty Sandy Gravel
703.9	48			
			22558	Gray Sand (Wash Sample)
701.9	50	100*	22559	Gray Sandy Gravelly Silt
701.4	52	100*	22560	Gray Gravelly Sandy Silt
698.9				
	54		22561	Gray Sand
696.9				
695.9	56	100*	22562	Gray Sandy Gravelly Silt
	58			BOTTOM OF BORING
	60			* Refusal
	62			
	64			
	66			
	68			
	70			
	72			
	74			
	76			
	78			
	80			
	82			

B-012-4-59

STATE OF OHIO
DEPARTMENT OF HIGHWAYS
TESTING LABORATORY

LOG OF BORING

CO., RT. NO., SEC. FRA-40-12.82 BRIDGE NO. FRA-40-1334
FORWARD ABUTMENT SOUTH INNERBELT UNDER FOURTH STREET

LOCATION: T.H. 12 B STA. 53+47 OFFSET 33'LT FED. NO.

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
751.2	0			Sidewalk
	2			
	4			
746.2	6			Fill Material-Brown Clay W/Gravel and Fragments of Stone and Brick
	8			
741.2	10			
	12	15/22	21450	Brownish-Gray Gravelly Sandy Silt W/ Stone Fragments and Brick
738.7	14	18/23	21451	Brownish-Gray Sandy Silt W/Stone Fragments
736.2	16	15/25	21452	Brownish-Gray Sandy Silt W/Stone Fragts
733.7	18	33/70	21453	Gray Silty Sandy Gravel
731.2	20			
	22	36/*	21454	Gray Gravel
728.7	24	25/50	21455	Gray Gravelly Sand
726.2	26	17/50	21456	Gray Gravelly Sand
723.7	28	23/42	21457	Gray Sandy Gravel
721.2	30			
	32	17/56	21458	Gray Sandy Gravel
	34			
716.2	36	27/53	21459	Gray Sandy Gravel

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7.7.59

B-012-4-59

LOG OF BORING (CONTINUED)

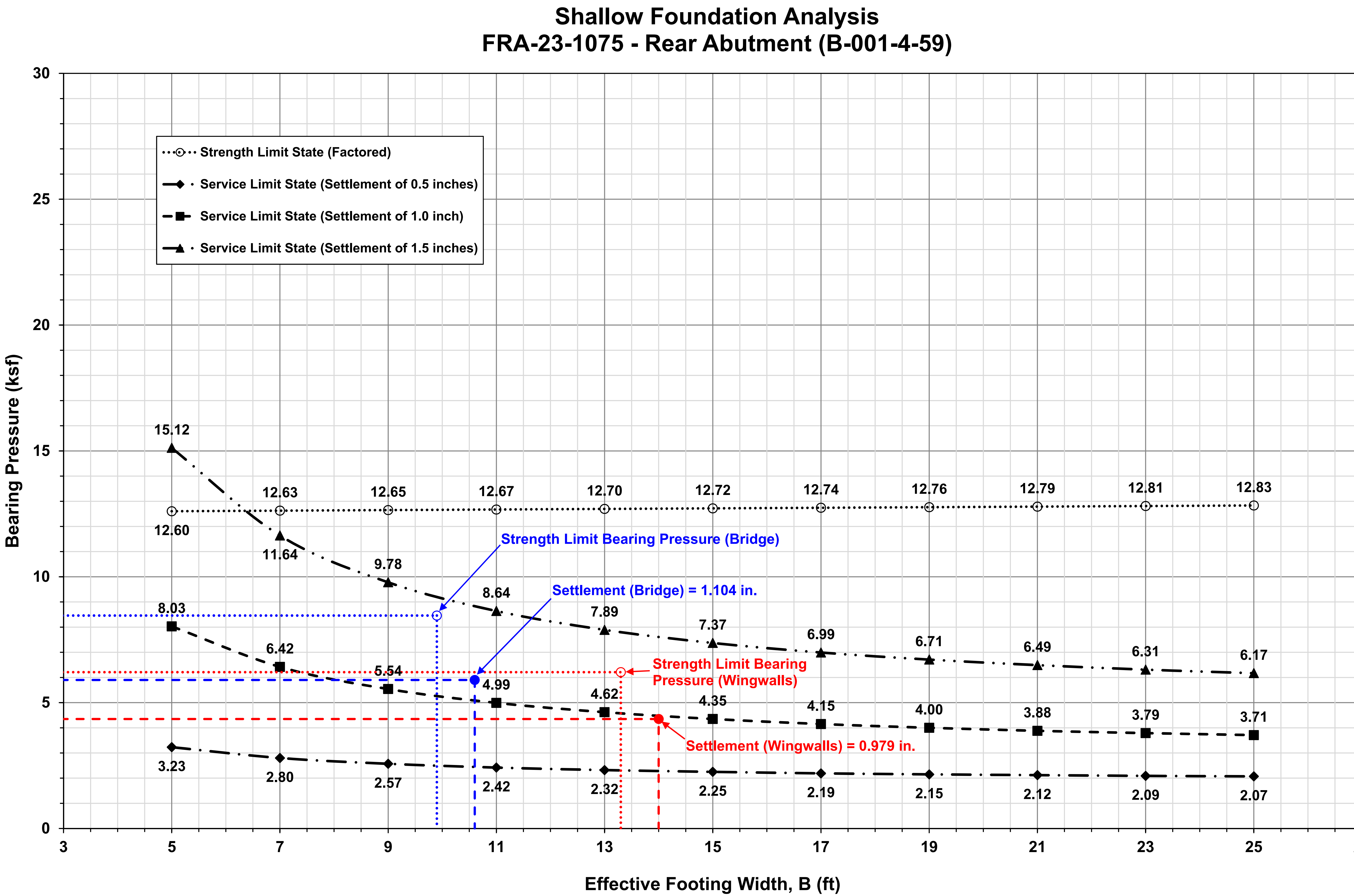
SHEET 9

BRIDGE NO. FRA-40-1334 T.H. 12 B

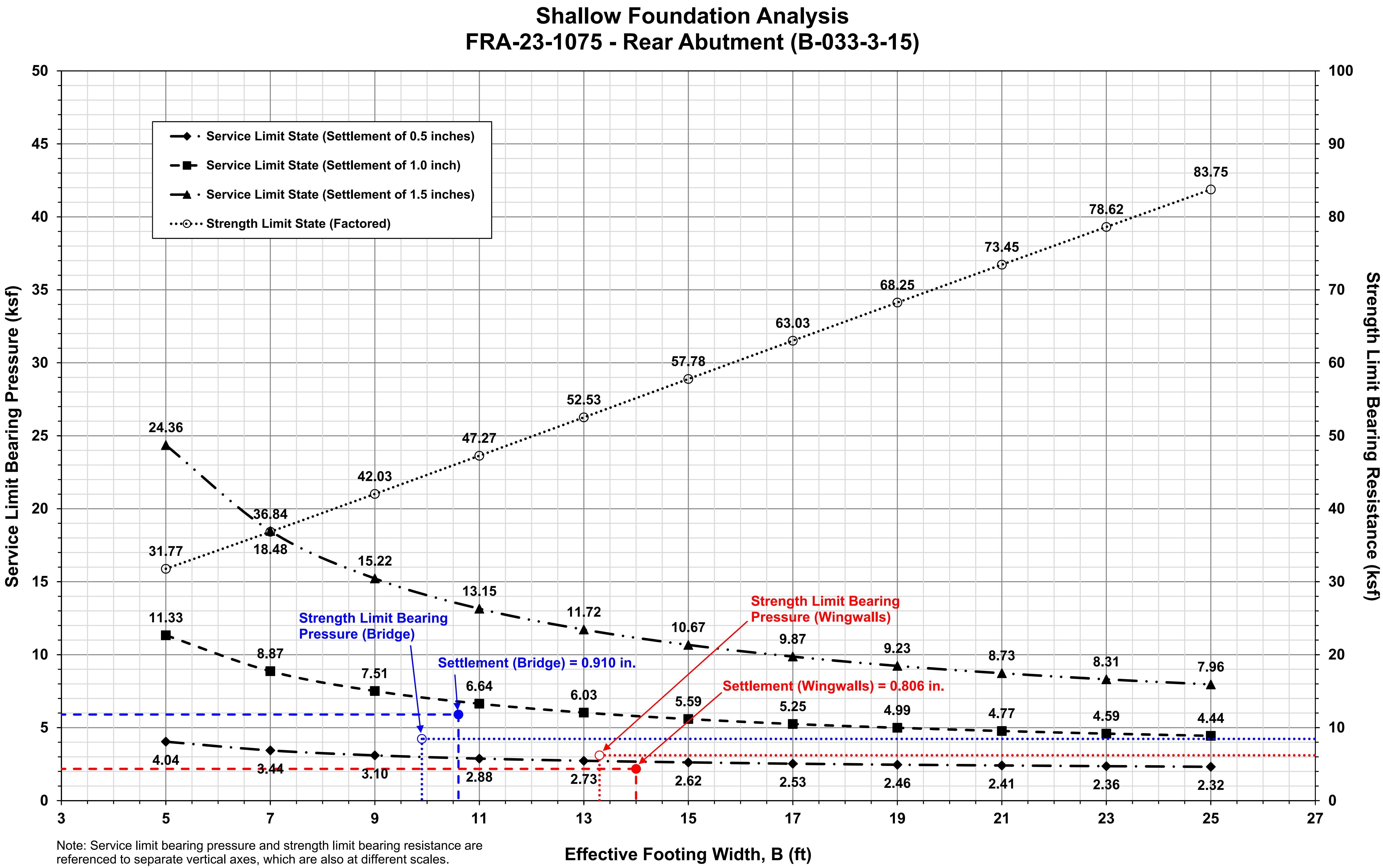
ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
	38	----	21460	Gray Gravel (Wash Sample)
711.2	40			
		44/78	21461	Gray Silty Sandy Gravel W/Stone Fragts.
	42			
		-----	21462	Gray Sand (Wash Sample)
	44			
706.2				
	46	50/*	21463	Gray Clayey Gravel
	48			
		----	21464	Gray Sand (Wash Sample)
701.2	50			
700.2		44/*	------	Gray Sandy Gravel
	52			BOTTOM OF BORING
	54			
	56			
	58			*Refusal
	60			
	62			
	64			
	66			
	68			
	70			
	72			
	74			
	76			
	78			
	80			
	82			

APPENDIX V

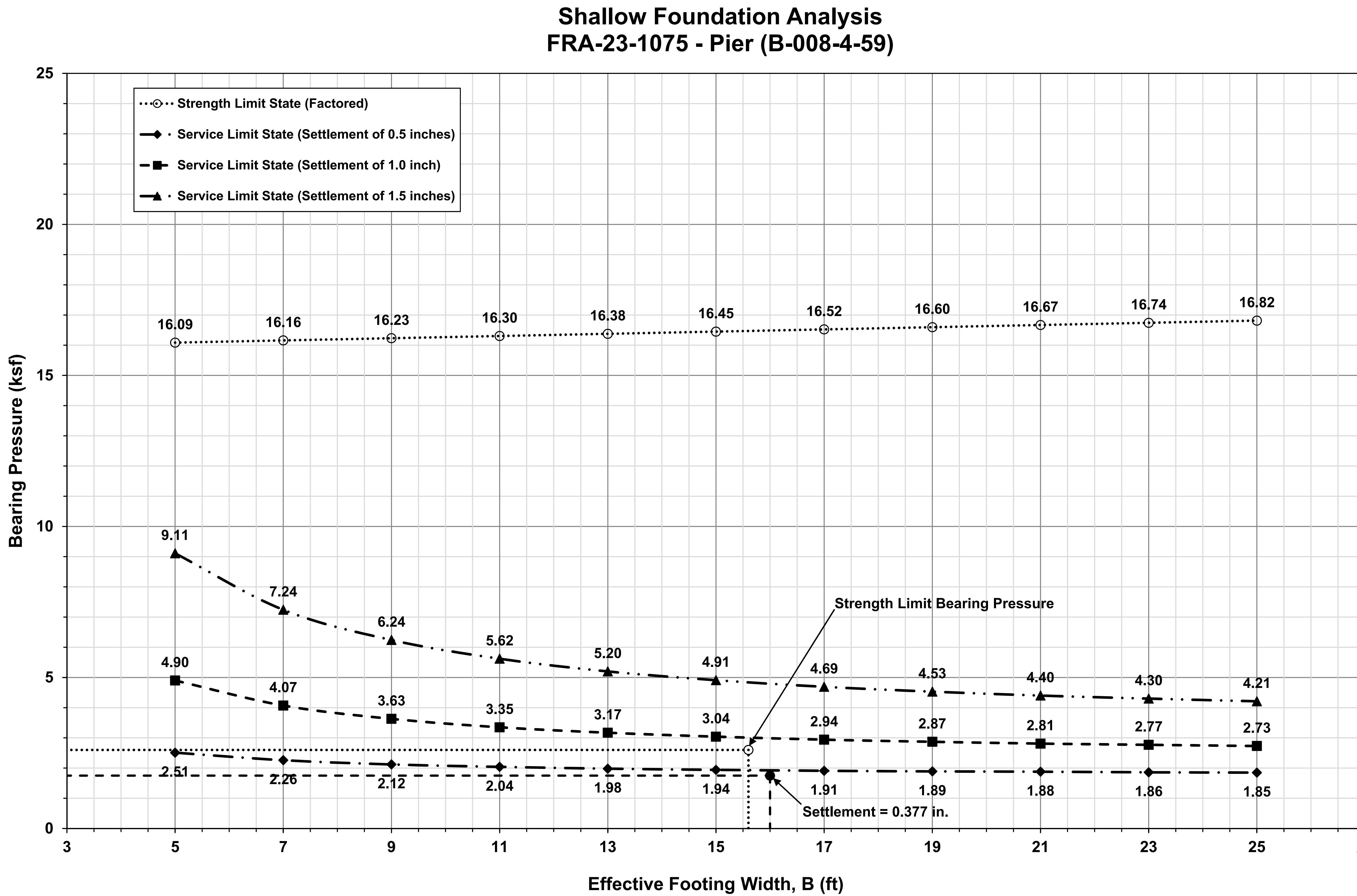
BEARING RESISTANCE CHARTS

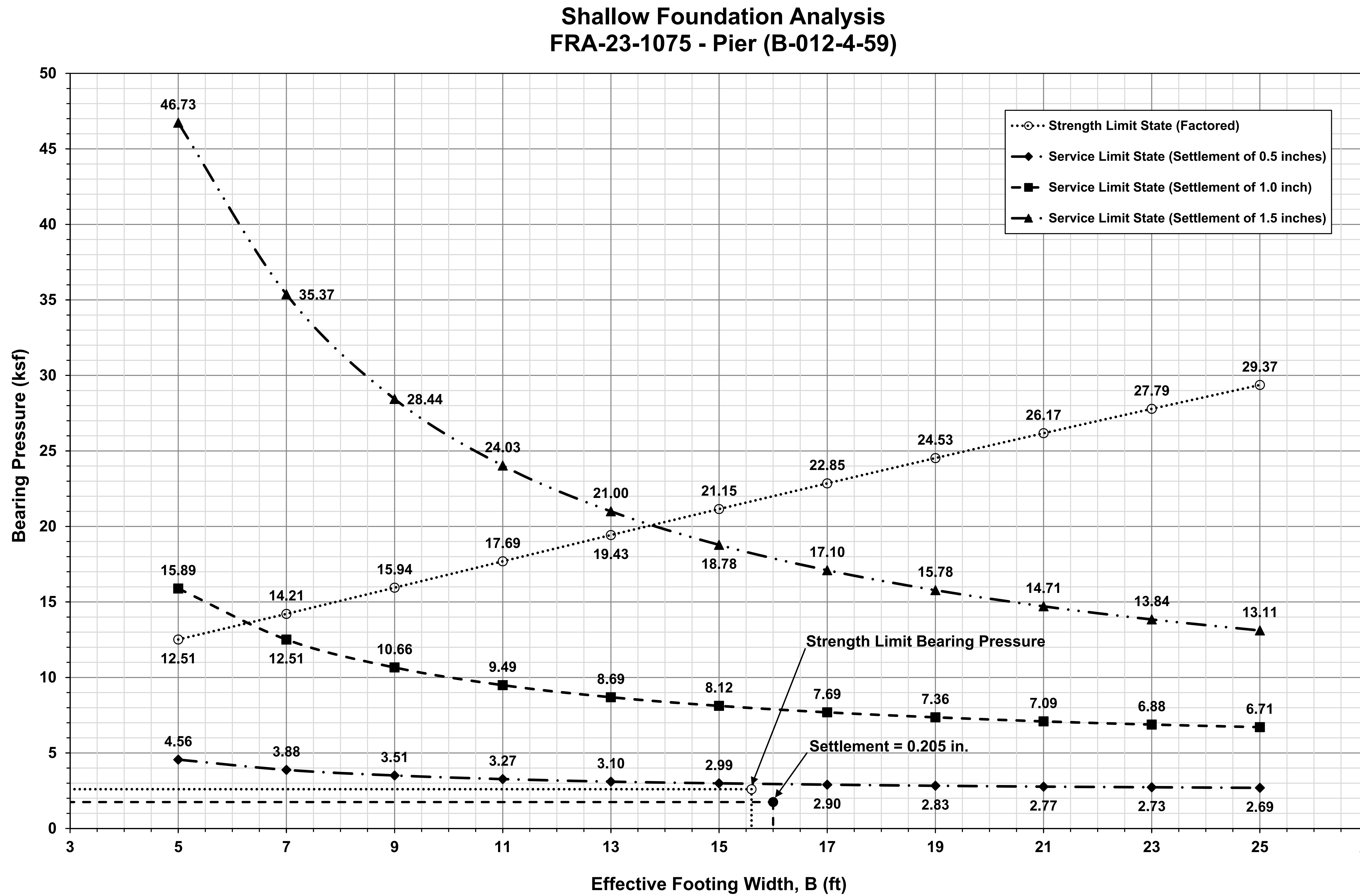


Shallow Foundation Analysis
FRA-23-1075 - Rear Abutment (B-001-4-59)
Strength Limit State (Factored)
Service Limit State (Settlement of 0.5 inches)
Service Limit State (Settlement of 1.0 inch)
Service Limit State (Settlement of 1.5 inches)
Strength Limit Bearing Pressure (Bridge)
Settlement (Bridge) = 1.104 in.
Strength Limit Bearing Pressure (Wingwalls)
Settlement (Wingwalls) = 0.979 in.
Bearing Pressure (ksf)
Effective Footing Width, B (ft)
15.12
12.60
12.63
12.65
12.67
12.70
12.72
12.74
12.76
12.79
12.81
12.83
11.64
9.78
8.64
7.89
7.37
6.99
6.71
6.49
6.31
6.17
8.03
6.42
5.54
4.99
4.62
4.35
4.15
4.00
3.88
3.79
3.71
3.23
2.80
2.57
2.42
2.32
2.25
2.19
2.15
2.12
2.09
2.07



Note: Service limit bearing pressure and strength limit bearing resistance are referenced to separate vertical axes, which are also at different scales.


Shallow Foundation Analysis
FRA-23-1075 - Pier (B-008-4-59)
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)
Bearing Pressure (ksf)
Effective Footing Width, B (ft)
0
5
10
15
20
25
3
5
7
9
11
13
15
17
19
21
23
25
27
16.09
16.16
16.23
16.30
16.38
16.45
16.52
16.60
16.67
16.74
16.82
9.11
7.24
6.24
5.62
5.20
4.91
4.69
4.53
4.40
4.30
4.21
4.90
4.07
3.63
3.35
3.17
3.04
2.94
2.87
2.81
2.77
2.73
2.51
2.26
2.12
2.04
1.98
1.94
1.91
1.89
1.88
1.86
1.85
Strength Limit Bearing Pressure
Settlement = 0.377 in.



Shallow Foundation Analysis
FRA-23-1075 - Pier (B-012-4-59)
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 = 0.205 in.
Bearing Pressure (ksf)
Effective Footing Width, B (ft)
46.73
35.37
28.44
24.03
21.00
18.78
17.10
15.78
14.71
13.84
13.11
12.51
14.21
15.94
17.69
19.43
21.15
22.85
24.53
26.17
27.79
29.37
15.89
12.51
10.66
9.49
8.69
8.12
7.69
7.36
7.09
6.88
6.71
4.56
3.88
3.51
3.27
3.10
2.99
2.90
2.83
2.77
2.73
2.69

APPENDIX VI

SHALLOW FOUNDATION CALCULATIONS

W-15-126 - FRA-70-14.05 Project 4B - FRA-23-1075 S. Fourth Street over I-70/71
Shallow Foundation Analysis - Rear Abutment - Bridge - Settlement

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

Boring B-001-4-59

B =	10.6	ft	Effective Footing width
D_w =	0.0	ft	Depth below bottom of footing
q =	5,900	psf	Service limit bearing pressure at bottom of wall
q_{net} =	4,588	psf	Net bearing pressure at bottom of footing (considers initial overburden stress of 1,312 psf from 10.5-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-3a	G	0.0	1.6	1.6	0.8	113	135	216	108	58							226	300	0.08	0.999	4,581	4,639	0.010	0.122
A-3a	G	1.6	3.2	1.6	2.4	113	135	432	324	174							205	300	0.23	0.969	4,443	4,617	0.008	0.091
A-1-a	G	3.2	5.7	2.5	4.5	115	135	770	601	323							185	300	0.42	0.869	3,986	4,309	0.009	0.113
A-1-a	G	5.7	8.2	2.5	7.0	115	135	1,107	938	505							168	300	0.66	0.722	3,311	3,816	0.007	0.088
A-4b	C	8.2	10.7	2.5	9.5	35	130	1,432	1,270	680	4,000	4,680	27	0.153	0.015	0.633			0.89	0.597	2,739	3,419	0.016	0.197
A-4b	C	10.7	13.2	2.5	12.0	35	135	1,770	1,601	855	4,000	4,855	27	0.153	0.015	0.633			1.13	0.502	2,302	3,157	0.013	0.159
A-6a	G	13.2	15.7	2.5	14.5	62	140	2,120	1,945	1,043							76	123	1.36	0.430	1,971	3,014	0.009	0.113
A-6a	G	15.7	18.2	2.5	17.0	62	140	2,470	2,295	1,237							72	118	1.60	0.374	1,717	2,954	0.008	0.097
A-4a	G	18.2	20.7	2.5	19.5	120	140	2,820	2,645	1,431							134	209	1.83	0.331	1,518	2,949	0.004	0.045
A-4a	G	20.7	23.2	2.5	22.0	120	140	3,170	2,995	1,625							129	202	2.07	0.296	1,358	2,983	0.003	0.039
A-4a	G	23.2	26.2	3.0	24.7	120	140	3,590	3,380	1,838							124	194	2.33	0.265	1,216	3,055	0.003	0.041
																					Total Settlement:			1.104 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-001-4-59

B =	9.9	ft	
L =	210	ft	
c =	4,375	psf	
γ =	130	pcf	
D_f =	5.0	ft	
φ =	0	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$$ = 23.02 ksf

$$N_{cm} = N_c s_c i_c$$ = 5.19

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

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

N_c =	5.14	s_c =	1+(9.9 ft/210 ft)(1/5.14) =	1.009	i_c =	1.000	d_q =	1+2tan(0°)[1-sin(0°)]²tan⁻¹(5 ft/9.9 ft) =	1.000
N_q =	1.00	s_q =	1+(9.9 ft/210 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(9.9 ft/210 ft) =	0.981	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(9.9 ft) + 5 ft =	0.500

$$q_R = q_n \cdot \phi_b$$ = 12.66 ksf

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

W-15-126 - FRA-70-14.05 Project 4B - FRA-23-1075 S. Fourth Street over I-70/71
Shallow Foundation Analysis - Rear Abutment - Wingwalls - Settlement

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

Boring B-001-4-59

B =	14.0	ft	Effective Footing width
D_w =	0.0	ft	Depth below bottom of footing
q =	4,350	psf	Service limit bearing pressure at bottom of wall
q_{net} =	3,038	psf	Net bearing pressure at bottom of footing (considers initial overburden stress of 1,312 psf from 10.5-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-3a	G	0.0	1.6	1.6	0.8	113	135	216	108	58							226	300	0.06	0.999	3,036	3,094	0.009	0.110
A-3a	G	1.6	3.2	1.6	2.4	113	135	432	324	174							205	300	0.17	0.985	2,992	3,166	0.007	0.081
A-1-a	G	3.2	5.7	2.5	4.5	115	135	770	601	323							185	300	0.32	0.928	2,818	3,141	0.008	0.099
A-1-a	G	5.7	8.2	2.5	7.0	115	135	1,107	938	505							168	300	0.50	0.821	2,493	2,997	0.006	0.077
A-4b	C	8.2	10.7	2.5	9.5	35	130	1,432	1,270	680	4,000	4,680	27	0.153	0.015	0.633			0.68	0.710	2,158	2,838	0.015	0.174
A-4b	C	10.7	13.2	2.5	12.0	35	135	1,770	1,601	855	4,000	4,855	27	0.153	0.015	0.633			0.85	0.615	1,868	2,723	0.012	0.141
A-6a	G	13.2	15.7	2.5	14.5	62	140	2,120	1,945	1,043							76	123	1.03	0.537	1,631	2,674	0.008	0.100
A-6a	G	15.7	18.2	2.5	17.0	62	140	2,470	2,295	1,237							72	118	1.21	0.474	1,440	2,676	0.007	0.086
A-4a	G	18.2	20.7	2.5	19.5	120	140	2,820	2,645	1,431							134	209	1.39	0.423	1,284	2,715	0.003	0.040
A-4a	G	20.7	23.2	2.5	22.0	120	140	3,170	2,995	1,625							129	202	1.57	0.381	1,157	2,782	0.003	0.035
A-4a	G	23.2	26.2	3.0	24.7	120	140	3,590	3,380	1,838							124	194	1.76	0.343	1,041	2,880	0.003	0.036
																					Total Settlement:			0.979 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-001-4-59

B =	13.3	ft	
L =	210	ft	
c =	4,375	psf	
γ =	130	pcf	
D_f =	5.0	ft	
φ =	0	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$$ = 23.09 ksf

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

N_c =	5.14	s_c =	1+(13.3 ft/210 ft)(1/5.14) =	1.012	i_c =	1.000	d_q =	1+2tan(0°)[1-sin(0°)]²tan⁻¹(5 ft/13.3 ft) =	1.000
N_q =	1.00	s_q =	1+(13.3 ft/210 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(13.3 ft/210 ft) =	0.975	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(13.3 ft) + 5 ft =	0.500

$q_R = q_n \cdot \phi_b$ = 12.70 ksf

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

W-15-126 - FRA-70-14.05 Project 4B - FRA-23-1075 S. Fourth Street over I-70/71 Calculated By: BRT Date: 6/27/2022

Shallow Foundation Analysis - Rear Abutment - Bridge - Settlement Checked By: JPS Date: 6/27/2022

Boring B-033-3-15

B =	10.6	ft	Effective Footing width
D_w =	0.0	ft	Depth below bottom of footing
q =	5,900	psf	Service limit bearing pressure at bottom of wall
q_{net} =	4,588	psf	Net bearing pressure at bottom of footing (considers initial overburden stress of 1,312 psf from 10.5-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-b	G	0.0	1.8	1.8	0.9	52	135	243	122	65							104	300	0.08	0.998	4,578	4,644	0.011	0.133
A-1-b	G	1.8	4.3	2.5	3.1	52	135	581	412	221							90	300	0.29	0.943	4,325	4,546	0.011	0.131
A-1-b	G	4.3	6.8	2.5	5.6	52	135	918	749	403							80	300	0.52	0.803	3,685	4,088	0.008	0.101
A-1-b	G	6.8	9.3	2.5	8.1	52	135	1,256	1,087	584							73	288	0.76	0.663	3,042	3,626	0.007	0.083
A-1-b	G	9.3	11.8	2.5	10.6	78	135	1,593	1,424	766							103	300	1.00	0.552	2,531	3,297	0.005	0.063
A-1-b	G	11.8	14.8	3.0	13.3	78	140	2,013	1,803	973							97	300	1.25	0.460	2,112	3,085	0.005	0.060
A-1-b	G	14.8	17.8	3.0	16.3	78	140	2,433	2,223	1,206							91	300	1.54	0.387	1,777	2,983	0.004	0.047
A-1-b	G	17.8	20.8	3.0	19.3	78	140	2,853	2,643	1,439							87	300	1.82	0.333	1,528	2,967	0.003	0.038
A-4b	C	20.8	23.3	2.5	22.1	107	140	3,203	3,028	1,652	4,000	5,652	19	0.081	0.008	0.500			2.08	0.295	1,353	3,005	0.004	0.042
A-4b	C	23.3	25.8	2.5	24.6	107	140	3,553	3,378	1,846	4,000	5,846	19	0.081	0.008	0.500			2.32	0.267	1,223	3,069	0.003	0.036
A-4b	C	25.8	28.3	2.5	27.1	107	140	3,903	3,728	2,040	4,000	6,040	19	0.081	0.008	0.500			2.55	0.243	1,116	3,156	0.003	0.031
A-1-b	G	28.3	33.3	5.0	30.8	117	140	4,603	4,253	2,331							111	300	2.91	0.215	986	3,317	0.003	0.031
A-1-b	G	33.3	38.3	5.0	35.8	117	140	5,303	4,953	2,719							105	300	3.38	0.186	852	3,571	0.002	0.024
A-1-b	G	38.3	43.3	5.0	40.8	117	140	6,003	5,653	3,107							100	300	3.85	0.164	750	3,857	0.002	0.019
A-1-b	G	43.3	48.3	5.0	45.8	117	140	6,703	6,353	3,495							95	300	4.32	0.146	670	4,165	0.001	0.015
A-6a	C	48.3	53.3	5.0	50.8	120	140	7,403	7,053	3,883	4,000	7,883	24	0.126	0.013	0.583			4.79	0.132	605	4,488	0.003	0.030
A-4a	G	53.3	58.3	5.0	55.8	113	140	8,103	7,753	4,271							85	136	5.26	0.120	551	4,823	0.002	0.023
A-1-b	G	58.3	60.3	2.0	59.3	120	140	8,383	8,243	4,543							87	300	5.59	0.113	519	5,062	0.000	0.004
																					Total Settlement:			0.910 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-033-3-15

B =	9.9	ft	
L =	210	ft	
c =	0	psf	
γ =	130	pcf	
D_f =	5.0	ft	
φ =	42	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma} = 80.69 \text{ ksf}$$

$$N_{cm} = N_c s_c i_c = 97.73 \qquad N_{qm} = N_q s_q d_q i_q = 97.20 \qquad N_{\gamma m} = N_\gamma s_\gamma i_\gamma = 152.61$$

N_c =	93.71	s_c =	1+(9.9 ft/210 ft)(85.37/93.71) =	1.043	i_c =	1.000	d_q =	1+2tan(42°)[1-sin(42°)]²tan⁻¹(5 ft/9.9 ft) =	1.092
N_q =	85.37	s_q =	1+(9.9 ft/210 ft)tan(42°) =	1.042	i_q =	1.000	C_{wq} =	0.0 ft < 5.0 ft =	0.500
N_γ =	155.54	s_γ =	1-0.4(9.9 ft/210 ft) =	0.981	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(9.9 ft) + 5 ft =	0.500

$$q_R = q_n \cdot \phi_b = 44.38 \text{ ksf}$$

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

W-15-126 - FRA-70-14.05 Project 4B - FRA-23-1075 S. Fourth Street over I-70/71
Shallow Foundation Analysis - Rear Abutment - Wingwalls - Settlement

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

Boring B-033-3-15

B =	14.0	ft	Effective Footing width
D_w =	0.0	ft	Depth below bottom of footing
q =	4,350	psf	Service limit bearing pressure at bottom of wall
q_{net} =	3,038	psf	Net bearing pressure at bottom of footing (considers initial overburden stress of 1,312 psf from 10.5-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-b	G	0.0	1.8	1.8	0.9	52	135	243	122	65							104	300	0.06	0.999	3,035	3,100	0.010	0.121
A-1-b	G	1.8	4.3	2.5	3.1	52	135	581	412	221							90	300	0.22	0.972	2,951	3,172	0.010	0.116
A-1-b	G	4.3	6.8	2.5	5.6	52	135	918	749	403							80	300	0.40	0.883	2,683	3,086	0.007	0.088
A-1-b	G	6.8	9.3	2.5	8.1	52	135	1,256	1,087	584							73	288	0.58	0.771	2,342	2,926	0.006	0.073
A-1-b	G	9.3	11.8	2.5	10.6	78	135	1,593	1,424	766							103	300	0.75	0.666	2,024	2,790	0.005	0.056
A-1-b	G	11.8	14.8	3.0	13.3	78	140	2,013	1,803	973							97	300	0.95	0.571	1,734	2,707	0.004	0.053
A-1-b	G	14.8	17.8	3.0	16.3	78	140	2,433	2,223	1,206							91	300	1.16	0.489	1,486	2,691	0.003	0.042
A-1-b	G	17.8	20.8	3.0	19.3	78	140	2,853	2,643	1,439							87	300	1.38	0.426	1,293	2,731	0.003	0.033
A-4b	C	20.8	23.3	2.5	22.1	107	140	3,203	3,028	1,652	4,000	5,652	19	0.081	0.008	0.500			1.58	0.379	1,152	2,804	0.003	0.037
A-4b	C	23.3	25.8	2.5	24.6	107	140	3,553	3,378	1,846	4,000	5,846	19	0.081	0.008	0.500			1.75	0.345	1,047	2,893	0.003	0.032
A-4b	C	25.8	28.3	2.5	27.1	107	140	3,903	3,728	2,040	4,000	6,040	19	0.081	0.008	0.500			1.93	0.316	959	2,999	0.002	0.027
A-1-b	G	28.3	33.3	5.0	30.8	117	140	4,603	4,253	2,331							111	300	2.20	0.280	850	3,181	0.002	0.027
A-1-b	G	33.3	38.3	5.0	35.8	117	140	5,303	4,953	2,719							105	300	2.56	0.243	738	3,457	0.002	0.021
A-1-b	G	38.3	43.3	5.0	40.8	117	140	6,003	5,653	3,107							100	300	2.91	0.214	651	3,758	0.001	0.017
A-1-b	G	43.3	48.3	5.0	45.8	117	140	6,703	6,353	3,495							95	300	3.27	0.192	582	4,077	0.001	0.013
A-6a	C	48.3	53.3	5.0	50.8	120	140	7,403	7,053	3,883	4,000	7,883	24	0.126	0.013	0.583			3.63	0.173	526	4,409	0.002	0.026
A-4a	G	53.3	58.3	5.0	55.8	113	140	8,103	7,753	4,271							85	136	3.99	0.158	480	4,751	0.002	0.020
A-1-b	G	58.3	60.3	2.0	59.3	120	140	8,383	8,243	4,543							87	300	4.24	0.149	452	4,995	0.000	0.003
																					Total Settlement:			0.806 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-033-3-15

B =	13.3	ft	
L =	210	ft	
c =	0	psf	
γ =	130	pcf	
D_f =	5.0	ft	
φ =	42	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$$ = 96.94 ksf

$$N_{cm} = N_c s_c i_c$$ = 99.12

$$N_{qm} = N_q s_q d_q i_q$$ = 96.64

$$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$$ = 151.60

N_c =	93.71	s_c =	1+(13.3 ft/210 ft)(85.37/93.71) =	1.058	i_c =	1.000	d_q =	1+2tan(42°)[1-sin(42°)]²tan⁻¹(5 ft/13.3 ft) =	1.071
N_q =	85.37	s_q =	1+(13.3 ft/210 ft)tan(42°) =	1.057	i_q =	1.000	C_{wq} =	0.0 ft < 5.0 ft =	0.500
N_γ =	155.54	s_γ =	1-0.4(13.3 ft/210 ft) =	0.975	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(13.3 ft) + 5 ft =	0.500

$$q_R = q_n \cdot \phi_b$$ = 53.31 ksf

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

W-15-126 - FRA-70-14.05 Project 4B - FRA-23-1075 S. Fourth Street over I-70/71 | Calculated By: BRT | Date: 6/27/2022

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

Boring B-008-4-59

B =	16.0	ft	Effective Footing width
D_w =	0.0	ft	Depth below bottom of footing
q =	1,750	psf	Service limit bearing pressure at bottom of wall
q_{net} =	375	psf	Net bearing pressure at bottom of footing (considers initial overburden stress of 1,375 psf from 11.0-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-1-b	G	0.0	1.5	1.5	0.8	97	135	203	101	54							194	300	0.05	1.000	375	429	0.004	0.054
A-1-b	G	1.5	3.0	1.5	2.3	97	135	405	304	163							178	300	0.14	0.991	372	535	0.003	0.031
A-1-b	G	3.0	5.2	2.2	4.1	97	135	702	554	298							159	300	0.26	0.957	359	656	0.003	0.030
A-6a	C	5.2	7.7	2.5	6.5	49	130	1,027	865	462	4,000	4,462	35	0.225	0.023	0.767			0.40	0.879	330	792	0.007	0.089
A-6a	C	7.7	10.2	2.5	9.0	49	130	1,352	1,190	631	4,000	4,631	35	0.225	0.023	0.767			0.56	0.781	293	924	0.005	0.063
A-6b	C	10.2	12.7	2.5	11.5	96	135	1,690	1,521	806	4,000	4,806	41	0.279	0.028	0.867			0.72	0.687	258	1,064	0.005	0.054
A-3	G	12.7	15.2	2.5	14.0	50	135	2,027	1,858	988							62	144	0.87	0.606	227	1,215	0.002	0.019
A-1-b	G	15.2	18.2	3.0	16.7	120	140	2,447	2,237	1,195							141	300	1.04	0.532	200	1,395	0.001	0.008
A-3	G	18.2	20.2	2.0	19.2	50	135	2,717	2,582	1,384							56	130	1.20	0.477	179	1,563	0.001	0.010
A-4a	G	20.2	23.2	3.0	21.7	120	140	3,137	2,927	1,573							130	204	1.36	0.431	162	1,735	0.001	0.008
A-3	G	23.2	25.2	2.0	24.2	50	140	3,417	3,277	1,767							52	121	1.51	0.393	147	1,914	0.001	0.007
A-4a	C	25.2	26.7	1.5	26.0	120	140	3,627	3,522	1,903	4,000	5,903	23	0.117	0.012	0.567			1.62	0.370	139	2,041	0.000	0.004
																					Total Settlement:			0.377 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-008-4-59

B =	15.6	ft	
L =	84	ft	
c =	6,125	psf	
γ =	130	pcf	
D_f =	5.0	ft	
φ =	0	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$$ = 32.95 ksf

$N_{cm} = N_c s_c i_c$ = 5.33

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

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

N_c =	5.14	s_c =	1+(15.6 ft/84 ft)(1/5.14) =	1.036	i_c =	1.000	d_q =	1+2tan(0°)[1-sin(0°)]²tan⁻¹(5 ft/15.6 ft) =	1.000
N_q =	1.00	s_q =	1+(15.6 ft/84 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(15.6 ft/84 ft) =	0.926	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(15.6 ft) + 5 ft =	0.500

$$q_R = q_n \cdot \phi_b$$ = 16.47 ksf

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

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

Boring B-012-4-59

B =	16.0	ft	Effective Footing width
D_w =	0.0	ft	Depth below bottom of footing
q =	1,750	psf	Service limit bearing pressure at bottom of wall
q_{net} =	375	psf	Net bearing pressure at bottom of footing (considers initial overburden stress of 1,375 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	2.0	2.0	1.0	73	135	270	135	73							146	300	0.06	0.999	375	447	0.005	0.063
A-1-b	G	2.0	4.0	2.0	3.0	73	135	540	405	218							127	300	0.19	0.981	368	586	0.003	0.034
A-1-b	G	4.0	6.0	2.0	5.0	73	135	810	675	363							115	300	0.31	0.931	349	712	0.002	0.023
A-1-b	G	6.0	8.4	2.4	7.2	73	135	1,134	972	523							106	300	0.45	0.850	319	841	0.002	0.020
A-1-a	G	8.4	10.9	2.5	9.7	120	135	1,472	1,303	701							162	300	0.60	0.753	283	983	0.001	0.015
A-1-a	G	10.9	13.4	2.5	12.2	120	140	1,822	1,647	888							153	300	0.76	0.663	249	1,137	0.001	0.011
A-3	G	13.4	15.4	2.0	14.4	50	135	2,092	1,957	1,058							61	141	0.90	0.593	222	1,280	0.001	0.014
A-2-6	G	15.4	18.4	3.0	16.9	120	140	2,512	2,302	1,247							139	300	1.06	0.528	198	1,445	0.001	0.008
A-3	G	18.4	20.9	2.5	19.7	50	135	2,849	2,680	1,454							55	128	1.23	0.468	176	1,630	0.001	0.012
A-1-b	G	20.9	24.4	3.5	22.7	120	140	3,339	3,094	1,681							127	300	1.42	0.416	156	1,837	0.000	0.005
																						Total Settlement:		0.205 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-012-4-59

B =	15.6	ft	
L =	84	ft	
c =	0	psf	
γ =	130	pcf	
D_f =	5.0	ft	
φ =	37	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$$ = 48.15 ksf

$$N_{cm} = N_c s_c i_c$$ = 63.60

$$N_{qm} = N_q s_q d_q i_q$$ = 52.55

$$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$$ = 61.27

N_c =	55.63	s_c =	1+(15.6 ft/84 ft)(42.92/55.63) =	1.143	i_c =	1.000	d_q =	1+2tan(37°)[1-sin(37°)]²tan⁻¹(5 ft/15.6 ft) =	1.074
N_q =	42.92	s_q =	1+(15.6 ft/84 ft)tan(37°) =	1.140	i_q =	1.000	C_{wq} =	0.0 ft < 5.0 ft =	0.500
N_γ =	66.19	s_γ =	1-0.4(15.6 ft/84 ft) =	0.926	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(15.6 ft) + 5 ft =	0.500

$$q_R = q_n \cdot \phi_b$$ = 21.67 ksf

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

APPENDIX VII

DRILLED SHAFT CALCULATIONS

FRA-23-1075 Bridge Replacement
Drilled Shaft Resistance Calculations

Boring	Proposed Top of Shaft Elevation (ft msl)	D_w (ft)	Shaft Diameter, D (ft)	Soil Class.	Material Type [1]	Stratum Depth, z (ft)	Stratum Thickness (ft)	Bottom Elevation (ft msl)	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]
				A-2-4	G	0.9	0.9	723.6	67	135	33	122				55	41	21,306	20.99		724.5-723.6	0.0-0.9	60	0.68	0.50	0.55
				A-1-b	G	10.9	10.0	713.6	73	135	428	1,472				56	42	23,214	4.31		723.6-713.6	0.9-10.9	60	1.84	0.50	0.55
				A-1-a	G	15.9	5.0	708.6	120	140	985	2,172				87	43	38,160	3.59		713.6-708.6	10.9-15.9	60	3.53	0.50	0.55
B-012-4-59	724.5	0.0	5.0	A-3	G	17.9	2.0	706.6	50	135	1,252	2,442				35	37	10,419	1.07	B-012-4-59	708.6-706.6	15.9-17.9	60	1.34	0.50	0.55
				A-2-6	G	20.9	3.0	703.6	120	140	1,441	2,862				83	41	38,160	2.57		706.6-703.6	17.9-20.9	60	3.69	0.50	0.55
				A-3	G	23.4	2.5	701.1	50	140	1,654	3,212				34	37	10,419	0.91		703.6-701.1	20.9-23.4	60	1.50	0.50	0.55
				A-1-b	G	24.9	1.5	699.6	120	140	1,810	3,422				80	42	38,160	2.29		701.1-699.6	23.4-24.9	60	4.14	0.50	0.55
				A-1-a	G	4.0	4.0	718.0	17	125	125	500				13	37	5,406	2.89		722.0-718.0	0.0-4.0	20	0.36	0.50	0.55
				A-1-b	G	9.0	5.0	713.0	49	130	419	1,150				37	40	15,582	3.06		718.0-713.0	4.0-9.0	58	1.28	0.50	0.55
				A-4b	G	11.5	2.5	710.5	80	135	679	1,488				58	38	33,182	3.29		713.0-710.5	9.0-11.5	60	2.23	0.50	0.55
				A-1-a	G	14.5	3.0	707.5	18	130	871	1,878				13	37	5,724	0.93		710.5-707.5	11.5-14.5	21	0.81	0.50	0.55
				A-6a	C	17.5	3.0	704.5	58	135	1,082	2,283	7,250	9.0	0.45						707.5-704.5	14.5-17.5	65	3.26	0.40	0.45
B-034-0-08	722.0	0.0	5.0	A-1-a	G	20.5	3.0	701.5	120	140	1,307	2,703				82	43	38,160	2.96	B-034-0-08	704.5-701.5	17.5-20.5	60	3.86	0.50	0.55
				A-4a	C	32.5	12.0	689.5	38	140	1,889	4,383	4,750	9.0	0.48						701.5-689.5	20.5-32.5	42	2.26	0.40	0.45
				A-1-a	G	37.5	5.0	684.5	120	140	2,549	5,083				73	43	38,160	1.88		689.5-684.5	32.5-37.5	60	4.78	0.50	0.55
				A-1-b	G	57.5	20.0	664.5	111	140	3,519	7,883				62	42	35,298	1.39		684.5-664.5	37.5-57.5	60	4.90	0.50	0.55
				A-4a	G	62.5	5.0	659.5	120	140	4,489	8,583				63	38	45,896	1.26		664.5-659.5	57.5-62.5	60	5.63	0.50	0.55
				A-1-b	G	85.5	23.0	636.5	107	140	5,575	11,803				51	42	34,026	1.00		659.5-636.5	62.5-85.5	60	5.57	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. $(N_1)_{60} = C_nN_{60}$, where $C_N = [0.77\log(40/\sigma_v')] \leq 2.0$ ksf, where σ_v' = vetical effective stress at midpoint of soil layer with respect to the entire soil profile for the respective boring; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS (granular soil layers)
7. 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. $\beta = \tan\varphi'_f(1-\sin\varphi'_f)(\sigma_p'/\sigma_v')\text{^}(\sin\varphi'_f)$, where σ_v' = vetical effective stress at midpoint of soil layer; Ref. Section 10.8.3.5.2b, AASHTO LRFD BDS (granular soil layers)
10. $q_p = N_CS_u \leq 80.0$ ksf; Ref. Section 10.8.3.5.1c, AASHTO LRFD BDS (cohesive soil layers)
11. $q_p = 1.2N_{60} \leq 60$ ksf; Ref. Section 10.8.3.5.2c, AASHTO LRFD BDS (granular soil layers)
12. $q_s = \alpha S_u$; Ref. Section 10.8.3.5.1b, AASHTO LRFD BDS (cohesive soil layers)
13. $q_s = \beta\sigma_v'$, where σ_v' = vetical effective stress at midpoint of soil layer; Ref. Section 10.8.3.5.2b, AASHTO LRFD BDS (granular soil layers)
14. φ_{qp} = 0.50 for granular soils layers and 0.40 for cohesive soil layers; Ref. Table 10.5.5.2.4-1, AASHTO LRFD BDS
15. φ_{qs} = 0.55 for granular soils layers and 0.45 for cohesive soil layers; Ref. Table 10.5.5.2.4-1, AASHTO LRFD BDS

FRA-23-1075 Bridge Replacement
Individual Drilled Shaft Resistance

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]
				A-1-a	G	4.0	4.0	718.0	17	125	125	500				13	37	5,406	2.89
				A-1-b	G	9.0	5.0	713.0	49	130	419	1,150				37	40	15,582	3.06
				A-4b	G	11.5	2.5	710.5	80	135	679	1,488				58	38	33,182	3.29
				A-1-a	G	14.5	3.0	707.5	18	130	871	1,878				13	37	5,724	0.93
				A-6a	C	17.5	3.0	704.5	58	135	1,082	2,283	7,250	9.0	0.45				
B-034-0-08	722.0	0.0	5.0	A-1-a	G	20.5	3.0	701.5	120	140	1,307	2,703				82	43	38,160	2.96
				A-4a	C	32.5	12.0	689.5	38	140	1,889	4,383	4,750	9.0	0.48				
				A-1-a	G	37.5	5.0	684.5	120	140	2,549	5,083				73	43	38,160	1.88
				A-1-b	G	57.5	20.0	664.5	111	140	3,519	7,883				62	42	35,298	1.39
				A-4a	G	62.5	5.0	659.5	120	140	4,489	8,583				63	38	45,896	1.26
				A-1-b	G	85.5	23.0	636.5	107	140	5,575	11,803				51	42	34,026	1.00

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]
	722.0-718.0	0.0-4.0	20	0.36	0.50	0.55
	718.0-713.0	4.0-9.0	58	1.28	0.50	0.55
	713.0-710.5	9.0-11.5	60	2.23	0.50	0.55
	710.5-707.5	11.5-14.5	21	0.81	0.50	0.55
	707.5-704.5	14.5-17.5	65	3.26	0.40	0.45
B-034-0-08	704.5-701.5	17.5-20.5	60	3.86	0.50	0.55
	701.5-689.5	20.5-32.5	42	2.26	0.40	0.45
	689.5-684.5	32.5-37.5	60	4.78	0.50	0.55
	684.5-664.5	37.5-57.5	60	4.90	0.50	0.55
	664.5-659.5	57.5-62.5	60	5.63	0.50	0.55
	659.5-636.5	62.5-85.5	60	5.57	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)
41.0	681.0			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. $N_C = 6[1+0.2(Z/D)] \leq 9$; 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. $(N_1)_{60} = C_nN_{60}$, where $C_N = [0.77\log(40/\sigma_v')] \leq 2.0$ ksf, where σ_v' = vetical effective stress at midpoint of soil layer with respect to the entire soil profile for the respective boring; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS (granular soil layers)
7. 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. $q_p = N_CS_u \leq 80.0$ 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

Boring B-034-0-08

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 =	215.0	ft	43 drilled shafts @ 5.0 ft diameter each bridge and adjacent wingwalls
c =	0	psf	
γ =	135	pcf	
D_f =	41.0	ft	
φ =	42	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma} = 330.31 \text{ ksf}$$

$N_{cm} = N_c s_c i_c =$ 95.26

$N_{qm} = N_q s_q d_q i_q =$ 112.01

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma =$ 154.41

N_c =	93.71	s_c =	1+(3.9 ft/215 ft)(85.37/93.71) =	1.017	i_c =	1.000	d_q =	1+2tan(42°)[1-sin(42°)]²tan⁻¹(41 ft/3.9 ft) =	1.291
N_q =	85.37	s_q =	1+(3.9 ft/215 ft)tan(42°) =	1.016	i_q =	1.000	C_{wq} =	0.0 ft < 41.0 ft =	0.500
N_γ =	155.54	s_γ =	1-0.4(3.9 ft/215 ft) =	0.993	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(3.9 ft) + 41 ft =	0.500

$q_R = q_n \cdot \phi_b =$ 165.16 ksf

$\varphi_b =$ 0.5

$\boldsymbol{R_R = q_R \cdot A_p} =$ 3,243 kips

APPENDIX VIII

LATERAL DESIGN PARAMETERS

FRA-23-1075 Bridge Replacement
Lateral Design Parameters

Boring No.	Elevation (feet msl)	Soil Class.	Soil Type	Strata	N_{60}	$N1_{60}$	γ (pcf)	γ' (pcf)	Strength Parameter	k (soil) k_{rm} (rock)	ε_{50} (soil) E_r (rock)	RQD (rock)
B-001-4-59	752.3 to 744.3	A-1-b	G	4	59	85	135	135	φ = 42°	355 pci	-	-
	744.3 to 740.3	A-2-4	G	4	60	68	135	135	φ = 41°	315 pci	-	-
	740.3 to 732.8	A-4a	C	3	63	63	135	135	Su = 7,875 psf	2,625 pci	0.0034	-
	732.8 to 730.3	A-1-a	G	4	82	73	140	140	φ = 43°	395 pci	-	-
	730.3 to 727.8	A-2-4	G	4	23	20	135	135	φ = 36°	160 pci	-	-
	727.8 to 725.3	A-1-a	G	4	103	84	140	140	φ = 43°	395 pci	-	-
	725.3 to 722.8	A-1-b	G	4	95	74	140	140	φ = 42°	355 pci	-	-
	722.8 to 719.3	A-3a	G	4	113	85	140	140	φ = 40°	280 pci	-	-
	719.3 to 714.3	A-1-a	G	4	115	83	140	78	φ = 43°	215 pci	-	-
	714.3 to 709.3	A-4b	C	2	35	35	140	78	Su = 4,375 psf	1,460 pci	0.0045	-
	709.3 to 704.3	A-6a	C	2	62	62	140	78	Su = 7,750 psf	2,585 pci	0.0034	-
	704.3 to 696.3	A-4a	G	4	120	77	140	78	φ = 38°	125 pci	-	-
B-008-4-59	751.3 to 743.3	A-1-b	G	4	7	10	120	120	φ = 34°	115 pci	-	-
	743.3 to 738.3	A-3a	G	4	42	48	130	130	φ = 39°	250 pci	-	-
	738.3 to 733.3	A-4a	C	3	56	56	135	135	Su = 7,000 psf	2,335 pci	0.0037	-
	733.3 to 726.8	A-3a	G	4	87	78	140	140	φ = 40°	280 pci	-	-
	726.8 to 716.8	A-1-b	G	4	97	76	140	140	φ = 42°	355 pci	-	-
	716.8 to 711.8	A-6a	C	2	49	49	140	78	Su = 6,125 psf	2,040 pci	0.0040	-
	711.8 to 709.3	A-6b	C	2	96	96	140	78	Su = 8,000 psf	2,665 pci	0.0033	-
	709.3 to 706.8	A-3	G	4	50	35	140	78	φ = 37°	110 pci	-	-
	706.8 to 703.8	A-1-b	G	4	120	82	140	78	φ = 42°	195 pci	-	-
	703.8 to 701.8	A-3	G	4	50	34	140	78	φ = 37°	110 pci	-	-
	701.8 to 698.8	A-4a	G	4	120	79	140	78	φ = 38°	125 pci	-	-
	698.8 to 696.8	A-3	G	4	50	32	140	78	φ = 36°	95 pci	-	-
	696.8 to 695.3	A-4a	C	2	120	120	140	78	Su = 8,000 psf	2,665 pci	0.0033	-
B-012-4-59	750.6 to 733.6	A-4a	C	3	39	39	130	130	Su = 4,875 psf	1,625 pci	0.0044	-
	733.6 to 731.1	A-1-b	G	4	103	97	140	140	φ = 42°	355 pci	-	-
	731.1 to 728.6	A-1-a	G	4	120	108	140	140	φ = 43°	395 pci	-	-
	728.6 to 726.1	A-3	G	4	75	64	140	140	φ = 39°	250 pci	-	-
	726.1 to 723.6	A-2-4	G	4	67	55	140	140	φ = 41°	315 pci	-	-
	723.6 to 713.6	A-1-b	G	4	73	56	140	78	φ = 42°	195 pci	-	-
	713.6 to 708.6	A-1-a	G	4	120	87	140	78	φ = 43°	215 pci	-	-
	708.6 to 706.6	A-3	G	4	50	35	140	78	φ = 37°	110 pci	-	-
	706.6 to 703.6	A-2-6	G	4	120	83	140	78	φ = 41°	175 pci	-	-
	703.6 to 701.1	A-3	G	4	50	34	140	78	φ = 37°	110 pci	-	-
	701.1 to 699.6	A-1-b	G	4	120	80	140	78	φ = 42°	195 pci	-	-
B-033-3-15	731.2 to 728.2	A-4a	C	3	24	24	125	125	Su = 3,000 psf	1,000 pci	0.0050	-
	728.2 to 713.2	A-1-b	G	4	52	58	135	135	φ = 42°	355 pci	-	-
	713.2 to 701.7	A-1-b	G	4	78	73	140	78	φ = 42°	195 pci	-	-
	701.7 to 694.2	A-4b	C	2	107	107	140	78	Su = 8,000 psf	2,665 pci	0.0033	-
	694.2 to 674.2	A-1-b	G	4	117	88	140	78	φ = 42°	195 pci	-	-
	674.2 to 669.2	A-6a	C	2	120	120	140	78	Su = 8,000 psf	2,665 pci	0.0033	-
	669.2 to 664.2	A-4a	G	4	113	74	140	78	φ = 38°	125 pci	-	-
	664.2 to 662.2	A-1-b	G	4	120	77	140	78	φ = 42°	195 pci	-	-
B-034-0-08	751.5 to 745.5	A-4a	C	3	17	17	125	125	Su = 2,125 psf	710 pci	0.0062	-
	745.5 to 740.5	A-1-b	G	4	8	10	120	120	φ = 34°	115 pci	-	-
	740.5 to 735.5	A-4a	C	3	41	41	135	135	Su = 5,125 psf	1,710 pci	0.0043	-
	735.5 to 733.0	A-1-b	G	4	6	6	125	125	φ = 33°	95 pci	-	-
	733.0 to 726.5	A-6a	C	3	54	54	140	140	Su = 6,750 psf	2,250 pci	0.0038	-
	726.5 to 718.0	A-1-a	G	4	17	13	135	73	φ = 37°	110 pci	-	-
	718.0 to 713.0	A-1-b	G	4	49	37	140	78	φ = 40°	155 pci	-	-
	713.0 to 710.5	A-4b	G	4	80	58	140	78	φ = 38°	125 pci	-	-
	710.5 to 707.5	A-1-a	G	4	18	13	135	73	φ = 37°	110 pci	-	-
	707.5 to 704.5	A-6a	C	2	58	58	140	78	Su = 7,250 psf	2,415 pci	0.0036	-
	704.5 to 701.5	A-1-a	G	4	120	82	140	78	φ = 43°	215 pci	-	-
	701.5 to 689.5	A-4a	C	2	38	38	140	78	Su = 4,750 psf	1,585 pci	0.0044	-
	689.5 to 684.5	A-1-a	G	4	120	73	140	78	φ = 43°	215 pci	-	-
	684.5 to 664.5	A-1-b	G	4	111	62	140	78	φ = 42°	195 pci	-	-
	664.5 to 659.5	A-4a	G	4	120	63	140	78	φ = 38°	125 pci	-	-
	659.5 to 636.5	A-1-b	G	4	107	51	140	78	φ = 42°	195 pci	-	-
	636.5 to 616.0	Shale	R	9	-	-	150	88	Qu = 4,000 psi	0.00025	250,000 psi	69