Resource International, Inc.

FRA-70-12.68 PROJECT 4R
RETAINING WALLS 4W5 AND 4W6
PID NO. 105523
FRANKLIN COUNTY, OHIO

STRUCTURE FOUNDATION EXPLORATION REPORT

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

July 2018

RESOURCE INTERNATIONAL, INC.
ISO 9001:2008 Certified QMS
An ISO 9001:2008 QMS Certified Firm

December 1, 2015 (Revised July 16, 2018)

Mr. Christopher W. Luzier, P.E.
Project Manager
GPD GROUP
1801 Watermark Drive, Suite 210
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Re: Structure Foundation Exploration Report
FRA-70-12.68 Project 4R
Retaining Walls 4W5 and 4W6
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 4W5 and 4W6 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 R. Trenner, P.E.
Director – Geotechnical Programming

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

Enclosure: Structure Foundation Exploration Report

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EXECUTIVE SUMMARY

Resource International, Inc. (Rii) has completed a structure foundation exploration for the design and construction of the proposed retaining walls 4W5 and 4W6 as part of the FRA-70-12.68 Project 4R. Based on the proposed plan information provided by GPD GROUP, Retaining Wall 4W5 begins at station 176+66.37, 39.84' Lt (BL I-70 EB) and continues south along the forward abutment of the proposed FRA-70-1373R bridge carrying I-70 eastbound over Short Street to Sta. 176+97.14, 60.00' Rt. (BL I-70 EB). The wall continues south along the forward abutment of the proposed FRA-70-1373A bridge carrying Ramp C5 over Short Street between Sta. 5081+23.26, 25.73' Lt. and Sta. 5081+37.65, 24.29' Rt. (BL Ramp C5), and turns east and continues along the south side of Ramp C5 to Sta. 5085+53.99, 24.29' Rt. (BL Ramp C5), where the wall connects to the rear abutment of the proposed FRA-70-1390C bridge structure. **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 bridge structure report, which is presented under a separate cover.**

Retaining Wall 4W6 begins at Sta. 177+05.64, 59.72' Rt. (BL I-70 EB) and extends east along the north side of Ramp C5 to Sta. 180+30.70, 53.22' Rt. (BL I-70 EB). The wall will connect to the forward abutment of the proposed FRA-70-1373A structure at the east end of the wall alignment and to the rear abutment of the proposed FRA-70-1390C bridge carrying Ramp C5 over I-70 eastbound and westbound at the west end of the wall alignment.

Exploration and Findings

Between July 1, 2013 and January 22, 2015, six (6) structural borings, designated as B-020-1-13, B-020-2-13, B-020-3-13, B-020-7-13, B-020-9-15 and B-023-1-13, were drilled to completion depths ranging from 48.1 to 86.0 feet below the existing ground surface. In addition to the borings performed by Rii for the current exploration, three (3) borings, designated as B-021-0-08, B-023-0-08 and B-024-0-08, were drilled to completion depths ranging from 35.0 to 111.5 feet below the existing ground surface by DLZ as part of the FRA-70-8.93 preliminary exploration. In addition to the project borings, two (2) borings, designated as B-001-A-59 and B-003-A-59, were drilled to a completion depth of 51.0 feet each below the existing grade at the respective boring location at the time of the exploration by the Department of Highways as part of the FRA-40-12.89 project.

Boring B-020-1-13 was on the property located at the southwest corner of Short Street and an access drive that extends west of Short Street along the south side of I-70 to the existing railroad tracks and encountered 6.0 inches of topsoil overlying 4.0 inches of brick pavers at the ground surface. Boring B-020-2-13 was drilled through existing pavement at the entry of the access drive and encountered 4.0 inches of asphalt overlying 6.0 inches of aggregate base. Borings B-020-3-13 and B-020-9-13 were drilled through the existing pavement of Short Street and encountered 4.0 inches of asphalt overlaying 4.0 inches of brick pavers in each boring followed by 9.0 and 3.0 inches of aggregate base, respectively, at the ground surface. Boring B-020-7-13 was drilled through the existing sidewalk along the east side of Short Street, below the existing structure and between the curb and pier columns, and encountered 8.0 inches of concrete at the ground surface. Boring B-023-0-08 was performed within the pavement of W. Fulton Street and encountered 4.0 inches of asphalt, overlying 8.0 inches of concrete followed by 6.0 inches of aggregate base at the ground surface. Borings B-021-0-08, B-023-1-13 and B-024-0-08 were drilled on the property located at the southeast corner of W. Fulton Street and Short Street. Boring B-021-0-08 encountered 3.0 inches of gravel at the ground surface and natural soils were encountered at the surface in borings B-023-1-13 and B-024-0-08.

Beneath the surface materials in borings B-020-1-13, B-020-2-13, B-020-3-13, B-020-7-13, B-020-9-15, B-021-0-08 and B-023-0-08, material identified as existing fill was encountered extending to depths ranging from 10.5 to 21.5 feet below existing grade, which corresponds to elevations ranging from 690.9 to 702.5 feet msl. The fill material consisted of dark brown, brown, black and gray gravel and sand, gravel with sand and silt, sandy silt, silt and clay, silty clay and clay (ODOT A-1-b, A-2-4, A-4a, A-6a, A-6b, A-7-6). The fill material was placed within the limits of the abandoned canal and contains construction debris and organics throughout. Additionally, the SPT blow counts were significantly lower and more variable within the fill depth of the borings where fill was encountered.

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

Top of bedrock was encountered in borings B-020-1-13, B-020-2-13, B-020-7-13, B-020-9-13, B-021-0-08 and B-024-0-08 at elevations ranging from 647.1 to 660.9 feet msl. The upper portion of the bedrock consists of weak shale which was able to be augered to competent shale or mudstone bedrock in borings B-020-2-13 and B-021-0-08 and competent limestone bedrock in boring B-020-1-13. The cored bedrock consists of shale and mudstone, which was encountered in borings B-020-2-13, B-020-7-13, B-020-9-13, B-021-0-08 and B-024-0-08 at elevations ranging from 641.9 to 657.9 feet msl, and limestone bedrock, which was encountered in borings B-020-1-13 and B-020-2-13 an elevation of 632.3 and 631.9 feet msl, respectively.

In general, the subsurface conditions encountered in the historic borings matched relatively closely with the subsurface conditions encountered in the current and preliminary engineering exploration borings.

Analyses and Recommendations

Based on the plan information provided, the lightweight cellular concrete modified MSE walls and fill will be placed along the abutments of the FRA-70-1373A and R structures and will extend east to Sta. 5083+25 (BL Ramp C5) and Sta. 178+10 (BL I-70 EB). The cellular concrete fill section will extend the full height from the leveling pad elevation to the profile grade of the roadway. Where the walls and I-70 eastbound embankment span over the influence of the Franklin Main, it is understood that significant undercut of the existing soils will be performed, which will be backfilled with lightweight cellular concrete as part of the Project 2B improvements. Cellular concrete will be placed to the profile grade of Ramp C5, as well as geofoam blocking to the profile grade of I-70 eastbound within the limits of influence, which will result in no net loading. From Sta. 5083+25 to the end of the wall alignments at the read abutment of the FRA-70-1390C bridge structure, traditional MSE walls will be utilized.

Lightweight (Cellular Concrete) Wall Recommendations

Given the presence of existing fill material to significant depths, as well as the significant amount of existing utilities present along the east side of Short Street, it is understood that lightweight fill material consisting of cellular concrete is being considered to be utilized as the backfill along the length of Walls 4W5 and 4W6 from the FRA-70-1373A and R bridge structures to Sta. 5083+25 (BL Ramp C5) and Sta. 178+10 (BL I-70 EB). The use of the lightweight cellular concrete will eliminate the need for undercut or ground improvement to stabilize the underlying existing fill material and control settlement to tolerable limits. Based on information provided by GPD GROUP, two types of lightweight cellular concrete will be utilized in lieu of typical embankment fill and select granular fill, which is typically used for MSE wall applications. The wall facing will be connected to geosynthetic straps that are embedded into the cellular concrete and supported on a leveling pad, similar to traditional MSE walls. It is recommended that the reinforcement extend the minimum length of 70 percent of the wall height into the cellular concrete backfill, similar to traditional MSE walls.

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

Based on the plan information provided, it is understood that the cellular concrete fill will be placed the full height of the embankment and full width of both Ramp C5 and I-70 eastbound. As such, external and global stability calculations will not be required for these sections of Wall 4W5 and 4W6. However, if bearing resistance must be checked, then a factored bearing resistance of 3.8 ksf should be utilized for design at the strength limit state.

Total settlements of 2.61 to 4.28 inches at the center of the wall mass and 1.98 to 3.15 inches at the facing of the wall are anticipated along Wall 4W5 where is crosses in front of the forward abutment of the two structures. Based on the results of the analysis, 90 percent of the total settlement is anticipated to occur over a period of approximately 0 to 35 days.

The beginning of Retaining Wall 4W6, from Sta. 177+05 to 178+40 (BL I-70 EB), will be supported on approximately 25 to 30 feet of cellular concrete backfill. Once the cellular concrete has setup and cured at the bottom of wall elevation (leveling pad), this material will be suitable for support of the proposed wall along this length.

MSE Wall Recommendations

It is understood that a traditional MSE wall type is being utilized for Retaining Wall 4W5 from Sta. 5083+25 (BL Ramp C5) and Retaining Wall 4W6 from Sta. 178+85 (BL I-70 EB) to the end of the wall alignments, where they tie into the rear abutment of the proposed FRA-70-1390C bridge structure. Based on the proposed plan and profile information, the maximum wall height along the alignment of Retaining Wall 4W5 east of Sta. 5081+38 (BL Ramp C5) will range from 30.1 feet at Sta. 5085+54 to 45.3 feet at Sta. 5082+00 (BL Ramp C5), and wall height along the alignment of Retaining Wall 4W6 range from 14.6 feet at Sta. 177+06 to 26.7 feet at Sta. 180+31 (BL I-70 EB), as measured from the top of the coping and the top of the leveling pad. Since the wall is located within an existing floodplain, the analysis was performed using a design groundwater level at the ground surface.

Natural cohesive soils consisting of very stiff to hard sandy silt, silty clay and clay (ODOT A-4a, A-6b, A-7-6) were encountered at the proposed bearing elevation in borings B-003-A-59, B-023-1-13 and B-024-0-08 along the alignment of Wall 4W5 between Sta. 5083+25 to 5085+54 (BL Ramp C5). Retaining Wall 4W6 will be supported on approximately 10.0 to 20.0 feet of new fill from Sta. 178+85 to 180+31 (BL I-70 EB). MSE wall foundations bearing on these natural soils or granular embankment, placed and compacted in accordance with ODOT Item 203, may be proportioned for a nominal bearing resistance as indicated in the following table. A geotechnical resistance factor of φ_b=0.65 was considered in calculating the factored bearing resistance at the strength limit state.

Retaining Wall 4W5 and 4W6 MSE Wall Design Parameters

Structure Reference	From Station / Offset [1]	To Station / Offset [1]	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]	
Retaining Wall 4W5	5083+25, 24.3' Rt.	5085+54, 24.3' Rt.	39.2	27.4 (0.70H)	14.16	9.20	8.91
Retaining Wall 4W6	178+85, 56.0' Rt.	180+30, 53.2' Rt.	26.7	18.7 (0.70H)	15.16	9.85	6.32

1. *Station and offset referenced to the baseline of Ramp C5 for Retaining Wall 4W5, and to the baseline of I-70 EB for Retaining Wall 4W6.*
2. *The required foundation width 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.*

Given that Walls 4W5 and 4W6 will both be constructed to the same profile grade and that Wall 4W6 will be constructed on granular embankment that extends to the same bottom of wall elevation as 4W5, settlement analysis was performed for Wall 4W5 only, and is considered representative of the settlement anticipated along Wall 4W6. Total settlements of 3.56 to 5.44 inches at the center of the reinforced mass and 2.27 to 2.37 inches at the facing of the wall are anticipated along the portion of the alignment of Retaining Walls 4W5 and 4W6 where the standard MSE walls are proposed. Based on the results of the analysis, 90 percent of the total settlement at the facing of the walls is anticipated to occur during construction of the wall or within 11 to 35 days following the completion of construction of the walls.

Based on the results of the external and global stability analysis performed for the MSE walls, the recommended controlling strap length is 0.70 times the height of the MSE walls (measured from the top of the leveling pad to top of the coping) along the alignment of Retaining Wall 4W5 between Sta. 5083+25 and 5085+54 (BL Ramp C5) and Retaining Wall 4W6 between Sta. 178+85 to 180+31 (BL I-70 EB). All of the external and global stability calculations indicate that adequate resistance is available for support of the proposed walls using the configurations and backfill materials outlined in the plan sheets provided by GPD GROUP.

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

1.0 INTRODUCTION

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

This report is a presentation of the structure foundation exploration performed for the design and construction of proposed retaining walls 4W5 and 4W6 as part of the FRA-70-12.68 Project 4R, as shown on the vicinity map and boring plan presented in Appendix I. The retaining walls will provide the required grade separation to support Ramp C5 adjacent to the proposed I-70 eastbound along the north side of the ramp and the existing terrain to the south. Based on the proposed plan information provided by GPD GROUP, Retaining Wall 4W5 begins at station 176+66.37, 39.84’ Lt (BL I-70 EB) and continues south in front of the forward abutment of the proposed FRA-70-1373R bridge carrying I-70 eastbound over Short Street to Sta. 176+97.14, 60.00’ Rt. (BL I-70 EB). The wall continues south in front of the forward abutment of the proposed FRA-70-1373A bridge carrying Ramp C5 over Short Street between Sta. 5081+23.39, 25.73’ Lt. and Sta. 5081+37.65, 24.29’ Rt. (BL Ramp C5), and turns east and continues along the south side of Ramp C5 to Sta. 5085+53.99, 24.29’ Rt. (BL Ramp C5), where the wall connects to the rear abutment of the proposed FRA-70-1390C bridge structure. The total wall length is approximately 580 lineal feet as measured along the facing of the wall. The maximum wall height where the wall crosses in front of the forward abutments of the FRA-70-1373R and FRA-70-1373A structures is 37.1 and 44.1 feet, respectively, and wall heights along the remainder of the wall alignment range from 30.1 feet at Sta. 5085+54 to 45.3 feet at Sta. 5082+00 (BL Ramp C5). **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 bridge structure report, which is presented under a separate cover.**

Retaining Wall 4W6 begins at Sta. 177+05.64, 59.72’ Rt. (BL I-70 EB) and extends east along the north side of Ramp C5 to Sta. 180+30.70, 53.22’ Rt. (BL I-70 EB). The wall will connect to the forward abutment of the proposed FRA-70-1373A structure at the west end of the wall alignment and to the rear abutment of the proposed FRA-70-1390C structure at the east end of the wall alignment. The wall height along the wall alignment ranges from 14.6 feet at Sta. 177+06 to 26.7 feet at Sta. 180+31 (BL I-70 EB), and the total wall length is approximately 325 lineal feet as measured along the facing of the wall. Due to the grade difference of the proposed I-70 eastbound from the existing grade, Retaining Wall 4W6 will be supported on new fill for the entire alignment.

Several wall type alternatives were considered during the Stage 1 design for these structures, including traditional mechanically stabilized earth (MSE) walls and walls constructed of lighter weight fill material such as geofoam and cellular concrete. Based on the evaluation performed as part of the Stage 1 design, it was determined that ground improvement would be required for a portion of the alignment if traditional MSE walls were utilized due to the presence of existing weak, highly variable fill soils encountered in several of the borings. Based on coordination with the Ohio Department of Transportation (ODOT) Office of Geotechnical Engineering (OGE), it was elected to utilize MSE wall types with lightweight cellular concrete backfill in lieu of typical soil backfill materials to reduce the bearing stress and settlement within the weak fill soils, where encountered along the wall alignments.

Additionally, it is understood that no net loading is permitted to be applied over the Franklin Main, which has been previously lined and subsequently loaded to the maximum permissible overburden pressure. Therefore, the lightweight cellular concrete modified MSE walls will be utilized to span Ramp C5 over the existing Franklin Main, where additional undercut of the existing soil and backfill with the lightweight cellular concrete will be provided to reduce the net loading on the existing 60-inch brick sewer. Where I-70 spans over the Franklin Main, geofoam blocking in conjunction with undercut and of the existing soil and backfill with lightweight cellular concrete will be utilized. On the east side of the Franklin Main, traditional MSE wall types with typical soil backfill be utilized up to the proposed FRA-70-1390C structure.

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 60.5 to 91.8 feet below the ground surface, which corresponds to elevations ranging from 647.1 feet to 660.9 feet msl.

2.2 Existing Conditions

The proposed retaining wall 4W5 and 4W6 structures will be situated along the south side of I-70 eastbound, east of Short Street, and will generally follow the existing alignment of W. Fulton Street to just west of 2nd Street. The existing Short Street and W. Fulton Avenue in the vicinity of the structures are two-lane, asphalt paved roadways that are aligned north-to-south and east-to-west, respectively. The existing I-70 eastbound in the vicinity of the structures is a three-lane, asphalt paved roadway that is aligned east-to-west. The existing I-70 roadway profile grade is elevated approximately 26 feet above the Short Street profile grade, and is supported by an existing cast-in-place (CIP) retaining wall along the south side of the alignment between the highway and W. Fulton Street. The terrain along I-70 slopes down gently to the east and along W. Fulton Street slopes up moderately to the east from Short Street, and the surrounding area in the vicinity of the intersection of Short Street and W. Fulton Street is relatively flat-lying.

Based on utility plans provided by GPD GROUP, there are many buried utilities within the Short Street and W. Fulton Street roadways and also beneath the surrounding sidewalks, including the Franklin Main, which is a 60-inch brick sewer that crosses I-70 at approximately Sta. 178+30 (BL I-70 EB). It is understood that this line was previously lined when I-70 was originally constructed and that additional lining cannot be added as the current capacity of the line cannot be reduced further.

Additionally, based on information provided by ODOT and GPD GROUP, it is understood that a canal was formerly located in the area of the Short Street and W. Fulton Street intersection, which was abandoned and filled in prior to construction of the original US 40 or I-70 roadways.

3.0 EXPLORATION

Between July 1, 2013 and January 22, 2015, six (6) structural borings, designated as B-020-1-13, B-020-2-13, B-020-3-13, B-020-7-13, B-020-9-15 and B-023-1-13, were drilled to completion depths ranging from 48.1 to 86.0 feet below the existing ground surface. In addition to the borings performed by Rii for the current exploration, three (3) borings, designated as B-021-0-08, B-023-0-08 and B-024-0-08, were performed by DLZ as part of the FRA-70-8.93 preliminary exploration and their findings were published in a report dated January 2010 and March 2010. The borings were performed between July 1 and September 2, 2008, and were advanced to completion depths ranging from 35.0 to 111.5 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	Reference Alignment	Station	Offset	Latitude	Longitude	Ground Elevation (feet msl)	Boring Depth (feet)
B-020-1-13	BL Ramp C5	5080+09.80	30.9' Rt.	39.952922218	-83.004665587	712.8	86.0
B-020-2-13	BL I-70 EB	176+13.92	34.0' Rt.	39.953155708	-83.004534664	711.4	84.5
B-020-3-13	BL Ramp C5	5081+15.55	85.3' Rt.	39.952760425	-83.004309070	712.3	49.8
B-020-7-13	BL I-70 WB	176+68.64	1.8' Rt.	39.953451540	-83.004376859	713.5	80.4
B-020-9-15	BL Ramp C5	5081+05.25	39.8' Rt.	39.952886963	-83.004333117	713.0	75.5
B-021-0-08	BL Ramp C5	5082+48.43	39.8' Rt.	39.952847911	-83.003831990	727.9	90.0
B-023-0-08	BL I-70 EB	179+52.56	1.9' Lt.	39.953151856	-83.003323535	722.0	35.0
B-023-1-13	BL Ramp C5	5084+74.16	15.0' Rt.	39.952844807	-83.003019835	732.4	48.1
B-024-0-08	BL Ramp C5	5085+90.21	3.1' Lt.	39.952928381	-83.002605586	743.4	111.5

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 were drilled using either 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 or 4.0-inch flush joint casing to advance the holes. In general, standard penetration test (SPT) and split spoon sampling were performed in the boring at 2.5-foot increments of depth to 20 to 30 feet and at 5.0-foot increments thereafter to the boring termination depth or top of bedrock. The SPT, per the American Society for Testing and Materials (ASTM) designation D1586, is conducted using a 140-pound hammer falling 30.0 inches to drive a 2.0-inch outside diameter split spoon sampler 18.0 inches. A calibrated automatic drop hammer was utilized to generate consistent energy transfer to the sampler. Driving resistance is recorded on the boring logs in terms of blows per 6.0-inch interval of the driving distance. The second and third intervals are added to obtain the number of blows per foot (N). Standard penetration blow counts aid in determining soil properties applicable in foundation system design. Measured blow count (N) values are corrected to an equivalent (60%) energy ratio, N_{60}, by the following equation. Both values are represented on boring logs in Appendix III.

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

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

The hammers for the Mobile B-53 and CME 750 drill rigs operated by Rii were calibrated on April 26, 2013, and have drill rod energy ratios of 77.7 and 82.6 percent, respectively. The hammer for the CME 55 drill rig operated by Rii was calibrated on October 20, 2014, and has a drill rod energy ratio of 92.0 percent. The hammers for the CME 750X and CME 55-LC drill rigs operated by Stock Drilling were calibrated on March 28, 2013, and have a drill rod energy ratios of 78.6 and 73.2 percent, respectively. The hammers for the CME 75 and CME 750X drill rigs used by DLZ have a drill rod energy ratios of 61.2 and 63.1 percent, respectively. No calibration date is available for the DLZ rig calibrations.

During drilling for the borings performed by Rii and Stock Drilling, 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. Rii Laboratory Test Schedule

Laboratory Test	Test Designation	Number of Tests Performed
Natural Moisture Content	ASTM D 2216	103
Plastic and Liquid Limits	AASHTO T89, T90	38
Gradation – Sieve/Hydrometer	AASHTO T88	38
Unconfined Compressive Strength of Cohesive Soil	ASTM D2166	1
One-Dimensional Consolidation	ASTM D2435	3
Consolidated Undrained (CU) Triaxial Test	ASTM D4767	2
Point Load Strength Index of Rock Specimens	ASTM D5731	1
Unconfined Compressive Strength of Intact Rock	ASTM D7012	1

The tests performed are necessary to classify existing soil according to the 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 and also in Appendix V. 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 was determined by split spoon sampler or auger refusal, or by visual inspection of the very weak to weak shale and mudstone samples in conjunction with the blow counts obtained from the SPT testing. 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. Auger refusal is defined as no or insignificant observable advancement of the augers with the weight of the drill rig driving the augers. Where borings were extended into the bedrock, an NQ or HQ-sized double-tube diamond bit core barrel (utilizing wire line equipment) was used to core the bedrock. Coring produced 1.8 or 2.5 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:

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

In addition to the project borings, historic borings performed in 1959 by the Department of Highways as part of the FRA-40-12.89 project were also obtained from the construction documents on record. Two (2) borings, designated as B-001-A-59 and B-003-A-59, were obtained along the alignment of the existing CIP wall separating I-70 eastbound and W. Fulton Street. The borings were extended to a depth of 51.0 feet each below the existing grade at the respective boring location at the time of the exploration. Please note that the elevations provided on the historic boring logs are referenced to the North American Datum (NAD) 27. The current design survey is referenced to NAD 83. The NAD 27 datum is 0.6 feet lower than the NAD 83 datum. **Therefore, all elevations noted in this report with respect to the historic borings are adjusted to the current NAD 83 datum.** The historic boring locations are shown on the boring plan provided in Appendix I, and the historic boring logs are provided in Appendix IV.

4.0 FINDINGS

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

4.1 Surface Materials

Boring B-020-1-13 was on the property located at the southwest corner of Short Street and an access drive that extends west of Short Street along the south side of I-70 to the existing railroad tracks and encountered 6.0 inches of topsoil overlying 4.0 inches of brick pavers at the ground surface. Boring B-020-2-13 was drilled through existing pavement at the entry of the access drive and encountered 4.0 inches of asphalt overlying 6.0 inches of aggregate base. Borings B-020-3-13 and B-020-9-13 were drilled through the existing pavement of Short Street and encountered 4.0 inches of asphalt overlaying 4.0 inches of brick pavers in each boring followed by 9.0 and 3.0 inches of aggregate base, respectively, at the ground surface. Boring B-020-7-13 was drilled through the existing sidewalk along the east side of Short Street, below the existing structure and between the curb and pier columns, and encountered 8.0 inches of concrete at the ground surface. Boring B-023-0-08 was performed within the pavement of W. Fulton Street and encountered 4.0 inches of asphalt, overlying 8.0 inches of concrete followed by 6.0 inches of aggregate base at the ground surface. Borings B-021-0-08, B-023-1-13 and B-024-0-08 were drilled on the property located at the southeast corner of W. Fulton Street and Short Street. Boring B-021-0-08 encountered 3.0 inches of gravel at the ground surface and natural soils were encountered at the surface in borings B-023-1-13 and B-024-0-08.

4.2 Subsurface Soils

Beneath the surface materials in borings B-020-1-13, B-020-2-13, B-020-3-13, B-020-7-13, B-020-9-15, B-021-0-08 and B-023-0-08, material identified as existing fill was encountered extending to depths ranging from 10.5 to 21.5 feet below existing grade, which corresponds to elevations ranging from 690.9 to 702.5 feet msl. The fill material consisted of dark brown, brown, black and gray gravel and sand, gravel with sand and silt, sandy silt, silt and clay, silty clay and clay (ODOT A-1-b, A-2-4, A-4a, A-6a, A-6b, A-7-6). The fill material was placed within the limits of the abandoned canal and contains construction debris and organics throughout. Additionally, the SPT blow counts were significantly lower and more variable within the fill depth of the borings where fill was encountered.

Underlying the surficial materials and existing fill, natural granular and cohesive soils were encountered. The granular soils were generally described as brown, gray, brownish gray, dark brown and black gravel, gravel and sand, gravel with sand and silt, gravel with sand, silt and clay, fine sand, coarse and fine sand, sandy silt and silt (ODOT A-1-a, A-1-b, A-2-4, A-2-6, A-3, A-3a, A-4a, A-4b). The cohesive soils were described as gray, brown, brownish gray and dark gray sandy silt, silt and clay, silty clay and clay (ODOT A-4a, A-6a, A-6b, A-7-6). Granite boulders were encountered in boring B-020-3-13 at an elevation of 686.3 feet msl and again at an elevation of 662.5 feet msl. Auger refusal was encountered at these elevations, and rock coring was performed for a 5.0-foot interval at both instances, which small boulder pieces were recovered from the core runs. Granite boulders were also encountered in boring B-020-7-13 at an elevation of 657.0 feet msl, and rock coring was performed below this elevation. Based on the lack of recovery and observation of the soil washout in the circulation fluid in core runs RC-1 through RC-3, it is anticipated that this material is a hard cohesive soil rather than highly weathered bedrock.

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} < 6 blows per foot [bpf]) to very dense (N_{60} > 50 bpf). Overall blow counts recorded from the SPT sampling ranged from 2 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 soils encountered ranged from very soft (HP ≤ 0.25 tsf) to hard (HP > 4.0 tsf). The unconfined compressive strength of the cohesive soil samples tested, obtained from the hand penetrometer, ranged from less than 0.25 to over 4.5 tsf (limit of instrument).

Natural moisture contents of the soil samples tested ranged from 4 to 37 percent. The natural moisture content of the cohesive soil samples tested for plasticity index ranged from 7 percent below to 7 percent above their corresponding plastic limits. In general, the soils exhibited natural moisture contents considered to be moderately 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		Top of Bedrock Core	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-020-1-13	712.8	64.2	648.6	80.5	632.3
B-020-2-13	711.4	64.3	647.1	69.5	641.9
B-020-3-13	712.3	N/A	N/A	N/A	N/A
B-020-7-13	713.5	65.4	648.1	65.4	648.1
B-020-9-15	713.0	60.5	652.5	60.5	652.5
B-021-0-08	727.9	67.0	660.9	70.0	657.9
B-023-0-08	722.0	N/A	N/A	N/A	N/A
B-023-1-13	732.4	N/A	N/A	N/A	N/A
B-024-0-08	743.4	91.8	651.6	91.5	651.9

Top of bedrock was encountered in borings B-020-1-13, B-020-2-13, B-020-7-13, B-020-9-13, B-021-0-08 and B-024-0-08 at elevations ranging from 647.1 to 660.9 feet msl. The upper portion of the bedrock consists of weak shale which was able to be augered to competent shale or mudstone bedrock in borings B-020-2-13 and B-021-0-08 and competent limestone bedrock in boring B-020-1-13. The cored bedrock consists of shale and mudstone, which was encountered in borings B-020-2-13, B-020-7-13, B-020-9-13, B-021-0-08 and B-024-0-08 at elevations ranging from 641.9 to 657.9 feet msl, and limestone bedrock, which was encountered in borings B-020-1-13 and B-020-2-13 an elevation of 632.3 and 631.9 feet msl, respectively.

The mudstone is described as gray, highly weathered, very weak, thinly laminated to thin bedded, arenaceous, calcareous, fissile, friable and slightly to highly fractured with tight, slightly rough apertures. The shale is described as dark gray, bluish gray and black, slightly to highly weathered, very weak to weak, thinly laminated to medium bedded, arenaceous, calcareous, fissile, friable, pyritic, jointed and moderately to highly fractured with tight to open, slightly rough to very rough apertures. The limestone is described as gray and tan, unweathered to slightly weathered, moderately strong to strong, very thin to medium bedded, calcareous, crystalline, dolomitic, pyritic and slightly fractured to fractured with narrow to open, slightly rough to very rough apertures.

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-020-1-13	RC-2	632.3 to 626.8	99	49	q_u @ 80.7' = 9,465 psi
B-020-2-13	RC-1	641.9 to 636.9	40	8	N/A
	RC-2	636.9 to 631.9	20	0	N/A
	RC-3	631.9 to 626.9	97	97	N/A
B-020-7-13	RC-4	648.1 to 638.1	97	89	q_u @ 69.4' = 224 psi [1]
	RC-5	638.1 to 633.1	100	45	N/A
B-020-9-13	RC-1	652.5 to 647.5	52	17	N/A
	RC-2	647.5 to 642.5	75	48	N/A
	RC-3	642.5 to 637.5	100	80	N/A
B-021-0-08	R1	657.9 to 652.9	100	82	N/A
	R2	652.9 to 647.9	100	90	N/A
	R3	647.9 to 642.9	100	75	N/A
	R4	642.9 to 637.9	100	87	q_u @ 82.8' = 1,536 psi
B-024-0-08	R2	651.9 to 641.9	97	81	q_u @ 97.7' = 1,650 psi
	R3	641.9 to 631.9	63	48	N/A

1. *Represents the mean unconfined compressive strength based on correlations with the mean point load strength index.*

It should be noted that bedrock experiences mechanical breaks during the drilling and coring processes. Rii attempted to account for fresh, manmade breaks during tabulation of the RQD analysis. The zones 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 cored mudstone and shale bedrock, according to the RQD values of the bedrock units, ranged from very poor (RQD ≤ 25%) to good (75% < RQD ≤ 90%), and the quality of the cored limestone bedrock ranged from very fair (50% < RQD ≤ 75%) to excellent (RQD > 90%).

4.4 Groundwater

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

Table 5. Groundwater Levels

Boring Number	Ground Surface Elevation (feet msl)	Initial Groundwater		Upon Completion	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-020-1-13	712.8	23.0	689.8	N/A [1]	N/A
B-020-2-13	711.4	18.5	692.9	N/A [1]	N/A
B-020-3-13	712.3	15.0	697.3	N/A [1]	N/A
B-020-7-13	713.5	N/A [2]	N/A	N/A [1]	N/A
B-020-9-15	713.0	24.5	688.5	N/A [1]	N/A
B-021-0-08	727.9	28.0	699.9	26.9	701.0
B-023-0-08	722.0	25.0	697.0	19.1	702.9
B-023-1-13	732.4	32.0	700.4	18.0	714.4
B-024-0-08	743.4	28.5	714.9	16.5	726.9

1. *Groundwater was not encountered in boring B-020-7-13 prior to introducing water to the borehole.*
2. *The groundwater level at completion could not be obtained due to the addition of water or mud as a drilling fluid.*

Groundwater was encountered initially during drilling in all of the borings, with the exception of boring B-020-7-13, at depths ranging from 15.0 to 32.0 feet below the existing ground surface, which corresponds to elevations ranging from 689.8 to 714.9 feet msl. Groundwater was not encountered in boring B-020-7-13 prior to introducing water to the borehole. At the completion of drilling and prior to beginning rock coring operations in borings B-021-0-08, B-023-0-08, B-023-1-13 and B-024-0-08, groundwater accumulated in the auger stems to depths ranging from 16.5 to 26.9 feet below the ground surface, which corresponds to elevations ranging from 701.0 to 726.9 feet msl. The groundwater levels at the completion of drilling could not be measured in the remainder of the borings due to the addition of 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.

4.5 Historic Borings

Historic boring B-001-A-59 encountered existing fill consisting of loose to medium dense, gray, dark gray and brown gravel and sand, gravel with sand and silt and sandy silt (ODOT A-1-b, A-2-4, A-4a) extending to a depth of 17.0 feet below existing grade at the time the boring was performed, which corresponds to an elevation of 696.9 feet msl. In general, the natural soils encountered below the fill in boring B-001-A-59 and from the ground surface in boring B-003-A-59 consisted of medium dense to very dense granular soils with intermittent seams of very stiff to hard cohesive soils. The granular soils were generally described as brown and gray gravel, gravel and sand, gravel with sand and silt, coarse and fine sand, sandy silt and silt (ODOT A-1-a, A-1-b, A-2-4, A-3a, A-4a, A-4b), and the cohesive soils were generally described as gray sandy silt, silt and silt and clay (ODOT A-4a, A-4b, A-6a). Boulders were noted throughout the natural granular soil deposits encountered below the fill in boring B-001-A-59 beginning at elevation 696.9 feet msl. Boulders were not noted on the log for boring B-003-A-59. Bedrock was not encountered in the historic borings prior to the termination depths. Groundwater levels were not noted in the borings performed during the 1959 exploration. In general, the subsurface conditions encountered in the historic borings matched relatively closely with the subsurface conditions encountered in the current and preliminary engineering exploration borings.

5.0 ANALYSES AND RECOMMENDATIONS

Data obtained from the current and preliminary exploration programs have been used to determine the foundation support capabilities and the settlement potential for the soil encountered at the site. These parameters have been used to provide guidelines for the design of foundation systems for the subject structure, 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 walls were provided by GPD GROUP. As noted in Section 1.0, traditional MSE walls were evaluated as part of the Stage 1 design and it was determined that ground improvement would be required to support this wall type due to the height of the walls and presence of existing weak, highly variable fill soils encountered in several of the borings. The existing fill is primarily located along the western half of the wall, west of the Franklin Main, where the former canal was abandoned in the vicinity of Short Street and W. Fulton Street. Based on coordination with the Ohio Department of Transportation (ODOT) Office of Geotechnical Engineering (OGE), it was elected to utilize MSE wall types with lightweight cellular concrete backfill in lieu of typical soil backfill materials to reduce the bearing stress and settlement within the weak fill soils, where encountered along the wall alignments.

Additionally, it is understood that no net loading is permitted to be applied over the Franklin Main, which has been previously lined and subsequently loaded to the maximum permissible overburden pressure. Therefore, the lightweight cellular concrete modified MSE walls will be utilized to span Ramp C5 over the existing Franklin Main, where additional undercut of the existing soil and backfill with the lightweight cellular concrete will be provided to reduce the net loading on the existing 60-inch brick sewer. Where I-70 spans over the Franklin Main, geofoam blocking in conjunction with undercut and of the existing soil and backfill with lightweight cellular concrete will be utilized. On the east side of the Franklin Main, traditional MSE wall types with typical soil backfill be utilized up to the proposed FRA-70-1390C structure.

Based on the plan information provided, the lightweight cellular concrete modified MSE walls and fill will be placed along the abutments of the FRA-70-1373A and R structures and will extend east to Sta. 5083+25 (BL Ramp C5) and Sta. 178+10 (BL I-70 EB). The cellular concrete fill section will extend the full height from the leveling pad elevation to the profile grade of the roadway. Where the walls and I-70 eastbound embankment span over the influence of the Franklin Main, it is understood that significant undercut of the existing soils will be performed, which will be backfilled with lightweight cellular concrete as part of the Project 2B improvements. Cellular concrete will be placed to the profile grade of Ramp C5, as well as geofoam blocking to the profile grade of I-70 eastbound within the limits of influence, which will result in no net loading. From Sta. 5083+25 to the end of the wall alignments at the read abutment of the FRA-70-1390C bridge structure, traditional MSE walls will be utilized.

5.1 Lightweight (Cellular Concrete) Wall Recommendations

Given the presence of existing fill material to significant depths, as well as the significant amount of existing utilities present along the east side of Short Street, it is understood that lightweight fill material consisting of cellular concrete is being considered to be utilized as the backfill along the length of Walls 4W5 and 4W6 from the FRA-70-1373A and R bridge structures to Sta. 5083+25 (BL Ramp C5) and Sta. 178+10 (BL I-70 EB). The use of the lightweight cellular concrete will eliminate the need for undercut or ground improvement to stabilize the underlying existing fill material and control settlement to tolerable limits. Based on information provided by GPD GROUP, two types of lightweight cellular concrete will be utilized in lieu of typical embankment fill and select granular fill, which is typically used for MSE wall applications. The wall facing will be connected to geosynthetic straps that are embedded into the cellular concrete and supported on a leveling pad, similar to traditional MSE walls.

A typical section of the proposed cellular concrete wall system was provided by GPD GROUP. Based on the information provided, the typical section will consist of an approximate 3.0-foot thick pavement section, including asphalt and/or concrete and aggregate base, overlying 2.0 feet of Class III cellular concrete, followed by Class II cellular concrete to the bottom of the embankment/wall elevation. A composite unit weight of 130 pcf was considered for the entire pavement section, and the unit weight of the Class III cellular concrete is 36 pcf and the Class II cellular concrete is 30 pcf. The pressure at the bottom of the embankment was calculated as follows:

$$\Delta\sigma = (130 \text{ pcf})(3.0 \text{ ft}) + (36 \text{ pcf})(2.0 \text{ ft}) + (H - 5 \text{ ft})(30 \text{ pcf})$$

Where,

$\Delta\sigma$ = induced pressure at the bottom of embankment/wall (psf)

H = height of embankment/wall from existing ground surface to profile grade of roadway (ft)

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

Following placement of the cellular concrete, the material will cure and harden similar to concrete and will become a rigid mass. The concept of active earth pressure within this mass is not valid, as it cannot substantially deform, develop an active wedge, and mobilize active earth pressure. Therefore, the entire cellular concrete mass must be treated as a solid block. The “reinforced zone” is not the same as a traditional MSE wall reinforced zone, as the reinforcement straps only need to extend back into the cellular mass far enough to fully develop resistance in tension as if it were a reinforcing bar embedded in reinforced concrete. However, it is recommended that the reinforcement extend the minimum length of 70 percent of the wall height into the cellular concrete backfill, similar to traditional MSE walls.

Considering the above commentary in regards to the external stability of the cellular concrete backfilled MSE walls, sliding, overturning, bearing and overall (global) stability of the wall must be performed for the entire mass as a single block. Therefore, consideration must be given to the effect of the backfill material behind the cellular concrete if it is only utilized within the reinforced zone of the wall.

The active earth pressure coefficient, and consequently the active pressure on the back of the cellular concrete mass, will greatly reduce as the slope of the backfill soil flattens. Once the slope of the backfill flattens more than the internal friction angle of the backfill soil, the active earth pressure coefficient will go to zero. Therefore, if the backslope of any backfill is reduced to the internal friction angle of the backfill material, analysis of external stability is not required, with the exception of bearing and overall (global) stability. Based on the plan information provided, it is understood that the cellular concrete fill will be placed the full height of the embankment and full width of both Ramp C5 and I-70 eastbound. As such, external and global stability calculations will not be required for these sections of Wall 4W5 and 4W6. However, if bearing resistance must be checked, then a factored bearing resistance of 3.8 ksf should be utilized for design at the strength limit state.

The compressibility parameters utilized in the settlement analysis of the proposed cellular concrete backfilled areas are provided in Table 10.

Table 6. Compressibility Parameters Utilized in Settlement Analysis

Material Type	γ (pcf)	LL (%)	C_c (1)	C_r (2)	e_o (3)	C_v (4) (ft²/yr)	N_{60}	C' (5)
Existing Fill: Very Soft to Soft Sandy Silt (ODOT A-4a)	115	28	0.162	0.024	0.491	800	N/A	N/A
Existing Fill: Very Soft to Stiff Silt and Clay (ODOT A-6a)	115 to 120	29 to 35	0.171 to 0.225	0.026 to 0.034	0.499 to 0.546	600	N/A	N/A
Existing Fill: Medium Stiff to Stiff Silty Clay (ODOT A-6b)	115	34 to 40	0.216 to 0.270	0.032 to 0.041	0.538 to 0.585	300	N/A	N/A
Existing Fill: Soft Clay (ODOT A-7-6)	115	41	0.279	0.042	0.593	150	N/A	N/A
Existing Fill: Loose Sandy Silt (ODOT A-4a)	115	N/A	N/A	N/A	N/A	N/A	9	29 to 32
Existing Fill: Very Loose to Loose Granular Soils (ODOT A-1-b, A-2-4)	120 to 125	N/A	N/A	N/A	N/A	N/A	4 to 21	51 to 136
Medium Dense to Very Dense Granular Soils (ODOT A-1-a, A-1-b, A-2-4, A-2-6, A-3, A-4a)	125 to 135	N/A	N/A	N/A	N/A	N/A	17 to 120	73 to 560
Very Stiff to Hard Sandy Silt (ODOT A-4a)	120 to 130	22 to 27	0.108 to 0.153	0.011 to 0.015	0.444 to 0.483	800	N/A	N/A
Hard Silty Clay (ODOT A-6b)	130	37 to 38	0.243 to 0.252	0.024 to 0.025	0.561 to 0.569	300	N/A	N/A
Hard Clay (ODOT A-7-6)	130	42 to 44	0.288 to 0.306	0.029 to 0.023	0.600 to 0.616	150	N/A	N/A

1. *Per Table 6-9, Section 6.14.1 of FHWA GEC 5.*
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 11. Total settlements of 2.61 to 4.28 inches at the center of the wall mass and 1.98 to 3.15 inches at the facing of the wall are anticipated along Wall 4W5 where is crosses in front of the forward abutment of the two structures. Based on the results of the analysis, 90 percent of the total settlement is anticipated to occur over a period of approximately 0 to 35 days. Please note that the consolidation settlement and time rate of consolidation are based on estimates using correlated compressibility parameters provided in Table 10 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 7. Retaining Wall 4W5 and 4W6 Settlement Results

Boring	Wall / Embankment Height (feet)	Pressure at Bottom of Wall / Embankment [1] (ksf)	Total Settlement Values (inches)		Time for 90% Consolidation (Days)
			Center of Wall Mass	Facing of Wall	
B-001-A-59	45.3	1,731	3.88	2.80	0
B-020-1-13	45.3	1,731	3.40	2.46	18
B-020-2-13	45.3	1,731	4.28	3.15	35
B-020-9-15	45.3	1,731	3.19	2.37	10
B-021-0-08	43.3	1,671	2.61	1.98	7
B-023-0-08	41.2	1,608	2.72	2.00	11

1. $\Delta\sigma$ = (130 pcf)(3.0 ft) + (36 pcf)(2.0 ft) + (H - 5 ft)(30 pcf).

Per Section 204.6.2.1 of the ODOT BDM, for traditional MSE walls “the maximum allowable differential settlement in the longitudinal direction (regardless of the size of panels) is one (1) percent.” Based on the total anticipated settlement at the facing of the walls, maximum differential settlements in the longitudinal directions are anticipated to be less than 1/1000, which is within the tolerable limit of 1/100. If localized bearing pressures exerted on the leveling pad from the wall facing panels will be higher than the pressure exerted by the wall mass, then there is a potential for differential settlement to occur given the variability in the fill material.

Results of the settlement analysis for the cellular concrete fil areas are provided in Appendix VI.

For the portions of Walls 4W5 and 4W6 that span over the influence area of the Franklin Main (between Sta. 5082+80 and 5083+25, BL Ramp C5), it is understood that approximately 17 feet of over excavation will be performed (to El. 708 feet msl) and backfilled with Class II cellular concrete to the proposed top of leveling pad elevation of 718.5 feet msl. Based on the depth of over excavation noted and considering a unit weight for the soil removed of 120 pcf, the reduction in overburden over the sewer main will be approximately 2,040 psf. The proposed profile grade where the alignment crosses the sewer main is approximately 759 feet msl, which results in a pressure increase of approximately 1,900 psf. Therefore, there is a slight net unload with respect to the proposed undercut and backfill with cellular concrete in the influence zone of the sewer main.

For the portion of I-70 eastbound that spans over the influence area of the Franklin Main (between Sta. 178+10 and 178+85), it is understood that approximately 13 feet of over excavation will be performed (to El. 702.5 feet msl) and backfilled with Class II cellular concrete to an elevation of 720 feet msl. It is understood that geofoam blocking (ASTM D6817, Type 19) will be utilized above this elevation. Based on the depth of over excavation noted and considering a unit weight for the soil removed of 120 pcf, the reduction in overburden over the sewer main will be approximately 1,560 psf. The proposed profile grade where the alignment crosses the sewer main is approximately 744 feet msl, which results in a pressure increase of approximately 1,200 psf considering a unit weight of 1.5 pcf for the geofoam blocking. Therefore, there is a net unload with respect to the proposed undercut and backfill with cellular concrete and geofoam blocking in the influence zone of the sewer main.

The beginning of Retaining Wall 4W6, from Sta. 177+05 to 178+40 (BL I-70 EB), will be supported on approximately 25 to 30 feet of cellular concrete backfill. Once the cellular concrete has setup and cured at the bottom of wall elevation (leveling pad), this material will be suitable for support of the proposed wall along this length.

5.2 MSE Wall Recommendations

It is understood that a traditional MSE wall type is being utilized for Retaining Wall 4W5 from Sta. 5083+25 (BL Ramp C5) and Retaining Wall 4W6 from Sta. 178+85 (BL I-70 EB) to the end of the wall alignments, where they tie into the rear abutment of the proposed FRA-70-1390C bridge structure. 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 Bridge Design Manual (BDM), where the wall alignment does not cross in from of an abutment the height of the MSE wall is defined as the elevation difference between the top of the 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 on the proposed plan and profile information, the maximum wall height along the alignment of Retaining Wall 4W5 east of Sta. 5081+38 (BL Ramp C5) will range from 30.1 feet at Sta. 5085+54 to 45.3 feet at Sta. 5082+00 (BL Ramp C5), and wall height along the alignment of Retaining Wall 4W6 range from 14.6 feet at Sta. 177+06 to 26.7 feet at Sta. 180+31 (BL I-70 EB), as measured from the top of the coping and the top of the leveling pad. For the analysis, the foundation width was set at 70 percent of the maximum wall height and the foundation width was increased, if required, until external and global stability requirements were satisfied.

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. Natural cohesive soils consisting of very stiff to hard sandy silt, silty clay and clay (ODOT A-4a, A-6b, A-7-6) were encountered at the proposed bearing elevation in borings B-003-A-59, B-023-1-13 and B-024-0-08 along the alignment of Wall 4W5 between Sta. 5083+25 to 5085+54 (BL Ramp C5). Based on the plan and profile information provided, Retaining Wall 4W6 will be supported on approximately 10.0 to 20.0 feet of new fill from Sta. 178+85 to 180+31 (BL I-70 EB). Based on plan information provided, the fill material that Wall 4W6 will be bearing on along the portion of the wall alignment will consist of ODOT Item 203 granular embankment. Provided the granular embankment is placed and compacted in accordance with applicable specifications, then the new fill should be adequate for support of the proposed wall.

5.2.1 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 8.

Table 8. Shear Strength Parameters Utilized in MSE Wall Stability Analyses

Material Type	γ (pcf)	φ' (1) (°)	c' (2) (psf)	S_u (3) (psf)
MSE Select Granular Backfill	120	34	0	N/A
Item 203 Granular Embankment	130	33	0	N/A
Very Stiff to Hard Natural Cohesive Soils (ODOT A-4a, A-6a, A-6b, A-7-6)	120 to 130	26 to 27	0	2,950 to 4,000
Dense to Very Dense Natural Granular Soils (ODOT A-1-a, A-1-b)	130 to 135	34 to 41	0	N/A

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 select granular 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 8. Based on the design plans provided by GPD Group, 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.

5.2.2 Bearing Stability

Natural cohesive soils consisting of very stiff to hard sandy silt, silty clay and clay (ODOT A-4a, A-6b, A-7-6) were encountered at the proposed bearing elevation in borings B-003-A-59, B-023-1-13 and B-024-0-08 along the alignment of Wall 4W5 between Sta. 5083+25 to 5085+54 (BL Ramp C5). Retaining Wall 4W6 will be supported on approximately 10.0 to 20.0 feet of new fill from Sta. 178+85 to 180+31 (BL I-70 EB). MSE wall foundations bearing on these natural soils or granular embankment, placed and compacted in accordance with ODOT Item 203, may be proportioned for a nominal bearing resistance as indicated in Table 9. A geotechnical resistance factor of φ_b=0.65 was considered in calculating the factored bearing resistance at the strength limit state. The reinforcement lengths presented in the following table represent the minimum foundation widths required to satisfy external and global stability requirements, expressed as a percentage of the wall height.

Table 9. Retaining Wall 4W5 and 4W6 MSE Wall Design Parameters

Structure Reference	From Station / Offset [1]	To Station / Offset [1]	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]	
Retaining Wall 4W5	5083+25, 24.3' Rt.	5085+54, 24.3' Rt.	39.2	27.4 (0.70H)	14.16	9.20	8.91
Retaining Wall 4W6	178+85, 56.0' Rt.	180+30, 53.2' Rt.	26.7	18.7 (0.70H)	15.16	9.85	6.32

1. *Station and offset referenced to the baseline of Ramp C5 for Retaining Wall 4W5, and to the baseline of I-70 EB for Retaining Wall 4W6.*
2. *The required foundation width 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 heights indicated in Table 9. Based on the minimum length of reinforcement presented, the factored equivalent bearing pressure exerted below the wall **will not exceed** the factored bearing resistance at the strength limit state.

5.2.3 Settlement Evaluation

The compressibility parameters utilized in the settlement analyses of the proposed MSE wall are provided in Table 10.

Table 10. 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)
Stiff to Hard Natural Cohesive Soils (ODOT A-4a, A-6b, A-7-6)	120 to 130	21 to 48	0.099 to 0.342	0.010 to 0.034	0.436 to 0.647	150 to 800	N/A	N/A
Medium Dense to Very Dense Natural Granular Soils (ODOT A-1-a, A-1-b, A-3a, A-4a)	130 to 135	N/A	N/A	N/A	N/A	N/A	28 to 100	49 to 300

1. *Per Table 6-9, Section 6.14.1 of FHWA GEC 5.*
2. *Estimated at 10% of C_c 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 11. Given that Walls 4W5 and 4W6 will both be constructed to the same profile grade and that Wall 4W6 will be constructed on granular embankment that extends to the same bottom of wall elevation as 4W5, settlement analysis was performed for Wall 4W5 only, and is considered representative of the settlement anticipated along Wall 4W6. Total settlements of 3.56 to 5.44 inches at the center of the reinforced mass and 2.27 to 2.37 inches at the facing of the wall are anticipated along the portion of the alignment of Retaining Walls 4W5 and 4W6 where the standard MSE walls are proposed. Based on the results of the analysis, 90 percent of the total settlement at the facing of the walls is anticipated to occur during construction of the wall or within 11 to 35 days following the completion of construction of the walls. Please note that the consolidation settlement and time rate of consolidation are based on estimates using correlated compressibility parameters provided in Table 10 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 11. Retaining Wall 4W5 and 4W6 MSE Wall Settlement Values

Structure Reference	From Station [1]	To Station [1]	Wall Height Analyzed (feet)	Service Limit Equivalent Bearing Pressure [2] (ksf)	Total Settlement Values [3] (inches)		Time for 90% Consolidation [3] (Days)
					Center of Wall Mass	Facing of Wall	
Retaining Walls 4W5 and 4W6	5083+25, 24.3' Rt.	5085+54, 24.3' Rt.	39.4	6.29	5.44 / 3.56	2.37 / 2.27	11 / 35

1. *Station and offset referenced to the baseline of Ramp C5.*
2. *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.*
3. *The first settlement value indicates the settlement based on the wall height and bearing elevation at boring B-023-1-13, and the second value indicates the settlement based on the wall height and bearing elevation at boring B-024-0-08.*

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." Based on the total anticipated settlement at the facing of the walls, maximum differential settlements in the longitudinal directions are anticipated to be less than 1/1000, which is within the tolerable limit of 1/100. If the total or differential settlement values predicted for the proposed walls present 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 walls. 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 VI.

5.2.4 Eccentricity (Overturning Stability)

The resistance of the MSE walls to overturning will be dependent 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 2018 AASHTO LRFD BDS, for foundations bearing on soil or cellular concrete, 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 retaining wall configurations indicated in Table 9. Based on the minimum length of reinforcement presented in Table 9 and utilizing the parameters listed in Section 5.2.1 for the retained material, the calculated eccentricity of the resultant force **will not exceed** the limiting eccentricity at the strength limit state.

5.2.5 Sliding Stability

The resistance of the MSE walls to sliding was evaluated per Section 11.10.5.3 of the 2018 AASHTO LRFD BDS. Given that the bearing soils will consist of cohesive material, the sliding resistance was evaluated under both drained and undrained conditions. 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 material and the reinforced backfill. Based on the material parameters listed in Section 5.2.1 for the foundation and retained material, a coefficient of sliding friction of 0.49 was utilized for design. 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 material parameters listed in Section 5.2.1, the undrained shear strength of the bearing soil is 2.95 ksf. A geotechnical resistance factor of φ_τ=1.0 was considered in calculating the factored shear resistance between the reinforced backfill and foundation material for sliding. Based on the minimum length of reinforced soil mass presented in Table 9 and utilizing the material parameters listed in Section 5.2.1 for the retained 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 or undrained conditions.

5.2.6 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 5.2.1. 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 6.0 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 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 is satisfied when a minimum factor of safety of 1.3 is obtained. For MSE walls designed with a minimum strap length listed in Table 9, the resulting factor of safety under drained conditions (long-term stability) and undrained conditions (short-term stability) using the Spencer’s analysis method was greater than 1.3.

5.2.7 *Final MSE Wall Considerations*

Based on the results of the external and global stability analysis performed for the MSE walls, the recommended controlling strap length is 0.70 times the height of the MSE walls (measured from the top of the leveling pad to top of the coping) along the alignment of Retaining Wall 4W5 between Sta. 5083+25 and 5085+54 (BL Ramp C5) and Retaining Wall 4W6 between Sta. 178+85 to 180+31 (BL I-70 EB). All of the external and global stability calculations indicate that adequate resistance is available for support of the proposed walls using the configurations and backfill materials outlined in the plan sheets provided by GPD GROUP.

Calculations for external (bearing and sliding resistance and limiting eccentricity) and overall (global) stability of the MSE 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 12 and Table 13.

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

Soil Type	γ (pcf) [1]	c (psf)	φ	k_a	k_o	k_p
Soft to Stiff Cohesive Soil	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 13. Estimated Drained (Long-term) Soil Parameters for Design

Soil Type	γ (pcf) [1]	c (psf)	φ'	k_a	k_o	k_p
Soft to Stiff Cohesive Soil	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 14. Excavation Back Slopes

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

5.4.2 Groundwater Considerations

Based on the groundwater observations made during drilling, groundwater may be encountered during construction of the drilled shafts. Where groundwater is encountered, proper groundwater control should be employed and maintained to prevent disturbance to excavation bottoms consisting of cohesive soil, and to prevent the possible development of a quick or "boiling" condition where soft silts and/or fine sands are encountered. It is preferable that the groundwater level, if encountered, be maintained at least 36 inches below the deepest excavation. In the case of drilled shafts, the utilization of casing will be required below the water table to maintain an open hole and prevent the sidewalls from collapse. In addition, concrete placed below the water table should be placed by tremie method using a rigid tremie pipe. Note that mitigating the water during construction and protecting the excavation is the responsibility of the contractor.

6.0 LIMITATIONS OF STUDY

The recommendations in this report 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 our recommendations.

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

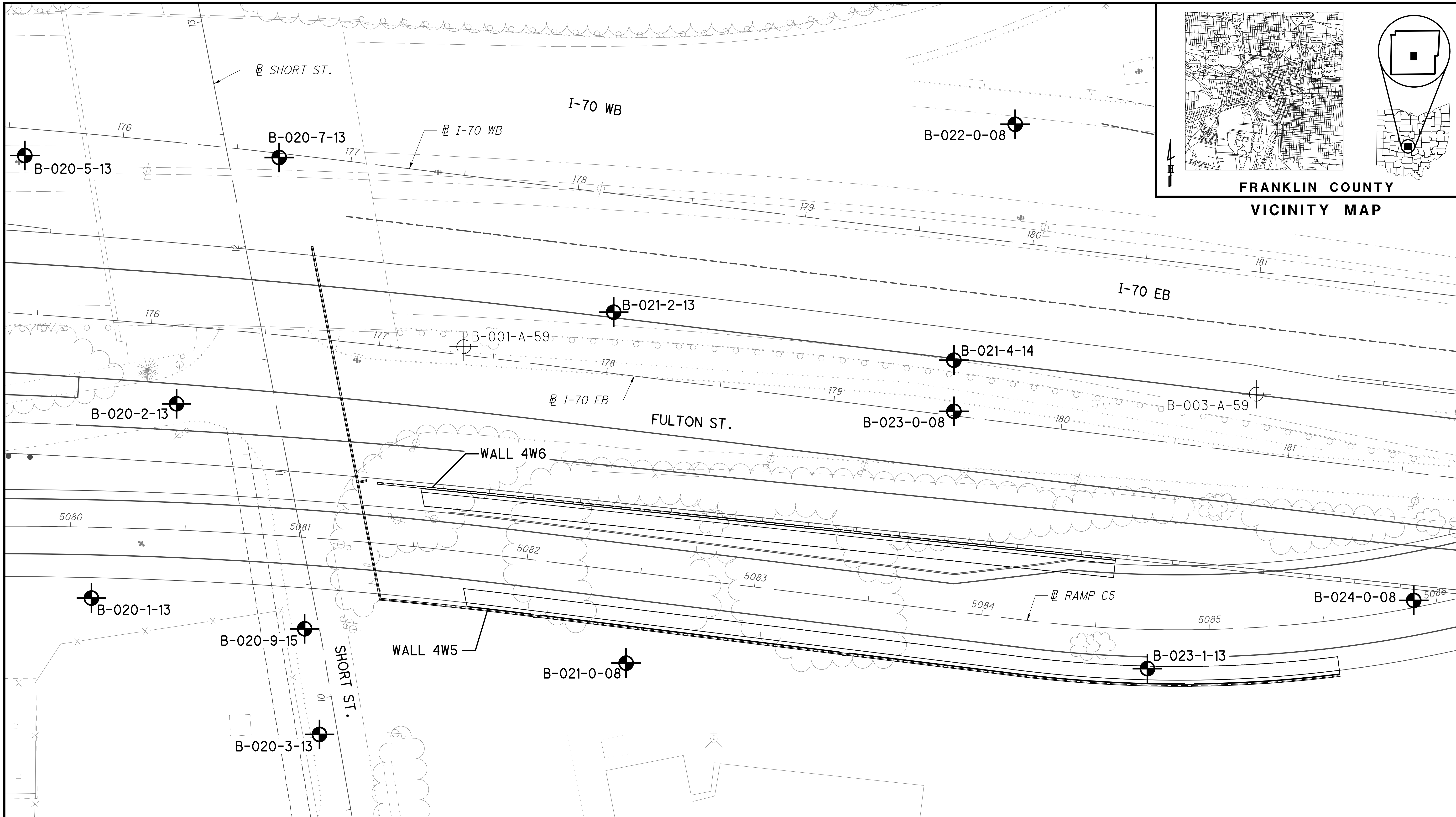
The conclusions and recommendations herein have been based upon the available soil and bedrock information and the design details furnished by a representative of the owner of the proposed project. Any revision in the plans for the proposed construction from those anticipated in this report should be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

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

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

APPENDIX I

VICINITY MAP AND BORING PLAN



BORING PLAN

FRA-70-12.68 - RETAINING WALL 4W5 AND 4W6

FRANKLIN COUNTY, OHIO

RII PROJECT NO. W-13-045	DRAWN RRM	RESOURCE INTERNATIONAL, INC.
SCALE: 1"=40' (0 20 40)	REVIEWED BRT	
	DATE 7-12-18	

APPENDIX II

DESCRIPTION OF SOIL TERMS

DESCRIPTION OF SOIL TERMS

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

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

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

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

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

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

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

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

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

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

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

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

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

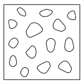
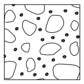
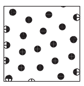
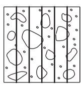
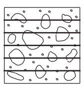
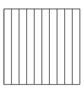
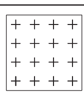
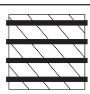
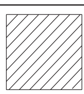
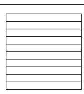
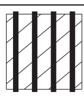
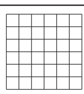
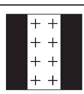
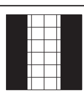
Bedrock – The following terms are used to describe the relative strength of bedrock:

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

CLASSIFICATION OF SOILS

Ohio Department of Transportation

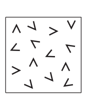
(The classification of a soil is found by proceeding from top to bottom of the chart. The first classification that the test data fits is the correct classification.)

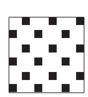
SYMBOL	DESCRIPTION	Classifcation AASHTO	Classifcation OHIO	LL_O/LL × 100*	% Pass #40	% Pass #200	Liquid Limit (LL)	Plastic Index (PI)	Group Index Max.	REMARKS
	Gravel and/or Stone Fragments	A-1-a			30 Max.	15 Max.		6 Max.	0	Min. of 50% combined gravel, cobble and boulder sizes
	Gravel and/or Stone Fragments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

MATERIAL CLASSIFIED BY VISUAL INSPECTION

 Sod and Topsoil

 Pavement or Base

 Uncontrolled Fill (Describe)

 Bouldery Zone

 Peat

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

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-020-1-13, B-020-2-13, B-020-3-13,
B-020-7-13, B-020-9-15, B-021-0-08,
B-023-0-08, B-023-1-13 and B-024-0-08

BORING LOGS

Definitions of Abbreviations

AS = Auger sample

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

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

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

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

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

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

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

RC = Rock core sample

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

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

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

S = Sulfate content (parts per million)

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

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

SS = Split spoon sample

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

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

TR = Top of rock

W = Initial water level measured during drilling

▼ = Water level measured at completion of drilling

Classification Test Data

Gradation (as defined on Description of Soil Terms):

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

Atterberg Limits:

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

WC = Water content (%)

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-12.68 - 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**

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'	

W (23)

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	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)

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)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
MEDIUM DENSE TO DENSE, BROWN **GRAVEL AND SAND**, LITTLE CLAY, TRACE SILT, MOIST. *(same as above)*	680.8	31–32																
HARD, GRAY **SANDY SILT**, SOME CLAY, LITTLE FINE GRAVEL, DAMP.		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	35–37																
VERY DENSE, GRAY TO BROWNISH GRAY **GRAVEL AND SAND**, TRACE SILT, DAMP TO MOIST.		39	7 13 34	62	100	SS-14	-	-	-	-	-	-	-	-	-	4	A-1-b (V)	
-INTRODUCED WATER @ 40.0'		44	20 43 50/5"	-	100	SS-15	-	-	-	-	-	-	-	-	-	11	A-1-b (V)	
		49	17 41 50/3"	-	100	SS-16	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	
	660.8	52																
HARD, GRAY **CLAY**, LITTLE SILT, TRACE COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP.																		
-SHALE FRAGMENTS PRESENT IN SS-17		54	18 30 25	72	100	SS-17	-	-	-	-	-	-	-	-	-	14	A-7-6 (V)	
		59	30 26 33	77	78	SS-18	4.5+	6	1	8	18	67	44	21	23	14	A-7-6 (14)	

Depth scale: 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48 49 50 51 52 53 54 55 56 57 58 59 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 (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
HARD, GRAY **CLAY**, LITTLE SILT, TRACE COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP. *(same as above)*	648.6	63 – 64	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'.		TR 64 – 75	0		0	RC-1											CORE	
		75	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 – 86 EOB	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

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-12.68 - PHASE 4A	DRILLING FIRM / OPERATOR: RII / S.M.	DRILL RIG: CME-750 (SN 98048)	STATION / OFFSET: 176+13.92 / 34' RT
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / C.H.	HAMMER: CME AUTOMATIC	ALIGNMENT: BL I-70 EB
PID: 77372 BR ID: FRA-70-1373R	DRILLING METHOD: 3.25" HSA / RC	CALIBRATION DATE: 4/26/13	ELEVATION: 711.4 (MSL) EOB: 84.5 ft.
START: 7/16/13 END: 7/17/13	SAMPLING METHOD: SPT / NQ	ENERGY RATIO (%): 82.6	LAT / LONG: 39.953155708, -83.004534664

EXPLORATION ID **B-020-2-13**

MATERIAL DESCRIPTION AND NOTES	ELEV. 711.4
0.3' - ASPHALT (4.0")	711.1
0.5' - AGGREGATE BASE (6.0")	710.6
FILL: STIFF, BLACK AND BROWN **SILT AND CLAY**, SOME COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP. -ROCK FRAGMENTS PRESENT	708.4
FILL: VERY LOOSE, BLACK AND BROWN **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST.	705.9
POSSIBLE FILL: STIFF, BROWN **SILT AND CLAY**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP.	703.4
POSSIBLE FILL: MEDIUM STIFF TO STIFF, BROWN **SILTY CLAY**, TRACE COARSE TO FINE SAND, MOIST.	698.4
POSSIBLE FILL: MEDIUM STIFF, BROWN **CLAY**, SOME SILT, AND FINE TO COARSE SAND, LITTLE FINE GRAVEL, MOIST. -CONSOLIDATION TEST PERFORMED @ 14.7'	693.4
POSSIBLE FILL: LOOSE, BROWN **GRAVEL WITH SAND AND SILT**, TRACE CLAY, WET. -HEAVING SANDS ENCOUNTERED @ 18.5' -COBBLES PRESENT @ 19.5'	690.9
DENSE TO VERY DENSE, BROWN **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST.	

DEPTHS: 1 – 29; W (at 18)

SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
4, 8, 6	19	72	SS-1	1.75	-	-	-	-	-	-	-	-	16	A-6a (V)
2, 2, 1	4	33	SS-2	-	47	16	12	19	6	26	23	3	15	A-1-b (0)
3, 4, 7	15	33	SS-3	2.00	-	-	-	-	-	-	-	-	15	A-6a (V)
2, 2, 3	7	83	SS-4	1.00	0	1	8	54	37	39	20	19	25	A-6b (12)
3, 4, 6	14	83	SS-5	1.75	-	-	-	-	-	-	-	-	27	A-6b (V)
		83	ST-6	0.75	17	31	13	24	15	41	16	25	22	A-7-6 (4)
1, 1, 2	4	33	SS-7	0.75	-	-	-	-	-	-	-	-	25	A-7-6 (V)
WOH, 2, 4	8	33	SS-8	-	33	21	13	26	7	31	25	6	30	A-2-4 (0)
4, 13, 10	32	67	SS-9	-	-	-	-	-	-	-	-	-	16	A-1-b (V)
50/5"	-	80	SS-10	-	-	-	-	-	-	-	-	-	14	A-1-b (V)
9, 12, 14	36	33	SS-11	-	-	-	-	-	-	-	-	-	14	A-1-b (V)
9, 24, 32	77	39	SS-12	-	47	27	10	12	4	21	18	3	7	A-1-b (0)

PID: 77372	BR ID: FRA-70-1373R	PROJECT: FRA-70-12.68 - PHASE 4A	STATION / OFFSET: 176+13.92 / 34 RT	START: 7/16/13	END: 7/17/13	PG 2 OF 3	B-020-2-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 681.4	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
DENSE TO VERY DENSE, BROWN **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST. *(same as above)*		31–33																
		34–35	14 29 33	85	61	SS-13	-	-	-	-	-	-	-	-	-	12	A-1-b (V)	
		36–38																
		39–40	22 48 50/5"	-	71	SS-14	-	-	-	-	-	-	-	-	-	14	A-1-b (V)	
		41–43																
	666.9	44	15 31 29	83	94	SS-15	-	-	-	-	-	-	-	-	-	12	A-1-b (V)	
HARD, BROWN **SILT AND CLAY**, LITTLE FINE GRAVEL, TRACE COARSE TO FINE SAND, DAMP.		45					4.5+	11	1	4	46	38	32	18	14	17	A-6a (10)	
	664.4	46–47																
HARD, BROWN **SILTY CLAY**, LITTLE FINE GRAVEL, TRACE COARSE TO FINE SAND, DAMP.		48																
		49–50	29 27 25	72	22	SS-16	-	-	-	-	-	-	-	-	-	18	A-6b (V)	
		51–53																
		54–55	11 20 20	55	56	SS-17	4.50	-	-	-	-	-	-	-	-	17	A-6b (V)	
		56–58																
		59	14 50/3"	-	89	SS-18	-	19	3	4	36	38	38	20	18	16	A-6b (11)	
	649.4	60–61																

PID: 77372	BR ID: FRA-70-1373R	PROJECT: FRA-70-12.68 - PHASE 4A	STATION / OFFSET: 176+13.92 / 34 RT	START: 7/16/13	END: 7/17/13	PG 3 OF 3	B-020-2-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 649.3	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
HARD, BROWN **SILT AND CLAY**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP. *(same as above)*	647.1	63 – 64 TR	35 50/3"	-	100	SS-19	-	-	-	-	-	-	-	-	-	11	A-6a (V)	
SHALE: GRAY, VERY WEAK, HIGHLY WEATHERED.	641.9	65 – 69	49 50/2"	-	100	SS-20	-	-	-	-	-	-	-	-	-	9	Rock (V)	
SHALE : GRAY AND BLACK, SLIGHTLY TO MODERATELY WEATHERED, WEAK AND STRONG, THINLY LAMINATED TO MEDIUM BEDDED, CALCAREOUS, PYRITIC, FISSILE, FRACTURED TO HIGHLY FRACTURED, NARROW TO OPEN APERTURE, SMOOTH TO SLIGHTLY ROUGH; RQD 4%, REC 30%.		70 – 74	8		40	RC-1											CORE	
	631.9	75 – 79	0		20	RC-2											CORE	
LIMESTONE : GRAY AND TAN, SLIGHTLY WEATHERED, MODERATELY STRONG, VERY THIN TO MEDIUM BEDDED, CALCAREOUS, PYRITIC, DOLOMITIC, CHERT NODULES, FRACTURED TO SLIGHTLY FRACTURED, NARROW APERTURE, SMOOTH; RQD 97%, REC 97%.	626.9	80 – 84 EOB	97		97	RC-3											CORE	

NOTES: GROUNDWATER INITIALLY ENCOUNTERED @ 18.5'

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

B-020-2-13 – RC-1, RC-2, and RC-3 – Depth from 69.5 to 84.5 feet

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-12.68 - PHASE 4A	DRILLING FIRM / OPERATOR: RII / J.B.	DRILL RIG: MOBILE B-53 (SN 624400)	STATION / OFFSET: 5081+15.55 / 85.3' RT	EXPLORATION ID	B-020-3-13
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / S.B.	HAMMER: AUTOMATIC	ALIGNMENT: BL RAMP C5	PAGE	
PID: 77372 BR ID: FRA-70-1373A	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 4/26/13	ELEVATION: 712.3 (MSL) EOB: 49.8 ft.	1 OF 2	
START: 8/15/13 END: 8/21/13	SAMPLING METHOD: SPT	ENERGY RATIO (%): 77.7	LAT / LONG: 39.952760425, -83.004309070		

MATERIAL DESCRIPTION AND NOTES	ELEV. 712.3
0.3' - ASPHALT (4.0")	712.0
0.3' - BRICK (4.0")	711.7
0.8' - AGGREGSTE BASE (9.0")	710.9
POSSIBLE FILL: STIFF, BROWN **SILT AND CLAY**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP. -TRACE ORGANICS IN SS-1	709.3
POSSIBLE FILL: STIFF TO VERY STIFF, GRAY TO BROWN **SILTY CLAY**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST.	699.3
POSSIBLE FILL: MEDIUM STIFF, BROWN **SILT AND CLAY**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST.	696.8
MEDIUM DENSE TO VERY DENSE, BROWN AND GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST. -COBBLES PRESENT THROUGHOUT	691.8
LOOSE TO VERY DENSE, GRAY **GRAVEL AND SAND**, TRACE SILT, MOIST. -COBBLES PRESENT THROUGHOUT -AUGER REFUSAL ENCOUNTERED @ 26.0'. ATTEMPTED 5.0' CORE RUN. GRANITE BOULDER PIECE RECOVERED IN CORE RUN. REMAINING SOIL WAS WASHED OUT DURING CORING OPERATION. CONTINUED SPT SAMPLING @ 31.3'.	

DEPTHS: W (water) at 15

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
3 3 6	12	72	SS-1	2.00	-	-	-	-	-	-	-	-	13	A-6a (V)	
3 2 3	6	61	SS-2	1.25	4	4	8	47	37	38	18	20	22	A-6b (12)	
1 2 4	8	83	SS-3	2.25	-	-	-	-	-	-	-	-	31	A-6b (V)	
4 3 5	10	78	SS-4	2.50	0	0	14	54	32	35	15	20	22	A-6b (12)	
1 2 3	6	44	SS-5	1.75	-	-	-	-	-	-	-	-	18	A-6b (V)	
WOH 2 4	8	44	SS-6	1.00	-	-	-	-	-	-	-	-	23	A-6a (V)	
14 22 20	54	39	SS-7	-	-	-	-	-	-	-	-	-	8	A-1-a (V)	
WOH 8 10	23	50	SS-8	-	-	-	-	-	-	-	-	-	9	A-1-a (V)	
2 3 4	9	50	SS-9	1.75	50	21	9	17	3	24	18	6	13	A-1-b (0)	
5 15 40	71	67	SS-10	-	-	-	-	-	-	-	-	-	11	A-1-b (V)	
50/3"	-	0	SS-11	-	-	-	-	-	-	-	-	-	-		
7		17	RC-1											CORE	

PID: 77372	BR ID: FRA-70-1373A	PROJECT: FRA-70-12.68 - PHASE 4A	STATION / OFFSET: 5081+15.55 / 85.3 RT	START: 8/15/13	END: 8/21/13	PG 2 OF 2	B-020-3-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 682.3	DEPTHS
LOOSE TO VERY DENSE, GRAY **GRAVEL AND SAND**, TRACE SILT, MOIST. *(same as above)*	680.3	32
VERY STIFF TO HARD, GRAY **SILT AND CLAY**, SOME COARSE TO FINE SAND, LITTLE FINE GRAVEL, DAMP TO MOIST.	672.8	39.5
VERY DENSE, BROWN **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, DAMP TO MOIST.	665.3	47
VERY DENSE, BROWN **GRAVEL WITH SAND AND SILT**, TRACE CLAY, DAMP. -AUGER REFUSAL ENCOUNTERED @ 49.8'. ATTEMPTED ROCK CORE RUN AT 49.8' AND RECOVERED GRANITE BOULDER PIECE. SOIL WAS OBSERVED TO BE WASHING OUT WITH THE CIRCULATION FUILD BELOW THE BOULDER. BORING TERMINATED AT 49.8' PRIOR TO ENCOUNTERING BEDROCK.	662.5	EOB

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)
10 30 50/5"	-	100	SS-12	4.5+	20	8	11	29	32	34	16	18	14	A-6b (8)
10 40 50/4"	-	100	SS-13	4.00	-	-	-	-	-	-	-	-	19	A-6b (V)
				-	-	-	-	-	-	-	-	-	8	A-1-a (V)
19 50/2"	-	38	SS-14	-	-	-	-	-	-	-	-	-	5	A-1-a (V)
25 36 50/3"	-	60	SS-15	-	47	20	12	14	7	22	14	8	10	A-2-4 (0)

NOTES: SEEPAGE ENCOUNTERED @ 15.0'

ABANDONMENT METHODS, MATERIALS, QUANTITIES: COMPACTED WITH THE AUGER 50 LBS BENTONITE CHIPS AND SOIL CUTTINGS

B-020-3-13 – RC-1 – Depth from 26.0 to 29.0 feet

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-13.10 - PHASE 6A	DRILLING FIRM / OPERATOR: STOCK / J/M	DRILL RIG: CME 55-LC (SN 360485)	STATION / OFFSET: 176+68.64 / 1.8' RT	EXPLORATION ID B-020-7-13
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / K.R.	HAMMER: AUTOMATIC	ALIGNMENT: BL I-70 WB	
PID: 89464 BR ID: FRA-70-1373L	DRILLING METHOD: 4.25" HSA / NQ	CALIBRATION DATE: 3/28/13	ELEVATION: 713.5 (MSL) EOB: 80.4 ft.	PAGE 1 OF 3
START: 1/19/15 END: 1/22/15	SAMPLING METHOD: SPT / RC	ENERGY RATIO (%): 73.2	LAT / LONG: 39.953451540, -83.004376859	

MATERIAL DESCRIPTION AND NOTES	ELEV. 713.5
0.7' - CONCRETE (8.0")	712.8
POSSIBBLE FILL: LOOSE TO MEDIUM DENSE, BROWN **GRAVEL WITH SAND AND SILT**, TRACE CLAY, MOIST.	706.5
POSSIBLE FILL: STIFF, DARK BROWN **CLAY**, "AND" SILT, TRACE COARSE TO FINE SAND, MOIST. -SWITCHED TO ROTARY DRILLING TECHNIQUES WITH WATER AND CASING ADVANCER @ 10.0' -CONSOLIDATION TEST PERFORMED @ 11.8' -CU TRIAXIAL COMPRESSION TEST PERFORMED @ 12.0'	700.5
POSSIBLE FILL: MEDIUM STIFF TO STIFF, BROWN **SILTY CLAY**, TRACE FINE SAND, TRACE FINE GRAVEL, MOIST.	696.4
POSSIBLE FILL: HARD, REDDISH BROWN **SANDY SILT**, LITTLE FINE GRAVEL, LITTLE CLAY, MOIST.	695.5
POSSIBLE FILL: MEDIUM STIFF, BROWN **CLAY**, "AND" SILT, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST.	693.0
VERY DENSE, BLACK **GRAVEL**, LITTLE COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST. -COBBLES PRESENT @ 24.0'	686.5
DENSE, BROWN AND BLACK **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST. -HEAVING SANDS ENCOUNTERED @ 28.5'	

DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
1-2	5 4 4	10	83	SS-1	-	-	-	-	-	-	-	-	-	12	A-2-4 (V)	
4-5	WOH 2 3	6	67	SS-2	-	25	31	17	19	8	NP	NP	NP	13	A-2-4 (0)	
9-10	2 4 6	12	11	SS-3	-	-	-	-	-	-	-	-	-	23	A-7-6 (V)	
11-13			71	ST-4	1.50	0	2	7	45	46	43	19	24	23	A-7-6 (14)	
14-15	2 3 4	9	81	S-5	1.25	1	0	8	49	42	38	19	19	26	A-6b (12)	
16-18			63	ST-6	0.75	-	-	-	-	-	-	-	-	-	A-6b (V)	
					4.50	19	14	12	40	15	26	21	5	21	A-4a (4)	
19-20	1 2 3	6	33	SS-7	-	-	-	-	-	-	-	-	-	24	A-7-6 (V)	
21-23			0	ST-8	-	-	-	-	-	-	-	-	-	-		
24-25	7 19 20	48	61	SS-9	-	78	11	5	4	2	NP	NP	NP	11	A-1-a (0)	
29-30	5 13 14	33	100	SS-10	-	-	-	-	-	-	-	-	-	15	A-1-b (V)	

2014 ODOT BORING LOG-RII NE BRIDGE ID - OH DOT.GDT - 4/21/15 08:35 - U:\GI8\PROJECTS\2013W-13-072.GPJ

MATERIAL DESCRIPTION AND NOTES	ELEV. 683.5	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
DENSE, BROWN AND BLACK **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST. *(same as above)*	681.5	31 – 32																
HARD, BROWNISH GRAY **SILT AND CLAY**, SOME COARSE TO FINE SAND, LITTLE FINE GRAVEL, MOIST.	679.5	32 – 34	11 20 33	65	72	SS-11	4.5+	11	11	19	33	26	27	15	12	13	A-6a (6)	
VERY DENSE, BLACK **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, MOIST.		34 – 35					-	-	-	-	-	-	-	-	-	9	A-1-b (V)	
		39 – 40	6 20 38	71	94	SS-12	-	54	14	22	7	3	NP	NP	NP	14	A-1-b (0)	
-COBBLES PRESENT @ 40.0'	671.5	40 – 42																
HARD, GRAY **SILT AND CLAY**, SOME COARSE TO FINE SAND, LITTLE FINE GRAVEL, DAMP.		44 – 45	4 6 21	33	44	SS-13	4.50	-	-	-	-	-	-	-	-	15	A-6a (V)	
	666.5	47																
VERY DENSE, GRAY AND BLACK **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST.		49 – 50	9 32 41	89	100	SS-14	-	70	13	9	5	3	NP	NP	NP	10	A-1-a (0)	
	659.5	54	5 20 39	72	78	SS-15	-	-	-	-	-	-	-	-	-	12	A-1-a (V)	
HARD, GRAY **SILT AND CLAY**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST. AUGER REFUSAL @ 55.0'		54 – 55					4.5+	-	-	-	-	-	-	-	-	13	A-6a (V)	
		55 – 56.5			25	RC-1	-	-	-	-	-	-	-	-	-	-	A-6a (V)	
-0.8' GRANITE BOULDER @ 56.5'		56.5 – 61.5			33	RC-2	-	-	-	-	-	-	-	-	-	-	A-6a (V)	
-0.8' MUDSTONE SEAM @ 60.7'		61																

PID: 89464	BR ID: FRA-70-1373L	PROJECT: FRA-70-13.10 - PHASE 6A	STATION / OFFSET: 176+68.64 / 1.8 RT	START: 1/19/15	END: 1/22/15	PG 3 OF 3	B-020-7-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 651.4	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
HARD, GRAY **SILT AND CLAY**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST. *(same as above)* -0.1' THICK PIECE OF GRANITE RECOVERED IN RC-3. REMAINING SOIL WASHED OUT DURING CORING.	648.1	63–65, TR			2	RC-3	-	-	-	-	-	-	-	-	-	-	A-6a (V)	
MUDSTONE : GRAY, HIGHLY WEATHERED, VERY WEAK, THINLY LAMINATED TO THIN BEDDED, ARENACEOUS, CALCAREOUS, FRIABLE, FISSILE, SLIGHTLY TO HIGHLY FRACTURED, THIGHT APERTURES, SLIGHTLY ROUGH; RQD 74%, REC 98%. -POINT LOAD STRENGTH @ 69.4' to 74.8' -MEAN QU = 224 PSI		66–75	89		97	RC-4											CORE	
	633.1	76–80, EOB	45		100	RC-5											CORE	

NOTES: GROUNDWATER NOT ENCOUNTERED PRIOR TO INTORDUCTION OF WATER AS A DRILLING FLUID

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

B-020-7-13 – RC-1, RC-2, RC-3 and RC-4 – Depth from 55.0 to 73.5 feet

B-020-7-13 – RC-4 (cont.) and RC-5 – Depth from 73.5 to 80.4 feet

B-020-9-13 ALT – RC-1 and RC-2 – Depth from 60.5 to 70.5 feet

B-020-9-13 ALT – RC-3 – Depth from 70.5 to 75.5 feet

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-12.68 - PHASE 4A	DRILLING FIRM / OPERATOR: RII / J.K.	DRILL RIG: CME 55 (SN 386345)	STATION / OFFSET: 5081+05.25 / 39.8' RT	EXPLORATION ID B-020-9-15
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / C.D.	HAMMER: CME AUTOMATIC	ALIGNMENT: BL RAMP C5	
PID: 77372 BR ID: FRA-70-1373A	DRILLING METHOD: 4.25" HSA / RC	CALIBRATION DATE: 10/20/14	ELEVATION: 713.0 (MSL) EOB: 75.5 ft.	PAGE 1 OF 3
START: 3/16/15 END: 3/17/15	SAMPLING METHOD: SPT / HQ	ENERGY RATIO (%): 92	LAT / LONG: 39.952886963, -83.004333117	

MATERIAL DESCRIPTION AND NOTES	ELEV. 713.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
0.3'- ASPHALT (4.0")	712.7																	
0.4' - BRICK (4.0")	712.3																	
0.3' - AGGREGATE BASE (3.0")	712.0	1–2	5 4 3	11	56	SS-1	1.25	-	-	-	-	-	-	-	-	22	A-6a (V)	
POSSIBLE FILL: STIFF, DARK BROWN **SILT AND CLAY**, SOME COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP TO MOIST.	707.5	3.5–5	4 4 4	12	100	SS-2	1.50	10	10	14	37	29	35	20	15	18	A-6a (8)	
POSSIBLE FILL: STIFF, DARK BROWN AND BLACK TO BROWNISH GRAY **SILTY CLAY**, LITTLE COARSE TO FINE SAND, MOIST.		6–7.5	3 2 4	9	61	SS-3	2.00	-	-	-	-	-	-	-	-	37	A-6b (V)	
	702.5	8.5–10	4 4 6	15	100	SS-4	2.00	0	1	13	50	36	34	18	16	21	A-6b (10)	
DENSE, GRAY **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, DAMP. -ROCK FRAGMENTS PRESENT IN SS-5	700.0	11–12.5	7 11 12	35	39	SS-5	-	-	-	-	-	-	-	-	-	5	A-1-b (V)	
MEDIUM DENSE, BROWN **SANDY SILT**, "AND" FINE GRAVEL, TRACE CLAY, MOIST. -ROCK FRAGMENTS PRESENT IN SS-6	697.5	13.5–15	6 5 13	28	100	SS-6	-	38	15	10	29	8	22	19	3	14	A-4a (0)	
VERY DENSE, BROWNISH GRAY **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST. -ROCK FRAGMENTS PRESENT IN SS-7	695.0	16–17.5	20 14 24	58	89	SS-7	-	-	-	-	-	-	-	-	-	6	A-1-b (V)	
MEDIUM DENSE, BROWNISH GRAY **SANDY SILT**, SOME FINE GRAVEL, TRACE CLAY, MOIST.	692.5	18.5–20	13 9 9	28	100	SS-8	-	34	18	12	26	10	NP	NP	NP	10	A-4a (0)	
MEDIUM DENSE TO VERY DENSE, BROWN **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST.		21–22.5	14 43 17	92	50	SS-9	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
-ROCK FRAGMENTS PRESENT THROUGHOUT		W 23.5–25	13 4 8	18	33	SS-10	-	33	38	9	13	7	NP	NP	NP	16	A-1-b (0)	
-PETROLEUM ODOR PRESENT IN SS-11	685.0	26–27.3	23 25 50/4"	-	88	SS-11	-	47	21	14	12	6	NP	NP	NP	11	A-1-b (0)	
VERY DENSE, BROWN **GRAVEL WITH SAND AND SILT**, TRACE CLAY, MOIST. -INTRODUCED MUD @ 30.0'		28.5–30	7 14 19	51	100	SS-12	-	-	-	-	-	-	-	-	-	8	A-2-4 (V)	

PID: 77372	BR ID: FRA-70-1373A	PROJECT: FRA-70-12.68 - PHASE 4A	STATION / OFFSET: 5081+05.25 / 39.8 RT	START: 3/16/15	END: 3/17/15	PG 2 OF 3	B-020-9-15

MATERIAL DESCRIPTION AND NOTES	ELEV. 683.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
VERY DENSE, BROWN **GRAVEL WITH SAND AND SILT**, TRACE CLAY, MOIST. *(same as above)*		31–36																
		34	5 30 42	110	100	SS-13	-	-	-	-	-	-	-	-	-	8	A-2-4 (V)	
	676.0	37																
VERY DENSE, GRAY **GRAVEL WITH SAND, SILT, AND CLAY**, MOIST.		38																
		39	34 37 50/5"	-	82	SS-14	-	35	18	14	15	18	26	13	13	9	A-2-6 (1)	
	671.0	40–42																
VERY DENSE, BROWNISH GRAY TO GRAY **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, MOIST.		43																
		44	17 32 15	72	100	SS-15	-	-	-	-	-	-	-	-	-	14	A-1-b (V)	
		45–48																
-ROCK FRAGMENTS PRESENT IN SS-16		49	17 50/5"	-	73	SS-16	-	-	-	-	-	-	-	-	-	14	A-1-b (V)	
	661.0	50–52																
VERY STIFF TO HARD, GRAY **CLAY**, SOME SILT, LITTLE FINE GRAVEL, TRACE COARSE TO FINE SAND, DAMP.		53																
		54	12 24 39	97	100	SS-17	4.5+	11	3	3	34	49	42	19	23	14	A-7-6 (14)	
-BECOMING SHALE WITH DEPTH		55–58																
		59	23 50/4"	-	100	SS-18	3.75	-	-	-	-	-	-	-	-	16	A-7-6 (V)	
	652.5	TR 60																
		61																

PID: 77372	BR ID: FRA-70-1373A	PROJECT: FRA-70-12.68 - PHASE 4A	STATION / OFFSET: 5081+05.25 / 39.8 RT	START: 3/16/15	END: 3/17/15	PG 3 OF 3	B-020-9-15

MATERIAL DESCRIPTION AND NOTES	ELEV. 650.9	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
SHALE : DARK GRAY TO BLACK, HIGHLY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED TO MEDIUM BEDDED, ARENACEOUS, CALCAREOUS, FRIABLE, FISSILE, JOINTED, MODERATELY TO HIGHLY FRACTURED, THIGHT TO OPEN APERTURES, SLIGHTLY TO VERY ROUGH; RQD 48%, REC 75%. *(same as above)* -0.4' GRANITE BOULDER @ 60.5' -0.2' CLAY SEAM @ 60.9' -0.4' LIMESTONE SEAM @ 61.5' -0.5' LIMESTONE SEAM @ 64.5'		63 64 65	17		52	RC-1											CORE	
		66 67 68 69 70	48		75	RC-2											CORE	
-SLIGHTLY WEATHERED AND SLIGHTLY FRACTURED IN RC-3	637.5	71 72 73 74 75 EOB	80		100	RC-3											CORE	

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 24.5'

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PUMPED 188 LBS CEMENT / 50 LBS BENTONITE POWDER / 40 GAL WATER

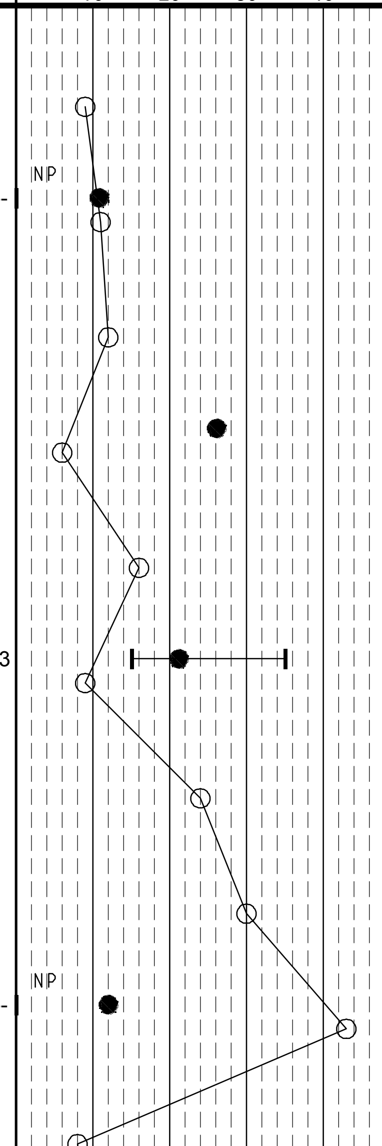
Client: ms consultants	Project: FRA-70-8.93	Job No. 0221-1004.01
LOG OF: Boring B-021-0-08	Location: Sta. 5082+48.43, 39.8' RT., BL RAMP C5	Date Drilled: 7/29/2008 to 7/31/2008

WATER OBSERVATIONS:
Water seepage at: 0.0-8.0, 18.0-23.0, 28.0-57.0
Water level at completion: 26.9' (beginning of shift, 7/30/08)
25.3' (includes drilling water)

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

Depth (ft)	Elev. (ft) 727.9	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0.3	727.6						Gravel - 3"						
							FILL: Loose to medium dense brown and gray GRAVEL WITH SAND (A-1-b), little silt; wet.						
		5 5 4	1	1			@ 1.0', drove splitspoon on gravel particle.						
5		6 5 6	6	2				63	13	---	8	--16--	
		10 6 6	3	3									
8.0	719.9						POSSIBLE FILL: Medium stiff to stiff brown SILTY CLAY (A-6b), trace to little fine to coarse sand, trace gravel; moist.						
10		6 3 3	10	4		1.0							
		5 7 9	1	5		--	@ 11.0'-12.0', drove gravel in spoon.						
							@ 13.0', very stiff.						
15		2 3 6	18	6		2.5		1	7	---	10	39	43
		3 10 14	6	7		2.75							
18.0	709.9						Medium dense to dense gray GRAVEL (A-1-a), trace to little silty clay, trace fine sand; wet.						
20		11 13 16	5	8									
							@ 21.0'-22.5', light brown, some fine to coarse sand.						
		11 19 23	18	9				50	20	---	15	--15--	
23.0	704.9						Stiff to very stiff brown SANDY SILT (A-4a), some clay, little gravel; moist.						
25	702.9	9 5 3	18	10		2.0							

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-021-0-08	*Location:* Sta. 5082+48.43, 39.8' RT., BL RAMP C5	*Date Drilled:* 7/29/2008 to 7/31/2008

WATER OBSERVATIONS:
Water seepage at: 0.0-8.0, 18.0-23.0, 28.0-57.0
Water level at completion: 26.9' (beginning of shift, 7/30/08) 25.3' (includes drilling water)

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

Surface Elev. (ft): 702.9

Depth (ft)	Elev. (ft)	DESCRIPTION
		Stiff to very stiff brown SANDY SILT (A-4a), some clay, little gravel; moist. @ 25.0'-35.0', set piezometer screen.
28.0	699.9	Medium dense brown GRAVEL WITH SAND (A-1-b), some fine to coarse sand, little to some silty clay; wet. @ 33.5'-35.0', drove spoon on boulder. No sample was recovered. @ 33.5'-43.0', very dense; boulders and cobble caused difficult drilling and poor sample recovery. @ 37.0', gray.
42.0	685.9	Very dense brown FINE SAND (A-3), some coarse sand, little gravel, trace silty clay; wet.
50	677.9	

Sample No. (Drive)	Blows per 6"	Recovery	Hand Penetrometer (tsf)	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	Standard Penetration (N60) / Moisture
11	5, 5, 5	18	1.5	11	12	---	32	19	26	
12	5, 8, 13	18								
13	50/1	0								50+
14	43, 48, 42	18		53	17	---	14	--15--		92; NP
15	25, 32, 38	10								71
16	22, 28, 35	16		14	35	---	42	--8--		64; NP

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

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

LOG OF: Boring B-021-0-08 | **Location:** Sta. 5082+48.43, 39.8' RT., BL RAMP C5 | **Date Drilled:** 7/29/2008 to 7/31/2008

WATER OBSERVATIONS:
Water seepage at: 0.0-8.0, 18.0-23.0, 28.0-57.0
Water level at completion: 26.9' (beginning of shift, 7/30/08)
25.3' (includes drilling water)

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

Depth (ft)	Elev. (ft) 677.9	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N60)
							Very dense brown FINE SAND (A-3), some coarse sand, little gravel, trace silty clay; wet.							
55		25 30 46	18	17										78
57.0	670.9						Hard gray SILTY CLAY (A-6b), trace to little gravel, trace fine to coarse sand; damp to moist.							
60		11 21 50/5	14	18		4.5+		0	1	---	1	31	67	50+
65		23 25 25	18	19		4.5+	@ 63.5'-65.0', brown, "and" fine to coarse sand; encountered sandstone fragments.	12	20	---	18	23	27	51
67.0	660.9						Shale, blue-gray, severely weathered, weak, laminated, pyritic.							
70		13 14 26	8	20										
71.2	656.7	Core 60"	Rec 60"	RQD 82%	R1		Shale, blue-gray, highly weathered, weak, thinly laminated, friable, fissile, pyritic, slightly calcareous, jointed, moderately fractured, tight, slightly rough; RQD 83%, Loss 0%. @ 73.1' - 73.3', high angle fracture.							
75	652.9													

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

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

LOG OF: Boring B-021-0-08 | *Location:* Sta. 5082+48.43, 39.8' RT., BL RAMP C5 | *Date Drilled:* 7/29/2008 to 7/31/2008

WATER OBSERVATIONS:
Water seepage at: 0.0-8.0, 18.0-23.0, 28.0-57.0
Water level at completion: 26.9' (beginning of shift, 7/30/08) 25.3' (includes drilling water)

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

Depth (ft)	Elev. (ft) 652.9	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	Graphic Log	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N60) Natural Moisture Content, % - ● PL ├─┤ LL Blows per foot - ○ / Non-Plastic - NP 10 20 30 40
		Core 60"	Rec 60"	RQD 90%	R2		Shale, blue-gray, highly weathered, weak, thinly laminated, friable, fissile, pyritic, slightly calcareous, jointed, moderately fractured, tight, slightly rough; RQD 83%, Loss 0%. @ 75.8', encountered large pyritic inclusions. @ 76.4'-76.8' and 78.8'-78.9', high angle fracture.								
80															
82.2	645.7	Core 60"	Rec 60"	RQD 75%	R3		Interbedded Shale (91%) and Limestone (9%) RQD 82%, LOSS 0%; Shale, dark gray, weak, moderately to highly weathered, laminated, calcareous, fissile, jointed, moderately fractured, tight, slightly rough; Limestone, light gray, slightly weathered, slightly strong to strong, thinly bedded, pyritic. @ 82.8' - 83.3', qu = 1536 psi.								
85															
		Core 60"	Rec 60"	RQD 87%	R4										
90.0	637.9						Bottom of Boring - 90.0'								
95															
100															

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

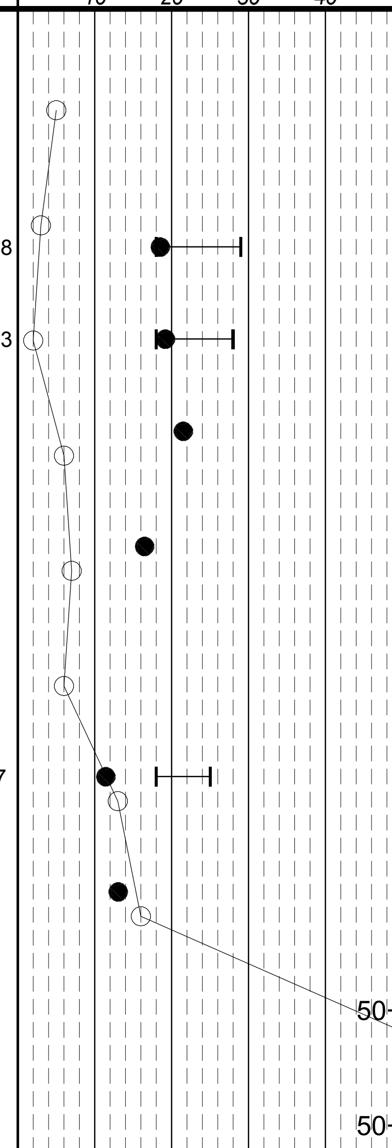
LOG OF: Boring B-023-0-08 | *Location:* Sta. 179+52.56, 1.9' LT., BL I-70 EB | *Date Drilled:* 8/20/2008 to 9/2/2008

WATER OBSERVATIONS:
Water seepage at: 25.0'
Water level at completion: 19.1'

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

Depth (ft)	Elev. (ft) 722.0	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
							Asphalt Concrete - 4" Portland Cement Concrete - 8" Aggregate Base - 6"						
1.5	720.5	7 2 3	4	1		3.0	FILL: Very soft to soft brown SILT AND CLAY (A-6a), some fine to coarse sand, trace to little gravel; contains few brick fragments; moist. @ 1.5'-2.5', stiff.						
5		1 1 2	12	2	P1	0.25		14	14	---	21	33	18
6.0	716.0	WOH WOH 2	2	3	P2	1.5 0.25	FILL: Very soft to soft brown SANDY SILT (A-4a), some to "and" fine to coarse sand, trace to little gravel; contains few brick fragments; moist. @ 6.0'-7.5', slight organic odor.	9	16	---	20	32	23
10		1 2 4	18	4		<0.25							
		WOH 3 4	18	5		--							
13.5	708.5	2 3 3	18	6			POSSIBLE FILL: Loose to medium dense brown GRAVEL WITH SAND AND SILT (A-2-4), trace clay; damp.						
15		7 6 6	10	7				40	15	---	14	24	7
20		7 7 8	8	8			@ 21.0'-21.5', contains turpentine odor.						
21.5	700.5	18 50/6	10	9A 9B			Very dense medium orangish-brown SANDY SILT (A-4a), some fine to coarse sand, some gravel; contains turpentine odor; moist.						
25	697.0	11 43 50/5	14	10									

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



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

LOG OF: Boring B-023-0-08 | *Location:* Sta. 179+52.56, 1.9' LT., BL I-70 EB | *Date Drilled:* 8/20/2008 to 9/2/2008

WATER OBSERVATIONS:
Water seepage at: 25.0'
Water level at completion: 19.1'

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

Depth (ft)	Elev. (ft) 697.0	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
							Very dense brown COARSE AND FINE SAND (A-3a), some silt, some gravel; wet.								
		18 32 33	18	11			@ 26.0'-27.5', contains turpentine odor.								68
30		7 31 36	15	12											70
		18 23 29	17	13			@ 31.0'-35.0', gray, little silt, little gravel.								55
35.0	687.0	18 28 35	15	14											66
							Bottom of Boring - 35.0'								

40

45

50

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-12.68 - PHASE 4A	DRILLING FIRM / OPERATOR: RII / S.M.	DRILL RIG: CME-750 (SN 98048)	STATION / OFFSET: 5084+74.16 / 15.0' RT
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / K.R.	HAMMER: CME AUTOMATIC	ALIGNMENT: BL RAMP C5
PID: 77372 BR ID: N/A	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 4/26/13	ELEVATION: 732.4 (MSL) EOB: 48.1 ft.
START: 8/6/13 END: 8/6/13	SAMPLING METHOD: SPT	ENERGY RATIO (%): 82.6	LAT / LONG: 39.952844807, -83.003019835

EXPLORATION ID **B-023-1-13**

MATERIAL DESCRIPTION AND NOTES	ELEV. 732.4	DEPTHS
VERY STIFF TO HARD, BROWN **CLAY**, "AND" SILT, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP.	724.4	0 – 8
VERY STIFF TO HARD, BROWN **SILT AND CLAY**, SOME COARSE TO FINE SAND, LITTLE FINE GRAVEL, MOIST. -COBBLES PRESENT @ 9.0' -QU @ 8.3' = 2.95 TSF -CONSOLIDATION TEST PERFORMED @ 8.9'	719.4	8 – 13
STIFF TO VERY STIFF, BROWN **CLAY**, SOME SILT, SOME COARSE TO FINE SAND, SOME FINE GRAVEL, DAMP TO MOIST.	714.4	13 – 18 (W at 18)
DENSE TO VERY DENSE, BROWN TO GRAY **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST TO WET. -COBBLES PRESENT THROUGHOUT		18 –

Depth (ft)	SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
1–2.5	5 7 13	28	53	SS-1	4.5+	-	-	-	-	-	-	-	-	14	A-7-6 (V)
3.5–5	4 4 6	14	58	SS-2	3.50	10	5	11	37	37	42	21	21	16	A-7-6 (13)
6–7.5	6 8 10	25	53	SS-3	4.5+	-	-	-	-	-	-	-	-	16	A-7-6 (V)
8–10			60	ST-4	4.5+	16	11	15	34	24	29	18	11	22	A-6a (5)
11–12.5	4 5 6	15	78	SS-5	3.00	-	-	-	-	-	-	-	-	18	A-6a (V)
13.5–15	4 6 6	17	81	SS-6	2.00	25	13	12	25	25	48	19	29	16	A-7-6 (10)
16–18			98	ST-7	3.00	-	-	-	-	-	-	-	-	21	A-7-6 (V)
18.5–20	4 8 16	33	100	SS-8	-	-	-	-	-	-	-	-	-	18	A-1-b (V)
23.5–25	4 17 20	51	100	SS-9	-	-	-	-	-	-	-	-	-	13	A-1-b (V)
28.5–29.5	22 50/5"	-	0	SS-10	-	-	-	-	-	-	-	-	-	-	
29.5–	45	-	100	SS-10A	-	54	18	10	14	4	25	22	3	12	A-1-b (0)

PID: 77372	BR ID: N/A	PROJECT: FRA-70-12.68 - PHASE 4A	STATION / OFFSET: 5084+74.16 / 15.0 RT	START: 8/6/13	END: 8/6/13	PG 2 OF 2	B-023-1-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 702.4	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
DENSE TO VERY DENSE, BROWN TO GRAY **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST TO WET. *(same as above)* -HEAVING SAND ENCOUNTERED @ 32.0' -COBBLES PRESENT THROUGHOUT		31 W 32 33																
		34 35	17 18 22	55	100	SS-11	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	
	695.4	36 37																
VERY STIFF, DARK GRAY **SILTY CLAY**, SOME COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP.		38																
		39 40	9 6 23	40	100	SS-12	3.50	4	8	20	46	22	36	16	20	11	A-6b (11)	
	690.4	41 42																
VERY DENSE, GRAY **GRAVEL**, LITTLE COARSE TO FINE SAND, TRACE SILT, DAMP.		43																
		44 45	12 30 36	91	78	SS-13	-	-	-	-	-	-	-	-	-	6	A-1-a (V)	
-COBBLES PRESENT @ 46.0'	684.3	46 47																
		EOB 48	50/1"	-	0	SS-14	-	-	-	-	-	-	-	-	-	-		

NOTES: SEEPAGE ENCOUNTERED @ 18.5'; GROUNDWATER ENCOUNTERED INITIALLY @ 32.0' AND AT COMPLETION @ 18.0'

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

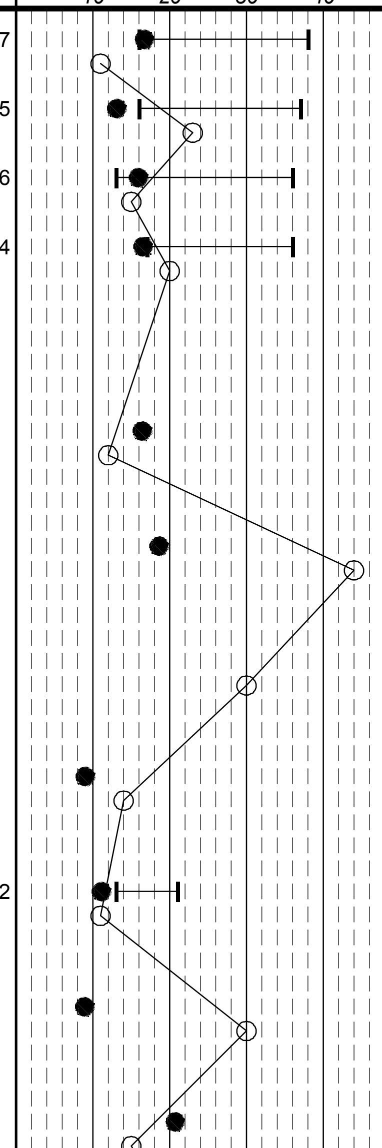
Client: ms consultants	Project: FRA-70-8.93	Job No. 0221-1004.01
LOG OF: Boring B-024-0-08	Location: Sta. 5085+90.21, 3.1' LT., BL RAMP C5	Date Drilled: 7/1/2008 to 7/2/2008

WATER OBSERVATIONS:
Water seepage at: 28.5'-80.0'
Water level at completion: 16.5' (prior to coring)
10.7' (includes drilling water)

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
	743.4	3 4 7	11	1		--	Stiff to very stiff brown SILTY CLAY (A-6b), trace to little fine to coarse sand, little gravel; damp to moist.		11	9	---	11	32	37
		7 12 11	12	2		4.5+	@ 0.0 - 3.0', contains roots. @ 1.5', hard.		15	8	---	10	32	35
		5 7 8	16	3		4.0	@ 3.0', very stiff to hard.		11	8	---	12	33	36
5		4 6 14	13	4		3.75			12	10	---	13	31	34
10		4 4 8	15	5		4.5								
		14 21 22	9	6		--	@ 11.0', encountered rock fragments.							
15		17 14 15	1	7		--								
15.5	727.9						Very stiff brownish gray SANDY SILT (A-4a), little to some fine to coarse sand, trace gravel; damp to moist.							
		4 7 7	18	8		3.5								
20		2 4 7	18	9		3.0			9	15	---	19	35	22
		7 13 16	12	10		--	@ 21.5', encountered possible large rock fragments, cobbles.							
25	718.4	3 7 8	18	11		3.25								

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

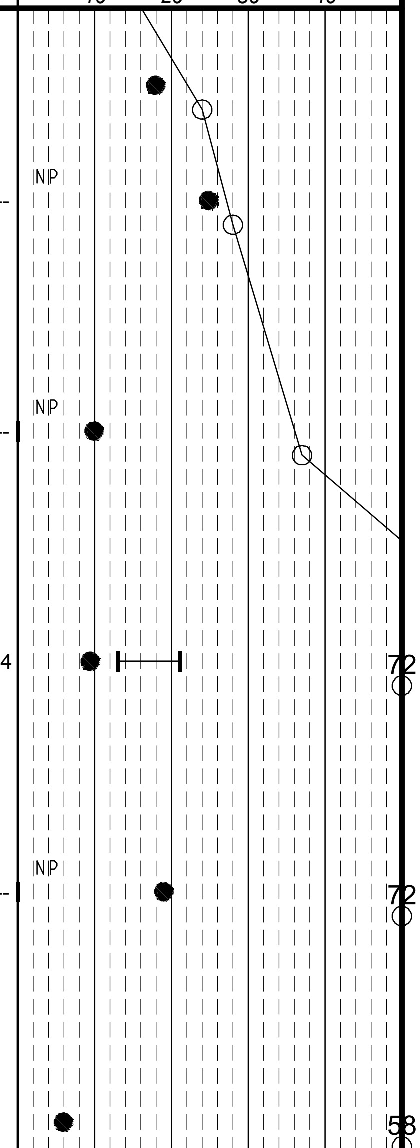
Client: ms consultants	*Project:* FRA-70-8.93	*Job No.* 0221-1004.01
LOG OF: Boring B-024-0-08	*Location:* Sta. 5085+90.21, 3.1' LT., BL RAMP C5	*Date Drilled:* 7/1/2008 to 7/2/2008

WATER OBSERVATIONS:
Water seepage at: 28.5'-80.0'
Water level at completion: 16.5' (prior to coring); 10.7' (includes drilling water)

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

Depth (ft)	Elev. (ft) 718.4	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
		8 10 14	18	12		--	Very stiff brownish gray SANDY SILT (A-4a), little to some fine to coarse sand, trace gravel; damp to moist.						
28.0	715.4												
30		3 12 15	16	13			Medium dense brownish gray SILT (A-4b), little fine sand; wet.	0	0	---	17	--83--	
32.0	711.4												
35		7 18 18	11	14			Dense brown GRAVEL (A-1-a), some fine to coarse sand, trace to little silty clay; wet.	60	22	---	8	--10--	
37.0	706.4												
40		13 26 45	15	15		--	Hard brownish gray SANDY SILT (A-4a), little gravel, trace clay; damp.	12	14	---	27	33	14
42.0	701.4												
45		17 26 45	18	16			Very dense brownish gray COARSE AND FINE SAND (A-3a), little silty clay, little gravel; wet. @ 43.5', 2.0 feet sand heave.	16	25	---	47	--12--	
47.0	696.4												
50	693.4	26 30 27	13	17			Very dense brownish gray SANDY SILT (A-4a), little gravel; wet.						

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



Client: ms consultants	Project: FRA-70-8.93	Job No. 0221-1004.01
LOG OF: Boring B-024-0-08	Location: Sta. 5085+90.21, 3.1' LT., BL RAMP C5	Date Drilled: 7/1/2008 to 7/2/2008

WATER OBSERVATIONS:
Water seepage at: 28.5'-80.0'
Water level at completion: 16.5' (prior to coring)
10.7' (includes drilling water)

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

Depth (ft)	Elev. (ft) 693.4	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N60) Blows per foot
							Very dense brownish gray GRAVEL WITH SAND (A-1-b), some silty clay; wet.							
55		15 40 50/4	13	18			@ 50.0'-60.0', difficulty advancing boring due to obstruction inside augers blocking rods; possible boulder zone.	30	31	---	16	15	8	50+
57.0	686.4						Very dense brownish gray SANDY SILT (A-4a), some gravel, some fine to coarse sand; damp.							
60		37 21 50	13	19				30	19	---	10	28	13	72
62.0	681.4						Very dense brown and gray GRAVEL (A-1-a), some fine to coarse sand, trace silt; wet.							
65		11 50/4	10	20			@ 63.5', one foot sand heave; encountered black shale fragments.	73	17	---	5	--5--		NP 50+
70		50/3	0	21			@ 68.5', possible cobbles or boulders.							50+
75	668.4	50 50/3	6	22			@ 73.5', 6.0 feet sand heave; washed out with tricone.							50+

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

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

LOG OF: Boring B-024-0-08 | *Location:* Sta. 5085+90.21, 3.1' LT., BL RAMP C5 | *Date Drilled:* 7/1/2008 to 7/2/2008

WATER OBSERVATIONS:
Water seepage at: 28.5'-80.0'
Water level at completion: 16.5' (prior to coring)
10.7' (includes drilling water)

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

Depth (ft)	Elev. (ft) 668.4	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	Graphic Log	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N60) Natural Moisture Content, % - ● PL ⊢—⊣ LL Blows per foot - ○ / Non-Plastic - NP 10 20 30 40
							Very dense brown and gray GRAVEL (A-1-a), some fine to coarse sand, trace silt; wet.								
		39 50/3	9	23			@ 78.5', 3.0 feet sand heave.		55	36	---	5	--4--		NP ● 50+
80															
							@ 80.0'-90.0', difficult drilling; possible cobbles.								
		50/0	0	24											50+
85															
		50/0	0	25			@ 88.5', 1.0 foot sand heave.								50+
90							@ 90.0' - 91.5', core loss.								
		Core 18"	Rec 0"	RQD 0%	R1		@ 91.5' - 91.8', encountered igneous-plutonic cobble/boulder; likely peridotite or gabbro.								
91.8	651.6						Interbedded Shale (90%) and Limestone (10%) RQD 60%, LOSS 26%; Shale, blue-gray, highly weathered, weak, laminated, slightly calcareous, contains abundant pyritic inclusions, moderately to highly fractured; Limestone, light gray, moderately weathered, moderately strong to strong, fractured.								
95															
		Core 120"	Rec 116"	RQD 81%	R2										
							@ 97.7' - 98.1', qu = 1650 psi								
100	643.4														

Client: ms consultants	Project: FRA-70-8.93	Job No. 0221-1004.01
LOG OF: **Boring B-024-0-08**	Location: Sta. 5085+90.21, 3.1' LT., BL RAMP C5	Date Drilled: 7/1/2008 to 7/2/2008

WATER OBSERVATIONS:
Water seepage at: 28.5'-80.0'
Water level at completion: 16.5' (prior to coring)
10.7' (includes drilling water)
FIELD NOTES:
Advanced boring using 3.25" diameter hollowstem augers.

Depth (ft)	Elev. (ft) 643.4	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
							Interbedded Shale (90%) and Limestone (10%) RQD 60%, LOSS 26%; Shale, blue-gray, highly weathered, weak, laminated, slightly calcareous, contains abundant pyritic inclusions, moderately to highly fractured; Limestone, light gray, moderately weathered, moderately strong to strong, fractured. @ 101.2', dark gray (shale).								
105		Core 120"	Rec 76"	RQD 48%	R3		@ 107.8' - 111.5', core loss due to pyritic inclusion lodged in core barrel.								
110															
111.5	631.9						Bottom of Boring - 111.5'								
115															
120															
125															

APPENDIX IV

HISTORIC BORING LOGS:

B-001-A-57 and B-003-A-57

B-001-A-59

STATE OF OHIO
DEPARTMENT OF HIGHWAYS
TESTING LABORATORY

SHEET 4

LOG OF BORING

CO., RT. NO., SEC. FRA-40-12.82 RETAINING WALL-A   BRIDGE NO. SOUTH-EAST INNERBELT

LOCATION: T.H.1-AB-A STA. 51+16 OFFSET 72' RT FED. NO.

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
714.5	0			
	2			
	4			
709.5	6	2/3	19687	Brown Silty Gravelly Sand
707.0	8	3/5	19688	Dk.Gr.Gravelly Sand Silt W/Coal Fragments
704.5	10	3/6	19689	Dk.Gr.Gravelly Sandy Silt
702.0	12	3/7	19690	Gr.Gravelly Sandy Silt Trace of Organic
699.5	14 / 16	4/9	19691	Dk.Gray Silty Sandy Gravel
697.0	18	10/13	19692	Brown Gravel W/Limestone Fragments
694.5	20	8/14	19693	Brown Sandy Gravel
692.0	22 / 24	12/18	19694	Brown Silty Sandy Gravel
689.5	26	10/16	19695	Brown Silty Sandy Gravel
	28			
684.5	30 / 32	19/32	19696	Gray Silty Gravelly Sand
	34			
679.5	36	30/55	19697	Gray Sandy Gravel

Fill (0–18 ft)

Boulders (18–36 ft)

MB
8-26-59

LOG OF BORING (CONTINUED)

SHEET 5

BRIDGE NO. ______ T.H. 1-A B

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION	
	38				Boulders
674.5	40				
		45/75	19698	Gray Silty Sandy Gravel	
	42				
	44				
669.5	46	37/95	19699	Gray Silty Sandy Gravel	
	48				
664.5	50				
663.5		31/40	19700	Gray Sandy Silt	
				BOTTOM OF BORING	
	52				
	54				
	56				
	58				
	60				
	62				
	64				
	66				
	68				
	70				
	72				
	74				
	76				
	78				
	80				
	82				

B-003-A-59

STATE OF OHIO
DEPARTMENT OF HIGHWAYS
TESTING LABORATORY

SHEET 6

LOG OF BORING

CO., RT. NO., SEC. FRA-40-12.82 BRIDGE NO.
RETAINING WALL-A SOUTH-EAST INNERBELT

LOCATION: T.H. 3-AB STA. 54+68 OFFSET 60'RT FED. NO.

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
735.5	0			
	2			
730.5	4			
	6	19/14	19701	Gray Silty Sandy Gravel
728.0	8	8/9	19702	Gray Sandy Gravelly Silt
725.5	10			
	12	9/9	19703	Gray Gravelly Sandy Silt
723.0	14	7/10	19704	Gray Silt and Clay
720.5	16	9/10	19705	Gray Silt
718.0	18	13/15	19706	Gray Silty Sand
715.5	20			
	22	16/22	19707	Gray Sandy Silt
713.0	24	42/36	19708	Gray Silty Sandy Gravel W/Boulders
710.5	26	25/25	19709	Gray Gravel W/Boulders
708.0	28	20/46	19710	Brown Sandy Gravel W/Boulders
705.5	30			
	32	40/75	19711	Brown Sandy Gravel W/Boulders
	34			
700.5	36	75/*	19712	Gray Silty Sand W/Boulders

*Refusal

MB
8.26.59

B-003-A-59

LOG OF BORING (CONTINUED)

SHEET 7

BRIDGE NO. ______ T.H. 3-A B

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
	38			
695.5	40			
	42	38/34	19713	Gray Sandy Silt
	44			
690.5	46	27/42	19714	Gray Silty Sandy Gravel
	48			
685.5	50			
	52	25/37	19715	Gray Silt W/Boulders
683.0				
	54	75/*	-----	Bouldery Gray Sand
680.5				BOTTOM OF BORING
	56			
	58			
	60			*Refusal
	62			
	64			
	66			
	68			
	70			
	72			
	74			
	76			
	78			
	80			
	82			

APPENDIX V

LABORATORY TEST RESULTS

UNCONFINED COMPRESSION

ASTM D -2166

6350 Presidential Gateway
Columbus, Ohio 43231
Telephone: (614) 823-4949
Fax Number: (614) 823-4990

PROJECT	FRA-70-12.68
JOB No.	W-13-045
BORING	B-023-1-13
STATION / OFFSET	5084+74.16 / 15.0' Rt.
SAMPLE No. / DEPTH	ST-4 / 8.3 ft
DATE OF TESTING	8/14/2013
TESTED BY	JJH

Soil Description: Brown SILT AND CLAY, some coarse to fine sand, little fine gravel.
Soil Classification: ODOT A-6a

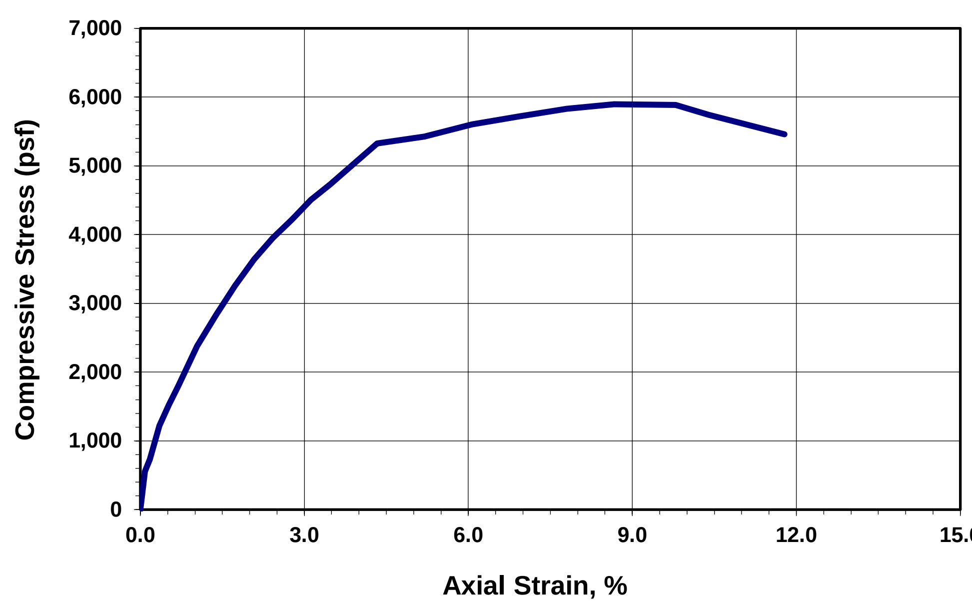
Physical Characteristics	L.L.	P.L.	P.I.	Gravel%	C. Sand%	F. Sand%	Silt%	Clay%
	29	18	11	16	11	15	34	24

DIAMETER, D_0	2.87 in	72.898 mm	STRAIN RATE	1.00	%/min
AREA, A_0	6.47 in^2	41.7 cm^2	WET SOIL + PAN MASS	1384.2	g
HEIGHT, L_0	5.77 in	146.58 mm	PAN MASS	90.2	g
VOLUME, V_0	37.33 in^3	611.8 cm^3	DRY SOIL + PAN MASS	1197.9	g
MACH. RATE	0.577	in/min	WET DENSITY	132.04	lb/ft^3
WATER CONT.	16.82	%	DRY DENSITY	113.03	lb/ft^3

UNCONFINED COMPRESSION STRESS, q_u	**5,896** **psf**	2.95	tsf
AXIAL STRAIN @ FAILURE		8.66	%
HAND PENETROMETER		4.5+	tsf

Failure Sketch

Unconfined Compression Test



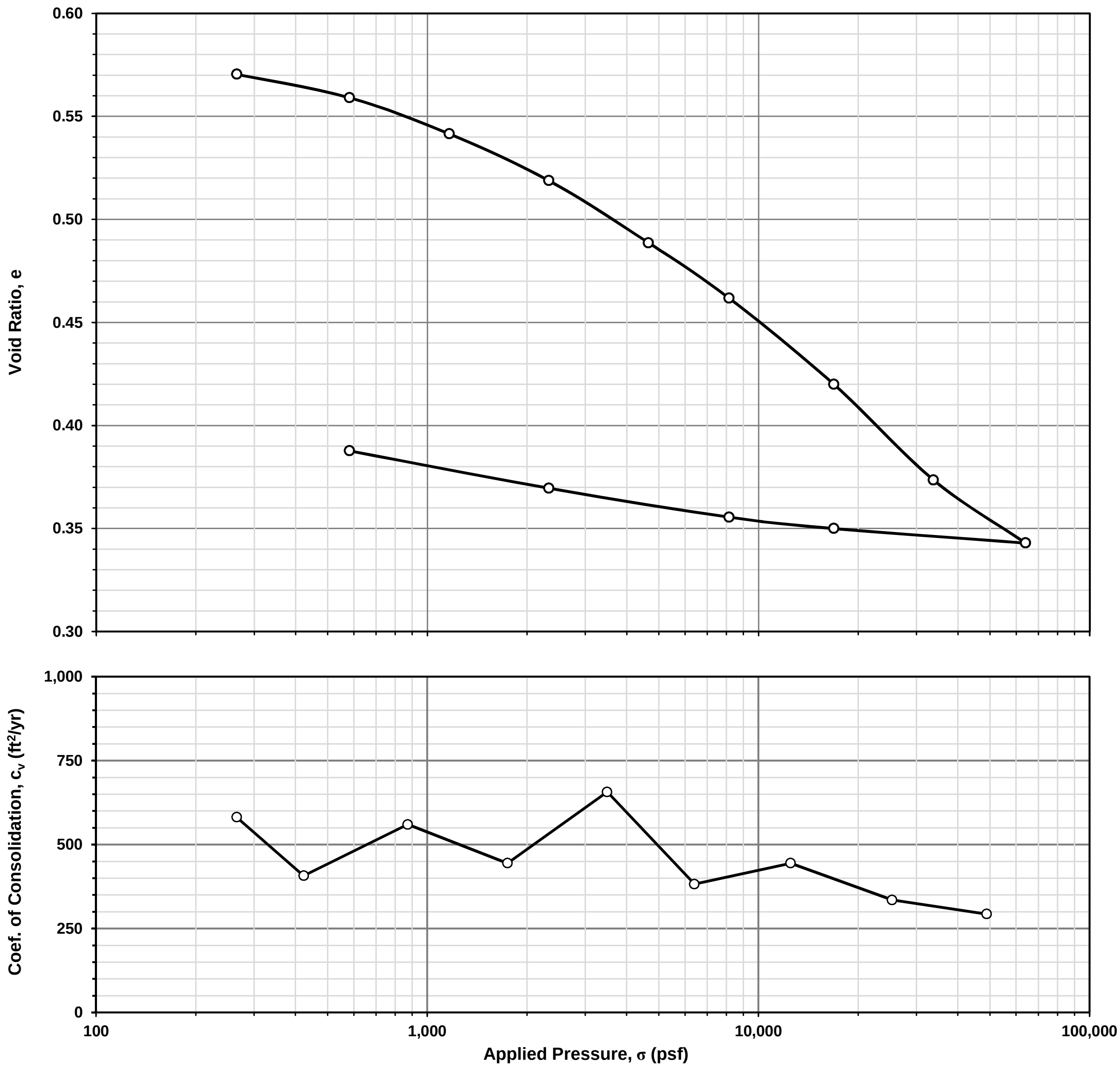
One-Dimensional Consolidation Test Report (ASTM D2435)

Project Number:	W-13-045	Boring Number:	B-020-2-13
Project Name:	FRA-70-12.68	Station / Offset:	176+13.62, 34.0' Rt.
Project Location:	Columbus, Ohio	Sample No. / Depth:	ST-6 / 14.7 ft
Client:	GPD GROUP	Date of Testing:	08/13/2013 to 08/30/2013

Soil Description: Brown CLAY, and coarse to fine sand, some silt, little fine gravel.
Soil Classification: ODOT A-7-6

Physical Characteristics	L.L.	P.L.	P.I.	Gravel%	C. Sand%	F. Sand%	Silt%	Clay%
	41	16	25	17	31	13	24	15

Natural		γ_d (pcf)	γ_{sat} (pcf)	σ_{vo}' (psf)	S_G	e_o	σ_p' (psf)	c_c	c_r
S_o	w_o								
101.6%	19.9%	105.6	128.9	1,617	2.67	0.578	2,470	0.154	0.022



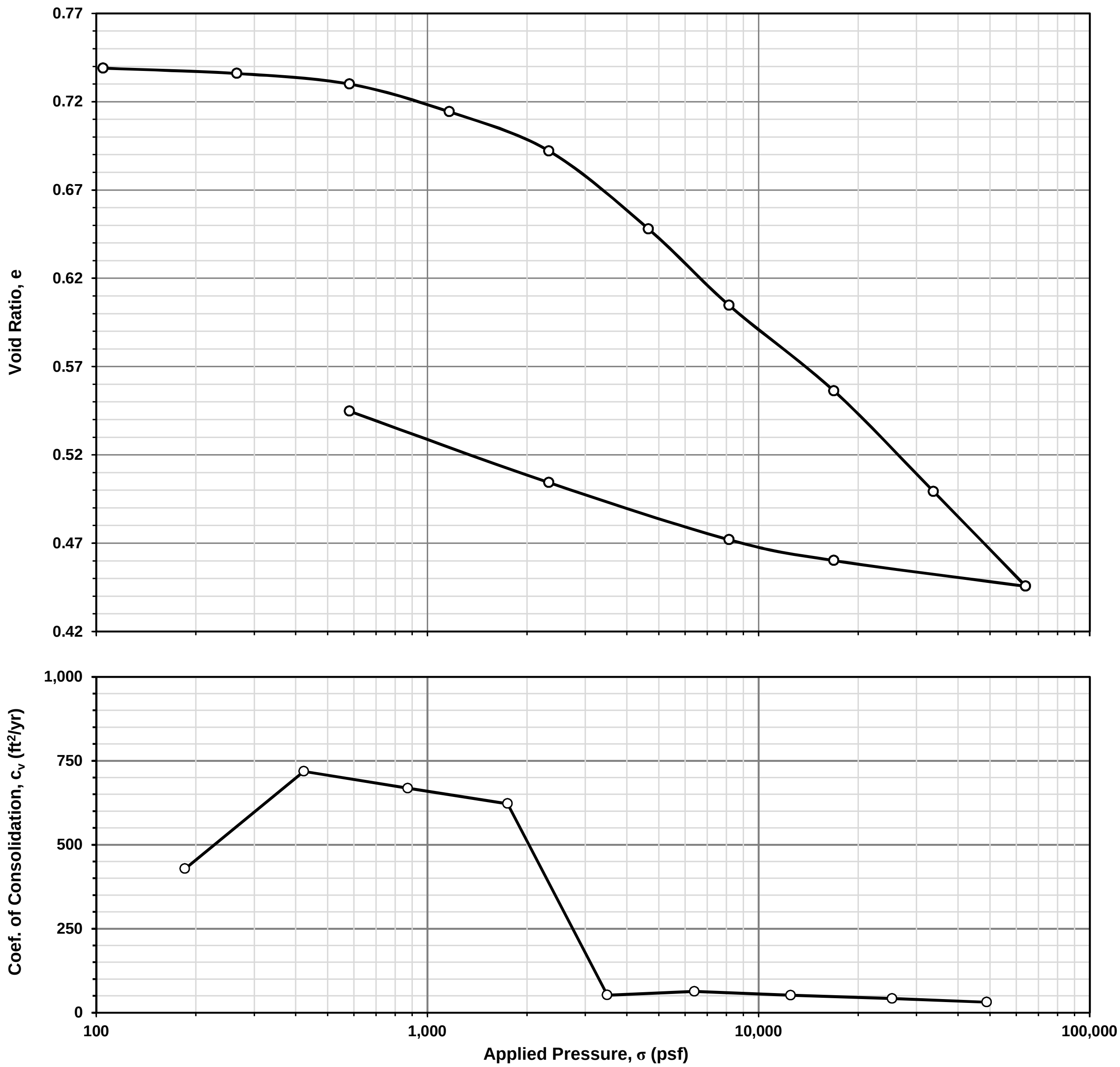
One-Dimensional Consolidation Test Report (ASTM D2435)

Project Number:	W-13-072	Boring Number:	B-020-7-13
Project Name:	FRA-70-13.10	Station / Offset:	176+68.64, 1.8' Rt.
Project Location:	Columbus, Ohio	Sample No. / Depth:	ST-4 / 11.8 ft
Client:	ms consultants, inc.	Date of Testing:	01/27/2015 to 02/12/2015

Soil Description: Dark brown CLAY, "and" silt, trace coarse to fine sand
Soil Classification: ODOT A-7-6

Physical Characteristics	L.L.	P.L.	P.I.	Gravel%	C. Sand%	F. Sand%	Silt%	Clay%
	43	19	24	0	2	7	45	46

Natural		γ_d	γ_{sat}	σ_{vo}'	S_G	e_o	σ_p'	c_c	c_r
S_o	w_o	(pcf)	(pcf)	(psf)			(psf)		
99.6%	23.3%	95.5	122.0	1,357	2.67	0.745	3,449	0.210	0.049



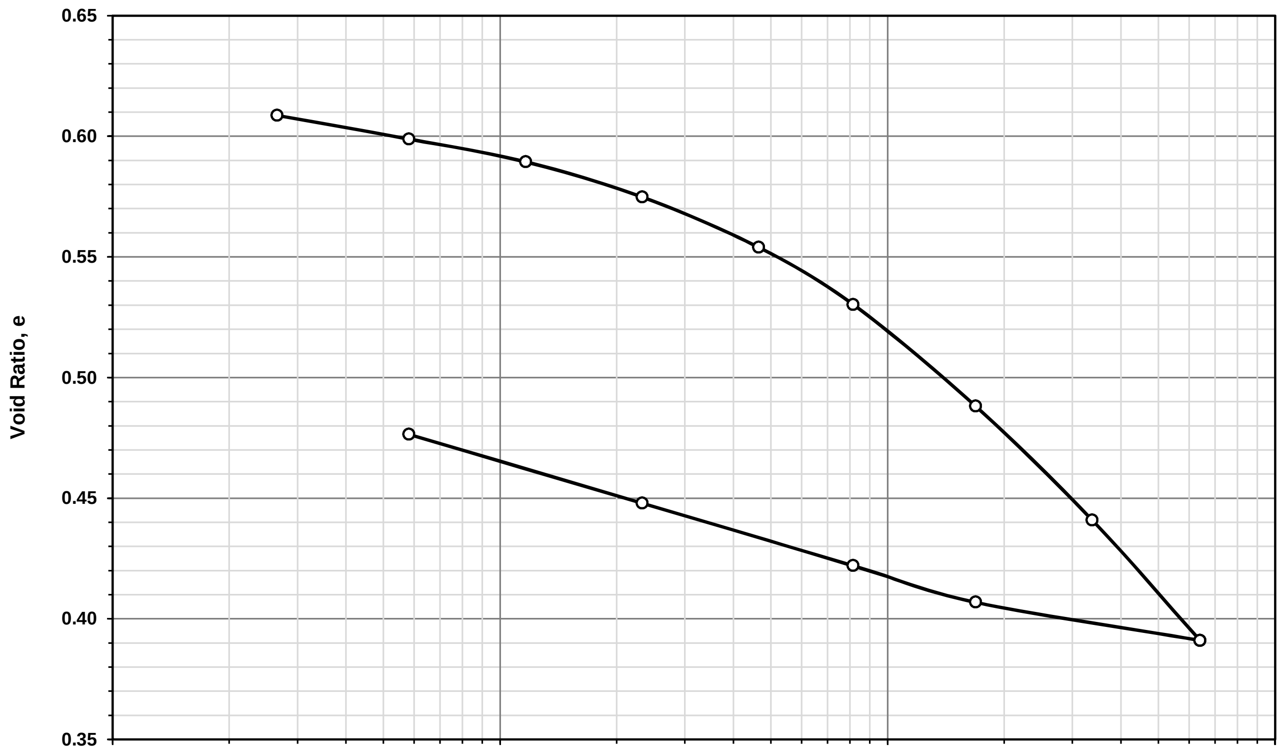
One-Dimensional Consolidation Test Report (ASTM D2435)

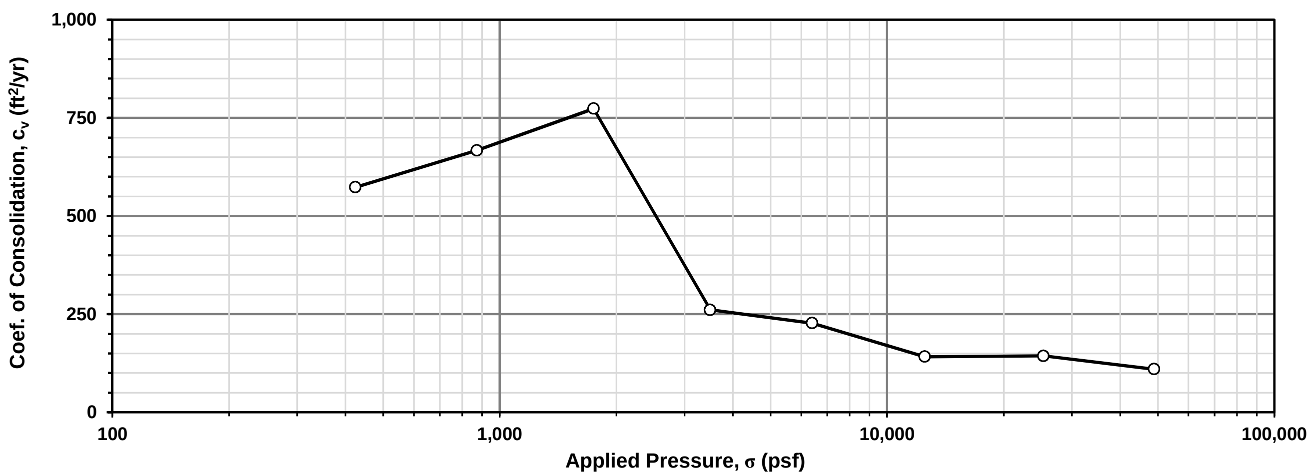
Project Number:	W-13-045	Boring Number:	B-023-1-13
Project Name:	FRA-70-12.68	Station / Offset:	5084+74.16, 15.0' Rt.
Project Location:	Columbus, Ohio	Sample No. / Depth:	ST-4 / 8.9 ft
Client:	GPD GROUP	Date of Testing:	08/21/2013 to 09/11/2013

Soil Description: Reddish brown SILT AND CLAY, some coarse to fine sand, little fine gravel.
Soil Classification: ODOT A-6a

Physical Characteristics	L.L.	P.L.	P.I.	Gravel%	C. Sand%	F. Sand%	Silt%	Clay%
	29	18	11	16	11	15	34	24

Natural		γ_d (pcf)	γ_{sat} (pcf)	σ_{vo}' (psf)	S_G	e_o	σ_p' (psf)	c_c	c_r
S_o	w_o								
89.5%	22.1%	102.6	124.1	1,068	2.67	0.624	7,680	0.191	0.043





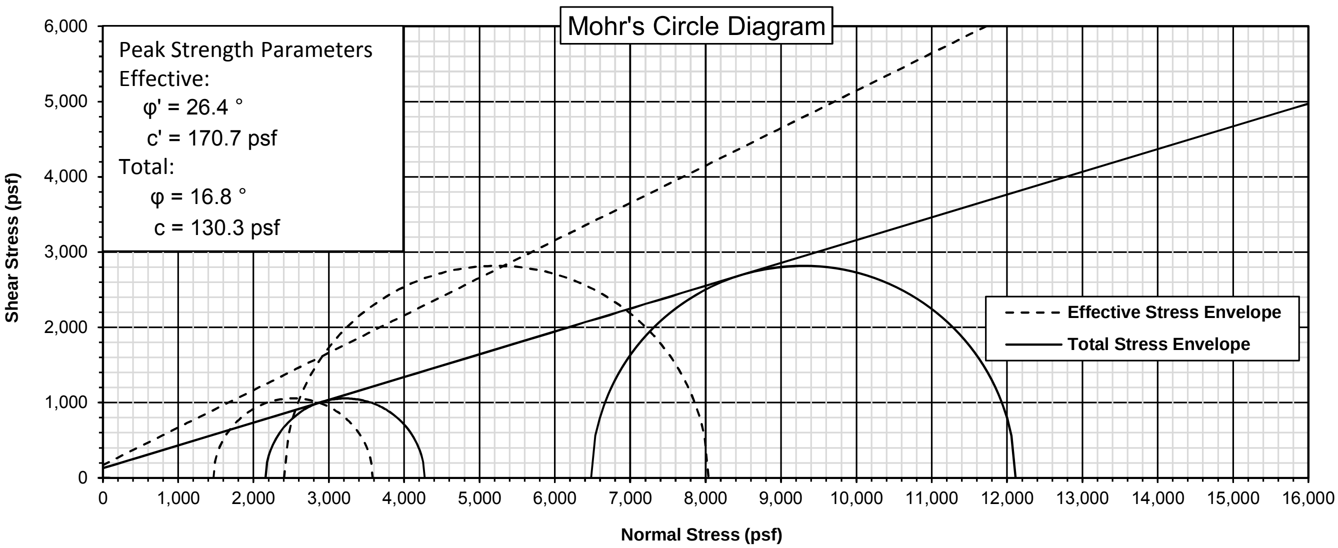
Consolidated, Undrained Triaxial Compression Test Report (ASTM D4767)

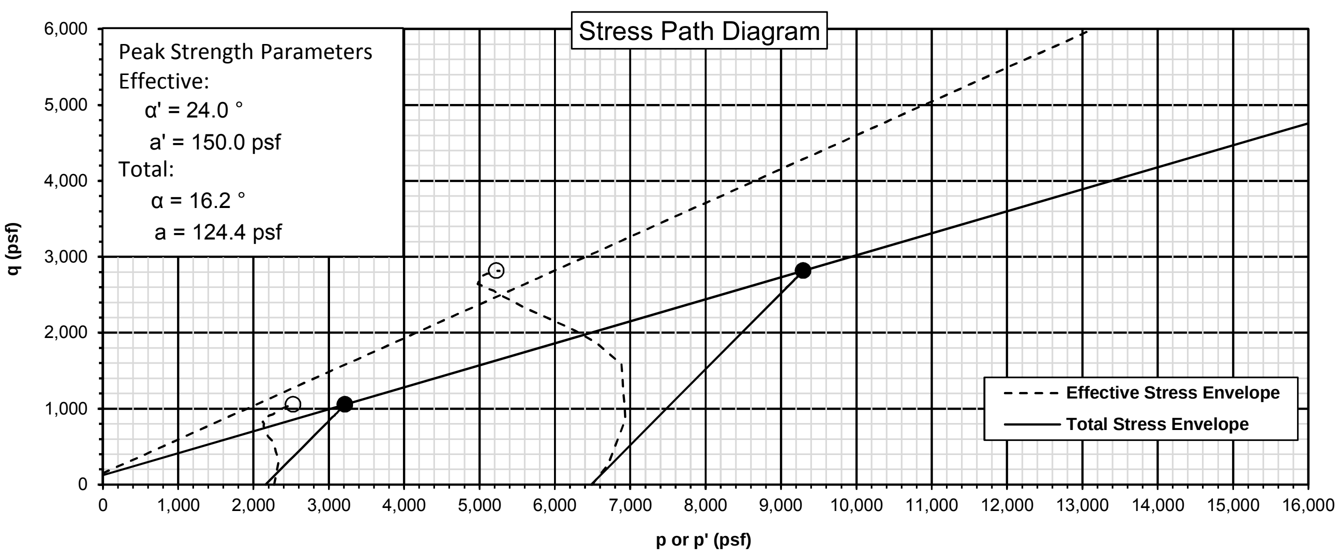
Project Number:	W-13-045	Boring Number:	B-020-2-13
Project Name:	FRA-70-12.68	Station / Offset:	176+13.92, 34.0' Rt.
Project Location:	Columbus, Ohio	Sample No. / Depth:	ST-6 / 13.5 ft to 14.0 ft
Client:	GPD GROUP	Date of Testing:	08/14/2013 to 08/21/2013

Soil Description: Brown CLAY, "and" coarse to fine sand, some silt, little fine gravel.
Soil Classification: ODOT A-7-6

Physical Characteristics	L.L.	P.L.	P.I.	Gravel%	C. Sand%	F. Sand%	Silt%	Clay%
	41	16	25	17	31	13	24	15

Stage	Boring No.	Sample No.	Depth (ft)	$(\sigma_3)_f$ (psf)	$(\sigma_1)_f$ (psf)	$(\sigma_3')_f$ (psf)	$(\sigma_1')_f$ (psf)	p'_f (psf)	q_f (psf)
1	B-020-2-13	ST-6	13.5	2,160.0	4,271.8	1,468.8	3,580.6	2,524.7	1,055.9
2	B-020-2-13	ST-6	14	6,480.0	12,114.6	2,404.8	8,039.4	5,222.1	2,817.3
3									





Notes:

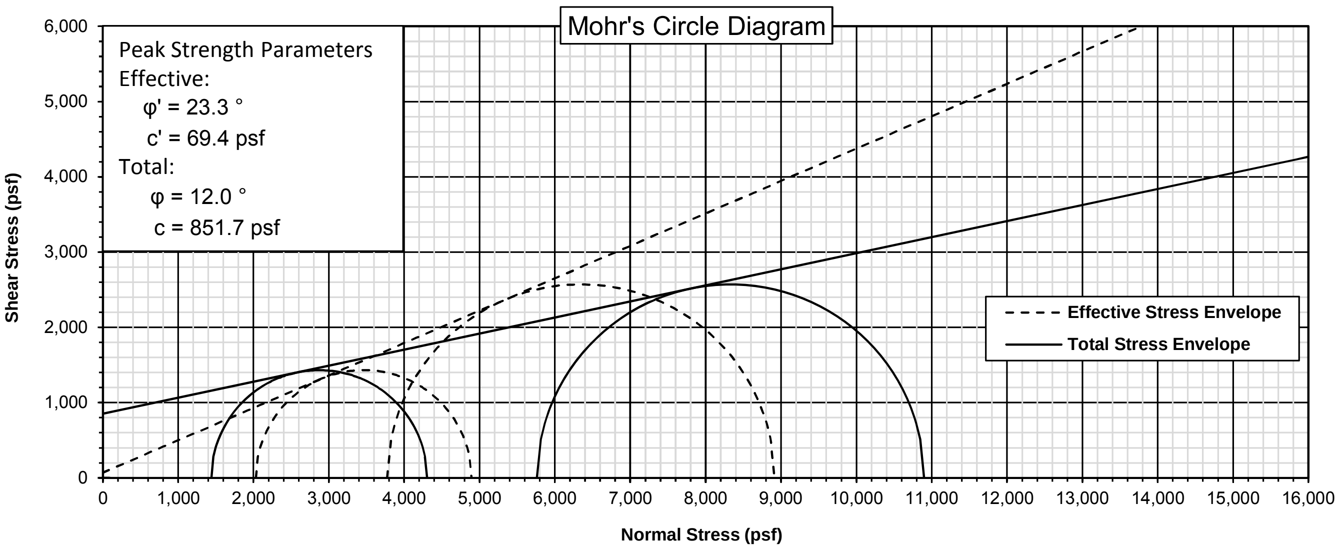
Consolidated, Undrained Triaxial Compression Test Report (ASTM D4767)

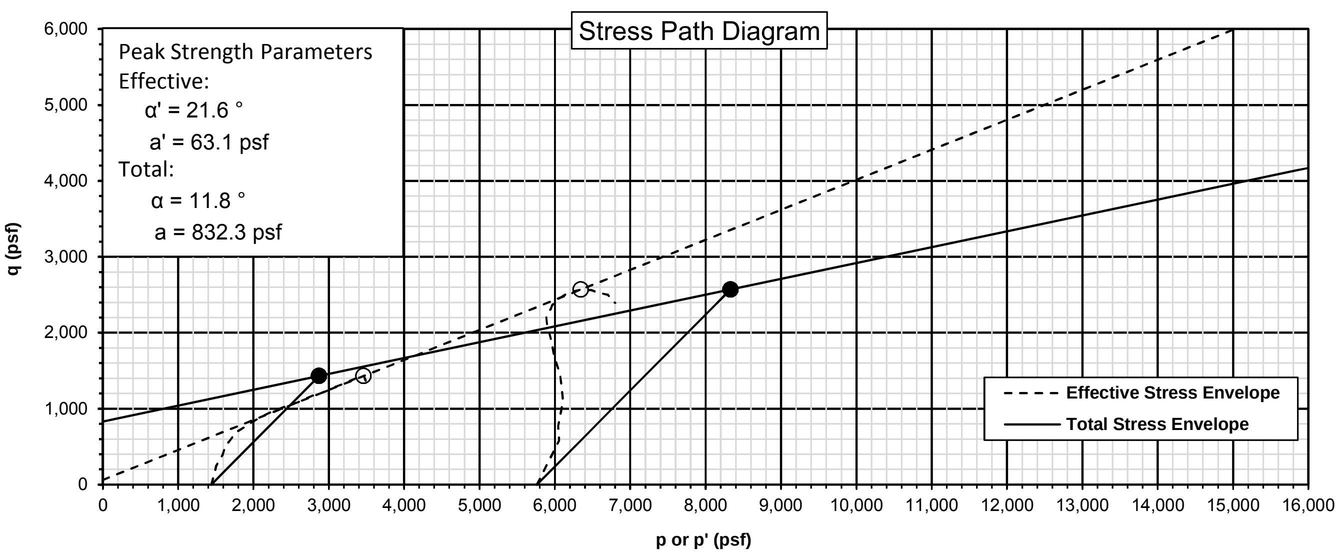
Project Number:	W-13-072	Boring Number:	B-020-7-13
Project Name:	FRA-70-13.10	Station / Offset:	176+68.64, 1.8' Rt.
Project Location:	Franklin County, Ohio	Sample No. / Depth:	ST-4 / 12.0 ft to 13.0 ft
Client:	ms consultants	Date of Testing:	01/28/2015 to 02/10/2015

Soil Description: Dark brown CLAY, "and" silt, trace coarse to fine sand
Soil Classification: ODOT A-7-6

Physical Characteristics	L.L.	P.L.	P.I.	Gravel%	C. Sand%	F. Sand%	Silt%	Clay%
	43	19	24	0	2	7	45	46

Stage	Boring No.	Sample No.	Depth (ft)	$(\sigma_3)_f$ (psf)	$(\sigma_1)_f$ (psf)	$(\sigma_3')_f$ (psf)	$(\sigma_1')_f$ (psf)	p'_f (psf)	q_f (psf)
1	B-020-7-13	ST-4	12.0-12.5	1,440.0	4,302.7	2,030.4	4,893.1	3,461.8	1,431.4
2	B-020-7-13	ST-4	12.5-13.0	5,760.0	10,900.3	3,772.8	8,913.1	6,343.0	2,570.2
3									





Notes:

Consolidated, Undrained Triaxial Compression Test (ASTM D4767)

Project Number:	W-13-072	Boring Number:	B-020-7-13
Project Name:	FRA-70-13.10	Station / Offset:	176+68.64, 1.8' Rt.
Project Location:	Franklin County, Ohio	Sample No. / Depth:	ST-4 / 12.0-12.5 ft
Client:	ms consultants	Date of Testing:	2/10/2015

Data for Specimen No. 1

Soil Description: Dark brown CLAY, "and" silt, trace coarse to fine sand
Soil Classification: ODOT A-7-6

Physical Characteristics	L.L.	P.L.	P.I.	Gravel%	C. Sand%	F. Sand%	Silt%	Clay%
	43	19	24	0	2	7	45	46

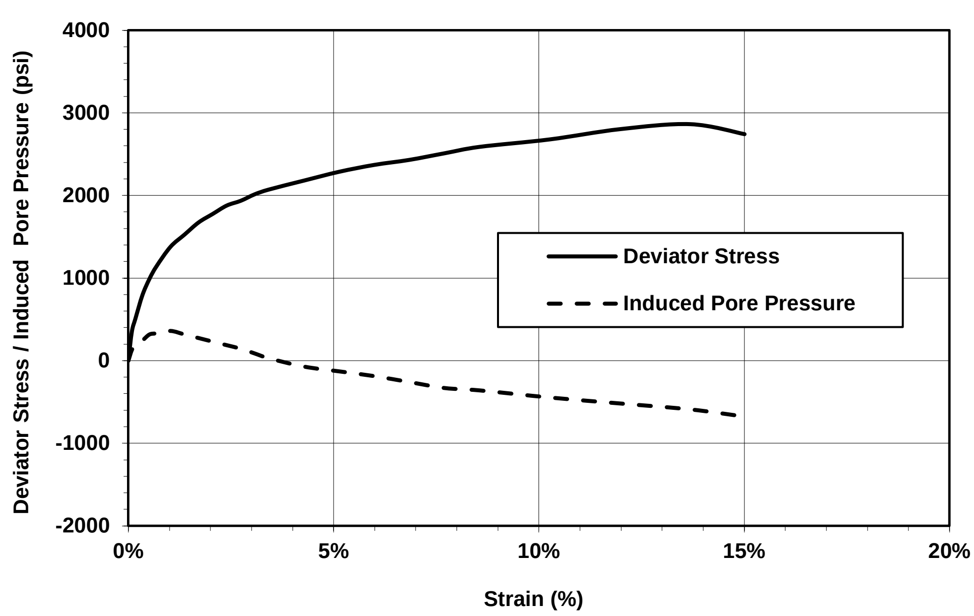
Diameter, D_0	2.872	in	Volume of Solids , V_s :	23.242	in^3
Area, A_0	6.478	in^2	Initial Volume of Voids, V_v :	15.246	in^3
Height, L_0	5.941	in	Initial Void Ratio, e_o :	0.656	
Volume, V_0	38.487	in^3	Initial Degree of Saturation, S_0 :	94.7	%

Water Content BEFORE Test			Water Content AFTER Test (Total Specimen)		
Tin No.:	X-16	g	Tin No.:	FUNKY	g
Wet Soil + Tin :	113.18	g	Wet Soil + Tin :	1316.50	g
Dry Soil + Tin :	97.47	g	Dry Soil + Tin :	1073.70	g
Tin Weight :	29.97	g	Tin Weight :	56.80	g
Dry Mass :	67.5	g	Dry Mass :	1016.90	g
Weight of water :	15.71	g	Weight of water :	242.80	g
Moisture :	23.27	%	Moisture :	23.88	%
			Wet Density :	124.69	pcf
			Dry Density :	100.65	pcf

Consolidation Cell Pressure:	140.0	psi	Deviator Stress @ Failure, Ds:	2,863	psf
Consolidation Back Pressure:	130.0	psi	Axial Strain @ Failure:	13.7	%
Effective Confining Stress, σ_3:	10.0	psi	Major Principal Stress @ Failure, σ_1:	4,303	psf
	1,440	psf	Induced Pore Pressure @ Failure:	-590	psf
Strain Rate:	0.0030	in/min	Effective Minor Principal Stress,σ'_3:	2,030	psf
			Effective Major Principal Stress, σ'_1:	4,893	psf

Failure Sketch

CU Compressive Strength and Induced Pore Pressure



Notes:

Consolidated, Undrained Triaxial Compression Test (ASTM D4767)

Project Number:	W-13-072	Boring Number:	B-020-7-13
Project Name:	FRA-70-13.10	Station / Offset:	176+68.64, 1.8' Rt.
Project Location:	Franklin County, Ohio	Sample No. / Depth:	ST-4 / 12.5-13.0 ft
Client:	ms consultants	Date of Testing:	10/11/2014

Data for Specimen No. 2

Soil Description: Dark brown CLAY, "and" silt, trace coarse to fine sand
Soil Classification: ODOT A-7-6

Physical Characteristics	L.L.	P.L.	P.I.	Gravel%	C. Sand%	F. Sand%	Silt%	Clay%
	43	19	24	0	2	7	45	46

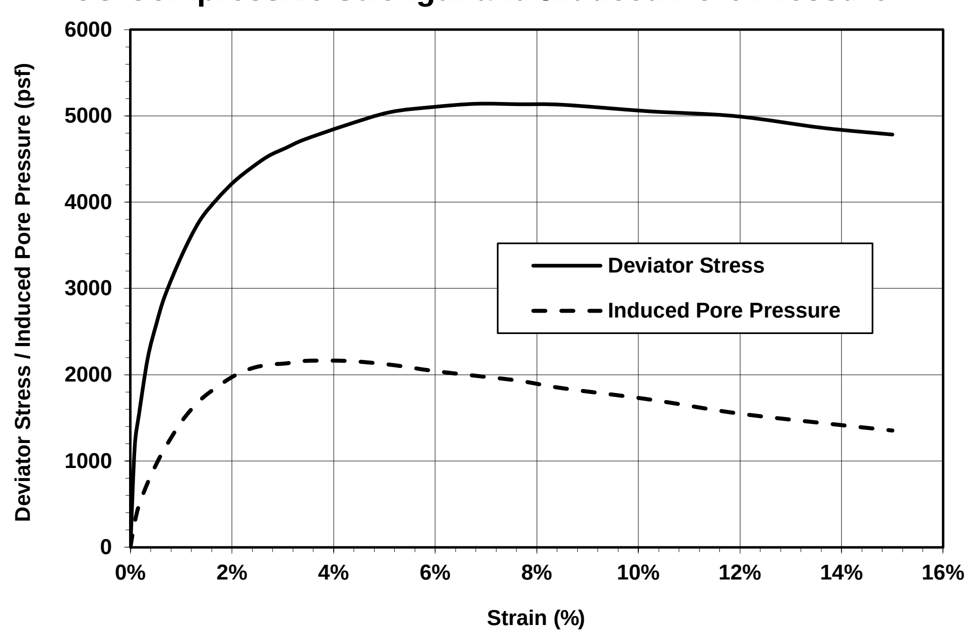
Diameter, D_0	2.875	in	Volume of Solids , V_s :	23.795	in^3
Area, A_0	6.492	in^2	Initial Volume of Voids, V_v :	14.946	in^3
Height, L_0	5.968	in	Initial Void Ratio, e_o :	0.628	
Volume, V_0	38.741	in^3	Initial Degree of Saturation, S_0 :	98.93	%

Water Content BEFORE Test			Water Content AFTER Test (Total Specimen)		
Tin No.:	X-16	g	Tin No.:	FUNKY	g
Wet Soil + Tin :	113.18	g	Wet Soil + Tin :	1325.30	g
Dry Soil + Tin :	97.47	g	Dry Soil + Tin :	1097.20	g
Tin Weight :	29.97	g	Tin Weight :	56.10	g
Dry Mass :	67.5	g	Dry Mass :	1041.10	g
Weight of water :	15.71	g	Weight of water :	228.10	g
Moisture :	23.27	%	Moisture :	21.91	%
			Wet Density :	124.80	pcf
			Dry Density :	102.37	pcf

Consolidation Cell Pressure:	143.0	psi	Deviator Stress @ Failure, Ds:	5,140	psf
Consolidation Back Pressure:	103.0	psi	Axial Strain @ Failure:	6.8	%
Effective Confining Stress, σ_3:	40.0	psi	Major Principal Stress @ Failure, σ_1:	10,900	psf
	5,760	psf	Induced Pore Pressure @ Failure:	1,987	psf
Strain Rate:	0.0030	in/min	Effective Minor Principal Stress,σ'_3:	3,773	psf
			Effective Major Principal Stress, σ'_1:	8,913	psf

Failure Sketch

CU Compressive Strength and Induced Pore Pressure



Notes:

Consolidated, Undrained Triaxial Compression Test (ASTM D4767)

Project Number:	W-13-045	Boring Number:	B-020-2-13
Project Name:	FRA-70-12.68	Station / Offset:	176+13.92, 34.0' Rt.
Project Location:	Columbus, Ohio	Sample No. / Depth:	ST-6 / 13.5 ft
Client:	GPD GROUP	Date of Testing:	6/21/2013

Data for Specimen No. 1

Soil Description: Brown CLAY, "and" coarse to fine sand, some silt, little fine gravel.

Soil Classification: ODOT A-7-6

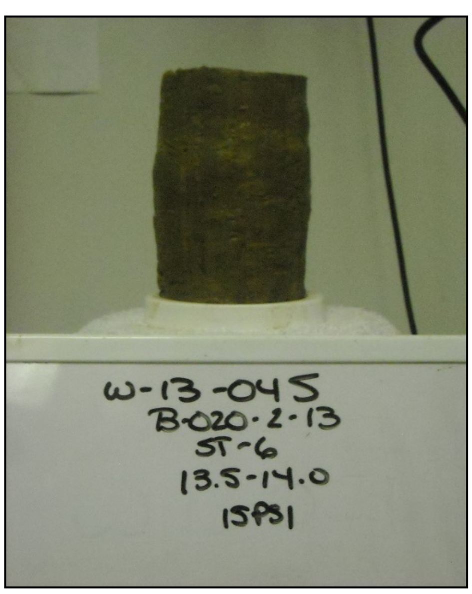
Physical Characteristics	L.L.	P.L.	P.I.	Gravel%	C. Sand%	F. Sand%	Silt%	Clay%
	41	16	25	17	31	13	24	15

Diameter, D_0	2.854	in	Volume of Solids , V_s :	21.813	in^3
Area, A_0	6.396	in^2	Initial Volume of Voids, V_v :	14.805	in^3
Height, L_0	5.725	in	Initial Void Ratio, e_o :	0.679	
Volume, V_0	36.618	in^3	Initial Degree of Saturation, S_0 :	88.2	%

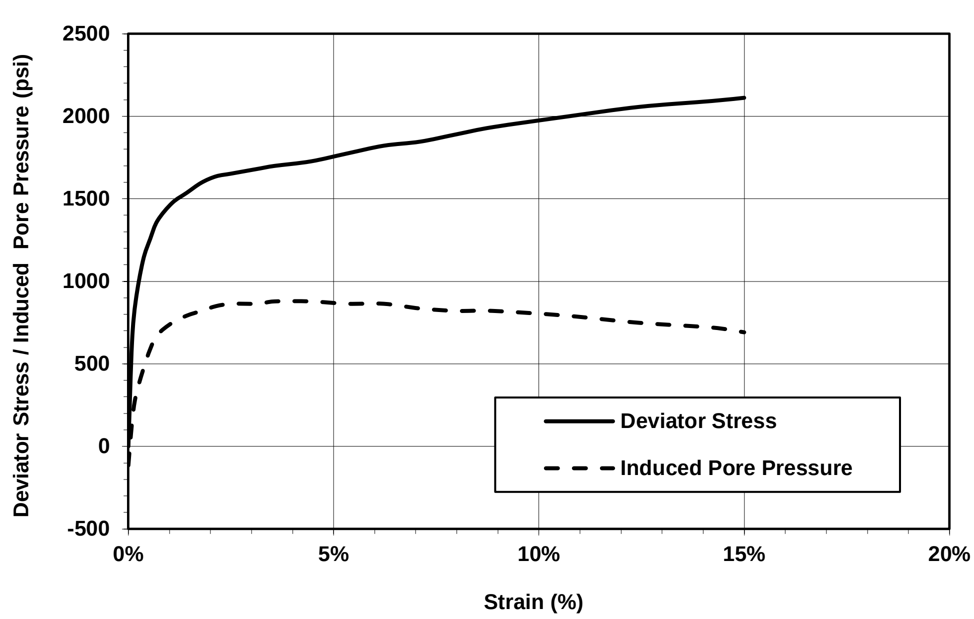
Water Content BEFORE Test			Water Content AFTER Test (Total Specimen)		
Tin No.:	M-74	g	Tin No.:	KDW	g
Wet Soil + Tin :	146.25	g	Wet Soil + Tin :	1262.80	g
Dry Soil + Tin :	124.58	g	Dry Soil + Tin :	1032.30	g
Tin Weight :	27.9	g	Tin Weight :	77.90	g
Dry Mass :	96.68	g	Dry Mass :	954.40	g
Weight of water :	21.67	g	Weight of water :	230.50	g
Moisture :	22.41	%	Moisture :	24.15	%
			Wet Density :	123.27	pcf
			Dry Density :	99.29	pcf

Consolidation Cell Pressure:	140.0	psi	Deviator Stress @ Failure, Ds:	2,112	psf
Consolidation Back Pressure:	125.0	psi	Axial Strain @ Failure:	15.0	%
Effective Confining Stress, σ_3:	15.0	psi	Major Principal Stress @ Failure, σ_1:	4,272	psf
	2,160	psf	Induced Pore Pressure @ Failure:	691	psf
Strain Rate:	0.0030	in/min	Effective Minor Principal Stress,σ'_3:	1,469	psf
			Effective Major Principal Stress, σ'_1:	3,581	psf

Failure Sketch



CU Compressive Strength and Induced Pore Pressure



Notes:

Consolidated, Undrained Triaxial Compression Test (ASTM D4767)

Project Number:	W-13-045	Boring Number:	B-020-2-13
Project Name:	FRA-70-12.68	Station / Offset:	176+13.92, 34.0' Rt.
Project Location:	Columbus, Ohio	Sample No. / Depth:	ST-6 / 14.0 ft
Client:	GPD GROUP	Date of Testing:	8/21/2013

Data for Specimen No. 2

Soil Description: Brown CLAY, "and" coarse to fine sand, some silt, little fine gravel.
Soil Classification: ODOT A-7-6

Physical Characteristics	L.L.	P.L.	P.I.	Gravel%	C. Sand%	F. Sand%	Silt%	Clay%
	41	16	25	17	31	13	24	15

Diameter, D_0	2.849	in	Volume of Solids , V_s :	23.322	in^3
Area, A_0	6.376	in^2	Initial Volume of Voids, V_v :	13.698	in^3
Height, L_0	5.806	in	Initial Void Ratio, e_o :	0.587	
Volume, V_0	37.019	in^3	Initial Degree of Saturation, S_0 :	101.89	%

Water Content BEFORE Test

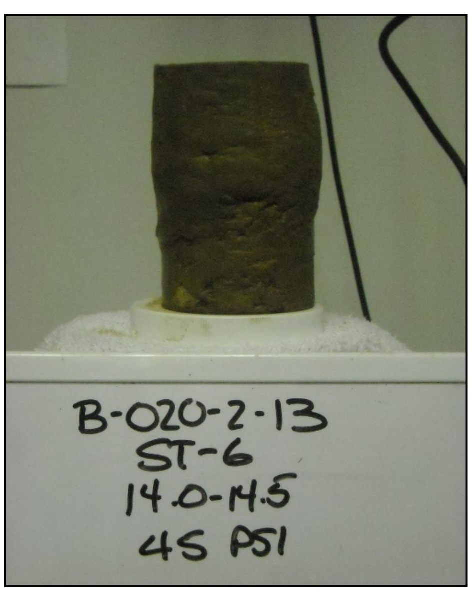
Tin No.:	M-74	g
Wet Soil + Tin :	146.25	g
Dry Soil + Tin :	124.58	g
Tin Weight :	27.9	g
Dry Mass :	96.68	g
Weight of water :	21.67	g
Moisture :	22.41	%

Water Content AFTER Test (Total Specimen)

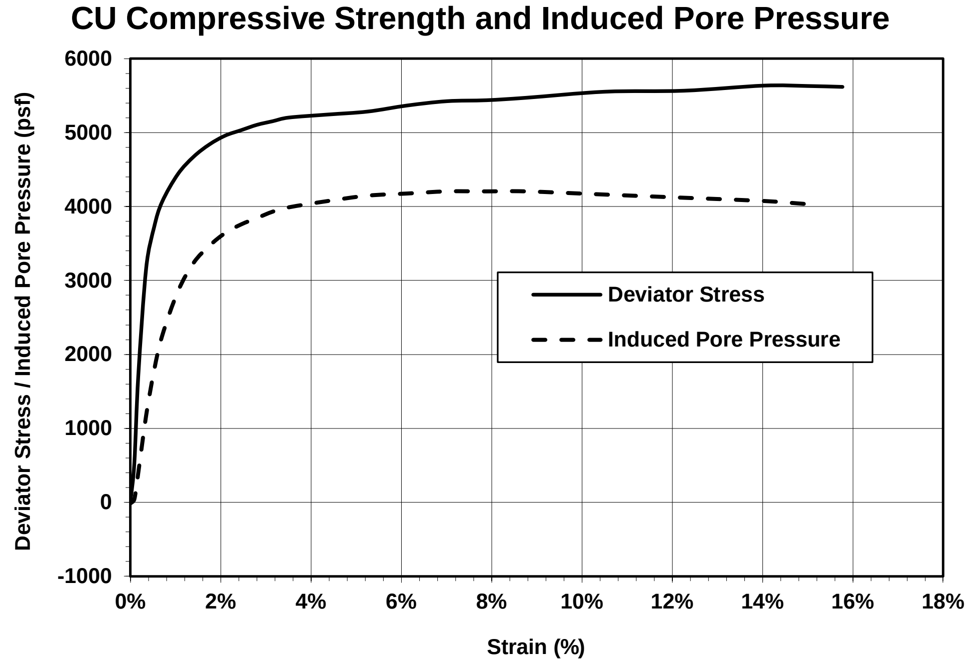
Tin No.:	BC	g
Wet Soil + Tin :	1307.40	g
Dry Soil + Tin :	1110.10	g
Tin Weight :	89.70	g
Dry Mass :	1020.40	g
Weight of water :	197.30	g
Moisture :	19.34	%
Wet Density :	125.31	pcf
Dry Density :	105.01	pcf

Consolidation Cell Pressure:	133.0	psi	Deviator Stress @ Failure, Ds:	5,635	psf
Consolidation Back Pressure:	88.0	psi	Axial Strain @ Failure:	14.0	%
Effective Confining Stress, σ_3:	45.0	psi	Major Principal Stress @ Failure, σ_1:	12,115	psf
	6,480	psf	Induced Pore Pressure @ Failure:	4,075	psf
Strain Rate:	0.0030	in/min	Effective Minor Principal Stress,σ'_3:	2,405	psf
			Effective Major Principal Stress, σ'_1:	8,039	psf

Failure Sketch



CU Compressive Strength and Induced Pore Pressure



Notes:

RESOURCE INTERNATIONAL, INC.
Engineering Consultants

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

Point Load Strength Index of Rock Specimens
(ASTM D 5731-08)

Project: FRA-70-13.10
Project No.: W-13-072
Date of Testing: 2/2/2015
Test Performed by: E.M.

Rock Description: Gray Mudstone

Boring No.: B-020-7-13
Station / Offset: 176+68.64, 1.8' Rt.
Sample No. / Depth: RC-4 / 69.4' to 74.8'

Test Apparatus: Forney-LA 0080
Serial Number: A125/AZ/0014
Date of Calibration: 8/9/2014

Sample No.	Test Type	Depth (ft)	Width (mm)	Diameter (mm)	Load (N)	D_e^2 (mm^2)	D_e (mm)	F	Is (MPa)	$Is_{(50)}$ (MPa)	σ_c (MPa)
1	a ⊥	69.4	37.0	46.5	70	2,192	46.8	0.97	0.03	0.03	0.38
2	a ⊥	70.6	37.1	46.0	185	2,174	46.6	0.97	0.09	0.08	1.02
3	a ⊥	70.9	35.8	45.5	195	2,078	45.6	0.96	0.09	0.09	1.13
4	a ⊥	73.8	36.7	45.9	105	2,143	46.3	0.97	0.05	0.05	0.59
5	a ⊥	74.8	34.2	45.6	110	1,983	44.5	0.95	0.06	0.05	0.67
6											
7											
8											
9											
10											

STATISTICS	
Mean $Is_{(50)}$ ⊥	**0.06 MPa (9 psi)**
Mean $Is_{(50)}$ ∥	
$Ia_{(50)}$	

Specific Specimen Shape:
d = diametrical
a = axial
b = block
i = irregular lump
⊥ = perpendicular to bedding plane
∥ = parallel to bedding plane

Estimated Unaxial Compression, σ_c = K*Is

K = 12
*Per Section 206.1.3 of 2011 ODOT Rock Slope Design Guide

Mean σ_c = 0.76 MPa (110 psi)

Remarks:

RESOURCE INTERNATIONAL, INC.

Engineering Consultants

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

6350 Presidential Gatew.	9885 Rockside Road	4480 Lake Forest Drive
Columbus, OH 43231	Cleveland, OH 44125	Cincinnati, Ohio 45242
Phone (614) 823-4949	Phone (216) 573-0955	Phone (513) 769-6998

Project: FRA-70-12.68

Project No.: W-13-045

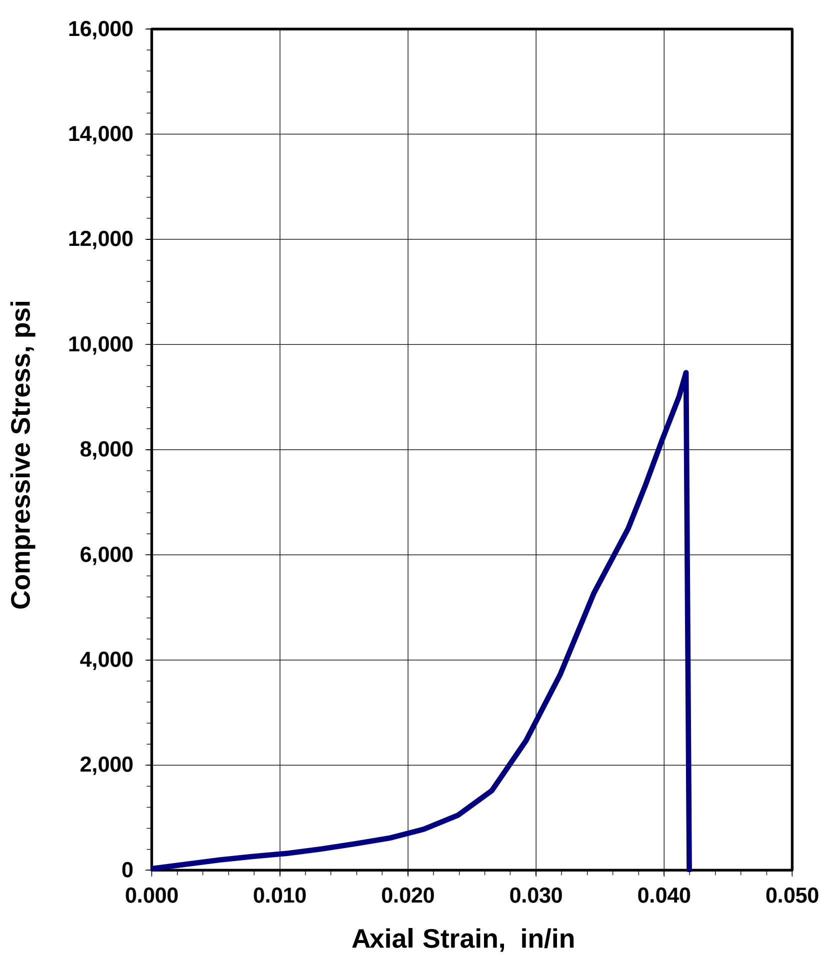
Date of Testing: 7/26/2013

Test Performed by: KR/TK

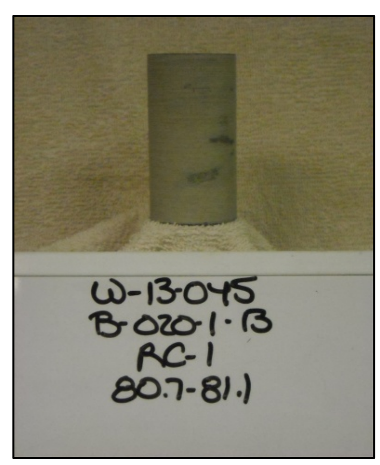
Rock Description: LIMESTONE: Light gray, unweathered, strong.

Boring No.:	B-020-1-13	Average Length:	3.765 in
Station / Offset:	5080+09.80, 30.9' Rt.	Average Diameter:	1.863 in
Sample No. / Depth:	RC-1 / 80.7 ft.	Length to diameter ratio:	2.021
Moisture condition:	As received	Cross Sectional Area:	2.725 in^2
		Failure Load:	25,800 lbs
Rate of Loading:	55.0 lbs/sec	Axial Strain at Failure:	0.0417 in/in
Testing Time:	469 sec	Stress:	9,465 psi
	(Rate 2-15 minutes to failure)		

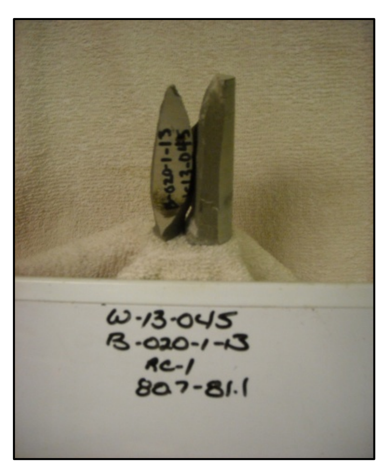
Unconfined Compression Test



Before Testing



After Failure



REMARKS: ______________________________

APPENDIX VI

CELLULAR CONCRETE WALL CALCULATIONS

W-13-045 - FRA-70-12.68 - Retaining Walls 4W5 and 4W6
MSE Wall with Cellular Concrete Backfill Settlement

Boring	Boring Elevation	Top of Wall / Profile Elevation (ft msl)	Bottom of Wall / Embankment Elevation (ft msl)	Wall / Embankment Height (ft)	Pressure at Bottom of Wall / Embankment [1] (psf)	Total Settlement at Center of Wall / Embankment (in)	Total Settlement at Wall Facing (in)	Time for 90% Consolidation (Days)
B-001-A-59	713.9	756.1	710.8	45.3	1,731	3.88	2.80	0
B-020-1-13	712.8	756.1	710.8	45.3	1,731	3.40	2.46	18
B-020-2-13	711.4	756.1	710.8	45.3	1,731	4.28	3.15	35
B-020-9-15	713.0	756.1	710.8	45.3	1,731	3.19	2.37	10
B-021-0-08	727.9	758.1	714.8	43.3	1,671	2.61	1.98	7
B-023-0-08	722.0	759.7	718.5	41.2	1,608	2.72	2.00	11

1. $\Delta\sigma$ = (130 pcf)(3.0 ft) + (36 pcf)(2.0 ft) + (H - 5 ft)(30 pcf)

W-13-045 - FRA-70-12.68 - Retaining Walls 4W5 and 4W6
MSE Wall with Cellular Concrete Backfill Settlement

Calculated By: BRT Date: 7/14/2018
Checked By: JPS Date: 7/15/2018

Boring B-020-7-13

H =	45.3	ft	Total wall/embankment height from profile grade to top of leveling pad
B =	31.7	ft	Wall/embankment width considered in analysis, equal to 70% of the wall height
D_w =	0.0	ft	Depth below bottom of wall/embankment
q =	1,731	psf	Bearing pressure at bottom of wall/embankment (see summary sheet)

																				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	4.0	4.0	2.0	120	480	240	115	2,115					5	10	56	0.06	0.999	1,730	1,845	0.086	1.026	0.500	865	981	0.066	0.792
2	A-4a	G	4.0	6.5	2.5	5.3	115	768	624	296	2,296					9	15	32	0.17	0.986	1,707	2,004	0.065	0.779	0.499	864	1,160	0.046	0.556
	A-4a	G	6.5	9.0	2.5	7.8	115	1,055	911	428	2,428					9	14	30	0.24	0.962	1,665	2,092	0.057	0.682	0.497	860	1,288	0.039	0.473
	A-4a	G	9.0	11.5	2.5	10.3	115	1,343	1,199	559	2,559					9	13	29	0.32	0.925	1,601	2,160	0.050	0.604	0.494	854	1,414	0.035	0.415
3	A-2-4	G	11.5	14.0	2.5	12.8	120	1,643	1,493	697	2,697					13	18	69	0.40	0.880	1,523	2,220	0.018	0.217	0.488	846	1,542	0.012	0.149
4	A-1-a	G	14.0	19.5	5.5	16.8	125	2,330	1,986	941	4,941					25	31	103	0.53	0.800	1,385	2,326	0.021	0.252	0.477	825	1,766	0.015	0.176
	A-1-a	G	19.5	25.0	5.5	22.3	125	3,018	2,674	1,285	5,285					25	29	95	0.70	0.695	1,203	2,488	0.017	0.198	0.455	787	2,073	0.012	0.144
5	A-1-b	G	25.0	30.0	5.0	27.5	135	3,693	3,355	1,639	5,639					94	100	472	0.87	0.608	1,053	2,692	0.002	0.027	0.430	745	2,384	0.002	0.021
	A-1-b	G	30.0	35.0	5.0	32.5	135	4,368	4,030	2,002	6,002					94	94	424	1.03	0.540	934	2,936	0.002	0.024	0.405	701	2,703	0.002	0.018
	A-1-b	G	35.0	40.0	5.0	37.5	135	5,043	4,705	2,365	6,365					94	89	387	1.18	0.483	836	3,201	0.002	0.020	0.380	658	3,023	0.001	0.017
	A-1-b	G	40.0	45.0	5.0	42.5	135	5,718	5,380	2,728	6,728					94	84	356	1.34	0.436	754	3,482	0.001	0.018	0.357	617	3,345	0.001	0.015
6	A-4a	G	45.0	48.0	3.0	46.5	135	6,123	5,920	3,018	7,018					71	61	102	1.47	0.404	699	3,717	0.003	0.032	0.339	586	3,605	0.002	0.027
																				Total Settlement:			3.880 in		Total Settlement:			2.803 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)

W-13-045 - FRA-70-12.68 - Retaining Walls 4W5 and 4W6
MSE Wall with Cellular Concrete Backfill Settlement

Calculated By: BRT Date: 7/14/2018
Checked By: JPS Date: 7/15/2018

Boring B-020-1-13

H =	45.3	ft	Total wall/embankment height from profile grade to top of leveling pad
B =	31.7	ft	Wall/embankment width considered in analysis, equal to 70% of the wall height
D_w =	0.0	ft	Depth below bottom of wall/embankment
q =	1,731	psf	Bearing pressure at bottom of wall/embankment (see summary sheet)

																				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-2-4	G	0.0	3.5	3.5	1.8	125	438	219	110	2,110					21	41	136	0.06	0.999	1,730	1,840	0.032	0.379	0.500	865	975	0.024	0.294
2	A-6a	C	3.5	6.0	2.5	4.8	115	725	581	285	2,285	35	0.225	0.034	0.546				0.15	0.990	1,713	1,998	0.046	0.554	0.499	864	1,149	0.033	0.397
	A-6a	C	6.0	8.5	2.5	7.3	115	1,013	869	416	2,416	35	0.225	0.034	0.546				0.23	0.968	1,675	2,091	0.038	0.459	0.498	861	1,278	0.027	0.319
	A-6b	C	8.5	11.0	2.5	9.8	115	1,300	1,156	548	2,548	40	0.270	0.041	0.585				0.31	0.933	1,615	2,163	0.038	0.457	0.494	856	1,404	0.026	0.313
	A-6b	C	11.0	13.5	2.5	12.3	115	1,588	1,444	679	2,679	40	0.270	0.041	0.585				0.39	0.889	1,539	2,218	0.033	0.394	0.490	848	1,527	0.022	0.270
5	A-1-b	G	13.5	16.0	2.5	14.8	120	1,888	1,738	817	2,817					4	5	51	0.47	0.840	1,455	2,272	0.022	0.263	0.483	836	1,653	0.015	0.181
6	A-1-b	G	16.0	21.0	5.0	18.5	125	2,513	2,200	1,046	5,046					17	21	76	0.58	0.766	1,325	2,371	0.023	0.281	0.470	814	1,860	0.017	0.198
	A-1-b	G	21.0	26.0	5.0	23.5	125	3,138	2,825	1,359	5,359					17	19	73	0.74	0.673	1,165	2,523	0.019	0.222	0.449	778	2,136	0.014	0.162
7	A-1-b	G	26.0	30.0	4.0	28.0	130	3,658	3,398	1,650	5,650					42	45	148	0.88	0.601	1,040	2,690	0.006	0.069	0.428	740	2,391	0.004	0.052
8	A-4a	C	30.0	35.0	5.0	32.5	130	4,308	3,983	1,955	5,955	22	0.108	0.011	0.444				1.03	0.540	934	2,889	0.006	0.076	0.405	701	2,656	0.005	0.060
9	A-1-b	G	35.0	42.5	7.5	38.8	135	5,320	4,814	2,396	6,396					87	82	340	1.22	0.470	814	3,210	0.003	0.034	0.374	648	3,044	0.002	0.028
	A-1-b	G	42.5	50.0	7.5	46.3	135	6,333	5,826	2,940	6,940					87	76	303	1.46	0.405	702	3,642	0.002	0.028	0.340	588	3,528	0.002	0.024
10	A-7-6	C	50.0	56.0	6.0	53.0	130	7,113	6,723	3,415	7,415	44	0.306	0.031	0.616				1.67	0.360	623	4,038	0.008	0.099	0.312	540	3,955	0.007	0.087
	A-7-6	C	56.0	62.0	6.0	59.0	130	7,893	7,503	3,821	7,821	44	0.306	0.031	0.616				1.86	0.327	565	4,386	0.007	0.082	0.290	501	4,322	0.006	0.073
																				Total Settlement:				3.397 in	Total Settlement:				2.457 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 $C_N = [0.77\log(40/\sigma_{vo}')] \le 2.0$ 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 Walls 4W5 and 4W6
MSE Wall with Cellular Concrete Backfill Settlement

Calculated By: BRT Date: 7/14/2018
Checked By: JPS Date: 7/15/2018

Boring B-020-1-13

H =	45.3	ft	Total wall/embankment height from profile grade to top of leveling pad
B =	31.7	ft	Wall/embankment width considered in analysis, equal to 70% of the wall height
D_w =	0.0	ft	Depth below bottom of wall/embankment
q =	1,731	psf	Bearing pressure at bottom of wall/embankment (see summary sheet)

	A-6a	A-6b	A-4a	A-7-6		
c_v =	600	300	800	150	ft^2/yr	Coefficient of consolidation
t =	18	18	18	18	days	Time following completion of construction
H_{dr} =	5.5	5	2.5	12	ft	Length of longest drainage path considered
T_v =	0.978	0.592	6.312	0.051		Time factor
U =	93	81	100	26	%	Degree of consolidation

$(S_c)_t$ = 2.389 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/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-2-4	G	0.0	3.5	3.5	1.8	125	438	219	110	2,110					21	41	136	0.06	0.500	865	975	0.024	0.294	0.294	0.294	0.294
2	A-6a	C	3.5	6.0	2.5	4.8	115	725	581	285	2,285	35	0.210	0.049	0.745				0.15	0.499	864	1,149	0.043	0.510	1.561	0.475	1.375
	A-6a	C	6.0	8.5	2.5	7.3	115	1,013	869	416	2,416	35	0.210	0.049	0.745				0.23	0.498	861	1,278	0.034	0.410		0.382	
	A-6b	C	8.5	11.0	2.5	9.8	115	1,300	1,156	548	2,548	40	0.210	0.049	0.745				0.31	0.494	856	1,404	0.029	0.344		0.279	
	A-6b	C	11.0	13.5	2.5	12.3	115	1,588	1,444	679	2,679	40	0.210	0.049	0.745				0.39	0.490	848	1,527	0.025	0.296		0.240	
5	A-1-b	G	13.5	16.0	2.5	14.8	120	1,888	1,738	817	4,817					4	5	51	0.47	0.483	836	1,653	0.015	0.181	0.181	0.181	0.181
6	A-1-b	G	16.0	21.0	5.0	18.5	125	2,513	2,200	1,046	5,046					17	21	76	0.58	0.470	814	1,860	0.017	0.198	0.360	0.198	0.360
	A-1-b	G	21.0	26.0	5.0	23.5	125	3,138	2,825	1,359	5,359					17	19	73	0.74	0.449	778	2,136	0.014	0.162		0.162	
7	A-1-b	G	26.0	30.0	4.0	28.0	130	3,658	3,398	1,650	5,650					42	45	148	0.88	0.428	740	2,391	0.004	0.052	0.052	0.052	0.052
8	A-4a	C	30.0	35.0	5.0	32.5	130	4,308	3,983	1,955	5,955	22	0.108	0.008	0.444				1.03	0.405	701	2,656	0.004	0.045	0.045	0.045	0.045
9	A-1-b	G	35.0	42.5	7.5	38.8	135	5,320	4,814	2,396	6,396					87	82	340	1.22	0.374	648	3,044	0.002	0.028	0.051	0.028	0.051
	A-1-b	G	42.5	50.0	7.5	46.3	135	6,333	5,826	2,940	6,940					87	76	303	1.46	0.340	588	3,528	0.002	0.024		0.024	
10	A-7-6	C	50.0	56.0	6.0	53.0	130	7,113	6,723	3,415	7,415	44	0.306	0.023	0.616				1.67	0.312	540	3,955	0.005	0.065	0.120	0.017	0.031
	A-7-6	C	56.0	62.0	6.0	59.0	130	7,893	7,503	3,821	7,821	44	0.306	0.023	0.616				1.86	0.290	501	4,322	0.005	0.055		0.014	

Settlement Remaining After Hold Period: 0.275 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 Walls 4W5 and 4W6

MSE Wall with Cellular Concrete Backfill Settlement

Calculated By: BRT Date: 7/14/2018

Checked By: JPS Date: 7/15/2018

Boring B-020-2-13

H =	45.3	ft	Total wall/embankment height from profile grade to top of leveling pad
B =	31.7	ft	Wall/embankment width considered in analysis, equal to 70% of the wall height
D_w =	0.0	ft	Depth below bottom of wall/embankment
q =	1,731	psf	Bearing pressure at bottom of wall/embankment (see summary sheet)

																				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				0.04	1.000	1,731	1,803	0.076	0.916	0.500	865	937	0.061	0.730
2	A-1-b	G	2.5	4.5	2.0	3.5	120	540	420	202	2,202					4	7	53	0.11	0.996	1,724	1,925	0.037	0.445	0.500	865	1,067	0.027	0.329
3	A-6a	C	4.5	7.0	2.5	5.8	115	828	684	325	2,325	35	0.225	0.034	0.546				0.18	0.983	1,701	2,026	0.043	0.521	0.499	863	1,188	0.031	0.369
4	A-6b	C	7.0	9.5	2.5	8.3	115	1,115	971	456	2,456	39	0.261	0.039	0.577				0.26	0.955	1,653	2,110	0.041	0.495	0.497	860	1,316	0.029	0.342
	A-6b	C	9.5	12.0	2.5	10.8	115	1,403	1,259	588	2,588	39	0.261	0.039	0.577				0.34	0.916	1,586	2,174	0.035	0.423	0.493	853	1,441	0.024	0.290
5	A-7-6	C	12.0	14.5	2.5	13.3	115	1,690	1,546	719	2,719	41	0.279	0.042	0.593				0.42	0.870	1,506	2,225	0.032	0.387	0.487	843	1,563	0.022	0.266
	A-7-6	C	14.5	17.0	2.5	15.8	115	1,978	1,834	851	2,851	41	0.279	0.042	0.593				0.50	0.820	1,420	2,271	0.028	0.336	0.480	831	1,682	0.019	0.233
6	A-2-4	G	17.0	19.5	2.5	18.3	120	2,278	2,128	989	4,989					8	10	57	0.58	0.770	1,334	2,322	0.016	0.197	0.471	816	1,805	0.012	0.139
7	A-1-b	G	19.5	27.0	7.5	23.3	130	3,253	2,765	1,314	5,314					34	39	127	0.73	0.677	1,172	2,486	0.016	0.197	0.450	780	2,094	0.012	0.144
8	A-1-b	G	27.0	35.0	8.0	31.0	135	4,333	3,793	1,858	5,858					91	93	419	0.98	0.559	968	2,826	0.003	0.042	0.413	714	2,572	0.003	0.032
	A-1-b	G	35.0	43.5	8.5	39.3	135	5,480	4,906	2,457	6,457					91	85	360	1.24	0.465	806	3,263	0.003	0.035	0.372	644	3,101	0.002	0.029
9	A-6b	C	43.5	49.5	6.0	46.5	130	6,260	5,870	2,968	6,968	38	0.252	0.025	0.569				1.47	0.404	699	3,667	0.009	0.106	0.339	586	3,555	0.008	0.090
	A-6b	C	49.5	55.5	6.0	52.5	130	7,040	6,650	3,374	7,374	38	0.252	0.025	0.569				1.66	0.363	628	4,002	0.007	0.086	0.314	543	3,917	0.006	0.075
	A-6b	C	55.5	63.5	8.0	59.5	130	8,080	7,560	3,847	7,847	38	0.252	0.025	0.569				1.88	0.324	561	4,408	0.008	0.091	0.288	498	4,346	0.007	0.082
																						Total Settlement:		4.276 in			Total Settlement:		3.150 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. $S_c = [C_r/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ 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 Walls 4W5 and 4W6

MSE Wall with Cellular Concrete Backfill Settlement

Calculated By: BRT Date: 7/14/2018

Checked By: JPS Date: 7/15/2018

Boring B-020-2-13

H =	45.3	ft	Total wall/embankment height from profile grade to top of leveling pad
B =	31.7	ft	Wall/embankment width considered in analysis, equal to 70% of the wall height
D_w =	0.0	ft	Depth below bottom of wall/embankment
q =	1,731	psf	Bearing pressure at bottom of wall/embankment (see summary sheet)

	A-6a	A-6b (Upper)	A-7-6	A-6b (Lower)		
c_v =	600	300	150	300	ft^2/yr	Coefficient of consolidation
t =	35	35	35	35	days	Time following completion of construction
H_{dr} =	2.5	5	5	20	ft	Length of longest drainage path considered
T_v =	9.205	1.151	0.575	0.072		Time factor
U =	100	95	80	30	%	Degree of consolidation

$(S_c)_t$ = 2.845 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				0.04	0.500	865	937	0.061	0.730	0.730	0.730	0.730
2	A-1-b	G	2.5	4.5	2.0	3.5	120	540	420	202	2,202					4	7	53	0.11	0.500	865	1,067	0.027	0.329	0.329	0.329	0.329
3	A-6a	C	4.5	7.0	2.5	5.8	115	828	684	325	2,325	35	0.225	0.034	0.546				0.18	0.499	863	1,188	0.031	0.369	0.369	0.369	0.369
4	A-6b	C	7.0	9.5	2.5	8.3	115	1,115	971	456	2,456	39	0.261	0.039	0.577				0.26	0.497	860	1,316	0.029	0.342	0.632	0.325	0.601
	A-6b	C	9.5	12.0	2.5	10.8	115	1,403	1,259	588	2,588	39	0.261	0.039	0.577				0.34	0.493	853	1,441	0.024	0.290		0.275	
5	A-7-6	C	12.0	14.5	2.5	13.3	115	1,690	1,546	719	2,719	41	0.279	0.042	0.593				0.42	0.487	843	1,563	0.022	0.266	0.499	0.212	0.399
	A-7-6	C	14.5	17.0	2.5	15.8	115	1,978	1,834	851	2,851	41	0.279	0.042	0.593				0.50	0.480	831	1,682	0.019	0.233		0.187	
6	A-2-4	G	17.0	19.5	2.5	18.3	120	2,278	2,128	989	4,989					8	10	57	0.58	0.471	816	1,805	0.012	0.139	0.139	0.139	0.139
7	A-1-b	G	19.5	27.0	7.5	23.3	130	3,253	2,765	1,314	5,314					34	39	127	0.73	0.450	780	2,094	0.012	0.144	0.144	0.144	0.144
8	A-1-b	G	27.0	35.0	8.0	31.0	135	4,333	3,793	1,858	5,858					91	93	419	0.98	0.413	714	2,572	0.003	0.032	0.061	0.032	0.061
	A-1-b	G	35.0	43.5	8.5	39.3	135	5,480	4,906	2,457	6,457					91	85	360	1.24	0.372	644	3,101	0.002	0.029		0.029	
9	A-6b	C	43.5	49.5	6.0	46.5	130	6,260	5,870	2,968	6,968	38	0.252	0.025	0.569				1.47	0.339	586	3,555	0.008	0.090	0.247	0.027	0.074
	A-6b	C	49.5	55.5	6.0	52.5	130	7,040	6,650	3,374	7,374	38	0.252	0.025	0.569				1.66	0.314	543	3,917	0.006	0.075		0.022	
	A-6b	C	55.5	63.5	8.0	59.5	130	8,080	7,560	3,847	7,847	38	0.252	0.025	0.569				1.88	0.288	498	4,346	0.007	0.082		0.024	

Settlement Remaining After Hold Period: 0.304 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 $C_N = [0.77\log(40/\sigma_{vo}')] \le 2.0$ 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)
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 Walls 4W5 and 4W6

MSE Wall with Cellular Concrete Backfill Settlement

Calculated By: BRT Date: 7/14/2018

Checked By: JPS Date: 7/15/2018

Boring B-020-9-15

H =	45.3	ft	Total wall/embankment height from profile grade to top of leveling pad
B =	31.7	ft	Wall/embankment width considered in analysis, equal to 70% of the wall height
D_w =	0.0	ft	Depth below bottom of wall/embankment
q =	1,731	psf	Bearing pressure at bottom of wall/embankment (see summary sheet)

																				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	3.5	3.5	1.8	115	403	201	92	2,092	35	0.225	0.034	0.546				0.06	0.999	1,730	1,822	0.099	1.189	0.500	865	957	0.078	0.933
2	A-6b	C	3.5	6.0	2.5	4.8	115	690	546	250	2,250	34	0.216	0.032	0.538				0.15	0.990	1,713	1,963	0.047	0.566	0.499	864	1,114	0.034	0.410
	A-6b	C	6.0	8.5	2.5	7.3	115	978	834	381	2,381	34	0.216	0.032	0.538				0.23	0.968	1,675	2,056	0.039	0.463	0.498	861	1,243	0.027	0.324
3	A-4a	G	8.5	11.0	2.5	9.8	130	1,303	1,140	532	4,532					37	53	90	0.31	0.933	1,615	2,147	0.017	0.203	0.494	856	1,388	0.012	0.139
	A-4a	G	11.0	13.5	2.5	12.3	130	1,628	1,465	701	4,701					37	50	85	0.39	0.889	1,539	2,240	0.015	0.179	0.490	848	1,548	0.010	0.122
	A-4a	G	13.5	16.0	2.5	14.8	130	1,953	1,790	870	4,870					37	47	81	0.47	0.840	1,455	2,324	0.013	0.159	0.483	836	1,706	0.009	0.109
	A-4a	G	16.0	18.5	2.5	17.3	130	2,278	2,115	1,039	5,039					37	45	77	0.54	0.790	1,368	2,407	0.012	0.141	0.475	822	1,861	0.008	0.098
4	A-1-b	G	18.5	26.0	7.5	22.3	135	3,290	2,784	1,395	5,395					77	86	370	0.70	0.695	1,203	2,598	0.005	0.066	0.455	787	2,183	0.004	0.047
5	A-2-4	G	26.0	30.5	4.5	28.3	135	3,898	3,594	1,831	5,831					80	83	344	0.89	0.597	1,034	2,865	0.003	0.031	0.426	738	2,569	0.002	0.023
	A-2-4	G	30.5	35.0	4.5	32.8	135	4,505	4,201	2,158	6,158					80	78	316	1.03	0.537	929	3,086	0.002	0.027	0.404	699	2,857	0.002	0.021
6	A-2-6	G	35.0	40.0	5.0	37.5	135	5,180	4,843	2,503	6,503					120	111	560	1.18	0.483	836	3,338	0.001	0.013	0.380	658	3,161	0.001	0.011
7	A-1-b	G	40.0	45.0	5.0	42.5	135	5,855	5,518	2,866	6,866					96	85	358	1.34	0.436	754	3,620	0.001	0.017	0.357	617	3,483	0.001	0.014
	A-1-b	G	45.0	50.0	5.0	47.5	135	6,530	6,193	3,229	7,229					96	81	333	1.50	0.396	686	3,914	0.001	0.015	0.334	579	3,807	0.001	0.013
8	A-7-6	C	50.0	58.5	8.5	54.3	130	7,635	7,083	3,697	7,697	42	0.288	0.029	0.600				1.71	0.352	610	4,307	0.010	0.122	0.307	531	4,229	0.009	0.107
																					Total Settlement:			3.189 in		Total Settlement:			2.372 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)

W-13-045 - FRA-70-12.68 - Retaining Walls 4W5 and 4W6 Calculated By: BRT Date: 7/14/2018

MSE Wall with Cellular Concrete Backfill Settlement Checked By: JPS Date: 7/15/2018

Boring B-020-9-15

H =	45.3	ft	Total wall/embankment height from profile grade to top of leveling pad
B =	31.7	ft	Wall/embankment width considered in analysis, equal to 70% of the wall height
D_w =	0.0	ft	Depth below bottom of wall/embankment
q =	1,731	psf	Bearing pressure at bottom of wall/embankment (see summary sheet)

	A-6a	A-6b	A-7-6		
c_v =	600	300	150	ft^2/yr	Coefficient of consolidation
t =	10	10	10	days	Time following completion of construction
H_{dr} =	3.5	4	8.5	ft	Length of longest drainage path considered
T_v =	1.342	0.514	0.057		Time factor
U =	97	77	27	%	Degree of consolidation

$(S_c)_t$ = 2.097 in Settlement complete at 88% 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/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	Total Settlement at Facing of Wall: S_c (9,10) (ft)	S_c (in)	Layer Settlement (in)	Settlement Complete at 88% of Primary Consolidation: $(S_c)_t$ (11) (in)	Layer Settlement (in)
1	A-6a	C	0.0	3.5	3.5	1.8	115	403	201	92	2,092	35	0.225	0.034	0.546				0.06	0.500	865	957	0.078	0.933	0.933	0.905	0.905
2	A-6b	C	3.5	6.0	2.5	4.8	115	690	546	250	2,250	34	0.216	0.032	0.538				0.15	0.499	864	1,114	0.034	0.410	0.735	0.316	0.566
	A-6b	C	6.0	8.5	2.5	7.3	115	978	834	381	2,381	34	0.216	0.032	0.538				0.23	0.498	861	1,243	0.027	0.324		0.250	
3	A-4a	G	8.5	11.0	2.5	9.8	130	1,303	1,140	532	4,532					37	53	90	0.31	0.494	856	1,388	0.012	0.139	0.468	0.139	0.468
	A-4a	G	11.0	13.5	2.5	12.3	130	1,628	1,465	701	4,701					37	50	85	0.39	0.490	848	1,548	0.010	0.122		0.122	
	A-4a	G	13.5	16.0	2.5	14.8	130	1,953	1,790	870	4,870					37	47	81	0.47	0.483	836	1,706	0.009	0.109		0.109	
	A-4a	G	16.0	18.5	2.5	17.3	130	2,278	2,115	1,039	5,039					37	45	77	0.54	0.475	822	1,861	0.008	0.098		0.098	
4	A-1-b	G	18.5	26.0	7.5	22.3	135	3,290	2,784	1,395	5,395					77	86	370	0.70	0.455	787	2,183	0.004	0.047	0.047	0.047	0.047
5	A-2-4	G	26.0	30.5	4.5	28.3	135	3,898	3,594	1,831	5,831					80	83	344	0.89	0.426	738	2,569	0.002	0.023	0.044	0.023	0.044
	A-2-4	G	30.5	35.0	4.5	32.8	135	4,505	4,201	2,158	6,158					80	78	316	1.03	0.404	699	2,857	0.002	0.021		0.021	
6	A-2-6	G	35.0	40.0	5.0	37.5	135	5,180	4,843	2,503	6,503					120	111	560	1.18	0.380	658	3,161	0.001	0.011	0.011	0.011	0.011
7	A-1-b	G	40.0	45.0	5.0	42.5	135	5,855	5,518	2,866	6,866					96	85	358	1.34	0.357	617	3,483	0.001	0.014	0.027	0.014	0.027
	A-1-b	G	45.0	50.0	5.0	47.5	135	6,530	6,193	3,229	7,229					96	81	333	1.50	0.334	579	3,807	0.001	0.013		0.013	
8	A-7-6	C	50.0	58.5	8.5	54.3	130	7,635	7,083	3,697	7,697	42	0.288	0.029	0.600				1.71	0.307	531	4,229	0.009	0.107	0.107	0.029	0.029

Settlement Remaining After Hold Period: 0.275 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 Walls 4W5 and 4W6
MSE Wall with Cellular Concrete Backfill Settlement

Calculated By: BRT Date: 7/14/2018
Checked By: JPS Date: 7/15/2018

Boring B-021-0-08

H =	43.3	ft	Total wall/embankment height from profile grade to top of leveling pad
B =	30.3	ft	Wall/embankment width considered in analysis, equal to 70% of the wall height
D_w =	0.0	ft	Depth below bottom of wall/embankment
q =	1,671	psf	Bearing pressure at bottom of wall/embankment (see summary sheet)

																			Total Settlement at Center of Reinforced 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-6b	C	0.0	2.5	2.5	1.3	115	288	144	66	2,066	35	0.225	0.034	0.546				0.04	1.000	1,671	1,736	0.078	0.931	0.500	835	901	0.062	0.745
	A-6b	C	2.5	5.0	2.5	3.8	115	575	431	197	2,197	35	0.225	0.034	0.546				0.12	0.994	1,661	1,858	0.053	0.638	0.500	835	1,032	0.039	0.471
2	A-1-b	G	5.0	7.5	2.5	6.3	130	900	738	348	4,348					36	57	201	0.21	0.975	1,630	1,977	0.009	0.113	0.498	833	1,180	0.007	0.079
	A-1-b	G	7.5	10.0	2.5	8.8	130	1,225	1,063	517	4,517					36	52	179	0.29	0.942	1,575	2,091	0.008	0.101	0.495	828	1,344	0.006	0.069
3	A-4a	C	10.0	12.5	2.5	11.3	120	1,525	1,375	673	4,673	27	0.153	0.015	0.483				0.37	0.898	1,501	2,174	0.013	0.158	0.491	820	1,493	0.009	0.107
	A-4a	C	12.5	15.0	2.5	13.8	120	1,825	1,675	817	4,817	27	0.153	0.015	0.483				0.45	0.848	1,416	2,233	0.011	0.135	0.484	809	1,626	0.008	0.093
4	A-1-b	G	15.0	19.0	4.0	17.0	125	2,325	2,075	1,014	5,014					21	26	88	0.56	0.780	1,303	2,317	0.016	0.196	0.473	791	1,805	0.011	0.137
5	A-1-b	G	19.0	24.0	5.0	21.5	135	3,000	2,663	1,321	5,321					95	108	536	0.71	0.691	1,154	2,475	0.003	0.031	0.454	758	2,079	0.002	0.022
	A-1-b	G	24.0	29.0	5.0	26.5	135	3,675	3,338	1,684	5,684					95	101	473	0.87	0.605	1,011	2,695	0.002	0.026	0.429	717	2,401	0.002	0.020
6	A-3	G	29.0	36.5	7.5	32.8	135	4,688	4,181	2,138	6,138					71	70	163	1.08	0.518	866	3,004	0.007	0.082	0.396	662	2,800	0.005	0.065
	A-3	G	36.5	44.0	7.5	40.3	135	5,700	5,194	2,682	6,682					71	64	149	1.33	0.439	734	3,416	0.005	0.063	0.358	599	3,281	0.004	0.053
7	A-6b	C	44.0	49.0	5.0	46.5	130	6,350	6,025	3,123	7,123	37	0.243	0.024	0.561				1.53	0.388	648	3,772	0.006	0.076	0.329	550	3,674	0.005	0.066
	A-6b	C	49.0	54.0	5.0	51.5	130	7,000	6,675	3,461	7,461	37	0.243	0.024	0.561				1.70	0.355	592	4,054	0.005	0.064	0.308	515	3,977	0.005	0.056
																					Total Settlement:			2.614 in		Total Settlement:			1.982 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 (Cohesive 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 Walls 4W5 and 4W6
MSE Wall with Cellular Concrete Backfill Settlement

Calculated By: BRT Date: 7/14/2018
Checked By: JPS Date: 7/15/2018

Boring B-021-0-08

H =	43.3	ft	Total wall/embankment height from profile grade to top of leveling pad
B =	30.3	ft	Wall/embankment width considered in analysis, equal to 70% of the wall height
D_w =	0.0	ft	Depth below bottom of wall/embankment
q =	1,671	psf	Bearing pressure at bottom of wall/embankment (see summary sheet)

	Ex. Fill A-6b	Nat. A-4a	Nat. A-6b		
c_v =	300	800	300	ft^2/yr	Coefficient of consolidation
t =	7	7	7	days	Time following completion of construction
H_{dr} =	2.5	2.5	10	ft	Length of longest drainage path considered
T_v =	0.921	2.455	0.058		Time factor
U =	92	100	27	%	Degree of consolidation

$(S_c)_t$ = 1.795 in 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)	Total Settlement at Facing of Wall: S_c (9,10) (ft)	S_c (in)	Layer Settlement (in)	Settlement Complete at 91% of Primary Consolidation: $(S_c)_t$ (11) (in)	Layer Settlement (in)
1	A-6b	C	0.0	2.5	2.5	1.3	115	288	144	66	2,066	35	0.225	0.034	0.546				0.04	0.500	835	901	0.062	0.745	1.216	0.685	1.118
	A-6b	C	2.5	5.0	2.5	3.8	115	575	431	197	2,197	35	0.225	0.034	0.546				0.12	0.500	835	1,032	0.039	0.471		0.433	
2	A-1-b	G	5.0	7.5	2.5	6.3	130	900	738	348	4,348					36	57	201	0.21	0.498	833	1,180	0.007	0.079	0.149	0.079	0.149
	A-1-b	G	7.5	10.0	2.5	8.8	130	1,225	1,063	517	4,517					36	52	179	0.29	0.495	828	1,344	0.006	0.069		0.069	
3	A-4a	C	10.0	12.5	2.5	11.3	120	1,525	1,375	673	4,673	27	0.153	0.015	0.483				0.37	0.491	820	1,493	0.009	0.107	0.200	0.107	0.200
	A-4a	C	12.5	15.0	2.5	13.8	120	1,825	1,675	817	4,817	27	0.153	0.015	0.483				0.45	0.484	809	1,626	0.008	0.093		0.093	
4	A-1-b	G	15.0	19.0	4.0	17.0	125	2,325	2,075	1,014	5,014					21	26	88	0.56	0.473	791	1,805	0.011	0.137	0.159	0.137	0.159
5	A-1-b	G	19.0	24.0	5.0	21.5	135	3,000	2,663	1,321	5,321					95	108	536	0.71	0.454	758	2,079	0.002	0.022		0.022	
	A-1-b	G	24.0	29.0	5.0	26.5	135	3,675	3,338	1,684	5,684					95	101	473	0.87	0.429	717	2,401	0.002	0.020	0.020	0.020	0.020
6	A-3	G	29.0	36.5	7.5	32.8	135	4,688	4,181	2,138	6,138					71	70	163	1.08	0.396	662	2,800	0.005	0.065	0.117	0.065	0.117
	A-3	G	36.5	44.0	7.5	40.3	135	5,700	5,194	2,682	6,682					71	64	149	1.33	0.358	599	3,281	0.004	0.053		0.053	
7	A-6b	C	44.0	49.0	5.0	46.5	130	6,350	6,025	3,123	7,123	37	0.243	0.024	0.561				1.53	0.329	550	3,674	0.005	0.066	0.122	0.018	0.033
	A-6b	C	49.0	54.0	5.0	51.5	130	7,000	6,675	3,461	7,461	37	0.243	0.024	0.561				1.70	0.308	515	3,977	0.005	0.056		0.015	

Settlement Remaining After Hold Period: 0.186 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 26, FHWA GEC 5
3. $C_r = 0.15(C_c)$ for medium stiff to stiff natural soil deposits and existing fill material, 0.075 to 0.10(C_c) for very stiff to hard natural soil deposits, and 0.05(C_c) for new embankment fill; Ref. Section 5.4.2.5 of FHWA GEC 5
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. $S_c = [C_r/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ 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 (Cohesive 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 Walls 4W5 and 4W6

MSE Wall with Cellular Concrete Backfill Settlement

Calculated By: BRT Date: 7/14/2018

Checked By: JPS Date: 7/15/2018

Boring B-023-0-08

H =	41.2	ft	Total wall/embankment height from profile grade to top of leveling pad
B =	28.8	ft	Wall/embankment width considered in analysis, equal to 70% of the wall height
D_w =	0.0	ft	Depth below bottom of wall/embankment
q =	1,608	psf	Bearing pressure at bottom of wall/embankment (see summary sheet)

																				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	115	288	144	66	2,066	29	0.171	0.026	0.499				0.04	1.000	1,608	1,673	0.060	0.722	0.500	804	870	0.048	0.576
2	A-4a	C	2.5	5.0	2.5	3.8	115	575	431	197	2,197	28	0.162	0.024	0.491				0.13	0.993	1,597	1,794	0.039	0.469	0.500	803	1,001	0.029	0.345
	A-4a	C	5.0	7.5	2.5	6.3	115	863	719	329	2,329	28	0.162	0.024	0.491				0.22	0.972	1,563	1,891	0.031	0.372	0.498	801	1,129	0.022	0.262
	A-4a	C	7.5	10.0	2.5	8.8	115	1,150	1,006	460	2,460	28	0.162	0.024	0.491				0.30	0.935	1,503	1,964	0.026	0.308	0.495	795	1,256	0.018	0.213
3	A-2-4	G	10.0	12.0	2.0	11.0	120	1,390	1,270	584	2,584					11	16	66	0.38	0.892	1,434	2,018	0.016	0.197	0.490	788	1,371	0.011	0.136
	A-2-4	G	12.0	14.0	2.0	13.0	120	1,630	1,510	699	2,699					11	15	64	0.45	0.849	1,365	2,064	0.015	0.175	0.484	779	1,478	0.010	0.121
	A-2-4	G	14.0	16.0	2.0	15.0	120	1,870	1,750	814	2,814					11	14	63	0.52	0.805	1,295	2,109	0.013	0.156	0.478	768	1,582	0.009	0.109
	A-2-4	G	16.0	18.0	2.0	17.0	120	2,110	1,990	929	2,929					11	14	63	0.59	0.761	1,224	2,154	0.012	0.140	0.470	755	1,684	0.008	0.099
4	A-4a	G	18.0	21.5	3.5	19.8	135	2,583	2,346	1,114	5,114					120	144	224	0.69	0.704	1,132	2,246	0.005	0.057	0.457	735	1,849	0.003	0.041
5	A-3a	G	21.5	24.5	3.0	23.0	135	2,988	2,785	1,350	5,350					62	70	212	0.80	0.642	1,033	2,383	0.003	0.042	0.441	709	2,058	0.003	0.031
	A-3a	G	24.5	28.0	3.5	26.3	135	3,460	3,224	1,586	5,586					62	67	199	0.91	0.588	945	2,531	0.004	0.043	0.423	681	2,266	0.003	0.033
	A-3a	G	28.0	31.5	3.5	29.8	135	3,933	3,696	1,840	5,840					62	64	187	1.03	0.537	863	2,703	0.003	0.037	0.404	649	2,489	0.002	0.029
																				Total Settlement:			2.718 in		Total Settlement:			1.996 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)

W-13-045 - FRA-70-12.68 - Retaining Walls 4W5 and 4W6
MSE Wall with Cellular Concrete Backfill Settlement

Calculated By: BRT Date: 7/14/2018
Checked By: JPS Date: 7/15/2018

Boring B-023-0-08

H =	41.2	ft	Total wall/embankment height from profile grade to top of leveling pad
B =	28.8	ft	Wall/embankment width considered in analysis, equal to 70% of the wall height
D_w =	0.0	ft	Depth below bottom of wall/embankment
q =	1,608	psf	Bearing pressure at bottom of wall/embankment (see summary sheet)

	Ex. Fill A-6a	Ex. Fill A-4a		
c_v =	600	800	ft^2/yr	Coefficient of consolidation
t =	11	11	days	Time following completion of construction
H_{dr} =	2.5	7	ft	Length of longest drainage path considered
T_v =	2.893	0.492		Time factor
U =	100	76	%	Degree of consolidation

$(S_c)_t$ = 1.799 in 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/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	Total Settlement at Facing of Wall: S_c (9,10) (ft)	S_c (in)	Layer Settlement (in)	Settlement Complete at 90% of Primary Consolidation: $(S_c)_t$ (11) (in)	Layer Settlement (in)
1	A-6a	C	0.0	2.5	2.5	1.3	115	288	144	66	2,066	29	0.171	0.026	0.499				0.04	0.500	804	870	0.048	0.576	0.576	0.576	0.576
2	A-4a	C	2.5	5.0	2.5	3.8	115	575	431	197	2,197	28	0.162	0.024	0.491				0.13	0.500	803	1,001	0.029	0.345	0.820	0.262	0.623
	A-4a	C	5.0	7.5	2.5	6.3	115	863	719	329	2,329	28	0.162	0.024	0.491				0.22	0.498	801	1,129	0.022	0.262		0.199	
	A-4a	C	7.5	10.0	2.5	8.8	115	1,150	1,006	460	2,460	28	0.162	0.024	0.491				0.30	0.495	795	1,256	0.018	0.213		0.162	
3	A-2-4	G	10.0	12.0	2.0	11.0	120	1,390	1,270	584	2,584					11	16	66	0.38	0.490	788	1,371	0.011	0.136	0.465	0.136	0.465
	A-2-4	G	12.0	14.0	2.0	13.0	120	1,630	1,510	699	2,699					11	15	64	0.45	0.484	779	1,478	0.010	0.121		0.121	
	A-2-4	G	14.0	16.0	2.0	15.0	120	1,870	1,750	814	2,814					11	14	63	0.52	0.478	768	1,582	0.009	0.109		0.109	
	A-2-4	G	16.0	18.0	2.0	17.0	120	2,110	1,990	929	2,929					11	14	63	0.59	0.470	755	1,684	0.008	0.099		0.099	
4	A-4a	G	18.0	21.5	3.5	19.8	135	2,583	2,346	1,114	5,114					120	144	224	0.69	0.457	735	1,849	0.003	0.041	0.041	0.041	0.041
5	A-3a	G	21.5	24.5	3.0	23.0	135	2,988	2,785	1,350	5,350					62	70	212	0.80	0.441	709	2,058	0.003	0.031	0.093	0.031	0.093
	A-3a	G	24.5	28.0	3.5	26.3	135	3,460	3,224	1,586	5,586					62	67	199	0.91	0.423	681	2,266	0.003	0.033		0.033	
	A-3a	G	28.0	31.5	3.5	29.8	135	3,933	3,696	1,840	5,840					62	64	187	1.03	0.404	649	2,489	0.002	0.029		0.029	

Settlement Remaining After Hold Period: 0.197 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 26, FHWA GEC 5
3. C_r = 0.15(C_c) for medium stiff to stiff natural soil deposits and existing fill material, 0.075 to 0.10(C_c) for very stiff to hard natural soil deposits, and 0.05(C_c) for new embankment fill; Ref. Section 5.4.2.5 of FHWA GEC 5
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 (Cohesive 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

B =	31.7	ft	
L =	142	ft	
c =	1,375	psf	
γ =	115	pcf	
D_f =	3.0	ft	
φ =	0	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma} = 7.55 \text{ ksf}$$

$$N_{cm} = N_c s_c i_c = 5.36$$

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

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

N_c =	5.14	s_c =	1+(31.71 ft/142 ft)(1/5.14) =	1.043	i_c =	1.000	d_q =	1+2tan(0°)[1-sin(0°)]²tan⁻¹(3 ft/31.71 ft) =	1.000
N_q =	1.00	s_q =	1+(31.71 ft/142 ft)tan(0°) =	1.000	i_q =	1.000	C_{wq} =	0.0 ft < 3.0 ft =	0.500
N_γ =	0.00	s_γ =	1-0.4(31.71 ft/142 ft) =	0.911	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(31.71 ft) + 3 ft =	0.500

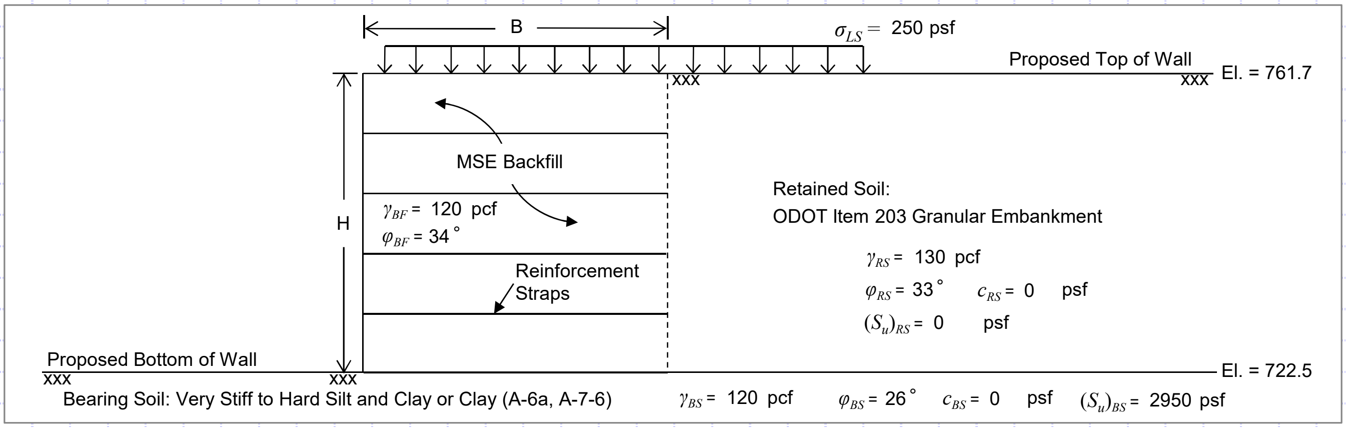
$$q_R = q_n \cdot \phi_b = 3.77 \text{ ksf}$$

φ_b = 0.5

APPENDIX VII

MSE WALL CALCULATIONS

FRA-70-12.68 - Retaining Wall 4W5 - Sta. 5083+25 to Sta. 5085+54 - B-003-A-59, B-023-1-13, B-024-0-08 - 39.2 ft. Wall Height



MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	39.2	ft
MSE Wall Width (Reinforcement Length), (B) =	27.4	ft
MSE Wall Length, (L) =	229	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}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(S_u)_{BS}]$ =	2950	psf
Embedment Depth, (D_f) =	3.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	8.0	ft

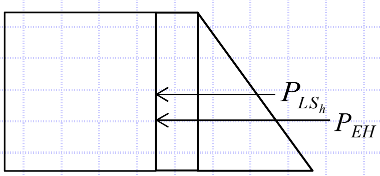
LRFD Load Factors

	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:



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

$$P_{EH} = \tfrac{1}{2}\gamma_{RS} H^2 K_a \gamma_{EH} = \tfrac{1}{2}(130\text{ pcf})(39.2\text{ ft})^2(0.264)(1.5) = 39.55 \text{ kip/ft}$$

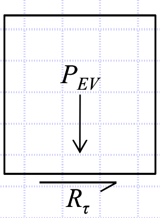
$$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS} = (250\text{ psf})(39.2\text{ ft})(0.264)(1.75) = 4.53 \text{ kip/ft}$$

$$P_H = 39.55\text{ kip/ft} + 4.53\text{ kip/ft} = 44.08 \text{ kip/ft}$$

Check Sliding Resistance - Drained Condition

Nominal Sliding Resistance:

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



$$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV} = (120\text{ pcf})(39.2\text{ ft})(27.4\text{ ft})(1.00) = 128.89 \text{ kip/ft}$$

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

$$\tan\delta = \tan(26) \le \tan(34) \longrightarrow 0.49 \le 0.67 \longrightarrow \tan\delta = 0.49$$

$$R_\tau = (128.89\text{ kip/ft})(0.49) = 63.16 \text{ kip/ft}$$

Verify Sliding Force Less Than Factored Sliding Resistance - Drained Condition

$$P_H \le R_\tau \cdot \phi_\tau \longrightarrow 44.08\text{ kip/ft} \le (63.16\text{ kip/ft})(1.0) = 63.16\text{ kip/ft} \longrightarrow 44.08\text{ kip/ft} \le 63.16\text{ kip/ft}$$ **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) =	39.2	ft
MSE Wall Width (Reinforcement Length), (B) =	27.4	ft
MSE Wall Length, (L) =	229	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}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	2950	psf
Embedment Depth, (D_f) =	3.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	8.0	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

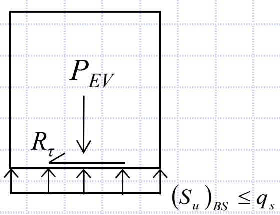
(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

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

Check Sliding Resistance - Undrained Condition

Nominal Sliding Resisting:

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



$(S_u)_{BS} =$ 2.95 ksf

$q_s = \sigma_v / 2$ = (4.70 ksf) / 2 = 2.35 ksf

$\sigma_v = P_{EV} / B$ = (128.89 kip/ft) / (27.4 ft) = 4.70 ksf

R_τ = (2.95 ksf ≤ 2.35 ksf)(27.4 ft) = 64.39 kip/ft

Verify Sliding Force Less Than Factored Sliding Resistance - Undrained Condition

$P_H \le R_\tau \cdot \phi_\tau$ ⟶ 44.08 kip/ft ≤ (64.39 kip/ft)(1.0) = 64.39 kip/ft ⟶ 44.08 kip/ft ≤ 64.39 kip/ft **OK**

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

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MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	39.2	ft
MSE Wall Width (Reinforcement Length), (B) =	27.4	ft
MSE Wall Length, (L) =	229	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:

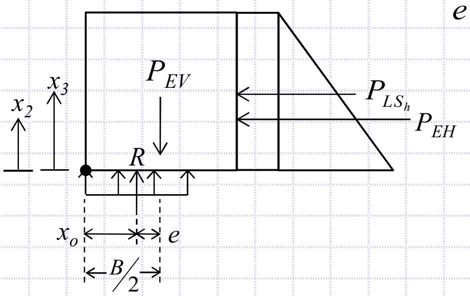
Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	2950	psf
Embedment Depth, (D_f) =	3.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	8.0	ft

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



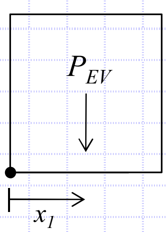
$$e = B/2 - x_o$$

$$x_o = \frac{M_{EV} - M_H}{P_{EV}}$$ = (1765.79 kip·ft/ft - 605.71 kip·ft/ft) / (128.89 kip/ft) = 9.00 ft

M_{EV} = 1765.79 kip·ft/ft
M_H = 605.71 kip·ft/ft
P_{EV} = 128.89 kip/ft
Defined below

$e =$ (27.4 ft)/2 - 9 ft = 4.70 ft

Resisting Moment, M_{EV}:



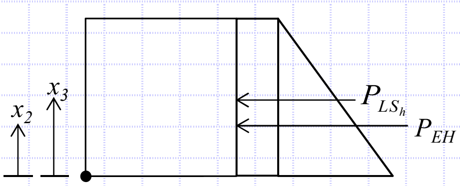
$$M_{EV} = P_{EV}(x_1)$$

$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV}$ = (120 pcf)(39.2 ft)(27.4 ft)(1.00) = 128.89 kip/ft

$x_1 = B/2$ = (27.4 ft) / 2 = 13.70 ft

M_{EV} = (128.89 kip/ft)(13.70 ft) = 1765.79 kip·ft/ft

Overturning Moment, M_H:



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

$P_{EH} = ½\gamma_{RS} H^2 K_a \gamma_{EH}$ = ½(130 pcf)(39.2 ft)²(0.264)(1.5) = 39.55 kip/ft

$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS}$ = (250 psf)(39.2 ft)(0.264)(1.75) = 4.53 kip/ft

$x_2 = H/3$ = (39.2 ft) / 3 = 13.07 ft

$x_3 = H/2$ = (39.2 ft) / 2 = 19.60 ft

M_H = (39.55 kip/ft)(13.07 ft) + (4.53 kip/ft)(19.60 ft) = 605.71 kip·ft/ft

Check Eccentricity

$e < e_{max} \longrightarrow$ 4.70 ft < 9.13 ft **OK**

Limiting Eccentricity: $e_{max} = B/3 \longrightarrow e_{max}$ = (27.4 ft) / 3 = 9.13 ft

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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 4W5 - Sta. 5083+25 to Sta. 5085+54

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	39.2	ft
MSE Wall Width (Reinforcement Length), (B) =	27.4	ft
MSE Wall Length, (L) =	229	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:

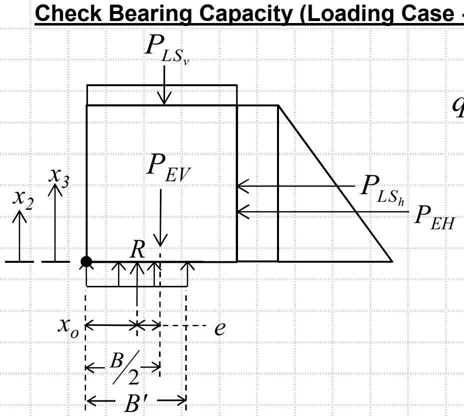
Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	2950	psf
Embedment Depth, (D_f) =	3.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	8.0	ft

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



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

$$B' = B - 2e \quad = \quad 27.4 \text{ ft} - 2(3.26 \text{ ft}) \quad = \quad 20.88 \text{ ft}$$

$$e = B/2 - x_o \quad = \quad (27.4 \text{ ft}) / 2 - 10.44 \text{ ft} \quad = \quad 3.26 \text{ ft}$$

$$x_o = \frac{M_V - M_H}{P_V} \quad = \quad (2548.04 \text{ kip·ft/ft} - 605.7 \text{ kip·ft/ft}) / 185.99 \text{ kip/ft} \quad = \quad 10.44 \text{ ft}$$

$$q_{eq} = \quad (185.99 \text{ kip/ft}) / (20.88 \text{ ft}) \quad = \quad 8.91 \text{ ksf}$$

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

$$M_V = \quad [(120 \text{ pcf})(39.2 \text{ ft})(27.4 \text{ ft})(1.35)](13.7 \text{ ft}) + [(250 \text{ psf})(27.4 \text{ ft})(1.75)](13.7 \text{ ft}) \quad = \quad 2548.04 \text{ kip·ft/ft}$$

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

$$M_H = \quad [\tfrac{1}{2}(130 \text{ pcf})(39.2 \text{ ft})^2(0.264)(1.5)](13.07 \text{ ft}) + [(250 \text{ psf})(39.2 \text{ ft})(0.264)(1.75)](19.6 \text{ ft}) \quad = \quad 605.70 \text{ kip·ft/ft}$$

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

$$P_V = \quad (120 \text{ pcf})(39.2 \text{ ft})(27.4 \text{ ft})(1.35) + (250 \text{ psf})(27.4 \text{ ft})(1.75) \quad = \quad 185.99 \text{ kip/ft}$$

Check Bearing Resistance - Drained Condition

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

$N_{cm} = N_c s_c i_c$ = 23.34

N_c = 22.25
s_c = 1+(20.88 ft/229 ft)(11.85/22.25)
= 1.049
i_c = 1.000 (Assumed)

$N_{qm} = N_q s_q d_q i_q$ = 12.92

N_q = 11.85
s_q = 1.044
d_q = 1+2tan(26°)[1-sin(26°)]²tan⁻¹(3.0 ft/20.88 ft)
1.044
i_q = 1.000 (Assumed)
C_{wq} = 8.0 ft > 3.0 ft = 1.000

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 12.09

N_γ = 12.54
s_γ = 0.964
i_γ = 1.000 (Assumed)
$C_{w\gamma}$ = 8.0 ft < 1.5(20.88 ft) + 3.0 ft = 0.628

$$q_n = \quad (0 \text{ psf})(23.340) + (120 \text{ pcf})(3.0 \text{ ft})(12.916)(1.000) + \tfrac{1}{2}(120 \text{ pcf})(20.9 \text{ ft})(12.089)(0.628) \quad = \quad 14.16 \text{ ksf}$$

Verify Equivalent Pressure Less Than Factored Bearing Resistance

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

$q_{eq} \le q_n \cdot \phi_b \longrightarrow$ 8.91 ksf ≤ (14.16 ksf)(0.65) = 9.20 ksf $\longrightarrow$ 8.91 ksf ≤ 9.20 ksf **OK**

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	39.2	ft
MSE Wall Width (Reinforcement Length), (B) =	27.4	ft
MSE Wall Length, (L) =	229	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}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	2950	psf
Embedment Depth, (D_f) =	3.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	8.0	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: $$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$$

$N_{cm} = N_c s_c i_c$ = 5.230

- N_c = 5.140
- s_c = 1+(20.88 ft/[(5)(229 ft)]) = 1.018
- 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(3.0 ft/20.88 ft) = 1.000
- i_q = 1.000 (Assumed)
- C_{wq} = 8.0 ft > 3.0 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)
- $C_{w\gamma}$ = 8.0 ft < 1.5(20.88 ft) + 3.0 ft = 0.628

$q_n =$ (2950 psf)(5.230) + (120 pcf)(3.0 ft)(1.000)(1.000) + ½(120 pcf)(20.9 ft)(0.000)(0.628) = 15.79 ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

$q_{eq} \le q_n \cdot \phi_b \longrightarrow$ 8.91 ksf ≤ (15.79 ksf)(0.65) = 10.26 ksf $\longrightarrow$ 8.91 ksf ≤ 10.26 ksf **OK**

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

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MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	39.2	ft
MSE Wall Width (Reinforcement Length), (B) =	27.4	ft
MSE Wall Length, (L) =	229	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:

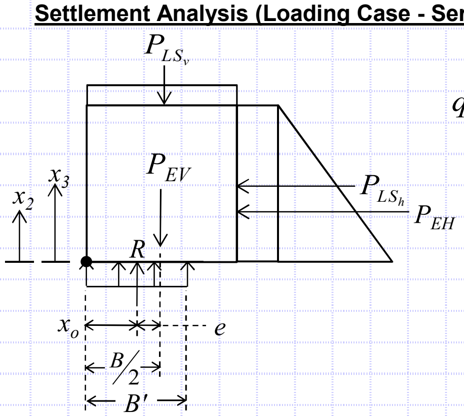
Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	2950	psf
Embedment Depth, (D_f) =	3.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	8.0	ft

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



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

$B' = B - 2e$ = 27.4 ft - 2(2.91 ft) = 21.58 ft

$e = B/2 - x_o$ = (27.4 ft) / 2 - 10.79 ft = 2.91 ft

$x_o = \dfrac{M_V - M_H}{P_V}$ = (1859.63 kip·ft/ft - 395.35 kip·ft/ft) / 135.74 kip/ft = 10.79 ft

$q_{eq} =$ (135.74 kip/ft) / (21.58 ft) = 6.29 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)(39.2 ft)(27.4 ft)(1.00)](13.7 ft) + [(250 psf)(27.4 ft)(1.00)](13.7 ft) = 1859.63 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)(39.2 ft)²(0.264)(1.00)](13.07 ft) + [(250 psf)(39.2 ft)(0.264)(1.00)](19.6 ft) = 395.35 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)(39.2 ft)(27.4 ft)(1.00) + (250 psf)(27.4 ft)(1.00) = 135.74 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-023-1-13	5.443 in	2.372 in	11 days		
B-024-0-08	3.563 in	2.226 in	35 days	125 ft	1/10270

W-13-045 - FRA-70-12.68 - Retaining Wall 4W5 - Sta. 5083+25 to Sta. 5085+54

Settlement - MSE Wall

Calculated By: BRT Date: 7/15/2018

Checked By: JPS Date: 7/15/2018

Boring B-023-1-13

H=	39.2	ft	Total wall height
B'=	21.6	ft	Effective footing width due to eccentricity
D_w =	7.5	ft	Depth below bottom of footing
q_e =	6,290	psf	Equivalent bearing pressure at bottom of wall

																				Total Settlement at Center of Reinforced 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-6b	C	0.0	3.0	3.0	1.5	120	360	180	180	4,180	29	0.171	0.017	0.499				0.07	0.999	6,283	6,463	0.112	1.338	0.500	3,145	3,325	0.043	0.520
2	A-7-6	C	3.0	5.5	2.5	4.3	120	660	510	510	4,510	48	0.342	0.034	0.647				0.20	0.978	6,153	6,663	0.137	1.645	0.498	3,135	3,645	0.044	0.532
	A-7-6	C	5.5	8.0	2.5	6.8	120	960	810	810	4,810	48	0.342	0.034	0.647				0.31	0.931	5,853	6,663	0.114	1.363	0.494	3,109	3,919	0.036	0.426
3	A-1-b	G	8.0	12.0	4.0	10.0	130	1,480	1,220	1,064	5,064					33	40	131	0.46	0.842	5,295	6,359	0.024	0.285	0.483	3,040	4,104	0.018	0.215
4	A-1-b	G	12.0	19.5	7.5	15.8	135	2,493	1,986	1,471	5,471					53	59	208	0.73	0.680	4,275	5,746	0.021	0.256	0.451	2,837	4,308	0.017	0.202
	A-1-b	G	19.5	27.0	7.5	23.3	135	3,505	2,999	2,016	6,016					53	53	182	1.08	0.520	3,271	5,287	0.017	0.207	0.397	2,497	4,513	0.014	0.173
5	A-6b	C	27.0	32.0	5.0	29.5	130	4,155	3,830	2,457	6,457	36	0.234	0.023	0.553				1.37	0.429	2,698	5,155	0.024	0.291	0.353	2,220	4,677	0.021	0.253
6	A-1-a	G	32.0	38.0	6.0	35.0	135	4,965	4,560	2,844	6,844					91	80	331	1.62	0.370	2,327	5,171	0.005	0.057	0.318	2,002	4,846	0.004	0.050
																					Total Settlement:			5.443 in		Total Settlement:			2.372 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 (Cohesive 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 4W5 - Sta. 5083+25 to Sta. 5085+54

Settlement - MSE Wall

Calculated By: BRT Date: 07/15/2018

Checked By: JPS Date: 07/15/2018

Boring B-023-1-13

H=	39.2	ft	Total wall height
B'=	21.6	ft	Effective footing width due to eccentricity
D_w =	7.5	ft	Depth below bottom of footing
q_e =	6,290	psf	Equivalent bearing pressure at bottom of wall

	Nat. A-6b	Nat. A-7-6			
c_v =	300	150		ft^2/yr	Coefficient of consolidation
t =	11	11		days	Time following completion of construction
H_{dr} =	2.5	4.0		ft	Length of longest drainage path considered
T_v =	1.447	0.283			Time factor
U =	98	60		%	Degree of consolidation

$(S_c)_t$ = 2.145 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-6b	C	0.0	3.0	3.0	1.5	120	360	180	180	4,180	29	0.171	0.017	0.499				0.07	0.500	3,145	3,325	0.043	0.520	0.520	0.510	0.510
2	A-7-6	C	3.0	5.5	2.5	4.3	120	660	510	510	4,510	48	0.342	0.034	0.647				0.20	0.498	3,135	3,645	0.044	0.532	0.958	0.319	0.746
	A-7-6	C	5.5	8.0	2.5	6.8	120	960	810	810	4,810	48	0.342	0.034	0.647				0.31	0.494	3,109	3,919	0.036	0.426		0.426	
3	A-1-b	G	8.0	12.0	4.0	10.0	130	1,480	1,220	1,064	5,064					33	40	131	0.46	0.483	3,040	4,104	0.018	0.215	0.215	0.215	0.215
4	A-1-b	G	12.0	19.5	7.5	15.8	135	2,493	1,986	1,471	5,471					53	59	208	0.73	0.451	2,837	4,308	0.017	0.202	0.375	0.202	0.371
	A-1-b	G	19.5	27.0	7.5	23.3	135	3,505	2,999	2,016	6,016					53	53	182	1.08	0.397	2,497	4,513	0.014	0.173		0.170	
5	A-6b	C	27.0	32.0	5.0	29.5	130	4,155	3,830	2,457	6,457	36	0.234	0.023	0.553				1.37	0.353	2,220	4,677	0.021	0.253	0.253	0.253	0.253
6	A-1-a	G	32.0	38.0	6.0	35.0	135	4,965	4,560	2,844	6,844					91	80	331	1.62	0.318	2,002	4,846	0.004	0.050	0.050	0.050	0.050

Settlement Remaining After Hold Period: 0.227 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 (Cohesive 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 4W5 - Sta. 5083+25 to Sta. 5085+54
Settlement - MSE Wall

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

Boring B-024-0-13

H=	30.1	ft	Total wall height
B=	16.5	ft	Effective footing width due to eccentricity
D_w =	20.0	ft	Depth below bottom of footing
q_e =	4,930	psf	Equivalent bearing pressure at bottom of wall

																				Total Settlement at Center of Reinforced 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-6b	C	0.0	3.0	3.0	1.5	125	375	188	188	4,188	38	0.252	0.025	0.569				0.09	0.998	4,918	5,105	0.106	1.278	0.500	2,464	2,652	0.055	0.665
	A-6b	C	3.0	6.5	3.5	4.8	125	813	594	594	4,594	38	0.252	0.025	0.569				0.29	0.943	4,648	5,241	0.082	0.986	0.495	2,442	3,036	0.040	0.478
2	A-4a	C	6.5	9.0	2.5	7.8	120	1,113	963	963	4,963	21	0.099	0.010	0.436				0.47	0.838	4,129	5,092	0.014	0.170	0.483	2,380	3,342	0.009	0.112
	A-4a	C	9.0	11.5	2.5	10.3	120	1,413	1,263	1,263	5,263	21	0.099	0.010	0.436				0.62	0.742	3,660	4,923	0.010	0.122	0.466	2,296	3,559	0.008	0.093
	A-4a	C	11.5	14.0	2.5	12.8	120	1,713	1,563	1,563	5,563	21	0.099	0.010	0.436				0.77	0.656	3,234	4,796	0.008	0.101	0.445	2,192	3,754	0.007	0.079
	A-4a	C	14.0	16.5	2.5	15.3	120	2,013	1,863	1,863	5,863	21	0.099	0.010	0.436				0.92	0.582	2,870	4,732	0.007	0.084	0.421	2,076	3,939	0.006	0.067
	A-4a	C	16.5	19.0	2.5	17.8	120	2,313	2,163	2,163	6,163	21	0.099	0.010	0.436				1.08	0.520	2,565	4,728	0.006	0.070	0.397	1,958	4,120	0.005	0.058
3	A-4b	G	19.0	23.0	4.0	21.0	130	2,833	2,573	2,510	6,510					28	26	49	1.27	0.455	2,243	4,753	0.023	0.274	0.367	1,807	4,318	0.019	0.232
4	A-1-a	G	23.0	28.0	5.0	25.5	135	3,508	3,170	2,827	6,827					37	33	107	1.55	0.386	1,901	4,728	0.010	0.125	0.328	1,617	4,444	0.009	0.110
5	A-4a	C	28.0	33.0	5.0	30.5	130	4,158	3,833	3,177	7,177	21	0.099	0.010	0.436				1.85	0.329	1,620	4,797	0.006	0.074	0.291	1,435	4,612	0.006	0.067
6	A-3a	G	33.0	38.0	5.0	35.5	135	4,833	4,495	3,528	7,528					72	58	168	2.15	0.286	1,409	4,936	0.004	0.052	0.260	1,283	4,810	0.004	0.048
7	A-4a	G	38.0	41.0	3.0	39.5	135	5,238	5,035	3,818	7,818					58	46	78	2.39	0.258	1,274	5,093	0.005	0.058	0.239	1,179	4,997	0.004	0.054
8	A-1-b	G	41.0	48.0	7.0	44.5	135	6,183	5,710	4,181	8,181					100	76	300	2.70	0.231	1,138	5,319	0.002	0.029	0.217	1,069	5,250	0.002	0.028
9	A-4a	G	48.0	53.0	5.0	50.5	135	6,858	6,520	4,617	8,617					72	52	88	3.06	0.204	1,008	5,624	0.005	0.059	0.194	959	5,576	0.005	0.056
10	A-1-a	G	53.0	63.0	10.0	58.0	135	8,208	7,533	5,161	9,161					100	68	259	3.52	0.179	881	6,042	0.003	0.032	0.172	848	6,009	0.003	0.031
	A-1-a	G	63.0	73.0	10.0	68.0	135	9,558	8,883	5,887	9,887					100	64	236	4.12	0.153	754	6,641	0.002	0.027	0.149	733	6,620	0.002	0.026
	A-1-a	G	73.0	83.0	10.0	78.0	135	10,908	10,233	6,613	10,613					100	60	216	4.73	0.134	659	7,272	0.002	0.023	0.131	645	7,258	0.002	0.022
																						Total Settlement:		3.563 in			Total Settlement:		2.226 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 $C_N = [0.77\log(40/\sigma_{vo}')] \le 2.0$ 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 (Cohesive 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 4W5 - Sta. 5083+25 to Sta. 5085+54
Settlement - MSE Wall

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

Boring B-024-0-13

H=	30.1	ft	Total wall height
B'=	16.5	ft	Effective footing width due to eccentricity
D_w =	20.0	ft	Depth below bottom of footing
q_e =	4,930	psf	Equivalent bearing pressure at bottom of wall

	Nat. A-6b	Nat. A-4a		
c_v =	300	800	ft^2/yr	Coefficient of consolidation
t =	35	35	days	Time following completion of construction
H_{dr} =	6.5	9.5	ft	Length of longest drainage path considered
T_v =	0.681	0.850		Time factor
U =	85	90	%	Degree of consolidation

$(S_c)_t$ = 2.007 in Settlement complete at 90% of primary consolidation

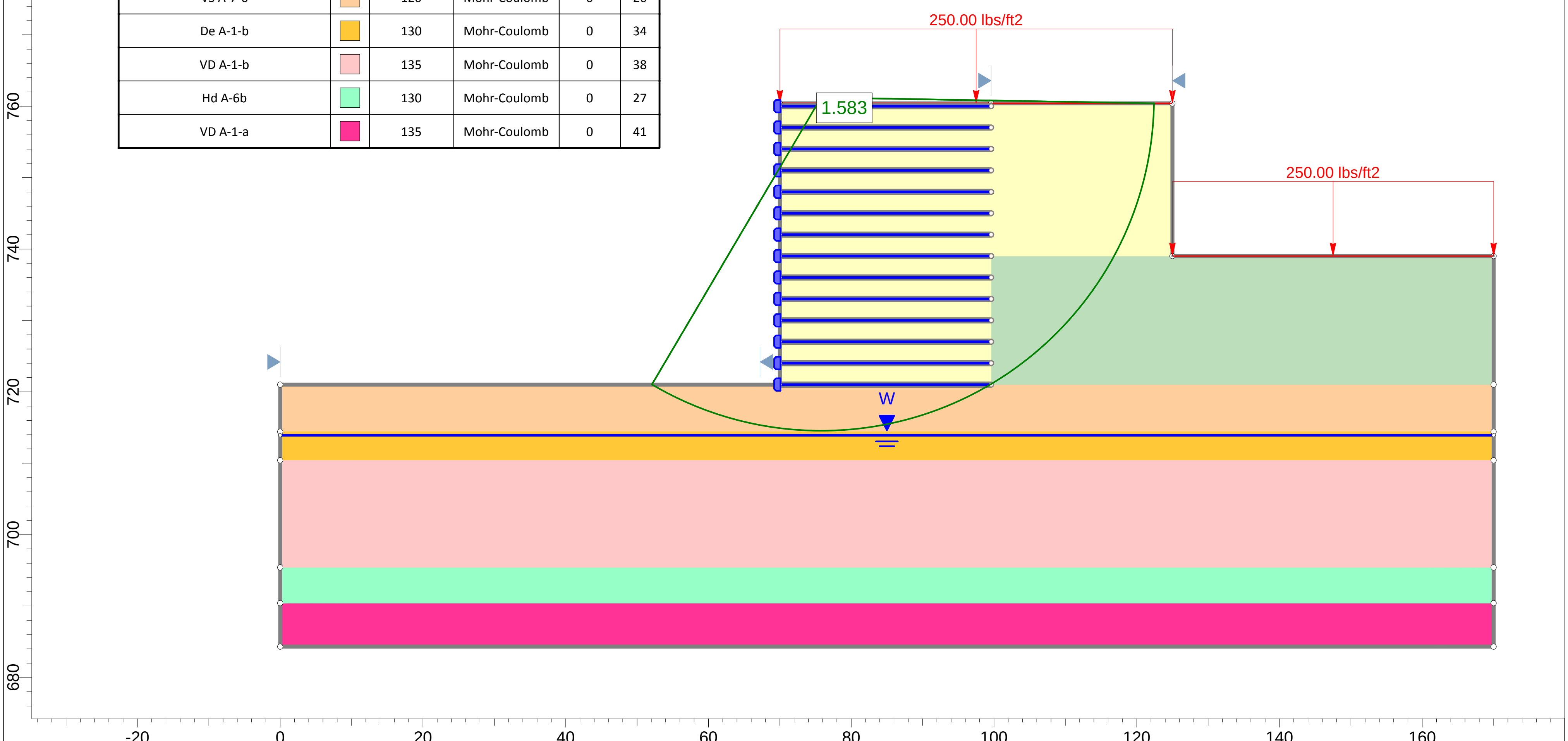
Layer	Soil Type	Soil Type	Layer Depth (ft)		Layer Thickness (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	Total Settlement at Facing of Wall: S_c (9,10) (ft)	Total Settlement at Facing of Wall: S_c (in)	Total Settlement at Facing of Wall: Layer Settlement (in)	Settlement Complete at 90% of Primary Consolidation: $(S_c)_t$ (11) (in)	Settlement Complete at 90% of Primary Consolidation: Layer Settlement (in)
1	A-6b	C	0.0	3.0	3.0	1.5	125	375	188	188	4,188	38	0.252	0.025	0.569				0.09	0.500	2,464	2,652	0.055	0.665	1.143	0.565	0.972
	A-6b	C	3.0	6.5	3.5	4.8	125	813	594	594	4,594	38	0.252	0.025	0.569				0.29	0.495	2,442	3,036	0.040	0.478		0.406	
2	A-4a	C	6.5	9.0	2.5	7.8	120	1,113	963	963	4,963	21	0.099	0.010	0.436				0.47	0.483	2,380	3,342	0.009	0.112	0.409	0.101	0.368
	A-4a	C	9.0	11.5	2.5	10.3	120	1,413	1,263	1,263	5,263	21	0.099	0.010	0.436				0.62	0.466	2,296	3,559	0.008	0.093		0.084	
	A-4a	C	11.5	14.0	2.5	12.8	120	1,713	1,563	1,563	5,563	21	0.099	0.010	0.436				0.77	0.445	2,192	3,754	0.007	0.079		0.071	
	A-4a	C	14.0	16.5	2.5	15.3	120	2,013	1,863	1,863	5,863	21	0.099	0.010	0.436				0.92	0.421	2,076	3,939	0.006	0.067		0.061	
	A-4a	C	16.5	19.0	2.5	17.8	120	2,313	2,163	2,163	6,163	21	0.099	0.010	0.436				1.08	0.397	1,958	4,120	0.005	0.058		0.052	
3	A-4b	G	19.0	23.0	4.0	21.0	130	2,833	2,573	2,510	6,510					28	26	49	1.27	0.367	1,807	4,318	0.019	0.232	0.232	0.232	0.232
4	A-1-a	G	23.0	28.0	5.0	25.5	135	3,508	3,170	2,827	6,827					37	33	107	1.55	0.328	1,617	4,444	0.009	0.110	0.110	0.110	0.110
5	A-4a	C	28.0	33.0	5.0	30.5	130	4,158	3,833	3,177	7,177	21	0.099	0.010	0.436				1.85	0.291	1,435	4,612	0.006	0.067	0.067	0.060	0.060
6	A-3a	G	33.0	38.0	5.0	35.5	135	4,833	4,495	3,528	7,528					72	58	168	2.15	0.260	1,283	4,810	0.004	0.048	0.048	0.048	0.048
7	A-4a	G	38.0	41.0	3.0	39.5	135	5,238	5,035	3,818	7,818					58	46	78	2.39	0.239	1,179	4,997	0.004	0.054	0.054	0.054	0.054
8	A-1-b	G	41.0	48.0	7.0	44.5	135	6,183	5,710	4,181	8,181					100	76	300	2.70	0.217	1,069	5,250	0.002	0.028	0.028	0.028	0.028
9	A-4a	G	48.0	53.0	5.0	50.5	135	6,858	6,520	4,617	8,617					72	52	88	3.06	0.194	959	5,576	0.005	0.056	0.056	0.056	0.056
10	A-1-a	G	53.0	63.0	10.0	58.0	135	8,208	7,533	5,161	9,161					100	68	259	3.52	0.172	848	6,009	0.003	0.031	0.079	0.031	0.079
	A-1-a	G	63.0	73.0	10.0	68.0	135	9,558	8,883	5,887	9,887					100	64	236	4.12	0.149	733	6,620	0.002	0.026		0.026	
	A-1-a	G	73.0	83.0	10.0	78.0	135	10,908	10,233	6,613	10,613					100	60	216	4.73	0.131	645	7,258	0.002	0.022		0.022	

Settlement Remaining After Hold Period: 0.219 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. $S_c = [C_r/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ 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 (Cohesive 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)	Strength Type	Cohesion (psf)	Phi (deg)
MSE Select Granular Backfill		120	Mohr-Coulomb	0	34
Item 203 Granular Embankment		130	Mohr-Coulomb	0	33
VS A-7-6		120	Mohr-Coulomb	0	26
De A-1-b		130	Mohr-Coulomb	0	34
VD A-1-b		135	Mohr-Coulomb	0	38
Hd A-6b		130	Mohr-Coulomb	0	27
VD A-1-a		135	Mohr-Coulomb	0	41

Support Name	Color	Type	Strip Coverage (%)	Tensile Strength (lbs/ft)
MSE Reinforcement Straps		GeoTextile	100	15000

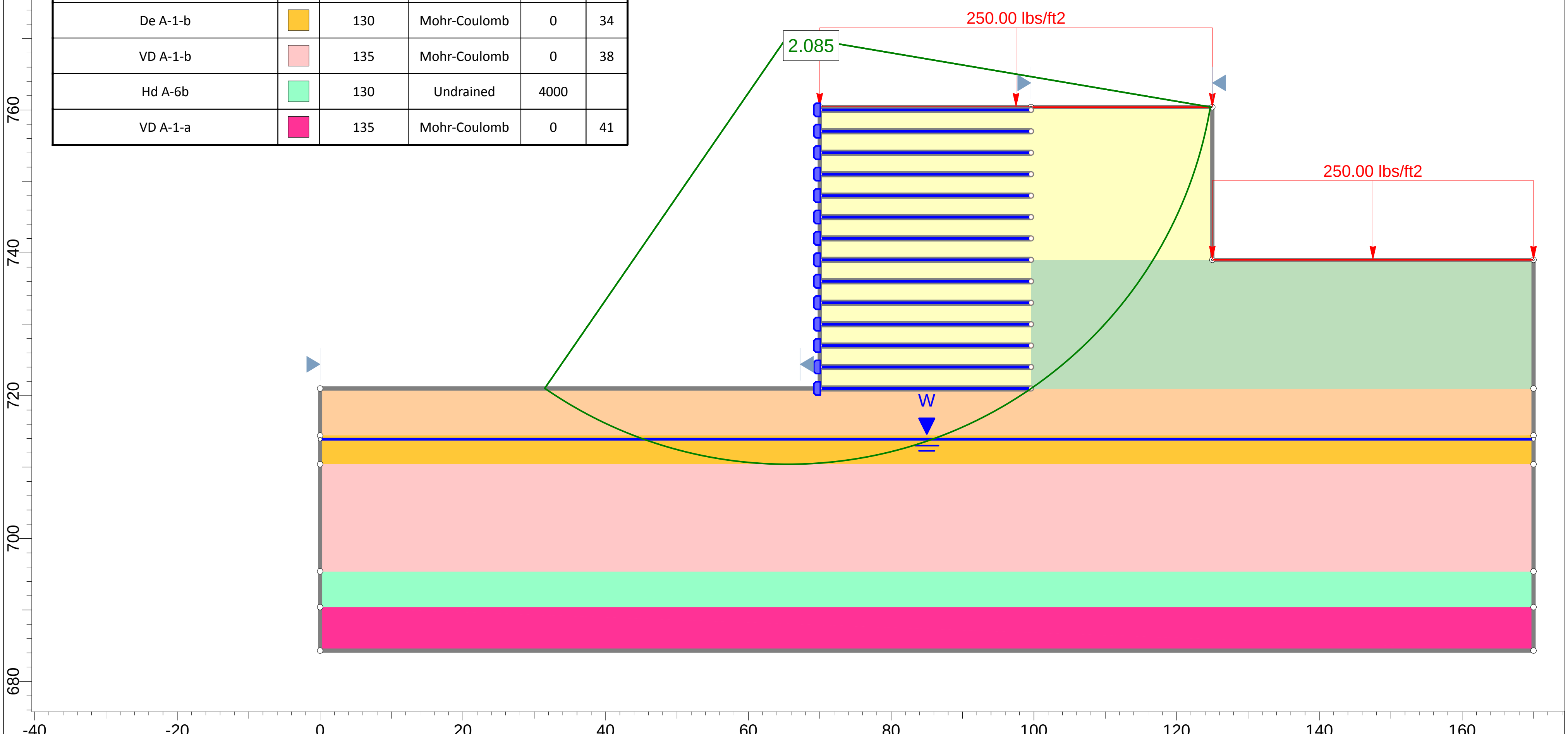


Project	FRA-70-12.68 - Retaining Wall 4W5 - Sta. 5083+25 to Sta. 5085+54
Analysis Description	B-003-A-59, B-023-1-13, B-024-0-08 - 39.4 ft. Wall Height - Drained - Spencer's
Drawn By	BRT
Scale	1:250
Company	Resource International, Inc.
Date	11/30/2015, 5:20:00 PM
File Name	Retaining Wall 4W5 - Sta. 5083+25 to 5085+54 - Global Stability.slim

SLIDEINTERPRET 6.031

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
MSE Select Granular Backfill		120	Mohr-Coulomb	0	34
Item 203 Granular Embankment		130	Mohr-Coulomb	0	33
VS A-7-6		120	Undrained	2950	
De A-1-b		130	Mohr-Coulomb	0	34
VD A-1-b		135	Mohr-Coulomb	0	38
Hd A-6b		130	Undrained	4000	
VD A-1-a		135	Mohr-Coulomb	0	41

Support Name	Color	Type	Strip Coverage (%)	Tensile Strength (lbs/ft)
MSE Reinforcement Straps		GeoTextile	100	15000



Project	FRA-70-12.68 - Retaining Wall 4W5 - Sta. 5083+25 to Sta. 5085+54
Analysis Description	B-003-A-59, B-023-1-13, B-024-0-08 - 39.4 ft. Wall Height - Undrained - Spencer's
Drawn By	BRT
Scale	1:250
Company	Resource International, Inc.
Date	11/30/2015, 5:20:00 PM
File Name	Retaining Wall 4W5 - Sta. 5083+25 to 5085+54 - Global Stability.slim

rocscience

SLIDEINTERPRET 6.031

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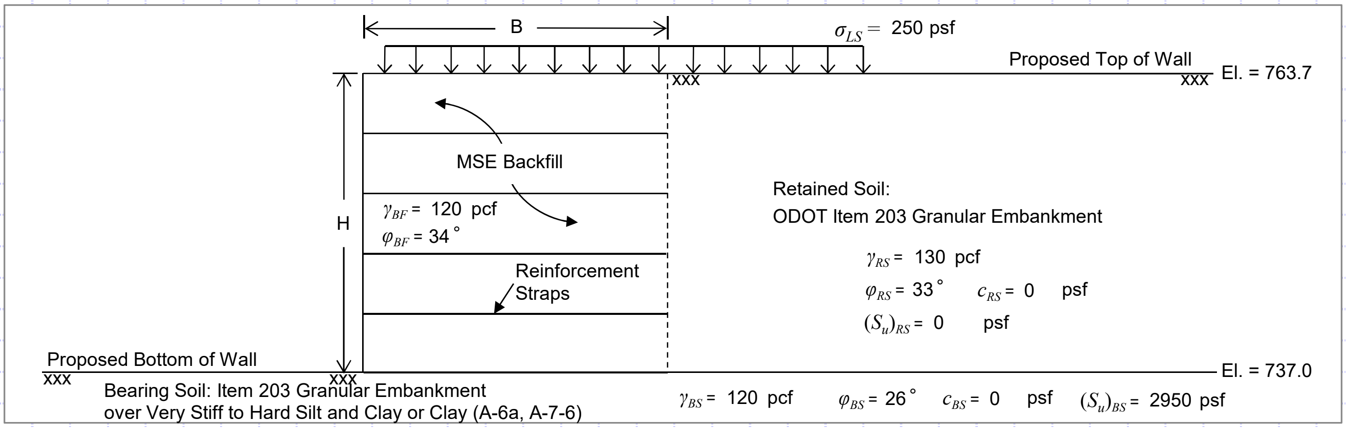
FRA-70-12.68 - Retaining Wall 4W6 - Sta. 178+85 to Sta. 180+30 - B-003-A-59 and B-023-1-13 - 26.7 ft. Wall Height



MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	26.7	ft
MSE Wall Width (Reinforcement Length), (B) =	18.7	ft
MSE Wall Length, (L) =	145	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}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(S_u)_{BS}]$ =	2950	psf
Embedment Depth, (D_f) =	3.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	22.0	ft

LRFD Load Factors

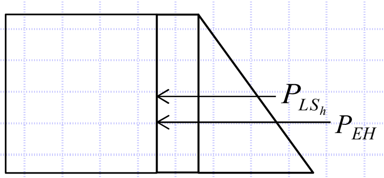
	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

(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:

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



$$P_{EH} = \tfrac{1}{2}\gamma_{RS} H^2 K_a \gamma_{EH} = \text{½(130 pcf)(26.7 ft)²(0.264)(1.5)} = 18.35 \text{ kip/ft}$$

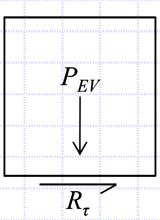
$$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS} = \text{(250 psf)(26.7 ft)(0.264)(1.75)} = 3.08 \text{ kip/ft}$$

$$P_H = \text{18.35 kip/ft + 3.08 kip/ft} = 21.43 \text{ kip/ft}$$

Check Sliding Resistance - Drained Condition

Nominal Sliding Resistance:

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



$$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV} = \text{(120 pcf)(26.7 ft)(18.7 ft)(1.00)} = 59.91 \text{ kip/ft}$$

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

$$\tan\delta = \tan(26) \le \tan(34) \longrightarrow 0.49 \le 0.67 \longrightarrow \tan\delta = 0.49$$

$$R_\tau = \text{(59.91 kip/ft)(0.49)} = 29.36 \text{ kip/ft}$$

Verify Sliding Force Less Than Factored Sliding Resistance - Drained Condition

$$P_H \le R_\tau \cdot \phi_\tau \longrightarrow \text{21.43 kip/ft} \le \text{(29.36 kip/ft)(1.0) = 29.36 kip/ft} \longrightarrow \text{21.43 kip/ft} \le \text{29.36 kip/ft}$$

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) =	26.7	ft
MSE Wall Width (Reinforcement Length), (B) =	18.7	ft
MSE Wall Length, (L) =	145	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}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	2950	psf
Embedment Depth, (D_f) =	3.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	22.0	ft

LRFD Load Factors

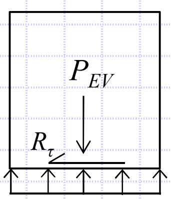
	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

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



$(S_u)_{BS} =$ 2.95 ksf

$q_s = \sigma_v / 2$ = (3.20 ksf) / 2 = 1.60 ksf

$\sigma_v = P_{EV} / B$ = (59.91 kip/ft) / (18.7 ft) = 3.20 ksf

R_τ = (2.95 ksf ≤ 1.60 ksf)(18.7 ft) = 29.92 kip/ft

Verify Sliding Force Less Than Factored Sliding Resistance - Undrained Condition

$P_H \le R_\tau \cdot \phi_\tau$ $\longrightarrow$ 21.43 kip/ft ≤ (29.92 kip/ft)(1.0) = 29.92 kip/ft $\longrightarrow$ 21.43 kip/ft ≤ 29.92 kip/ft **OK**

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

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MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	26.7	ft
MSE Wall Width (Reinforcement Length), (B) =	18.7	ft
MSE Wall Length, (L) =	145	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:

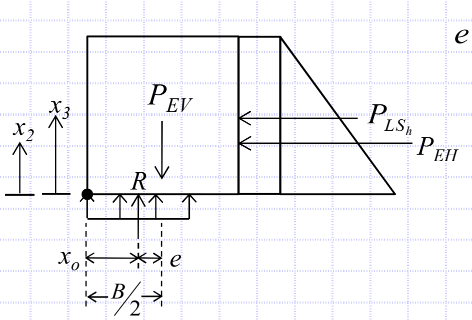
Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	2950	psf
Embedment Depth, (D_f) =	3.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	22.0	ft

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



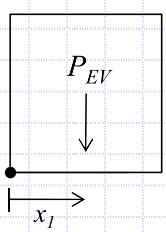
$$e = B/2 - x_o$$

$$x_o = \frac{M_{EV} - M_H}{P_{EV}}$$ = (560.16 kip·ft/ft - 204.43 kip·ft/ft) / (59.91 kip/ft) = 5.94 ft

M_{EV} = 560.16 kip·ft/ft
M_H = 204.43 kip·ft/ft
P_{EV} = 59.91 kip/ft
Defined below

$e =$ (18.7 ft)/2 - 5.94 ft = 3.41 ft

Resisting Moment, M_{EV}:



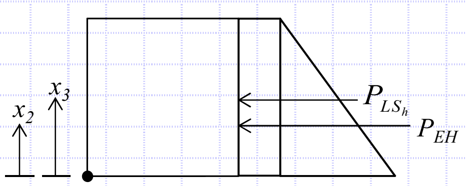
$$M_{EV} = P_{EV}(x_1)$$

$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV}$ = (120 pcf)(26.7 ft)(18.7 ft)(1.00) = 59.91 kip/ft

$x_1 = B/2$ = (18.7 ft) / 2 = 9.35 ft

$M_{EV} =$ (59.91 kip/ft)(9.35 ft) = 560.16 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)(26.7 ft)²(0.264)(1.5) = 18.35 kip/ft

$P_{LS_h} = \sigma_{LS}HK_a\gamma_{LS}$ = (250 psf)(26.7 ft)(0.264)(1.75) = 3.08 kip/ft

$x_2 = H/3$ = (26.7 ft) / 3 = 8.90 ft

$x_3 = H/2$ = (26.7 ft) / 2 = 13.35 ft

$M_H =$ (18.35 kip/ft)(8.9 ft) + (3.08 kip/ft)(13.35 ft) = 204.43 kip·ft/ft

Check Eccentricity

$e < e_{max} \longrightarrow$ 3.41 ft < 6.23 ft **OK**

Limiting Eccentricity: $e_{max} = B/3 \longrightarrow e_{max} =$ (18.7 ft) / 3 = 6.23 ft

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Retaining Wall 4W6 - Sta. 178+85 to Sta. 180+30

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	26.7	ft
MSE Wall Width (Reinforcement Length), (B) =	18.7	ft
MSE Wall Length, (L) =	145	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:

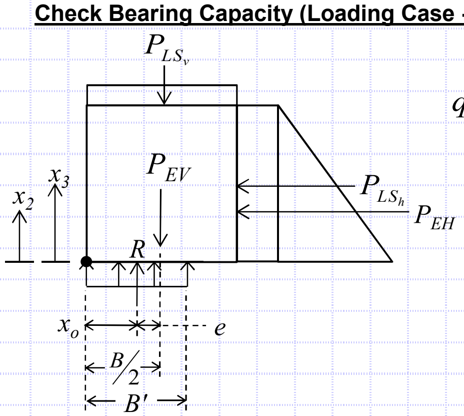
Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	2950	psf
Embedment Depth, (D_f) =	3.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	22.0	ft

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



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

$$B' = B - 2e \quad = \quad 18.7\text{ ft} - 2(2.3\text{ ft}) \quad = \quad 14.10 \quad \text{ft}$$

$$e = B/2 - x_o \quad = \quad (18.7\text{ ft}) / 2 - 7.05\text{ ft} \quad = \quad 2.30 \quad \text{ft}$$

$$x_o = \frac{M_V - M_H}{P_V} \quad = \quad (832.77\text{ kip·ft/ft} - 204.48\text{ kip·ft/ft}) / 89.07\text{ kip/ft} \quad = \quad 7.05 \quad \text{ft}$$

$$q_{eq} = \quad (89.07\text{ kip/ft}) / (14.1\text{ ft}) \quad = \quad 6.32 \quad \text{ksf}$$

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

$$M_V = \quad [(120\text{ pcf})(26.7\text{ ft})(18.7\text{ ft})(1.35)](9.35\text{ ft}) + [(250\text{ psf})(18.7\text{ ft})(1.75)](9.35\text{ ft}) \quad = \quad 832.77 \quad \text{kip·ft/ft}$$

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

$$M_H = \quad [\tfrac{1}{2}(130\text{ pcf})(26.7\text{ ft})^2(0.264)(1.5)](8.9\text{ ft}) + [(250\text{ psf})(26.7\text{ ft})(0.264)(1.75)](13.35\text{ ft}) \quad = \quad 204.48 \quad \text{kip·ft/ft}$$

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

$$P_V = \quad (120\text{ pcf})(26.7\text{ ft})(18.7\text{ ft})(1.35) + (250\text{ psf})(18.7\text{ ft})(1.75) \quad = \quad 89.07 \quad \text{kip/ft}$$

Check Bearing Resistance - Drained Condition

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

$N_{cm} = N_c s_c i_c$ = 23.41

- N_c = 22.25
- s_c = 1+(14.1 ft/145 ft)(11.85/22.25)
- = 1.052
- i_c = 1.000 (Assumed)

$N_{qm} = N_q s_q d_q i_q$ = 13.21

- N_q = 11.85
- s_q = 1.047
- d_q = 1+2tan(26°)[1-sin(26°)]²tan⁻¹(3.0 ft/14.1 ft)
- 1.065
- i_q = 1.000 (Assumed)
- C_{wq} = 22.0 ft > 3.0 ft = 1.000

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 12.05

- N_γ = 12.54
- s_γ = 0.961
- i_γ = 1.000 (Assumed)
- $C_{w\gamma}$ = 22.0 ft < 1.5(14.1 ft) + 3.0 ft = 1.020

$$q_n = \quad (0\text{ psf})(23.407) + (120\text{ pcf})(3.0\text{ ft})(13.213)(1.000) + \tfrac{1}{2}(120\text{ pcf})(14.1\text{ ft})(12.051)(1.020) \quad = \quad 15.16 \quad \text{ksf}$$

Verify Equivalent Pressure Less Than Factored Bearing Resistance

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

$q_{eq} \le q_n \cdot \phi_b \longrightarrow$ 6.32 ksf ≤ (15.16 ksf)(0.65) = 9.85 ksf $\longrightarrow$ 6.32 ksf ≤ 9.85 ksf **OK**

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	26.7	ft
MSE Wall Width (Reinforcement Length), (B) =	18.7	ft
MSE Wall Length, (L) =	145	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}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	2950	psf
Embedment Depth, (D_f) =	3.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	22.0	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: $$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$$

$N_{cm} = N_c s_c i_c$ = 5.240

- N_c = 5.140
- s_c = 1+(14.1 ft/[(5)(145 ft)]) = 1.019
- 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(3.0 ft/14.1 ft) = 1.000
- i_q = 1.000 (Assumed)
- C_{wq} = 22.0 ft > 3.0 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)
- $C_{w\gamma}$ = 22.0 ft < 1.5(14.1 ft) + 3.0 ft = 1.020

q_n = (2950 psf)(5.240) + (120 pcf)(3.0 ft)(1.000)(1.000) + ½(120 pcf)(14.1 ft)(0.000)(1.020) = 15.82 ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

$q_{eq} \le q_n \cdot \phi_b$ ⟶ 6.32 ksf ≤ (15.82 ksf)(0.65) = 10.28 ksf ⟶ 6.32 ksf ≤ 10.28 ksf **OK**

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

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	26.7	ft
MSE Wall Width (Reinforcement Length), (B) =	18.7	ft
MSE Wall Length, (L) =	145	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_{RS}) =	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:

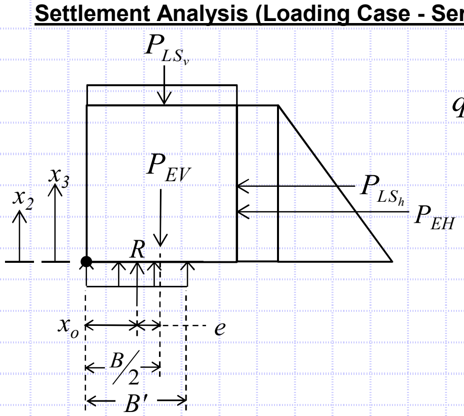
Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	2950	psf
Embedment Depth, (D_f) =	3.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	22.0	ft

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



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

$$B' = B - 2e \quad = \quad \text{18.7 ft - 2(2.05 ft)} \quad = \quad 14.60 \quad \text{ft}$$

$$e = B/2 - x_o \quad = \quad \text{(18.7 ft) / 2 - 7.3 ft} \quad = \quad 2.05 \quad \text{ft}$$

$$x_o = \frac{M_V - M_H}{P_V} \quad = \quad \text{(603.91 kip·ft/ft - 132.4 kip·ft/ft) / 64.59 kip/ft} \quad = \quad 7.3 \quad \text{ft}$$

$$q_{eq} = \quad \text{(64.59 kip/ft) / (14.6 ft)} \quad = \quad 4.42 \quad \text{ksf}$$

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

$$M_V = \quad \text{[(120 pcf)(26.7 ft)(18.7 ft)(1.00)](9.4 ft) + [(250 psf)(18.7 ft)(1.00)](9.4 ft)} \quad = \quad 603.91 \quad \text{kip·ft/ft}$$

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

$$M_H = \quad \text{[½(130 pcf)(26.7 ft)²(0.264)(1.00)](8.9 ft) + [(250 psf)(26.7 ft)(0.264)(1.00)](13.35 ft)} \quad = \quad 132.40 \quad \text{kip·ft/ft}$$

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

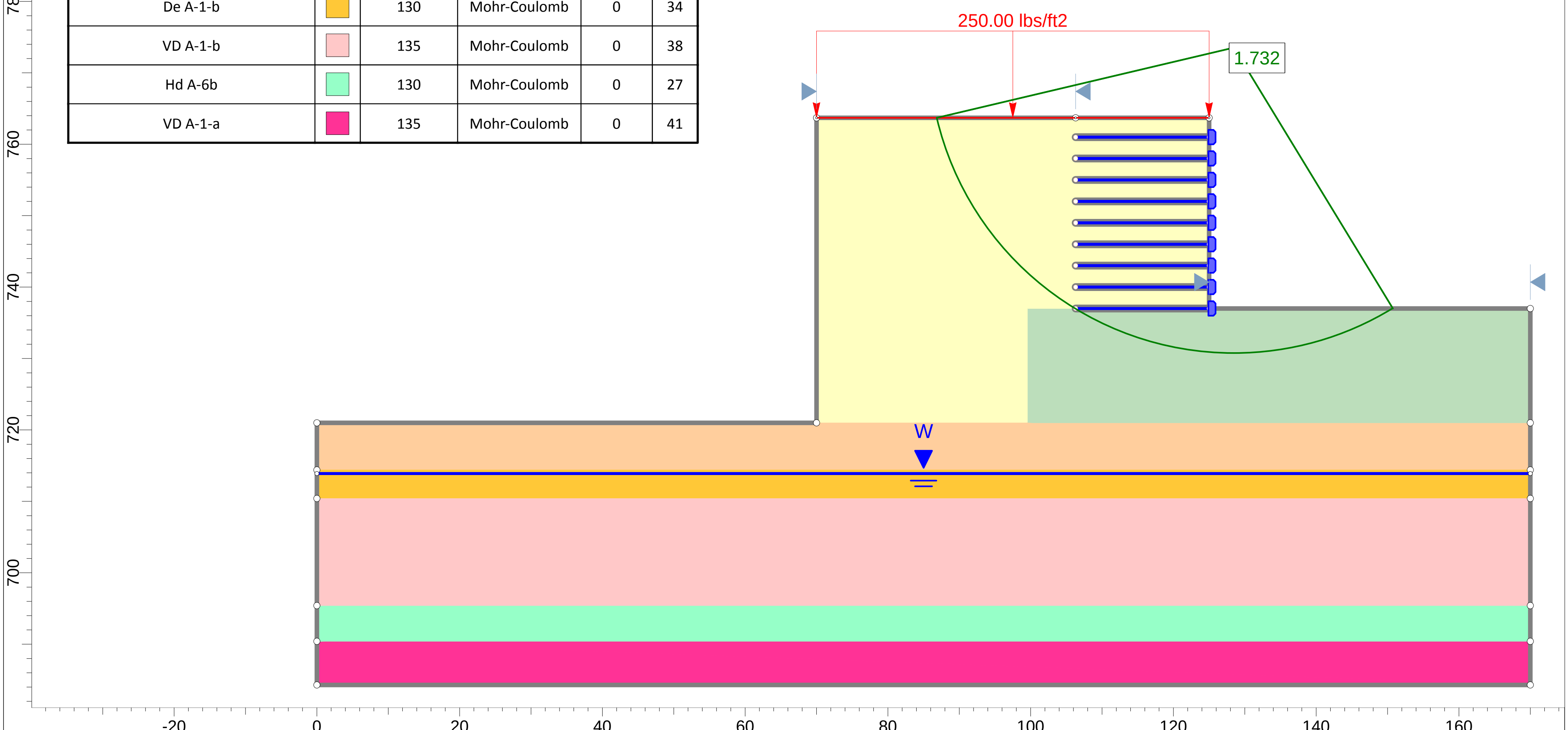
$$P_V = \quad \text{(120 pcf)(26.7 ft)(18.7 ft)(1.00) + (250 psf)(18.7 ft)(1.00)} \quad = \quad 64.59 \quad \text{kip/ft}$$

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

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
MSE Select Granular Backfill		120	Mohr-Coulomb	0	34
Item 203 Granular Embankment		130	Mohr-Coulomb	0	33
VS A-7-6		120	Mohr-Coulomb	0	26
De A-1-b		130	Mohr-Coulomb	0	34
VD A-1-b		135	Mohr-Coulomb	0	38
Hd A-6b		130	Mohr-Coulomb	0	27
VD A-1-a		135	Mohr-Coulomb	0	41

Support Name	Color	Type	Strip Coverage (%)	Tensile Strength (lbs/ft)
MSE Reinforcement Straps		GeoTextile	100	15000



Project	FRA-70-12.68 - Retaining Wall 4W6 - Sta. 178+85 to Sta. 180+30		
Analysis Description	B-003-A-59 and B-023-1-13 - 26.7 ft. Wall Height - Drained - Spencer's		
Drawn By: BRT	Scale: 1:250	Company: Resource International, Inc.	
Date: 11/30/2015, 5:20:00 PM	File Name: Retaining Wall 4W6 - Sta. 178+85 to 180+30 - Global Stability.slim		

SLIDEINTERPRET 6.031