FRA-71-14.36 PHASE 6R
RETAINING WALL E7
PID NO. 105588
FRANKLIN COUNTY, OHIO

STRUCTURE FOUNDATION EXPLORATION REPORT (REV. 1)

Prepared For:
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Prepared By:
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Rii Project No. W-13-072

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RESOURCE INTERNATIONAL, INC.
ISO | ISO 9001:2015 Certified QMS
An ISO 9001:2015 QMS Certified Firm

Planning

Engineering

Construction Management

Technology

June 19, 2015 (Revised July 19, 2019)

Mr. Gary Gardner, P.E.
ms consultants, inc.
2221 Schrock Road
Columbus, OH 43229-1547

Re: Structure Foundation Exploration Report (Rev. 1)
FRA-71-14.36 Phase 6R
Retaining Wall E7
PID No. 105588
Rii Project No. W-13-072

Mr. Gardner:

Resource International, Inc. (Rii) is pleased to submit this revised 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 Wall E7 as part of the FRA-71-14.36 Phase 6R project 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

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Enclosure: Structure Foundation Exploration Report (Rev. 1)

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EXECUTIVE SUMMARY

Resource International, Inc. (Rii) has completed a structure foundation exploration for the design and construction of the proposed Retaining Wall E7. Based on plan information provided by the Rii design group and ms consultants, Retaining Wall E7 will be located along the north side of Ramp D6, and will provide the required grade separation between the ramp and the adjacent power substation to the north of the ramp alignment. The wall begins at Sta. 6003+60 (BL Ramp D6) and extends west along the north side of Ramp D6 and I-71 southbound to Sta. 277+55 (BL I-71 SB), where the wall alignment turns south and crosses under the forward abutment of the proposed FRA-71-1503L structure as well as the proposed FRA-70-1373L structure. The total wall length for Retaining Wall E7, including the portion of the wall that crosses in front of the abutments of the proposed bridge structures, is approximately 593 lineal feet, and the total length from the beginning of the wall to the end of the approach slab leading to the forward abutment of the FRA-71-1503L structure is approximately 383 feet. **Please note that the design of the MSE wall between Sta. 704+21 and 706+31 (BL Wall E7), where it crosses the abutments of the proposed bridge structures, will be governed by the recommendations in the respective bridge structure reports, which are presented under separate covers.** The wall heights along the portion of the wall alignment that is considered for this exploration report will range from 8.5 feet at Sta. 700+38 (BL Wall E7) to 47.3 feet at Sta. 704+21 (BL Wall E7).

Exploration and Findings

Between May 13, 2014, and April 17, 2015, four (4) structural borings, designated as B-021-1-13, B-021-5-14, B-023-3-14, and B-115-2-13, were drilled to completion depths ranging from 35.0 to 90.7 feet below the existing ground surface at the locations shown on the boring plan provided in Appendix I of the full report.

All of the borings for this exploration were drilled in the grass area at the toe of the existing embankment supporting the ramp from Mound Street to I-70 westbound and encountered 5.0 to 12.0 inches of topsoil at the existing ground surface, as identified by the significant presence of vegetation and organic material.

Beneath the surface materials in borings B-021-1-13, B-023-3-14 and B-115-2-13, material identified as existing fill was encountered extending to a depth of 3.0, 5.5 and 23.0 feet below existing grade, which corresponds to an elevation of 716.7, 727.7 and 693.1 feet msl. The existing fill material was generally described as dark brown, brown, black and gray gravel with sand, gravel with sand and silt, coarse and fine sand, silt, and silt and clay. The fill contained debris consisting of brick and slag fragments, cinders and organics. Asphalt, brick, cinder, coal, concrete, rock, slag, and wood fragments were observed to be present within the fill materials in addition to organic material. It should also be noted that a petroleum odor was noted within the fill material encountered in boring B-115-2-13 at a depth of 16.0 feet beneath the ground surface.

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

Top of bedrock in boring B-115-2-13 was encountered at a depth of 63.5 feet below the existing ground surface, which corresponds to an elevation of 652.6 feet msl. The upper 19.7 feet of the bedrock encountered consists of claystone and shale overlying limestone bedrock at an elevation of 632.9 feet msl.

Analyses and Recommendations

It is understood that a MSE wall type is being utilized between Sta. 700+38 and 702+50 (BL Wall E7), with a wall height ranging from 8.5 to 34.3 feet. A lightweight cellular concrete modified MSE wall system will be utilized between Sta. 702+50 and 703+00 (BL Wall E5), where the wall will span over two existing electrical duct banks, with a wall height ranging from 34.2 to 39.3 feet. A modified MSE wall system consisting of geofoam on top of lightweight cellular concrete is being utilized between Sta. 703+00 and 704+21 (BL Wall E7), where the alignment spans over the influence zone of the Franklin Main, with a wall height ranging from 39.2 to 47.3 feet. It is understood that precast facing panels will be utilized, which will be supported on pedestal footings and anchored into the distribution slab at the top of the wall for composite geofoam and cellular concrete modified MSE wall segment.

MSE Wall Recommendations

Based on the proposed plan and profile information, the proposed standard MSE wall section will have wall heights ranging from 8.5 to 34.2 feet, as measured from the top of the leveling pad to the top of the coping. The anticipated bearing materials along the proposed alignment of Retaining Wall E7 between Sta. 700+38 and 702+50 (BL Wall E7) consists of medium dense to dense gravel and gravel with sand (ODOT A-1-a, A-1-b) and existing fill consisting of medium stiff silt and clay (ODOT A-6a) near the east end of the wall. MSE wall foundations bearing on these soils may be proportioned for a factored 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 E7 MSE Wall Design Parameters

From Station [1]	To Station [1]	Wall Height Analyzed (feet)	Backslope Behind Wall in Analysis	Minimum Required Reinforcement Length [2] (feet)	Bearing Resistance at Strength Limit (ksf)		Strength Limit Equivalent Bearing Pressure [4] (ksf)
					Nominal	Factored [3]	
700+38	702+50	34.2	Level	23.9 (0.70H ≥ 8.0)	65.4	42.5	8.01

1. *Stationing referenced to the baseline of Retaining Wall E7.*
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.*

Total settlements of up to 2.16 inches at the center of the reinforced soil mass and 1.79 inches at the facing of the wall are anticipated along the alignment of Retaining Wall E7 between Sta. 700+38 and 702+50 (BL Wall E7). Based on the results of the analysis, 100 percent of the total settlement at the facing of the wall is anticipated to occur during or immediately following construction of the wall or within eight (8) days following the completion of construction of the wall.

Based on the results of the external and global stability analysis performed for the MSE wall, the recommended controlling strap length is 0.70 times the height of the MSE wall (measured from the top of the leveling pad to the proposed profile grade of the roadway) between Sta. 700+38 and 702+50 (BL Wall E7). All of the external and global stability calculations indicate that adequate resistance is available for support of the MSE wall.

Lightweight (Cellular Concrete) Wall Recommendations

It is understood that a lightweight cellular concrete modified MSE system wall will be utilized between Sta. 702+50 and 703+00 (BL Wall E5), where the wall will span over two existing electrical duct banks, with a wall height ranging from 34.2 to 39.3 feet. Based on information provided by the Rii design team, 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.

Provided that all backslopes cut into the existing I-70 embankment are graded no steeper than 2H:1V, external and global stability calculations will not be required for this section of Retaining Wall E7. However, if bearing resistance must be checked, then a factored bearing resistance of 37.9 ksf should be utilized for design at the strength limit state.

Total settlements of 0.57 to 1.24 inches at the center of the wall mass and 0.41 to 0.91 inches at the facing of the wall are anticipated along Retaining Wall E7 between Sta. 702+50 and 703+00 (BL Wall E5). Based on the results of the analysis, 90 percent of the total settlement is anticipated to occur over a period of approximately five (5) days following the completion of construction of the wall.

Composite Geofoam/ Cellular Concrete Wall Recommendations

A modified MSE wall system consisting of geofoam on top of lightweight cellular concrete is being utilized between Sta. 703+00 and 704+21 (BL Wall E7), where the alignment spans over the influence zone of the Franklin Main, with a wall height ranging from 39.2 to 47.3 feet. Based on the information provided, the typical section for this segment will consist of an approximate 5.0-foot thick pavement section, including asphalt and/or concrete and aggregate base on top of a concrete distribution slab, overlying geofoam blocking (ASTM D6817, Type 19) to El. 725 feet msl, overlying Class II cellular concrete to the bottom of wall elevation. Where the Franklin Main crosses the wall alignment, at approximately Sta. 703+70 (BL Wall E7), the wall height is approximately 45.0 feet and the bottom of wall (top of leveling pad) is at El. 713.5 feet msl. Considering a unit weight of 30 pcf for the Class II cellular concrete, 1.5 pcf for the geofoam blocking and 130 pcf for the pavement/aggregate base/distribution slab, the pressure at the bottom of the wall is approximately 1,040 psf. Therefore, to provide for no net loading on the Franklin Main, it is recommended to over excavate 11.5 feet below the bottom of the wall and backfill with Class II cellular concrete. Please note that this over excavation would only need to occur at the wall facing, and can be laid back where the lightweight fill extends up the existing I-70 embankment.

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-70/71-13.10/14.36 (Projects 6A/6R) project in Columbus, Ohio. The projects represent the central portion of FRA-70-8.93 (PID 77369) I-70/71 south innerbelt improvements project, which includes all improvements along I-70 westbound from the I-71/SR-315 interchange to Front Street and along I-71 southbound from I-70 to Greenlawn Avenue. The FRA-71-14.36 (Project 6R) phase will consist of all work associated with the reconfiguration and construction of I-71 southbound from downtown (Front Street) to Greenlawn Avenue, including Ramps C3, D6 and D7. This project includes the construction of two (2) new bridge structures, one (1) for I-71 southbound over Short Street, NS/CXS Railroad and the Scioto River (FRA-71-1503L) and one (1) for Ramp D7 over Short Street (FRA-70-1373B), as well as the construction of five (5) new retaining walls (Walls E4, E5, E7, W2 and W5) to accommodate the new configuration.

This report is a presentation of the structure foundation exploration performed for the design and construction of the proposed Retaining Wall E7 as shown on the vicinity map and boring plan presented in Appendix I. Retaining Wall E7 will be located along the north side of Ramp D6, and will provide the required grade separation between the ramp and the adjacent power substation to the north of the ramp alignment. The wall begins at Sta. 6003+60 (BL Ramp D6) and extends west along the north side of Ramp D6 and I-71 southbound to Sta. 277+55 (BL I-71 SB), where the wall alignment turns south and crosses under the forward abutment of the proposed FRA-71-1503L structure as well as the proposed FRA-70-1373L structure. The total wall length for Retaining Wall E7, including the portion of the wall that crosses in front of the abutments of the proposed bridge structures, is approximately 593 lineal feet, and the total length from the beginning of the wall to the end of the approach slab leading to the forward abutment of the FRA-71-1503L structure is approximately 383 feet. **Please note that the design of the MSE wall between Sta. 704+21 and 706+31 (BL Wall E7), where it crosses the abutments of the proposed bridge structures, will be governed by the recommendations in the respective bridge structure reports, which are presented under separate covers.** The wall heights along the portion of the wall alignment that is considered for this exploration report will range from 8.5 feet at Sta. 700+38 (BL Wall E7) to 47.3 feet at Sta. 704+21 (BL Wall E7).

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), as well as in conjunction with the adjacent FRA-70-12.68 Project 4R (PID 105523), 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. In addition, the lightweight cellular concrete will also be utilized along the segment of wall that crosses over two existing electrical duct banks at approximately Sta. 702+55 and 702+62 (BL Wall E7).

Additionally, it is understood that no net loading is permitted to be applied over the Franklin Main, which is a 60-inch brick sewer that crosses under the wall alignment at approximately Sta. 703+65. It is understood that the sewer has been previously lined and subsequently loaded to the maximum permissible overburden pressure. Therefore, geofoam blocking in conjunction with undercut of the existing soil and backfill with lightweight cellular concrete will be utilized within the zone of influence of the existing sewer pipe. 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, it is understood that a standard mechanically stabilized earth (MSE) wall type is being utilized between Sta. 700+38 and 702+50 (BL Wall E7). A retaining wall system consisting of geofoam on top of lightweight backfill is being utilized between Sta. 703+00 and 704+21 (BL Wall E7) in order to limit the loading imparted on the Franklin Main. Additionally, the lightweight cellular concrete will also be utilized for a short segment just east of the geofoam section of wall, between Sta. 702+50 and 703+00 (BL Wall E7) in order to limit the loading imparted on the existing electrical duct banks.

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 boring B-020-5-13 performed for this current project, shale and mudstone bedrock was encountered beginning at a depth of 76.6 feet below the existing ground surface, which corresponds to an elevation of 656.8 feet msl.

2.2 Existing Conditions

The proposed Retaining Wall E7 structure will be situated along the north side of the existing ramp from Mound Street to I-70 westbound. The existing ramp is a single lane, asphalt pave roadway that is supported on engineered embankment that grades up from the Mound Street to the I-70 westbound profile grade. The existing I-70 westbound in the vicinity of the structure 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. There is an existing electrical substation located along the north side of the existing ramp, which is owned and operated by American Electric Power (AEP). The terrain along I-70 slopes gently to the east and the surrounding area is relatively flat-lying, and dense vegetation covers the existing embankment slope that supports the onramp.

3.0 EXPLORATION

Between May 13, 2014, and April 17, 2015, four (4) structural borings, designated as B-021-1-13, B-021-5-14, B-023-3-14, and B-115-2-13, were drilled at the locations shown on the boring plan provided in Appendix I of this report and summarized in Table 1. The borings were advanced to completion depths ranging from 35.0 to 90.7 feet below the existing ground surface along the toe of the existing embankment supporting the ramp from Mound Street to I-70 westbound.

Table 1. Test Boring Summary

Boring Number	Reference Alignment	Station [1]	Offset [1]	Latitude	Longitude	Ground Elevation (feet msl)	Boring Depth (feet)
B-021-1-13	BL Ramp D6	6000+84.54	7.6' Lt.	39.953691691	-83.003720238	719.7	49.8
B-021-5-14	BL Ramp D6	6001+99.15	32.1' Lt.	39.953766206	-83.003343013	727.5	40.0
B-023-3-14	BL Ramp D6	6003+43.74	39.1' Lt.	39.953952398	-83.002987438	733.2	35.0
B-115-2-13	BL I-71 SB	277+71.50	20.2' Lt.	39.953734364	-83.004256897	716.1	90.7

The boring locations 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 ms consultants.

The borings were drilled using a truck or an all-terrain vehicle (ATV) mounted rotary drilling machine, utilizing a 3.25-inch inside diameter, hollow-stem auger to advance the holes. Standard penetration test (SPT) and split spoon were performed in the borings at 2.5-foot intervals to a depth of 20.0 or 30.0 feet, and at 5.0-foot intervals 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. Rii utilized a calibrated automatic drop hammer to generate consistent energy transfer to the sampler. Driving resistance is recorded on the boring logs in terms of blow per 6.0-inch interval of the driving distance. The second and third intervals are added to obtain the number of blows per foot (N). Standard penetration blow counts aid in determining soil properties applicable in foundation system design. Measured blow count (N) values are corrected to an equivalent (60%) energy ratio, N_{60}, by the following equation. Both values are represented on boring logs in Appendix III.

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

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

The hammer for the Mobile B-53 drill rig was calibrated on April 26, 2013, and has a drill rod energy ratio of 77.7 percent. The hammer for the CME 750 drill rig was calibrated on October 20, 2014, and has a drill rod energy ratio of 85.7 percent.

During 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. Laboratory Test Schedule

Laboratory Test	Test Designation	Number of Tests Performed
Natural Moisture Content	ASTM D 2216	52
Plastic and Liquid Limits	AASHTO T89, T90	20
Gradation – Sieve/Hydrometer	AASHTO T88	20
Unconfined Compressive Strength of Intact Rock	ASTM D7012	1

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

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

The depth to bedrock in boring B-115-2-13 was determined by split spoon sampler refusal. Split spoon sampler refusal is defined as exceeding 50 blows from the hammer with less than 6.0 inches of penetration by the split spoon sampler. An NQ-sized double-tube diamond bit core barrel (utilizing wire line equipment) was used to core the bedrock in boring B-115-2-13. Coring produced 1.85-inch diameter cores, from which the type of rock and geological characteristics were determined.

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

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

4.0 FINDINGS

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

4.1 Surface Materials

All of the borings for this exploration were drilled in the grass area at the toe of the existing embankment supporting the ramp from Mound Street to I-70 westbound and encountered 5.0 to 12.0 inches of topsoil at the existing ground surface, as identified by the significant presence of vegetation and organic material.

4.2 Subsurface Soils

Beneath the surface materials in borings B-021-1-13, B-023-3-14 and B-115-2-13, material identified as existing fill was encountered extending to a depth of 3.0, 5.5 and 23.0 feet below existing grade, which corresponds to an elevation of 716.7, 727.7 and 693.1 feet msl. The existing fill material was generally described as dark brown, brown, black and gray gravel with sand, gravel with sand and silt, coarse and fine sand, silt, and silt and clay. The fill contained debris consisting of brick and slag fragments, cinders and organics. Asphalt, brick, cinder, coal, concrete, rock, slag, and wood fragments were observed to be present within the fill materials in addition to organic material. It should also be noted that a petroleum odor was noted within the fill material encountered in boring B-115-2-13 at a depth of 16.0 feet beneath the ground surface.

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

The relative density of granular soils is primarily derived from SPT blow counts (N_{60}). Based on the SPT blow counts obtained, the granular soil encountered ranged from very loose (N_{60} < 5 blows per foot [bpf]) to very dense (N_{60} > 50 bpf). Overall blow counts recorded from the SPT sampling ranged from 3 bpf to split spoon sampler refusal. The shear strength and consistency of the cohesive soils are primarily derived from the hand penetrometer values (HP). The cohesive soil encountered ranged from stiff ($1.0 < HP \leq 2.0$ tsf) to hard (HP > 4.0 tsf). The unconfined compressive strength of the cohesive soil samples tested, obtained from the hand penetrometer, ranged from 1.25 to over 4.5 tsf (limit of instrument).

Natural moisture contents of the soil samples tested ranged from 3 to 68 percent. The natural moisture content of the cohesive soil samples tested for plasticity index ranged from 9 percent below to 5 percent above their corresponding plastic limits. In general, the soil exhibited natural moisture contents considered to be significantly below to moderately above optimum moisture levels.

4.3 Bedrock

Bedrock was encountered in boring B-115-2-13 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-115-2-13	716.1	63.5	652.6	64.2	651.9

Top of bedrock in boring B-115-2-13 was encountered at a depth of 63.5 feet below the existing ground surface, which corresponds to an elevation of 652.6 feet msl. The upper 19.7 feet of the bedrock encountered consists of claystone and shale overlying limestone bedrock at an elevation of 632.9 feet msl. The claystone is described as gray, unweathered, very weak to slightly strong, medium bedded and moderately to highly fractured with open, slightly rough apertures. The shale is described as gray, highly weathered, weak, medium bedded, calcareous, fissile and fractured to highly fractured with open, rough apertures. The limestone is described as grayish brown, slightly weathered, moderately strong, thin to medium bedded, cherty, pyritic and moderately fractured with open, slightly to very rough apertures.

The percent recovery, RQD values and unconfined compressive strengths of the bedrock core runs in boring B-115-2-13 are summarized in Table 4.

Table 4. Rock Core Summary

Boring	Core No.	Elevation (feet msl)	Recovery (%)	RQD (%)	Unconfined Compressive Strength
B-115-2-13	RC-1	651.9 to 650.4	100	58	N/A
	RC-2	650.4 to 645.4	88	0	N/A
	RC-3	645.4 to 640.4	100	86	N/A
	RC-4	640.4 to 635.4	30	0	N/A
	RC-5	635.4 to 630.4	75	45	q_u @ 83.2’ = 8,867 psi
	RC-6	630.4 to 625.4	100	92	N/A

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 quality of the claystone and shale bedrock, according to the RQD values, ranged from very poor (RQD ≤ 25%) to good (75% < RQD ≤ 90%), and the quality of the limestone bedrock was excellent (90% < RQD ≤ 100%).

4.4 Groundwater

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

Table 5. Groundwater

Boring Number	Ground Elevation (feet msl)	Initial Groundwater		Upon Completion	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-021-1-13	719.7	22.0	697.7	N/A [1]	N/A
B-021-5-14	727.5	24.5	703.0	N/A [1]	N/A
B-023-3-14	733.2	28.5	704.7	N/A [1]	N/A
B-115-2-13	716.1	8.0	708.1	N/A [1]	N/A

1. *The groundwater level at completion could not be obtained due to the addition of mud as a drilling fluid and water during the rock coring process.*

Groundwater was encountered initially during drilling in all four borings at depths ranging from 8.0 to 28.5 feet below the ground surface, which corresponds to elevations ranging from 697.7 to 708.1 feet msl. The groundwater level at the completion of drilling could not be measured due to the addition of mud during to counteract heaving sands during drilling as well as water as a circulating fluid during the rock coring process. Please note that short-term water level readings, especially in cohesive soils, are not necessarily an accurate indication of the actual groundwater level. In addition, groundwater levels or the presence of groundwater are considered to be dependent on seasonal fluctuations in precipitation.

A more comprehensive description of what was encountered during the drilling process may be found on the boring logs in Appendix III.

5.0 ANALYSES AND RECOMMENDATIONS

Data obtained from the subsurface exploration has been used to determine the foundation support capabilities and the settlement potential for the soil encountered at the site. These parameters have been used to provide guidelines for the design of 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 wall were provided by the Rii design team and ms consultants. It is understood that a standard MSE wall type is being utilized between Sta. 700+38 and 702+50 (BL Wall E7), with a wall height ranging from 8.5 to 34.3 feet. A lightweight cellular concrete modified MSE wall system will be utilized between Sta. 702+50 and 703+00 (BL Wall E5), where the wall will span over two existing electrical duct banks, with a wall height ranging from 34.2 to 39.3 feet. A modified MSE wall system consisting of geofoam on top of lightweight cellular concrete is being utilized between Sta. 703+00 and 704+21 (BL Wall E7), where the alignment spans over the influence zone of the Franklin Main, with a wall height ranging from 39.2 to 47.3 feet. It is understood that precast facing panels will be utilized, which will be supported on pedestal footings and anchored into the distribution slab at the top of the wall for composite geofoam and cellular concrete modified MSE wall segment.

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 composite geofoam and cellular concrete modified MSE wall will be utilized to span Ramp D6 and I-71 southbound 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. The design of a retaining wall system that incorporates geofoam and lightweight cellular concrete fill is considered proprietary. Therefore, only calculations for settlement and bearing capacity for these segments of the wall are provided in this report. If additional analyses for internal stability or external sliding, overturning or global stability are required, they should be performed by a specialty contractor that is qualified to design these systems.

5.1 MSE Wall Recommendations

It is understood that a standard MSE wall type is being utilized between Sta. 700+38 and 702+50 (BL Wall E7). 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 2019 ODOT BDM, the height of the MSE wall is defined as the elevation difference between the top of coping and the top of the leveling pad. However, it is noted that the reinforced soil mass only extends from the foundation bearing elevation (top of leveling pad) to the roadway subgrade elevation where the roadway is supported on the top of the wall, and the reinforced soil mass extends to the top of the coping where the roadway is not supported on top of the wall. 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 2019 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 proposed standard MSE wall section will have wall heights ranging from 8.5 to 34.2 feet, as measured from the top of the leveling pad to the top of the coping. For the analysis, the foundation width was set at 70 percent of the 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. The anticipated soils at the proposed bearing elevation along the wall alignment between Sta. 700+38 and 702+50 (BL Wall E7) consists of medium dense to dense gravel and gravel with sand (ODOT A-1-a, A-1-b) and existing fill consisting of medium stiff silt and clay (ODOT A-6a) near the east end of the wall. These materials are considered suitable for support of the proposed wall in their current condition.

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

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

5.1.1 Strength Parameters Utilized in External and Global Stability Analyses

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

Table 6. Shear Strength Parameters Utilized in Stability Analyses

Material Type	**γ (pcf)**	**φ' (1) (°)**	**c' (2) (psf)**	**S_u (3) (psf)**
MSE Wall Backfill (Select granular backfill)	120	34	0	N/A
Item 203 Embankment Fill (Retained soil)	120	30	0	2,000
Medium Dense to Very Dense Granular Soils (ODOT A-1-a, A-1-b)	125 to 135	35 to 42	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 reinforced soil backfill and retained embankment are provided in ODOT SS 840. Per SS 840, the select granular backfill in the reinforced zone and the retained embankment must meet the shear strength requirements provided in Table 6. 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.

5.1.2 Bearing Stability

The anticipated bearing materials along the proposed alignment of Retaining Wall E7 between Sta. 700+38 and 702+50 (BL Wall E7) consists of medium dense to dense gravel and gravel with sand (ODOT A-1-a, A-1-b) and existing fill consisting of medium stiff silt and clay (ODOT A-6a) near the east end of the wall. MSE wall foundations bearing on these soils may be proportioned for a factored bearing resistance as indicated in Table 7. A geotechnical resistance factor of φ_b=0.65 was considered in calculating the factored bearing resistance at the strength limit state. The reinforcement length presented in the following table represents the minimum foundation width required to satisfy external and global stability requirements, expressed as a percentage of the wall height.

Table 7. Retaining Wall E7 MSE Wall Design Parameters

From Station [1]	To Station [1]	Wall Height Analyzed (feet)	Backslope Behind Wall in Analysis	Minimum Required Reinforcement Length [2] (feet)	Bearing Resistance at Strength Limit (ksf)		Strength Limit Equivalent Bearing Pressure [4] (ksf)
					Nominal	Factored [3]	
700+38	702+50	34.2	Level	23.9 (0.70H ≥ 8.0)	65.4	42.5	8.01

1. *Stationing referenced to the baseline of Retaining Wall E7.*
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 height indicated in Table 7. Based on the minimum length of reinforced soil mass presented, the factored equivalent bearing pressure exerted below the wall **will not exceed** the factored bearing resistance at the strength limit state.

5.1.3 Settlement Evaluation

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

Table 8. Compressibility Parameters Utilized in Settlement Analysis

Material Type	γ (pcf)	LL (%)	C_c (1)	C_r (2)	e_o (3)	C_v (4) (ft^2/yr)	N_{60}	C' (5)
Medium Dense to Very Dense Granular Soils (ODOT A-1-a, A-1-b, A-3a)	125 to 135	N/A	N/A	N/A	N/A	N/A	14 to 120	59 to 543
Stiff to Hard Silt and Clay (ODOT A-6a)	125	28 to 34	0.162 to 0.216	0.016 to 0.022	0.491 to 0.538	600	N/A	N/A
Very Stiff Sandy Silt (ODOT A-4a)	130	22	0.108	0.011	0.444	1,000	N/A	N/A

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 9. Total settlements of up to 2.16 inches at the center of the reinforced soil mass and 1.79 inches at the facing of the wall are anticipated along the alignment of Retaining Wall E7 between Sta. 700+38 and 702+50 (BL Wall E7). Based on the results of the analysis, 100 percent of the total settlement at the facing of the wall is anticipated to occur during or immediately following construction of the wall or within eight (8) days following the completion of construction of the wall. Please note that the consolidation settlement and time rate of consolidation are based on estimates using correlated compressibility parameters provided in Table 8 for the underlying soils. Actual settlement and time rate of consolidation should be determined by monitoring the settlement of the wall using settlement platforms.

Table 9. Retaining Wall E7 MSE Wall Settlement Values

From Station [1]	To Station [1]	Service Limit Equivalent Bearing Pressure [2] (ksf)	Total Settlement Values (inches)		Time for 100% Consolidation (Days)
			Center of Wall Mass	Facing of Wall	
700+38	702+50	1.50 to 5.62	1.35 to 2.16	1.11 to 1.79	0 to 8

1. *Stationing referenced to the baseline of Retaining Wall E7.*
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.*

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 1/1,000, 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 V.

5.1.4 Eccentricity (Overturning Stability)

The resistance of the MSE wall 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, the location of the resultant of the reaction forces shall be within the middle two-thirds ($^2/_3$) of the base width. Therefore, the limiting eccentricity is one-third ($^1/_3$) of the base width of the wall. Rii performed a verification of the eccentricity of the resultant force for the specified wall height indicated in Table 7. Based on the minimum length of reinforced soil mass presented in Table 7 and utilizing the soil parameters listed in Section 5.1.1 for the retained embankment material, the calculated eccentricity of the resultant force **will not exceed** the limiting eccentricity at the strength limit state.

5.1.5 Sliding Stability

The resistance of the MSE wall to sliding was evaluated per Section 11.10.5.3 of the 2018 AASHTO LRFD BDS. For drained conditions, the sliding resistance is determined by multiplying a coefficient of sliding friction "f" times the total vertical force at the base of the wall. The coefficient of sliding friction is determined based on the limiting friction angle between the foundation soil and the reinforced soil backfill. Based on the soil parameters listed in Section 5.1.1 for the foundation and reinforced soil backfill, a coefficient of sliding friction of 0.67 was utilized for design.

A geotechnical resistance factor of φ_τ=1.0 was considered in calculating the factored shear resistance between the reinforced soil backfill and foundation soil for sliding. Based on the minimum length of reinforced soil mass presented in Table 7 and utilizing the soil parameters listed in Section 5.1.1 for the retained embankment material, the resultant horizontal forces on the back of the MSE wall **will not exceed** the factored shear resistance at the strength limit state.

5.1.6 Overall (Global) Stability

A slope stability analysis was performed to check the global stability of the wall. As per the AASHTO LRFD BDS, safety against soil failure shall be evaluated at the service limit state by assuming the reinforced soil mass to be a rigid body. Soil parameters utilized in the global stability analyses are presented in Section 5.1.1. For the global stability condition, it was considered that the failure plane will not cross through the reinforced soil mass. The computer software program Slide 2018 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 an MSE wall designed with the minimum strap length listed in Table 7, the resulting factor of safety under drained conditions (long-term stability) was greater than 1.3. Given the granular nature of the subsurface profile, an undrained analysis was not performed.

5.1.7 Final MSE Wall Considerations

Based on the results of the external and global stability analysis performed for the MSE wall, the recommended controlling strap length is 0.70 times the height of the MSE wall (measured from the top of the leveling pad to the proposed profile grade of the roadway) between Sta. 700+38 and 702+50 (BL Wall E7). All of the external and global stability calculations indicate that adequate resistance is available for support of the MSE wall.

Calculations for external (bearing and sliding resistance and limiting eccentricity) and overall (global) stability of the MSE wall are provided in Appendix V.

5.2 Cellular Concrete Wall Recommendations

It is understood that a lightweight cellular concrete modified MSE system wall will be utilized between Sta. 702+50 and 703+00 (BL Wall E5), where the wall will span over two existing electrical duct banks, with a wall height ranging from 34.2 to 39.3 feet. Based on information provided by the Rii design team, 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 the Rii design team. 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 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 this section of 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 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 D6 and I-71 southbound. Provided that all backslopes cut into the existing I-70 embankment are graded no steeper than 2H:1V, external and global stability calculations will not be required for this section of Retaining Wall E7. However, if bearing resistance must be checked, then a factored bearing resistance of 37.9 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 8.

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)
Medium Dense to Very Dense Granular Soils (ODOT A-1-a, A-1-b, A-3a)	125 to 135	N/A	N/A	N/A	N/A	N/A	16 to 120	63 to 942
Very Stiff Sandy Silt (ODOT A-4a)	130	22	0.108	0.011	0.444	1,000	N/A	N/A

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 9. Total settlements of 0.57 to 1.24 inches at the center of the wall mass and 0.41 to 0.91 inches at the facing of the wall are anticipated along Retaining Wall E7 between Sta. 702+50 and 703+00 (BL Wall E5). Based on the results of the analysis, 100 percent of the total settlement at the facing of the wall is anticipated to occur during or immediately following construction of the wall or within five (5) days following the completion of construction of the wall. Please note that the consolidation settlement and time rate of consolidation are based on estimates using correlated compressibility parameters provided in Table 8 for the underlying soils. Actual settlement and time rate of consolidation should be determined by monitoring the settlement of the wall using settlement platforms.

Table 11. Retaining Wall E7 Settlement Results

Boring	Wall Height (feet)	Pressure at Bottom of Wall [1] (ksf)	Total Settlement Values (inches)		Time for 100% Consolidation (Days)
			Center of Wall Mass	Facing of Wall	
B-021-1-13	39.3	1,551	1.24	0.91	0
B-021-5-14	34.2	1,398	0.57	0.41	5

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 1/1,000, which is within the tolerable limit of 1/100.

Results of the settlement analysis and bearing resistance for the cellular concrete MSE wall are provided in Appendix VI.

5.3 Composite Geofoam/Cellular Concrete Wall Recommendations

A modified MSE wall system consisting of geofoam on top of lightweight cellular concrete is being utilized between Sta. 703+00 and 704+21 (BL Wall E7), where the alignment spans over the influence zone of the Franklin Main, with a wall height ranging from 39.2 to 47.3 feet. Based on the information provided, the typical section for this segment will consist of an approximate 5.0-foot thick pavement section, including asphalt and/or concrete and aggregate base on top of a concrete distribution slab, overlying geofoam blocking (ASTM D6817, Type 19) to El. 725 feet msl, overlying Class II cellular concrete to the bottom of wall elevation. Where the Franklin Main crosses the wall alignment, at approximately Sta. 703+70 (BL Wall E7), the wall height is approximately 45.0 feet and the bottom of wall (top of leveling pad) is at El. 713.5 feet msl. Considering a unit weight of 30 pcf for the Class II cellular concrete, 1.5 pcf for the geofoam blocking and 130 pcf for the pavement/aggregate base/distribution slab, the pressure at the bottom of the wall is approximately 1,040 psf. Therefore, to provide for no net loading on the Franklin Main, it is recommended to over excavate 11.5 feet below the bottom of the wall and backfill with Class II cellular concrete. Please note that this over excavation would only need to occur at the wall facing, and can be laid back where the lightweight fill extends up the existing I-70 embankment.

5.4 Lateral Earth Pressure

For the soil types encountered in the borings, the “in-situ” unit weight (γ), cohesion (c), effective angle of friction (φ'), and lateral earth pressure coefficients for at-rest conditions (k_o), active conditions (k_a), and passive conditions (k_p) have been estimated and are provided in Table 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
Very Loose to 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	120	0	32°	0.27	0.47	6.82

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

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

Soil Type	γ (pcf) [1]	c (psf)	φ'	k_a	k_o	k_p
Soft to Stiff Cohesive Soil	115	0	26°	0.35	0.56	4.53
Very Stiff to Hard Cohesive Soil	125	0	28°	0.32	0.53	5.07
Very Loose to 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	120	0	32°	0.27	0.47	6.82

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

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

5.5 Construction Considerations

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

5.5.1 Excavation Considerations

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

Table 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

5.5.2 Groundwater Considerations

Based on the groundwater observations made during drilling, groundwater is not anticipated to be encountered during construction of the proposed retaining wall. However, where/if groundwater is encountered, proper groundwater control should be employed and maintained to prevent disturbance to excavation bottoms consisting of cohesive soil, and to prevent the possible development of a quick or "boiling" condition where soft silts and/or fine sands are encountered. It is preferable that the groundwater level, if encountered, be maintained at least 36 inches below the deepest excavation. Any seepage or groundwater encountered at this site should be able to be controlled by pumping from temporary sumps. Additional measures may be required depending on seasonal fluctuations of the groundwater level. Note that determining and maintaining actual groundwater levels during construction is the responsibility of the contractor.

6.0 LIMITATIONS OF STUDY

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

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

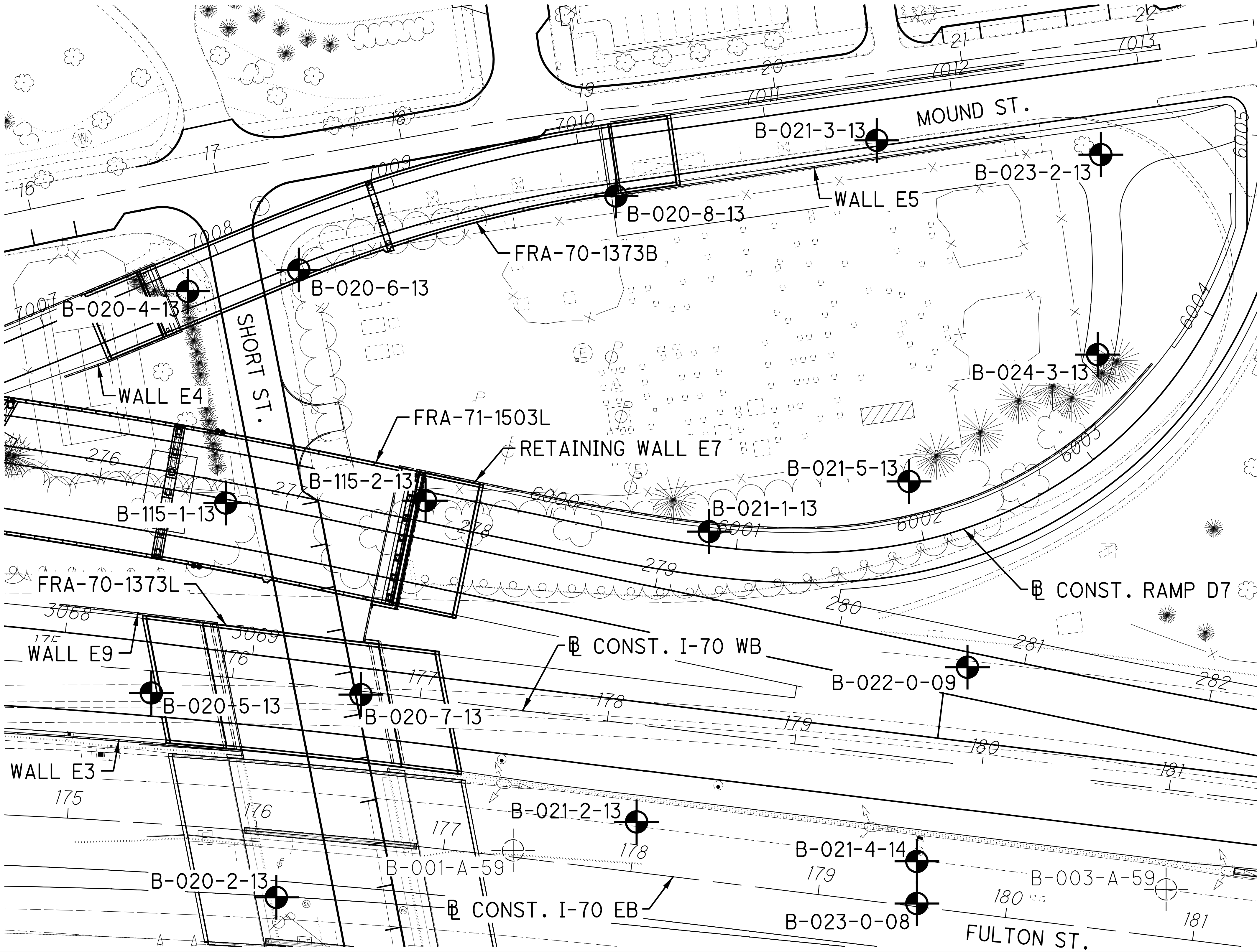
The conclusions and recommendations herein have been based upon the available soil and bedrock information and the design details furnished by a representative of the owner of the proposed project. Any revision in the plans for the proposed construction from those anticipated in this report should be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

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

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

APPENDIX I

VICINITY MAP AND BORING PLAN



BORINGS B-021-1-13, B-021-5-13 AND B-024-3-13 WERE PEFORMED FOR WALL E7.

BORINGS B-020-4-13, B-020-6-13 AND B-020-8-13 WERE PERFORMED FOR THE FRA-70-1373B STRUCTURE.

BORINGS B-020-5-13 AND B-020-7-13 WERE PERFORMED FOR THE FRA-70-1373L STRUCTURE.

BORINGS B-021-3-13 AND B-023-2-13 WERE PERFORMED FOR WALL E5.

BORINGS B-115-1-13 AND B-115-2-13 WERE PERFORMED FOR THE FRA-71-1503L STRUCTURE.

BORINGS B-020-2-13, B-021-2-13 AND B-021-4-14 WERE PERFORMED AS PART OF THE FRA-70-12.68 PROJECT 4A/4R.

BORINGS B-022-0-09, B-023-0-08, B-025-0-08 AND B-026-0-08 WERE PERFORMED AS PART OF THE FRA-70-8.93 PRELIMINARY EXPLORATION.

LEGEND

- PROJECT BORING
- HISTORIC BORING

BORING PLAN

FRA-70-13.10 - RETAINING WALL E7

FRANKLIN COUNTY, OHIO

PROJECT NO. Rii W-13-072	DRAWN RRM
SCALE: 1″=60′ (0 30 60)	REVIEWED BRT
N	DATE 7-17-19

RESOURCE INTERNATIONAL Rii

APPENDIX II

DESCRIPTION OF SOIL TERMS

DESCRIPTION OF SOIL TERMS

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

Granular Soils - The relative compactness of granular soils is described as:
ODOT A-1, A-2, A-3, A-4 (non-plastic) or USCS GW, GP, GM, GC, SW, SP, SM, SC, ML (non-plastic)

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 - The relative consistency of cohesive soils is described as:
ODOT A-4, A-5, A-6, A-7, A-8 or USCS ML, CL, OL, MH, CH, OH, PT

Description	Blows per foot – SPT (N_{60})			Unconfined Compression (tsf)
Very Soft	Below		2	UCS ≤ 0.25
Soft	2	-	4	0.25 < UCS ≤ 0.5
Medium Stiff	5	-	8	0.5 < UCS ≤ 1.0
Stiff	9	-	15	1.0 < UCS ≤ 2.0
Very Stiff	16	-	30	2.0 < UCS ≤ 4.0
Hard	Over		30	UCS > 4.0

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

Soil Fraction		USCS Size	ODOT Size
Boulders		Larger than 12”	Larger than 12”
Cobbles		12” to 3”	12” to 3”
Gravel	coarse	3” to ¾”	3” to ¾“
	fine	¾” to 4.75 mm (¾” to #4 Sieve)	¾” to 2.0 mm (¾” to #10 Sieve)
Sand	coarse	4.75 mm to 2.0 mm (#4 to #10 Sieve)	2.0 mm to 0.42 mm (#10 to #40 Sieve)
	medium	2.0 mm to 0.42 mm (#10 to #40 Sieve)	-
	fine	0.42 mm to 0.074 mm (#40 to #200 Sieve)	0.42 mm to 0.074 mm (#40 to #200 Sieve)
Silt		0.074 mm to 0.005 mm (#200 to 0.005 mm)	0.074 mm to 0.005 mm (#200 to 0.005 mm)
Clay		Smaller than 0.005 mm	Smaller than 0.005 mm

Modifiers of Components - Modifiers of components are as follows:

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 - USCS	Range - ODOT
Dry	0% to 10%	Well below Plastic Limit
Damp	>2% below Plastic Limit	Below Plastic Limit
Moist	2% below to 2% above Plastic Limit	Above PL to 3% below LL
Very Moist	>2% above Plastic Limit	
Wet	^{3} Liquid Limit	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 bedrock hardness:

Term	Blows per foot – SPT (N)		
Very Soft	Below		50
Soft	50/5”	–	50/6”
Medium Hard	50/3”	–	50/4”
Hard	50/1”	–	50/2”
Very Hard 50/0”			

DESCRIPTION OF ROCK TERMS

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

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	

Condition of Fractures

Aperature 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:

RQD %	Rock Index Property Classification
0 – 25%	Very Poor
26 – 50%	Poor
51 – 70%	Fair
71 – 85%	Good
86 – 100%	Very Good

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

MATERIAL CLASSIFIED BY VISUAL INSPECTION

- Sod and Topsoil
- Pavement or Base
- Uncontrolled Fill (Describe)
- Bouldery Zone
- Peat, S-Sedimentary W-Woody F-Fibrous L-Loamy & etc

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

APPENDIX III

PROJECT BORING LOGS:

B-021-1-13, B-021-5-14, B-023-3-14
and B-115-2-13

BORING LOGS

Definitions of Abbreviations

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

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

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

<u>Classification Test Data</u>

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-70-13.10 - PHASE 6A	DRILLING FIRM / OPERATOR: RII / T.F.	DRILL RIG: CME 750X (SN 310218)	STATION / OFFSET: 6000+84.54 / 7.6' LT	EXPLORATION ID B-021-1-13
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / J.P.	HAMMER: AUTOMATIC	ALIGNMENT: BL RAMP D6	
PID: 89464 BR ID: N/A	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 10/20/14	ELEVATION: 719.7 (MSL) EOB: 49.8 ft.	PAGE 1 OF 2
START: 1/23/15 END: 1/23/15	SAMPLING METHOD: SPT	ENERGY RATIO (%): 85.7	LAT / LONG: 39.953692, -83.003720	

MATERIAL DESCRIPTION AND NOTES	ELEV. 719.7
1.0' - TOPSOIL (12.0")	718.7
FILL: DENSE, DARK BROWN **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, MOIST. -BRICK FRAGMENTS AND CINDERS PRESENT IN SS-1	716.7
MEDIUM DENSE TO DENSE, GRAYISH BROWN **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, DAMP.	711.7
MEDIUM DENSE TO DENSE, BROWN **GRAVEL WITH SAND**, TRACE SILT, TRACE CLAY, DAMP TO MOIST.	704.2
DENSE TO VERY DENSE, BROWN **GRAVEL WITH SAND**, TRACE TO LITTLE SILT, TRACE CLAY, DAMP TO MOIST.	

DEPTHS: 1 – 29 ft; W (water) at approx. 22

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
5 11 16	39	44	SS-1	-	-	-	-	-	-	-	-	-	11	A-1-b (V)
15 14 11	36	67	SS-2	-	63	25	4	6	2	NP	NP	NP	3	A-1-a (0)
5 4 4	11	33	SS-3	-	-	-	-	-	-	-	-	-	3	A-1-a (V)
11 14 11	36	56	SS-4	-	-	-	-	-	-	-	-	-	6	A-1-b (V)
5 6 8	20	67	SS-5	-	46	39	5	7	3	NP	NP	NP	9	A-1-b (0)
6 8 8	23	83	SS-6	-	-	-	-	-	-	-	-	-	4	A-1-b (V)
17 50/5"	-	91	SS-7	-	-	-	-	-	-	-	-	-	5	A-1-b (V)
17 21 26	67	100	SS-8	-	21	51	13	8	7	NP	NP	NP	7	A-1-b (0)
5 14 16	43	100	SS-9	-	-	-	-	-	-	-	-	-	10	A-1-b (V)
8 16 26	60	33	SS-10	-	42	26	12	11	9	NP	NP	NP	13	A-1-b (0)

BACK FILL

PID: 89464	BR ID: N/A	PROJECT: FRA-70-13.10 - PHASE 6A	STATION / OFFSET: 6000+84.54 / 7.6 LT	START: 1/23/15	END: 1/23/15	PG 2 OF 2	B-021-1-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 689.7	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
DENSE TO VERY DENSE, BROWN **GRAVEL WITH SAND**, TRACE TO LITTLE SILT, TRACE CLAY, DAMP TO MOIST. *(same as above)*		31, 32, 33																
		34	42 34 32	94	22	SS-11	-	-	-	-	-	-	-	-	-	6	A-1-b (V)	
		35, 36, 37, 38																
		39	13 12 33	64	83	SS-12	-	-	-	-	-	-	-	-	-	15	A-1-b (V)	
	677.7	40, 41																
MEDIUM DENSE, BROWNISH GRAY **GRAVEL WITH SAND**, TRACE SILT, TRACE CLAY, MOIST.		42, 43																
		44	5 5 6	16	100	SS-13	-	38	34	19	6	3	NP	NP	NP	10	A-1-b (0)	
	672.7	45, 46																
VERY DENSE, BROWN **GRAVEL WITH SAND**, TRACE SILT, TRACE CLAY, DAMP.		47, 48																
-WC NOT PERFORMED ON SS-14 DUE TO BROKEN JAR	669.9	49 EOB	10 37 50/3"	-	100	SS-14	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 22.0'

ABANDONMENT METHODS, MATERIALS, QUANTITIES: COMPACTED WITH THE AUGER SOIL CUTTINGS

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-13.10 - PHASE 6A	DRILLING FIRM / OPERATOR: RII / T.F./S.B.	DRILL RIG: CME 750X (SN 310218)	STATION / OFFSET: 6001+99.15 / 32.1' LT	EXPLORATION ID **B-021-5-14**
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / J.P./C.D.	HAMMER: AUTOMATIC	ALIGNMENT: BL RAMP D6	
PID: 89464 BR ID: N/A	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 10/20/14	ELEVATION: 727.5 (MSL) EOB: 39.9 ft.	PAGE
START: 1/22/15 END: 4/9/15	SAMPLING METHOD: SPT	ENERGY RATIO (%): 85.7	LAT / LONG: 39.953766, -83.003343	1 OF 2

MATERIAL DESCRIPTION AND NOTES	ELEV. 727.5	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
								GRADATION (%)					ATTERBERG					
0.4' - TOPSOIL (5.0")	727.1																	
MEDIUM DENSE, BROWN TO REDDISH BROWN **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, DAMP TO MOIST. -ROOT FIBERS PRESENT IN SS-1		1, 2	3 7 7	20	44	SS-1	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	
		3, 4, 5	6 11 7	26	22	SS-2	-	-	-	-	-	-	-	-	-	6	A-1-b (V)	
	720.0	6, 7	5 7 8	21	0	SS-3	-	-	-	-	-	-	-	-	-	-		
HARD, BROWN **SILT AND CLAY**, TRACE COARSE TO FINE SAND, MOIST.	719.0	8	19	-	100	2S-3A	4.5+	0	1	6	49	44	28	17	11	20	A-6a (8)	
MEDIUM DENSE TO DENSE, BROWN **GRAVEL WITH SAND**, TRACE SILT, TRACE CLAY, DAMP.		9, 10	7 9 9	26	67	SS-4	-	-	-	-	-	-	-	-	-	5	A-1-b (V)	
	714.5	11, 12	7 12 15	39	67	SS-5	-	41	47	5	4	3	NP	NP	NP	5	A-1-b (0)	
MEDIUM DENSE TO VERY DENSE, BROWN **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, DAMP.		13, 14, 15	16 36 39	107	83	SS-6	-	-	-	-	-	-	-	-	-	5	A-1-a (V)	
		16, 17	7 30 33	90	100	SS-7	-	-	-	-	-	-	-	-	-	4	A-1-a (V)	
-ROCK FRAGMENTS PRESENT THROUGHOUT		18, 19, 20	9 22 11	47	83	SS-8	-	55	28	7	7	3	NP	NP	NP	5	A-1-a (0)	
	705.5	21																
VERY STIFF, BROWN **SANDY SILT**, LITTLE FINE GRAVEL, TRACE CLAY, DAMP.		22, 23, 24	21 31 50/5"	-	106	SS-9	2.50	-	-	-	-	-	-	-	-	15	A-4a (V)	
-BORING TERMINATED @ 24.9' ON 1-22-15 PRIOR TO PLANNED TERMINATION DEPTH. ON 4-9-15, CONTINUED SAMPLING @ 28.5' TO PLANNNED TERMINATION DEPTH.	700.5	25, 26, 27																
VERY DENSE, BROWN **COARSE AND FINE SAND**, TRACE SILT, TRACE CLAY, TRACE FINE GRAVEL, MOIST TO WET.		28, 29	18 31 43	106	100	SS-10	-	-	-	-	-	-	-	-	-	16	A-3a (V)	

PID: 89464	BR ID: N/A	PROJECT: FRA-70-13.10 - PHASE 6A	STATION / OFFSET: 6001+99.15 / 32.1 LT	START: 1/22/15	END: 4/9/15	PG 2 OF 2	B-021-5-14

MATERIAL DESCRIPTION AND NOTES	ELEV. 697.5	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
VERY DENSE, BROWN **COARSE AND FINE SAND**, TRACE SILT, TRACE CLAY, TRACE FINE GRAVEL, MOIST TO WET. *(same as above)*		W 34	9 19 38	81	100	SS-11	-	1	9	79	9	2	NP	NP	NP	19	A-3a (0)	
	687.6	39 EOB	3 21 50/5"	-	106	SS-12	-	-	-	-	-	-	-	-	-	14	A-3a (V)	

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 34.5'; CAVE-IN DEPTH @ 37.5'

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

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-13.10 - PHASE 6A	DRILLING FIRM / OPERATOR: RII / T.F./S.B.	DRILL RIG: CME 750X (SN 310218)	STATION / OFFSET: 6003+43.74 / 39.1' LT	EXPLORATION ID B-023-3-14
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / J.P./C.D.	HAMMER: AUTOMATIC	ALIGNMENT: BL RAMP D6	
PID: 89464 BR ID: N/A	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 10/20/14	ELEVATION: 733.2 (MSL) EOB: 35.0 ft.	PAGE
START: 1/22/15 END: 4/17/15	SAMPLING METHOD: SPT	ENERGY RATIO (%): 85.7	LAT / LONG: 39.953952, -83.002987	1 OF 2

MATERIAL DESCRIPTION AND NOTES	ELEV. 733.2
0.7' - TOPSOIL (8.0")	732.5
FILL: MEDIUM STIFF TO VERY STIFF, BROWN **SILT AND CLAY**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP. -BRICK FRAGMENTS AND ROOT FIBERS PRESENT IN SS-1 -ROOT FIBERS PRESENT IN SS-2	727.7
MEDIUM DENSE, BROWN **GRAVEL WITH SAND**, TRACE SILT, TRACE CLAY, MOIST. -ROCK FRAGMENTS PRESENT IN SS-3	722.7
LOOSE TO MEDIUM DENSE, BROWN **GRAVEL WITH SAND, SILT, AND CLAY**, WET.	721.2
STIFF TO VERY STIFF, GRAY **SILT AND CLAY**, TRACE FINE GRAVEL, MOIST.	715.2
MEDIUM DENSE, BLACK AND GRAY **GRAVEL WITH SAND**, TRACE SILT, TRACE CLAY, WET.	711.2
DENSE TO VERY DENSE, BLACK AND GRAY TO GRAY **GRAVEL WITH SAND**, TRACE SILT, TRACE CLAY, MOIST. -ROCK FRAGMENTS PRESENT IN SS-9 -BORING TERMINATED @ 25.0' ON 1-22-15. ON 4-17-15 OFFSET BORING 5.0' NORTH AND CONTINUED SAMPLING @ 28.5'. -COBBLES AND BOULDERS PRESENT FROM 26.0' TO 30.0'	

DEPTHS: 1 – 29 (W at ~10.8 and ~28.5)

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
4 6 6	17	39	SS-1	4.00	-	-	-	-	-	-	-	-	16	A-6a (V)	
2 3 2	7	44	SS-2	-	4	12	15	47	22	34	22	12	13	A-6a (8)	
6 7 8	21	33	SS-3	-	-	-	-	-	-	-	-	-	8	A-1-b (V)	
7 8 7	21	33	SS-4	-	-	-	-	-	-	-	-	-	11	A-1-b (V)	
4 3 3	9	67	SS-5	-	-	-	-	-	-	-	-	-	22	A-2-6 (V)	
				1.25	-	-	-	-	-	-	-	-	24	A-6a (V)	
2 3 5	11	100	SS-6	2.50	2	0	0	44	54	34	20	14	22	A-6a (10)	
1 2 3	7	89	SS-7	1.50	-	-	-	-	-	-	-	-	24	A-6a (V)	
2 5 5	14	33	SS-8	-	16	37	36	8	3	NP	NP	NP	19	A-1-b (0)	
33 27 27	77	67	SS-9	-	-	-	-	-	-	-	-	-	7	A-1-b (V)	
50/5"	-	0	SS-10	-	-	-	-	-	-	-	-	-	-		

PID: 89464	BR ID: N/A	PROJECT: FRA-70-13.10 - PHASE 6A	STATION / OFFSET: 6003+43.74 / 39.1 LT	START: 1/22/15	END: 4/17/15	PG 2 OF 2	B-023-3-14

MATERIAL DESCRIPTION AND NOTES	ELEV. 703.2	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
DENSE TO VERY DENSE, BLACK AND GRAY TO GRAY **GRAVEL WITH SAND**, TRACE SILT, TRACE CLAY, MOIST. *(same as above)*		31 32 33																
		34	7 12 20	46	56	SS-11	-	44	22	18	10	6	NP	NP	NP	17	A-1-b (0)	
	698.2	EOB 35																

NOTES: SEEPAGE ENCOUNTERED @ 11.0'; GROUNDWATER ENCOUNTERED INITIALLY @ 28.5'

ABANDONMENT METHODS, MATERIALS, QUANTITIES: COMPACTED WITH THE AUGER SOIL CUTTINGS

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-13.10 - PHASE 6A	DRILLING FIRM / OPERATOR: RII / T.F.	DRILL RIG: MOBILE B-53 (SN 624400)	STATION / OFFSET: 277+71.50 / 20.2' LT	EXPLORATION ID **B-115-2-13**
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / S.B.	HAMMER: AUTOMATIC	ALIGNMENT: BL I-71 SB	
PID: 89464 BR ID: FRA-71-1503L	DRILLING METHOD: 3.25" HSA / NQ	CALIBRATION DATE: 4/26/13	ELEVATION: 716.1 (MSL) EOB: 90.7 ft.	PAGE 1 OF 3
START: 5/13/14 END: 5/16/14	SAMPLING METHOD: SPT / RC	ENERGY RATIO (%): 77.7	LAT / LONG: 39.953734, -83.004257	

MATERIAL DESCRIPTION AND NOTES	ELEV. 716.1
0.6' - TOPSOIL (7.0")	715.5
FILL: MEDIUM DENSE, BROWN AND BLACK **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, MOIST. -ROCK FRAGMENTS AND ROOT FIBERS PRESENT IN SS-1	713.1
FILL: MEDIUM DENSE, BROWN TO DARK BROWN **GRAVEL WITH SAND AND SILT**, LITTLE CLAY, MOIST. -CINDERS, BRICK AND SLAG FRAGMENTS PRESENT THROUGHOUT	708.1
FILL: VERY LOOSE TO LOOSE, GRAY AND BLACK **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, WET. -ORGANICS PRESENT THROUGHOUT	703.1
FILL: VERY LOOSE TO LOOSE, GRAY AND BLACK **COARSE AND FINE SAND**, SOME SILT, TRACE CLAY, TRACE FINE GRAVEL, WET. -ROCK FRAGMENTS AND ORGANICS PRESENT IN SS-6 -PETROLEUM ODOR AND WOOD FRAGMENTS PRESENT IN SS-7	698.1
FILL: VERY LOOSE TO LOOSE, BLACK **SILT**, SOME COARSE TO FINE SAND, SOME CLAY, TRACE FINE GRAVEL, WET. -ORGANICS PRESENT THROUGHOUT	693.1
MEDIUM DENSE TO DENSE, BROWNISH GRAY TO BROWN **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST. -CERAMIC TILE FRAGMENTS PRESENT IN SS-10 -ROCK FRAGMENTS PRESENT IN SS-11	

DEPTHS: 1 – 29 (W at 8)

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
7-8-9	22	67	SS-1	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
4-5-4	12	72	SS-2	-	29	26	11	18	16	NP	NP	NP	17	A-2-4 (0)	
4-5-6	14	56	SS-3	-	-	-	-	-	-	-	-	-	13	A-2-4 (V)	
2-2-3	6	100	SS-4	-	30	33	13	15	9	NP	NP	NP	21	A-1-b (0)	
3-1-2	4	56	SS-5	-	-	-	-	-	-	-	-	-	34	A-1-b (V)	
WOH-1-1	3	100	SS-6	-	4	33	29	24	10	NP	NP	NP	49	A-3a (0)	
2-2-6	10	78	SS-7	-	-	-	-	-	-	-	-	-	22	A-3a (V)	
3-2-1	4	56	SS-8	-	1	11	14	53	21	NP	NP	NP	68	A-4b (8)	
2-3-3	8	83	SS-9	-	-	-	-	-	-	-	-	-	41	A-4b (V)	
6-9-13	28	44	SS-10	-	-	-	-	-	-	-	-	-	13	A-1-a (V)	
11-11-13	31	44	SS-11	-	61	21	6	7	5	NP	NP	NP	13	A-1-a (0)	
10-13-15	36	11	SS-12	-	-	-	-	-	-	-	-	-	-	A-1-a (V)	

PID: 89464	BR ID: FRA-71-1503L	PROJECT: FRA-70-13.10 - PHASE 6A	STATION / OFFSET: 277+71.50 / 20.2 LT	START: 5/13/14	END: 5/16/14	PG 2 OF 3	B-115-2-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 686.1
MEDIUM DENSE TO DENSE, BROWNISH GRAY TO BROWN **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST. *(same as above)*	684.1
VERY DENSE, DARK BROWN TO GRAY **GRAVEL**, "AND" COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST. -COBBLES PRESENT @ 33.5' -ROCK FRAGMENTS PRESENT THROUGHOUT -HEAVING SANDS ENCOUNTERED @ 38.5' -INTRODUCED MUD @ 38.5'	674.1
HARD, GRAY **SILTY CLAY**, LITTLE COARSE TO FINE SAND, MOIST.	669.1
DENSE, GRAY **GRAVEL WITH SAND AND SILT**, TRACE CLAY, WET.	664.1
GRAY **SILTY CLAY.** -BOULDER ENCOUNTERED @ 55.0' -SOIL TYPE DETERMINED FROM FIELD OBSERVATION OF AUGER CUTTINGS	

DEPTHS: 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

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
25	-	0	3S-12A	-	-	-	-	-	-	-	-	-	-		
10 30 35	84	72	SS-13	-	50	32	5	7	6	NP	NP	NP	12	A-1-a (0)	
22 32 48	104	94	SS-14	-	-	-	-	-	-	-	-	-	8	A-1-a (V)	
10 12 45	74	78	SS-15	4.50	0	5	5	32	58	40	19	21	24	A-6b (12)	
12 12 18	39	94	SS-16	-	-	-	-	-	-	-	-	-	23	A-2-4 (V)	
50/2"	-	0	SS-17	-	-	-	-	-	-	-	-	-	-		
50/3"	-	0	SS-18	-	-	-	-	-	-	-	-	-	-		

PID: 89464	BR ID: FRA-71-1503L	PROJECT: FRA-70-13.10 - PHASE 6A	STATION / OFFSET: 277+71.50 / 20.2 LT	START: 5/13/14	END: 5/16/14	PG 3 OF 3	B-115-2-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 654.0	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
GRAY **SILTY CLAY.** *(same as above)*	652.6	TR 63																
SHALE : GRAY, HIGHLY WEATHERED, VERY WEAK. AUGER REFUSAL @ 64.2'	651.9	64	45 50/2"	-	100	SS-19	-	-	-	-	-	-	-	-	-	9	Rock (V)	
CLAYSTONE : GRAY, UNWEATHERED, VERY WEAK TO SLIGHTLY STRONG, MEDIUM BEDDED, MODERATELY TO HIGHLY FRACTURED, OPEN APERTURES, SLIGHTLY ROUGH; RQD 45%, REC 95%.		65	58		100	RC-1											CORE	
		66–70	0		88	RC-2											CORE	
	640.4	71–75	86		100	RC-3											CORE	
SHALE : GRAY, HIGHLY WEATHERED, WEAK, MEDIUM BEDDED, CALCAREOUS, FISSILE, FRACTURED TO HIGHLY FRACTURED, OPEN APERTURES, SLIGHTLY ROUGH; RQD 0%, REC 37%.		76–80	0		30	RC-4											CORE	
	632.9	81–85	45		75	RC-5											CORE	
LIMESTONE : GRAYISH BROWN, SLIGHTLY WEATHERED, MODERATELY STRONG, THIN TO MEDIUM BEDDED, CHERTY, PYRITIC,, MODERATELY FRACTURED, OPEN APERTURES, SLIGHTLY TO VERY ROUGH; RQD 91%, REC 100%. -QU @ 83.2' = 8,867 PSI	625.4	86–90 EOB	92		100	RC-6											CORE	

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 8.5'

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

B-115-2-13 – RC-1 and RC-2 – Depth from 64.2 to 70.7 feet

B-115-2-13 – RC-3 and RC-4 – Depth from 70.7 to 80.7 feet

B-115-2-13 – RC-5 and RC-6 – Depth from 80.7 to 90.7 feet

APPENDIX IV

LABORATORY TEST RESULTS

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Engineering Consultants

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Columbus, OH 43231
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9885 Rockside Road
Cleveland, OH 44125
Phone (216) 573-0955

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

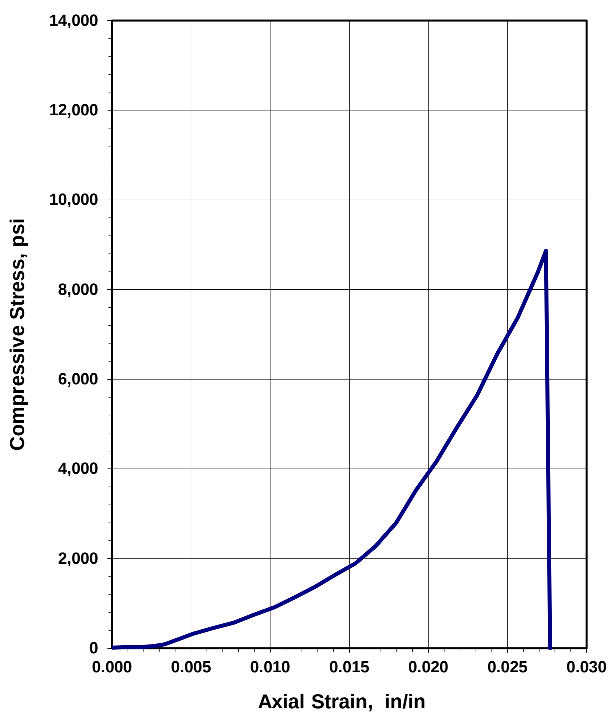
Unconfined Compressive Strength
of Intact Rock Core Specimens (ASTM D 7012-04)

Project: FRA-70-13.10 - Project 6A
Project No.: W-13-072
Date of Testing: 6/23/2014
Test Performed by: K.R./T.K.

Rock Description: Dolomitic Limestone

Boring No.:	B-115-2-13	Average Length:	3.9 in
Sample No:	RC-5	Average Diameter:	1.778 in
Depth (ft):	83.2	Length to diameter ratio:	2.193
Moisture condition:	As received	Cross Sectional Area:	2.482 in^2
Rate of Loading:	56.9 lbs/sec	Failure Load:	22,010 lbs
Testing Time:	387 sec	Axial Strain at Failure:	0.0274 in/in
(Rate 2-15 minutes to failure)		Stress:	8,867 psi

Unconfined Compression Test



Before Testing

After Failure

REMARKS:

APPENDIX V

MSE WALL CALCULATIONS

RESOURCE INTERNATIONAL, INC.
6350 PRESIDENTIAL GATEWAY
COLUMBUS, OHIO 43231
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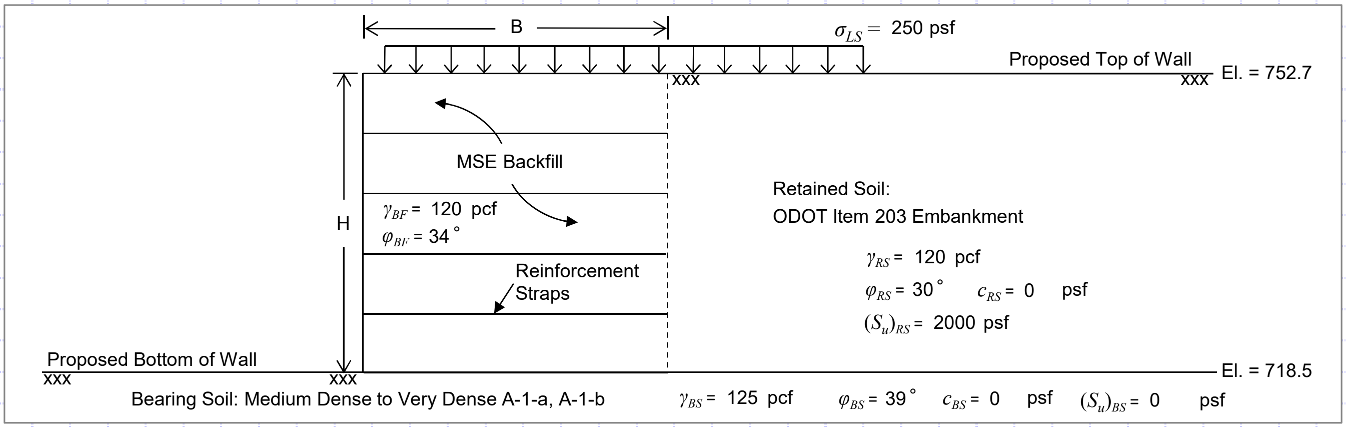
Retaining Wall E7 - Sta. 700+38 to 702+50 - B-021-1-13, B-021-5-14, B-023-3-14 - 34.2 ft. Wall Height



MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	34.2	ft
MSE Wall Width (Reinforcement Length), (B) =	23.9	ft
MSE Wall Length, (L) =	212	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion [1], (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	39	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(S_u)_{BS}$] =	0	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

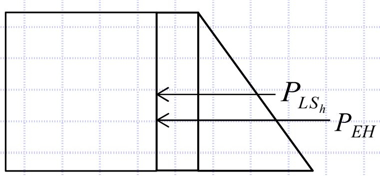
	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

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

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

Sliding Force:

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



$$P_{EH} = \tfrac{1}{2}\gamma_{RS} H^2 K_a \gamma_{EH} = \text{½(120 pcf)(34.2 ft)²(0.297)(1.5)} = 31.26 \text{ kip/ft}$$

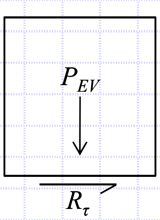
$$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS} = \text{(250 psf)(34.2 ft)(0.297)(1.75)} = 4.44 \text{ kip/ft}$$

$$P_H = 31.26 \text{ kip/ft} + 4.44 \text{ kip/ft} = 35.70 \text{ kip/ft}$$

Check Sliding Resistance - Drained Condition

Nominal Sliding Resistance:

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



$$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV} = \text{(120 pcf)(34.2 ft)(23.9 ft)(1.00)} = 98.09 \text{ kip/ft}$$

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

$$\tan\delta = \tan(39) \le \tan(34) \longrightarrow 0.81 \le 0.67 \longrightarrow \tan\delta = 0.67$$

$$R_\tau = \text{(98.09 kip/ft)(0.67)} = 65.72 \text{ kip/ft}$$

Verify Sliding Force Less Than Factored Sliding Resistance - Drained Condition

$$P_H \le R_\tau \cdot \phi_\tau \longrightarrow 35.70 \text{ kip/ft} \le (65.72 \text{ kip/ft})(1.0) = 65.72 \text{ kip/ft} \longrightarrow 35.70 \text{ kip/ft} \le 65.72 \text{ kip/ft}$$ **OK**

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

RESOURCE INTERNATIONAL, INC.
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MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	34.2	ft
MSE Wall Width (Reinforcement Length), (B) =	23.9	ft
MSE Wall Length, (L) =	212	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	39	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	0	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

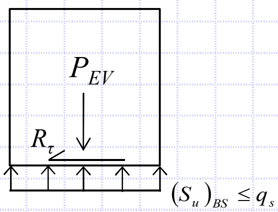
	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

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

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

Check Sliding Resistance - Undrained Condition

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



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

$q_s = \sigma_v / 2$ = (4.10 ksf) / 2 = 2.05 ksf

$\sigma_v = P_{EV} / B$ = (98.09 kip/ft) / (23.9 ft) = 4.10 ksf

R_τ = (N/A ksf ≤ 2.05 ksf)(23.9 ft) = N/A kip/ft

Verify Sliding Force Less Than Factored Sliding Resistance - Undrained Condition

$P_H \le R_\tau \cdot \phi_\tau$ $\longrightarrow$ N/A $\longrightarrow$ N/A

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) =	34.2	ft
MSE Wall Width (Reinforcement Length), (B) =	23.9	ft
MSE Wall Length, (L) =	212	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, $[(S_u)_{RS}]$ =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

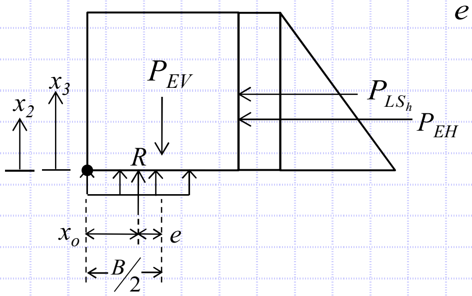
Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	39	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	0	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

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

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

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



$$e = B/2 - x_o$$

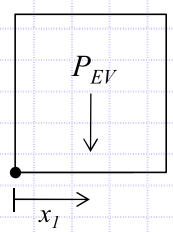
$$x_o = \frac{M_{EV} - M_H}{P_{EV}}$$ = (1172.18 kip·ft/ft - 432.29 kip·ft/ft) / (98.09 kip/ft) = 7.54 ft

M_{EV}	=	1172.18	kip·ft/ft
M_H	=	432.29	kip·ft/ft
P_{EV}	=	98.09	kip/ft

Defined below

$e =$ (23.9 ft)/2 - 7.54 ft = 4.41 ft

Resisting Moment, M_{EV}:



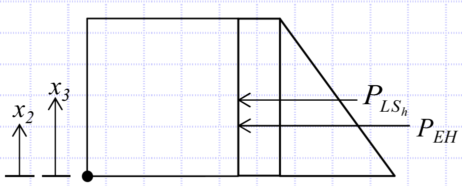
$$M_{EV} = P_{EV}(x_1)$$

$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV}$ = (120 pcf)(34.2 ft)(23.9 ft)(1.00) = 98.09 kip/ft

$x_1 = B/2$ = (23.9 ft) / 2 = 11.95 ft

$M_{EV} =$ (98.09 kip/ft)(11.95 ft) = 1172.18 kip·ft/ft

Overturning Moment, M_H:



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

$P_{EH} = \frac{1}{2}\gamma_{RS} H^2 K_a \gamma_{EH}$ = ½(120 pcf)(34.2 ft)²(0.297)(1.5) = 31.26 kip/ft

$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS}$ = (250 psf)(34.2 ft)(0.297)(1.75) = 4.44 kip/ft

$x_2 = H/3$ = (34.2 ft) / 3 = 11.40 ft

$x_3 = H/2$ = (34.2 ft) / 2 = 17.10 ft

$M_H =$ (31.26 kip/ft)(11.4 ft) + (4.44 kip/ft)(17.10 ft) = 432.29 kip·ft/ft

Check Eccentricity

$e < e_{max} \longrightarrow$ 4.41 ft < 7.97 ft **OK**

Limiting Eccentricity: $e_{max} = B/3 \longrightarrow e_{max} =$ (23.9 ft) / 3 = 7.97 ft

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	34.2	ft
MSE Wall Width (Reinforcement Length), (B) =	23.9	ft
MSE Wall Length, (L) =	212	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

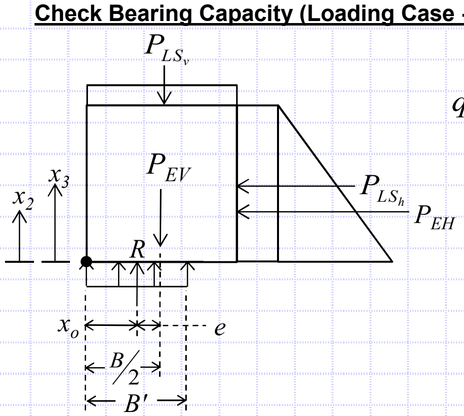
Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	39	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	0	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

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

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

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



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

$B' = B - 2e$ = 23.9 ft - 2(3.03 ft) = 17.84 ft

$e = B/2 - x_o$ = (23.9 ft) / 2 - 8.92 ft = 3.03 ft

$x_o = \dfrac{M_V - M_H}{P_V}$ = (1707.32 kip·ft/ft - 432.41 kip·ft/ft) / 142.87 kip/ft = 8.92 ft

q_{eq} = (142.87 kip/ft) / (17.84 ft) = 8.01 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)(34.2 ft)(23.9 ft)(1.35)](11.95 ft) + [(250 psf)(23.9 ft)(1.75)](11.95 ft) = 1707.32 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(120 pcf)(34.2 ft)²(0.297)(1.5)](11.4 ft) + [(250 psf)(34.2 ft)(0.297)(1.75)](17.1 ft) = 432.41 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)(34.2 ft)(23.9 ft)(1.35) + (250 psf)(23.9 ft)(1.75) = 142.87 kip/ft

Check Bearing Resistance - Drained Condition

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

$N_{cm} = N_c s_c i_c$ = 72.55

N_c = 67.87

s_c = 1+(17.84 ft/212 ft)(55.96/67.87)

= 1.069

i_c = 1.000 (Assumed)

$N_{qm} = N_q s_q d_q i_q$ = 62.69

N_q = 55.96

s_q = 1.068

d_q = 1+2tan(39°)[1-sin(39°)]²tan⁻¹(4.0 ft/17.84 ft)

1.049

i_q = 1.000 (Assumed)

C_{wq} = 0.0 ft > 4.0 ft = 0.500

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 89.11

N_γ = 92.25

s_γ = 0.966

i_γ = 1.000 (Assumed)

$C_{w\gamma}$ = 0.0 ft < 1.5(17.84 ft) + 4.0 ft = 0.500

q_n = (0 psf)(72.553) + (125 pcf)(4.0 ft)(62.694)(0.500) + ½(125 pcf)(17.8 ft)(89.114)(0.500) = 65.35 ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

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

$q_{eq} \le q_n \cdot \phi_b$ ⟶ 8.01 ksf ≤ (65.35 ksf)(0.65) = 42.48 ksf ⟶ 8.01 ksf ≤ 42.48 ksf **OK**

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MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	34.2	ft
MSE Wall Width (Reinforcement Length), (B) =	23.9	ft
MSE Wall Length, (L) =	212	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	39	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	0	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

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

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

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

Check Bearing Resistance - Undrained Condition

Nominal Bearing Resistance: $$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + ½\gamma B N_{\gamma m} C_{w\gamma}$$

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

- N_c = 5.140
- s_c = 1+(17.84 ft/[(5)(212 ft)]) = 1.017
- i_c = 1.000 (Assumed)

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

- N_q = 1.000
- s_q = 1.000
- d_q = 1+2tan(0°)[1-sin(0°)]²tan⁻¹(4.0 ft/17.84 ft) = 1.000
- i_q = 1.000 (Assumed)
- C_{wq} = 0.0 ft > 4.0 ft = 0.500

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

- N_γ = 0.000
- s_γ = 1.000
- i_γ = 1.000 (Assumed)
- $C_{w\gamma}$ = 0.0 ft < 1.5(17.84 ft) + 4.0 ft = 0.500

$q_n =$ (0 psf)(5.230) + (125 pcf)(4.0 ft)(1.000)(0.500) + ½(125 pcf)(17.8 ft)(0.000)(0.500) = N/A ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

$q_{eq} \leq q_n \cdot \phi_b$ $\longrightarrow$ 8.01 ksf ≤ (N/A ksf)(0.65) = N/A ksf $\longrightarrow$ N/A

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) =	34.2	ft
MSE Wall Width (Reinforcement Length), (B) =	23.9	ft
MSE Wall Length, (L) =	212	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, $[(S_u)_{RS}]$ =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

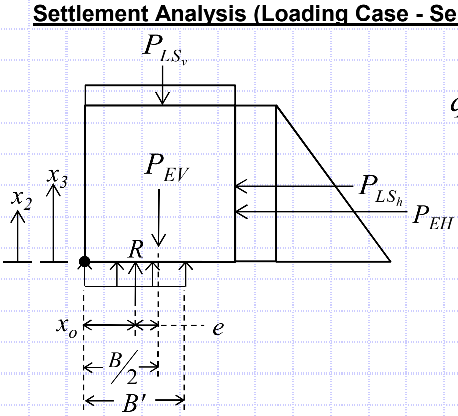
Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	39	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	0	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

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

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

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



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

$$B' = B - 2e \quad = \quad 23.9 \text{ ft} - 2(2.7 \text{ ft}) \quad = \quad 18.50 \quad \text{ft}$$

$$e = B/2 - x_o \quad = \quad (23.9 \text{ ft}) / 2 - 9.25 \text{ ft} \quad = \quad 2.70 \quad \text{ft}$$

$$x_o = \frac{M_V - M_H}{P_V} \quad = \quad (1243.52 \text{ kip·ft/ft} - 281.03 \text{ kip·ft/ft}) / 104.06 \text{ kip/ft} \quad = \quad 9.25 \quad \text{ft}$$

$$q_{eq} = \quad (104.06 \text{ kip/ft}) / (18.5 \text{ ft}) \quad = \quad 5.62 \quad \text{ksf}$$

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

$$M_V = \quad [(120 \text{ pcf})(34.2 \text{ ft})(23.9 \text{ ft})(1.00)](12.0 \text{ ft}) + [(250 \text{ psf})(23.9 \text{ ft})(1.00)](12.0 \text{ ft}) \quad = \quad 1243.52 \quad \text{kip·ft/ft}$$

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

$$M_H = \quad [\tfrac{1}{2}(120 \text{ pcf})(34.2 \text{ ft})^2(0.297)(1.00)](11.4 \text{ ft}) + [(250 \text{ psf})(34.2 \text{ ft})(0.297)(1.00)](17.1 \text{ ft}) \quad = \quad 281.03 \quad \text{kip·ft/ft}$$

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

$$P_V = \quad (120 \text{ pcf})(34.2 \text{ ft})(23.9 \text{ ft})(1.00) + (250 \text{ psf})(23.9 \text{ ft})(1.00) \quad = \quad 104.06 \quad \text{kip/ft}$$

Settlement, Time Rate of Consolidation and Differential Settlement:

Boring	Total Settlement at Center of Reinforced Soil Mass	Total Settlement at Wall Facing	Time for 100% Consolidation	Distance Between Borings Along Wall Facing	Differential Settlement Along Wall Facing
B-021-1-13	2.159 in	1.787 in	0 days		
B-021-5-14	1.349 in	1.112 in	5 days	105 ft	1/1870
B-023-3-14	1.469 in	1.219 in	8 days	135 ft	1/15140

W-13-072 - FRA-70-13.10 - Retaining Wall E7
MSE Wall Settlement - Sta. 700+38 to 702+50

Calculated By: BRT Date: 7/9/2019
Checked By: JPS Date: 7/11/2019

Boring B-021-1-13

H=	34.2	ft	Total wall height
B'=	18.5	ft	Effective footing width due to eccentricity
D_w =	0.0	ft	Depth below bottom of footing
q_e =	5,620	psf	Equivalent bearing pressure at bottom of wall

																				Total Settlement at Center of Reinforced Soil Mass					Total Settlement at Facing of Wall				
Layer	Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
1	A-1-b	G	0.0	2.0	2.0	1.0	130	260	130	68	4,068					39	78	315	0.05	0.999	5,617	5,685	0.012	0.147	0.500	2,810	2,877	0.010	0.124
2	A-1-a	G	2.0	4.5	2.5	3.3	125	573	416	213	4,213					23	40	131	0.18	0.984	5,530	5,744	0.027	0.326	0.499	2,804	3,017	0.022	0.262
	A-1-a	G	4.5	7.0	2.5	5.8	125	885	729	370	4,370					23	36	117	0.31	0.931	5,234	5,604	0.025	0.302	0.494	2,778	3,148	0.020	0.238
3	A-1-b	G	7.0	9.5	2.5	8.3	125	1,198	1,041	526	4,526					23	33	109	0.45	0.853	4,791	5,318	0.023	0.277	0.485	2,725	3,251	0.018	0.218
	A-1-b	G	9.5	12.0	2.5	10.8	125	1,510	1,354	683	4,683					23	31	103	0.58	0.767	4,311	4,994	0.021	0.253	0.471	2,645	3,328	0.017	0.201
	A-1-b	G	12.0	14.5	2.5	13.3	125	1,823	1,666	839	4,839					23	30	98	0.72	0.687	3,860	4,699	0.019	0.229	0.453	2,545	3,384	0.015	0.185
4	A-1-b	G	14.5	19.5	5.0	17.0	135	2,498	2,160	1,099	5,099					65	78	316	0.92	0.585	3,285	4,384	0.010	0.114	0.422	2,372	3,471	0.008	0.095
	A-1-b	G	19.5	24.5	5.0	22.0	135	3,173	2,835	1,462	5,462					65	72	279	1.19	0.481	2,702	4,165	0.008	0.098	0.379	2,132	3,594	0.007	0.084
	A-1-b	G	24.5	32.5	8.0	28.5	135	4,253	3,713	1,934	5,934					65	66	245	1.54	0.387	2,173	4,107	0.011	0.128	0.329	1,847	3,781	0.010	0.114
	A-1-b	G	32.5	41.0	8.5	36.8	135	5,400	4,826	2,533	6,533					65	60	215	1.99	0.308	1,729	4,262	0.009	0.107	0.276	1,553	4,086	0.008	0.099
5	A-1-b	G	41.0	46.0	5.0	43.5	125	6,025	5,713	2,998	6,998					16	14	63	2.35	0.263	1,478	4,476	0.014	0.167	0.243	1,364	4,362	0.013	0.156
6	A-1-b	G	46.0	48.8	2.8	47.4	135	6,403	6,214	3,256	7,256					120	101	474	2.56	0.242	1,362	4,618	0.001	0.011	0.226	1,272	4,528	0.001	0.010
																						Total Settlement:		2.159 in			Total Settlement:		1.787 in

1. $\sigma_p' = \sigma_{vo}'+\sigma_m$; Estimate σ_m of 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 = 0.10(Cc) for 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-13-072 - FRA-70-13.10 - Retaining Wall E7
MSE Wall Settlement - Sta. 700+38 to 702+50

Calculated By: BRT Date: 7/9/2019
Checked By: JPS Date: 7/11/2019

Boring B-021-5-14

H=	25.5	ft	Total wall height
B'=	13.8	ft	Effective footing width due to eccentricity
D_w =	0.0	ft	Depth below bottom of footing
q_e =	4,310	psf	Equivalent bearing pressure at bottom of wall

																				Total Settlement at Center of Reinforced Soil Mass					Total Settlement at Facing of Wall				
Layer	Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
1	A-1-b	G	0.0	3.5	3.5	1.8	125	438	219	110	4,110					23	45	151	0.13	0.994	4,282	4,392	0.037	0.447	0.500	2,153	2,263	0.031	0.367
2	A-6a	C	3.5	4.5	1.0	4.0	125	563	500	250	4,250	28	0.162	0.016	0.491				0.29	0.942	4,059	4,309	0.014	0.168	0.495	2,135	2,385	0.011	0.128
3	A-1-b	G	4.5	6.5	2.0	5.5	125	813	688	344	4,344					33	52	180	0.40	0.882	3,801	4,145	0.012	0.144	0.489	2,107	2,451	0.009	0.114
	A-1-b	G	6.5	9.0	2.5	7.8	125	1,125	969	485	4,485					33	49	164	0.56	0.779	3,359	3,844	0.014	0.165	0.473	2,039	2,524	0.011	0.131
4	A-1-a	G	9.0	13.5	4.5	11.3	135	1,733	1,429	727	4,727					98	131	746	0.82	0.634	2,732	3,459	0.004	0.049	0.438	1,889	2,615	0.003	0.040
	A-1-a	G	13.5	18.0	4.5	15.8	135	2,340	2,036	1,053	5,053					47	57	201	1.14	0.497	2,141	3,195	0.011	0.129	0.387	1,667	2,721	0.009	0.110
5	A-4a	C	18.0	23.0	5.0	20.5	130	2,990	2,665	1,386	5,386	22	0.108	0.011	0.444				1.49	0.399	1,720	3,106	0.013	0.157	0.336	1,448	2,834	0.012	0.139
6	A-3a	G	23.0	29.0	6.0	26.0	135	3,800	3,395	1,773	5,773					102	106	388	1.88	0.323	1,392	3,165	0.004	0.047	0.287	1,238	3,010	0.004	0.043
	A-3a	G	29.0	35.9	6.9	32.5	135	4,732	4,266	2,241	6,241					102	98	344	2.35	0.263	1,133	3,374	0.004	0.043	0.243	1,046	3,287	0.003	0.040
																						Total Settlement:		1.349 in			Total Settlement:		1.112 in

1. $\sigma_p' = \sigma_{vo}'+\sigma_m$; Estimate σ_m of 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 = 0.10(Cc) for 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)

W-13-072 - FRA-70-13.10 - Retaining Wall E7
MSE Wall Settlement - Sta. 700+38 to 702+50

Calculated By: BRT Date: 07/09/2019
Checked By: JPS Date: 07/11/2019

Boring B-021-5-14

H=	25.5	ft	Total wall height
B'=	13.8	ft	Effective footing width due to eccentricity
D_w =	0.0	ft	Depth below bottom of footing
q_e =	4,310	psf	Equivalent bearing pressure at bottom of wall

	A-6a	A-4a			
c_v =	600	1000		ft^2/yr	Coefficient of consolidation
t =	5	5		days	Time following completion of construction
H_{dr} =	1	2.5		ft	Length of longest drainage path considered
T_v =	8.219	2.192			Time factor
U =	100	100		%	Degree of consolidation

$(S_c)_t$ = 1.112 in Settlement complete at 100% 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 100% of Primary Consolidation: $(S_c)_t$ [11] (in)	Layer Settlement (in)
1	A-1-b	G	0.0	3.5	3.5	1.8	125	438	219	110	4,110					23	45	151	0.13	0.500	2,153	2,263	0.031	0.367	0.367	0.367	0.367
2	A-6a	C	3.5	4.5	1.0	4.0	125	563	500	250	4,250	28	0.162	0.016	0.491				0.29	0.495	2,135	2,385	0.011	0.128	0.128	0.128	0.128
3	A-1-b	G	4.5	6.5	2.0	5.5	125	813	688	344	4,344					33	52	180	0.40	0.489	2,107	2,451	0.009	0.114	0.245	0.114	0.245
	A-1-b	G	6.5	9.0	2.5	7.8	125	1,125	969	485	4,485					33	49	164	0.56	0.473	2,039	2,524	0.011	0.131		0.131	
4	A-1-a	G	9.0	13.5	4.5	11.3	135	1,733	1,429	727	4,727					98	131	746	0.82	0.438	1,889	2,615	0.003	0.040	0.151	0.040	0.151
	A-1-a	G	13.5	18.0	4.5	15.8	135	2,340	2,036	1,053	5,053					47	57	201	1.14	0.387	1,667	2,721	0.009	0.110		0.110	
5	A-4a	C	18.0	23.0	5.0	20.5	130	2,990	2,665	1,386	5,386	22	0.108	0.011	0.444				1.49	0.336	1,448	2,834	0.012	0.139	0.139	0.139	0.139
6	A-3a	G	23.0	29.0	6.0	26.0	135	3,800	3,395	1,773	5,773					102	106	388	1.88	0.287	1,238	3,010	0.004	0.043	0.083	0.043	0.083
	A-3a	G	29.0	35.9	6.9	32.5	135	4,732	4,266	2,241	6,241					102	98	344	2.35	0.243	1,046	3,287	0.003	0.040		0.040	

Settlement Remaining After Hold Period: 0.000 in

1. $\sigma_p' = \sigma_{vo}' + \sigma_m$; Estimate σ_m of 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 = 0.10(Cc) for 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. $S_c = [C_r/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ 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)
11. $(S_c)_t = S_c(U/100)$; U = 100 for all granular soils at time t = 0

W-13-072 - FRA-70-13.10 - Retaining Wall E7
MSE Wall Settlement - Sta. 700+38 to 702+50

Calculated By: BRT Date: 7/9/2019
Checked By: JPS Date: 7/11/2019

Boring B-023-3-14

H=	8.5	ft	Total wall height
B'=	6.8	ft	Effective footing width due to eccentricity
D_w =	0.0	ft	Depth below bottom of footing
q_e =	1,500	psf	Equivalent bearing pressure at bottom of wall

																				Total Settlement at Center of Reinforced Soil Mass					Total Settlement at Facing of Wall				
Layer	Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
1	A-6a	C	0.0	3.5	3.5	1.8	115	403	201	92	4,092	34	0.216	0.022	0.538				0.26	0.956	1,435	1,527	0.060	0.720	0.497	745	837	0.047	0.566
2	A-1-b	G	3.5	6.0	2.5	4.8	125	715	559	262	4,262					21	35	115	0.70	0.697	1,045	1,308	0.015	0.182	0.455	683	945	0.012	0.145
	A-1-b	G	6.0	8.5	2.5	7.3	125	1,028	871	419	4,419					21	32	105	1.07	0.524	786	1,205	0.011	0.131	0.399	598	1,017	0.009	0.110
3	A-6a	C	8.5	11.0	2.5	9.8	115	1,315	1,171	563	4,563	34	0.216	0.022	0.538				1.43	0.412	617	1,180	0.011	0.135	0.343	515	1,078	0.010	0.119
	A-6a	C	11.0	13.5	2.5	12.3	115	1,603	1,459	694	4,694	34	0.216	0.022	0.538				1.80	0.336	505	1,199	0.008	0.100	0.296	445	1,139	0.008	0.091
	A-6a	C	13.5	16.0	2.5	14.8	115	1,890	1,746	826	4,826	34	0.216	0.022	0.538				2.17	0.284	425	1,251	0.006	0.076	0.259	388	1,214	0.006	0.070
4	A-1-b	G	16.0	20.0	4.0	18.0	125	2,390	2,140	1,017	5,017					14	17	69	2.65	0.235	352	1,369	0.008	0.090	0.220	330	1,347	0.007	0.085
5	A-1-b	G	20.0	25.0	5.0	22.5	135	3,065	2,728	1,324	5,324					77	88	379	3.31	0.190	284	1,608	0.001	0.013	0.182	272	1,596	0.001	0.013
	A-1-b	G	25.0	30.0	5.0	27.5	135	3,740	3,403	1,687	5,687					77	82	338	4.04	0.156	234	1,920	0.001	0.010	0.151	227	1,914	0.001	0.010
6	A-1-b	G	30.0	33.0	3.0	31.5	135	4,145	3,943	1,977	5,977					46	46	154	4.63	0.136	205	2,181	0.001	0.010	0.133	200	2,177	0.001	0.010
																						Total Settlement:		1.469 in			Total Settlement:		1.219 in

1. $\sigma_p' = \sigma_{vo}' + \sigma_m$. Estimate σ_m of 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 = 0.10(Cc) for 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-13-072 - FRA-70-13.10 - Retaining Wall E7
MSE Wall Settlement - Sta. 700+38 to 702+50

Calculated By: BRT   Date: 07/09/2019
Checked By: JPS   Date: 07/11/2019

Boring B-023-3-14

H=	8.5	ft	Total wall height
B'=	6.8	ft	Effective footing width due to eccentricity
D_w =	0.0	ft	Depth below bottom of footing
q_e =	1,500	psf	Equivalent bearing pressure at bottom of wall

	A-6a				
c_v =	600			ft^2/yr	Coefficient of consolidation
t =	8			days	Time following completion of construction
H_{dr} =	2.5			ft	Length of longest drainage path considered
T_v =	2.104				Time factor
U =	100			%	Degree of consolidation

$(S_c)_t$ = 1.219 in Settlement complete at 100% of primary consolidation

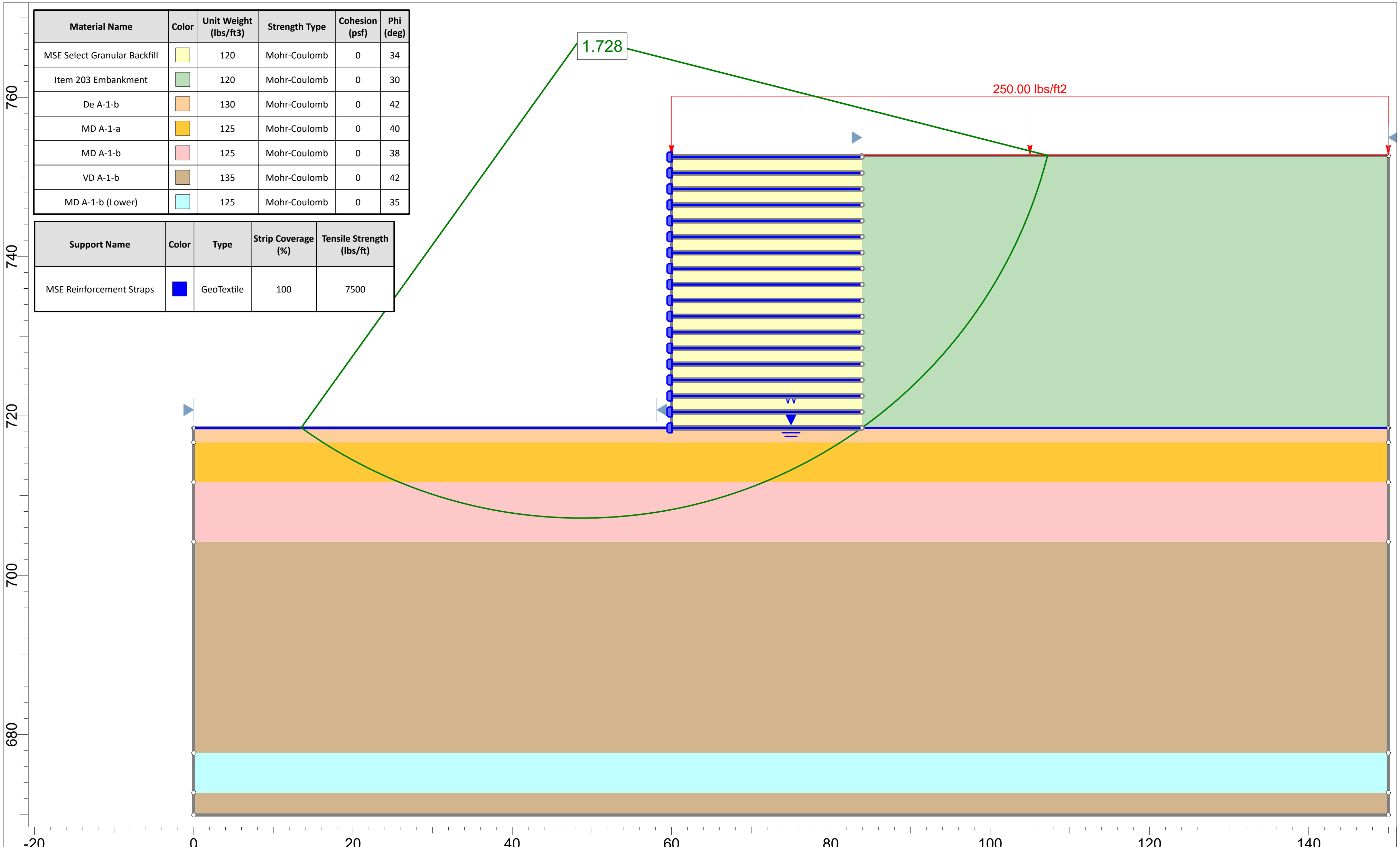
Layer	Soil Type	Soil Type	Layer Depth (ft)		Layer Thickness (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	Total Settlement at Facing of Wall: S_c (9,10) (ft)	S_c (in)	Layer Settlement (in)	Settlement Complete at 100% 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	4,092	34	0.216	0.022	0.538				0.26	0.497	745	837	0.047	0.566	0.566	0.566	0.566
2	A-1-b	G	3.5	6.0	2.5	4.8	125	715	559	262	4,262					21	35	115	0.70	0.455	683	945	0.012	0.145	0.256	0.145	0.256
	A-1-b	G	6.0	8.5	2.5	7.3	125	1,028	871	419	4,419					21	32	105	1.07	0.399	598	1,017	0.009	0.110		0.110	
3	A-6a	C	8.5	11.0	2.5	9.8	115	1,315	1,171	563	4,563	34	0.216	0.022	0.538				1.43	0.343	515	1,078	0.010	0.119	0.280	0.119	0.280
	A-6a	C	11.0	13.5	2.5	12.3	115	1,603	1,459	694	4,694	34	0.216	0.022	0.538				1.80	0.296	445	1,139	0.008	0.091		0.091	
	A-6a	C	13.5	16.0	2.5	14.8	115	1,890	1,746	826	4,826	34	0.216	0.022	0.538				2.17	0.259	388	1,214	0.006	0.070		0.070	
4	A-1-b	G	16.0	20.0	4.0	18.0	125	2,390	2,140	1,017	5,017					14	17	69	2.65	0.220	330	1,347	0.007	0.085	0.085	0.085	0.085
5	A-1-b	G	20.0	25.0	5.0	22.5	135	3,065	2,728	1,324	5,324					77	88	379	3.31	0.182	272	1,596	0.001	0.013	0.023	0.013	0.023
	A-1-b	G	25.0	30.0	5.0	27.5	135	3,740	3,403	1,687	5,687					77	82	338	4.04	0.151	227	1,914	0.001	0.010		0.010	
6	A-1-b	G	30.0	33.0	3.0	31.5	135	4,145	3,943	1,977	5,977					46	46	154	4.63	0.133	200	2,177	0.001	0.010	0.010	0.010	0.010

Settlement Remaining After Hold Period: 0.000 in

1. $\sigma_p' = \sigma_{vo}' + \sigma_m$; Estimate σ_m of 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. $C_r = 0.10(Cc)$ for 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 Embankment		120	Mohr-Coulomb	0	30
De A-1-b		130	Mohr-Coulomb	0	42
MD A-1-a		125	Mohr-Coulomb	0	40
MD A-1-b		125	Mohr-Coulomb	0	38
VD A-1-b		135	Mohr-Coulomb	0	42
MD A-1-b (Lower)		125	Mohr-Coulomb	0	35

Support Name	Color	Type	Strip Coverage (%)	Tensile Strength (lbs/ft)
MSE Reinforcement Straps		GeoTextile	100	7500



Resource International, Inc.
Planning | Engineering | Construction Management | Technology

SLIDEINTERPRET 8.020

Project	Retaining Wall E5 - Sta. 700+38 to 701+50 - MSE Wall Global Stability
Analysis Description	34.2 ft Wall Height - Drained - Circular - Spencer
Drawn By	BRT
Scale	1:200
Company	Resource International, Inc.
Date	07/18/2019
File Name	Retaining Wall E7 - Global Stability.slim

APPENDIX VI

CELLULAR CONCRETE WALL CALCULATIONS

W-13-072 - FRA-70-13.10 - Retaining Wall E7
MSE Wall with Cellular Concrete Backfill Settlement - Sta. 702+50 to 703+00

Boring	Boring Elevation	Top of Wall Elevation (ft msl)	Bottom of Wall Elevation (ft msl)	Wall Height (ft)	Pressure at Bottom of Wall [1] (psf)	Total Settlement at Center of Wall (in)	Total Settlement at Wall Facing (in)	Time for 100% Consolidation (Days)
B-021-1-13	719.7	755.3	716.0	39.3	1,551	1.24	0.91	0
B-021-5-14	727.5	752.7	718.5	34.2	1,398	0.57	0.41	5

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

W-13-072 - FRA-70-13.10 - Retaining Wall E7
MSE Wall with Cellular Concrete Backfill Settlement - Sta. 702+50 to 703+00

Calculated By: BRT Date: 7/9/2019
Checked By: JPS Date: 7/11/2019

Boring B-021-1-13

H =	39.3	ft	Total wall height from profile grade to top of leveling pad
B =	27.5	ft	Wall width considered in analysis, equal to 70% of the wall height
D_w =	0.0	ft	Depth below bottom of wall
q =	1,551	psf	Bearing pressure at bottom of wall (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-a	G	0.0	2.0	2.0	1.0	125	250	125	63	4,063					23	46	153	0.04	1.000	1,551	1,613	0.018	0.221	0.500	775	838	0.015	0.177
	A-1-a	G	2.0	4.5	2.5	3.3	125	563	406	203	4,203					23	41	133	0.12	0.995	1,543	1,746	0.018	0.211	0.500	775	978	0.013	0.154
2	A-1-b	G	4.5	7.0	2.5	5.8	125	875	719	360	4,360					23	36	118	0.21	0.974	1,511	1,871	0.015	0.182	0.498	773	1,133	0.011	0.127
	A-1-b	G	7.0	9.5	2.5	8.3	125	1,188	1,031	516	4,516					23	33	109	0.30	0.937	1,453	1,969	0.013	0.160	0.495	767	1,284	0.009	0.109
	A-1-b	G	9.5	12.0	2.5	10.8	125	1,500	1,344	673	4,673					23	31	103	0.39	0.886	1,375	2,048	0.012	0.141	0.489	759	1,432	0.008	0.096
3	A-1-b	G	12.0	17.0	5.0	14.5	135	2,175	1,838	933	4,933					65	82	339	0.53	0.801	1,242	2,175	0.005	0.065	0.477	740	1,672	0.004	0.045
	A-1-b	G	17.0	22.0	5.0	19.5	135	2,850	2,513	1,296	5,296					65	75	294	0.71	0.691	1,071	2,367	0.004	0.053	0.454	704	2,000	0.003	0.038
	A-1-b	G	22.0	30.0	8.0	26.0	135	3,930	3,390	1,768	5,768					65	68	256	0.95	0.573	888	2,656	0.006	0.066	0.418	648	2,416	0.004	0.051
	A-1-b	G	30.0	38.5	8.5	34.3	135	5,078	4,504	2,367	6,367					65	61	222	1.25	0.463	718	3,085	0.004	0.053	0.371	575	2,942	0.004	0.043
4	A-1-b	G	38.5	43.5	5.0	41.0	125	5,703	5,390	2,832	6,832					16	14	63	1.49	0.398	617	3,449	0.007	0.081	0.335	520	3,352	0.006	0.070
5	A-1-b	G	43.5	46.3	2.8	44.9	135	6,081	5,892	3,090	7,090					120	103	490	1.63	0.367	570	3,660	0.000	0.005	0.317	491	3,581	0.000	0.004
																						Total Settlement:		1.239 in			Total Settlement:		0.914 in

1. $\sigma_p' = \sigma_{vo}' + \sigma_m$; Estimate σ_m of 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. $C_r = 0.10(Cc)$ for 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-13-072 - FRA-70-13.10 - Retaining Wall E7
MSE Wall with Cellular Concrete Backfill Settlement - Sta. 702+50 to 703+00

Calculated By: BRT Date: 7/9/2019
Checked By: JPS Date: 7/11/2019

Boring B-021-5-14

H =	34.2	ft	Total wall height from profile grade to top of leveling pad
B =	23.9	ft	Wall width considered in analysis, equal to 70% of the wall height
D_w =	0.0	ft	Depth below bottom of wall
q =	1,398	psf	Bearing pressure at bottom of wall (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	2.0	2.0	1.0	125	250	125	63	2,063					33	66	246	0.04	1.000	1,398	1,460	0.011	0.133	0.500	699	762	0.009	0.106
	A-1-b	G	2.0	4.0	2.0	3.0	125	500	375	188	2,188					33	59	211	0.13	0.994	1,389	1,577	0.009	0.105	0.500	698	886	0.006	0.077
2	A-1-a	G	4.0	8.5	4.5	6.3	135	1,108	804	414	2,414					98	150	942	0.26	0.955	1,335	1,748	0.003	0.036	0.496	694	1,108	0.002	0.025
	A-1-a	G	8.5	13.0	4.5	10.8	135	1,715	1,411	740	2,740					47	63	229	0.45	0.850	1,189	1,929	0.008	0.098	0.485	677	1,418	0.006	0.067
3	A-4a	C	13.0	18.0	5.0	15.5	130	2,365	2,040	1,073	3,073	22	0.108	0.011	0.444				0.65	0.726	1,015	2,088	0.011	0.130	0.462	646	1,719	0.008	0.092
4	A-3a	G	18.0	24.0	6.0	21.0	135	3,175	2,770	1,460	3,460					102	113	426	0.88	0.603	843	2,303	0.003	0.033	0.428	599	2,058	0.002	0.025
	A-3a	G	24.0	30.9	6.9	27.5	135	4,107	3,641	1,928	3,928					102	103	371	1.15	0.494	691	2,619	0.002	0.030	0.386	539	2,467	0.002	0.024
																						Total Settlement:		0.566 in			Total Settlement:		0.415 in

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

W-13-072 - FRA-70-13.10 - Retaining Wall E7

MSE Wall with Cellular Concrete Backfill Settlement - Sta. 702+50 to 703+00

Calculated By: BRT Date: 7/9/2019

Checked By: JPS Date: 7/11/2019

Boring B-021-5-14

H =	34.2	ft	Total wall height from profile grade to top of leveling pad
B =	23.9	ft	Wall width considered in analysis, equal to 70% of the wall height
D_w =	0.0	ft	Depth below bottom of wall
q =	1,398	psf	Bearing pressure at bottom of wall (see summary sheet)

	A-4a				
c_v =	1,000			ft^2/yr	Coefficient of consolidation
t =	5			days	Time following completion of construction
H_{dr} =	2.5			ft	Length of longest drainage path considered
T_v =	2.192				Time factor
U =	100			%	Degree of consolidation

$(S_c)_t$ = 0.415 in Settlement complete at 100% 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 100% of Primary Consolidation: $(S_c)_t$ [11] (in)	Layer Settlement (in)
1	A-1-b	G	0.0	2.0	2.0	1.0	125	250	125	63	2,063					33	66	246	0.04	0.500	699	762	0.009	0.106	0.183	0.106	0.183
	A-1-b	G	2.0	4.0	2.0	3.0	125	500	375	188	2,188					33	59	211	0.13	0.500	698	886	0.006	0.077		0.077	
2	A-1-a	G	4.0	8.5	4.5	6.3	135	1,108	804	414	2,414					98	150	942	0.26	0.496	694	1,108	0.002	0.025	0.091	0.025	0.091
	A-1-a	G	8.5	13.0	4.5	10.8	135	1,715	1,411	740	2,740					47	63	229	0.45	0.485	677	1,418	0.006	0.067		0.067	
3	A-4a	C	13.0	18.0	5.0	15.5	130	2,365	2,040	1,073	3,073	22	0.108	0.011	0.444				0.65	0.462	646	1,719	0.008	0.092	0.092	0.092	0.092
4	A-3a	G	18.0	24.0	6.0	21.0	135	3,175	2,770	1,460	3,460					102	113	426	0.88	0.428	599	2,058	0.002	0.025	0.049	0.025	0.049
	A-3a	G	24.0	30.9	6.9	27.5	135	4,107	3,641	1,928	3,928					102	103	371	1.15	0.386	539	2,467	0.002	0.024		0.024	

Settlement Remaining After Hold Period: 0.000 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. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ 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 =	23.9	ft	
L =	50	ft	
c =	0	psf	
γ =	125	pcf	
D_f =	4.0	ft	
φ =	39	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$$ = 75.85 ksf

$N_{cm} = N_c s_c i_c$ = 94.62

$N_{qm} = N_q s_q d_q i_q$ = 80.49

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 74.61

N_c =	67.87	s_c =	1+(23.9 ft/50 ft)(55.96/67.87) =	1.394	i_c =	1.000	d_q =	1+2tan(39°)[1-sin(39°)]²tan⁻¹(4 ft/23.9 ft) =	1.037
N_q =	55.96	s_q =	1+(23.9 ft/50 ft)tan(39°) =	1.387	i_q =	1.000	C_{wq} =	0.0 ft < 4.0 ft =	0.500
N_γ =	92.25	s_γ =	1-0.4(23.9 ft/50 ft) =	0.809	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(23.9 ft) + 4 ft =	0.500

$q_R = q_n \cdot \phi_b$ = 37.92 ksf

φ_b = 0.5