FRA-70-12.68 PROJECT 4R
RETAINING WALLS 4W8 AND 4W9
PID NO. 105523
FRANKLIN COUNTY, OHIO

STRUCTURE FOUNDATION EXPLORATION REPORT

Prepared For:
GPD GROUP
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Columbus, OH 43215

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

July 2018

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March 2, 2018 (Revised July 15, 2018)

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

Re: Structure Foundation Exploration Report
FRA-70-12.68 Project 4R
Retaining Walls 4W8 and 4W9
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. 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 proposed Retaining Walls 4W8 and 4W9 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.

Brian Trenner

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

Jonathan P. Sterenberg, P.E.
Director – Geotechnical Planning

Enclosure: Structure Foundation Exploration Report

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APPENDICIES

EXECUTIVE SUMMARY

Resource International, Inc. (Rii) has completed a structure foundation exploration for the design and construction of proposed Retaining Walls 4W8 and 4W9 as part of the FRA-70-12.68 Project 4R. Based on proposed plan information provided by Dynotec and GPD GROUP, it is understood that the wall will wrap around in front of the forward abutment of the proposed FRA-70-1358A and FRA-70-1358R bridge structures at the west end of the wall alignment and to the rear abutment of the FRA-70-1373A and FRA-70-1373R bridge structures at the east end of the wall alignment. The total wall length for Retaining Wall 4W8, including the portions of the wall that cross in front of the abutments of the proposed bridge structures, is approximately 969 lineal feet, and the total length between the abutments of the two crossings, from Sta. 5074+12 to 5080+60 (BL Ramp C5), is approximately 648 feet. It is understood that a mechanically stabilized earth (MSE) wall is being considered as the preferred wall type for Retaining Wall 4W8. **Please note that the design of the MSE wall where it crosses the abutments of the proposed bridge structures will be governed by the recommendations in the respective bridge structure reports, which are presented under separate covers.**

Based on proposed plan information provided by Dynotec and GPD GROUP, Retaining Wall 4W9 will support Ramp C5 along the north side of the ramp and will provide grade separation between the ramp and the proposed I-70 eastbound where the two grades separate. The wall begins at Sta. 5080+41 (BL Ramp C5) where it will be connected to the north side of the rear abutment of the proposed FRA-70-1373A bridge structure, and ends at Sta. 5079+00 (BL Ramp C5). The total length of Wall 4W9 is approximately 143 feet. It is understood that a cast-in-place (CIP) wall is being considered as the preferred wall type for Retaining Wall 4W9.

Exploration and Findings

Between July 3 and August 7, 2013, four (4) structural borings, designated as B-017-6-13, B-017-7-13, B-019-1-13 and B-020-1-13, were drilled to completion depths ranging from 50.0 to 96.7 feet below the existing ground surface along the proposed alignment of retaining wall 4W8. In addition to the borings performed by Rii and Stock as part of the current exploration, one (1) boring, designated as B-019-0-08, was advanced to a completion depth of 75.8 feet below the existing ground surface by DLZ as part of the FRA-70-8.93 preliminary exploration.

Boring B-017-7-13 was drilled through the existing pavement of I-70 eastbound and encountered 11.0 inches of asphalt overly 6.0 inches of aggregate base. Boring B-019-0-08 was drilled through an access drive that extends south off of the asphalt access road between Short Street and the railroad tracks to the west and encountered 12.0 inches of aggregate base at the ground surface. Boring B-019-1-13 was performed in the grass area just south of the asphalt access road and encountered 12.0 inches of topsoil at the ground surface. Boring B-020-1-13 was performed in a grass yard between a new building and the asphalt access drive and encountered 6.0 inches of topsoil overlying 4.0 inches of brick pavers.

Beneath the pavement section in boring B-017-7-13, existing embankment fill consisting of brown and brownish gray silt and clay and silty clay (ODOT A-6a, A-6b) with seams of brown and gray gravel (ODOT A-1-a). Beneath the surface materials in the remaining borings, material identified as existing fill was encountered extending to depths ranging from 10.5 feet below the existing ground surface in boring B-017-6-13 to 18.0 feet below the ground surface in borings B-019-0-08, which corresponds to elevations ranging from 695.5 to 704.4 feet msl. The fill material consisted of brown, dark brown, gray and black gravel and sand, gravel with sand and silt, sandy silt, silt and clay and silty clay (ODOT A-1-b, A-2-4, A-4a, A-6a, A-6b). Material identified as organic clay (A-8b) was encountered in boring B-019-0-08 at a depth of 8.0 feet and extended to a depth of 15.5 feet below existing grade. The organic content in this layer ranged from 7 to 9 percent, and large wood fragments were encountered throughout this layer as well.

Underlying the surficial materials and existing fill, natural granular soils were encountered overlying cohesive soils. The granular soils were generally described as brown, gray and dark brown gravel, gravel and sand, gravel with sand and silt, gravel with sand, silt and clay and coarse and fine sand (ODOT A-1-a, A-1-b, A-2-4, A-2-6, A-3a). The cohesive soils were described as gray, brown and brownish gray sandy silt, silt and clay, silty clay, elastic clay and clay (ODOT A-4a, A-6a, A-6b, A-7-5, A-7-6).

Shale bedrock was encountered along the proposed wall alignment at a depths ranging from 54.0 to 90.0 feet below the ground surface, which corresponds to elevations ranging from 648.6 feet msl in boring B-020-1-13 to 659.5 feet msl in boring B-019-0-08. Competent limestone bedrock was encountered in borings B-017-6-13 and B-020-1-13 at a depth of 74.5 and 80.5 feet below the ground surface, which corresponds to elevations of 640.4 and 632.3 feet msl, respectively.

Analyses and Recommendations

MSE Wall Recommendations (Retaining Wall 4W8)

Based upon the proposed plan information, the proposed retaining wall will have a maximum height of approximately 42.3 feet, as measured from the top of the leveling pad to the top of the coping.

Material identified as existing fill consisting of loose gravel and sand and sandy silt (ODOT A-1-b, A-4a), soft silt and clay (ODOT A-6a) and stiff organic clay (ODOT A-8a) in boring B-019-0-08, which contained a significant amount of organics and wood fibers throughout, was encountered at the proposed bearing elevation. These unsuitable soils extend to a depths ranging from 6.1 to 13.2 feet below the bottom of wall elevation (El. 694.8 to 701.9 feet). As noted in Section 5.1 of the full report, it is understood that ground improvement techniques will be implemented along the alignment of Retaining Wall 4W8. As this is a proprietary design, the analysis for this wall considers the existing fill material will remain in place. MSE wall foundations bearing on existing fill material may be proportioned for a factored bearing resistance as indicated in Table 7. A geotechnical resistance factor of φ_b=0.65 was considered in calculating the factored bearing resistance at the strength limit state.

Retaining Wall 4W8 MSE Wall Design Parameters

From Station [1]	**To Station [1]**	**Representative Borings**	**Wall Height Analyzed (feet)**	**Minimum Required Reinforcement Length [2] (feet)**	**Bearing Resistance at Strength Limit (ksf)**		**Strength Limit Equivalent Bearing Pressure [4] (ksf)**
					Nominal	**Factored [3]**	
5074+12	5080+60	B-019-0-08, B-019-1-13 and B-020-1-13	42.3	29.6 (0.7H)	5.03	3.21	9.54

1. *Limits of wall determined from plan information provided by Dynotec. Stationing listed is referenced to Ramp C5 and reflects only the portion of the wall between the abutment substructures of the FRA-70-1358A and FRA-70-1373A bridge structures.*
2. *The minimum reinforcement length is expressed as a percentage of the wall height, H.*
3. *A geotechnical resistance factor of φ_b=0.65 was considered in calculating the factored bearing resistance at the strength limit state.*
4. *The strength limit equivalent bearing pressure is the uniformly distributed pressure asserted by the wall over an effective base width based on the eccentricity of the wall system at the strength limit state.*

Total settlements ranging from 10.98 to 16.95 inches at the center of the reinforced soil mass and 7.45 to 10.29 inches at the facing of the wall are anticipated along the alignment of retaining wall 4W8. Based on the results of the analysis, 90 percent of the total settlement is anticipated to occur over a period of approximately 20 to 90 days, with the longer durations being anticipated along the western half of the wall alignment in the area of borings B-017-6-13 and B-019-0-08.

Based on the results of the external and global stability analysis performed for the MSE wall, sliding under undrained conditions as well as bearing and global stability under both drained and undrained conditions were not satisfied at a strap length equal to 0.7 times the wall height. Increasing the width of the wall up to 1.0 times the wall height still did not satisfy all of the external and global stability requirements.

Consideration was given to over excavating the existing fill and unsuitable soils and replacing it with granular embankment; however, given the depth and extent of this material the alignment of the wall, this a very expensive and uneconomical option. Therefore, it is recommended that ground improvement techniques be implemented to increase the strength of the soil mass and reduce settlement potential within these layers. Additional considerations for the ground improvement design, including required performance criteria, are provided in Section 5.1.8 of the full report.

CIP Wall Recommendations

Based on the proposed plan and cross section information provided by Dynotec, a maximum wall height of 10.4 feet is anticipated along the alignment. It is understood that up to 35 feet of new embankment fill will be placed to bring the site to proposed grade along Ramp C5. Therefore, the bearing soils below retaining wall 4W9 will consist of new embankment fill. CIP wall foundations bearing on newly placed granular embankment fill may be proportioned for a factored bearing resistance as indicated in the following table. A geotechnical resistance factor of φ_b=0.55 was considered in calculating the factored bearing resistance at the strength limit state. Given that the wall will be bearing on approximately 30 to 35 feet of new embankment fill, little to no settlement is anticipated under the loading from the proposed wall along the alignment.

Retaining Wall 4W9 CIP Wall Design Parameters

From Station [1]	**To Station [1]**	**Representative Borings**	**Wall Height Analyzed (feet)**	**Minimum Required Foundation Width [2] (feet)**	**Bearing Resistance at Strength Limit (ksf)**		**Strength Limit Equivalent Bearing Pressure [4] (ksf)**
					Nominal	**Factored [3]**	
5079+00	5080+41	N/A	10.4	7.5 (0.72H)	27.34	15.04	2.39

1. *Limits of wall determined from plan information provided by Dynotec Stationing listed is referenced to Ramp C5.*
2. *The foundation width based on the wall section provided in the design sheets.*
3. *A geotechnical resistance factor of φ_b=0.55 was considered in calculating the factored bearing resistance at the strength limit state.*
4. *The strength limit equivalent bearing pressure is the uniformly distributed pressure asserted by the wall over an effective base width based on the eccentricity of the wall system at the strength limit state.*

Based on the results of the external and global stability analysis performed for Retaining Wall 4W9, the wall section provided in the design sheet for Wall 4W9, which includes a base width of 7.5 feet (0.73 times the height of the CIP wall) meets all of the external and global stability requirements.

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 proposed Retaining Walls 4W8 and 4W9 as part of the FRA-70-12.68 Project 4R, as shown on the vicinity map and boring plan presented in Appendix I. Based on proposed plan information provided by Dynotec and GPD GROUP, Retaining Wall 4W8 will support Ramp C5 along the south side of the ramp, and will provide the required grade separation in lieu of graded embankments in this area. Additionally, it is understood that the wall will wrap around in front of the forward abutment of the proposed FRA-70-1358A and FRA-70-1358R bridge structures at the west end of the wall alignment and to the rear abutment of the FRA-70-1373A and FRA-70-1373R bridge structures at the east end of the wall alignment. The total wall length for Retaining Wall 4W8, including the portions of the wall that cross in front of the abutments of the proposed bridge structures, is approximately 969 lineal feet, and the total length between the abutments of the two crossings, from Sta. 5074+12 to 5080+60 (BL Ramp C5), is approximately 648 feet. It is understood that a mechanically stabilized earth (MSE) wall is being considered as the preferred wall type for Retaining Wall 4W8. **Please note that the design of the MSE wall where it crosses the abutments of the proposed bridge structures will be governed by the recommendations in the respective bridge structure reports, which are presented under separate covers.**

Based on proposed plan information provided by Dynotec and GPD GROUP, Retaining Wall 4W9 will support Ramp C5 along the north side of the ramp and will provide grade separation between the ramp and the proposed I-70 eastbound where the two grades separate. The wall begins at Sta. 5080+41 (BL Ramp C5) where it will be connected to the north side of the rear abutment of the proposed FRA-70-1373A bridge structure, and ends at Sta. 5079+00 (BL Ramp C5). The total length of Wall 4W9 is approximately 143 feet. It is understood that a cast-in-place (CIP) wall is being considered as the preferred wall type for Retaining Wall 4W9.

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. Within the borings performed for this current project, shale bedrock was encountered at depths ranging from 54.3 to 64.3 feet below the ground surface, which corresponds to elevations ranging from 648.6 feet to 659.5 feet msl.

2.2 Existing Conditions

The proposed Retaining Wall 4W8 and 4W9 structures will be situated along the south side of I-70 eastbound between the existing CSX/Norfolk Southern Railroad tracks and Short Street. The existing I-70 eastbound in the vicinity of the structure is a four-lane, asphalt paved roadway that is aligned east-to-west. The existing I-70 roadway profile grade is elevated approximately 25 feet above the railroad and surrounding terrain on graded embankments. An asphalt/gravel access road is situated at the toe of the existing embankment which provides access to the railroad tracks from Short Street. There is a commercial property situated along the south side of the access road. The terrain along I-70 slopes gently to the west and the surrounding area is relatively flat-lying, and dense vegetation covers the existing I-70 embankment slope.

3.0 EXPLORATION

Between July 3 and August 7, 2013, four (4) structural borings, designated as B-017-6-13, B-017-7-13, B-019-1-13 and B-020-1-13, were drilled to completion depths ranging from 50.0 to 96.7 feet below the existing ground surface along the proposed alignment of retaining wall 4W8. Borings B-017-6-13 and B-017-7-13 were performed by Rii, while B-019-1-13 and B-020-1-13 were performed by Stock Drilling under the direction of Rii. In addition to the borings performed by Rii and Stock as part of the current exploration, one (1) boring, designated as B-019-0-08, was performed by DLZ along the proposed alignment of retaining wall 4W8 as part of the FRA-70-8.93 preliminary exploration and their findings were published in a report dated March 18, 2010. The boring was advanced to a completion depth of 75.8 feet below the existing ground surface. The current project boring locations are shown on the boring plan provided in Appendix I of this report and summarized in Table 1 below.

Table 1. Test Boring Summary

Boring Number	Station [1]	Offset [1]	Latitude	Longitude	Ground Elevation (feet msl)	Boring Depth (feet)
B-017-6-13	5074+21.50	19.1' Rt.	39.952895767	-83.006754692	714.9	84.5
B-017-7-13	170+79.36	23.3’ Rt.	39.953200568	-83.006425064	743.1	96.7
B-019-0-08	5076+04.30	29.0' Rt.	39.952897615	-83.006105991	713.5	75.8
B-019-1-13	5077+15.33	3.2' Rt.	39.952978442	-83.005713188	712.5	50.0
B-020-1-13	5080+09.80	30.9' Rt.	39.952922218	-83.004665587	712.8	86.0

1. *Station and offset for boring B-017-7-13 is referenced to the proposed baseline of I-70 EB. The remaining borings are referenced to the proposed baseline of Ramp C5.*

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

The borings performed by Stock Drilling and Rii for the current exploration were drilled using a truck or an all-terrain vehicle (ATV) mounted rotary drilling machine, utilizing a 3.25 or 4.25-inch inside diameter, hollow-stem auger to advance the holes. Standard penetration test (SPT) and split spoon sampling were performed in the borings at 2.5-foot increments of depth to 20 feet in boring B-019-1-13 and 30 feet in borings B-017-6-13, B-017-7-13 and B-020-1-13 and at 5.0-foot increments thereafter to the boring termination depth. The SPT, per the American Society for Testing and Materials (ASTM) designation D1586, is conducted using a 140-pound hammer falling 30.0 inches to drive a 2.0-inch outside diameter split spoon sampler 18.0 inches. Stock Drilling and Rii utilized a calibrated automatic drop hammer to generate consistent energy transfer to the sampler. Driving resistance is recorded on the boring logs in terms of 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 Mobile B-53 drill rig used by Rii was calibrated on April 26, 2013, and has a drill rod energy ratio of 77.7, and the CME 750X used by Stock Drilling was calibrated on March 28, 2013 and has a drill rod energy ratio of 86.8 percent.

During drilling for the borings performed by Stock Drilling and Rii, field logs were prepared by Rii personnel showing the encountered subsurface conditions. Soil samples obtained from the drilling operation were preserved and sealed in glass jars and delivered to the soil laboratory. In the laboratory, the soil samples were visually classified and select samples were tested, as noted in Table 2.

Table 2. Laboratory Test Schedule

Laboratory Test	Test Designation	Number of Tests Performed
Natural Moisture Content	ASTM D 2216	82
Plastic and Liquid Limits	AASHTO T89, T90	25
Gradation – Sieve/Hydrometer	AASHTO T88	25
Unconfined Compressive Strength of Intact Rock	ASTM D7012	2

The tests performed are necessary to classify existing soil according to the Ohio Department of Transportation (ODOT) classification system and to estimate engineering properties of importance in determining foundation design and construction recommendations. Results of the laboratory testing are presented on the boring logs in Appendix III. A description of the soil terms used throughout this report is presented in Appendix II.

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 borings B-017-6-13, B-017-7-13, B-019-0-08 and B-020-1-13 was determined by split spoon sampler refusal. Split spoon sampler refusal is defined as exceeding 50 blows from the hammer with less than 6.0 inches of penetration by the split spoon sampler.

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

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

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

4.0 FINDINGS

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

4.1 Surface Materials

Boring B-017-7-13 was drilled through the existing pavement of I-70 eastbound and encountered 11.0 inches of asphalt overly 6.0 inches of aggregate base. Boring B-019-0-08 was drilled through an access drive that extends south off of the asphalt access road between Short Street and the railroad tracks to the west and encountered 12.0 inches of aggregate base at the ground surface. Boring B-019-1-13 was performed in the grass area just south of the asphalt access road and encountered 12.0 inches of topsoil at the ground surface. Boring B-020-1-13 was performed in a grass yard between a new building and the asphalt access drive and encountered 6.0 inches of topsoil overlying 4.0 inches of brick pavers.

4.2 Subsurface Soils

Beneath the pavement section in boring B-017-7-13, existing embankment fill consisting of brown and brownish gray silt and clay and silty clay (ODOT A-6a, A-6b) with seams of brown and gray gravel (ODOT A-1-a). Beneath the surface materials in the remaining borings, material identified as existing fill was encountered extending to depths ranging from 10.5 feet below the existing ground surface in boring B-017-6-13 to 18.0 feet below the ground surface in borings B-019-0-08, which corresponds to elevations ranging from 695.5 to 704.4 feet msl. The fill material consisted of brown, dark brown, gray and black gravel and sand, gravel with sand and silt, sandy silt, silt and clay and silty clay (ODOT A-1-b, A-2-4, A-4a, A-6a, A-6b). Material identified as organic clay (A-8b) was encountered in boring B-019-0-08 at a depth of 8.0 feet and extended to a depth of 15.5 feet below existing grade. The organic content in this layer ranged from 7 to 9 percent, and large wood fragments were encountered throughout this layer as well.

Underlying the surficial materials and existing fill, natural granular soils were encountered overlying cohesive soils. The granular soils were generally described as brown, gray and dark brown gravel, gravel and sand, gravel with sand and silt, gravel with sand, silt and clay and coarse and fine sand (ODOT A-1-a, A-1-b, A-2-4, A-2-6, A-3a). The cohesive soils were described as gray, brown and brownish gray sandy silt, silt and clay, silty clay, elastic clay and clay (ODOT A-4a, A-6a, A-6b, A-7-5, 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 very loose (N_{60} < 5 blows per foot [bpf]) to very dense (N_{60} > 50 bpf). Overall blow counts recorded from the SPT sampling ranged from 4 bpf to split spoon sampler refusal. The shear strength and consistency of the cohesive soils are primarily derived from the hand penetrometer values (HP). The cohesive soil encountered ranged from soft ($0.25 \leq HP \leq 0.5$ tsf) to hard (HP > 4.0 tsf). The unconfined compressive strength of the cohesive soil samples tested, obtained from the hand penetrometer, ranged from 0.5 to over 4.5 tsf (limit of instrument).

Natural moisture contents of the soil samples tested ranged from 4 to 29 percent, and the moisture content within the organic clay (ODOT A-8b) layer encountered in boring B-019-0-08 ranged from 35 to 47 percent. The natural moisture content of the cohesive soil samples tested for plasticity index ranged from 13 percent below to 4 percent above their corresponding plastic limits. In general, the soil exhibited natural moisture contents considered to be significantly below to moderately above optimum moisture levels.

4.3 Bedrock

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

Table 3. Top of Bedrock Elevations

Boring Number	Ground Surface Elevation (feet msl)	Top of Bedrock (Sampler Refusal)		Top of Bedrock Core (Auger Refusal)	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-017-6-13	714.9	64.3	650.6	74.5	640.4
B-017-7-13	743.1	90.0	653.1	90.0	653.1
B-019-0-08	713.5	54.0	659.5	55.0	658.5
B-019-1-13	712.5	N/A	N/A	N/A	N/A
B-020-1-13	712.8	64.2	648.6	80.5	632.3

Shale bedrock was encountered along the proposed wall alignment at a depths ranging from 54.0 to 90.0 feet below the ground surface, which corresponds to elevations ranging from 648.6 feet msl in boring B-020-1-13 to 659.5 feet msl in boring B-019-0-08. However, this weathered bedrock material was able to be augered in borings B-017-6-13 and B-020-1-13, while this material required rock coring techniques to advance the borings in B-017-713 and B-019-0-08. Competent limestone bedrock was encountered in borings B-017-6-13 and B-020-1-13 at a depth of 74.5 and 80.5 feet below the ground surface, which corresponds to elevations of 640.4 and 632.3 feet msl, respectively. The cored shale bedrock encountered in borings B-017-7-13 and

B-019-0-08 was described as light gray to blue gray, dark gray, highly to severely weathered, very weak to weak and moderately fractured to highly fractured. The cored limestone bedrock encountered in borings B-017-6-13 and B-020-1-13 was described as light brown, light gray, gray and dark gray, unweathered, strong to very strong and slightly fractured to highly fractured.

The percent recovery, RQD values and unconfined compressive strengths of the bedrock core runs are summarized in Table 4.

Table 4. Rock Core Summary

Boring	Core No.	Elevation (feet msl)	Recovery (%)	RQD (%)	Unconfined Compressive Strength
B-017-6-13	RC-1	640.4 to 635.4	93	83	q_u @ 75.6' = 12,261 psi
	RC-2	635.4 to 630.4	100	100	N/A
B-017-7-13	RC-1	653.1 to 652.1	100	79	N/A
	RC-2	652.1 to 651.1	100	66	N/A
	RC-3	651.1 to 646.1	74	15	N/A
B-019-0-08	R-2	657.7 to 652.7	47	23	N/A
	R-3	652.7 to 647.7	38	17	N/A
	R-4	647.7 to 642.7	100	67	N/A
	R-5	642.7 to 637.7	50	22	N/A
B-020-1-13	RC-2	632.3 to 626.8	99	49	q_u @ 80.7' = 9,465 psi

It should be noted that bedrock naturally experiences mechanical breaks during the drilling and coring processes. Rii attempted to account for fresh, manmade breaks during tabulation of the RQD analysis. The zone within borings B-019-0-08 and B-020-1-13 where boulders were encountered that required rock coring techniques to advance through these zones are not included in the RQD tabulation above. The quality of the shale bedrock, according to the RQD values, was very poor (RQD ≤ 25%) to fair (50% < RQD ≤ 75%) and the quality of the limestone bedrock was poor (25% < RQD ≤ 50%) to excellent (90% < RQD ≤ 100%).

4.4 Groundwater

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

Table 5. Groundwater

Boring Number	Ground Elevation (feet msl)	Initial Groundwater		Upon Completion	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-017-6-13	714.9	28.0	686.9	N/A [1]	N/A
B-017-7-13	743.1	57.0	686.1	N/A [1]	N/A
B-019-0-08	713.5	12.5	701.0	12.0	701.5
B-019-1-13	712.5	23.5	689.0	N/A [1]	N/A
B-020-1-13	712.8	23.0	689.8	N/A [1]	N/A

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

Groundwater was encountered initially during drilling in all five borings at depths ranging from 12.5 to 57.0 feet below the ground surface, which corresponds to elevations ranging from 686.1 to 701.0 feet msl. The groundwater level at the completion of drilling in boring B-019-0-08 was 12.5 feet below existing grade prior to adding water for the rock coring process, which corresponds to an elevation of 701.5 feet msl. The groundwater levels at the completion of drilling in the remaining borings could not be measured due to the addition of water or mud to counteract heaving sands as well as water as a circulating fluid during the rock coring process.

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 current and preliminary exploration programs 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 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 Dynotec. It is understood that a mechanically stabilized earth (MSE) wall is being considered as the preferred wall type for Retaining Wall 4W8. The wall extends between the FRA-70-1358A and FRA-70-1373A bridge structure, and will wrap around in front of the forward abutment of the proposed FRA-70-1358A and FRA-70-1358R bridge structures at the west end of the wall alignment and to the rear abutment of the FRA-70-1373A and FRA-70-1373R bridge structures at the east end of the wall alignment. The total wall length for Retaining Wall 4W8, including the portions of the wall that cross in front of the abutments of the proposed bridge structures, is approximately 969 lineal feet, and the total length between the abutments of the two crossings, from Sta. 5074+12 to 5080+60 (BL Ramp C5), is approximately 648 feet.

It is understood that a cast-in-place (CIP) wall is being considered as the preferred wall type for Retaining Wall 4W9. The wall begins at Sta. 5080+41 (BL Ramp C5) where it will be connected to the north side of the rear abutment of the proposed FRA-70-1373A bridge structure, and ends at Sta. 5079+00 (BL Ramp C5). The total length of Wall 4W9 is approximately 143 feet.

5.1 MSE Wall Recommendations (Retaining Wall 4W8)

It is understood that a MSE retaining wall is being considered for use in supporting the proposed Ramp C5 alignment. MSE walls are constructed on earthen foundations at a minimum depth of 3.0 feet below grade, as defined by the top of the leveling pad to the ground surface located 4.0 feet from the face of the wall. Per Section 204.6.2.1 of the 2007 ODOT BDM, the height of the MSE wall is defined as the elevation difference between the top of coping and the top of the leveling pad. However, it is noted that the reinforced soil mass only extends from the foundation bearing elevation (top of leveling pad) to the roadway subgrade elevation. The width of the MSE wall foundation (B) is defined by the length of the reinforced soil mass. Per the Section 204.6.2.1 of the 2007 ODOT BDM and Supplemental Specification (SS) 840, the minimum length of the reinforced soil mass is equal to 70 percent of the height of the MSE wall or 8.0 feet whichever is greater. A non-structural bearing leveling pad consisting of a minimum of 6.0-inches of unreinforced concrete should be placed at the base of the wall facing for constructability purposes. Please note that the leveling pad is not a structural foundation.

Based upon the proposed plan information provided by Dynotec, the proposed retaining wall will have a maximum height of approximately 42.3 feet, as measured from the top of the leveling pad to the top of the coping. Therefore, it is considered that the minimum reinforcement length and the effective foundation width (B) for external and global stability calculations will be 29.6 feet.

Per Section 840.06.D of ODOT SS 840, the foundation subgrade should be inspected to verify that the subsurface conditions are the same as those anticipated in this report. Material identified as existing fill or possible fill containing soft soils and organic matter was encountered at the proposed bearing elevation in borings B-017-6-13, B-019-0-08, B-019-1-13, and B-020-1-13. The unsuitable material extends to a depth of 3.6 feet below the proposed bearing elevation in boring B-017-6-13 (El. 704.4 feet), 12.5 feet below the proposed bearing elevation in boring B-019-0-08 (El. 695.5 feet), 8.5 feet below the proposed bearing elevation in boring B-019-1-13 (El. 699.5 feet), and 13.2 feet below the proposed bearing elevation in boring B-020-1-13 (El. 694.8 feet). The fill material consisted of loose gravel and sand and sandy silt (ODOT A-1-b, A-4a), soft silt and clay (ODOT A-6a) and stiff organic clay (ODOT A-8a) in boring B-019-0-08, which contained a significant amount of organics and wood fibers throughout. In addition, stiff silt and clay (ODOT A-6a) was encountered below the existing fill in boring B-017-6-13, which extended to a depth of 6.1 feet below the proposed bearing elevation (El. 701.9 feet). These soils are not considered suitable for foundation support for a wall of this size.

A study was performed by GPD GROUP, dated March 2, 2018, to investigate the use of ground improvement techniques (stone columns/rigid inclusions) as well as the use of lightweight fill consisting of cellular concrete to control settlement within the fill material and meet strength requirements. Analyses for both alternatives were provided in the report, as well as a cost comparison between the two alternatives. Based on the results of the study, it is understood that ground improvement techniques will be a cheaper option for this wall.

The ground improvement techniques, which will consist of stone columns or rigid inclusion, will increase the bearing resistance of the bearing soils and also reduce settlement. Based on the information provided in the study, it is understood that the ground improvement elements will be installed along the entire footprint of Wall 4W8 and extend all of the way up to the existing I-70 embankment. The design of such a system is proprietary and beyond the scope of this investigation. Based on discussions with the ODOT Office of Geotechnical Engineering (OGE), the analysis for the wall was performed assuming that the existing fill and unsuitable soils will remain in place and not be stabilized. Additional considerations for the ground improvement design, including required performance criteria, are provided in Section 5.1.8.

Per ODOT SS 840, following foundation subgrade inspection and acceptance, a minimum of 12.0 inches of ODOT Item 703.16.C, Granular Material Type C, should be placed and compacted in accordance with ODOT Item 204.07.

Since the wall is located within an existing floodplain, the analysis was performed using a design groundwater level at the ground surface.

5.1.1 Strength Parameters Utilized in External and Global Stability Analyses

The shear strength parameters utilized in the external and global stability analyses for the MSE walls at the abutments are provided in Table 6.

Table 6. Shear Strength Parameters Utilized in Stability Analyses

Material Type	γ (pcf)	φ' (1) (°)	c' (2) (psf)	S_u (3) (psf)
MSE Wall Backfill (Select Granular Fill)	120	34	0	N/A
Item 203 Granular Embankment (Retained Soil)	130	33	0	N/A
Existing I-70 Embankment Stiff to Very Stiff Silt and Clay and Silty Clay (ODOT A-6a, A-6b)	120	28	0	2,500
Existing Fill: Soft to Stiff Silt and Clay (ODOT A-6a)	115	26	0	875 to 1,500
Existing Fill: Soft to Stiff Silty Clay (ODOT A-6b)	115	25	0	1,000 to 1,250
Existing Fill: Stiff Organic Clay (ODOT A-8b)	100	20	0	1,000
Existing Fill: Very Loose to Medium Dense Granular Soils (ODOT A-1-b, A-2-4, A-4a)	120 to 125	30 to 39	0	N/A
Medium Dense to Very Dense Granular Soils (ODOT A-1-b, A-2-4, A-3a)	125 to 135	35 to 43	0	N/A
Stiff to Hard Sandy Silt (ODOT A-4a)	120	28	0	3,125 to 7,000
Hard Silty Clay (ODOT A-6b)	125	27	0	4,375
Very Stiff to Hard Elastic Clay (ODOT A-7-5)	125	24	0 to 50	3,375 to 8,000
Very Stiff to Hard Clay (ODOT A-7-6)	130	26	50	8,000

1. *Per Figure 7-45, Section 7.6.9 of FHWA GEC 5 for cohesive soils and Table 10.4.6.2.4-1 of the 2018 AASHTO LRFS BDS for granular soils.*
2. *Estimated based on overconsolidated nature of soil.*
3. *$S_u = 125(N_{60})$, Terzaghi and Peck (1967).*

Shear strength parameters for the reinforced soil backfill are provided in ODOT SS 840. Per SS 840, the select granular backfill in the reinforced zone must meet the shear strength requirements provided in Table 6. Based on the design plans provided by GPD Group and Dynotec, it is understood that Item 203 granular embankment will be utilized where any new embankment will be placed behind the reinforced soil backfill at both MSE walls. Therefore, the shear strength parameters for the retained fill will be modeled using a friction angle of 33 degrees since granular embankment is being specified, instead of using the shear strength parameters provided in ODOT SS 840.

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 and based on past experience in the vicinity of the site with projects performed in similar subsurface profiles. However, the friction angle for the existing fill that consisted of medium dense gravel with sand and silt was conservatively assigned since there no records of the material origin or how it was placed.

5.1.2 Bearing Stability

Material identified as existing fill consisting of loose gravel and sand and sandy silt (ODOT A-1-b, A-4a), soft silt and clay (ODOT A-6a) and stiff organic clay (ODOT A-8a) in boring B-019-0-08, which contained a significant amount of organics and wood fibers throughout, was encountered at the proposed bearing elevation. These unsuitable soils extend to a depths ranging from 6.1 to 13.2 feet below the bottom of wall elevation (El. 694.8 to 701.9 feet). As noted in Section 5.1, it is understood that ground improvement techniques will be implemented along the alignment of Retaining Wall 4W8. As this is a proprietary design, the analysis for this wall considers the existing fill material will remain in place. MSE wall foundations bearing on existing fill material may be proportioned for a factored bearing resistance as indicated in Table 7. A geotechnical resistance factor of φ_b=0.65 was considered in calculating the factored bearing resistance at the strength limit state.

Table 7. Retaining Wall 4W8 MSE Wall Design Parameters

From Station [1]	**To Station [1]**	**Representative Borings**	**Wall Height Analyzed (feet)**	**Minimum Required Reinforcement Length [2] (feet)**	**Bearing Resistance at Strength Limit (ksf)**		**Strength Limit Equivalent Bearing Pressure [4] (ksf)**
					Nominal	**Factored [3]**	
5074+12	5080+60	B-019-0-08, B-019-1-13 and B-020-1-13	42.3	29.6 (0.7H)	5.03	3.21	9.54

1. *Limits of wall determined from plan information provided by Dynotec. Stationing listed is referenced to Ramp C5 and reflects only the portion of the wall between the abutment substructures of the FRA-70-1358A and FRA-70-1373A bridge structures.*
2. *The minimum reinforcement length is expressed as a percentage of the wall height, H.*
3. *A geotechnical resistance factor of φ_b=0.65 was considered in calculating the factored bearing resistance at the strength limit state.*
4. *The strength limit equivalent bearing pressure is the uniformly distributed pressure asserted by the wall over an effective base width based on the eccentricity of the wall system at the strength limit state.*

Rii performed a verification of the bearing pressure exerted on the subgrade material for the maximum specified wall height indicated in Table 7. Based on the minimum length of reinforced soil mass presented, the factored equivalent bearing pressure exerted below the wall **will exceed** the factored bearing resistance at the strength limit state under drained and undrained conditions, considering the wall will bear on the existing fill material and unsuitable soils.

5.1.3 Settlement Evaluation

The compressibility parameters utilized in the settlement analyses of the proposed MSE wall are provided in Table 8.

Table 8. Compressibility Parameters Utilized in Settlement Analysis

Material Type	γ (pcf)	LL (%)	C_c (1)	C_r (2)	e_o (3)	C_v (4) (ft^2/yr)	N_{60}	C' (5)
Existing Fill: Veru Stiff Sandy Silt (ODOT A-4a)	120	24	0.126	0.013	0.460	800	N/A	N/A
Existing Fill: Soft to Stiff Silt and Clay (ODOT A-6a)	115	33 to 35	0.207 to 0.225	0.031 to 0.034	0.530 to 0.546	600	N/A	N/A
Existing Fill: Soft to Stiff Silty Clay (ODOT A-6b)	115	40	0.270	0.041	0.585	300	N/A	N/A
Existing Fill: Stiff Organic Clay (ODOT A-8b)	100	41	0.460	0.069	0.750	50	N/A	N/A
Existing Fill: Very Loose to Medium Dense Granular Soils (ODOT A-1-b, A-2-4, A-4a)	120 to 125	N/A	N/A	N/A	N/A	N/A	4 to 18	34 to 81
Medium Dense to Very Dense Granular Soils (ODOT A-1-b, A-2-4, A-3a)	125 to 135	N/A	N/A	N/A	N/A	N/A	15 to 100	72 to 494
Stiff to Hard Sandy Silt (ODOT A-4a)	120	22	0.108	0.011	0.444	800	N/A	N/A
Hard Silty Clay (ODOT A-6b)	125	33	0.207	0.021	0.530	300	N/A	N/A
Very Stiff to Hard Elastic Clay (ODOT A-7-5)	125	57	0.423	0.042	0.718	100	N/A	N/A
Very Stiff to Hard Clay (ODOT A-7-6)	130	44 to 51	0.306 to 0.369	0.031 to 0.037	0.616 to 0.671	150	N/A	N/A

1. *Per Table 6-9, Section 6.14.1 of FHWA GEC 5. For the organic soil layer (A-8b) encountered in boring B-019-0-08, $C_c = 0.0115w_n$ per Table 8.2 of Holtz and Kovacs 1981.*
2. *Estimated at 10% of C_c for natural soils and 15% C_c for existing fill per Section 8.11 of Holtz and Kovacs (1981).*
3. *Per Table 8-2 of Holtz and Kovacs (1981).*
4. *Per Figure 6-37, Section 6.14.2 of FHWA GEC 5.*
5. *Per Figure 10.6.2.4.2-1 of 2018 AASHTO LRFD BDS.*

Results of the settlement analysis are tabulated in Table 9. Total settlements ranging from 10.98 to 16.95 inches at the center of the reinforced soil mass and 7.45 to 10.29 inches at the facing of the wall are anticipated along the alignment of retaining wall 4W8. Based on the results of the analysis, 90 percent of the total settlement is anticipated to occur over a period of approximately 20 to 90 days, with the longer durations being anticipated along the western half of the wall alignment in the area of borings B-017-6-13 and B-019-0-08. The presence of a 20-foot thick layer of elastic clay (ODOT A-7-5) in boring B-017-6-13 from El. 657.9 to 677.9 feet, which is not as thick in the remaining borings along the wall alignment, is the likely reason for the large variation in the estimated settlement and time rate of settlement at this location. The thick layer of organic clay, which was not observed in the remaining borings, is the reason for the longer duration at this location. Additionally, given the significant presence of organic matter at this location, it is anticipated that significant consolidation of this layer may take place over the design life of the structure. Please note that the consolidation settlement and time rate of consolidation are based on estimates using correlated compressibility parameters provided in Table 8 for the underlying soils. Actual settlement and time rate of consolidation should be determined by monitoring the settlement of the wall using settlement platforms.

Table 9. Retaining Wall 4W8 MSE Wall Settlement Results

From Station [1]	To Station [1]	Wall Height Analyzed (feet)	Service Limit Equivalent Bearing Pressure [1] (ksf)	Total Settlement Values (inches)		Time for 90% Consolidation (Days)
				Center of Wall Mass	Facing of Wall	
5074+12	5080+50	42.3	6.90	10.98 to 16.95	7.45 to 10.29	20 to 90

1. *The service limit equivalent bearing pressure is the uniformly distributed pressure asserted by the wall over an effective base width based on the eccentricity of the wall system at the service limit state.*

Per Section 204.6.2.1 of the ODOT BDM, “the maximum allowable differential settlement in the longitudinal direction (regardless of the size of panels) is one (1) percent.” Give the amount of settlement anticipated at the facing along the wall alignment, as well as the presence of existing fill material and organic soils that may vary significantly over the footprint of the wall, differential settlement greater than 1/100 may occur if the fill material is not stabilized or over excavated and replaced with embankment fill. If either the total or differential settlement predicted presents an issue with respect to the deformation tolerances that the walls can withstand, then measures should be taken to minimize the amount of settlement that will occur. This can be achieved by preloading the site and consolidating the underlying soils prior to constructing the wall. If preloading the site is not a desired option, then consideration could be given to ground improvement through the use of stone columns. Settlement calculations are provided in Appendix IV.

5.1.4 Eccentricity (Overturning Stability)

The resistance of the MSE wall to overturning will be dependent on the on the location of the resultant force at the bottom of the wall due to the overturning and resisting moments acting on the wall. For MSE walls, overturning stability is determined by calculating the eccentricity of the resultant force from the midpoint of the base of the wall and comparing this value to a limiting eccentricity value. Per Section 11.10.5.5 of the 2014 AASHTO LRFD BDS, for foundations bearing on soil, the location of the resultant of the reaction forces shall be within the middle two-thirds ($^2/_3$) of the base width. Therefore, the limiting eccentricity is one-third ($^1/_3$) of the base width of the wall. Rii performed a verification of the eccentricity of the resultant force for the specified wall height indicated in Table 7. Based on the minimum length of reinforced soil mass presented in Table 7 and utilizing the soil parameters listed in Section 5.1.1 for the retained embankment material, the calculated eccentricity of the resultant force **will not exceed** the limiting eccentricity at the strength limit state.

5.1.5 Sliding Stability

The resistance of the MSE walls to sliding was evaluated per Section 11.10.5.3 of the 2018 AASHTO LRFD BDS. For drained conditions, the sliding resistance is determined by multiplying a coefficient of sliding friction “f” times the total vertical force at the base of the wall. The coefficient of sliding friction is determined based on the limiting friction angle between the foundation soil and the reinforced soil backfill. Based on the soil parameters listed in Section 5.1.1 for the foundation and reinforced soil backfill, a coefficient of sliding friction of 0.36 was utilized for design based on the consideration that the organic clay layer encountered in boring B-019-0-08 is close enough to the bearing elevation to effect sliding under this condition. The sliding resistance at was also evaluated under undrained conditions as well. For undrained conditions, the sliding resistance is taken as the limiting value between the undrained shear strength of the bearing soil and half of the vertical stress applied by the wall multiplied by the width of the MSE wall. Based on the soil parameters listed in Section 5.1.1, the undrained shear strength of the existing fill material and organic soils encountered at the proposed bearing elevation is estimated to be 1,000 psf. A geotechnical resistance factor of φ_{τ}=1.0 was considered in calculating the factored shear resistance between the reinforced backfill material and foundation for sliding. Based on the minimum length of reinforced soil mass presented in Table 7 and utilizing the soil parameters listed in Section 5.1.1 for the retained embankment material, the resultant horizontal forces on the back of the MSE walls **will not exceed** the factored shear resistance at the strength limit state under drained conditions. However, the resultant horizontal forces on the back of the MSE wall **will exceed** the factored shear resistance at the strength limit state under undrained conditions.

5.1.6 Overall (Global) Stability

A slope stability analysis was performed to check the global stability of the wall. As per the AASHTO LRFD BDS, safety against soil failure shall be evaluated at the service limit state by assuming the reinforced soil mass to be a rigid body. Soil parameters utilized in the global stability analyses are presented in Section 5.1.1. For the global stability condition, it was considered that the failure plane will not cross through the reinforced soil mass. The computer software program Slide 7.0 manufactured by Rocscience Inc. was utilized to perform the analyses.

Per Section 11.6.2.3 of the 2014 AASHTO LRFD BDS, for MSE walls that are not integrated with or supporting structural foundations or elements, global stability is satisfied if the product of the factor of safety from the slope stability output multiplied by the resistance factor φ=0.75 is greater than 1.0. Therefore, global stability for the portions of the wall that are adjacent to the abutment substructure is satisfied when a minimum factor of safety of 1.3 is obtained. For an MSE wall designed with the minimum strap length listed in Table 7, the resulting factor of safety under drained conditions (long-term stability) using the Spencer's analysis method was less than 1.3, and was just over 1.0 for both conditions, with the critical failure plane passing through the organic clay layer.

5.1.7 Final MSE Wall Considerations

Based on the results of the external and global stability analysis performed for the MSE wall, sliding under undrained conditions as well as bearing and global stability under both drained and undrained conditions were not satisfied at a strap length equal to 0.7 times the wall height. Increasing the width of the wall up to 1.0 times the wall height still did not satisfy all of the external and global stability requirements. Calculations for external (bearing and sliding resistance and limiting eccentricity) and overall (global) stability of the MSE wall are provided in Appendix IV.

As noted in Section 5.1, consideration was given to over excavating the existing fill and unsuitable soils and replacing it with granular embankment; however, given the depth and extent of this material the alignment of the wall, this a very expensive and uneconomical option. Therefore, it is recommended that ground improvement techniques be implemented to increase the strength of the soil mass and reduce settlement potential within these layers. Additional considerations for the ground improvement design, including required performance criteria, are provided in Section 5.1.8 below.

5.1.8 Ground Improvement Considerations

The design of the ground improvement should result in the improved soil matrix meeting the design criteria for bearing resistance and compressibility for the MSE wall. The improved soil matrix will need to provide a factored bearing resistance greater than or equal to the factored bearing pressure at the strength limit state of 9.54 ksf. Additionally, the improved soil matrix will need to limit settlement to the required maximum differential settlement of 1/100 along the wall facing and to tolerable limits for maximum settlement of the wall based on the wall manufacturers specifications or for constructability of the roadway. In the absence of specific settlement from the wall manufacturer, the ground improvement design should limit total settlement of the embankment and back of the reinforced soil mass to 5.0 inches, and total settlement at the facing of the wall to 2.5 inches.

As noted above, total settlements of up to approximately 11 to 17 inches at the center of the reinforced soil mass and 7.5 to 10.25 inches at the facing of the wall are anticipated along the alignment of Retaining Wall 4W8 based on a service limit bearing pressure of 6.75 ksf without stabilization of the existing fill and unsuitable soils. About 90 percent of the estimated settlement is occurring within these upper layers. Therefore, it is recommended that the ground improvement elements be extended through the existing fill layers and any underlying compressive (cohesive) layers. Based on the conditions encountered, the ground improvement elements should be extended to an approximate elevation of 695 feet. Additionally, it is recommended that ground improvement elements be located along the length of the leveling pad if concentrated loads will be imparted along the pad to ensure that differential settlement does not occur.

5.2 CIP Wall Recommendations (Retaining Wall 4W9)

It is understood that a CIP retaining wall is being considered as the preferred wall type for Retaining Wall 4W9. Based on the proposed plan and cross section information provided by Dynotec, a maximum wall height of 10.4 feet is anticipated along the alignment. For CIP walls bearing on earthen foundations, footings should be proportioned such that the factored equivalent bearing pressure exerted at the front of the wall will not exceed the factored bearing resistance at the strength limit state. Further, the footings should also be proportioned such that the entire footing width remains in compression (no tensile stresses form under the footing, pulling the footing up and away from the bearing surface). It is understood that the foundations for CIP walls will bear approximately 4.0 to 5.0 feet below the finished grade. In general, the typical width of a CIP wall foundation (B) is equal to 50 to 70 percent the wall height.

It is understood that up to 35 feet of new embankment fill will be placed to bring the site to proposed grade along Ramp C5. Therefore, the bearing soils below retaining wall 4W9 will consist of new embankment fill. As stated in Section 5.1.1, based on the design plans provided by GPD Group and Dynotec, it is understood that Item 203 granular embankment will be utilized where any new embankment will be placed behind the reinforced soil backfill zone for Retaining Wall 4W8.

The shear strength parameters utilized in the external and global stability analysis of the retaining wall are provided in Table 6 from Section 5.1.1.

5.2.1 Bearing Stability

The bearing materials along the proposed alignment will consist of newly placed granular embankment fill. CIP wall foundations bearing on this material may be proportioned for a factored bearing resistance as indicated in Table 10. A geotechnical resistance factor of φ_b=0.55 was considered in calculating the factored bearing resistance at the strength limit state. The foundation width presented in the following table is based on the wall section provided in the design sheets.

Table 10. Retaining Wall 4W9 CIP Wall Design Parameters

From Station [1]	To Station [1]	Representative Borings	Wall Height Analyzed (feet)	Foundation Width [2] (feet)	Bearing Resistance at Strength Limit (ksf)		Strength Limit Equivalent Bearing Pressure [4] (ksf)
					Nominal	Factored [3]	
5079+00	5080+41	N/A	10.4	7.5 (0.72H)	27.34	15.04	2.39

1. *Limits of wall determined from plan information provided by Dynotec Stationing listed is referenced to Ramp C5.*
2. *The foundation width based on the wall section provided in the design sheets.*
3. *A geotechnical resistance factor of φ_b=0.55 was considered in calculating the factored bearing resistance at the strength limit state.*
4. *The strength limit equivalent bearing pressure is the uniformly distributed pressure asserted by the wall over an effective base width based on the eccentricity of the wall system at the strength limit state.*

Rii performed a verification of the bearing pressure exerted on the subgrade material for the maximum specified wall height indicated in Table 10. Based on the minimum footing width presented, the factored equivalent bearing pressure exerted below the wall **will not exceed** the factored bearing resistance at the strength limit state.

Given that the wall will be bearing on approximately 30 to 35 feet of newly placed granular embankment fill, little to no settlement is anticipated under the loading from the proposed wall along the alignment. However, it is recommended that settlement monitoring of the embankment fill placed up to bottom of footing elevation be performed to verify that settlement of the embankment up to that level is complete prior to constructing the wall.

5.2.2 Eccentricity (Overturning Stability)

The resistance of the CIP wall to overturning will be dependent on the on the location of the resultant force at the bottom of the wall due to the overturning and resisting moments acting on the wall. For CIP walls, overturning stability is determined by calculating the eccentricity of the resultant force from the midpoint of the base of the wall and comparing this value to a limiting eccentricity value. Per Section 11.6.3.3 of the 2018 AASHTO LRFD BDS, for foundations bearing on soil, the location of the resultant of the reaction forces shall be within the middle two-thirds ($^2/_3$) of the base width. Therefore, the limiting eccentricity is one-third ($^1/_3$) of the base width of the wall. Based on the required foundation width presented in Table 7 and utilizing the soil parameters listed in Section 5.1.1 for the retained embankment material, the calculated eccentricity of the resultant force **will not exceed** the limiting eccentricity at the strength limit state.

5.2.3 Sliding Stability

The resistance of the CIP wall to sliding was evaluated per Section 11.6.3.6 of the 2018 AASHTO LRFD BDS. Given that the bearing soils along the wall alignment will consist of newly placed granular embankment fill, the bearing resistance was evaluated under drained conditions only. For drained conditions, the sliding resistance is determined by multiplying a coefficient of sliding friction "f" times the total vertical force at the base of the wall. The coefficient of sliding friction is determined based on the friction angle of the foundation soil. Based on the soil parameters listed in Section 5.1.1 for ODOT Item 203 granular embankment, a coefficient of sliding friction of 0.65 was utilized for design. A geotechnical resistance factor of φ_τ=1.0 was considered in calculating the factored shear resistance along the base of the wall. Based on the minimum foundation width presented in Table 7 and utilizing the soil parameters listed in Section 5.1.1 for the retained embankment material, the resultant horizontal forces on the back of the CIP wall **will not exceed** the factored shear resistance at the strength limit state under drained conditions.

5.2.4 Global (Overall) Stability

A slope stability analysis was performed to check the global stability of the wall along the alignment. As per 2018 AASHTO LRFD BDS, safety against soil failure shall be evaluated at the service limit state by assuming the concrete and soil backfill to be a rigid body. Soil parameters utilized in external stability analyses are presented Section 5.1.1. For the global stability condition, it was considered that the failure plane will not cross through any portion of the supported soil mass above the concrete or through the concrete footing itself. The computer software program Slide 7.0 manufactured by Rocscience Inc. was utilized to perform the analyses.

Per Section 11.6.2.3 of the 2018 AASHTO LRFD BDS, overall (global) stability for CIP walls that are not supporting structural foundations or elements is satisfied if the product of the factor of safety from the slope stability output multiplied by the resistance factor φ=0.75 is greater than 1.0. Therefore, global stability is satisfied when a minimum factor of safety of 1.3 is obtained. Based on the recommended footing dimensions listed in Table 7, the resulting factor of safety under drained conditions (long-term stability) using the Spencer's analysis method was greater than 1.3.

5.2.5 Final CIP Wall Considerations

Based on the results of the external and global stability analysis performed for Retaining Wall 4W9, the wall section provided in the design sheet for Wall 4W9, which includes a base width of 7.5 feet (0.73 times the height of the CIP wall) meets all of the external and global stability requirements.

Calculations for external (bearing and sliding resistance and limiting eccentricity) and overall (global) stability of the CIP walls are provided in Appendix VI.

5.3 Lateral Earth Pressure

For the soil types encountered in the borings, the "in-situ" unit weight (γ), cohesion (c), effective angle of friction (φ'), and lateral earth pressure coefficients for at-rest conditions (k_o), active conditions (k_a), and passive conditions (k_p) have been estimated and are provided in Table 11 and Table 12.

Table 11. Estimated Undrained (Short-term) Soil Parameters for Design

Soil Type	γ (pcf) [1]	c (psf)	φ	k_a	k_o	k_p
Soft to Stiff Cohesive Soil	115	1,500	0°	N/A	N/A	N/A
Very Stiff to Hard Cohesive Soil	125	3,000	0°	N/A	N/A	N/A
Loose Granular Soil	120	0	28°	0.32	0.53	5.07
Medium Dense Granular Soil	125	0	32°	0.27	0.47	6.82
Dense to Very Dense Granular Soil	130	0	36°	0.23	0.41	9.09
Compacted Cohesive Engineered Fill	120	2,000	0°	N/A	N/A	N/A
Compacted Granular Engineered Fill	130	0	33°	0.26	0.46	7.41

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

Table 12. Estimated Drained (Long-term) Soil Parameters for Design

Soil Type	γ (pcf) [1]	c (psf)	φ'	k_a	k_o	k_p
Soft to Stiff Cohesive Soil	115	0	26°	0.35	0.56	4.53
Very Stiff to Hard Cohesive Soil	125	50	28°	0.32	0.53	5.07
Loose Granular Soil	120	0	28°	0.32	0.53	5.07
Medium Dense Granular Soil	125	0	32°	0.27	0.47	6.82
Dense to Very Dense Granular Soil	130	0	36°	0.23	0.41	9.09
Compacted Cohesive Engineered Fill	120	0	30°	0.30	0.50	5.58
Compacted Granular Engineered Fill	130	0	33°	0.26	0.46	7.41

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

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

5.4 Construction Considerations

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

5.4.1 Excavation Considerations

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

Table 13. 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
Rock to 3.0' +/- below Auger Refusal	0.75 : 1.0	Above Ground Water Table and No Seepage
Stable Rock	Vertical	Above Ground Water Table and No Seepage

5.4.2 Groundwater Considerations

Based on the groundwater observations made during drilling, groundwater may be encountered during overexcavation of the existing fill material. 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. 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.

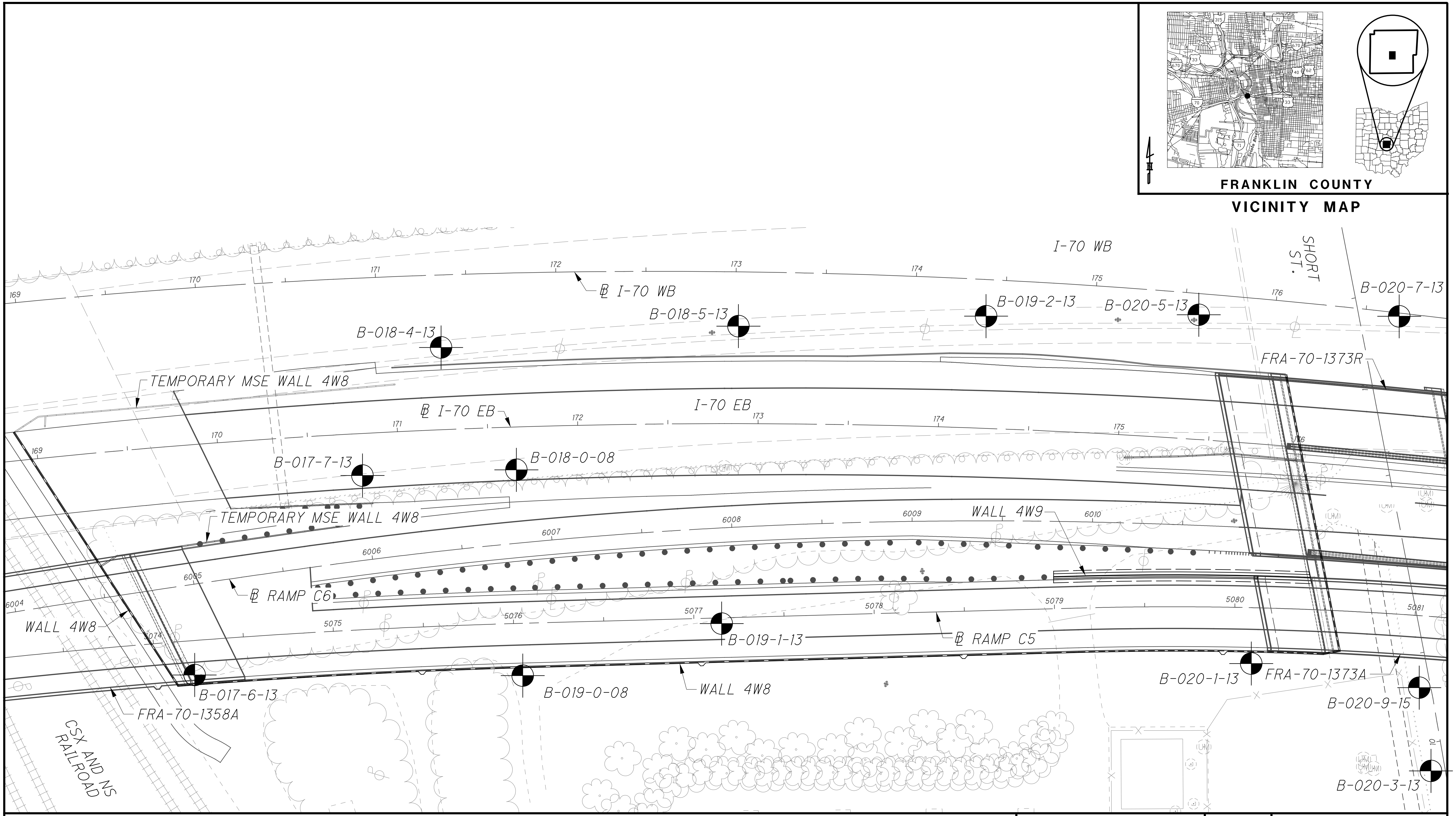
The conclusions and recommendations herein have been based upon the available soil and bedrock information and the design details furnished by a representative of the owner of the proposed project. Any revision in the plans for the proposed construction from those anticipated in this report should be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



VICINITY MAP

BORING PLAN

WALL 4W8 AND WALL 4W9

FRANKLIN COUNTY, OHIO

RII PROJECT NO.
W-13-045

SCALE: 1"=50'
0 25 50

N

DRAWN
RRM

REVIEWED
BRT

DATE
7-12-18

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		$LL_O/LL \times 100$*	% Pass #40	% Pass #200	Liquid Limit (LL)	Plastic Index (PI)	Group Index Max.	REMARKS
		AASHTO	OHIO							
○	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	
FS	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	^	Uncontrolled Fill (Describe)	▚	Bouldery Zone	P	Peat
▩	Pavement or Base						

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

DESCRIPTION OF ROCK TERMS

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

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

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

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

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

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

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

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

Degree of Fracturing

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

Aperture Width

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

Surface Roughness

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

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

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

APPENDIX III

PROJECT BORING LOGS:

B-017-6-13 through B-020-1-13

BORING LOGS

Definitions of Abbreviations

AS = Auger sample

GI = Group index as determined from the Ohio Department of Transportation classification system

HP = Unconfined compressive strength as determined by a hand penetrometer (tons per square foot)

LL_o = Oven-dried liquid limit as determined by ASTM D4318. Per ASTM D2487, if LL_o/LL is less than 75 percent, soil is classified as “organic”.

LOI = Percent organic content (by weight) as determined by ASTM D2974 (loss on ignition test)

PID = Photo-ionization detector reading (parts per million)

QR = Unconfined compressive strength of intact rock core sample as determined by ASTM D2938 (pounds per square inch)

QU = Unconfined compressive strength of soil sample as determined by ASTM D2166 (pounds per square foot)

RC = Rock core sample

REC = Ratio of total length of recovered soil or rock to the total sample length, expressed as a percentage

RQD = Rock quality designation – estimate of the degree of jointing or fracture in a rock mass, expressed as a percentage:

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

S = Sulfate content (parts per million)

SPT = Standard penetration test blow counts, per ASTM D1586. Driving resistance recorded in terms of blows per 6-inch interval while letting a 140-pound hammer free fall 30 inches to drive a 2-inch outer diameter (O.D.) split spoon sampler a total of 18 inches. The second and third intervals are added to obtain the number of blows per foot (N_m).

N_{60} = Measured blow counts corrected to an equivalent (60 percent) energy ratio (ER) by the following equation: $N_{60} = N_m{*}(ER/60)$

SS = Split spoon sample

2S = For instances of no recovery from standard SS interval, a 2.5 inch O.D. split spoon is driven the full length of the standard SS interval plus an additional 6.0 inches to obtain a representative sample. Only the final 6.0 inches of sample is retained. Blow counts from 2S sampling are not correlated with N_{60} values.

3S = Same as 2S, but using a 3.0 inch O.D. split spoon sampler.

TR = Top of rock

W = Initial water level measured during drilling

▼ = Water level measured at completion of drilling

Classification Test Data

Gradation (as defined on Description of Soil Terms):

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

Atterberg Limits:

LL = Liquid limit
PL = Plastic limit
PI = Plasticity Index

WC = Water content (%)

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-12.68 - PHASE 4A
TYPE: STRUCTURE
PID: 77372 BR ID: FRA-70-1358A
START: 12/31/13 END: 1/7/14

DRILLING FIRM / OPERATOR: RII / J.K.
SAMPLING FIRM / LOGGER: RII / K.S.
DRILLING METHOD: 3.25" HSA / RC
SAMPLING METHOD: SPT / NQ

DRILL RIG: MOBILE B-53 (SN 624400)
HAMMER: AUTOMATIC
CALIBRATION DATE: 4/26/13
ENERGY RATIO (%): 77.7

STATION / OFFSET: 5074+21.50 / 19.1' RT
ALIGNMENT: BL RAMP C5
ELEVATION: 714.9 (MSL) EOB: 84.5 ft.
LAT / LONG: 39.952895767, -83.006754692

EXPLORATION ID
B-017-6-13

PAGE
1 OF 3

MATERIAL DESCRIPTION AND NOTES	ELEV. 714.9	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
FILL: MEDIUM DENSE, BLACK **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, DAMP TO MOIST.		1 2	5 6 8	18	44	SS-1	-	-	-	-	-	-	-	-	-	14	A-1-b (V)	
-SLAG, CINDERS AND ROCK FRAGMENTS PRESENT THROUGHOUT		3 4 5	9 10 11	27	100	SS-2	-	34	25	20	18	3	NP	NP	NP	19	A-1-b (0)	
	708.2	6	13 8 4	16	78	SS-3	-	-	-	-	-	-	-	-	-	5	A-1-b (V)	
FILL: STIFF, BROWN **SILT AND CLAY**, LITTLE COARSE TO FINE SAND, LITTLE FINE GRAVEL, DAMP.		7					2.00	13	4	13	42	28	33	18	15	17	A-6a (9)	
-ORGANICS PRESENT THROUGHOUT	705.5	8 9	22 13 9	28	89	SS-4	1.75	-	-	-	-	-	-	-	-	15	A-6a (V)	
FILL: MEDIUM DENSE, BROWN **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, DAMP. -SLAG, CINDERS AND ORGANICS PRESENT THROUGHOUT	704.4	10					-	52	15	13	15	5	NP	NP	NP	6	A-1-b (0)	
STIFF, REDDISH BROWN **SILT AND CLAY**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST.	701.9	11 12 13	4 4 5	12	33	SS-5	1.50	-	-	-	-	-	-	-	-	21	A-6a (V)	
MEDIUM DENSE, BLACK AND BROWN **COARSE AND FINE SAND**, LITTLE SILT, TRACE CLAY, TRACE FINE GRAVEL, MOIST.	699.4	14 15	5 6 8	18	100	SS-6	-	4	28	47	16	5	NP	NP	NP	10	A-3a (0)	
VERY STIFF, BROWN **SANDY SILT**, SOME FINE GRAVEL, TRACE CLAY, DAMP.	696.9	16 17	7 9 10	25	89	SS-7	-	27	15	14	35	9	24	19	5	14	A-4a (2)	
MEDIUM DENSE TO VERY DENSE, BROWN **GRAVEL AND SAND**, TRACE TO LITTLE SILT, TRACE CLAY, DAMP TO MOIST.		18 19 20	14 10 11	27	83	SS-8	-	-	-	-	-	-	-	-	-	11	A-1-b (V)	
		21 22 23	11 14 25	51	89	SS-9	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	
		24 25	29 26 26	67	78	SS-10	-	57	17	9	12	5	22	17	5	9	A-1-b (0)	
		26 27	18 16 16	41	67	SS-11	-	-	-	-	-	-	-	-	-	13	A-1-b (V)	
		W 28 29	8 9 10	25	83	SS-12	-	-	-	-	-	-	-	-	-	15	A-1-b (V)	

PID: 77372	BR ID: FRA-70-1358A	PROJECT: FRA-70-12.68 - PHASE 4A	STATION / OFFSET: 5074+21.50 / 19.1 RT	START: 12/31/13	END: 1/7/14	PG 2 OF 3	B-017-6-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 684.9	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
MEDIUM DENSE TO VERY DENSE, BROWN **GRAVEL AND SAND**, TRACE TO LITTLE SILT, TRACE CLAY, DAMP TO MOIST. *(same as above)*		31 32 33																
		34 35	2 6 12	23	67	SS-13	-	10	47	33	8	2	NP	NP	NP	19	A-1-b (0)	
	677.9	36 37																
VERY STIFF TO HARD, GRAY **ELASTIC CLAY**, TRACE SILT, TRACE COARSE SAND, DAMP.		38																
		39 40	2 6 12	23	67	SS-14	3.00	-	-	-	-	-	-	-	-	24	A-7-5 (V)	
		41 42 43																
		44 45	7 10 13	30	83	SS-15	3.75	-	-	-	-	-	-	-	-	27	A-7-5 (V)	
		46 47 48																
		49 50	8 10 12	28	100	SS-16	3.75	0	1	0	10	89	57	34	23	27	A-7-5 (17)	
		51 52 53																
		54 55	12 18 32	65	100	SS-17	4.50	-	-	-	-	-	-	-	-	25	A-7-5 (V)	
	657.9	56 57																
VERY DENSE, GRAY AND BLACK **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, DAMP.		58																
		59	50/4"	-	75	SS-18	-	-	-	-	-	-	-	-	-	8	A-1-b (V)	
	652.9	60 61																

2014 ODOT BORING LOG-RII NE BRIDGE ID - OH DOT.GDT - 4/2/15 17:50 - U:\GI8\PROJECTS\2013\W-13-045.GPJ

PID: 77372	BR ID: FRA-70-1358A	PROJECT: FRA-70-12.68 - PHASE 4A	STATION / OFFSET: 5074+21.50 / 19.1 RT	START: 12/31/13	END: 1/7/14	PG 3 OF 3	B-017-6-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 652.8	DEPTHS		SPT/ RQD	N60	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	HOLE SEALED
									GR	CS	FS	SI	CL	LL	PL	PI			
HARD, GRAY **CLAY**, "AND" FINE GRAVEL, LITTLE SILT, TRACE COARSE TO FINE SAND, DRY. *(same as above)*	650.6	TR	63 64	32 50/3"	-	22	SS-19	4.5+	56	1	1	15	27	51	23	28	10	A-7-6 (7)	
SHALE : GRAY, HIGHLY WEATHERED, VERY WEAK.			65 66 67 68 69	33 50/3"	-	56	SS-20	-	-	-	-	-	-	-	-	-	12	Rock (V)	
			70 71 72 73 74	50/3"	-	100	SS-21	-	-	-	-	-	-	-	-	-	16	Rock (V)	
AUGER REFUSAL @ 74.5'	640.4																		
LIMESTONE : LIGHT BROWN AND GRAY, UNWEATHERED, VERY STRONG, VERY THICK BEDDED, DOLOMITIC, CALCAREOUS, CHERTY, MICACEOUS, SLIGHTLY TO HIGHLY FRACTURED, OPEN APERTURES, ROUGH; RQD 92%, REC 97%. -QU @ 75.6' = 12,261 PSI			75 76 77 78 79	83		93	RC-1											CORE	
			80 81 82 83 84	100		100	RC-2											CORE	
	630.4	EOB																	

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 28.0'

ABANDONMENT METHODS, MATERIALS, QUANTITIES: TREMIED 282 LBS CEMENT / 50 LBS BENTONITE POWDER / 50 GAL WATER

B-017-6-13 – RC-1 and RC-2 – Depth from 74.5 to 84.5 feet

PROJECT: FRA-70-12.68 - PHASE 4A	DRILLING FIRM / OPERATOR: RII / J.B.	DRILL RIG: MOBILE B-53 (SN 624400)	STATION / OFFSET: 170+79.36 / 23.3' RT	EXPLORATION ID B-017-7-13
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: 743.1 (MSL) EOB: 96.7 ft.	PAGE 1 OF 4
START: 8/4/13 END: 8/7/13	SAMPLING METHOD: SPT / HQ	ENERGY RATIO (%): 77.7	LAT / LONG: 39.953200568, -83.006425064	

MATERIAL DESCRIPTION AND NOTES	ELEV. 743.1
0.9' - ASPHALT (11.0")	742.2
0.5' - AGGREGATE BASE (6.0")	741.7
FILL: MEDIUM DENSE, GRAY **GRAVEL**, LITTLE FINE TO COARSE SAND, TRACE SILT, TRACE CLAY, MOIST.	739.1
FILL: STIFF TO VERY STIFF, BROWNISH GRAY TO BROWN **SILT AND CLAY**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP.	735.1
FILL: VERY DENSE, BROWN **GRAVEL**, TRACE SILT, DAMP.	732.6
FILL: STIFF TO HARD, BROWN TO DARK BROWNISH GRAY **SILTY CLAY**, SOME FINE GRAVEL, LITTLE COARSE TO FINE SAND, DRY TO MOIST.	

DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)
1.5-3	4 5 6	14	67	SS-1	-	69	13	5	10	3	NP	NP	NP	7	A-1-a (0)
3.5-5	3 3 5	10	50	SS-2	-	-	-	-	-	-	-	-	-	9	A-1-a (V)
					3.00	-	-	-	-	-	-	-	-	13	A-6a (V)
6-7.5	6 8 12	26	50	SS-3	2.00	-	-	-	-	-	-	-	-	12	A-6a (V)
8.5-10	18 25 25	65	44	SS-4	-	-	-	-	-	-	-	-	-	5	A-1-a (V)
11-12.5	3 9 10	25	50	SS-5	2.00	-	-	-	-	-	-	-	-	12	A-6b (V)
13.5-15	1 2 5	9	56	SS-6	1.50	31	11	8	30	20	37	18	19	19	A-6b (6)
16-17.5	5 15 15	39	39	SS-7	2.00	-	-	-	-	-	-	-	-	9	A-6b (V)
18.5-20	15 50 28	101	39	SS-8	2.00	-	-	-	-	-	-	-	-	19	A-6b (V)
21-22.5	6 12 9	27	56	SS-9	1.75	-	-	-	-	-	-	-	-	20	A-6b (V)
23.5-25	21 6 8	18	56	SS-10	4.50	24	10	9	33	24	40	20	20	18	A-6b (8)
26-27.5	7 8 10	23	83	SS-11	2.50	-	-	-	-	-	-	-	-	16	A-6b (V)
28.5-30	WOH 2 10	16	72	SS-12	2.75	-	-	-	-	-	-	-	-	18	A-6b (V)

BACK FILL

PID: 77372 | BR ID: FRA-70-1358R | PROJECT: FRA-70-12.68 - PHASE 4A | STATION / OFFSET: 170+79.36 / 23.3 RT | START: 8/4/13 | END: 8/7/13 | PG 2 OF 4 | B-017-7-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 713.1	DEPTHS		SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	BACK FILL
									GR	CS	FS	SI	CL	LL	PL	PI			
FILL: STIFF TO HARD, BROWN TO DARK BROWNISH GRAY **SILTY CLAY**, SOME FINE GRAVEL, LITTLE COARSE TO FINE SAND, DRY TO MOIST. *(same as above)*	711.1		31 32																
FILL: STIFF, DARK BROWN **SILT AND CLAY**, SOME COARSE TO FINE SAND, SOME FINE GRAVEL, DAMP.			33 34 35	4 4 5	12	50	SS-13	1.75	24	14	12	31	19	33	18	15	20	A-6a (5)	
-WOOD FRAGMENTS RECOVERED FROM 38.5' TO 39.0'	704.1		36 37 38																
HARD, DARK BROWN **SILTY CLAY**, SOME FINE TO COARSE SAND, SOME FINE GRAVEL, MOIST.			39 40	10 8 8	21	33	SS-14	- 4.50	- -	- -	- -	- -	- -	- -	- -	- -	110 13	 A-6b (V)	
	701.1		41 42																
MEDIUM DENSE TO DENSE, BROWN **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, MOIST.			43 44 45	2 8 11	25	78	SS-15	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	
		W	46 47 48 49 50	10 14 16	39	61	SS-16	-	52	18	10	16	4	21	18	3	10	A-1-b (0)	
			51 52 53 54 55	20 30 45	97	72	SS-17	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	
	686.1	W	56 57																
VERY DENSE, GRAY **GRAVEL WITH SAND AND SILT**, TRACE SILT, MOIST.			58 59 60	41 21 50/4"	-	100	SS-18	-	-	-	-	-	-	-	-	-	17	A-2-4 (V)	
-COBBLES PRESENT @ 60.0'			61																

PID: 77372 | BR ID: FRA-70-1358R | PROJECT: FRA-70-12.68 - PHASE 4A | STATION / OFFSET: 170+79.36 / 23.3 RT | START: 8/4/13 | END: 8/7/13 | PG 3 OF 4 | B-017-7-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 681.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI			
VERY DENSE, GRAY **GRAVEL WITH SAND AND SILT**, TRACE SILT, MOIST. *(same as above)*		63																
		64	8 20 20	52	83	SS-19	-	-	-	-	-	-	-	-	-	17	A-2-4 (V)	
		65																
		66																
	676.1	67																
HARD, GRAY **CLAY**, SOME SILT, TRACE COARSE TO FINE SAND, DAMP.		68																
		69	8 9 16	32	78	SS-20	4.5+	-	-	-	-	-	-	-	-	20	A-7-6 (V)	
		70																
		71																
		72																
		73																
		74	12 14 20	44	89	SS-21	4.5+	-	-	-	-	-	-	-	-	17	A-7-6 (V)	
		75																
		76																
		77																
		78																
		79	12 22 50	93	44	SS-22	4.5+	-	-	-	-	-	-	-	-	17	A-7-6 (V)	
		80																
		81																
		82																
		83																
		84	40 35 50/4"	-	81	SS-23	4.5+	0	2	5	34	59	54	25	29	15	A-7-6 (18)	
		85																
		86																
		87																
		88																
AUGER REFUSAL @ 90.0'		89	25 50/4"	-	100	SS-24	4.5+	-	-	-	-	-	-	-	-	15	A-7-6 (V)	
MUDSTONE : GRAY, SLIGHTLY WEATHERED, VERY WEAK, THINLY LAMINATED TO LAMINATED, FRIABLE, FISSILE, HIGHLY FRACTURED TO FRACTURED, OPEN APERTURE, ROUGH; RQD 73%, REC 100%.	653.1	TR 90	79		100	RC-1											CORE	
	651.1	91	66		100	RC-2											CORE	
		92																
		93																
		94																

PID: 77372 | BR ID: FRA-70-1358R | PROJECT: FRA-70-12.68 - PHASE 4A | STATION / OFFSET: 170+79.36 / 23.3 RT | START: 8/4/13 | END: 8/7/13 | PG 4 OF 4 | B-017-7-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 648.8	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
SHALE : GRAY TO BLACK, HIGHLY WEATHERED, VERY WEAK, THINLY LAMINATED TO LAMINATED, FRIABLE, MODERATELY TO HIGHLY FRACTURED, OPEN APERTURE, SLIGHTLY ROUGH TO ROUGH; RQD 15%, REC 74%. *(same as above)*	646.4	95 96 EOB	15		74	RC-3											CORE	

NOTES: SEEPAGE ENCOUNTERED @ 48.5'; GROUNDWATER ENCOUNTERED INITIALLY @ 57.0'

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PUMPED 188 LBS PORTLAND CEMENT / 50 LBS BENTONITE POWDER / 50 GAL WATER

B-017-7-13 – RC-1, RC-2, and RC-3 – Depth from 90.0 to 96.7 feet

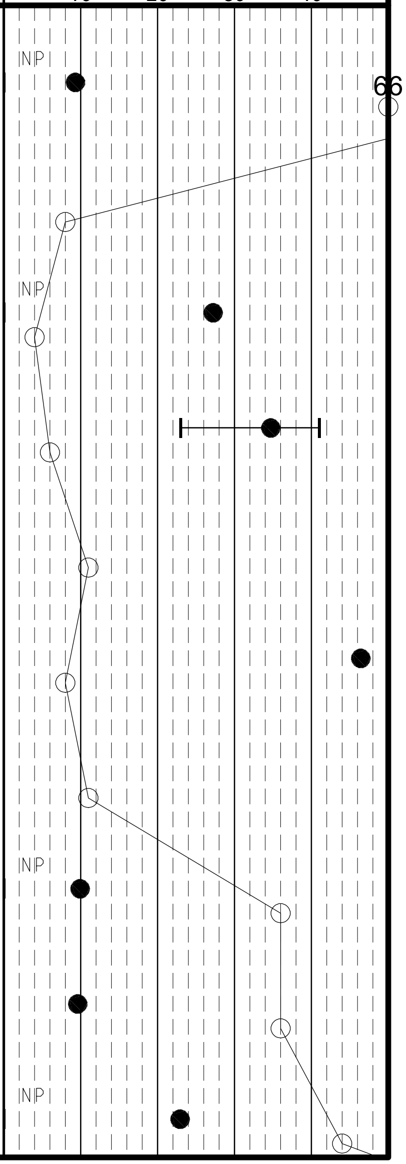
Client: ms consultants | *Project:* FRA-70-8.93 | *Job No.* 0221-1004.01

LOG OF: Boring B-019-0-08 | *Location:* Sta. 5076+04.30, 29.0' RT., RAMP C5 | *Date Drilled:* 6/30/2008

WATER OBSERVATIONS:
Water seepage at: 12.5'-50.0'
Water level at completion: 12.0' (prior to coring)
11.0' (24 hours after completion)
FIELD NOTES:
Advanced boring using 3.25" diameter hollowstem augers.

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	Graphic Log	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
	713.5													
1.0	712.5						Aggregate road base - 12"							
		9 28 37	16	1			FILL: Very dense black GRAVEL WITH SAND (A-1-b), little silt; contains coal particles and cinders; damp.		21	30	---	35	--14--	
4.0	709.5	9 5 3	2	2			POSSIBLE FILL: Loose black GRAVEL WITH SAND (A-1-b), little silty clay; damp.							
		2 2 2	11	3					37	27	---	26	--10--	
8.0	705.5	2 2 4	11	4		1.5	POSSIBLE FILL: Stiff dark brown to black ORGANIC CLAY (A-8b), trace fine sand; moderately organic; damp. @ 8.5', LOI @ 440 deg = 7.2%, LOI @ 750 deg = 9.1 %.		0	4	---	11	57	28
		10 6 5	8	5		--	@ 11.0', encountered large, decomposing wood fragment.							
		5 4 4	3	6		--								
15.5	698.0	8 7 4	8	7			POSSIBLE FILL: Medium dense brown GRAVEL WITH SAND (A-1-b), trace silty clay; wet. @ 16.0', encountered large, decomposing wood fragment.							
18.0	695.5	10 17 18	12	8			Dense brown and gray GRAVEL (A-1-a), some fine to coarse sand, little silty clay; wet.		64	12	---	11	--13--	
		11 15 20	12	9										
23.0	690.5	21 22 21	7	10			Dense dark brown and gray GRAVEL WITH SAND (A-1-b), trace silty clay; wet.		26	39	---	30	--5--	
25	688.5													

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



DLZ Ohio, Inc. * 6121 Huntley Road, Columbus, Ohio 43229 * (614) 888-0040

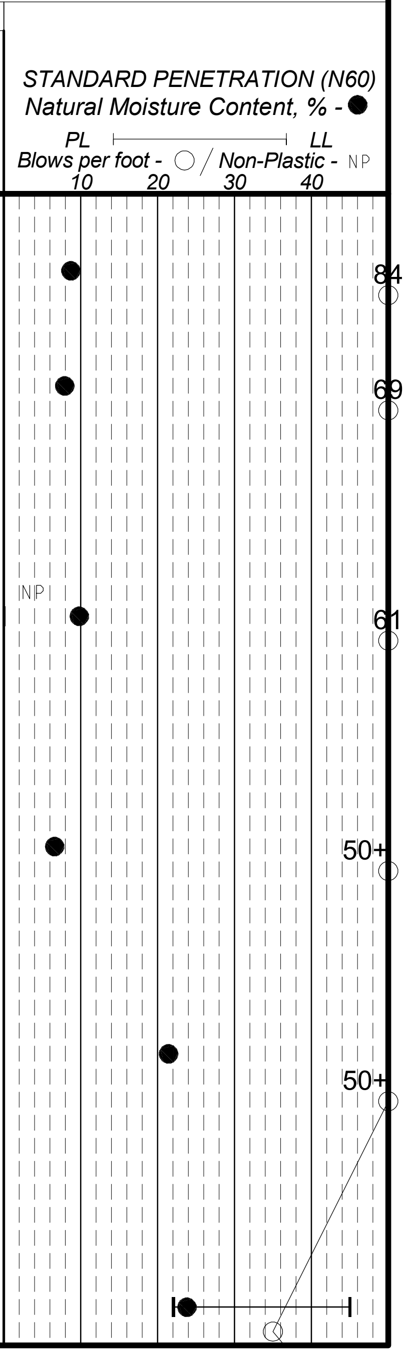
Client: ms consultants | *Project:* FRA-70-8.93 | *Job No.* 0221-1004.01

LOG OF: Boring B-019-0-08 | *Location:* Sta. 5076+04.30, 29.0' RT., RAMP C5 | *Date Drilled:* 6/30/2008

WATER OBSERVATIONS:
Water seepage at: 12.5'-50.0'
Water level at completion: 12.0' (prior to coring)
11.0' (24 hours after completion)

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

Depth (ft)	Elev. (ft) 688.5	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
							Very dense brown and gray GRAVEL WITH SAND (A-1-b), trace silty clay; wet. @ 26.0', encountered 1.8 feet sand heave.						
		14 36 46	12	11									
30		25 32 36	10	12									
32.0	681.5						Very dense brownish gray GRAVEL (A-1-a), some fine to coarse sand, trace silty clay; wet. @ 33.5', 38.5', encountered 2 feet sand heave.						
35		22 28 32	9	13				66	22	---	8	--4--	
40		32 30 50/5	15	14									
45		50/3	3	15			@ 43.5', possible cobbles.						
47.0	666.5						Very stiff brownish gray CLAY (A-7-6), trace to little silt, trace fine sand; damp to moist.						
50	663.5	8 12 22	10	16		3.5		0	0	---	0	9	91



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

LOG OF: Boring B-019-0-08 | *Location:* Sta. 5076+04.30, 29.0' RT., RAMP C5 | *Date Drilled:* 6/30/2008

WATER OBSERVATIONS:
Water seepage at: 12.5'-50.0'
Water level at completion: 12.0' (prior to coring)
11.0' (24 hours after completion)

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

Depth (ft)	Elev. (ft) 663.5	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	STANDARD PENETRATION (N60)
							Very stiff brownish gray CLAY (A-7-6), trace to little silt, trace fine sand; damp to moist.	
54.0	659.5	8 23 36	17	17		4.5+	Shale, blue-gray, severely weathered, weak to very weak. @ 55.0'-55.8', core loss.	60
55		Core 10"	Rec 0"	RQD 0%	R1			
55.8	657.7	Core 60"	Rec 28"	RQD 23%	R2		Interbedded Shale (90%) and Limestone (10%) RQD 19% Loss 60%. Shale, light gray to blue-gray, highly to severely weathered, very weak to weak, slightly calcareous, laminated, fissile, fractured to highly fractured; Limestone, light gray, moderately weathered, moderately strong to strong, fractured. @ 55.8'-60.8', core barrel damaged while coring.	
60		Core 60"	Rec 23"	RQD 17%	R3		@ 62.3'-62.5', encountered large pyritic inclusion. @ 62.5'-65.8', core loss. Pyrite lodged in core barrel, eroded sample.	
65								
65.8	647.7	Core 60"	Rec 60"	RQD 67%	R4		Shale dark gray highly weathered, weak, laminated, slighly calcareous, contains abundant pyritic inclusions, fissile, moderately fractured to fractured, RQD 45%, Loss 25%.	
70		Core 60"	Rec 30"	RQD 22%	R5		@ 71.0'-71.3', encountered large pyritic inclusion.	
75	638.5							

GRADATION: % Aggregate, % C. Sand, % M. Sand, % F. Sand, % Silt, % Clay

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-019-0-08 | *Location:* Sta. 5076+04.30, 29.0' RT., RAMP C5 | *Date Drilled:* 6/30/2008

WATER OBSERVATIONS:
Water seepage at: *12.5'-50.0'*
Water level at completion: *12.0' (prior to coring)*
11.0' (24 hours after completion)

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

Depth (ft)	Elev. (ft) 638.5	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	Graphic Log	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N60) Natural Moisture Content, % - ● PL ⊢⊣ LL Blows per foot - ○ / Non-Plastic - NP 10 20 30 40
75.8	637.7	Core 60"	Rec 30"	RQD 22%											
							Bottom of Boring - 75.8'								
80															
85															
90															
95															
100															

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-12.68 - PHASE 4A	DRILLING FIRM / OPERATOR: STOCK / A/J	DRILL RIG: CME-750X (SN 375128)	STATION / OFFSET: 5077+15.33 / 3.2' RT	**EXPLORATION ID** **B-019-1-13**
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / K.R.	HAMMER: CME AUTOMATIC	ALIGNMENT: BL RAMP C5	
PID: 77372 BR ID: N/A	DRILLING METHOD: 4.25" HSA	CALIBRATION DATE: 3/28/13	ELEVATION: 712.5 (MSL) EOB: 50.0 ft.	PAGE 1 OF 2
START: 7/8/13 END: 7/11/13	SAMPLING METHOD: SPT	ENERGY RATIO (%): 78.6	LAT / LONG: 39.952978442, -83.005713188	

MATERIAL DESCRIPTION AND NOTES	ELEV. 712.5	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
1.0' - TOPSOIL (12.0")	711.5																	
FILL: LOOSE TO MEDIUM DENSE, DARK BROWN TO BLACK AND GRAY **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST. -BRICK AND ASPHALT FRAGMENTS PRESENT THROUGHOUT		1 2 3	20 13 10	30	94	SS-1	-	-	-	-	-	-	-	-	-	13	A-1-b (V)	
	707.0	4 5	3 3 4	9	44	SS-2	-	35	24	17	16	8	21	16	5	10	A-1-b (0)	
FILL: LOOSE, BROWN **SANDY SILT**, SOME FINE GRAVEL, TRACE CLAY, WET. -STONE AND CONCRETE FRAGMENTS PRESENT IN SS-3	704.5	6 7 8	6 4 2	8	83	SS-3	-	-	-	-	-	-	-	-	-	29	A-4a (V)	
POSSIBLE FILL: SOFT TO MEDIUM STIFF, DARK BROWNISH GRAY **SILTY CLAY**, "AND" COARSE TO FINE SAND, LITTLE FINE GRAVEL, MOIST.		9 10	2 3 6	12	94	SS-4	0.75	-	-	-	-	-	-	-	-	26	A-6b (V)	
	699.5	11 12 13	3 4 3	9	83	SS-5	0.50	16	25	20	15	24	40	16	24	19	A-6b (4)	
MEDIUM DENSE, BROWN **GRAVEL WITH SAND AND SILT**, TRACE CLAY, MOIST. -STONE FRAGMENTS PRESENT IN SS-6		14 15	4 5 6	14	69	SS-6	-	-	-	-	-	-	-	-	-	12	A-2-4 (V)	
		16 17 18	5 5 7	16	67	SS-7	-	43	19	11	21	6	NP	NP	NP	13	A-2-4 (0)	
	690.5	19 20 21 22	6 6 6	16	86	SS-8	-	-	-	-	-	-	-	-	-	10	A-2-4 (V)	
MEDIUM DENSE, BROWN **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST. -COBBLES PRESENT THROUGHOUT		23 W 24 25 26 27 28	7 6 8	18	78	SS-9	-	55	17	7	17	4	NP	NP	NP	16	A-1-b (0)	
-HEAVING SANDS ENCOUNTERED @ 28.5'		29	5 3 14	22	100	SS-10	-	-	-	-	-	-	-	-	-	17	A-1-b (V)	

PID: 77372	BR ID: N/A	PROJECT: FRA-70-12.68 - PHASE 4A	STATION / OFFSET: 5077+15.33 / 3.2 RT	START: 7/8/13	END: 7/11/13	PG 2 OF 2	B-019-1-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 682.5	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
MEDIUM DENSE, BROWN **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST. *(same as above)*	680.5	31 – 32																
VERY DENSE, GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, MOIST. -DROVE SPOON ON COBBLE @ 33.5' -INTRODUCED MUD @ 35.0'		33 – 34	50/2"	-	0	SS-11	-	-	-	-	-	-	-	-	-	-		
		39 – 40	7 30 28	76	67	SS-12	-	-	-	-	-	-	-	-	-	9	A-1-a (V)	
	670.5	41 – 42																
HARD, GRAY TO MOTTLED BROWN AND GRAY **SILTY CLAY**, TRACE COARSE AND FINE SAND, TRACE FINE GRAVEL, MOIST.		44 – 45	3 8 26	45	89	SS-13	4.5+	1	2	2	32	63	33	16	17	20	A-6b (11)	
	662.5	49 – 50 EOB	5 9 11	26	100	SS-14	4.5+	-	-	-	-	-	-	-	-	23	A-6b (V)	

NOTES: GROUNDWATER INITIALLY ENCOUNTERED @ 23.5'; CAVE-IN DEPTH @ 32.0'

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PUMPED 94 LBS PORTLAND CEMENT / 100 LBS BENTONITE POWDER / 50 GAL WATER

RESOURCE INTERNATIONAL, INC.

PROJECT:	FRA-70-12.68 - PHASE 4A	DRILLING FIRM / OPERATOR:	STOCK / A/M	DRILL RIG:	CME 750X (SN 375128)	STATION / OFFSET:	5080+09.80 / 30.9' RT
TYPE:	STRUCTURE	SAMPLING FIRM / LOGGER:	RII / K.R.	HAMMER:	AUTOMATIC	ALIGNMENT:	BL RAMP C5
PID: 77372	BR ID: FRA-70-1373A	DRILLING METHOD:	4.25" HSA / RC	CALIBRATION DATE:	3/28/13	ELEVATION:	712.8 (MSL) EOB: 86.0 ft.
START: 7/1/13	END: 7/3/13	SAMPLING METHOD:	SPT / NQ	ENERGY RATIO (%):	78.6	LAT / LONG:	39.952922218, -83.004665587

EXPLORATION ID
B-020-1-13

PAGE
1 OF 3

MATERIAL DESCRIPTION AND NOTES	ELEV. 712.8
0.5' - TOPSOIL (6.0")	712.3
0.3' - BRICK (4.0")	712.0
FILL: HARD, BROWN **SILT AND CLAY**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP. -ROOT AND GRASS FIBERS PRESENT IN SS-1	709.8
POSSIBLE FILL: MEDIUM DENSE, GRAY **GRAVEL WITH SAND AND SILT**, TRACE CLAY, MOIST.	707.3
POSSIBLE FILL: SOFT TO STIFF, DARK BROWN **SILT AND CLAY**, SOME COARSE TO FINE SAND, LITTLE FINE GRAVEL, DAMP TO MOIST. -STONE FRAGMENTS PRESENT IN SS-4	702.3
POSSIBLE FILL: STIFF TO VERY STIFF, DARK BROWNISH GRAY TO BROWN **SILTY CLAY**, SOME COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST.	697.3
POSSIBLE FILL: VERY LOOSE, DARK BROWN **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, WET.	694.8
MEDIUM DENSE, DARK BROWN **GRAVEL WITH SAND, SILT, AND CLAY**, MOIST. -STONE FRAGMENTS PRESENT IN SS-8	692.3
MEDIUM DENSE TO DENSE, BROWN **GRAVEL AND SAND**, LITTLE CLAY, TRACE SILT, MOIST. -COBBLES PRESENT @ 22.0'	

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

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)
6 50/5"	-	55	SS-1	-	-	-	-	-	-	-	-	-	11	A-6a (V)
15 11 5	21	81	SS-2	-	-	-	-	-	-	-	-	-	12	A-2-4 (V)
2 1 2	4	64	SS-3	0.50	20	13	13	22	32	35	20	15	20	A-6a (6)
2 5 3	10	33	SS-4	-	-	-	-	-	-	-	-	-	12	A-6a (V)
1 2 4	8	56	SS-5	2.75	-	-	-	-	-	-	-	-	23	A-6b (V)
2 3 3	8	78	SS-6	2.00	6	9	12	23	50	40	17	23	19	A-6b (13)
1 1 2	4	67	SS-7	-	-	-	-	-	-	-	-	-	21	A-1-b (V)
5 9 6	20	39	SS-8	-	-	-	-	-	-	-	-	-	15	A-2-6 (V)
10 7 5	16	39	SS-9	-	-	-	-	-	-	-	-	-	17	A-1-b (V)
3 6 8	18	53	SS-10	-	-	-	-	-	-	-	-	-	15	A-1-b (V)
5 4 8	16	56	SS-11	-	36	30	12	9	13	NP	NP	NP	15	A-1-b (0)
8 16 16	42	83	SS-12	-	-	-	-	-	-	-	-	-	14	A-1-b (V)

BACK FILL

PID: 77372	BR ID: FRA-70-1373A	PROJECT: FRA-70-12.68 - PHASE 4A	STATION / OFFSET: 5080+09.80 / 30.9 RT	START: 7/1/13	END: 7/3/13	PG 2 OF 3	B-020-1-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 682.8	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI			
MEDIUM DENSE TO DENSE, BROWN **GRAVEL AND SAND**, LITTLE CLAY, TRACE SILT, MOIST. *(same as above)*	680.8	31																
HARD, GRAY **SANDY SILT**, SOME CLAY, LITTLE FINE GRAVEL, DAMP.		32 33																
		34	15 18 25	56	100	SS-13	4.5+	20	13	17	21	29	22	13	9	10	A-4a (3)	
-HEAVING SANDS ENCOUNTERED @ 35.0'	675.8	35 36																
VERY DENSE, GRAY TO BROWNISH GRAY **GRAVEL AND SAND**, TRACE SILT, DAMP TO MOIST.		37 38																
		39	7 13 34	62	100	SS-14	-	-	-	-	-	-	-	-	-	4	A-1-b (V)	
-INTRODUCED WATER @ 40.0'		40 41 42 43																
		44	20 43 50/5"	-	100	SS-15	-	-	-	-	-	-	-	-	-	11	A-1-b (V)	
		45 46 47 48																
		49	17 41 50/3"	-	100	SS-16	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	
	660.8	50 51																
HARD, GRAY **CLAY**, LITTLE SILT, TRACE COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP.		52 53																
-SHALE FRAGMENTS PRESENT IN SS-17		54	18 30 25	72	100	SS-17	-	-	-	-	-	-	-	-	-	14	A-7-6 (V)	
		55 56 57 58																
		59	30 26 33	77	78	SS-18	4.5+	6	1	8	18	67	44	21	23	14	A-7-6 (14)	
		60 61																

2014 ODOT BORING LOG-RII NE BRIDGE ID - OH DOT.GDT - 3/31/15 08:54 - U:\GI8\PROJECTS\2013\W-13-045.GPJ

PID: 77372	BR ID: FRA-70-1373A	PROJECT: FRA-70-12.68 - PHASE 4A	STATION / OFFSET: 5080+09.80 / 30.9 RT	START: 7/1/13	END: 7/3/13	PG 3 OF 3	B-020-1-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 650.7	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI			
HARD, GRAY **CLAY**, LITTLE SILT, TRACE COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP. *(same as above)*	648.6	63 64 TR	24 50/2"	-	100	SS-19	4.50	-	-	-	-	-	-	-	-	14	A-7-6 (V)	
SHALE: GRAY, VERY WEAK, HIGHLY WEATHERED. -AUGER REFUSAL @ 65.0'. ATTEMPTED 10.0' ROCK CORE RUN. NO RECOVERY FROM CORE RUN. MAY HAVE WASHED OUT HIGHLY WEATHERED SHALE MATERIAL DURING THE CORING OPERATION. CONTINUED SPT SAMPLING @ 75.0'.		65 66 67 68 69 70 71 72 73 74	0		0	RC-1											CORE	
		75 76 77 78 79	50/4"	-	100	SS-20	-	-	-	-	-	-	-	-	-	15	Rock (V)	
AUGER REFUSAL @ 80.5'	632.3	80	50/1"	-	0	SS-21	-	-	-	-	-	-	-	-	-	-		
LIMESTONE : LIGHT AND DARK GRAY, UNWEATHERED, STRONG, VERY THIN TO THIN BEDDED, CRYSTALLINE, DOLOMITIC, MODERATELY FRACTURED TO FRACTURED, OPEN APERTURE, VERY ROUGH; RQD 49%, REC 99%. -QU @ 80.7' = 9,465 PSI -CHERT NODULES PRESENT THROUGHOUT	626.8	81 82 83 84 85 EOB 86	49		99	RC-2											CORE	

NOTES: GROUNDWATER INITIALLY ENCOUNTERED @ 23.0'

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PUMPED 376 PORTLAND CEMENT / 200 LBS BENTONITE POWDER / 150 GAL WATER

B-020-1-13 – RC-1 – Depth from 65.0 to 75.0 feet

B-020-1-13 – RC-2 – Depth from 81.0 to 86.0 feet

APPENDIX IV

UNCONFINED COMPRESSION TEST RESULTS

RESOURCE INTERNATIONAL, INC.
Engineering Consultants

Unconfined Compressive Strength
of Intact Rock Core Specimens (ASTM D 7012-04)

6350 Presidential Gatew.
Columbus, OH 43231
Phone (614) 823-4949

9885 Rockside Road
Cleveland, OH 44125
Phone (216) 573-0955

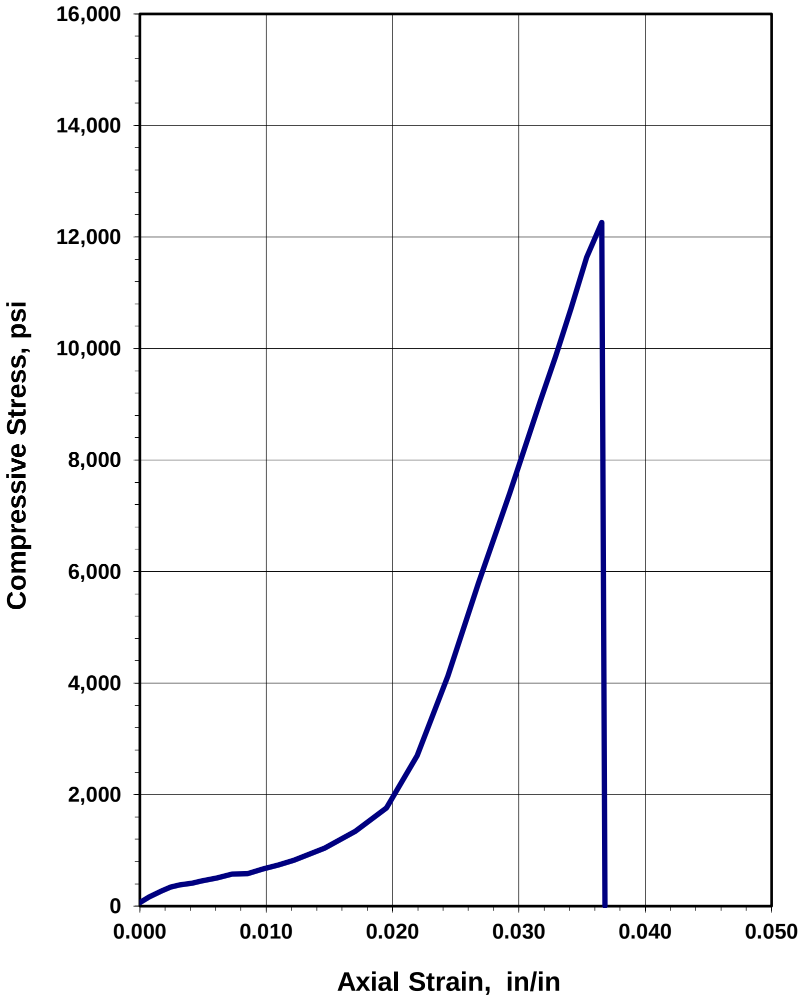
4480 Lake Forest Drive
Cincinnati, Ohio 45242
Phone (513) 769-6998

Project: FRA-70-12.68
Project No.: W-13-045
Date of Testing: 2/13/2014
Test Performed by: J.H./T.K.

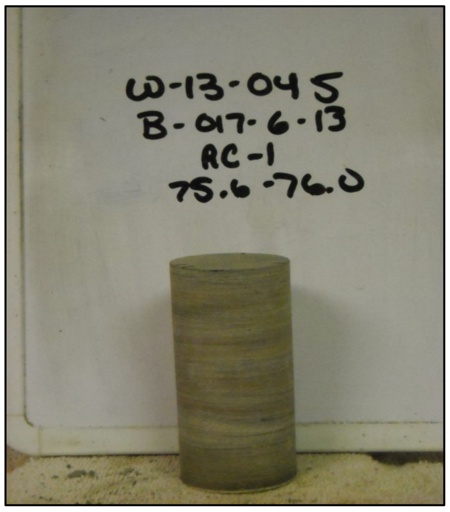
Rock Description: LIMESTONE: Light brown, unweathered, very strong.

Boring No.:	B-017-6-13	Average Length:	4.102 in
Station / Offset:	5074+21.50, 19.1' Rt.	Average Diameter:	1.87 in
Sample No. / Depth:	RC-1 / 75.6 ft.	Length to diameter ratio:	2.194
Moisture condition:	As received	Cross Sectional Area:	2.745 in^2
Rate of Loading:	62.1 lbs/sec	Failure Load:	33,670 lbs
Testing Time:	542 sec	Axial Strain at Failure:	0.0366 in/in
(Rate 2-15 minutes to failure)		Stress:	12,261 psi

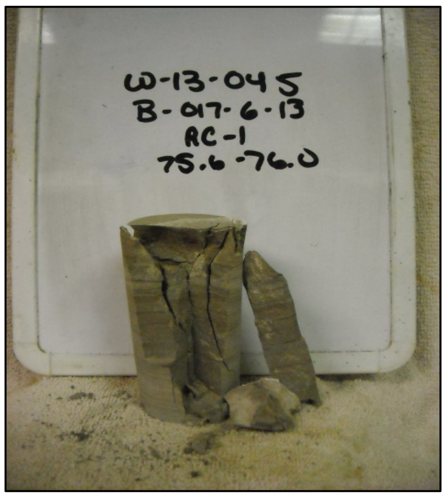
Unconfined Compression Test



Before Testing



After Failure



REMARKS:

RESOURCE INTERNATIONAL, INC.
Engineering Consultants

Unconfined Compressive Strength
of Intact Rock Core Specimens (ASTM D 7012-04)

6350 Presidential Gatew.
Columbus, OH 43231
Phone (614) 823-4949

9885 Rockside Road
Cleveland, OH 44125
Phone (216) 573-0955

4480 Lake Forest Drive
Cincinnati, Ohio 45242
Phone (513) 769-6998

Project: FRA-70-12.68
Project No.: W-13-045
Date of Testing: 7/26/2013
Test Performed by: KR/TK

Rock Description: LIMESTONE: Light gray, unweathered, strong.

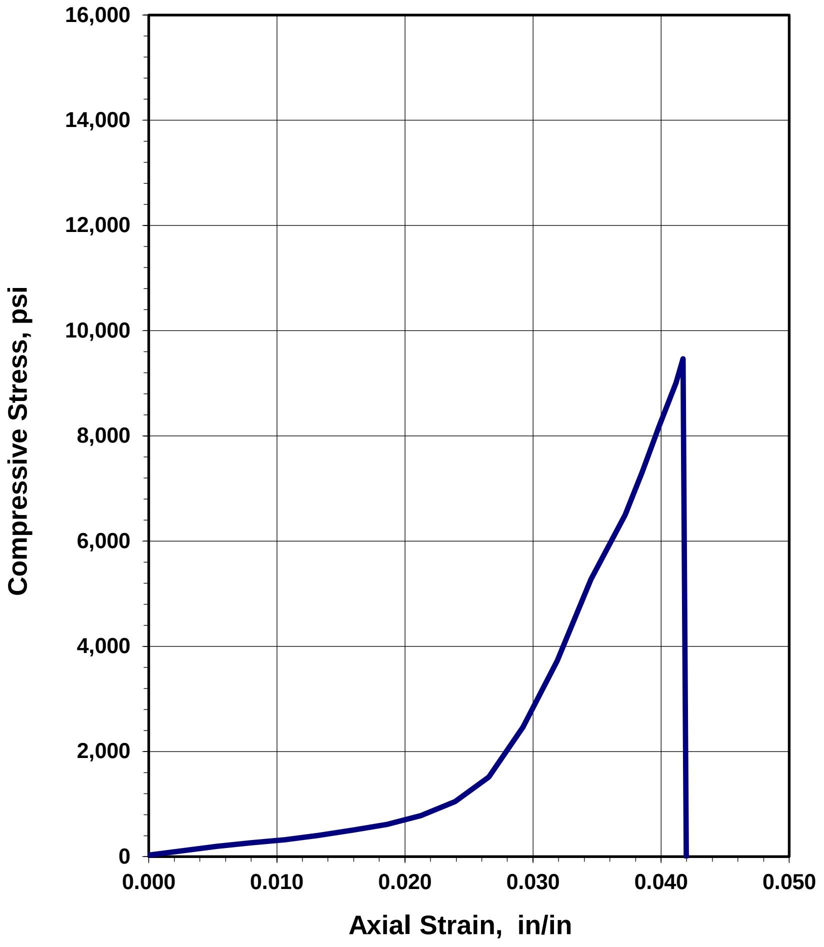
Boring No.: B-020-1-13
Station / Offset: 5080+09.80, 30.9' Rt.
Sample No. / Depth: RC-1 / 80.7 ft.
Moisture condition: As received

Average Length: 3.765 in
Average Diameter: 1.863 in
Length to diameter ratio: 2.021
Cross Sectional Area: 2.725 in^2

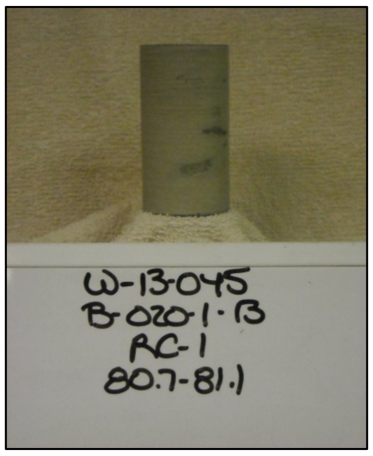
Failure Load: 25,800 lbs
Axial Strain at Failure: 0.0417 in/in
Stress: 9,465 psi

Rate of Loading: 55.0 lbs/sec
Testing Time: 469 sec
(Rate 2-15 minutes to failure)

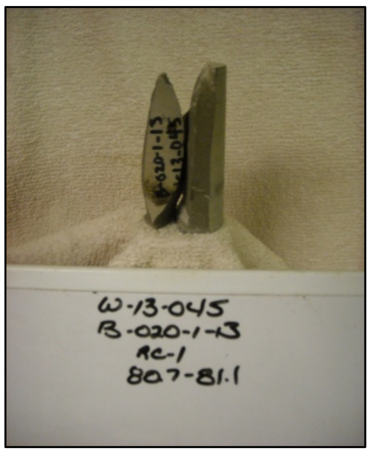
Unconfined Compression Test



Before Testing



After Failure

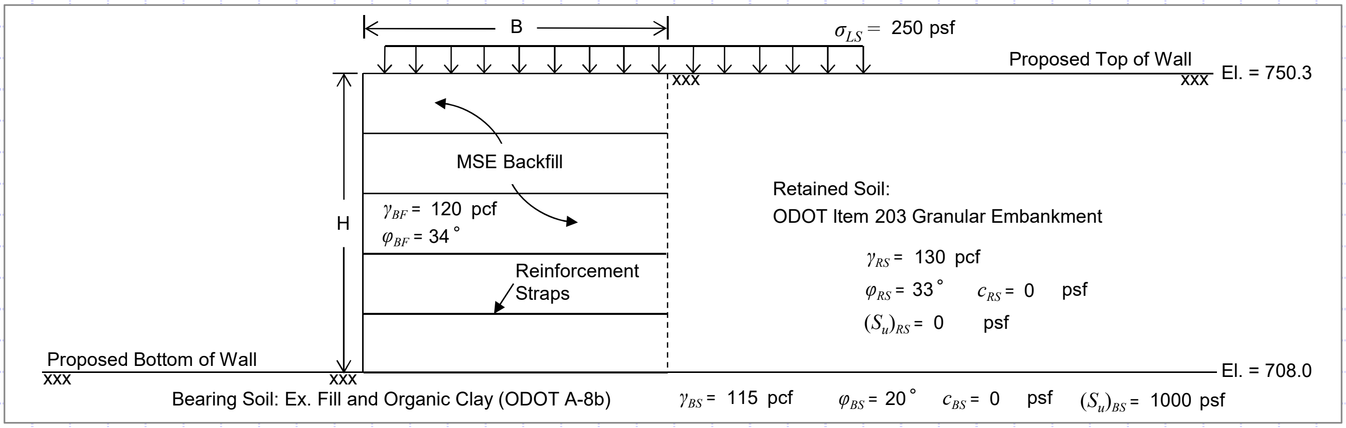


REMARKS: ____________________

APPENDIX V

MSE WALL CALCULATIONS

FRA-70-12.68 - Retaining Wall 4W8 - B-017-6-13, B-019-0-08, B-019-1-13 and B-20-1-13 - Maximum 42.3 ft. Wall Height



MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) = 42.3 ft
MSE Wall Width (Reinforcement Length), (B) = 29.6 ft
MSE Wall Length, (L) = 648 ft
Live Surcharge Load, (σ_{LS}) = 250 psf
Retained Soil Unit Weight, (γ_{RS}) = 130 pcf
Retained Soil Friction Angle, (φ_{RS}) = 33 °
Retained Soil Drained Cohesion [1], (c_{BS}) = 0 psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] = 0 psf
Retained Soil Active Earth Pressure Coeff., (K_a) = 0.264
MSE Backfill Unit Weight, (γ_{BF}) = 120 pcf
MSE Backfill Friction Angle, (φ_{BF}) = 34 °

Bearing Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) = 115 pcf
Bearing Soil Friction Angle, (φ_{BS}) = 20 °
Bearing Soil Drained Cohesion, (c_{BS}) = 0 psf
Bearing Soil Undrained Shear Strength, [$(S_u)_{BS}$] = 1000 psf
Embedment Depth, (D_f) = 4.0 ft
Depth to Grounwater (Below Bot. of Wall), (D_W) = 0.0 ft

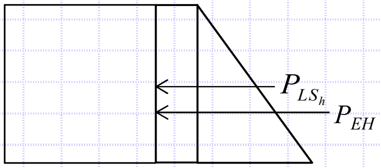
LRFD Load Factors

	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

Check Sliding (Loading Case - Strength Ia) - AASHTO LRFD BDM Section 11.10.5.3

Sliding Force:



$$P_H = P_{EH} + P_{LS_h}$$

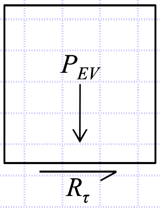
$$P_{EH} = \tfrac{1}{2}\gamma_{RS} H^2 K_a \gamma_{EH} = \text{½(130 pcf)(42.3 ft)²(0.264)(1.5)} = 46.06 \text{ kip/ft}$$

$$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS} = \text{(250 psf)(42.3 ft)(0.264)(1.75)} = 4.89 \text{ kip/ft}$$

$$P_H = \text{46.06 kip/ft + 4.89 kip/ft} = 50.95 \text{ kip/ft}$$

Check Sliding Resistance - Drained Condition

Nominal Sliding Resistance:



$$R_\tau = P_{EV} \cdot \tan\delta$$

$$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV} = \text{(120 pcf)(42.3 ft)(29.6 ft)(1.00)} = 150.25 \text{ kip/ft}$$

$$\tan\delta = \left(\tan\varphi_{BS} \le \tan\varphi_{BF}\right)$$

$$\tan\delta = \tan(20) \le \tan(34) \longrightarrow 0.36 \le 0.67 \longrightarrow \tan\delta = 0.36$$

$$R_\tau = \text{(150.25 kip/ft)(0.36)} = 54.09 \text{ kip/ft}$$

Verify Sliding Force Less Than Factored Sliding Resistance - Drained Condition

$P_H \le R_\tau \cdot \phi_\tau$ ⟶ 50.95 kip/ft ≤ (54.09 kip/ft)(1.0) = 54.09 kip/ft ⟶ 50.95 kip/ft ≤ 54.09 kip/ft **OK**

Use φ_τ = **1.0** *(Per AASHTO LRFD BDM Table 11.5.7-1)*

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	42.3	ft
MSE Wall Width (Reinforcement Length), (B) =	29.6	ft
MSE Wall Length, (L) =	648	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	130	pcf
Retained Soil Friction Angle, (φ_{RS}) =	33	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, $[(S_u)_{RS}]$ =	0	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.264	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	115	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	20	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	1000	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

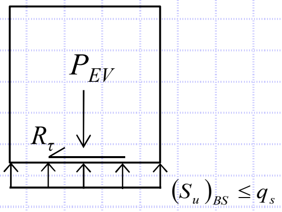
(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

Check Sliding (Loading Case - Strength Ia) - AASHTO LRFD BDM Section 11.10.5.3 *(Continued)*

Check Sliding Resistance - Undrained Condition

Nominal Sliding Resisting:

$$R_\tau = ((S_u)_{BS} \le q_s) \cdot B$$



$$(S_u)_{BS} = 1.00 \text{ ksf}$$

$$q_s = \sigma_v / 2 = (5.08 \text{ ksf}) / 2 = 2.54 \text{ ksf}$$

$$\sigma_v = P_{EV} / B = (150.25 \text{ kip/ft}) / (29.6 \text{ ft}) = 5.08 \text{ ksf}$$

$$R_\tau = (1.00 \text{ ksf} \le 2.54 \text{ ksf})(29.6 \text{ ft}) = 29.60 \text{ kip/ft}$$

Verify Sliding Force Less Than Factored Sliding Resistance - Undrained Condition

$P_H \le R_\tau \cdot \phi_\tau$ ⟶ 50.95 kip/ft ≤ (29.60 kip/ft)(1.0) = 29.60 kip/ft ⟶ 50.95 kip/ft ≤ 29.60 kip/ft **ERROR!!**

Use φ_τ = **1.0** *(Per AASHTO LRFD BDM Table 11.5.7-1)*

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) = 42.3 ft
MSE Wall Width (Reinforcement Length), (B) = 29.6 ft
MSE Wall Length, (L) = 648 ft
Live Surcharge Load, (σ_{LS}) = 250 psf
Retained Soil Unit Weight, (γ_{RS}) = 130 pcf
Retained Soil Friction Angle, (φ_{RS}) = 33 °
Retained Soil Drained Cohesion, (c_{BS}) = 0 psf
Retained Soil Undrained Shear Strength, $[(S_u)_{RS}]$ = 0 psf
Retained Soil Active Earth Pressure Coeff., (K_a) = 0.264
MSE Backfill Unit Weight, (γ_{BF}) = 120 pcf
MSE Backfill Friction Angle, (φ_{BF}) = 34 °

Bearing Soil Properties:

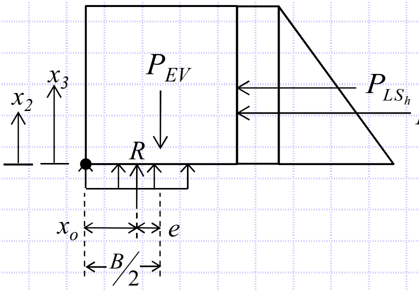
Bearing Soil Unit Weight, (γ_{BS}) = 115 pcf
Bearing Soil Friction Angle, (φ_{BS}) = 20 °
Bearing Soil Drained Cohesion, (c_{BS}) = 0 psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ = 1000 psf
Embedment Depth, (D_f) = 4.0 ft
Depth to Grounwater (Below Bot. of Wall), (D_W) = 0.0 ft

LRFD Load Factors

	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

Check Eccentricity (Loading Case - Strength Ia) - AASHTO LRFD BDM Section 11.10.5.5



$$e = B/2 - x_o$$

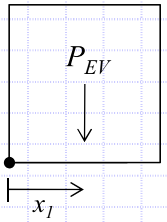
$$x_o = \frac{M_{EV} - M_H}{P_{EV}}$$ = (2223.7 kip·ft/ft - 752.87 kip·ft/ft) / (150.25 kip/ft) = 9.79 ft

M_{EV} = 2223.70 kip·ft/ft
M_H = 752.87 kip·ft/ft
P_{EV} = 150.25 kip/ft
Defined below

$e =$ (29.6 ft)/2 - 9.79 ft = 5.01 ft

Resisting Moment, M_{EV}:

$$M_{EV} = P_{EV}(x_1)$$



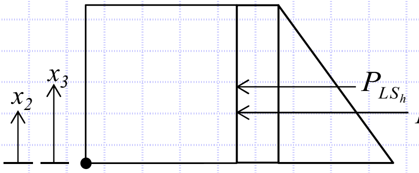
$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV}$ = (120 pcf)(42.3 ft)(29.6 ft)(1.00) = 150.25 kip/ft

$x_1 = B/2$ = (29.6 ft) / 2 = 14.80 ft

M_{EV} = (150.25 kip/ft)(14.80 ft) = 2223.70 kip·ft/ft

Overturning Moment, M_H:

$$M_H = P_{EH}(x_2) + P_{LS_h}(x_3)$$



$P_{EH} = ½\gamma_{RS}H^2K_a\gamma_{EH}$ = ½(130 pcf)(42.3 ft)²(0.264)(1.5) = 46.06 kip/ft

$P_{LS_h} = \sigma_{LS}HK_a\gamma_{LS}$ = (250 psf)(42.3 ft)(0.264)(1.75) = 4.89 kip/ft

$x_2 = H/3$ = (42.3 ft) / 3 = 14.10 ft

$x_3 = H/2$ = (42.3 ft) / 2 = 21.15 ft

M_H = (46.06 kip/ft)(14.1 ft) + (4.89 kip/ft)(21.15 ft) = 752.87 kip·ft/ft

Check Eccentricity

$e < e_{\max} \longrightarrow$ 5.01 ft < 9.87 ft **OK**

Limiting Eccentricity: $e_{\max} = B/3 \longrightarrow e_{\max}$ = (29.6 ft) / 3 = 9.87 ft

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) = 42.3 ft
MSE Wall Width (Reinforcement Length), (B) = 29.6 ft
MSE Wall Length, (L) = 648 ft
Live Surcharge Load, (σ_{LS}) = 250 psf
Retained Soil Unit Weight, (γ_{RS}) = 130 pcf
Retained Soil Friction Angle, (φ_{RS}) = 33 °
Retained Soil Drained Cohesion, (c_{BS}) = 0 psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] = 0 psf
Retained Soil Active Earth Pressure Coeff., (K_a) = 0.264
MSE Backfill Unit Weight, (γ_{BF}) = 120 pcf
MSE Backfill Friction Angle, (φ_{BF}) = 34 °

Bearing Soil Properties:

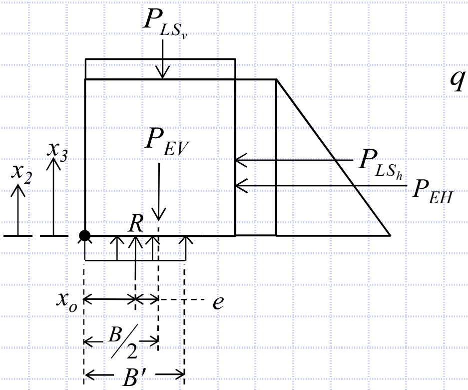
Bearing Soil Unit Weight, (γ_{BS}) = 115 pcf
Bearing Soil Friction Angle, (φ_{BS}) = 20 °
Bearing Soil Drained Cohesion, (c_{BS}) = 0 psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] = 1000 psf
Embedment Depth, (D_f) = 4.0 ft
Depth to Grounwater (Below Bot. of Wall), (D_W) = 0.0 ft

LRFD Load Factors

	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

Check Bearing Capacity (Loading Case - Strength Ib) - AASHTO LRFD BDM Section 11.10.5.4



$$q_{eq} = P_V / B'$$

$$B' = B - 2e \quad = \quad 29.6\text{ ft} - 2(3.49\text{ ft}) \quad = \quad 22.62 \quad \text{ft}$$

$$e = B/2 - x_o \quad = \quad (29.6\text{ ft}) / 2 - 11.31\text{ ft} \quad = \quad 3.49 \quad \text{ft}$$

$$x_o = \frac{M_V - M_H}{P_V} \quad = \quad (3193.65\text{ kip·ft/ft} - 752.73\text{ kip·ft/ft}) / 215.79\text{ kip/ft} \quad = \quad 11.31 \quad \text{ft}$$

$$q_{eq} = \quad (215.79\text{ kip/ft}) / (22.62\text{ ft}) \quad = \quad 9.54 \quad \text{ksf}$$

$$M_V = P_{EV}(x_1) + P_{LS_v}(x_1) = (\gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV})(x_1) + (\sigma_{LS} \cdot B \cdot \gamma_{LS})(x_1)$$

$$M_V = \quad [(120\text{ pcf})(42.3\text{ ft})(29.6\text{ ft})(1.35)](14.8\text{ ft}) + [(250\text{ psf})(29.6\text{ ft})(1.75)](14.8\text{ ft}) \quad = \quad 3193.65 \quad \text{kip·ft/ft}$$

$$M_H = P_{EH}(x_2) + P_{LS_h}(x_3) = (\tfrac{1}{2}\gamma_{RS} H^2 K_a \gamma_{EH})(x_2) + (\sigma_{LS} H K_a \gamma_{LS})(x_3)$$

$$M_H = \quad [\tfrac{1}{2}(130\text{ pcf})(42.3\text{ ft})^2(0.264)(1.5)](14.1\text{ ft}) + [(250\text{ psf})(42.3\text{ ft})(0.264)(1.75)](21.15\text{ ft}) \quad = \quad 752.73 \quad \text{kip·ft/ft}$$

$$P_V = P_{EV} + P_{LS} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV} + \sigma_{LS} \cdot B \cdot \gamma_{LS}$$

$$P_V = \quad (120\text{ pcf})(42.3\text{ ft})(29.6\text{ ft})(1.35) + (250\text{ psf})(29.6\text{ ft})(1.75) \quad = \quad 215.79 \quad \text{kip/ft}$$

Check Bearing Resistance - Drained Condition

Nominal Bearing Resistance: $q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$

$N_{cm} = N_c s_c i_c$ = 15.05

N_c = 14.83
s_c = 1+(22.62 ft/648 ft)(6.4/14.83)
= 1.015
i_c = 1.000 (Assumed)

$N_{qm} = N_q s_q d_q i_q$ = 6.84

N_q = 6.40
s_q = 1.013
d_q = 1+2tan(20°)[1-sin(20°)]²tan⁻¹(4.0 ft/22.62 ft)
1.055
i_q = 1.000 (Assumed)
C_{wq} = 0.0 ft > 4.0 ft = 0.500

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 5.31

N_γ = 5.39
s_γ = 0.986
i_γ = 1.000 (Assumed)
$C_{w\gamma}$ = 0.0 ft < 1.5(22.62 ft) + 4.0 ft = 0.500

$$q_n = \quad (0\text{ psf})(15.052) + (115\text{ pcf})(4.0\text{ ft})(6.840)(0.500) + \tfrac{1}{2}(115\text{ pcf})(22.6\text{ ft})(5.315)(0.500) \quad = \quad 5.03 \quad \text{ksf}$$

Verify Equivalent Pressure Less Than Factored Bearing Resistance

Use φ_b = **0.65** *(Per AASHTO LRFD BDM Table 11.5.7-1)*

$q_{eq} \le q_n \cdot \phi_b$ ⟶ 9.54 ksf ≤ (5.03 ksf)(0.65) = 3.27 ksf ⟶ 9.54 ksf ≤ 3.27 ksf **ERROR!!**

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) = 42.3 ft
MSE Wall Width (Reinforcement Length), (B) = 29.6 ft
MSE Wall Length, (L) = 648 ft
Live Surcharge Load, (σ_{LS}) = 250 psf
Retained Soil Unit Weight, (γ_{RS}) = 130 pcf
Retained Soil Friction Angle, (φ_{RS}) = 33 °
Retained Soil Drained Cohesion, (c_{BS}) = 0 psf
Retained Soil Undrained Shear Strength, $[(S_u)_{RS}]$ = 0 psf
Retained Soil Active Earth Pressure Coeff., (K_a) = 0.264
MSE Backfill Unit Weight, (γ_{BF}) = 120 pcf
MSE Backfill Friction Angle, (φ_{BF}) = 34 °

Bearing Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) = 115 pcf
Bearing Soil Friction Angle, (φ_{BS}) = 20 °
Bearing Soil Drained Cohesion, (c_{BS}) = 0 psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ = 1000 psf
Embedment Depth, (D_f) = 4.0 ft
Depth to Grounwater (Below Bot. of Wall), (D_W) = 0.0 ft

LRFD Load Factors

	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

Check Bearing Capacity (Loading Case - Strength Ib) - AASHTO LRFD BDM Section 11.10.5.4 *(Continued)*

Check Bearing Resistance - Undrained Condition

Nominal Bearing Resistance: $q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \frac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$

$N_{cm} = N_c s_c i_c$ = 5.180

N_c = 5.140
s_c = 1+(22.62 ft/[(5)(648 ft)]) = 1.007
i_c = 1.000 (Assumed)

$N_{qm} = N_q s_q d_q i_q$ = 1.000

N_q = 1.000
s_q = 1.000
d_q = 1+2tan(0°)[1-sin(0°)]²tan⁻¹(4.0 ft/22.62 ft)
1.000
i_q = 1.000 (Assumed)
C_{wq} = 0.0 ft > 4.0 ft = 0.500

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 0.000

N_γ = 0.000
s_γ = 1.000
i_γ = 1.000 (Assumed)
$C_{w\gamma}$ = 0.0 ft < 1.5(22.62 ft) + 4.0 ft = 0.500

$q_n =$ (1000 psf)(5.180) + (115 pcf)(4.0 ft)(1.000)(0.500) + ½(115 pcf)(22.6 ft)(0.000)(0.500) = 5.41 ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

$q_{eq} \leq q_n \cdot \phi_b \longrightarrow$ 9.54 ksf ≤ (5.41 ksf)(0.65) = 3.52 ksf $\longrightarrow$ 9.54 ksf ≤ 3.52 ksf **ERROR!!**

Use φ_b = **0.65** *(Per AASHTO LRFD BDM Table 11.5.7-1)*

JOB	FRA-70-12.68	NO.	W-13-045
SHEET NO.	6	OF	6
CALCULATED BY	BRT	DATE	7/15/2018
CHECKED BY	JPS	DATE	7/15/2018

Retaining Wall 4W8 - Sta. 5017+12 to 5080+60

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	42.3	ft
MSE Wall Width (Reinforcement Length), (B) =	29.6	ft
MSE Wall Length, (L) =	648	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	130	pcf
Retained Soil Friction Angle, (φ_{RS}) =	33	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	0	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.264	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

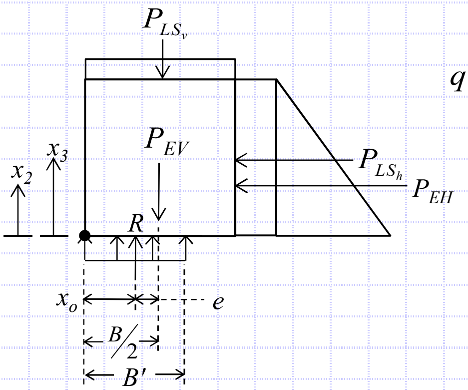
Bearing Soil Unit Weight, (γ_{BS}) =	115	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	20	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	1000	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

Settlement Analysis (Loading Case - Service I) - AASHTO LRFD BDM Section 11.10.4.1



$$q_{eq} = P_V / B'$$

$B' = B - 2e$ = 29.6 ft - 2(3.12 ft) = 23.36 ft

$e = B/2 - x_o$ = (29.6 ft) / 2 - 11.68 ft = 3.12 ft

$x_o = \dfrac{M_V - M_H}{P_V}$ = (2333.21 kip·ft/ft - 491.98 kip·ft/ft) / 157.65 kip/ft = 11.68 ft

q_{eq} = (157.65 kip/ft) / (23.36 ft) = 6.75 ksf

$$M_V = P_{EV}(x_1) + P_{LS_v}(x_1) = (\gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV})(x_1) + (\sigma_{LS} \cdot B \cdot \gamma_{LS})(x_1)$$

M_V = [(120 pcf)(42.3 ft)(29.6 ft)(1.00)](14.8 ft) + [(250 psf)(29.6 ft)(1.00)](14.8 ft) = 2333.21 kip·ft/ft

$$M_H = P_{EH}(x_2) + P_{LS_h}(x_3) = (\tfrac{1}{2}\gamma_{RS} H^2 K_a \gamma_{EH})(x_2) + (\sigma_{LS} H K_a \gamma_{LS})(x_3)$$

M_H = [½(130 pcf)(42.3 ft)²(0.264)(1.00)](14.1 ft) + [(250 psf)(42.3 ft)(0.264)(1.00)](21.15 ft) = 491.98 kip·ft/ft

$$P_V = P_{EV} + P_{LS} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV} + \sigma_{LS} \cdot B \cdot \gamma_{LS}$$

P_V = (120 pcf)(42.3 ft)(29.6 ft)(1.00) + (250 psf)(29.6 ft)(1.00) = 157.65 kip/ft

Settlement, Time Rate of Consolidation and Differential Settlement:

Boring	Total Settlement at Center of Reinforced Soil Mass	Total Settlement at Wall Facing	Time for 90% Consolidation	Distance Between Borings Along Wall Facing	Differential Settlement Along Wall Facing
B-017-6-13	11.582 in	7.561 in	80 days		
B-019-0-08	16.951 in	10.294 in	90 days	185 ft	1/810
B-019-0-13	10.978 in	7.450 in	20 days	105 ft	1/440
B-020-1-13	14.770 in	9.089 in	22 days	295 ft	1/2160

W-13-045 - FRA-70-12.68 - Retaining Wall 4W8
MSE Wall Settlement - Sta. 5074+12 to 5080+60

Calculated By: BRT Date: 7/15/2018
Checked By: JPS Date: 7/15/2018

Borings B-017-6-13

H=	42.3	ft	Total wall height
B'=	23.4	ft	Effective footing width due to eccentricity
D_w =	0.0	ft	Depth below bottom of footing
q_e =	6,750	psf	Equivalent bearing pressure at bottom of wall

																				Total Settlement at Center of Reinforced Soil Mass					Total Settlement at Facing of Wall				
Layer	Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
1	A-6a	C	0.0	4.0	4.0	2.0	120	480	240	115	2,115	33	0.207	0.031	0.530				0.09	0.998	6,736	6,851	0.379	4.546	0.500	3,374	3,489	0.220	2.643
2	A-6a	C	4.0	6.5	2.5	5.3	115	768	624	296	2,296	33	0.207	0.031	0.530				0.22	0.969	6,543	6,839	0.205	2.465	0.498	3,360	3,656	0.113	1.361
3	A-3a	G	6.5	9.0	2.5	7.8	125	1,080	924	440	2,440					18	27	81	0.33	0.921	6,214	6,655	0.037	0.439	0.493	3,329	3,769	0.029	0.347
4	A-4a	C	9.0	11.5	2.5	10.3	120	1,380	1,230	590	2,590	24	0.126	0.013	0.460				0.44	0.858	5,788	6,379	0.098	1.180	0.486	3,278	3,868	0.051	0.617
5	A-1-b	G	11.5	16.5	5.0	14.0	130	2,030	1,705	831	4,831					46	60	213	0.60	0.756	5,106	5,937	0.020	0.240	0.469	3,163	3,995	0.016	0.192
	A-1-b	G	16.5	21.5	5.0	19.0	130	2,680	2,355	1,169	5,169					46	54	188	0.81	0.636	4,290	5,460	0.018	0.213	0.439	2,961	4,130	0.015	0.175
6	A-1-b	G	21.5	26.0	4.5	23.8	125	3,243	2,961	1,479	5,479					24	26	89	1.01	0.544	3,670	5,150	0.027	0.327	0.407	2,746	4,225	0.023	0.275
	A-1-b	G	26.0	30.5	4.5	28.3	125	3,805	3,524	1,761	5,761					24	25	86	1.21	0.475	3,206	4,967	0.024	0.283	0.377	2,542	4,303	0.020	0.244
7	A-7-5	C	30.5	40.5	10.0	35.5	125	5,055	4,430	2,215	6,215	57	0.423	0.042	0.718				1.52	0.392	2,646	4,860	0.084	1.009	0.332	2,239	4,454	0.075	0.897
	A-7-5	C	40.5	50.5	10.0	45.5	125	6,305	5,680	2,841	6,841	57	0.423	0.042	0.718				1.94	0.314	2,118	4,959	0.060	0.715	0.281	1,894	4,735	0.055	0.656
8	A-1-b	G	50.5	54.5	4.0	52.5	135	6,845	6,575	3,299	7,299					100	83	350	2.24	0.275	1,855	5,154	0.002	0.027	0.252	1,700	4,999	0.002	0.025
9	A-7-6	C	54.5	57.5	3.0	56.0	130	7,235	7,040	3,546	7,546	51	0.369	0.037	0.671				2.39	0.259	1,745	5,291	0.012	0.138	0.239	1,615	5,160	0.011	0.130
																				Total Settlement:			11.582 in		Total Settlement:			7.561 in	

1. $\sigma_p' = \sigma_{vo}'+\sigma_{m;}$ Estimate σ_m of 2,000 psf in existing fill material and 4,000 psf (moderately overconsolidated) for natural soil deposits; Ref. Table 11.2, Coduto 2003
2. $C_c = 0.009(LL-10)$; Ref. Table 6-9, FHWA GEC 5
3. $C_r = 0.15(Cc)$ for the existing fill and 0.10(Cc) for the natural soil deposits; Ref. Section 8.11, Holtz and Kovacs 1981
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_nN_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \leq \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (Cohesiv soil layers)
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)

W-13-045 - FRA-70-12.68 - Retaining Wall 4W8
MSE Wall Settlement - Sta. 5074+12 to 5080+60

Calculated By: BRT   Date: 07/15/2018
Checked By: JPS   Date: 07/15/2018

Borings B-017-6-13

H=	42.3	ft	Total wall height
B'=	23.4	ft	Effective footing width due to eccentricity
D_w =	0.0	ft	Depth below bottom of footing
q_e =	6,750	psf	Equivalent bearing pressure at bottom of wall

	A-6a	A-4a	A-7-5	A-7-6		
c_v =	600	800	100	150	ft^2/yr	Coefficient of consolidation
t =	80	80	80	80	days	Time following completion of construction
H_{dr} =	3	1	10	3	ft	Length of longest drainage path considered
T_v =	14.612	175.342	0.219	3.653		Time factor
U =	100	100	53	100	%	Degree of consolidation

$(S_c)_t$ = 6.832 in   Settlement complete at 90% of primary consolidation

																							Total Settlement at Facing of Wall			Settlement Complete at 90% of Primary Consolidation	
Layer	Soil Type	Soil Type	Layer Depth (ft)		Layer Thickness (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	Layer Settlement (in)	$(S_c)_t$ (11) (in)	Layer Settlement (in)
1	A-6a	C	0.0	4.0	4.0	2.0	120	480	240	115	2,115	33	0.207	0.031	0.530				0.09	0.500	3,374	3,489	0.220	2.643	2.643	2.643	2.643
2	A-6a	C	4.0	6.5	2.5	5.3	115	768	624	296	2,296	33	0.207	0.031	0.530				0.22	0.498	3,360	3,656	0.113	1.361	1.361	1.361	1.361
3	A-3a	G	6.5	9.0	2.5	7.8	125	1,080	924	440	2,440					18	27	81	0.33	0.493	3,329	3,769	0.029	0.347	0.347	0.347	0.347
4	A-4a	C	9.0	11.5	2.5	10.3	120	1,380	1,230	590	2,590	24	0.126	0.013	0.460				0.44	0.486	3,278	3,868	0.051	0.617	0.617	0.617	0.617
5	A-1-b	G	11.5	16.5	5.0	14.0	130	2,030	1,705	831	4,831					46	60	213	0.60	0.469	3,163	3,995	0.016	0.192	0.367	0.192	0.367
	A-1-b	G	16.5	21.5	5.0	19.0	130	2,680	2,355	1,169	5,169					46	54	188	0.81	0.439	2,961	4,130	0.015	0.175		0.175	
6	A-1-b	G	21.5	26.0	4.5	23.8	125	3,243	2,961	1,479	5,479					24	26	89	1.01	0.407	2,746	4,225	0.023	0.275	0.519	0.275	0.519
	A-1-b	G	26.0	30.5	4.5	28.3	125	3,805	3,524	1,761	5,761					24	25	86	1.21	0.377	2,542	4,303	0.020	0.244		0.244	
7	A-7-5	C	30.5	40.5	10.0	35.5	125	5,055	4,430	2,215	6,215	57	0.423	0.042	0.718				1.52	0.332	2,239	4,454	0.075	0.897	1.552	0.475	0.823
	A-7-5	C	40.5	50.5	10.0	45.5	125	6,305	5,680	2,841	6,841	57	0.423	0.042	0.718				1.94	0.281	1,894	4,735	0.055	0.656		0.348	
8	A-1-b	G	50.5	54.5	4.0	52.5	135	6,845	6,575	3,299	7,299					100	83	350	2.24	0.252	1,700	4,999	0.002	0.025	0.025	0.025	0.025
9	A-7-6	C	54.5	57.5	3.0	56.0	130	7,235	7,040	3,546	7,546	51	0.369	0.037	0.671				2.39	0.239	1,615	5,160	0.011	0.130	0.130	0.130	0.130

Settlement Remaining After Hold Period: 0.730 in

1. $\sigma_p' = \sigma_{vo}'+\sigma_{m;}$ Estimate σ_m of 2,000 psf in existing fill material and 4,000 psf (moderately overconsolidated) for natural soil deposits; Ref. Table 11.2, Coduto 2003
2. $C_c = 0.009(LL-10)$; Ref. Table 6-9, FHWA GEC 5
3. $C_r = 0.15(Cc)$ for the existing fill and $0.10(Cc)$ for the natural soil deposits; Ref. Section 8.11, Holtz and Kovacs 1981
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_N N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \leq \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (Cohesiv soil layers)
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)
11. $(S_c)_t = S_c(U/100)$; U = 100 for all granular soils at time t = 0

W-13-045 - FRA-70-12.68 - Retaining Wall 4W8
MSE Wall Settlement - Sta. 5074+12 to 5080+60

Calculated By: BRT Date: 7/15/2018
Checked By: JPS Date: 7/15/2018

Borings B-019-0-08

H=	42.3	ft	Total wall height
B'=	23.4	ft	Effective footing width due to eccentricity
D_w =	0.0	ft	Depth below bottom of footing
q_e =	6,750	psf	Equivalent bearing pressure at bottom of wall

																			Total Settlement at Center of Reinforced Soil Mass					Total Settlement at Facing of Wall				
Layer	Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
1	A-1-b	G	0.0	2.5	2.5	1.3	120	300	150	72	2,072					4	8	54	0.05	0.999	6,747	6,819	0.092	1.098	0.500	3,375	3,447	0.078	0.934
2	A-8b	C	2.5	5.0	2.5	3.8	100	550	425	191	2,191	41	0.460	0.069	0.750				0.16	0.988	6,666	6,857	0.430	5.161	0.499	3,369	3,560	0.243	2.916
	A-8b	C	5.0	7.5	2.5	6.3	100	800	675	285	2,285	41	0.460	0.069	0.750				0.27	0.952	6,428	6,713	0.397	4.760	0.496	3,350	3,635	0.222	2.659
	A-8b	C	7.5	10.0	2.5	8.8	100	1,050	925	379	2,379	41	0.460	0.069	0.750				0.37	0.897	6,052	6,431	0.362	4.349	0.491	3,311	3,690	0.204	2.447
3	A-1-b	G	10.0	12.5	2.5	11.3	120	1,350	1,200	498	2,498					11	16	67	0.48	0.831	5,606	6,104	0.041	0.490	0.482	3,251	3,749	0.033	0.395
4	A-1-a	G	12.5	19.5	7.0	16.0	130	2,260	1,805	807	4,807					38	50	168	0.68	0.705	4,761	5,567	0.035	0.420	0.457	3,087	3,894	0.029	0.343
5	A-1-b	G	19.5	23.0	3.5	21.3	135	2,733	2,496	1,170	5,170					76	90	393	0.91	0.589	3,978	5,149	0.006	0.069	0.424	2,860	4,030	0.005	0.057
	A-1-b	G	23.0	26.5	3.5	24.8	135	3,205	2,969	1,424	5,424					76	85	359	1.06	0.527	3,558	4,982	0.005	0.064	0.400	2,700	4,124	0.005	0.054
6	A-1-a	G	26.5	34.0	7.5	30.3	135	4,218	3,711	1,824	5,824					100	103	494	1.29	0.449	3,032	4,855	0.006	0.077	0.364	2,454	4,278	0.006	0.067
	A-1-a	G	34.0	41.5	7.5	37.8	135	5,230	4,724	2,368	6,368					100	95	427	1.61	0.371	2,507	4,875	0.006	0.066	0.319	2,154	4,523	0.005	0.059
7	A-7-6	C	41.5	45.0	3.5	43.3	125	5,668	5,449	2,750	6,750	45	0.315	0.032	0.624				1.85	0.329	2,219	4,968	0.017	0.209	0.291	1,965	4,715	0.016	0.191
	A-7-6	C	45.0	48.5	3.5	46.8	125	6,105	5,886	2,969	6,969	45	0.315	0.032	0.624				2.00	0.306	2,066	5,035	0.016	0.187	0.275	1,857	4,826	0.014	0.172
																				Total Settlement:			16.951 in		Total Settlement:			10.294 in	

1. $\sigma_p' = \sigma_{vo}' + \sigma_{m;}$ Estimate σ_m of 2,000 psf in existing fill material and 4,000 psf (moderately overconsolidated) for natural soil deposits; Ref. Table 11.2, Coduto 2003
2. $C_c = 0.009(LL-10)$; Ref. Table 6-9, FHWA GEC 5; Cc = 0.0115wn for organic soils (A-8); Ref. Table 8.2, Holtz and Kovacs 1981
3. $C_r = 0.15(Cc)$ for the existing fill and 0.10(Cc) for the natural soil deposits; Ref. Section 8.11, Holtz and Kovacs 1981
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_n N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \leq \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (Cohesiv soil layers)
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)

W-13-045 - FRA-70-12.68 - Retaining Wall 4W8
MSE Wall Settlement - Sta. 5074+12 to 5080+60

Calculated By: BRT Date: 07/15/2018
Checked By: JPS Date: 07/15/2018

Borings B-019-0-08

H=	42.3	ft	Total wall height
B'=	23.4	ft	Effective footing width due to eccentricity
D_w =	0.0	ft	Depth below bottom of footing
q_e =	6,750	psf	Equivalent bearing pressure at bottom of wall

	A-8b	A-7-6		
c_v =	50	150	ft^2/yr	Coefficient of consolidation
t =	90	90	days	Time following completion of construction
H_{dr} =	4	7	ft	Length of longest drainage path considered
T_v =	0.771	0.755		Time factor
U =	88	87	%	Degree of consolidation

$(S_c)_t$ = 9.284 in Settlement complete at 90% of primary consolidation

																							Total Settlement at Facing of Wall			Settlement Complete at 90% of Primary Consolidation	
Layer	Soil Type	Soil Type	Layer Depth (ft)		Layer Thickness (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	Layer Settlement (in)	$(S_c)_t$ (11) (in)	Layer Settlement (in)
1	A-1-b	G	0.0	2.5	2.5	1.3	120	300	150	72	2,072					4	8	54	0.05	0.500	3,375	3,447	0.078	0.934	0.934	0.934	0.934
2	A-8b	C	2.5	5.0	2.5	3.8	100	550	425	191	2,191	41	0.460	0.069	0.750				0.16	0.499	3,369	3,560	0.243	2.916	8.022	2.566	7.059
	A-8b	C	5.0	7.5	2.5	6.3	100	800	675	285	2,285	41	0.460	0.069	0.750				0.27	0.496	3,350	3,635	0.222	2.659		2.340	
	A-8b	C	7.5	10.0	2.5	8.8	100	1,050	925	379	2,379	41	0.460	0.069	0.750				0.37	0.491	3,311	3,690	0.204	2.447		2.153	
3	A-1-b	G	10.0	12.5	2.5	11.3	120	1,350	1,200	498	2,498					11	16	67	0.48	0.482	3,251	3,749	0.033	0.395	0.395	0.395	0.395
4	A-1-a	G	12.5	19.5	7.0	16.0	130	2,260	1,805	807	4,807					38	50	168	0.68	0.457	3,087	3,894	0.029	0.343	0.343	0.343	0.343
5	A-1-b	G	19.5	23.0	3.5	21.3	135	2,733	2,496	1,170	5,170					76	90	393	0.91	0.424	2,860	4,030	0.005	0.057	0.111	0.057	0.111
	A-1-b	G	23.0	26.5	3.5	24.8	135	3,205	2,969	1,424	5,424					76	85	359	1.06	0.400	2,700	4,124	0.005	0.054		0.054	
6	A-1-a	G	26.5	34.0	7.5	30.3	135	4,218	3,711	1,824	5,824					100	103	494	1.29	0.364	2,454	4,278	0.006	0.067	0.127	0.067	0.127
	A-1-a	G	34.0	41.5	7.5	37.8	135	5,230	4,724	2,368	6,368					100	95	427	1.61	0.319	2,154	4,523	0.005	0.059		0.059	
7	A-7-6	C	41.5	45.0	3.5	43.3	125	5,668	5,449	2,750	6,750	45	0.315	0.032	0.624				1.85	0.291	1,965	4,715	0.016	0.191	0.363	0.166	0.315
	A-7-6	C	45.0	48.5	3.5	46.8	125	6,105	5,886	2,969	6,969	45	0.315	0.032	0.624				2.00	0.275	1,857	4,826	0.014	0.172		0.150	

Settlement Remaining After Hold Period: 1.010 in

1. $\sigma_p' = \sigma_{vo}'+\sigma_{m;}$ Estimate σ_m of 2,000 psf in existing fill material and 4,000 psf (moderately overconsolidated) for natural soil deposits; Ref. Table 11.2, Coduto 2003
2. C_c = 0.009(LL-10); Ref. Table 6-9, FHWA GEC 5; Cc = 0.0115wn for organic soils (A-8); Ref. Table 8.2, Holtz and Kovacs 1981
3. C_r = 0.15(Cc) for the existing fill and 0.10(Cc) for the natural soil deposits; Ref. Section 8.11, Holtz and Kovacs 1981
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_nN_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \leq \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (Cohesiv soil layers)
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)
11. $(S_c)_t = S_c(U/100)$; U = 100 for all granular soils at time t = 0

W-13-045 - FRA-70-12.68 - Retaining Wall 4W8
MSE Wall Settlement - Sta. 5074+12 to 5080+60

Calculated By: BRT Date: 7/15/2018
Checked By: JPS Date: 7/15/2018

Borings B-019-1-13

H=	42.3	ft	Total wall height
B'=	23.4	ft	Effective footing width due to eccentricity
D_w =	0.0	ft	Depth below bottom of footing
q_e =	6,750	psf	Equivalent bearing pressure at bottom of wall

																				Total Settlement at Center of Reinforced Soil Mass					Total Settlement at Facing of Wall				
Layer	Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
1	A-4a	G	0.0	3.5	3.5	1.8	120	420	210	101	2,101					8	16	34	0.07	0.999	6,741	6,841	0.189	2.274	0.500	3,374	3,475	0.159	1.909
2	A-6b	C	3.5	6.0	2.5	4.8	115	708	564	267	2,267	40	0.270	0.041	0.585				0.20	0.976	6,591	6,858	0.264	3.169	0.498	3,364	3,631	0.146	1.757
	A-6b	C	6.0	8.5	2.5	7.3	115	995	851	399	2,399	40	0.270	0.041	0.585				0.31	0.932	6,290	6,689	0.239	2.874	0.494	3,337	3,736	0.132	1.581
3	A-2-4	G	8.5	13.0	4.5	10.8	125	1,558	1,276	605	4,605					15	21	76	0.46	0.844	5,698	6,303	0.060	0.719	0.484	3,265	3,870	0.047	0.569
	A-2-4	G	13.0	17.5	4.5	15.3	125	2,120	1,839	887	4,887					15	19	72	0.65	0.724	4,887	5,775	0.051	0.607	0.462	3,117	4,004	0.041	0.488
4	A-1-b	G	17.5	22.5	5.0	20.0	125	2,745	2,433	1,185	5,185					20	24	82	0.85	0.614	4,147	5,332	0.040	0.477	0.432	2,917	4,101	0.033	0.394
	A-1-b	G	22.5	27.5	5.0	25.0	125	3,370	3,058	1,498	5,498					20	22	79	1.07	0.523	3,531	5,028	0.033	0.402	0.398	2,688	4,186	0.028	0.341
5	A-1-a	G	27.5	32.5	5.0	30.0	135	4,045	3,708	1,836	5,836					98	101	476	1.28	0.452	3,053	4,888	0.004	0.054	0.365	2,465	4,301	0.004	0.047
	A-1-a	G	32.5	37.5	5.0	35.0	135	4,720	4,383	2,199	6,199					98	95	431	1.50	0.397	2,678	4,877	0.004	0.048	0.335	2,259	4,457	0.004	0.043
6	A-6b	C	37.5	41.5	4.0	39.5	125	5,220	4,970	2,505	6,505	33	0.207	0.021	0.530				1.69	0.357	2,408	4,913	0.016	0.190	0.310	2,091	4,596	0.014	0.171
	A-6b	C	41.5	45.5	4.0	43.5	125	5,720	5,470	2,756	6,756	33	0.207	0.021	0.530				1.86	0.327	2,207	4,962	0.014	0.166	0.290	1,957	4,712	0.013	0.151
																				Total Settlement:			10.978 in		Total Settlement:			7.450 in	

1. $\sigma_p' = \sigma_{vo}'+\sigma_{m;}$ Estimate σ_m of 2,000 psf in existing fill material and 4,000 psf (moderately overconsolidated) for natural soil deposits; Ref. Table 11.2, Coduto 2003
2. $C_c = 0.009(LL-10)$; Ref. Table 6-9, FHWA GEC 5
3. $C_r = 0.15(Cc)$ for the existing fill and 0.10(Cc) for the natural soil deposits; Ref. Section 8.11, Holtz and Kovacs 1981
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_nN_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \leq \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (Cohesiv soil layers)
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)

W-13-045 - FRA-70-12.68 - Retaining Wall 4W8
MSE Wall Settlement - Sta. 5074+12 to 5080+60

Calculated By: BRT Date: 07/15/2018
Checked By: JPS Date: 07/15/2018

Borings B-019-1-13

H=	42.3	ft	Total wall height
B'=	23.4	ft	Effective footing width due to eccentricity
D_w =	0.0	ft	Depth below bottom of footing
q_e =	6,750	psf	Equivalent bearing pressure at bottom of wall

	A-6b (Upper)	A-6b (Lower)		
c_v =	300	300	ft^2/yr	Coefficient of consolidation
t =	20	20	days	Time following completion of construction
H_{dr} =	5	8	ft	Length of longest drainage path considered
T_v =	0.658	0.257		Time factor
U =	84	57	%	Degree of consolidation

$(S_c)_t$ = 6.777 in Settlement complete at 91% of primary consolidation

																						Total Settlement at Facing of Wall			Settlement Complete at 91% of Primary Consolidation	
Layer	Soil Type	Soil Type	Layer Depth (ft)		Layer Thickness (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	Layer Settlement (in)	$(S_c)_t$ (11) (in)	Layer Settlement (in)
1	A-4a	G	0.0	3.5	3.5	1.8	120	420	210	101	2,101					8	16	34	0.07	0.500	3,374	3,475	0.159	1.909	1.909	1.909	1.909
2	A-6b	C	3.5	6.0	2.5	4.8	115	708	564	267	2,267	40	0.270	0.041	0.585				0.20	0.498	3,364	3,631	0.146	1.757	3.338	1.476	2.804
	A-6b	C	6.0	8.5	2.5	7.3	115	995	851	399	2,399	40	0.270	0.041	0.585				0.31	0.494	3,337	3,736	0.132	1.581		1.328	
3	A-2-4	G	8.5	13.0	4.5	10.8	125	1,558	1,276	605	4,605					15	21	76	0.46	0.484	3,265	3,870	0.047	0.569	1.057	0.569	1.057
	A-2-4	G	13.0	17.5	4.5	15.3	125	2,120	1,839	887	4,887					15	19	72	0.65	0.462	3,117	4,004	0.041	0.488		0.488	
4	A-1-b	G	17.5	22.5	5.0	20.0	125	2,745	2,433	1,185	5,185					20	24	82	0.85	0.432	2,917	4,101	0.033	0.394	0.735	0.394	0.735
	A-1-b	G	22.5	27.5	5.0	25.0	125	3,370	3,058	1,498	5,498					20	22	79	1.07	0.398	2,688	4,186	0.028	0.341		0.341	
5	A-1-a	G	27.5	32.5	5.0	30.0	135	4,045	3,708	1,836	5,836					98	101	476	1.28	0.365	2,465	4,301	0.004	0.047	0.089	0.047	0.089
	A-1-a	G	32.5	37.5	5.0	35.0	135	4,720	4,383	2,199	6,199					98	95	431	1.50	0.335	2,259	4,457	0.004	0.043		0.043	
6	A-6b	C	37.5	41.5	4.0	39.5	125	5,220	4,970	2,505	6,505	33	0.207	0.021	0.530				1.69	0.310	2,091	4,596	0.014	0.171	0.323	0.098	0.184
	A-6b	C	41.5	45.5	4.0	43.5	125	5,720	5,470	2,756	6,756	33	0.207	0.021	0.530				1.86	0.290	1,957	4,712	0.013	0.151		0.086	

Settlement Remaining After Hold Period: 0.673 in

1. $\sigma_p' = \sigma_{vo}'+\sigma_{m;}$ Estimate σ_m of 2,000 psf in existing fill material and 4,000 psf (moderately overconsolidated) for natural soil deposits; Ref. Table 11.2, Coduto 2003
2. $C_c = 0.009(LL-10)$; Ref. Table 6-9, FHWA GEC 5
3. $C_r = 0.15(Cc)$ for the existing fill and $0.10(Cc)$ for the natural soil deposits; Ref. Section 8.11, Holtz and Kovacs 1981
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_nN_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \leq \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (Cohesiv soil layers)
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)
11. $(S_c)_t = S_c(U/100)$; U = 100 for all granular soils at time t = 0

W-13-045 - FRA-70-12.68 - Retaining Wall 4W8
MSE Wall Settlement - Sta. 5074+12 to 5080+60

Calculated By: BRT Date: 7/15/2018
Checked By: JPS Date: 7/15/2018

Borings B-020-1-13

H=	42.3	ft	Total wall height
B'=	23.4	ft	Effective footing width due to eccentricity
D_w =	0.0	ft	Depth below bottom of footing
q_e =	6,750	psf	Equivalent bearing pressure at bottom of wall

																				Total Settlement at Center of Reinforced Soil Mass					Total Settlement at Facing of Wall				
Layer	Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
1	A-6a	C	0.0	2.5	2.5	1.3	120	300	150	72	2,072	35	0.225	0.034	0.546				0.05	0.999	6,747	6,819	0.268	3.215	0.500	3,375	3,447	0.160	1.921
	A-6a	C	2.5	5.5	3.0	4.0	120	660	480	230	2,230	35	0.225	0.034	0.546				0.17	0.985	6,650	6,880	0.278	3.339	0.499	3,368	3,598	0.155	1.864
2	A-6b	C	5.5	8.0	2.5	6.8	120	960	810	389	2,389	40	0.270	0.041	0.585				0.29	0.942	6,361	6,750	0.243	2.910	0.495	3,344	3,733	0.133	1.595
	A-6b	C	8.0	10.5	2.5	9.3	120	1,260	1,110	533	2,533	40	0.270	0.041	0.585				0.40	0.884	5,966	6,499	0.218	2.611	0.489	3,301	3,834	0.120	1.439
3	A-1-b	G	10.5	13.0	2.5	11.8	120	1,560	1,410	677	2,677					4	5	51	0.50	0.817	5,514	6,191	0.047	0.566	0.480	3,237	3,914	0.037	0.449
4	A-1-b	G	13.0	18.0	5.0	15.5	125	2,185	1,873	905	4,905					17	22	78	0.66	0.718	4,845	5,750	0.052	0.621	0.460	3,107	4,012	0.042	0.500
	A-1-b	G	18.0	23.0	5.0	20.5	125	2,810	2,498	1,218	5,218					17	20	74	0.88	0.604	4,079	5,297	0.043	0.518	0.429	2,894	4,112	0.036	0.429
5	A-1-b	G	23.0	27.0	4.0	25.0	130	3,330	3,070	1,510	5,510					42	46	153	1.07	0.523	3,531	5,041	0.014	0.164	0.398	2,688	4,198	0.012	0.139
6	A-4a	C	27.0	32.0	5.0	29.5	130	3,980	3,655	1,814	5,814	22	0.108	0.011	0.444				1.26	0.459	3,095	4,909	0.016	0.194	0.368	2,487	4,301	0.014	0.168
7	A-1-b	G	32.0	39.5	7.5	35.8	135	4,993	4,486	2,255	6,255					100	96	439	1.53	0.390	2,629	4,885	0.006	0.069	0.330	2,230	4,485	0.005	0.061
	A-1-b	G	39.5	47.0	7.5	43.3	135	6,005	5,499	2,800	6,800					100	89	387	1.85	0.329	2,219	5,018	0.005	0.059	0.291	1,965	4,765	0.004	0.054
8	A-7-6	C	47.0	53.0	6.0	50.0	130	6,785	6,395	3,275	7,275	44	0.306	0.031	0.616				2.14	0.288	1,941	5,216	0.023	0.276	0.262	1,765	5,040	0.021	0.255
	A-7-6	C	53.0	59.0	6.0	56.0	130	7,565	7,175	3,681	7,681	44	0.306	0.031	0.616				2.39	0.259	1,745	5,426	0.019	0.230	0.239	1,615	5,295	0.018	0.215
																						Total Settlement:		14.770 in			Total Settlement:		9.089 in

1. $\sigma_p' = \sigma_{vo}'+\sigma_{m;}$ Estimate σ_m of 2,000 psf in existing fill material and 4,000 psf (moderately overconsolidated) for natural soil deposits; Ref. Table 11.2, Coduto 2003
2. C_c = 0.009(LL-10); Ref. Table 6-9, FHWA GEC 5
3. C_r = 0.15(Cc) for the existing fill and 0.10(Cc) for the natural soil deposits; Ref. Section 8.11, Holtz and Kovacs 1981
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_nN_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \le 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$for $\sigma_p' \le \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \le \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (Cohesiv soil layers)
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)

W-13-045 - FRA-70-12.68 - Retaining Wall 4W8
MSE Wall Settlement - Sta. 5074+12 to 5080+60

Calculated By: BRT Date: 07/15/2018
Checked By: JPS Date: 07/15/2018

Borings B-020-1-13

H=	42.3	ft	Total wall height
B'=	23.4	ft	Effective footing width due to eccentricity
D_w =	0.0	ft	Depth below bottom of footing
q_e =	6,750	psf	Equivalent bearing pressure at bottom of wall

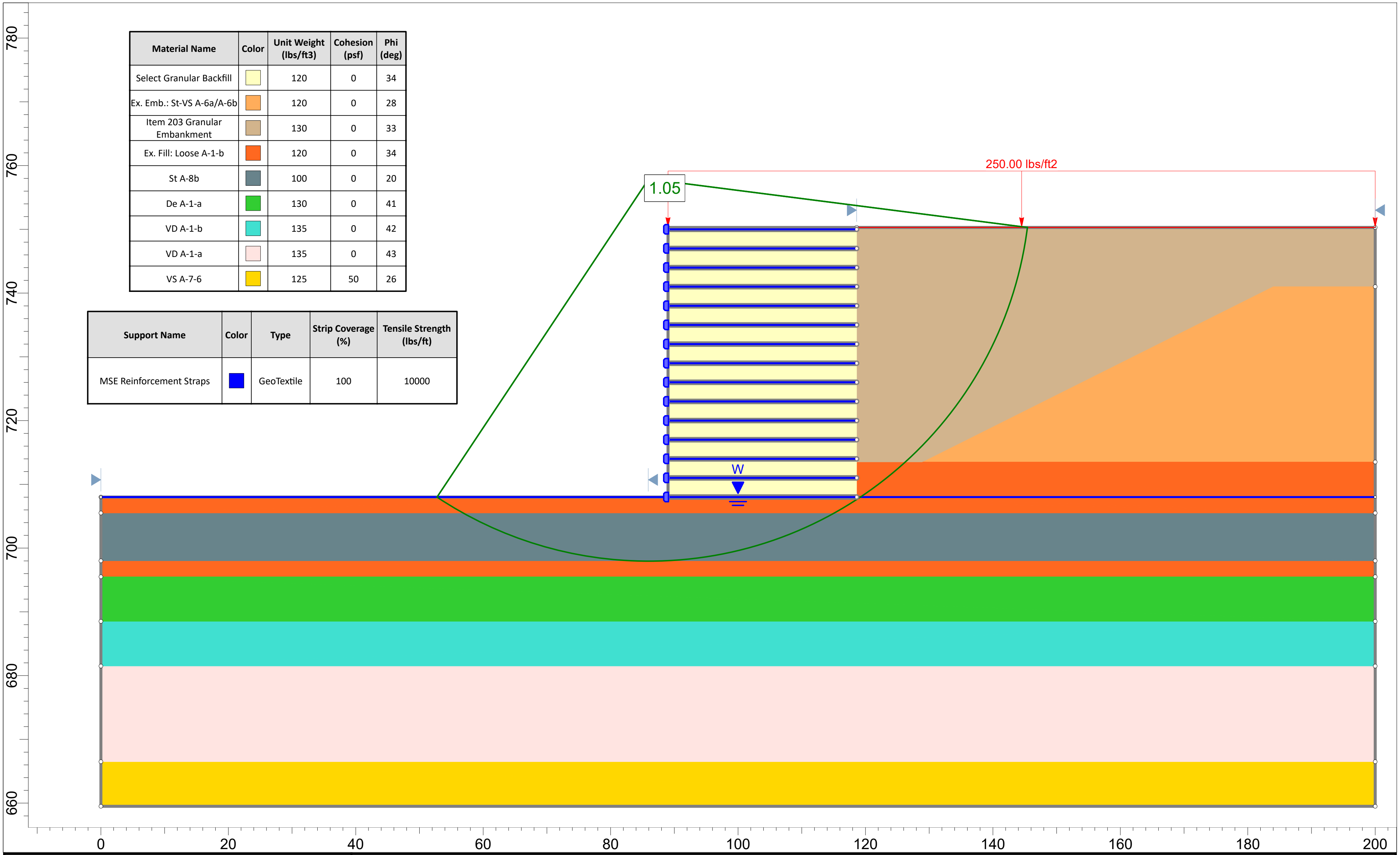
	A-6a	A-6b	A-4a	A-7-6		
c_v =	600	300	800	150	ft^2/yr	Coefficient of consolidation
t =	22	22	22	22	days	Time following completion of construction
H_{dr} =	5.5	5	2.5	12	ft	Length of longest drainage path considered
T_v =	1.196	0.723	7.715	0.063		Time factor
U =	96	86	100	28	%	Degree of consolidation

$(S_c)_t$ = 8.174 in Settlement complete at 90% of primary consolidation

																						Total Settlement at Facing of Wall			Settlement Complete at 90% of Primary Consolidation	
Layer	Soil Type	Soil Type	Layer Depth (ft)		Layer Thickness (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	Layer Settlement (in)	$(S_c)_t$ (11) (in)	Layer Settlement (in)
1	A-6a	C	0.0	2.5	2.5	1.3	120	300	150	72	2,072	35	0.225	0.034	0.546				0.05	0.500	3,375	3,447	0.160	1.921	3.785	1.844	3.633
	A-6a	C	2.5	5.5	3.0	4.0	120	660	480	230	2,230	35	0.225	0.034	0.546				0.17	0.499	3,368	3,598	0.155	1.864		1.789	
2	A-6b	C	5.5	8.0	2.5	6.8	120	960	810	389	2,389	40	0.270	0.041	0.585				0.29	0.495	3,344	3,733	0.133	1.595	3.034	1.372	2.609
	A-6b	C	8.0	10.5	2.5	9.3	120	1,260	1,110	533	2,533	40	0.270	0.041	0.585				0.40	0.489	3,301	3,834	0.120	1.439		1.238	
3	A-1-b	G	10.5	13.0	2.5	11.8	120	1,560	1,410	677	2,677					4	5	51	0.50	0.480	3,237	3,914	0.037	0.449	0.449	0.449	0.449
4	A-1-b	G	13.0	18.0	5.0	15.5	125	2,185	1,873	905	4,905					17	22	78	0.66	0.460	3,107	4,012	0.042	0.500	0.929	0.500	0.929
	A-1-b	G	18.0	23.0	5.0	20.5	125	2,810	2,498	1,218	5,218					17	20	74	0.88	0.429	2,894	4,112	0.036	0.429		0.429	
5	A-1-b	G	23.0	27.0	4.0	25.0	130	3,330	3,070	1,510	5,510					42	46	153	1.07	0.398	2,688	4,198	0.012	0.139	0.139	0.139	0.139
6	A-4a	C	27.0	32.0	5.0	29.5	130	3,980	3,655	1,814	5,814	22	0.108	0.011	0.444				1.26	0.368	2,487	4,301	0.014	0.168	0.168	0.168	0.168
7	A-1-b	G	32.0	39.5	7.5	35.8	135	4,993	4,486	2,255	6,255					100	96	439	1.53	0.330	2,230	4,485	0.005	0.061	0.115	0.061	0.115
	A-1-b	G	39.5	47.0	7.5	43.3	135	6,005	5,499	2,800	6,800					100	89	387	1.85	0.291	1,965	4,765	0.004	0.054		0.054	
8	A-7-6	C	47.0	53.0	6.0	50.0	130	6,785	6,395	3,275	7,275	44	0.306	0.031	0.616				2.14	0.262	1,765	5,040	0.021	0.255	0.471	0.071	0.132
	A-7-6	C	53.0	59.0	6.0	56.0	130	7,565	7,175	3,681	7,681	44	0.306	0.031	0.616				2.39	0.239	1,615	5,295	0.018	0.215		0.060	

Settlement Remaining After Hold Period: 0.915 in

1. $\sigma_p' = \sigma_{vo}'+\sigma_{m;}$ Estimate σ_m of 2,000 psf in existing fill material and 4,000 psf (moderately overconsolidated) for natural soil deposits; Ref. Table 11.2, Coduto 2003
2. $C_c = 0.009(LL-10)$; Ref. Table 6-9, FHWA GEC 5
3. $C_r = 0.15(Cc)$ for the existing fill and 0.10(Cc) for the natural soil deposits; Ref. Section 8.11, Holtz and Kovacs 1981
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_n N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \leq \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (Cohesiv soil layers)
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)
11. $(S_c)_t = S_c(U/100)$; U = 100 for all granular soils at time t = 0

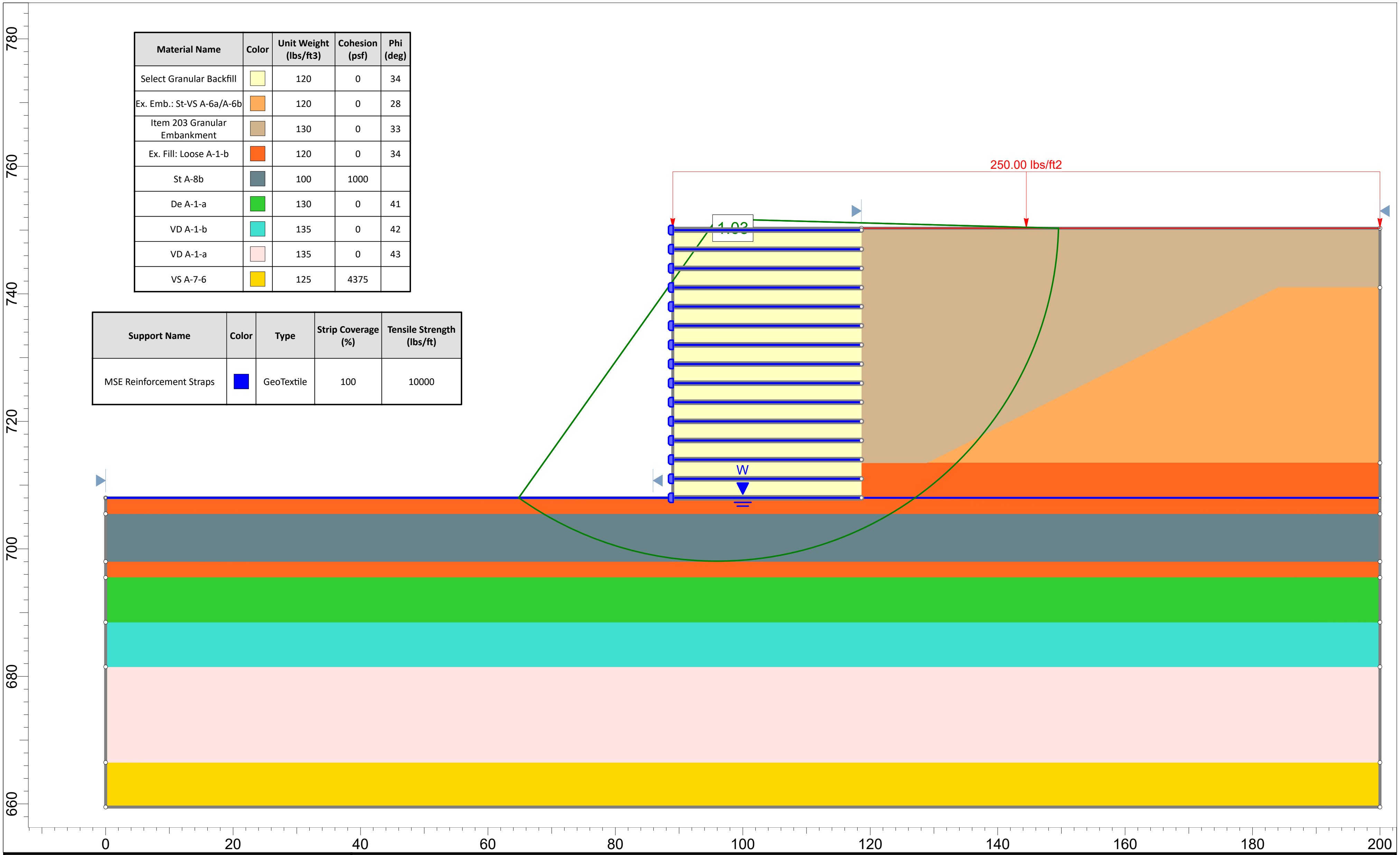


Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Select Granular Backfill	🟨	120	0	34
Ex. Emb.: St-VS A-6a/A-6b	🟧	120	0	28
Item 203 Granular Embankment	🟫	130	0	33
Ex. Fill: Loose A-1-b	🟧	120	0	34
St A-8b	⬛	100	0	20
De A-1-a	🟩	130	0	41
VD A-1-b	🟦	135	0	42
VD A-1-a	⬜	135	0	43
VS A-7-6	🟨	125	50	26

Support Name	Color	Type	Strip Coverage (%)	Tensile Strength (lbs/ft)
MSE Reinforcement Straps	🟦	GeoTextile	100	10000

SLIDEINTERPRET 7.020

Project	FRA-70-12.68 - Retaining Wall 4W8 - MSE Wall Global Stability		
Analysis Description	Sta. 5076+00 (BL Ramp C5) - Borings B-017-7-13 and B-019-0-08 - Spencer's Method - Drained Conditions		
Drawn By	BRT	*Scale*	1:250
Company	Resource International, Inc.		
Date	07/15/2018	*File Name*	Retaining Wall 4W8 - Global Stability.slim



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Select Granular Backfill	🟨	120	0	34
Ex. Emb.: St-VS A-6a/A-6b	🟧	120	0	28
Item 203 Granular Embankment	🟫	130	0	33
Ex. Fill: Loose A-1-b	🟧	120	0	34
St A-8b	⬛	100	1000	
De A-1-a	🟩	130	0	41
VD A-1-b	🟦	135	0	42
VD A-1-a	⬜	135	0	43
VS A-7-6	🟨	125	4375	

Support Name	Color	Type	Strip Coverage (%)	Tensile Strength (lbs/ft)
MSE Reinforcement Straps	🟦	GeoTextile	100	10000

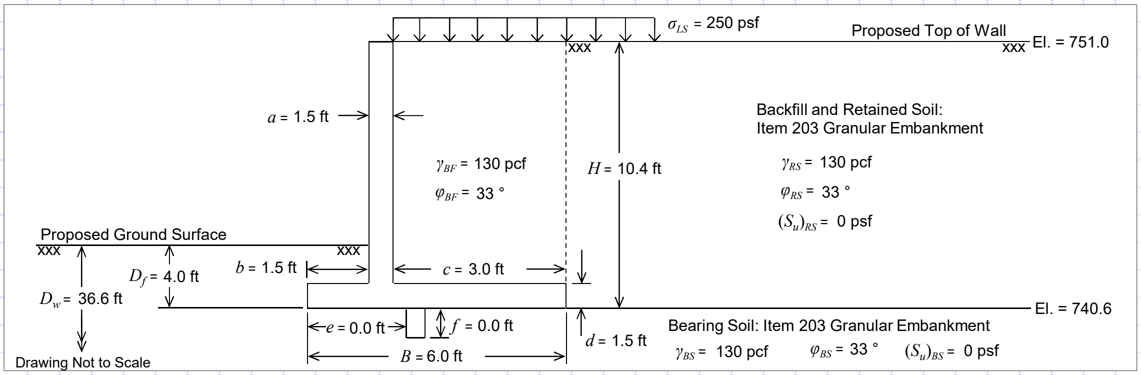
SLIDEINTERPRET 7.020

Project	FRA-70-12.68 - Retaining Wall 4W8 - MSE Wall Global Stability
Analysis Description	Sta. 5076+00 (BL Ramp C5) - Borings B-017-7-13 and B-019-0-08 - Spencer's Method - Undrained Conditions
Drawn By	BRT
Scale	1:250
Company	Resource International, Inc.
Date	07/15/2018
File Name	Retaining Wall 4W8 - Global Stability.slim

APPENDIX VI

CIP WALL CALCULATIONS

Retaining Wall 4W9 - CIP Wall Without Shear Key - 10.4 ft. Maximum Wall Height



CIP Wall Dimensions and Surcharge Loading

Wall Height, (H) =	10.4	ft
Foundation Width (Entire Base Width), (B) =	6.0	ft
Stem Width, (a) =	1.5	ft
Toe Width, (b) =	1.5	ft
Heel Width, (c) =	3.0	ft
Footing Thickness, (d) =	1.5	ft
Location of Shear Key, (e) =	0.0	ft
Depth of Shear Key, (f) =	0.0	ft
Embedment Depth, (D_f) =	4.0	ft
Wall Length, (L) =	143	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Depth to Groundwater, (D_w) =	36.6	ft

Bearing and Retained/Backfill Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	130	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	33	°
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	0	psf
Backfill and Retained Soil Unit Weight, (γ_{BF}, γ_{RS}) =	130	pcf
Retained Soil Friction Angle, (φ_{RS}) =	33	°
Retained Soil Undrained Shear Strength, [$(s_u)_{RS}$] =	0	psf
Active Earth Pressure Coefficient, (K_a) =	0.264	
Passive Earth Pressure Coefficient, (K_p) =	7.410	

LRFD Load Factors

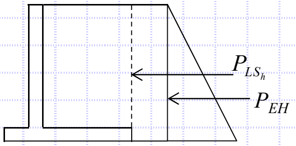
	DC	EV	EH	LS	EP
Strength Ia	0.90	1.00	1.50	1.75	0.90
Strength Ib	1.25	1.35	1.50	1.75	0.90
Service I	1.00	1.00	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

Check Sliding (Loading Case - Strength Ia) - AASHTO LRFD BDM Section 10.6.3.4

Sliding Force:

$$P_H = P_{EH} + P_{LS_h}$$



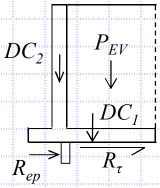
$P_{EH} = \frac{1}{2}\gamma_{RS}H^2K_a\gamma_{EH}$ = $\frac{1}{2}(130\text{ pcf})(10.4\text{ ft})^2(0.264)(1.50)$ = 2.78 kip/ft

$P_{LS_h} = \sigma_{LS}HK_a\gamma_{LS}$ (250 psf)(10.4 ft)(0.264)(1.75) = 1.2 kip/ft

P_H = 2.78 kip/ft + 1.2 kip/ft = 3.98 kip/ft

Check Sliding Resistance

Nominal Sliding Resisting: $R_n = R_\tau + R_{ep}$



$$R_{ep} = \gamma_{BS}D_f fK_p\gamma_{ep} + \tfrac{1}{2}\gamma_{BS}f^2K_p\gamma_{ep}$$

R_{ep} = (130 pcf)(4.0 ft)(0.0 ft)(7.41)(0.90) + $\frac{1}{2}(130\text{ pcf})(0.0\text{ ft})^2(7.41)(0.90)$ = 0.00 kip/ft

Check Drained Condition: $R_\tau = P_V \tan\delta$

$$P_V = DC_1 + DC_2 + P_{EV} = \gamma_c \cdot [B \cdot d + (H-d)\cdot a]\cdot \gamma_{DC} + \gamma_{BF}\cdot (H-d)\cdot c \cdot \gamma_{EV}$$

P_V = (150 pcf) [(6.0 ft)(1.5 ft) + (10.4 ft - 1.5 ft)(1.5 ft)] (0.90) + (130 pcf)(10.4 ft - 1.5 ft)(3.0 ft)(1.00) = 6.49 kip/ft

$\tan\delta = \tan\varphi_{BS}$ = tan(33) = 0.65

R_τ = (6.49 kip/ft)(0.65) = 4.22 kip/ft

Verify Sliding Force Less Than Factored Sliding Resistance - Drained Condition

$P_H \le \phi_n \cdot R_n \longrightarrow P_H \le \phi_\tau \cdot R_\tau + \phi_{ep}\cdot R_{ep} \longrightarrow$ 3.98 kip/ft ≤ (4.22 kip/ft)(1.00) + (0.00 kip/ft)(0.50) = 4.22 kip/ft

= 3.98 kip/ft ≤ 4.22 kip/ft **OK**

Use φ_τ = **1.00** Use φ_{ep} = **0.50** *(Per AASHTO LRFD BDM Tables 10.5.5.2.2-1 and 11.5.7-1)*

CIP Wall Dimensions and Surcharge Loading

Wall Height, (H) =	10.4	ft
Foundation Width (Entire Base Width), (B) =	6.0	ft
Stem Width, (a) =	1.5	ft
Toe Width, (b) =	1.5	ft
Heel Width, (c) =	3.0	ft
Footing Thickness, (d) =	1.5	ft
Location of Shear Key, (e) =	0.0	ft
Depth of Shear Key, (f) =	0.0	ft
Embedment Depth, (D_f) =	4.0	ft
Wall Length, (L) =	143	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Depth to Groundwater, (D_w) =	36.6	ft

Bearing and Retained/Backfill Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	130	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	33	°
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	0	psf
Backfill and Retained Soil Unit Weight, $(\gamma_{BF}, \gamma_{RS})$ =	130	pcf
Retained Soil Friction Angle, (φ_{RS}) =	33	°
Retained Soil Undrained Shear Strength, $[(s_u)_{RS}]$ =	0	psf
Active Earth Pressure Coefficient, (K_a) =	0.264	
Passive Earth Pressure Coefficient, (K_p) =	7.410	

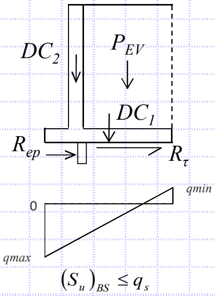
LRFD Load Factors

	DC	EV	EH	LS	EP
Strength Ia	0.90	1.00	1.50	1.75	0.90
Strength Ib	1.25	1.35	1.50	1.75	0.90
Service I	1.00	1.00	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

Check Sliding (Loading Case - Strength Ia) - AASHTO LRFD BDM Section 10.6.3.4 *(Continued)*

Check Undrained Condition: $R_\tau = \left((S_u)_{BS} \leq q_s\right) \cdot B$



$(S_u)_{BS}$ = N/A ksf

$q_{max} = \frac{1}{2}\sigma_{vmax}$ = (3.09 ksf) / 2 = 1.55 ksf

$q_{min} = \frac{1}{2}\sigma_{vmin}$ = (-0.92 ksf) / 2 = -0.46 ksf

$\sigma_{vmax} = \frac{P_V}{B}\left(1 + 6\frac{e}{B}\right)$ = (6.49 kip/ft / 6.0 ft)[1 + 6(1.86 ft / 6.0 ft)] = 3.09 ksf

$\sigma_{vmin} = \frac{P_V}{B}\left(1 - 6\frac{e}{B}\right)$ = (6.49 kip/ft / 6.0 ft)[1 - 6(1.86 ft / 6.0 ft)] = -0.92 ksf

R_τ = N/A = N/A kip/ft

Verify Sliding Force Less Than Factored Sliding Resistance - Undrained Condition

$P_H \leq \phi_n \cdot R_n \longrightarrow P_H \leq \phi_\tau \cdot R_\tau + \phi_{ep} \cdot R_{ep} \longrightarrow$ N/A

= N/A

Use φ_τ = **1.00** Use φ_{ep} = **0.50** *(Per AASHTO LRFD BDM Tables 10.5.5.2.2-1 and 11.5.7-1)*

CIP Wall Dimensions and Surcharge Loading

Wall Height, (H) =	10.4	ft
Foundation Width (Entire Base Width), (B) =	6.0	ft
Stem Width, (a) =	1.5	ft
Toe Width, (b) =	1.5	ft
Heel Width, (c) =	3.0	ft
Footing Thickness, (d) =	1.5	ft
Location of Shear Key, (e) =	0.0	ft
Depth of Shear Key, (f) =	0.0	ft
Embedment Depth, (D_f) =	4.0	ft
Wall Length, (L) =	143	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Depth to Groundwater, (D_w) =	36.6	ft

Bearing and Retained/Backfill Soil Properties:

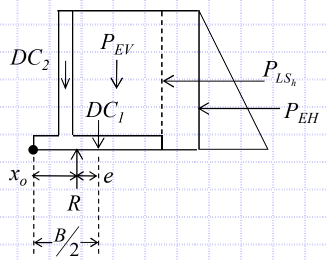
Bearing Soil Unit Weight, (γ_{BS}) =	130	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	33	°
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	0	psf
Backfill and Retained Soil Unit Weight, $(\gamma_{BF}, \gamma_{RS})$ =	130	pcf
Retained Soil Friction Angle, (φ_{RS}) =	33	°
Retained Soil Undrained Shear Strength, $[(s_u)_{RS}]$ =	0	psf
Active Earth Pressure Coefficient, (K_a) =	0.264	
Passive Earth Pressure Coefficient, (K_p) =	7.410	

LRFD Load Factors

	DC	EV	EH	LS	EP
Strength Ia	0.90	1.00	1.50	1.75	0.90
Strength Ib	1.25	1.35	1.50	1.75	0.90
Service I	1.00	1.00	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

Check Eccentricity (Loading Case - Strength Ia) - AASHTO LRFD BDM Section 11.6.3.3



$$e = B/2 - x_o$$

$$x_o = \frac{M_V - M_H}{P_V} = \text{(23.32 kip·ft/ft - 15.89 kip·ft/ft) / (6.49 kip/ft)} = 1.15 \text{ ft}$$

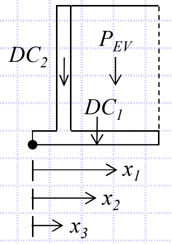
$$\left.\begin{array}{lll} M_V = & 23.32 & \text{kip·ft/ft} \\ M_H = & 15.89 & \text{kip·ft/ft} \end{array}\right\} \text{Defined below}$$

$$P_V = P_{EV} + DC_1 + DC_2 = \text{3.47 kip/ft + 1.22 kip/ft + 1.80 kip/ft} = 6.49 \text{ kip/ft}$$

$$e = \text{(6.0 ft / 2) - 1.15 ft} = 1.86 \text{ ft}$$

Resisting Moment, M_V:

$$M_V = P_{EV}(x_1) + DC_1(x_2) + DC_2(x_3)$$



$$P_{EV} = \gamma_{BF} \cdot (H - d) \cdot c \cdot \gamma_{EV} = \text{(130 pcf)(10.4 ft - 1.5 ft)(3.0 ft)(1.00)} = 3.47 \text{ kip/ft}$$

$$DC_1 = \gamma_c \cdot B \cdot d \cdot \gamma_{DC} = \text{(150 pcf)(6.0 ft)(1.5 ft)(0.90)} = 1.22 \text{ kip/ft}$$

$$DC_2 = \gamma_c \cdot (H - d) \cdot a \cdot \gamma_{DC} = \text{(150 pcf)(10.4 ft - 1.5 ft)(1.5 ft)(0.90)} = 1.80 \text{ kip/ft}$$

$$x_1 = a + b + c/2 = \text{1.5 ft + 1.5 ft + (3.0 ft / 2)} = 4.5 \text{ ft}$$

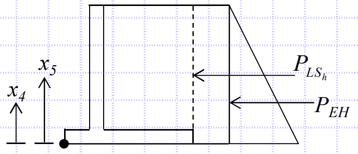
$$x_2 = B/2 = \text{6.0 ft / 2} = 3.0 \text{ ft}$$

$$x_3 = b + a/2 = \text{1.5 ft + (1.5 ft / 2)} = 2.3 \text{ ft}$$

$$M_V = \text{(3.47 kip/ft)(4.5 ft) + (1.22 kip/ft)(3.0 ft) + (1.80 kip/ft)(2.3 ft)} = 23.32 \text{ kip·ft/ft}$$

Overturning Moment, M_H:

$$M_H = P_{EH}(x_2) + P_{LS_h}(x_3)$$



$$P_{EH} = \tfrac{1}{2}\gamma_{RS} H^2 K_a \gamma_{EH} = \text{½(130 pcf)(10.4 ft)²(0.264)(1.50)} = 2.78 \text{ kip/ft}$$

$$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS} = \text{(250 psf)(10.4 ft)(0.264)(1.75)} = 1.2 \text{ kip/ft}$$

$$x_2 = H/3 = \text{(10.4 ft) / 3} = 3.47 \text{ ft}$$

$$x_3 = H/2 = \text{(10.4 ft) / 2} = 5.20 \text{ ft}$$

$$M_H = \text{(2.78 kip/ft)(3.47 ft) + (1.2 kip/ft)(5.20 ft)} = 15.89 \text{ kip·ft/ft}$$

Limiting Eccentricity:

$$e_{\max} = B/3 \longrightarrow e_{\max} = \text{(6.0 ft) / 3} = 2.00 \text{ ft}$$

Check Eccentricity

$$e < e_{\max} \longrightarrow \text{1.86 ft < 2.00 ft} \quad \textbf{OK}$$

JOB FRA-70-12.68 NO. W-13-045
SHEET NO. 4 OF 6
CALCULATED BY BRT DATE 7/15/2018
CHECKED BY JPS DATE 7/15/2018
Retaining Wall 4W9 - 10.4 ft Maximum Wall Height

CIP Wall Dimensions and Surcharge Loading

Wall Height, (H) =	10.4	ft
Foundation Width (Entire Base Width), (B) =	6.0	ft
Stem Width, (a) =	1.5	ft
Toe Width, (b) =	1.5	ft
Heel Width, (c) =	3.0	ft
Footing Thickness, (d) =	1.5	ft
Location of Shear Key, (e) =	0.0	ft
Depth of Shear Key, (f) =	0.0	ft
Embedment Depth, (D_f) =	4.0	ft
Wall Length, (L) =	143	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Depth to Groundwater, (D_w) =	36.6	ft

Bearing and Retained/Backfill Soil Properties:

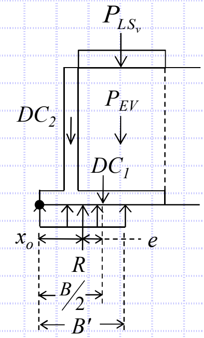
Bearing Soil Unit Weight, (γ_{BS}) =	130	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	33	°
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	0	psf
Backfill and Retained Soil Unit Weight, (γ_{BF}, γ_{RS}) =	130	pcf
Retained Soil Friction Angle, (φ_{RS}) =	33	°
Retained Soil Undrained Shear Strength, [$(s_u)_{RS}$] =	0	psf
Active Earth Pressure Coefficient, (K_a) =	0.264	
Passive Earth Pressure Coefficient, (K_p) =	7.410	

LRFD Load Factors

	DC	EV	EH	LS	EP
Strength Ia	0.90	1.00	1.50	1.75	0.90
Strength Ib	1.25	1.35	1.50	1.75	0.90
Service I	1.00	1.00	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

Check Bearing Capacity (Loading Case - Strength Ib) - AASHTO LRFD BDM Section 11.6.3.2



$$q_{eq} = P_V \big/ B'$$

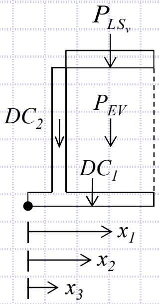
$$B' = B - 2e$$ = 6.0 ft - 2(0.59 ft) = 4.82 ft

$$e = B \big/ 2 - x_o$$ = (6.0 ft / 2) - 2.41 ft = 0.59 ft

$$x_o = \frac{M_V - M_H}{P_V}$$ = (43.59 kip·ft/ft - 15.89 kip·ft/ft) / (11.50 kip/ft) = 2.41 ft

q_{eq} = (11.50 kip/ft) / (4.82 ft) = 2.39 ksf

Resisting Moment, M_V:



$$M_V = P_{EV}(x_1) + P_{LS_v}(x_1) + DC_1(x_2) + DC_2(x_3)$$

$$P_{EV} = \gamma_{BF} \cdot (H - d) \cdot c \cdot \gamma_{EV}$$ = (130 pcf)(10.4 ft - 1.5 ft)(3.0 ft)(1.35) = 4.69 kip/ft

$$P_{LS_v} = \sigma_{LS} \cdot B \cdot \gamma_{LS}$$ = (250 psf)(6.0 ft)(1.75) = 2.625 kip/ft

$$DC_1 = \gamma_c \cdot B \cdot d \cdot \gamma_{DC}$$ = (150 pcf)(6.0 ft)(1.5 ft)(1.25) = 1.69 kip/ft

$$DC_2 = \gamma_c \cdot (H - d) \cdot a \cdot \gamma_{DC}$$ = (150 pcf)(10.4 ft - 1.5 ft)(1.5 ft)(1.25) = 2.50 kip/ft

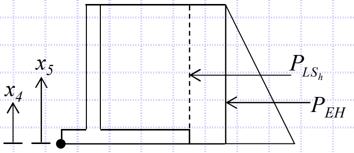
$$x_1 = a + b + c \big/ 2$$ = 1.5 ft + 1.5 ft + (3.0 ft / 2) = 4.5 ft

$$x_2 = B \big/ 2$$ = 6.0 ft / 2 = 3.0 ft

$$x_3 = b + a \big/ 2$$ = 1.5 ft + (1.5 ft / 2) = 2.3 ft

M_V = (4.69 kip/ft)(4.5 ft) + (2.63 kip/ft)(4.5 ft) + (1.69 kip/ft)(3.0 ft) + (2.50 kip/ft)(2.3 ft) = 43.59 kip·ft/ft

Overturning Moment, M_H:



$$M_H = P_{EH}(x_4) + P_{LS_h}(x_5)$$

$$P_{EH} = \tfrac{1}{2} \gamma_{RS} H^2 K_a \gamma_{EH}$$ = ½(130 pcf)(10.4 ft)²(0.264)(1.50) = 2.78 kip/ft

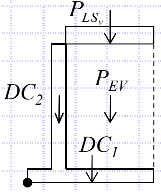
$$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS}$$ = (250 psf)(10.4 ft)(0.264)(1.75) = 1.2 kip/ft

$$x_4 = H \big/ 3$$ = (10.4 ft) / 3 = 3.47 ft

$$x_5 = H \big/ 2$$ = (10.4 ft) / 2 = 5.20 ft

M_H = (2.78 kip/ft)(3.47 ft) + (1.2 kip/ft)(5.20 ft) = 15.89 kip·ft/ft

Vertical Force, P_V:



$$P_V = P_{EV} + P_{LS_v} + DC_1 + DC_2$$

P_V = 4.69 kip/ft + 2.63 kip/ft + 1.69 kip/ft + 2.50 kip/ft

P_V = 11.50 kip/ft

JOB	FRA-70-12.68	NO.	W-13-045
SHEET NO.	5	OF	6
CALCULATED BY	BRT	DATE	7/15/2018
CHECKED BY	JPS	DATE	7/15/2018

Retaining Wall 4W9 - 10.4 ft Maximum Wall Height

CIP Wall Dimensions and Surcharge Loading

Wall Height, (H) =	10.4	ft
Foundation Width (Entire Base Width), (B) =	6.0	ft
Stem Width, (a) =	1.5	ft
Toe Width, (b) =	1.5	ft
Heel Width, (c) =	3.0	ft
Footing Thickness, (d) =	1.5	ft
Location of Shear Key, (e) =	0.0	ft
Depth of Shear Key, (f) =	0.0	ft
Embedment Depth, (D_f) =	4.0	ft
Wall Length, (L) =	143	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Depth to Groundwater, (D_w) =	36.6	ft

Bearing and Retained/Backfill Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	130	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	33	°
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	0	psf
Backfill and Retained Soil Unit Weight, (γ_{BF}, γ_{RS}) =	130	pcf
Retained Soil Friction Angle, (φ_{RS}) =	33	°
Retained Soil Undrained Shear Strength, [$(s_u)_{RS}$] =	0	psf
Active Earth Pressure Coefficient, (K_a) =	0.264	
Passive Earth Pressure Coefficient, (K_p) =	7.410	

LRFD Load Factors

	DC	EV	EH	LS	EP
Strength Ia	0.90	1.00	1.50	1.75	0.90
Strength Ib	1.25	1.35	1.50	1.75	0.90
Service I	1.00	1.00	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

Check Bearing Capacity (Loading Case - Strength Ib) - AASHTO LRFD BDM Section 11.6.3.2 *(Continued)*

Check Bearing Resistance - Drained Condition

Nominal Bearing Resistance: $q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \frac{1}{2}\gamma B' N_{\gamma m} C_{w\gamma}$

$N_{cm} = N_c s_c i_c$ = 39.527

N_c = 38.638
s_c = 1+(4.82 ft/143 ft)(26.092/38.638)
= 1.023
i_c = 1.000 (Assumed)

$N_{qm} = N_q s_q d_q i_q$ = 31.653

N_q = 26.092
s_q = 1+(4.82 ft/143 ft)tan(33°) = 1.022
d_q = 1+2tan(33°)[1-sin(33°)]²tan⁻¹(4.0 ft/4.82 ft)
= 1.187
i_q = 1.000 (Assumed)
C_{wq} = 36.6 ft > 4.0 ft = 1.000

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 34.731

N_γ = 35.188
s_γ = 1-0.4(4.82 ft/143 ft) = 0.987
i_γ = 1.000 (Assumed)
$C_{w\gamma}$ = 36.6 ft > 1.5(4.82 ft) + 4.0 ft = 1.000

$q_n =$ (0 psf)(39.527) + (130 pcf)(4.0 ft)(31.653)(1.000) + ½(130 pcf)(4.8 ft)(34.731)(1.000) = 27.34 ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

$q_{eq} \leq q_n \cdot \phi_b$ ⟶ 2.39 ksf ≤ (27.34 ksf)(0.55) = 15.04 ksf ⟶ 2.39 ksf ≤ 15.04 ksf **OK**

Use φ_b = **0.55** *(Per AASHTO LRFD BDM Table 11.5.7-1)*

Check Bearing Resistance - Undrained Condition

Nominal Bearing Resistance: $q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \frac{1}{2}\gamma B' N_{\gamma m} C_{w\gamma}$

$N_{cm} = N_c s_c i_c$ = 5.258

N_c = 5.140
s_c = 1+(4.82 ft/[(5)(143 ft)]) = 1.023
i_c = 1.000 (Assumed)

$N_{qm} = N_q s_q d_q i_q$ = 1.000

N_q = 1.000
s_q = 1.000
d_q = 1+2tan(0°)[1-sin(0°)]²tan⁻¹(4.0 ft/4.82 ft)
= 1.000
i_q = 1.000 (Assumed)
C_{wq} = 36.6 ft > 4.0 ft = 1.000

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 0.000

N_γ = 0.000
s_γ = 1.000
i_γ = 1.000 (Assumed)
$C_{w\gamma}$ = 36.6 ft > 1.5(4.82 ft) + 4.0 ft = 1.000

$q_n =$ (0 psf)(5.258) + (130 pcf)(4.0 ft)(1.000)(1.000) + ½(130 pcf)(4.8 ft)(0.000)(1.000) = N/A ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

$q_{eq} \leq q_n \cdot \phi_b$ ⟶ N/A ⟶ N/A

Use φ_b = **0.55** *(Per AASHTO LRFD BDM Table 11.5.7-1)*

Retaining Wall 4W9 - 10.4 ft Maximum Wall Height

CIP Wall Dimensions and Surcharge Loading

Wall Height, (H) =	10.4	ft
Foundation Width (Entire Base Width), (B) =	6.0	ft
Stem Width, (a) =	1.5	ft
Toe Width, (b) =	1.5	ft
Heel Width, (c) =	3.0	ft
Footing Thickness, (d) =	1.5	ft
Location of Shear Key, (e) =	0.0	ft
Depth of Shear Key, (f) =	0.0	ft
Embedment Depth, (D_f) =	4.0	ft
Wall Length, (L) =	143	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Depth to Groundwater, (D_w) =	36.6	ft

Bearing and Retained/Backfill Soil Properties:

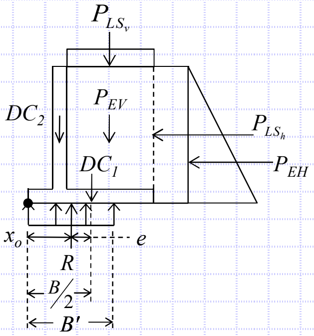
Bearing Soil Unit Weight, (γ_{BS}) =	130	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	33	°
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	0	psf
Backfill and Retained Soil Unit Weight, (γ_{BF}, γ_{RS}) =	130	pcf
Retained Soil Friction Angle, (φ_{RS}) =	33	°
Retained Soil Undrained Shear Strength, [$(s_u)_{RS}$] =	0	psf
Active Earth Pressure Coefficient, (K_a) =	0.264	
Passive Earth Pressure Coefficient, (K_p) =	7.410	

LRFD Load Factors

	DC	EV	EH	LS	EP
Strength Ia	0.90	1.00	1.50	1.75	0.90
Strength Ib	1.25	1.35	1.50	1.75	0.90
Service I	1.00	1.00	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

Check Bearing Capacity (Loading Case - Strength Ib) - AASHTO LRFD BDM Section 11.6.3.2



$$q_{eq} = P_V / B'$$

$$B' = B - 2e \quad = \quad 6.0\text{ ft} - 2(0.48\text{ ft}) \quad = \quad 5.04 \quad \text{ft}$$

$$e = B/2 - x_o \quad = \quad (6.0\text{ ft} / 2) - 2.52\text{ ft} \quad = \quad 0.48 \quad \text{ft}$$

$$x_o = \frac{M_V - M_H}{P_V} \quad = \quad (30.93\text{ kip·ft/ft} - 10.00\text{ kip·ft/ft}) / (8.32\text{ kip/ft}) \quad = \quad 2.52 \quad \text{ft}$$

$$q_{eq} \quad = \quad (8.32\text{ kip/ft}) / (5.04\text{ ft}) \quad = \quad 1.65 \quad \text{ksf}$$

$$M_V = [(\gamma_{BF} \cdot (H-d) \cdot c \cdot \gamma_{EV}) + (\sigma_{LS} \cdot B \cdot \gamma_{LS})](a + b + c/2) + (\gamma_c \cdot B \cdot d \cdot \gamma_{DC})(B/2) + (\gamma_c \cdot (H-d) \cdot a \cdot \gamma_{DC})(b + a/2)$$

M_V = [(130 pcf)(10.4 ft - 1.5 ft)(3.0 ft)(1.00) + (250 psf)(6.0 ft)(1.00)](1.5 ft + 1.5 ft + (3.0 ft / 2))
+ [(150 pcf)(6.0 ft)(1.5 ft)(1.00)](6.0 ft / 2) + [(150 pcf)(10.4 ft - 1.5 ft)(1.5 ft)(1.00)](1.5 ft + (1.5 ft / 2)) = 30.93 kip·ft/ft

$$M_H = (\tfrac{1}{2}\gamma_{RS} \cdot H^2 \cdot K_a \cdot \gamma_{EH})(H/3) + (\sigma_{LS} \cdot H \cdot K_a \cdot \gamma_{LS})(H/2)$$

M_H = [½(130 pcf)(10.4 ft)²(0.264)(1.00)](10.4 ft / 3) + [(250 psf)(10.4 ft)(0.264)(1.00)](10.4 ft / 2) = 10 kip·ft/ft

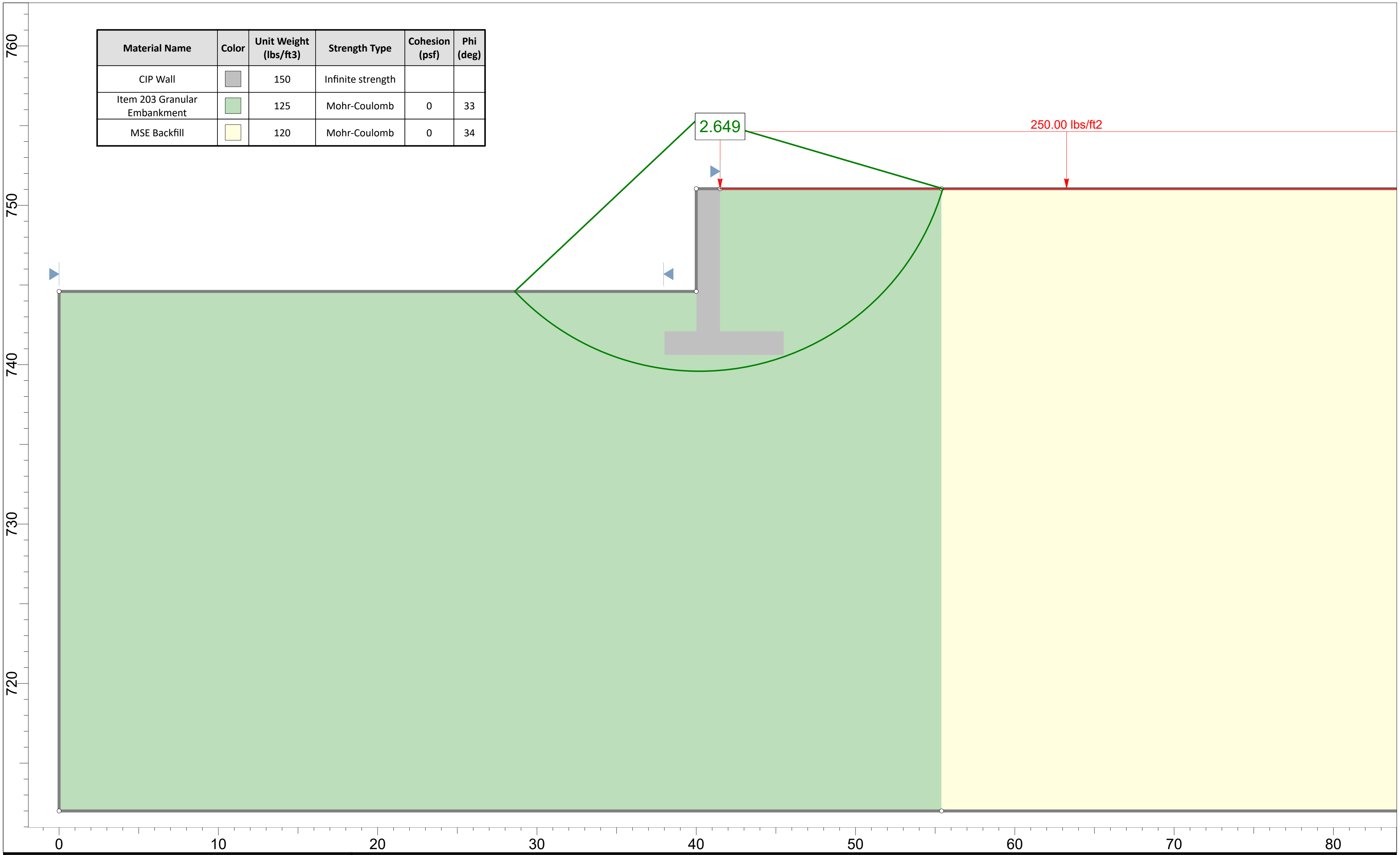
$$P_V = (\gamma_{BF} \cdot (H-d) \cdot c \cdot \gamma_{EV}) + (\sigma_{LS} \cdot B \cdot \gamma_{LS}) + (\gamma_c \cdot B \cdot d \cdot \gamma_{DC}) + (\gamma_c \cdot (H-d) \cdot a \cdot \gamma_{DC})$$

P_V = (130 pcf)(10.4 ft - 1.5 ft)(3.0 ft)(1.00) + (250 psf)(6.0 ft)(1.00) + (150 pcf)(6.0 ft)(1.5 ft)(1.00)
+ (150 pcf)(10.4 ft - 1.5 ft)(1.5 ft)(1.00) = 8.32 kip/ft

Settlement (See Attached Spreadsheet Calculations):

Total Settlement at Maximum Wall Height:	$(S_t)_{\max}$ =	N/A	in
Total Settlement at Minimum Wall Height:	$(S_t)_{\min}$ =	N/A	in
Differential Settlement Along Wall Alignment:	δ_s =	N/A	

$\delta_s < 1/500 \longrightarrow$ N/A < 1/500



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
CIP Wall	▨	150	Infinite strength		
Item 203 Granular Embankment	🟩	125	Mohr-Coulomb	0	33
MSE Backfill	🟨	120	Mohr-Coulomb	0	34

Project	FRA-70-12.68 - Retaining Wall 4W9 - CIP Wall Global Stability		
Analysis Description	Sta. 5080+00 (BL Ramp C5) - Spencer's Method - Drained Conditions		
Drawn By	BRT	*Scale* 1:100	*Company* Resource International, Inc.
Date	7/15/2018	*File Name*	Retaining Wall 4W9 - Global Stability.slim

SLIDEINTERPRET 7.020