Resource International, Inc.

FRA-70-14.05 PROJECT 4B
RETAINING WALL 4W18
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
Retaining Wall 4W18
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 Retaining Wall 4W18 as part of the FRA-70-14.05 Project 4B in Columbus, Ohio.

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

Sincerely,

RESOURCE INTERNATIONAL, INC.

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

Jonathan P. Sterenberg, P.E.
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Enclosure: Structure Foundation Exploration Report

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EXECUTIVE SUMMARY

Resource International, Inc. (Rii) has completed a structure foundation exploration for the design and construction of the proposed Retaining Wall 4W18 located along the north side of I-70 westbound on the east side of the S. Fourth Street bridge (FRA-23-1075) over I-70/I-71. Based on design information provided by GPD GROUP, it is understood that the proposed structure will consist of a tangent drilled shaft retaining wall type, which will connect to the forward abutment of the proposed FRA-23-1075 bridge structure at Sta. 205+71.43 (BL I-70 WB) at the west end of the retaining wall alignment and extend east to connect with a retaining wall that is currently under construction as part of the FRA-70-14.48 project at Sta. 206+14.13. The wall height along the alignment of the wall ranges from 26.4 to 27.8 feet, as measured from the bottom of footing (bottom of wall facing panels) to the top of the wall, and the overall length of the wall is 49.8 feet.

Drilled Shaft Recommendations

While the design of a tangent shaft retaining wall is controlled by lateral check of the shaft elements, the drilled shafts may be designed using the axial design parameters provided in Table 3 in Section 5.1 of the full report. Based on the subsurface conditions encountered, the embedded sections of the shafts will bear in dense to very dense gravel and sand (ODOT A-1-b) with seams of hard sandy silt (ODOT A-4a). 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 5.1.1 of the full report. Lateral analysis of the shafts should be performed to determine the required embedment depth and cross section of the shafts as outlined in Section 5.1.2 of the full report.

Please note that this executive summary does not contain all the information presented in the report. The unabridged subsurface exploration report should be read in its entirety to obtain a more complete understanding of the information presented.

1.0 INTRODUCTION

The overall purpose of this project is to provide detailed subsurface information and recommendations for the design and construction of the FRA-70-14.05 Project 4B in Columbus, Ohio. The projects represent the central portion of FRA-70-8.93 (PID 77369) I-70/71 South Innerbelt improvements project. The FRA-70-14.05 Project 4B phase will consist of all work associated with the construction of the I-70/I-71 corridor from just east of S. High Street to just west of Grant Avenue, as well as a minimal amount of work Fulton Street and at the intersections of S. Third Street and S. Fourth Street with Livingston Avenue. This project includes the replacement of the FRA-33-1747 (S. Third Street) and FRA-23-1075 (S. Fourth Street) bridge structures over I-70/71, as well as the construction of three (3) new retaining walls along the north side and two (2) new retaining walls along the south side of I-70/71 to accommodate the new configuration.

This report is a presentation of the structure foundation exploration performed for the design and construction of the proposed Retaining Wall 4W18 located along the north side of I-70 westbound on the east side of the S. Fourth Street bridge (FRA-23-1075) over I-70/I-71, as shown on the vicinity map and boring plan presented in Appendix I. Based on design information provided by GPD GROUP, it is understood that the proposed structure will consist of a tangent drilled shaft retaining wall type, which will connect to the forward abutment of the proposed FRA-23-1075 bridge structure at Sta. 205+71.43 (BL I-70 WB) at the west end of the retaining wall alignment and extend east to connect with a retaining wall that is currently under construction as part of the FRA-70-14.48 project at Sta. 206+14.13. The wall height along the alignment of the wall ranges from 26.4 to 27.8 feet, as measured from the bottom of footing (bottom of wall facing panels) to the top of the wall, and the overall length of the wall is 49.8 feet.

2.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1 Site Geology

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

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

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

2.2 Existing Conditions

The proposed Retaining Wall 4W18 is located along the north side of I-70/I-71 on the east side of S. Fourth Street, 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 I-70 profile is lowered from the surrounding terrain, as the existing corridor was cut approximately 18 feet below the existing grade of S. Fourth Street as well as the surrounding downtown area. The proposed wall alignment is situated along the top of the grass covered graded slope that extends down from the south side of former exit ramp from I-70 westbound to S. Fourth Street. 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 surrounding terrain across the site is relatively flat-lying.

3.0 EXPLORATION

On August 2 and 3, 2011, one (1) boring, designated as B-035-1-10, was performed by DLZ at the location shown on the boring plan provided in Appendix I of this report and summarized in Table 1 below as part of the FRA-70-14.48 exploration. The report associated with the performance of this boring was not available at the time of this report. The boring was advanced to a completion depth of 75.0 feet below the existing ground surface on the east side of S. Fourth Street and within the limits of the proposed structure.

Table 1. Test Boring Summary

Boring Number	Reference Alignment	Station	Offset	Latitude	Longitude	Ground Elevation (feet msl)	Boring Depth (feet)
B-035-1-10	BL I-70 WB	206+09.36	52.28' LT.	39.953920	-82.994016	751.7	75.0

The boring was drilled using a truck-mounted rotary drilling machine, utilizing a 3.25-inch inside diameter, hollow-stem auger to advance the hole. Standard penetration test (SPT) and split spoon sampling was performed in the boring at 2.5-foot increments of depth to 50.0 feet and at 5.0-foot increments thereafter to the boring termination depth. An automatic drop hammer was utilized by DLZ to generate consistent energy transfer to the sampler. Driving resistance is recorded on the boring logs in terms of blow per 6.0-inch interval of the driving distance. The second and third intervals are added to obtain the number of blows per foot (N). Standard penetration blow counts aid in determining soil properties applicable in foundation system design. Measured blow count (N) values are corrected to an equivalent (60%) energy ratio, N_{60}, by the following equation. Both values are represented on boring logs in Appendix III.

$$N_{60} = N_m * (ER/60)$$

Where:
N_m = measured N value
ER = drill rod energy ratio, expressed as a percent, for the system used

The hammer for the CME 75 truck-mounted drill rig used by DLZ was calibrated on January 7, 2010, and has a drill rod energy ratio of 79.0 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.

Laboratory testing was performed by DLZ in order to classify existing soil according to the Ohio Department of Transportation (ODOT) classification system, which is utilized 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.

4.0 FINDINGS

An engineering log has been prepared by DLZ as part of the FRA-70-14.48 exploration. Classification follows the respective version of the ODOT Specifications for Geotechnical Explorations (SGE) at the time the exploration boring was performed. The following is a summary of what was found in the test boring and what is represented on the boring log.

4.1 Surface Materials

Boring B-035-1-10 was performed in the pavement of the former ramp from I-70 westbound to S. Fourth Street and encountered 5.0 inches of asphalt overlying 10.0 inches of concrete.

4.2 Subsurface Soils

Beneath the pavement in boring B-035-1-10, material identified as existing fill was encountered extending to a depth of 8.5 feet below exiting grade, which corresponds to an elevation of 743.2 feet msl. The fill material was described as very stiff to hard, brown silty clay (ODOT A-6b) and contained brick fragments throughout, as noted on the visual descriptions provided on the boring logs.

Underlying the existing fill, natural cohesive soils were encountered overlying granular soils with intermittent seams of cohesive material. The cohesive soils were described as hard, gray sandy silt (ODOT A-4a). The granular soils were described as dense to very dense, brown and gray gravel and sand (ODOT A-1-b).

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 stiff (2.0 < HP ≤ 4.0 tsf) to hard (HP > 4.0 tsf). The unconfined compressive strength of the cohesive soil samples tested, obtained from the hand penetrometer, ranged from 3.25 to over 4.5 tsf (limit of instrument). 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 dense ($31 \leq N_{60} \leq 50$ blows per foot [bpf]) to very dense ($N_{60} > 50$ bpf). Blow counts recorded from the SPT sampling within the granular soils ranged from 45 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.

Natural moisture contents of the cohesive soil samples ranged from 8 to 19 percent. The natural moisture content of the cohesive soil samples tested for plasticity index were 5 percent below their corresponding plastic limits. The cohesive soil exhibited natural moisture contents considered to be moderately below optimum moisture levels. Natural moisture contents of the granular soil samples ranged from 5 to 13 percent, which were visually described as damp to wet.

4.3 Bedrock

Bedrock was not encountered in boring B-035-1-10.

4.4 Groundwater

Groundwater was encountered in boring B-035-1-10 as presented in Table 2.

Table 2. Groundwater Levels

Boring Number	Ground Elevation (feet msl)	Initial Groundwater		Upon Completion	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-035-1-10	751.7	26.0	725.7	27.2 [1]	724.5

1. *Groundwater level at completion of drilling was likely influenced by the introduction of water during drilling.*

Groundwater was encountered initially during the drilling process in borings B-035-1-10 at a depth of 26.0 feet below the existing ground surface, which corresponds to an elevation of 725.7 feet msl. At the completion of drilling, groundwater was encountered at a depth of 27.2 feet below grade, which corresponds to an elevation of 724.5 feet msl. It should be noted that the groundwater level at completion of drilling was likely influenced by the introduction of water during drilling.

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.

5.0 ANALYSES AND RECOMMENDATIONS

Data obtained from the 2010 DLZ exploration has been used to determine the foundation support capabilities and the settlement potential for the soil encountered at the site. These parameters have been used to provide guidelines for the design of 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 proposed retaining wall were provided by GPD GROUP. Based on the information provided, it is understood that Retaining Wall 4W18 will be a tangent drilled shaft wall type. The roadway profile grade along the I-70 westbound will be cut approximately 20.5 feet from the existing ground surface grade to the proposed roadway profile grade, as measured from the top of the wall to the toe of the barrier at the facing of the wall. Based the top of wall elevations provided, it is anticipated that no cut or fill will be required behind the wall.

5.1 Drilled Shaft Recommendations

While the design of a tangent drilled shaft retaining wall is controlled by lateral check of the shaft elements, the drilled shafts may be designed using the axial design parameters provided in Table 3. 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 and sand (ODOT A-1-b) with seams of hard sandy silt (ODOT A-4a). The drilled shafts should be proportioned for a nominal bearing resistance as presented in Table 3.

Table 3. Retaining Wall 4W18 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-035-1-10	725.5-699.7	0.0-25.8	A-1-b	60	2.93	0.50	0.55
	699.7-694.7	25.8-30.8	A-4a	72	3.60	0.40	0.45
	694.7-689.7	30.8-35.8	A-1-b	54	2.27	0.50	0.55
	689.7-676.7	35.8-48.8	A-1-b	60	4.64	0.50	0.55

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

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

It is anticipated that 100 percent of the side friction resistance will be mobilized at a displacement of 1.0 percent of the diameter of the shaft, which is approximately 0.4 inches for a 3.5-foot diameter shaft. At this displacement, approximately 30 percent of the end bearing resistance will be mobilized. Therefore, the nominal end bearing resistance noted in Table 3 should be reduced to 30 percent of the values provided for the respective tip elevation in the determination of the design shaft resistance. Drilled shaft calculations are provided in Appendix IV.

5.1.1 Group Efficiency

The axial resistance of a group of shafts may be less than the sum of the individual shaft resistance within a group of shafts. Per Section 10.8.3.6.3 of the 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 AASHTO LRFD BDS. The following criteria are recommended for the group resistance of any shaft groups:

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

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

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

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

5.1.2 Lateral Design

Lateral load and/or moments are expected to be applied on the proposed tangent drilled shafts, they should be analyzed to verify the shaft has enough lateral and bending resistance against these loads. A tabulation of parameters, based on boring B-035-1-10, that should be used for lateral loading design is provided in Appendix V. In order to evaluate the lateral capacity, it is recommended that a derivation of COM624, such as LPILE, be utilized to determine the proper embedment depth and cross section required to resist the lateral load for a given end condition and deflection. Table 4 lists the eleven different soil types internal to the LPILE program. These strata were utilized to define the soil strata in the soil profile for each boring provided in Appendix V.

Table 4. Subsurface Strata Description

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

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

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

In which:

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

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

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

5.1.3 Drilled Shaft Axial Resistance

The nominal and factored drilled shaft axial resistance has been calculated for Retaining Wall 4W18, which is summarized in Table 5 below. The tip elevation for the 42-inch diameter shafts is 697.0 feet msl based on the plan information provided, and the 30-inch diameter shaft has a tip elevation of 722.0 feet msl. 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 42-inch diameter drilled shafts will end bear within a layer of hard sandy silt (ODOT A-4a), which has a calculated nominal end bearing resistance of 72 ksf, overlying medium dense gravel with sand (ODOT A-1-b), which has a calculated nominal end bearing resistance of 54 ksf. The 30-inch diameter shaft will end bear in 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 for the 42-inch diameter shafts since the drilled shafts will be constructed tangent to each other. Based on a shaft tip elevation of 697.0 feet msl, the shafts will be bearing in a layer of hard sandy silt (ODOT A-4a) overlying medium dense gravel with sand (ODOT A-1-b). Using the undrained shear strength for the sandy silt (ODOT A-4a), the resulting nominal unit bearing resistance is 43.6 ksf and the factored unit bearing resistance is 19.6 ksf, considering a resistance factor of 0.45.

Table 5. Retaining Wall 4W18 Drilled Shaft Recommendations

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	2.5	725.5	722.9	3.5	2.5	265	N/A	265	133	N/A	133
Traditional	3.5	725.5	697.0	28.5	3.5	536	N/A	536	214	N/A	214
Block	3.5	725.5	697.0	28.5	3.5	378	N/A	378	189	N/A	189

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 0.4 or 0.5 was utilized for the traditional drilled shaft analysis methodology and a resistance factor of 0.45 was utilized for the block failure mode.*

The controlling resistance between the traditional drilled shaft analysis methodology and block failure mode is 133 and 189 kips/shaft for the 30 and 42-inch diameter shafts, respectively. The maximum factored load per shaft is 31 and 63 kips/shaft for the 30 and 42-inch diameter shafts, respectively, based on the structural loading information provided by GPD GROUP. Calculations for the drilled shaft axial resistance are provided in Appendix IV.

5.1.4 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 boring B-035-1-10 and other borings performed within the corridor, cobbles and boulders were encountered throughout the very dense gravel with sand deposits. Therefore, difficult drilling conditions or obstructions should be anticipated to be encountered during installation of the drilled shafts. If boulders are encountered during installation of the drilled shafts, specialized drilling/coring equipment may be required to advance the drilled shaft excavation beyond the obstruction.

5.2 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 6 and Table 7.

Table 6. 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 7. 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.

5.3 Construction Considerations

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

5.3.1 Excavation Considerations

All excavations should be shored / braced or laid back at a safe angle in accordance to Occupational Safety and Health Administration (OSHA) guidelines. During excavation, if slopes cannot be laid back to OSHA Standards due to adjacent structures or other obstructions, 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 8. Excavation Back Slopes

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

5.3.2 Groundwater Considerations

Based on the groundwater observations made during drilling, groundwater is anticipated during construction of the drilled shafts. Where groundwater is encountered, proper groundwater control should be employed and maintained to prevent disturbance to excavation bottoms consisting of cohesive soil, and to prevent the possible development of a quick or "boiling" condition where soft silts and/or fine sands are encountered. It is preferable that the groundwater level, if encountered, be maintained at least 36 inches below the deepest excavation.

In the case of drilled shafts, the utilization of casing will be required below the water table to maintain an open hole and prevent the sidewalls from collapse. In addition, concrete placed below the water table should be placed by tremie method using a rigid tremie pipe. 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.

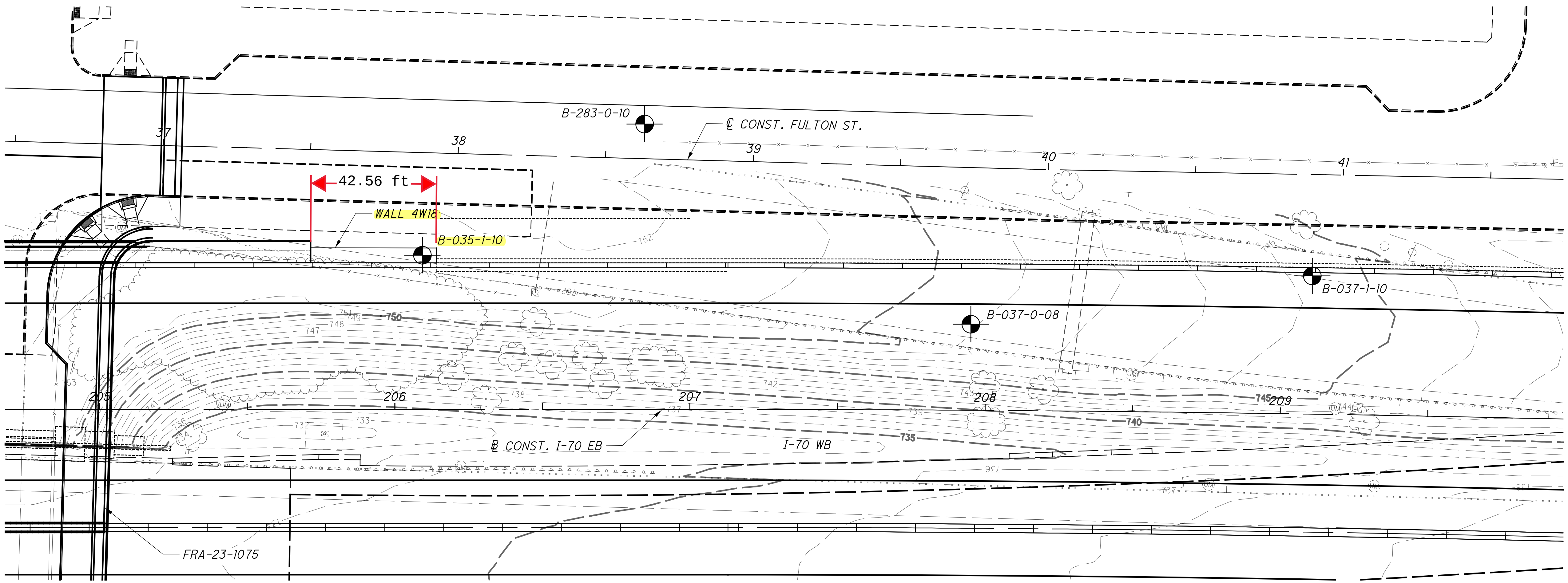
The conclusions and recommendations herein have been based upon the available soil and bedrock information and the design details furnished by a representative of the owner of the proposed project. Any revision in the plans for the proposed construction from those anticipated in this report should be 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 - WALL 4W18 1-70 WB

FRA-70-14.05

FRANKLIN COUNTY, OHIO

RII PROJECT NO. W-15-126	DRAWN JAS	
SCALE: 1"=20' 0 10 20	REVIEWED BRT	
N	DATE 11-27-18	

RESOURCE Rii INTERNATIONAL

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	
	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
- Peat

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

APPENDIX III

PROJECT BORING LOGS:

B-035-1-10

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 (%)

PROJECT: FRA-70-14.48	DRILLING FIRM / OPERATOR: DLZ / K. CONRAD	DRILL RIG: CME 75 TRUCK	STATION / OFFSET: 206+09 / 52.28' LT	EXPLORATION ID B-035-1-10
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: DLZ / M. EVENER	HAMMER: CME AUTOMATIC	ALIGNMENT: BL I-70 WB	
PID: 77370 BR ID: NA	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 1/7/10	ELEVATION: 751.7 (MSL) EOB: 75.0 ft.	PAGE 1 OF 3
START: 8/2/11 END: 8/3/11	SAMPLING METHOD: SPT	ENERGY RATIO (%): 79	COORD: 712045.650 N, 1830004.680 E	

MATERIAL DESCRIPTION AND NOTES	ELEV. 751.7	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
Asphalt - 5" Concrete - 10"	750.4	1																
FILL: Very stiff to hard brown SILTY CLAY (A-6b), little fine to coarse sand, trace gravel; damp.		2–3	4 5 6	14	56	SS-1	4.50	-	-	-	-	-	-	-	-	19	A-6b (V)	
@ 3.5'-7.5', contains occasional rust stains.		4–5	4 5 7	16	100	SS-2	4.5+	-	-	-	-	-	-	-	-	15	A-6b (V)	
@ 6.0'-7.5', contains occasional brick fragments.	743.2	6–8	7 7 9	21	17	SS-3	3.25	-	-	-	-	-	-	-	-	13	A-6b (V)	
Hard gray SANDY SILT (A-4a), some fine to coarse sand, trace to little gravel; damp.		9–10	13 15 20	46	83	SS-4	4.5+	12	11	20	33	24	24	15	9	10	A-4a (4)	
		11–13	10 12 13	33	100	SS-5	4.5+	-	-	-	-	-	-	-	-	10	A-4a (V)	
		14–15	6 11 12	30	100	SS-6	4.5+	-	-	-	-	-	-	-	-	10	A-4a (V)	
		16–18	8 14 15	38	100	SS-7	4.5+	9	12	21	36	22	22	14	8	9	A-4a (5)	
		19–20	10 24 26	66	83	SS-8	4.5+	-	-	-	-	-	-	-	-	8	A-4a (V)	
	728.2	21–23	8 13 19	42	100	SS-9	4.5+	-	-	-	-	-	-	-	-	9	A-4a (V)	
Very dense brown GRAVEL WITH SAND (A-1-b), trace to little silt; damp.	726.7	24–25	14 33 40	96	100	SS-10	-	44	29	17	- 10 -		NP	NP	NP	5	A-1-b (0)	
Very dense brown GRAVEL WITH SAND (A-1-b), trace to little silt; wet.		26–28 (W)	20 23 25	63	100	SS-11	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
		29 (W)	28 46 50/5"	-	100	SS-12	-	-	-	-	-	-	-	-	-	11	A-1-b (V)	

PID: 77370	BR ID:	PROJECT: FRA-70-14.48	STATION / OFFSET: 206+09 / 52.28 LT	START: 8/2/11	END: 8/3/11	PG 2 OF 3	B-035-1-10

MATERIAL DESCRIPTION AND NOTES	ELEV. 721.7	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
Very dense brown GRAVEL WITH SAND (A-1-b), trace to little silt; wet. *(continued)*		31 – 32	21 50/5"	-	100	SS-13	-	28	33	30	- 9 -		NP	NP	NP	13	A-1-b (0)	
		33 – 35	12 30 50/5"	-	100	SS-14	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
@ 36.0', 3.0 feet sand heave; washed out with tricone.		36 – 38	32 43 33	100	78	SS-15	-	-	-	-	-	-	-	-	-	11	A-1-b (V)	
		39 – 40	24 32 23	72	100	SS-16	-	29	41	17	- 13 -		NP	NP	NP	11	A-1-b (0)	
NOTE: Hole caved to 40.0' when augers removed.		41 – 43	17 28 28	74	56	SS-17	-	-	-	-	-	-	-	-	-	13	A-1-b (V)	
		44 – 45	19 34 31	86	78	SS-18	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
		46 – 48	25 35 31	87	89	SS-19	-	49	26	15	- 10 -		NP	NP	NP	10	A-1-b (0)	
@ 48.5', poor recovery; possible cobble/boulder.		49 – 51	50/5"	-	100	SS-20	-	-	-	-	-	-	-	-	-	7		
	699.7	52																
Hard gray SANDY SILT (A-4a), little fine to coarse sand, trace gravel; damp. @ 53.5', poor recovery; possible cobble/boulder.		53 – 56	50/3"	-	100	SS-21	4.5+	-	-	-	-	-	-	-	-	9	A-4a (V)	
	694.7	57																
Dense to very dense gray GRAVEL WITH SAND (A-1-b), trace silt; wet.		58 – 61	13 13 21	45	44	SS-22	-	41	33	19	- 7 -		NP	NP	NP	10	A-1-b (0)	

PID: 77370	BR ID:	PROJECT: FRA-70-14.48	STATION / OFFSET: 206+09 / 52.28 LT	START: 8/2/11	END: 8/3/11	PG 3 OF 3	B-035-1-10

MATERIAL DESCRIPTION AND NOTES	ELEV. 689.6	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
Dense to very dense gray GRAVEL WITH SAND (A-1-b), trace silt; wet. *(continued)*		63																
		64	34 50/5"	-	100	SS-23	-	-	-	-	-	-	-	-	-	11	A-1-b (V)	
		65 – 68																
		69 – 70	25 37 31	90	28	SS-24	-	-	-	-	-	-	-	-	-	11	A-1-b (V)	
		71 – 73																
	676.7	74 – 75	25 35 48	109	100	SS-25	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
		EOB 75																

NOTES: SEEPAGE AT 26.0 FEET AND 58.5 FEET; WATER LEVEL PRIOR TO ADDING WATER = 29.4 FEET; FINAL WATER LEVEL INCLUDING DRILLING WATER = 27.2 FEET.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: 1 BAG ASPHALT PATCH; 3 BAGS BENTONITE CHIPS; 1 BAG QUICKCRETE

APPENDIX IV

DRILLED SHAFT CALCULATIONS

Boring	Proposed Top of Shaft Elevation (ft msl)	D_w (ft)	Shaft Diameter, D (ft)	Soil Class.	Material Type [1]	Stratum Depth, z (ft)	Stratum Thickness (ft)	Bottom Elevation (ft msl)	N_{60} [2]	γ (pcf)	σ_v' (Midpoint) (psf)	σ_v (Bottom) (psf)	S_u [3] (psf)	N_c [4]	α [5]	$(N_1)_{60}$ [6]	φ'_f [7]	σ_p' [8] (psf)	β [9]
B-035-1-10	725.5	0.0	2.5	A-1-b	G	25.8	25.8	699.7	96	140	1,001	3,612				71	42	30,528	2.93
				A-4a	C	30.8	5.0	694.7	120	140	2,196	4,312	8,000	9.0	0.45				
				A-1-b	G	35.8	5.0	689.7	45	140	2,584	5,012				28	39	14,310	0.88
				A-1-b	G	48.8	13.0	676.7	106	140	3,282	6,832				63	42	33,708	1.42

Boring	Elevation (ft msl)	Shaft Length (ft)	Nominal Unit Tip Resistance, q_p [10,11] (ksf)	Nominal Unit Side Resistance, q_s [12,13] (ksf)	φ_{qp} [14]	φ_{qs} [15]
B-035-1-10	725.5-699.7	0.0-25.8	60	2.93	0.50	0.55
	699.7-694.7	25.8-30.8	72	3.60	0.40	0.45
	694.7-689.7	30.8-35.8	54	2.27	0.50	0.55
	689.7-676.7	35.8-48.8	60	4.64	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)
3.5	722.0	265		265	133		133

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. $S_u = 125(N_{60}) \leq 8,000$ psf (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	Proposed Top of Shaft Elevation (ft msl)	D_w (ft)	Shaft Diameter, D (ft)	Soil Class.	Material Type [1]	Stratum Depth, z (ft)	Stratum Thickness (ft)	Bottom Elevation (ft msl)	N_{60} [2]	γ (pcf)	σ_v' (Midpoint) (psf)	σ_v (Bottom) (psf)	S_u [3] (psf)	N_c [4]	α [5]	$(N_1)_{60}$ [6]	φ'_f [7]	σ_p' [8] (psf)	β [9]
B-035-1-10	725.5	0.0	3.5	A-1-b	G	25.8	25.8	699.7	96	140	1,001	3,612				71	42	30,528	2.93
				A-4a	C	30.8	5.0	694.7	120	140	2,196	4,312	8,000	9.0	0.45				
				A-1-b	G	35.8	5.0	689.7	45	140	2,584	5,012				28	39	14,310	0.88
				A-1-b	G	48.8	13.0	676.7	106	140	3,282	6,832				63	42	33,708	1.42

Boring	Elevation (ft msl)	Shaft Length (ft)	Nominal Unit Tip Resistance, q_p [10,11] (ksf)	Nominal Unit Side Resistance, q_s [12,13] (ksf)	φ_{qp} [14]	φ_{qs} [15]
B-035-1-10	725.5-699.7	0.0-25.8	60	2.93	0.50	0.55
	699.7-694.7	25.8-30.8	72	3.60	0.40	0.45
	694.7-689.7	30.8-35.8	54	2.27	0.50	0.55
	689.7-676.7	35.8-48.8	60	4.64	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)
28.5	697.0			536			214
		536			214		

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. $S_u = 125(N_{60}) \leq 8{,}000$ psf (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-035-1-10

D =	3.5	ft	Diameter of individual drilled shafts
B' =	2.8	ft	Equivalent footing width based on overall end bearing area of drilled shafts
L =	33.8	ft	
c =	8,000	psf	
γ =	130	pcf	
D_f =	28.5	ft	
φ =	0	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma} = 43.64 \text{ ksf}$$

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

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

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

N_c =	5.14	s_c =	1+(2.8 ft/33.8 ft)(1/5.14) =	1.016	i_c =	1.000	d_q =	1+2tan(0°)[1-sin(0°)]²tan⁻¹(28.5 ft/2.8 ft) =	1.000
N_q =	1.00	s_q =	1+(2.8 ft/33.8 ft)tan(0°) =	1.000	i_q =	1.000	C_{wq} =	0.0 ft < 28.5 ft =	0.500
N_γ =	0.00	s_γ =	1-0.4(2.8 ft/33.8 ft) =	0.967	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(2.8 ft) + 28.5 ft =	0.500

$q_R = q_n \cdot \phi_b$ = 19.64 ksf

φ_b = 0.45

$\boldsymbol{R_R = q_R \cdot A_p}$ = 189 kips

377.838

APPENDIX V

LATERAL DESIGN PARAMETERS

Boring No.	Elevation (feet msl)	Soil Class.	Soil Type	Strata	N_{60}	$N1_{60}$	γ (pcf)	γ' (pcf)	Strength Parameter	k (soil) k_{rm} (rock)	ε_{50} (soil) E_r (rock)	RQD (rock)
B-035-1-10	751.7 to 743.2	A-6b	C	3	17	17	125	125	Su = 2,125 psf	710 pci	0.0062	-
	743.2 to 728.2	A-4a	C	3	38	38	135	135	Su = 4,750 psf	1,585 pci	0.0044	-
	728.2 to 699.7	A-1-b	G	4	96	71	140	78	φ = 42°	195 pci	-	-
	699.7 to 694.7	A-4a	C	2	120	120	140	78	Su = 8,000 psf	2,665 pci	0.0033	-
	694.7 to 689.7	A-1-b	G	4	45	28	140	78	φ = 39°	140 pci	-	-
	689.7 to 676.7	A-1-b	G	4	106	63	140	78	φ = 42°	195 pci	-	-