Resource International, Inc.

FRA-70-12.68 PROJECT 4R
FRA-70-1357A – RAMP C5 OVER ELECTRICAL VAULT
PID NO. 105523
FRANKLIN COUNTY, OHIO

STRUCTURE FOUNDATION EXPLORATION REPORT

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

January 2019

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July 16, 2018 (Revised January 31, 2019)

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

Re: Structure Foundation Exploration Report
FRA-70-12.68 Project 4R
FRA-70-1357A – Ramp C5 over Electrical Vault
PID No. 105523
Rii Project No. W-13-045

Mr. Luzier:

Resource International, Inc. (Rii) is pleased to submit this structure foundation exploration report for the above referenced project. This report includes recommendations for the design and construction of the proposed FRA-70-1357A culvert structure carrying Ramp C5 over an electrical vault as part of the FRA-70-12.68 Project 4R in Columbus, Ohio.

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

Sincerely,

RESOURCE INTERNATIONAL, INC.

Hanumanth S. Kulkarni, Ph.D., P.E.
Project Engineer – Geotechnical Services

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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 FRA-70-1357A culvert structure carrying Ramp C5 over an electrical vault as part of the FRA-70-12.68 Project 4R. It is understood that the proposed structure will be constructed as a three-sided box culvert along the south side of Ramp C5 and will be located at approximately Sta. 5068+35. The proposed culvert will have an approximate height of 15 feet and width of 20 feet and will provide access to a buried electrical vault, which will remain following construction of Ramp C5.

Shallow Foundation Recommendations for Culvert

It is understood that the shallow spread foundations will be utilized for the proposed FRA-70-1357A culvert structure. The bearing soils are anticipated to consist a thin layer of medium stiff sandy silt (ODOT A-4a), extending to a depth of 2.5 feet below the proposed bearing elevation, overlying very dense gravel with sand (ODOT A-1-b). It is recommended to perform an undercut to remove the existing fill material and expose the competent underlying granular soils and replaced with ODOT Item 203 granular embankment. Shallow spread foundations bearing on these competent natural soils may be proportioned for a nominal bearing resistance as presented in Table 3 in Section 5.1 of the full report.

Based on the maximum service limit bearing pressures provided in the design documents and as noted in Section 5.0 a total settlement of 1.20 inches is anticipated under the proposed culvert. Additionally, the maximum factored bearing pressure will not exceed the factored bearing resistance at the strength limit.

Embankment Settlement

Settlement due to construction of new embankment fill was evaluated and a total settlement of approximately 2.12 inches should be expected in the existing natural soil. Approximately 1.75 inches of the settlement is anticipated within the medium dense to very dense gravel and sand and underlying very stiff to hard clay layers. Therefore, the effects of this magnitude of settlement on the existing electrical duct banks that connect to the existing vault structure should be evaluated. If needed, reinforcement of the duct banks may be required to withstand the additional loading and resist the deflections associated with settlement of the underlying soils.

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

1.0 INTRODUCTION

The overall purpose of this project is to provide detailed subsurface information and recommendations for the design and construction of the FRA-70-12.68/13.11/14.05C (Project 4R/4H/4A) projects in Columbus, Ohio. The projects represent the central portion of FRA-70-8.93 (PID 77369) I-70/71 south innerbelt improvements project. The FRA-70-12.68 (Project 4R) phase will consist of all work associated with the construction of Ramp C5, starting at the bridge over Souder Avenue and extending east to Front Street. The proposed Ramp C5 will be a two-lane to four-lane ramp that will collect and direct traffic from I-71 northbound and SR-315 southbound as well as I-70 eastbound to exit in downtown at the intersection of Front Street and W. Fulton Avenue. This project includes the construction of six (6) new bridge structures for the proposed Ramp C5 alignment and replacement of three (3) bridge structures, two along I-70 and the Front Street Structure over I-70, as well as the construction of fourteen (14) new retaining walls and a culvert structure to accommodate the new configuration.

This report is a presentation of the structure foundation exploration performed for the design and construction of the proposed FRA-70-1357A culvert structure carrying Ramp C5 over an electrical vault, as shown on the vicinity map and boring plan presented in Appendix I. It is understood that the proposed structure will be constructed as a three-sided box culvert along the south side of Ramp C5 and will be located at approximately Sta. 5068+35. The proposed culvert will have an approximate height of 15 feet and width of 20 feet and will provide access to a buried electrical vault, which will remain following construction of Ramp C5.

2.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1 Site Geology

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

According to the bedrock geology and topography maps obtained from the Ohio Department of Natural Resources (ODNR), the underlying bedrock consists predominantly of the Middle to Lower Devonian-aged Columbus Limestone. This formation is further subdivided into two members in the central portion of the state, known as the Delhi and Bellepoint Members. The Delhi Member consists of light gray, finely to coarsely crystalline, irregularly bedded, fossiliferous limestone. The Bellepoint Member consists of variable brown, finely crystalline, massively bedded limy dolomite. Both of these members contain chert nodules. Just east of the Scioto River, the underlying bedrock consists of the Upper Devonian Ohio Shale Formation overlying the Middle Devonian-aged Delaware Limestone Formation. The Ohio Shale formation consists of brownish black to greenish gray, thinly bedded, fissile, carbonaceous shale. The Delaware Limestone consists of bluish gray, thin to medium bedded dolomitic limestone with nodules and layers of chert. Regionally, the bedrock surface forms a broad valley aligned roughly north-to-south beneath the Scioto River. According to bedrock topography mapping, the elevation of the bedrock surface ranges from approximately 600 feet mean sea level (msl) in the valley to approximately 625 feet msl near the project limits. While bedrock was not noted as being encountered in boring B-017-1-09, based on the final sample obtained at a depth of 53.8 feet, this may be representative of the top of shale bedrock.

2.2 Existing Conditions

The proposed FRA-70-1537A culvert structure will be situated along the south side of proposed Ramp C5. The existing site is just east of an existing electrical substation and north of the Scioto Audubon Park. The project is located along the I-70/71 south innerbelt alignment, primarily along I-70 eastbound between Scioto River and CSX Railroad. The roadway is an eight-lane expressway in the area, which continues into downtown Columbus and crosses under Front Street and High Street. The existing I-70 is elevated from the surrounding terrain from east of the Scioto River to just west of Front Street and there are existing overpass bridges where the roadway crosses the existing CSX and Norfolk Southern Railroads and Short Street. The daily traffic volume along the project alignment is very high. The alignment traverses primarily commercial and government properties. The surrounding terrain across the site is relatively flat-lying, with general slope toward the Scioto River.

3.0 EXPLORATION

On September 23, 2009, one (1) boring, designated as B-017-1-09, was performed at the location shown on the boring plan provided in Appendix I of this report and summarized in Table 1 below by DLZ as part of the FRA-70-8.93 preliminary exploration and their findings were published in a report dated September 24, 2009. The boring was advanced to a completion depth of 53.8 feet below the existing ground surface, on the east side of the proposed structure. Additionally, between July 30 and August 2, 2013, boring B-017-3-13 was drilled by Rii to a depth of 87.0 feet below the existing grade on the northeast side 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-017-1-09	BL Ramp C5	5069+00.59	22.12' Rt.	39.952673635	-83.008577874	713.7	53.8
B-017-3-13	BL I-70 EB	166+20.53	31.80' Rt.	39.953028358	-83.008033736	740.3	87.0

The boring B-017-1-09 was drilled by DLZ using a CME 75 truck-mounted rotary drilling machine, utilizing a 3.25-inch inside diameter, hollow-stem auger to advance the holes. Boring B-017-3-13 was drilled by Rii using a Mobile B-53 truck-mounted rotary drilling machine, utilizing a 4.25-inch inside diameter, hollow-stem auger to advance the holes. Standard penetration testing (SPT) and split spoon sampling were performed in the borings at 2.5-foot increments of depth to 30.0 feet, and at 5.0-foot increments thereafter to the boring termination depth or top of bedrock. An automatic drop hammer was utilized 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 drill rig used by DLZ has a drill rod energy ratio of 62.0 percent. The hammer for the Mobile B-53 drill rig used by Rii has drill rod energy ratio of 77.7 percent.

Laboratory testing was performed by DLZ and Rii 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.

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.

The depth to bedrock in in boring B-017-3-13 was determined by split spoon sampler refusal. Split spoon sampler refusal is defined as exceeding 50 blows with less than 6 inches of penetration by the split spoon sampler. Where the borings were extended into the bedrock, an NQ-sized double-tube diamond bit core barrel (utilizing wire line equipment) was used to core the bedrock. Coring produced a 1.85-inch diameter core from which the type of rock and its geological characteristics were determined.

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

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

The RQD value aids in estimating the general quality of the rock and is used in conjunction with other parameters to designate the quality of the rock mass.

4.0 FINDINGS

Engineering logs have been prepared by DLZ as part of the preliminary engineering exploration and Rii as a part of the current exploration for the FRA-70-12.68 – Phase 4A project. 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 what is represented on the boring logs.

4.1 Surface Materials

Boring B-017-1-09 was drilled on the south of I-70/71 by toe of the existing I-70/71 eastbound embankment and encountered 8.0 inches of topsoil at the ground surface. Boring B-017-3-13 was drilled in the south shoulder of eastbound I-70 and encountered 6.0 inches of asphalt overlain by 4.0 inches of aggregate base.

4.2 Subsurface Soils

Beneath the topsoil, material identified as existing fill consisting of medium stiff, dark brown sandy silt (ODOT A-4a) was encountered extending to a depth of 6.0 feet below existing grade, which corresponds to an elevation of 707.7 feet msl, as shown in boring B-017-1-09. Beneath the existing pavement in boring B-017-3-13, material identified as existing fill consisting of stiff to very stiff brown clay (ODOT A-7-6) was encountered overlying medium dense black gravel and sand (ODOT A-1-b), extending to a depth of 32.0 feet below the existing grade, which corresponds to an elevation of 708.3 feet msl.

Underlying the surficial materials and existing fill, natural granular soils were encountered up to a depth of 32 feet below existing grade, which were generally described as dense to very dense, brown gravel and gravel with sand (ODOT A-1-a and A-1-b). Cohesive soils were encountered were generally described as stiff to hard, brown and gray silt and clay, silty clay and clay (ODOT A-6a, A-6b, A-7-6).

The relative density of granular soils is primarily derived from SPT blow counts (N_{60}). Based on the SPT blow counts obtained, the granular soil encountered ranged from medium dense ($11 \leq N_{60} < 30$ blows per foot [bpf]) to very dense ($N_{60} > 50$ bpf). Overall blow counts recorded from the SPT sampling ranged from 21 bpf to 55 bpf. The shear strength and consistency of the cohesive soils are primarily derived from the hand penetrometer values (HP). The cohesive soil encountered ranged from stiff ($1.0 \leq HP \leq 2.0$ tsf) to hard ($HP > 4.0$ tsf). The unconfined compressive strength of the cohesive soil samples tested, obtained from the hand penetrometer, ranged from 1.5 to over 4.5 tsf (limit of instrument).

Natural moisture contents of the soil samples tested ranged from 5 to 26 percent. The natural moisture content of the cohesive soil samples tested for plasticity index ranged from 5 percent below to at their corresponding plastic limits. In general, the soil exhibited natural moisture contents considered to be significantly below to near optimum moisture levels.

4.3 Bedrock

Bedrock was not noted to have been encountered in boring B-017-1-09. However, the boring was terminated at a depth of 53.8 feet, corresponding to an elevation of 659.9 feel msl, where split spoon sampler refusal was encountered. Based on the depth of bedrock encountered in other borings within the vicinity of this structure, this may represent the top of shale bedrock.

Boring B-017-3-13, performed in the northeast section of the proposed culvert, encountered split spoon sampler and auger refusal on the underlying highly weathered and fractured black and gray shale at a depth of 83.5 feet beneath the ground surface. Upon encountering the corable bedrock in boring B-017-3-13, a changeover to rock coring technique was made, and 3.5 feet of rock core was obtained. The shale encountered was described as slightly to highly weathered, very weak to slightly strong, thinly laminated to thin bedded, fissile, highly to moderately fractured and open aperture.

4.4 Groundwater

Groundwater was encountered in the borings as presented in Table 2.

Table 2. Groundwater Levels

Boring Number	Ground Surface Elevation (feet msl)	Initial Groundwater		Upon Completion	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-017-1-09	713.7	16.0	697.7	16.4 [1]	697.3
B-017-3-13	740.3	58.5	681.8	N/A [2]	--

1. *The groundwater level at completion measured includes drilling water as noted by DLZ due to advanced wash boring.*
2. *Groundwater at completion was not obtained due to introduction of water during the coring operation.*

Groundwater was encountered initially during the drilling process in borings B-017-1-09 and B-017-3-13 at a depth of 16.0 and 58.5 feet below the existing ground surface, which corresponds to an elevation of 697.7 and 681.8 feet msl, respectively. The groundwater level at the completion of drilling in boring B-017-1-09 was 16.4 feet below existing grade, which includes water that was added during the drilling process as noted by DLZ. The groundwater level at the completion of drilling in boring B-017-3-13 was not obtained due to the addition of water to the borehole during the coring operation.

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 drilling and testing programs have been used to determine the foundation support capabilities and the settlement potential for the soil encountered at the site. These parameters have been used to provide guidelines for the design of foundation systems for the subject three-sided box culvert, 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 culvert and embankment cross sections for Ramp C5 were provided by GPD GROUP. Based on plan information provided by GPD GROUP, the footings for the culvert structure have been designed to produce a maximum service limit bearing pressure of 5.0 ksf and a maximum factored bearing pressure of 8.7 ksf at the strength limit state. The culvert is proposed to be constructed as cast-in-place (CIP), and carrying embankment for the future Ramp C5 with the maximum embankment height of 28.5 feet from the existing ground surface.

5.1 Shallow Foundation Recommendations

It is understood that shallow spread foundations will be utilized for the three-sided box culvert (vault) structure. Based on plan information provided by GPD GROUP, the bottom of footing elevation of the substructure unit will bear at elevation of 710 feet msl, which is approximately 3.0 feet below existing grade. At this elevation, the bearing soils are anticipated to consist of a thin layer of existing fill comprised of soft sandy silt (ODOT A-4a), extending to a depth of 2.5 feet below the proposed bearing elevation, overlying very dense gravel with sand (ODOT A-1-b). It is recommended that the existing fill material be over excavated to expose the competent underlying granular soils and replaced with ODOT Item 203 granular embankment. Shallow spread foundations bearing on these competent natural soils may be proportioned for a nominal bearing resistance as presented in Table 3.

Table 3. FRA-70-1357A Spread Footing Design Parameters

Effective Footing Width (feet)	Service Limit Bearing Pressure (ksf) [1]			Nominal Bearing Resistance (ksf)	Factored Bearing Resistance [2] (ksf)
	1.0-inch	1.5-inch	2.0-inch		
3.0	5.74	11.24	18.44	34.36	15.46
4.0	4.44	8.61	14.07	38.83	17.48
5.0	3.65	7.03	11.45	43.26	19.47
6.0	3.12	5.98	9.70	47.61	21.42
7.0	2.74	5.23	8.45	51.86	23.34
8.0	2.46	4.66	7.51	55.99	25.19
9.0	2.24	4.22	6.79	59.99	26.99
10.0	2.06	3.87	6.20	63.85	28.73
11.0	1.91	3.58	5.73	67.58	30.41
12.0	1.79	3.34	5.33	71.16	32.02
13.0	1.69	3.14	5.00	74.60	33.57

1. *The service limit bearing pressure was calculated at total settlement values of 1.0, 1.5 and 2.0 inches.*
2. *A resistance factor of φ_b = 0.45 was utilized in calculating the factored bearing resistance at the strength limit state.*

The service limit bearing pressure that results in a maximum total settlement of 1.0, 1.5 and 2.0 inches was calculated and presented in Table 3. A geotechnical resistance factor of φ_b = 0.45 has been considered in calculating the factored bearing resistance at the strength limit state for the culvert and wing wall footings. Based on the bearing pressures provided in Table 3 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 1.0 and 1.5 inches of total settlement considered in the analysis. The bearing resistance will likely control the design for smaller footing widths (less than 4.0 feet) that are highly loaded. A graphical representation of the service limit bearing pressures and nominal and factored bearing resistance at the strength limit state for the culvert is presented in Appendix IV. Settlement calculations utilizing a footing width of 5.5 feet and an effective footing width of 4.95 feet (90% of the footing width), as shown on the plan sheets provided, as well as bearing resistance are included in Appendix V.

Based on the maximum service limit bearing pressures provided in the design documents and noted in Section 5.0, a total settlement of 1.20 inches is anticipated under the proposed culvert. Additionally, the maximum factored bearing pressure will not exceed the factored bearing resistance at the strength limit.

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 0.90 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.

5.2 Embankment Settlement Evaluation

In general, the soil profile in the area of the proposed embankment consists of natural granular soils to a depth of 32.0 feet below the existing ground surface overlain by cohesive material in the deeper layers. As noted in Section 4.2, the natural granular soils encountered were generally described as medium dense to very dense, brown gravel with sand (ODOT A-1-b). The cohesive soils encountered were generally described as stiff to hard, brown and gray silt and clay, silty clay and clay (ODOT A-6a, A-7-6). A settlement analysis of the proposed embankment fill was performed at a location to represent the maximum fill height at Sta. 5069+00, where boring B-017-1-09 was performed, to predict the long term consolidation settlement that will result after the embankment fill has been placed. Based on cross section information provided by GPD Group, the maximum anticipated fill of 28.5 feet is anticipated.

Results of the settlement analysis indicate that a total settlement of approximately 2.12 inches should be expected due to the weight of the new embankment fill. Approximately 1.75 inches of the settlement is anticipated within the medium dense to very dense gravel and sand and underlying very stiff to hard clay layers. Therefore, the effects of this magnitude of settlement on the existing electrical duct banks that connect to the existing vault structure should be evaluated. If needed, reinforcement of the duct banks may be required to withstand the additional loading and resist the deflections associated with settlement of the underlying soils. Results of the settlement analysis are provided in Appendix VI.

5.3 Embankment Slope Stability Evaluation

5.3.1 Strength Parameters

The shear strength parameters utilized in the slope stability analysis for the placement of the embankment fill to bring the site to the final grade are provided in Table 4.

Table 4. Shear Strength Parameters Utilized in Slope Stability Analysis

Material Type	γ (pcf)	φ' [1] (°)	c' [2] (psf)	S_u [3] (psf)
Item 203 Embankment Fill	120	32	0	2,000
Medium Dense to Very Dense Granular Soils (ODOT A-1-a, A-1-b)	130 to 140	39 to 42	0	N/A
Medium Stiff to Very Stiff Sandy Silt and Silt (ODOT A-4a)	125	26	0	1000
Very Stiff Silt and Clay (ODOT A-6a)	135	28	0	1500
Very Stiff to Hard Clay (ODOT A-7-6)	135 to 140	26 to 28	50	3500

1. *Per Figure 7-45, Section 7.6.9 of FHWA GEC 5 for cohesive soils, and per Table 10.4.6.2.4-1 of the 2017 AASHTO LRFD BDS for granular soils.*
2. *Estimated based on overconsolidated nature of soil.*
3. *S_u = 125(N_{60}).*

Shear strength parameters for new embankment fill were determined using ODOT GB-6 as a guide. It is understood that the proposed embankment will be constructed using Item 203 granular backfill material. The shear strength parameters for the natural soils were assigned using correlations provided in FHWA Geotechnical Engineering Circular (GEC) No. 5 (FHWA-NHI-16-072) Evaluation of Soil and Rock Properties, the 2017 AASHTO LRFD BDS and based on past experience in the vicinity of the site with projects performed in similar subsurface profiles.

5.3.2 Slope Stability Analysis

A slope stability analysis was performed to evaluate the stability of the proposed embankment slope. The critical cross section near the vicinity of the culvert was analyzed. The slope geometry was determined using proposed cross section information provided by GPD Group. Based on the information provided, the proposed embankment supporting Ramp C5 will be constructed using 2:1 side slope. For slopes not supporting a structural foundation, the minimum factor of safety against slope stability is 1.3. The resulting factor of safety under drained conditions was considered to be prevailing due to the site conditions. Based on the results of the analysis, the factor of safety of the proposed embankment slope under drained conditions is greater than 1.3. Results of the slope stability analysis for the cross-section analyzed is provided in Appendix VII.

5.4 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 5 and Table 6.

Table 5. Estimated Undrained (Short-term) Soil Parameters for Design

Soil Type	γ (pcf) [1]	c (psf)	φ	k_a	k_o	k_p
Very Stiff to Hard Cohesive Soil	125	3,000	0°	N/A	N/A	N/A
Medium Dense to Dense Granular Soil	130	0	32°	0.27	0.47	6.82
Very Dense Granular Soil	135	0	35°	0.24	0.43	8.56
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 6. Estimated Drained (Long-term) Soil Parameters for Design

Soil Type	γ (pcf) [1]	c (psf)	φ'	k_a	k_o	k_p
Very Stiff to Hard Cohesive Soil	120	0	28°	0.32	0.53	5.07
Medium Dense to Dense Granular Soil	130	0	32°	0.27	0.47	6.82
Very Dense Granular Soil	135	0	35°	0.24	0.43	8.56
Compacted Cohesive Engineered Fill	120	100	28°	0.32	0.53	5.07
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.

5.5 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).

5.5.1 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 7. 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

5.5.2 Groundwater Considerations

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

6.0 LIMITATIONS OF STUDY

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

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

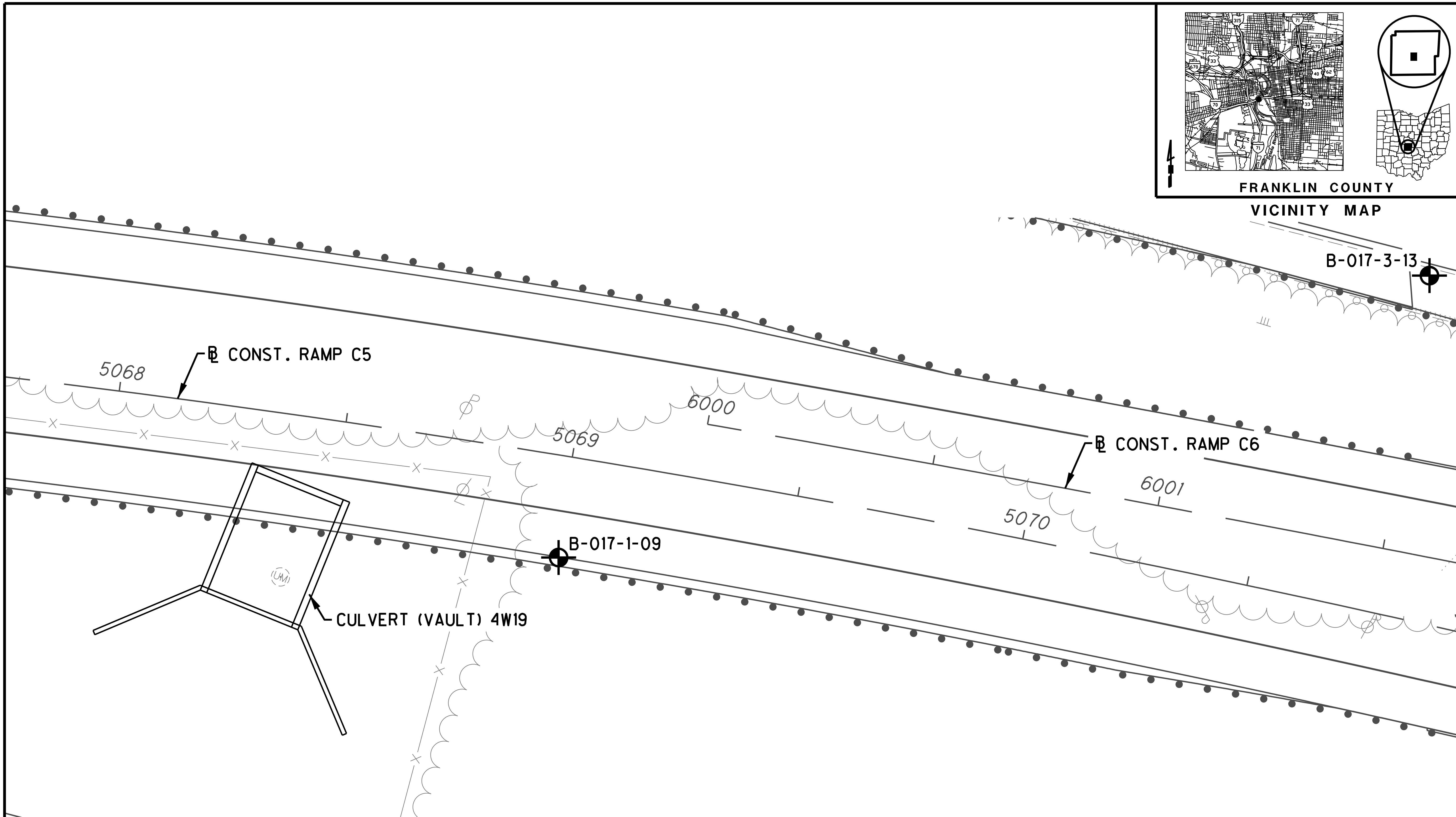
The conclusions and recommendations herein have been based upon the available soil and bedrock information and the design details furnished by a representative of the owner of the proposed project. Any revision in the plans for the proposed construction from those anticipated in this report should be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

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

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

APPENDIX I

VICINITY MAP AND BORING PLAN



BORING PLAN

FRA-70-12.68 CULVERT (VAULT) 4W19

FRANKLIN COUNTY, OHIO

RII PROJECT NO. W-13-045		DRAWN RRM
SCALE: 1"=20' (0 10 20)		REVIEWED BRT
		DATE 1-31-19

RESOURCE INTERNATIONAL, INC.

APPENDIX II

DESCRIPTION OF SOIL TERMS

DESCRIPTION OF SOIL TERMS

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

CLASSIFICATION OF SOILS

Ohio Department of Transportation

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

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

MATERIAL CLASSIFIED BY VISUAL INSPECTION

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

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

APPENDIX III

BORING LOG:

B-017-1-09 and B-017-3-13

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

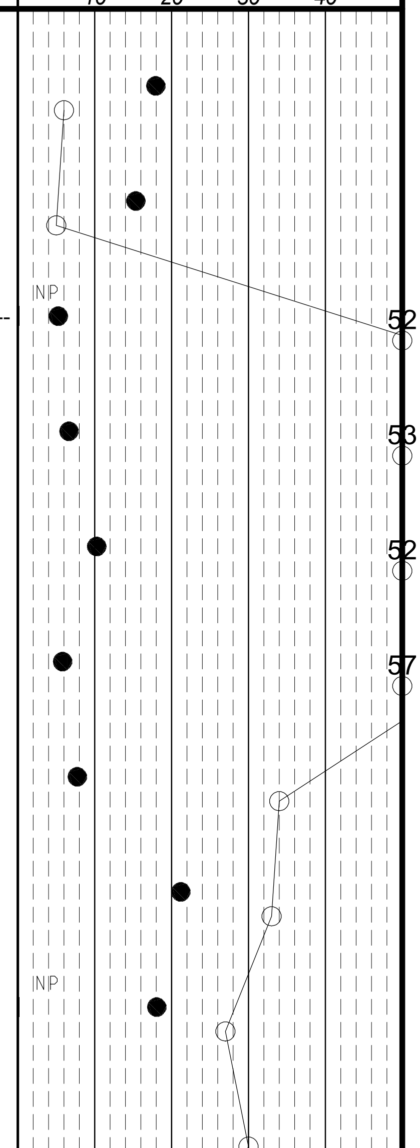
LOG OF: Boring B-017-1-09 | Location: Sta. 5069+00.59, 22.12' RT., BL RAMP C5 | Date Drilled: 9/23/2009

WATER OBSERVATIONS:
Water seepage at: 16.0'
Water level at completion: 18.5' (prior to adding water)
16.4' (includes drilling water)

FIELD NOTES:

Depth (ft)	Elev. (ft) 713.7	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	Graphic Log	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0.7	713.0						Topsoil - 8"							
		9 4 2	18	1		1.0	FILL: Medium stiff dark brown to brown SANDY SILT (A-4a), trace to little gravel; damp to moist.							
5		3 2 3	9	2		0.5								
6.0	707.7	7 27 23	13	3			Dense to very dense brown GRAVEL WITH SAND (A-1-b), trace to little silt; damp.		58	14	---	11	--17--	
10		16 27 24	14	4										
		7 27 23	13	5										
15		10 30 25	13	6										
		7 17 16	14	7			@ 16.0', becomes wet.							
20		9 16 16	4	8										
		2 8 18	12	9			@ 21.0'-22.5', medium dense.		33	45	---	15	--7--	
25	688.7	10 20 9	11	10										

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-017-1-09 | Location: Sta. 5069+00.59, 22.12' RT., BL RAMP C5 | Date Drilled: 9/23/2009

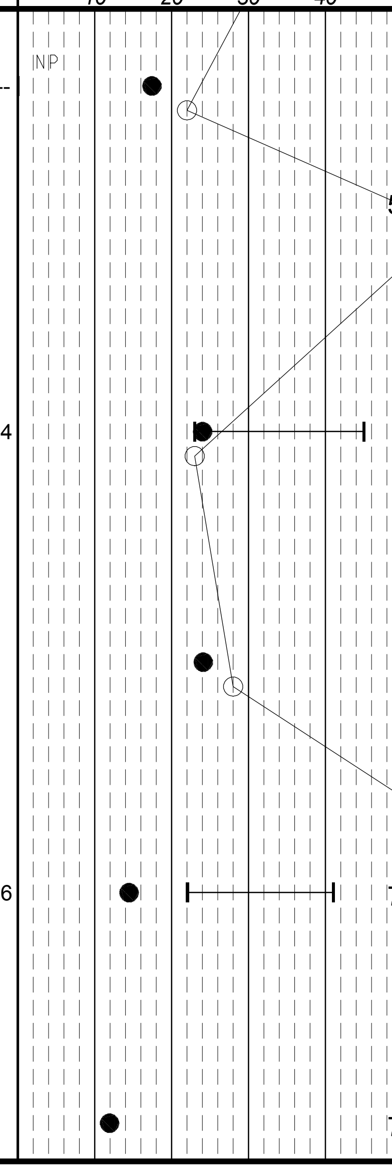
WATER OBSERVATIONS:
Water seepage at: 16.0'
Water level at completion: 18.5' (prior to adding water)
16.4' (includes drilling water)

FIELD NOTES:

Surface elevation: 688.7

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
							Medium dense brown GRAVEL WITH SAND (A-1-b), little silt; wet.						
		11 10 11	12	11				25	47	---	17	--11--	
30		8 14 40	9	12									
32.0	681.7						Very stiff gray CLAY (A-7-6), little silt, moist.						
35		4 9 13	18	13		2.5		0	0	---	0	16	84
37.0	676.7						Very stiff brown SILT AND CLAY (A-6a), little fine to coarse sand, trace to little gravel; moist.						
40		11 12 15	18	14		2.5							
42.0	671.7						Very stiff to hard gray CLAY (A-7-6), little gravel, trace fine to coarse sand; possible decomposed shale; damp.						
45		13 34 38	18	15		3.5		11	2	---	2	38	46
50	663.7	26 34 36	18	16		4.5+							

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-017-1-09	Location: Sta. 5069+00.59, 22.12' RT., BL RAMP C5	Date Drilled: 9/23/2009

WATER OBSERVATIONS:
Water seepage at: 16.0'
Water level at completion: 18.5' (prior to adding water)
16.4' (includes drilling water)
FIELD NOTES:

Depth (ft)	Elev. (ft) 663.7	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	Graphic Log	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N60) / Natural Moisture Content, % - ● / PL ⊢⊣ LL / Blows per foot - ○ / Non-Plastic - NP (10 20 30 40)
							Very stiff to hard gray CLAY (A-7-6), little gravel, trace fine to coarse sand; possible decomposed shale; damp.								
53.8	659.9	50/4	4	17		--	@ 53.5'; gray.								50+
							Bottom of Boring - 53.8'								

55

60

65

70

75

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-12.68 - PHASE 4A	DRILLING FIRM / OPERATOR: RII / J.B.	DRILL RIG: MOBILE B-53 (SN 624400)	STATION / OFFSET: 166+20.53 / 31.8' RT	EXPLORATION ID B-017-3-13
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / S.B.	HAMMER: AUTOMATIC	ALIGNMENT: BL I-70 EB	
PID: 77372 BR ID: FRA-70-1358R	DRILLING METHOD: 4.25" HSA / RC	CALIBRATION DATE: 4/26/13	ELEVATION: 740.3 (MSL) EOB: 87.0 ft.	PAGE 1 OF 3
START: 7/30/13 END: 8/2/13	SAMPLING METHOD: SPT / HQ	ENERGY RATIO (%): 77.7	LAT / LONG: 39.953028358, -83.008033736	

MATERIAL DESCRIPTION AND NOTES	ELEV. 740.3
0.5'- ASPHALT (6.0")	739.8
0.3'- AGGREGATE BASE (4.0")	739.5
FILL: STIFF TO VERY STIFF, BROWN **CLAY**, "AND" SILT, TRACE TO LITTLE FINE TO COARSE SAND, TRACE TO SOME FINE GRAVEL, DAMP TO MOIST.	713.3
FILL: MEDIUM DENSE, BLACK **GRAVEL AND SAND**, TRACE SILT, MOIST.	

DEPTHS: 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
4 4 10	18	78	SS-1	3.25	-	-	-	-	-	-	-	-	17	A-7-6 (V)	
5 2 4	8	50	SS-2	1.50	26	13	6	36	19	43	16	27	16	A-7-6 (11)	
2 3 5	10	33	SS-3	2.25	-	-	-	-	-	-	-	-	18	A-7-6 (V)	
3 4 5	12	44	SS-4	2.75	-	-	-	-	-	-	-	-	15	A-7-6 (V)	
2 3 3	8	50	SS-5	2.00	-	-	-	-	-	-	-	-	16	A-7-6 (V)	
1 3 3	8	56	SS-6	1.25	21	9	5	36	29	43	17	26	19	A-7-6 (13)	
3 4 5	12	17	SS-7	1.25	-	-	-	-	-	-	-	-	17	A-7-6 (V)	
4	-	67	3S-7A	1.25	-	-	-	-	-	-	-	-	16	A-7-6 (V)	
2 7 5	16	50	SS-8	3.00	5	3	3	35	54	50	18	32	22	A-7-6 (18)	
3 4 5	12	83	SS-9	3.00	-	-	-	-	-	-	-	-	24	A-7-6 (V)	
3 3 5	10	89	SS-10	2.00	-	-	-	-	-	-	-	-	26	A-7-6 (V)	
3 9 10	25	83	SS-11	3.75	-	-	-	-	-	-	-	-	21	A-7-6 (V)	
				-	-	-	-	-	-	-	-	-	16	A-1-b (V)	
3 5 9	18	33	SS-12	-	-	-	-	-	-	-	-	-	13	A-1-b (V)	

PID: 77372	BR ID: FRA-70-1358R	PROJECT: FRA-70-12.68 - PHASE 4A	STATION / OFFSET: 166+20.53 / 31.8 RT	START: 7/30/13	END: 8/2/13	PG 2 OF 3	B-017-3-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 710.3	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
FILL: MEDIUM DENSE, BLACK **GRAVEL AND SAND**, TRACE SILT, MOIST. *(same as above)*	708.3	31–32																
STIFF, BROWN **SILTY CLAY**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST.		34–35	1 4 7	14	67	SS-13	1.50	5	7	12	41	35	37	17	20	20	A-6b (12)	
	703.3	36–37																
DENSE TO VERY DENSE, BROWN **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, DAMP TO MOIST.		39–40	5 13 16	38	61	SS-14	-	-	-	-	-	-	-	-	-	7	A-1-b (V)	
		44–45	20 25 20	58	94	SS-15	-	59	16	8	14	3	NP	NP	NP	6	A-1-b (0)	
		W 48																
		49–50	13 26 26	67	56	SS-16	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
		54–55	30 25 28	69	78	SS-17	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	
	683.3	56–57																
VERY STIFF TO HARD, GRAY **SILT AND CLAY**, SOME COARSE TO FINE SAND, SOME FINE GRAVEL, DAMP.		W 58																
		59–60	15 22 20	54	67	SS-18	3.50	28	13	18	30	11	22	11	11	10	A-6a (1)	
		61																

PID: 77372	BR ID: FRA-70-1358R	PROJECT: FRA-70-12.68 - PHASE 4A	STATION / OFFSET: 166+20.53 / 31.8 RT	START: 7/30/13	END: 8/2/13	PG 3 OF 3	B-017-3-13

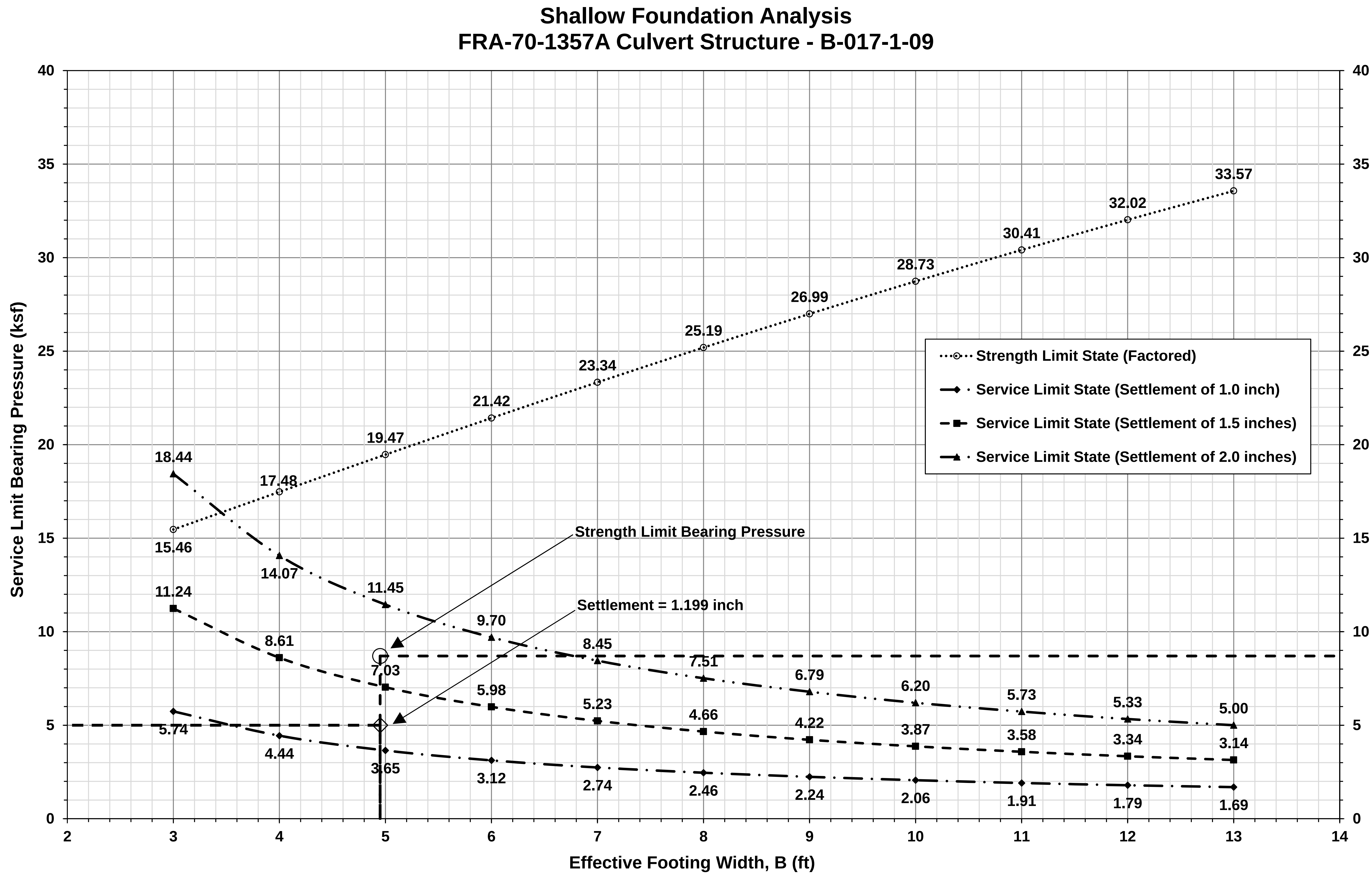
MATERIAL DESCRIPTION AND NOTES	ELEV. 678.2	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
VERY STIFF TO HARD, GRAY **SILT AND CLAY**, SOME COARSE TO FINE SAND, SOME FINE GRAVEL, DAMP. *(same as above)*		63																
		64 – 65	20 25 40	84	67	SS-19	4.50	-	-	-	-	-	-	-	-	9	A-6a (V)	
	673.3	66 – 67																
VERY DENSE, BROWN AND GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, MOIST.		68																
		69 – 70	3 30 36	85	67	SS-20	-	-	-	-	-	-	-	-	-	9	A-1-a (V)	
	668.3	71 – 72																
HARD, BROWN TO GRAY **CLAY**, TRACE COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP.		73																
		74 – 75	4 19 19	49	72	SS-21	4.50	-	-	-	-	-	-	-	-	17	A-7-6 (V)	
		76 – 78																
		79 – 80	10 24 40	83	72	SS-22	4.5+	8	2	3	41	46	41	19	22	10	A-7-6 (13)	
		81																
AUGER REFUSAL @ 83.5'	656.8	82 – 83 (TR)	4 20 50/5"	-	59	SS-23	4.5+	-	-	-	-	-	-	-	-	15	A-7-6 (V)	
SHALE : BLACK AND GRAY, SLIGHTLY TO HIGHLY WEATHERED, VERY WEAK TO SLIGHTLY STRONG, THINLY LAMINATED TO THIN BEDDED, FISSILE, HIGHLY TO MODERATELY FRACTURED, OPEN APERTURE, SLIGHTLY ROUGH; RQD 26%, REC 81%. -QU @ 86.0' = 222 PSI		84 – 85	0		56	RC-1											CORE	
	653.3	86 – 87 (EOB)	46		100	RC-2											CORE	

NOTES: SEEPAGE ENCOUNTERED @ 48.5'; GROUNDWATER ENCOUNTERED INITIALLY @ 58.5'

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

APPENDIX IV

BEARING RESISTANCE CHART


Shallow Foundation Analysis
FRA-70-1357A Culvert Structure - B-017-1-09
Service Limit Bearing Pressure (ksf)
Effective Footing Width, B (ft)
Strength Limit State (Factored)
Service Limit State (Settlement of 1.0 inch)
Service Limit State (Settlement of 1.5 inches)
Service Limit State (Settlement of 2.0 inches)
Strength Limit Bearing Pressure
Settlement = 1.199 inch
15.46
17.48
19.47
21.42
23.34
25.19
26.99
28.73
30.41
32.02
33.57
18.44
14.07
11.45
9.70
8.45
7.51
6.79
6.20
5.73
5.33
5.00
11.24
8.61
7.03
5.98
5.23
4.66
4.22
3.87
3.58
3.34
3.14
5.74
4.44
3.65
3.12
2.74
2.46
2.24
2.06
1.91
1.79
1.69

APPENDIX V

SHALLOW FOUNDATION CALCULATIONS FOR CULVERT

W-13-045 - FRA-70-1357A - Ramp C5 over Electical Vault
Shallow Foundation Analysis - Footing

Calculated By: HSK Date: 1/29/2019
Checked By: BRT Date: 1/31/2019

Boring B-017-1-09

B =	4.95	ft	Effective Footing width
D_w =	0.0	ft	Depth below bottom of footing
q =	5,000	psf	Service limit bearing pressure at bottom of wall

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	δ (7)	β (7)	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	2.5	2.5	1.3	30	125	313	156	78							60	215	0.25	-1.10	2.21	0.958	4,792	4,870	0.021	0.250
A-1-b	G	2.5	4.0	1.5	3.3	53	135	515	414	211							93	300	0.66	-0.65	1.30	0.721	3,606	3,817	0.006	0.075
A-1-b	G	4.0	6.5	2.5	5.3	53	135	853	684	356							84	300	1.06	-0.44	0.88	0.526	2,630	2,986	0.008	0.092
A-1-b	G	6.5	9.0	2.5	7.8	53	135	1,190	1,021	538							76	300	1.57	-0.31	0.62	0.381	1,906	2,444	0.005	0.066
A-1-b	G	9.0	11.5	2.5	10.3	29	125	1,503	1,346	707							39	128	2.07	-0.24	0.47	0.296	1,480	2,187	0.010	0.115
A-1-b	G	11.5	14.0	2.5	12.8	29	130	1,828	1,665	869							37	121	2.58	-0.19	0.38	0.241	1,206	2,075	0.008	0.094
A-1-b	G	14.0	16.5	2.5	15.3	29	130	2,153	1,990	1,038							35	115	3.08	-0.16	0.32	0.203	1,015	2,054	0.006	0.077
A-1-b	G	16.5	19.0	2.5	17.8	29	130	2,478	2,315	1,207							34	111	3.59	-0.14	0.28	0.175	876	2,084	0.005	0.064
A-1-b	G	19.0	21.5	2.5	20.3	29	130	2,803	2,640	1,376							33	107	4.09	-0.12	0.24	0.154	770	2,147	0.005	0.054
A-1-b	G	21.5	24.0	2.5	22.8	56	140	3,153	2,978	1,558							61	219	4.60	-0.11	0.22	0.137	687	2,245	0.002	0.022
A-1-b	G	24.0	26.0	2.0	25.0	56	140	3,433	3,293	1,733							59	209	5.05	-0.10	0.20	0.125	626	2,359	0.001	0.015
A-7-6	C	26.0	28.0	2.0	27.0	54	140	3,713	3,573	1,888	8,000	9,888	45	0.315	0.032	0.933			5.45	-0.09	0.18	0.116	580	2,468	0.004	0.046
A-7-6	C	28.0	31.0	3.0	29.5	22	135	4,118	3,915	2,074	8,000	10,074	45	0.315	0.032	0.933			5.96	-0.08	0.17	0.106	532	2,606	0.005	0.058
A-6a	C	31.0	36.0	5.0	33.5	27	135	4,793	4,455	2,365	8,000	10,365	35	0.225	0.023	0.767			6.77	-0.07	0.15	0.094	469	2,833	0.005	0.060
A-7-6	C	36.0	39.0	3.0	37.5	72	140	5,213	5,003	2,663	8,000	10,663	41	0.279	0.028	0.867			7.58	-0.07	0.13	0.084	419	3,081	0.003	0.034
A-7-6	C	39.0	44.0	5.0	41.5	70	140	5,913	5,563	2,973	8,000	10,973	41	0.279	0.028	0.867			8.38	-0.06	0.12	0.076	379	3,352	0.004	0.047
A-7-6	C	44.0	47.8	3.8	45.9	150	140	6,445	6,179	3,314	8,000	11,314	41	0.279	0.028	0.867			9.27	-0.05	0.11	0.069	343	3,657	0.002	0.029
																								Total Settlement:		1.199 in

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

W-13-045 - FRA-70-1357A - Ramp C5 over Electical Vault
Shallow Foundations - Strength Limit State

Calculated By: HSK   Date: 7/16/2018
Checked By: JPS   Date: 7/16/2018

Boring B-017-1-09

B =	4.95	ft	
L =	25	ft	
c =	0	psf	
γ =	125	pcf	
D_f =	3.0	ft	
φ =	42	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$$ = 43.04 ksf

$$N_{cm} = N_c s_c i_c$$ = 110.61

$$N_{qm} = N_q s_q d_q i_q$$ = 111.39

$$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$$ = 143.22

N_c =	93.71	s_c =	1+(4.95 ft/25 ft)(85.37/93.71) =	1.180	i_c =	1.000	d_q =	1+2tan(42°)[1-sin(42°)]²tan⁻¹(3 ft/4.95 ft) =	1.107
N_q =	85.37	s_q =	1+(4.95 ft/25 ft)tan(42°) =	1.178	i_q =	1.000	C_{wq} =	0.0 ft < 3.0 ft =	0.500
N_γ =	155.54	s_γ =	1-0.4(4.95 ft/25 ft) =	0.921	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(4.95 ft) + 3 ft =	0.500

$$q_R = q_n \cdot \phi_b$$ = 19.37 ksf

φ_b = 0.45

APPENDIX VI

EMBANKMENT SETTLEMENT CALCULATIONS

W-13-045 - FRA-70-1357A - Ramp C5 over Electical Vault
Shallow Foundation Analysis - Settlement

Calculated By: HSK Date: 1/29/2019
Checked By: BRT Date: 1/31/2019

Boring B-017-1-09

B =	30.0	ft	Effective Footing width = Considered as the maximum height of the embankment fill
D_w =	0.0	ft	Depth below bottom of footing
q =	3,600	psf	Service limit bearing pressure at foundation level of the embankment
q_{net} =	3,600	psf	Net bearing pressure at bottom of embankment

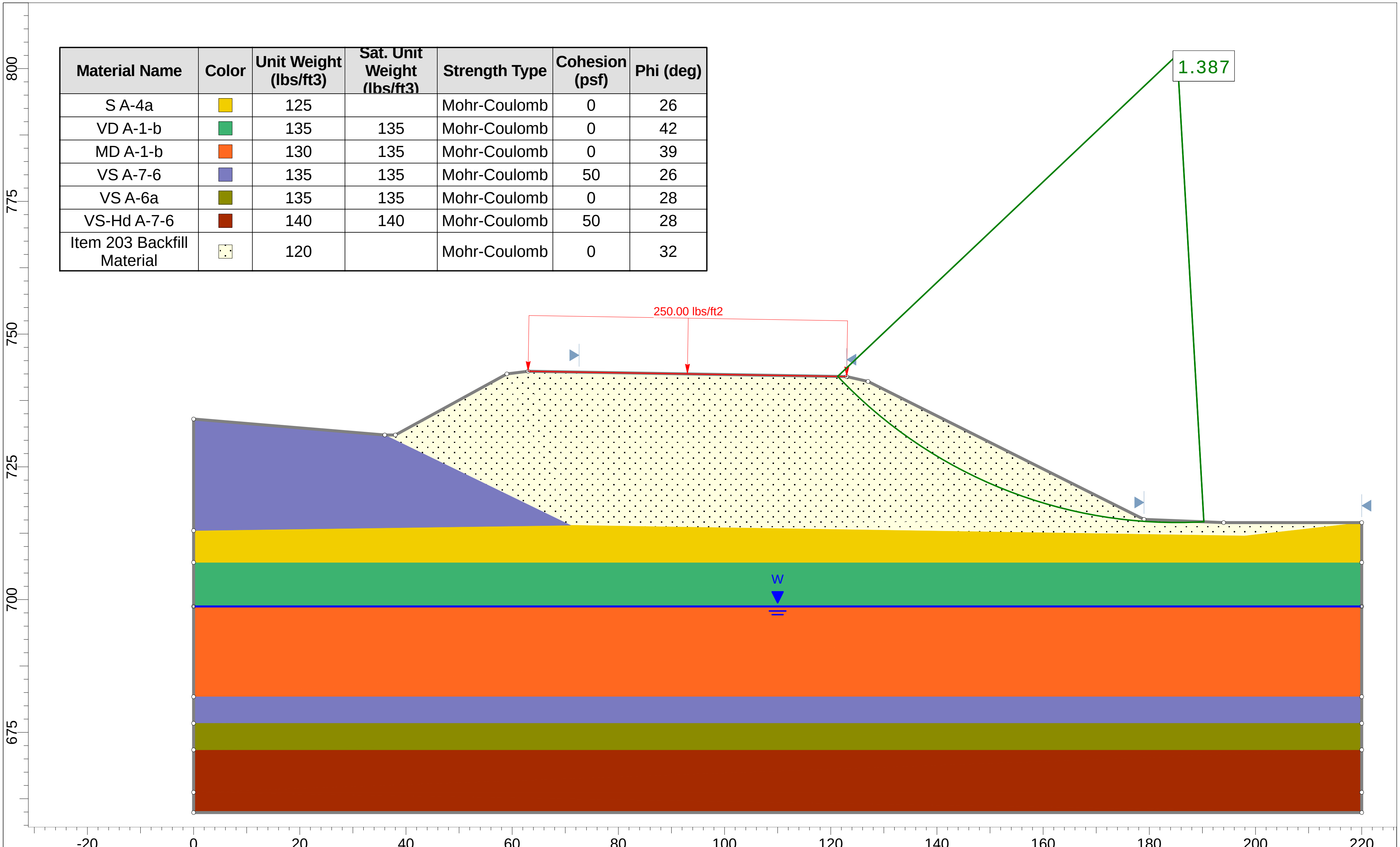
Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	N_{60}	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_m (1) (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
A-4a	C	0.0	3.0	3.0	1.5	6	115	345	173	79			15	0.045	0.005	0.389			0.05	1.000	3,598	3,677	0.016	0.195
A-4a	C	3.0	6.0	3.0	4.5	5	115	690	518	237			15	0.045	0.005	0.389			0.15	0.990	3,563	3,800	0.012	0.141
A-1-b	G	6.0	8.5	2.5	7.3	30	125	1,003	846	394							46	154	0.24	0.963	3,466	3,860	0.016	0.193
A-1-b	G	8.5	11.5	3.0	10.0	53	135	1,408	1,205	581							75	297	0.33	0.919	3,310	3,891	0.008	0.100
A-1-b	G	11.5	15.0	3.5	13.3	53	135	1,880	1,644	817							69	262	0.44	0.855	3,079	3,896	0.009	0.109
A-1-b	G	15.0	17.5	2.5	16.3	29	125	2,193	2,036	1,022							36	116	0.54	0.792	2,851	3,873	0.013	0.150
A-1-b	G	17.5	20.5	3.0	19.0	29	130	2,583	2,388	1,202							34	111	0.63	0.735	2,646	3,848	0.014	0.164
A-1-b	G	20.5	23.5	3.0	22.0	29	130	2,973	2,778	1,405							32	106	0.73	0.677	2,438	3,843	0.012	0.148
A-1-b	G	23.5	27.5	4.0	25.5	29	130	3,493	3,233	1,641							31	102	0.85	0.617	2,220	3,861	0.015	0.175
A-1-b	G	27.5	30.0	2.5	28.8	56	140	3,843	3,668	1,874							57	202	0.96	0.567	2,042	3,916	0.004	0.048
A-1-b	G	30.0	32.0	2.0	31.0	56	140	4,123	3,983	2,048							56	194	1.03	0.537	1,931	3,980	0.003	0.036
A-7-6	C	32.0	34.0	2.0	33.0	54	140	4,403	4,263	2,203	8,000	10,203	45	0.315	0.032	0.624			1.10	0.511	1,841	4,044	0.010	0.123
A-7-6	C	34.0	37.0	3.0	35.5	22	135	4,808	4,605	2,390	8,000	10,390	45	0.315	0.032	0.624			1.18	0.483	1,738	4,128	0.014	0.166
A-6a	C	37.0	42.0	5.0	39.5	27	135	5,483	5,145	2,680	8,000	10,680	20	0.090	0.009	0.428			1.32	0.442	1,592	4,273	0.006	0.077
A-7-6	C	42.0	45.0	3.0	43.5	72	140	5,903	5,693	2,978	8,000	10,978	20	0.090	0.009	0.428			1.45	0.408	1,467	4,445	0.003	0.039
A-7-6	C	45.0	50.0	5.0	47.5	70	140	6,603	6,253	3,289	8,000	11,289	41	0.279	0.028	0.593			1.58	0.378	1,359	4,648	0.013	0.158
A-7-6	C	50.0	53.8	3.8	51.9	150	140	7,135	6,869	3,630	8,000	11,630	41	0.279	0.028	0.593			1.73	0.349	1,256	4,886	0.009	0.103
																					Total Settlement:			2.123 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)

APPENDIX VII

GLOBAL STABILITY ANALYSIS OUTPUT

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
S A-4a		125		Mohr-Coulomb	0	26
VD A-1-b		135	135	Mohr-Coulomb	0	42
MD A-1-b		130	135	Mohr-Coulomb	0	39
VS A-7-6		135	135	Mohr-Coulomb	50	26
VS A-6a		135	135	Mohr-Coulomb	0	28
VS-Hd A-7-6		140	140	Mohr-Coulomb	50	28
Item 203 Backfill Material		120		Mohr-Coulomb	0	32



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SLIDEINTERPRET 8.020

Project	FRA-70-12.68 Project 4R		
Analysis Description	FRA-70-1357A - Ramp C5 over Electrical Vault - Boring B-017-1-09 & B-017-3-13 - Global Stability - Drained Spencer's		
Drawn By HSK	*Scale* 1:300	*Company* Resource International, Inc	
Date 1/29/2019, 8:54:52 AM	*File Name* FRA-70-12.68 Culvert (Vault) 4W19 -Global Stability-Drained.slmd		