FRA-71-14.36 PHASE 6R
RETAINING WALLS E5
PID NO. 105588
FRANKLIN COUNTY, OHIO

STRUCTURE FOUNDATION EXPLORATION REPORT (REV. 1)

Prepared For:
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Columbus, OH 43229-1547

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Rii Project No. W-13-072

July 2019

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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 Walls E5
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 E5 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 Trenner

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

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

Enclosure: Structure Foundation Exploration Report (Rev. 1)

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APPENDICIES

EXECUTIVE SUMMARY

Resource International, Inc. (Rii) has completed a structure foundation exploration for the design and construction of the proposed Retaining Wall E5. Based on proposed plan information provided by Rii and ms consultants, Retaining Wall E5 will support the proposed Ramp D7 from just west of the intersection with Ramp D6 to the east end of the proposed FRA-70-1373B bridge structure carrying Ramp D7 over Short Street. The proposed Ramp D7 will be a two-lane ramp that will carry traffic from Mound Street to I-70 westbound. The proposed structure will consist of two independent retaining walls that will be constructed parallel to each other and will be connected to the forward abutment of the proposed FRA-70-1373B structure at the west end of the wall alignments. Both wall sections connect to the forward abutment of the FRA-70-1373B structure at Sta. 7010+19 (BL Ramp D7) and extend east along the north side of Mound Street. The northern and southern wall sections terminate at Sta. 7012+40 and 7012+35 (BL Ramp D7), respectively. It is understood that a mechanically stabilized earth (MSE) wall type is being considered as the preferred wall type for both sections of Retaining Wall E5. The wall heights along the northern wall section will range from 7.0 feet at Sta. 501+18 (BL Wall E5) to 19.3 feet at Sta. 503+40 (BL Wall E5), and wall heights along the southern wall section will range from 6.6 feet at Sta. 506+12 (BL Wall E5) to 18.5 feet at Sta. 503+96 (BL Wall E5). The total wall length for the northern and southern wall sections is approximately 222 and 216 lineal feet, respectively.

Exploration and Findings

Between March 20 and December 19, 2014, three (3) structure borings, designated as B-020-8-13, B-021-3-13 and B-023-2-13, were drilled to completion depths ranging from 25.0 to 102.0 feet below the existing ground surface at the locations shown on the boring plan provided in Appendix I of the full report.

Boring B-020-8-13 was drilled through the existing sidewalk that runs along the south side of Mound Street and encountered 8.0 inches of concrete overlying 4.0 inches of aggregate base at the ground surface. Boring B-021-3-13 was drilled along the south side of Mound Street and encountered 3.0 inches of asphalt overlying 8.0 inches of concrete at the ground surface. Boring B-023-2-13 was drilled in grass area along the south side of Mound Street, between the entrance ramp to I-70 westbound and the AEP power substation, and encountered 11.0 inches of topsoil at the ground surface.

Beneath the surface materials in all three borings, material identified as existing fill was encountered extending to depths ranging from 3.0 to 5.5 feet below the existing ground surface. The fill material was generally described as brown and dark brown gravel with sand, gravel with sand and silt and silt and clay (ODOT A-1-b, A-2-4, A-6a). Potential buried construction debris was encountered at a depth of 1.0 feet in boring B-023-2-13, which prevented the possibility of obtaining the first split spoon sample at this depth. Additionally, a petroleum odor was noted in sample SS-1 at a depth of 3.5 feet in boring B-023-2-13.

Underlying the surficial materials and existing fill, natural granular soils were encountered with intermittent seams of cohesive soil. The granular soils were generally described as brown and gray gravel, gravel with sand, gravel with sand and silt and coarse and fine sand (ODOT A-1-a, A-1-b, A-2-4, A-3a). The cohesive soils were generally described as gray and brown sandy silt, silt and clay, silty clay and clay (ODOT A-4a, A-6a, A-6b, A-7-6). Cobbles and boulders were encountered above the bedrock in boring B-020-8-13 starting at an elevation of 689 feet msl. Due to the significant presence of large boulders in boring B-020-8-13 at elevation 682 feet msl, mud rotary drilling techniques with a tricone bit was utilized to advance the boring to bedrock.

Top of bedrock in boring B-020-8-13 was encountered at a depth of 65.0 feet below the existing ground surface, which corresponds to an elevation of 656.0 feet msl. The upper 27.0 feet of the bedrock encountered consists of shale and mudstone overlying limestone bedrock at an elevation of 629.0 feet msl.

Analyses and Recommendations

Design details of the proposed retaining wall were provided by the Rii design team and ms consultants. It is understood that Retaining Wall E5 is proposed to be a MSE wall type, which will support the proposed Ramp D7 from just west of the intersection with Ramp D6 to the east end of the proposed FRA-70-1373B bridge structure over Short Street. This structure will consist of two independent retaining walls that will be constructed parallel to each other and will be connected to the forward abutment of the proposed FRA-70-1373B structure at the west end of the wall alignments. Based on the proposed plan and profile information, wall heights along the alignment of the proposed structure are anticipated to range from 6.6 feet to a maximum height of 19.3 feet where the walls will connect to the forward abutment of the proposed FRA-70-1373B structure.

MSE Wall Recommendations

Based on the proposed plan and profile information, the proposed retaining wall will have a maximum height of approximately 18.8 feet, as measured from the top of the leveling pad to the top of the coping. The bearing materials along the proposed alignment of Retaining Wall E5 are anticipated to consist of dense gravel and gravel with sand (ODOT A-1-a, A-1-b). As noted in Section 5.1 of the full report, existing fill consisting of loose gravel with sand and silt (ODOT A-2-6) was encountered at the proposed bearing elevation in boring B-020-8-13, which extended to a depth of 2.0 feet below the proposed bottom of wall elevation. Given the shallow depth of this existing fill material, it is recommended that this material, where encountered, be completely over excavated to expose the underlying competent granular soils and replaced ODOT Item 203 granular embankment. MSE wall foundations bearing on competent natural soils or engineered fill, placed and compacted in accordance with ODOT Item 203, may be proportioned for a nominal bearing resistance as indicated in the following table. A geotechnical resistance factor of φ_b=0.65 was considered in calculating the factored bearing resistance at the strength limit state.

Retaining Wall E5 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]	
501+18	506+12	18.8	Level	13.2 (0.70H ≥ 8.0)	47.8	31.1	4.78

1. *Stationing referenced to the baseline of Retaining Wall E5.*
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 0.82 inches at the center of the reinforced soil mass and 0.69 inches at the facing of the wall are anticipated along the alignment of Retaining Wall E5. Based on the results of the analysis, 90 percent of the total settlement at the facing of the wall is anticipated to occur within 30 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 maximum height of the MSE wall (measured from the top of the leveling pad to the proposed profile grade of the roadway). All of the external and global stability calculations indicate that adequate resistance is available for support of the MSE wall.

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 E5, as shown on the vicinity map and boring plan presented in Appendix I. Based on proposed plan information provided by Rii and ms consultants, Retaining Wall E5 will support the proposed Ramp D7 from just west of the intersection with Ramp D6 to the east end of the proposed FRA-70-1373B bridge structure carrying Ramp D7 over Short Street. The proposed Ramp D7 will be a two-lane ramp that will carry traffic from Mound Street to I-70 westbound. The proposed structure will consist of two independent retaining walls that will be constructed parallel to each other and will be connected to the forward abutment of the proposed FRA-70-1373B structure at the west end of the wall alignments. Both wall sections connect to the forward abutment of the FRA-70-1373B structure at Sta. 7010+19 (BL Ramp D7) and extend east along the north side of Mound Street. The northern and southern wall sections terminate at Sta. 7012+40 and 7012+35 (BL Ramp D7), respectively. It is understood that a mechanically stabilized earth (MSE) wall type is being considered as the preferred wall type for both sections of Retaining Wall E5. The wall heights along the northern wall section will range from 7.0 feet at Sta. 501+18 (BL Wall E5) to 19.3 feet at Sta. 503+40 (BL Wall E5), and wall heights along the southern wall section will range from 6.6 feet at Sta. 506+12 (BL Wall E5) to 18.5 feet at Sta. 503+96 (BL Wall E5). The total wall length for the northern and southern wall sections is approximately 222 and 216 lineal feet, respectively.

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 west of the Scioto River 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 limey dolomite. Both of these members contain chert nodules. 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.

2.2 Existing Conditions

The proposed Retaining Wall E5 structure will be situated along the south side Mound Street, between Civic Center Drive and 2nd Street. Mound Street in the vicinity of the proposed structure is currently a three-lane, asphalt paved roadway that is aligned east-to-west, with concrete sidewalks that run along both sides of the roadway. There is an existing electrical substation located along the south side of Mound Street, between the I-70 westbound entrance ramp and Short Street, which is owned and operated by American Electric Power (AEP). The terrain along the Mound Street roadway and the surrounding area is relatively flat-lying, and the existing Mound Street entrance ramp and I-70 roadway are elevated above the surrounding terrain on engineered embankments. Based on utility plans provided by ms consultants, there are many underground utilities within the Mound Street roadway and also beneath the surrounding sidewalks.

3.0 EXPLORATION

Between March 20 and December 19, 2014, three (3) structure borings, designated as B-020-8-13, B-021-3-13 and B-023-2-13, 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 25.0 to 102.0 feet below the existing ground surface within the existing roadway, sidewalk or grass along the south side of Mound Street. Boring B-020-8-13 was performed following completion of Subsurface Utility Engineering (SUE) Level A locating, in order to identify the physical location of existing electrical duct banks in the vicinity of the boring location, which was required by AEP prior to performing the boring.

Table 1. Test Boring Summary

Boring Number	Station [1]	Offset [1]	Latitude	Longitude	Ground Elevation (feet msl)	Boring Depth (feet)
B-020-8-13	7010+15.89	30.7' Rt.	39.954179475	-83.003899896	721.0	102.0
B-021-3-13	7011+57.17	20.9' Rt.	39.954262476	-83.003407384	727.0	34.4
B-023-2-13	7012+73.67	45.0' Rt.	39.954243564	-83.002983734	733.1	25.0

1. Station and offset reference to the proposed baseline of Ramp D7.

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-mounted rotary drilling machine, utilizing a 3.25 or 4.25-inch inside diameter, hollow-stem auger to advance the holes. Standard penetration test (SPT) and split spoon were performed in the borings at 2.5-foot intervals to a depth of 25.0 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 55 drill rig was calibrated on October 20, 2014, and has a drill rod energy ratio of 92.0 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	39
Plastic and Liquid Limits	AASHTO T89, T90	15
Gradation – Sieve/Hydrometer	AASHTO T88	15
Unconfined Compressive Strength of Intact Rock	ASTM D7012	3

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-020-8-13 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 HQ-sized double-tube diamond bit core barrel (utilizing wire line equipment) was used to core the bedrock in boring B-020-8-13. Coring produced 2.5-inch diameter cores, from which the type of rock and geological characteristics were determined.

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

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

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

Boring B-020-8-13 was drilled through the existing sidewalk that runs along the south side of Mound Street and encountered 8.0 inches of concrete overlying 4.0 inches of aggregate base at the ground surface. Boring B-021-3-13 was drilled along the south side of Mound Street and encountered 3.0 inches of asphalt overlying 8.0 inches of concrete at the ground surface. Boring B-023-2-13 was drilled in grass area along the south side of Mound Street, between the entrance ramp to I-70 westbound and the AEP power substation, and encountered 11.0 inches of topsoil at the ground surface.

4.2 Subsurface Soils

Beneath the surface materials in all three borings, material identified as existing fill was encountered extending to depths ranging from 3.0 to 5.5 feet below the existing ground surface. The fill material was generally described as brown and dark brown gravel with sand, gravel with sand and silt and silt and clay (ODOT A-1-b, A-2-4, A-6a). Potential buried construction debris was encountered at a depth of 1.0 feet in boring B-023-2-13, which prevented the possibility of obtaining the first split spoon sample at this depth. Additionally, a petroleum odor was noted in sample SS-1 at a depth of 3.5 feet in boring B-023-2-13.

Underlying the surficial materials and existing fill, natural granular soils were encountered with intermittent seams of cohesive soil. The granular soils were generally described as brown and gray gravel, gravel with sand, gravel with sand and silt and coarse and fine sand (ODOT A-1-a, A-1-b, A-2-4, A-3a). The cohesive soils were generally described as gray and brown sandy silt, silt and clay, silty clay and clay (ODOT A-4a, A-6a, A-6b, A-7-6). Cobbles and boulders were encountered above the bedrock in boring B-020-8-13 starting at an elevation of 689 feet msl. Due to the significant presence of large boulders in boring B-020-8-13 at elevation 682 feet msl, mud rotary drilling techniques with a tricone bit was utilized to advance the boring to bedrock.

The relative density of granular soils is primarily derived from SPT blow counts (N_{60}). Based on the SPT blow counts obtained, the granular soil encountered ranged from loose ($5 \leq N_{60} \leq 10$ blows per foot [bpf]) to very dense ($N_{60} > 50$ bpf). Overall blow counts recorded from the SPT sampling ranged from 6 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 medium stiff ($0.5 < HP \leq 1.0$ tsf) to hard ($HP > 4.0$ tsf). The unconfined compressive strength of the cohesive soil samples tested, obtained from the hand penetrometer, ranged from 0.75 to over 4.5 tsf (limit of instrument).

Natural moisture contents of the soil samples tested ranged from 1 to 28 percent. The natural moisture content of the cohesive soil samples tested for plasticity index ranged from 8 percent below to 10 percent above their corresponding plastic limits. In general, the soil exhibited natural moisture contents considered to be significantly below to significantly above optimum moisture levels.

4.3 Bedrock

Bedrock was encountered in boring B-020-8-13 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-020-8-13	721.0	65.0	656.0	65.0	656.0

Top of bedrock in boring B-020-8-13 was encountered at a depth of 65.0 feet below the existing ground surface, which corresponds to an elevation of 656.0 feet msl. The upper 27.0 feet of the bedrock encountered consists of shale and mudstone overlying limestone bedrock at an elevation of 629.0 feet msl. The mudstone is described as gray, highly weathered, very weak to weak, medium bedded, calcareous, fissile, argillaceous and moderately fractured with tight to open, very rough apertures. The shale is described as gray, slightly weathered to unweathered, very weak to strong, laminated to very thick bedded, argillaceous, arenaceous, fissile, and slightly fractured to fractured with tight to open, very rough apertures. The limestone is described as gray, unweathered, very strong, very thick bedded, arenaceous, siliceous, pyritic, ferriferous and slightly fractured with narrow, slightly rough apertures.

The percent recovery, RQD values and unconfined compressive strengths of the bedrock core runs in boring B-020-5-13 are summarized in Table 4.

Table 4. Rock Core Summary

Boring	Core No.	Elevation (feet msl)	Recovery (%)	RQD (%)	Unconfined Compressive Strength
B-020-8-13	RC-1	656.0 to 654.0	52	33	N/A
	RC-2	654.0 to 649.0	97	72	q_u @ 71.2’ = 275 psi
	RC-3	649.0 to 644.0	100	83	N/A
	RC-4	644.0 to 639.0	90	68	N/A
	RC-5	639.0 to 634.0	95	78	q_u @ 85.9’ = 318 psi
	RC-6	634.0 to 629.0	52	8	N/A
	RC-7	629.0 to 624.0	100	100	N/A
	RC-8	624.0 to 619.0	100	100	q_u @ 97.1’ = 4,737 psi

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 shale and mudstone bedrock, according to the RQD values, ranged from poor (25% < RQD ≤ 50%) 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-020-8-13	721.0	18.5	702.5	N/A [1]	N/A
B-021-3-13	727.0	21.0	706.0	N/A [1]	N/A
B-023-2-13	733.1	13.5	719.6	N/A [1]	N/A

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

Groundwater was encountered initially during drilling in all three borings at depths ranging from 13.5 to 21.0 feet below the existing ground surface, which corresponds to elevations ranging from 702.5 to 719.6 feet msl. The groundwater levels at the completion of drilling could not be measured due to the addition of water or mud to counteract heaving sands. 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 Retaining Wall E5 is proposed to be a MSE wall type, which will support the proposed Ramp D7 from just west of the intersection with Ramp D6 to the east end of the proposed FRA-70-1373B bridge structure over Short Street. This structure will consist of two independent retaining walls that will be constructed parallel to each other and will be connected to the forward abutment of the proposed FRA-70-1373B structure at the west end of the wall alignments. Based on the proposed plan and profile information, wall heights along the alignment of the proposed structure are anticipated to range from 6.6 feet to a maximum height of 19.3 feet where the walls will connect to the forward abutment of the proposed FRA-70-1373B structure.

5.1 MSE Wall Recommendations

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. The width of the MSE wall foundation (B) is defined by the length of the reinforced soil mass. Per 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 retaining wall will have a maximum height of approximately 18.8 feet, as measured from the top of the leveling pad to the top of coping. For the analysis, the foundation width was set at 70 percent of the maximum wall height and the foundation width was increased, if required, until external and global stability requirements were satisfied.

Per Section 840.06.D of ODOT SS 840, the foundation subgrade should be inspected to verify that the subsurface conditions are the same as those anticipated in this report. The anticipated soils at the proposed bearing elevation along the wall alignment consists of dense gravel and gravel with sand (ODOT A-1-a, A-1-b). This material is considered suitable for support of the proposed wall in its current condition. However, existing fill consisting of loose gravel with sand and silt (ODOT A-2-4) was encountered at the proposed bearing elevation in boring B-020-8-13, which extended to an elevation of 715.5 feet msl. Given the shallow depth of this existing fill material, it is recommended that this material, where encountered, be completely over excavated to expose the underlying competent granular soils and replaced ODOT Item 203 granular embankment. Over excavation depths on the order of 2.0 feet are anticipated. Additionally, it should be noted that borings were not obtained within the footprint of the northern wall section due to the dense presence of utilities within the roadway. Existing fill depths may extend deeper in this area. The actual limits and depth of over excavation will need to be determined during the construction of the wall based on observation of the subgrade condition by a qualified soil technician or geotechnical engineer.

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.

5.1.1 Strength Parameters Utilized in External and Global Stability Analyses

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

Table 6. Shear Strength Parameters Utilized in Stability Analyses

Material Type	γ (pcf)	φ' (1) (°)	c' (2) (psf)	S_u (3) (psf)
MSE Wall Backfill (Select granular fill)	120	34	0	N/A
Item 203 Embankment Fill (Retained soil)	120	30	0	2,000
Medium Dense to Very Dense Granular Soils (ODOT A-1-a, A-1-b, A-3a)	125 to 135	39 to 43	0	N/A
Hard Clay (ODOT A-7-6)	130	27	50	8,000

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

Shear strength parameters for the reinforced soil backfill 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 bearing materials along the proposed alignment of Retaining Wall E5 are anticipated to consist of dense gravel and gravel with sand (ODOT A-1-a, A-1-b). As noted in Section 5.1, existing fill consisting of loose gravel with sand and silt (ODOT A-2-4) was encountered at the proposed bearing elevation in boring B-020-8-13, which extended to a depth of 2.0 feet below the proposed bottom of wall elevation. Given the shallow depth of this existing fill material, it is recommended that this material, where encountered, be completely over excavated to expose the underlying competent granular soils and replaced ODOT Item 203 granular embankment. MSE wall foundations bearing on competent natural soils or engineered fill, placed and compacted in accordance with ODOT Item 203, 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 E5 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]	
501+18	506+12	18.8	Level	13.2 (0.70H ≥ 8.0)	47.8	31.1	4.78

1. *Stationing referenced to the baseline of Retaining Wall E5.*
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 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 walls 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 Soil (ODOT A-1-a, A-1-b, A-3a)	125 to 135	N/A	N/A	N/A	N/A	N/A	12 to 120	59 to 945
Hard Sandy Silt (ODOT A-4a)	120 to 130	24 to 25	0.126 to 0.135	0.013 to 0.014	0.460 to 0.467	1,000	N/A	N/A
Hard Silt and Clay (ODOT A-6a)	130	25	0.135	0.014	0.467	600	N/A	N/A
Medium Stiff to Stiff Silty Clay (ODOT A-6b)	120	33	0.207	0.021	0.530	300	N/A	N/A
Hard Clay (ODOT A-7-6)	120	43	0.297	0.030	0.608	150	N/A	N/A

1. *Per Table 6-9, Section 6.14.1 of FHWA GEC 5.*
2. *Estimated at 10% of C_c 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 0.82 inches at the center of the reinforced soil mass and 0.69 inches at the facing of the wall are anticipated along the alignment of Retaining Wall E5. Based on the results of the analysis, 90 percent of the total settlement at the facing of the wall is anticipated to occur within 30 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 E5 MSE Wall Settlement Values

From Station [1]	To Station [1]	Service Limit Equivalent Bearing Pressure [2] (ksf)	Total Settlement Values (inches)		Time for 90% Consolidation (Days)
			Center of Wall Mass	Facing of Wall	
501+18	506+12	1.14 to 3.30	0.57 to 0.82	0.47 to 0.69	1 to 30

1. *Station referenced to the baseline of Retaining Wall E5.*
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. Given the dense presence of existing utilities within the footprint of the wall and Ramp D7, preloading the site or ground improvement techniques will likely not be viable options for settlement control. Therefore, if settlement or additional loading on the existing utilities will be a concern, then consideration should be given to using lightweight fill such as cellular concrete. 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 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 2014 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 soil and the reinforced soil backfill, a coefficient of sliding friction of 0.67 was utilized for design.

A geotechnical resistance factor of $\varphi_{\tau}=1.0$ 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 $\varphi=0.75$ is greater than 1.0. Therefore, global stability is satisfied when a minimum factor of safety of 1.3 is obtained. For MSE walls designed with a minimum strap length listed in Table 7, the resulting factor of safety under drained conditions (long-term stability) was greater than or equal to 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 maximum height of the MSE wall (measured from the top of the leveling pad to the proposed profile grade of the roadway). 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 walls are provided in Appendix V.

5.2 Lateral Earth Pressure

For the soil types encountered in the borings, the "in-situ" unit weight (γ), cohesion (c), effective angle of friction (φ'), and lateral earth pressure coefficients for at-rest conditions (k_o), active conditions (k_a), and passive conditions (k_p) have been estimated and are provided in Table 10 and Table 11.

Table 10. Estimated Undrained (Short-term) Soil Parameters for Design

Soil Type	γ (pcf) [1]	c (psf)	φ	k_a	k_o	k_p
Soft to Stiff Cohesive Soil	115	1,500	0°	N/A	N/A	N/A
Very Stiff to Hard Cohesive Soil	125	3,000	0°	N/A	N/A	N/A
Loose Granular Soil	120	0	28°	0.32	0.53	5.07
Medium Dense Granular Soil	125	0	32°	0.27	0.47	6.82
Dense to Very Dense Granular Soil	130	0	36°	0.23	0.41	9.09
Compacted Cohesive Engineered Fill	120	2,000	0°	N/A	N/A	N/A
Compacted Granular Engineered Fill	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 11. 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
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.

5.3 Construction Considerations

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

5.3.1 Excavation Considerations

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

Table 12. Excavation Back Slopes

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

5.3.2 Groundwater Considerations

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

6.0 LIMITATIONS OF STUDY

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

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

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

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

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

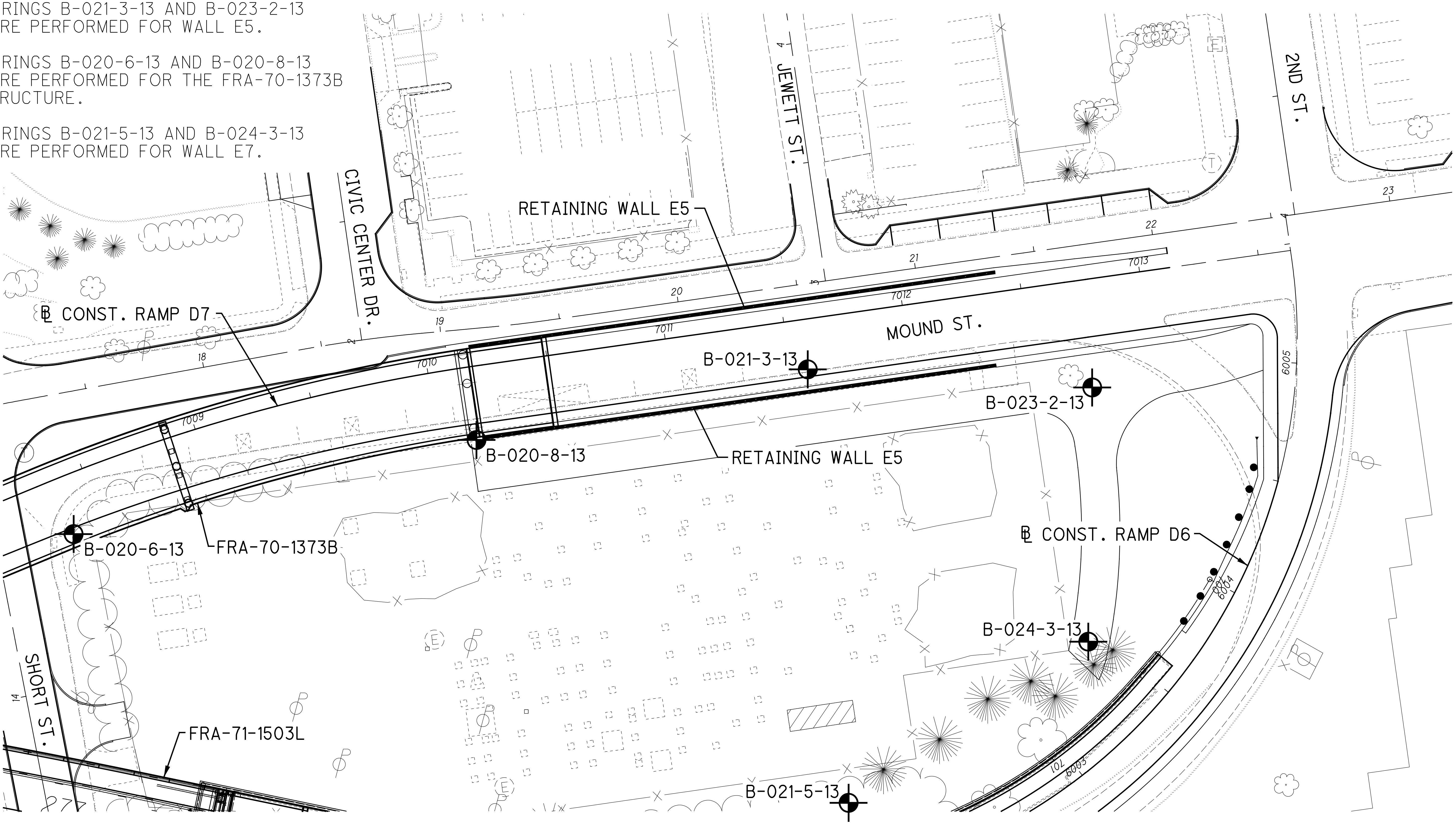
APPENDIX I

VICINITY MAP AND BORING PLAN

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

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

BORINGS B-021-5-13 AND B-024-3-13
WERE PERFORMED FOR WALL E7.



BORING PLAN

FRA-70-13.10 - RETAINING WALL E5

FRANKLIN COUNTY, OHIO

PROJECT NO.
Rii W-13-072

SCALE: 1″=40′
0 20 40

N

DRAWN
RRM

REVIEWED
BRT

DATE
7-17-19

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		LL_O/LL × 100*	% Pass *40	% Pass *200	Liquid Limit (LL)	Plastic Index (PI)	Group Index Max.	REMARKS
		AASHTO	OHIO							
○	Gravel and/or Stone Fragments	A-1-a			30 Max.	15 Max.		6 Max.	0	Min. of 50% combined gravel, cobble and boulder sizes
◌	Gravel and/or Stone Fragments with Sand	A-1-b			50 Max.	25 Max.		6 Max.	0	
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-020-8-13, B-021-3-13 and B-023-2-13

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:

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

S = Sulfate content (parts per million)

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

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

SS = Split spoon sample

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

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

TR = Top of rock

W = Initial water level measured during drilling

▼ = Water level measured at completion of drilling

Classification Test Data

Gradation (as defined on Description of Soil Terms):

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

Atterberg Limits:

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

WC = Water content (%)

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-13.10 - PHASE 6A
TYPE: STRUCTURE
PID: 89464 BR ID: FRA-70-1373B
START: 12/15/14 END: 12/19/14

DRILLING FIRM / OPERATOR: RII / J.K.
SAMPLING FIRM / LOGGER: RII / N.A.
DRILLING METHOD: 4.25" HSA / HQ
SAMPLING METHOD: SPT / RC

DRILL RIG: CME-55 (SN 386345)
HAMMER: AUTOMATIC
CALIBRATION DATE: 10/20/14
ENERGY RATIO (%): 92

STATION / OFFSET: 7010+15.89 / 30.7' RT
ALIGNMENT: BL RAMP D7
ELEVATION: 721.0 (MSL) EOB: 102.0 ft.
LAT / LONG: 39.954179, -83.003900

EXPLORATION ID
B-020-8-13

PAGE
1 OF 4

MATERIAL DESCRIPTION AND NOTES	ELEV. 721.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
0.7' - CONCRETE (8.0")	720.3																	
0.3' - AGGREGATE BASE (4.0")	720.0																	
FILL: LOOSE, BROWN **GRAVEL WITH SAND AND SILT**, LITTLE CLAY, DAMP TO MOIST.		1 2 3	2 3 2	8	56	SS-1	-	-	-	-	-	-	-	-	-	8	A-2-4 (V)	
-ROCK FRAGMENTS PRESENT THROUGHOUT	715.5	4 5	2 2 3	8	56	SS-2	-	40	19	12	13	16	28	18	10	15	A-2-4 (0)	
MEDIUM DENSE, GRAY **GRAVEL**, DRY. -LIMESTONE FRAGMENTS PRESENT IN SS-3	713.0	6 7 8	5 7 8	23	33	SS-3	-	-	-	-	-	-	-	-	-	1	A-1-a (V)	
DENSE TO VERY DENSE, BROWN **GRAVEL**, LITTLE COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, DAMP TO MOIST.		9 10	50/6"	-	0	SS-4	-	-	-	-	-	-	-	-	-	-		
		11 12 13	26 16 13	44	67	SS-5	-	77	12	4	3	4	20	19	1	6	A-1-a (0)	
		14 15	13 24 23	71	33	SS-6	-	-	-	-	-	-	-	-	-	6	A-1-a (V)	
		W 16 17	50/6"	-	100	SS-7	-	-	-	-	-	-	-	-	-	9	A-1-a (V)	
		W 18 19	15 16 16	48	0	SS-8	-	-	-	-	-	-	-	-	-	-		
-INTRODUCED MUD @ 20.0'	700.5	20																
DENSE TO VERY DENSE, BROWN **COARSE AND FINE SAND**, TRACE TO LITTLE CLAY, TRACE SILT, TRACE FINE GRAVEL, WET.		21 22 23	25 26 26	78	100	SS-9	-	1	10	70	9	10	NP	NP	NP	21	A-3a (0)	
		24 25	10 13 18	47	100	SS-10	-	-	-	-	-	-	-	-	-	21	A-3a (V)	
		26 27 28	8 11 15	39	100	SS-11	-	1	14	61	10	14	NP	NP	NP	22	A-3a (0)	
		29	11 16 16	48	100	SS-12	-	-	-	-	-	-	-	-	-	20	A-3a (V)	

PID: 89464	BR ID: FRA-70-1373B	PROJECT: FRA-70-13.10 - PHASE 6A	STATION / OFFSET: 7010+15.89 / 30.7 RT	START: 12/15/14	END: 12/19/14	PG 2 OF 4	B-020-8-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 691.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI			
DENSE TO VERY DENSE, BROWN **COARSE AND FINE SAND**, TRACE TO LITTLE CLAY, TRACE SILT, TRACE FINE GRAVEL, WET. *(same as above)*	689.0	31																
VERY DENSE, GRAY TO BROWN **GRAVEL WITH SAND**, TRACE SILT, TRACE CLAY, MOIST.		32																
		33																
		34	27 20 37	86	100	SS-13	-	36	38	13	6	7	NP	NP	NP	12	A-1-b (0)	
		35																
	684.0	36																
-GRANITE BOULDER ENCOUNTERED @ 37.0'		37																
		38																
-SWITCHED TO MUD ROTARY DRILLING WITH TRICONE BIT @ 39.0'		39	50/1"	-	0	SS-14	-	-	-	-	-	-	-	-	-	-		
		40																
		41																
-COBBLES AND BOULDERS PRESENT THROUGHOUT		42																
		43																
		44	43 50/5"	-	73	SS-15	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
		45																
		46																
		47																
		48																
		49	47 50/1"	-	71	SS-16	-	-	-	-	-	-	-	-	-	7	A-1-b (V)	
		50																
	669.0	51																
HARD, GRAY **CLAY**, SOME SILT, TRACE COARSE TO FINE SAND, DAMP TO MOIST.		52																
		53																
		54	8 30 40	105	100	SS-17	4.5+	-	-	-	-	-	-	-	-	24	A-7-6 (V)	
		55																
		56																
		57																
		58																
		59	40 50/6"	-	100	SS-18	4.5+	0	1	2	32	65	43	21	22	17	A-7-6 (13)	
		60																
		61																

PID: 89464	BR ID: FRA-70-1373B	PROJECT: FRA-70-13.10 - PHASE 6A	STATION / OFFSET: 7010+15.89 / 30.7 RT	START: 12/15/14	END: 12/19/14	PG 3 OF 4	B-020-8-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 658.9	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)
HARD, GRAY **CLAY**, SOME SILT, TRACE COARSE TO FINE SAND, DAMP TO MOIST. *(same as above)*	656.0	63 64	50/1"	-	0	SS-19	-	-	-	-	-	-	-	-	-	-	
MUDSTONE : GRAY, HIGHLY WEATHERED, VERY WEAK TO WEAK, MEDIUM BEDDED, CALCAREOUS, FISSILE, ARGILLACEOUS, MODERATELY FRACTURED, TIGHT TO OPEN APERTURES, VERY ROUGH; RQD 29%, REC 58%.	653.7	TR 65 66 67	33		52	RC-1											CORE
SHALE : GRAY, SLIGHTLY WEATHERED TO UNWEATHERED, VERY WEAK TO STRONG, LAMINATED TO VERY THICK BEDDED, ARENACEOUS, ARGILLACEOUS, FISSILE, SLIGHTLY FRACTURED TO FRACTURED, TIGHT TO OPEN APERTURES, VERY ROUGH; RQD 63%, REC 86%. -QU @ 71.2' = 275 PSI -0.4' LIMESTONE SEAM @ 72.0'		68 69 70 71 72	72		97	RC-2											CORE
		73 74 75 76 77	83		100	RC-3											CORE
-CALCAREOUS FROM 77.0' TO 82.0'		78 79 80 81 82	68		90	RC-4											CORE
-QU @ 85.9' = 318 PSI		83 84 85 86 87	78		95	RC-5											CORE
	629.0	88 89 90 91 92	8		52	RC-6											CORE
LIMESTONE : GRAY, UNWEATHERED, VERY STRONG, VERY THICK BEDDED, ARENACEOUS, FERRIFEROUS, SILICEOUS, PYRITIC, INTACT, NARROW APERTURES, SLIGHTLY ROUGH; RQD 100%, REC 100%.		93 94															

PID: 89464 | BR ID: FRA-70-1373B | PROJECT: FRA-70-13.10 - PHASE 6A | STATION / OFFSET: 7010+15.89 / 30.7 RT | START: 12/15/14 | END: 12/19/14 | PG 4 OF 4 | B-020-8-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 626.7	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
LIMESTONE : GRAY, UNWEATHERED, VERY STRONG, VERY THICK BEDDED, ARENACEOUS, FERRIFEROUS, SILICEOUS, PYRITIC, INTACT, NARROW APERTURES, SLIGHTLY ROUGH; RQD 100%, REC 100%. *(same as above)* -QU @ 97.1' = 4,737 PSI		95 96 97	100		100	RC-7											CORE	
	619.0	98 99 100 101 EOB 102	100		100	RC-8											CORE	

NOTES: SEEPAQGE ENCOUNTERED @ 16.0'; GROUNDWATER INITIALLY ENCOUNTERED @ 18.5'

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

B-020-8-13 – RC-1 and RC-2 – Depth from 65.0 to 72.0 feet

B-020-8-13 – RC-3 – Depth from 72.0 to 77.0 feet

B-020-8-13 – RC-4 and RC-5 – Depth from 77.0 to 87.0 feet

B-020-8-13 – RC-6 and RC-7 – Depth from 87.0 to 97.0 feet

B-020-8-13 – RC-8 – Depth from 97.0 to 102.0 feet

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-13.10 - PHASE 6A
TYPE: STRUCTURE
PID: 89464 BR ID: N/A
START: 3/20/14 END: 3/20/14

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

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

STATION / OFFSET: 7011+57.17 / 20.9' RT
ALIGNMENT: BL RAMP D7
ELEVATION: 727.0 (MSL) EOB: 34.4 ft.
LAT / LONG: 39.954262, -83.003407

EXPLORATION ID
B-021-3-13

PAGE
1 OF 2

MATERIAL DESCRIPTION AND NOTES	ELEV. 727.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)
0.2' - ASPHALT (3.0")	726.8																
0.7' - CONCRETE (8.0")	726.1																
FILL: STIFF, DARK BROWN **SILT AND CLAY**, SOME COARSE TO FINE SAND, LITTLE FINE GRAVEL, DRY. -BRICK AND ROCK FRAGMENTS PRESENT IN SS-1	724.0		12 6 5	14	67	SS-1	1.75	19	15	13	30	23	28	16	12	8	A-6a (4)
MEDIUM DENSE TO DENSE, GRAY TO BROWN **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, DAMP. -PETROLEUM ODOR PRESENT IN SS-2			8 14 15	38	56	SS-2	-	-	-	-	-	-	-	-	-	5	A-1-b (V)
-ROCK FRAGMENTS PRESENT THROUGHOUT			17 17 15	41	44	SS-3	-	-	-	-	-	-	-	-	-	5	A-1-b (V)
			11 12 14	34	100	SS-4	-	-	-	-	-	-	-	-	-	6	A-1-b (V)
	714.0		24 12 10	28	78	SS-5	-	18	46	17	11	8	NP	NP	NP	7	A-1-b (0)
VERY DENSE, BROWN **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, DAMP.			17 50/5"	-	73	SS-6	-	-	-	-	-	-	-	-	-	7	A-1-b (V)
-ROCK FRAGMENTS PRESENT THROUGHOUT	709.0		50/5"	-	100	SS-7	-	-	-	-	-	-	-	-	-	5	A-1-b (V)
MEDIUM DENSE, BROWN **COARSE AND FINE SAND**, SOME CLAY, LITTLE SILT, MOIST.		W	6 6 5	14	0	SS-8	-	-	-	-	-	-	-	-	-	-	
-ROCK FRAGMENTS PRESENT 2S-8A	706.5		36	-	67	2S-8A	-	-	-	-	-	-	-	-	-	12	A-3a (V)
VERY DENSE, BROWN **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, MOIST. -ROCK FRAGMENTS PRESENT SS-9	704.0	W	48 50/0"	-	83	SS-9	-	-	-	-	-	-	-	-	-	8	A-1-b (V)
HARD, GRAY **SILT AND CLAY**, SOME COARSE TO FINE SAND, SOME FINE GRAVEL, DAMP. -ROCK FRAGMENTS PRESENT IN SS-10	700.5		24 40 41	105	56	SS-10	4.5+	23	14	15	27	21	25	14	11	9	A-6a (3)
VERY DENSE, BROWN **COARSE AND FINE SAND**, LITTLE SILT, TRACE CLAY, TRACE FINE GRAVEL, WET.	699.0		29 26 30	73	83	SS-11	4.5+ -	- 1	- 24	- 54	- 14	- 7	- NP	- NP	- NP	11 20	A-6a (V) A-3a (0)
HARD, BROWN TO GRAY **SANDY SILT**, SOME CLAY, LITTLE FINE GRAVEL, DAMP.			16 18 24	54	94	SS-12	4.5+	-	-	-	-	-	-	-	-	11	A-4a (V)

BACK FILL

PID: 89464	BR ID: N/A	PROJECT: FRA-70-13.10 - PHASE 6A	STATION / OFFSET: 7011+57.17 / 20.9 RT	START: 3/20/14	END: 3/20/14	PG 2 OF 2	B-021-3-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 697.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI			
HARD, BROWN TO GRAY **SANDY SILT**, SOME CLAY, LITTLE FINE GRAVEL, DAMP. *(same as above)* -INTRODUCED WATER @ 30.0'		31 32 33	18 24 50	96	94	SS-13	4.5+	12	12	19	30	27	24	14	10	10	A-4a (4)	
-GRANITE FRAGMENTS PRESENT IN SS-14	692.6	EOB 34	19 50/5"	-	45	SS-14	4.5+	-	-	-	-	-	-	-	-	11	A-4a (V)	

NOTES: SEEPAGE ENCOUNTERED @ 18.5'; GROUNDWATER ENCOUNTERED INITIALLY @ 21.0'; CAVE-IN DEPTH @ 25.0'

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

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-13.10 - PHASE 6A
TYPE: STRUCTURE
PID: 89464 BR ID: N/A
START: 3/20/14 END: 3/20/14

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

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

STATION / OFFSET: 7012+73.67 / 45.0' RT
ALIGNMENT: BL RAMP D7
ELEVATION: 733.1 (MSL) EOB: 25.0 ft.
LAT / LONG: 39.954244, -83.002984

EXPLORATION ID
B-023-2-13

PAGE
1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 733.1	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	HOLE SEALED
								GR	CS	FS	SI	CL	LL	PL	PI			
0.9' - TOPSOIL (11.0")	732.2																	
FILL: MEDIUM DENSE, BROWN **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, DAMP. -COULD NOT ADVANCE AUGERS OR ATTEMPT SPLIT SPOON SAMPLE @ 1.0' DUE TO SIGNIFICANT PRESENCE OF CONSTRUCTION DEBRIS. OBTAINED REPRESENTATIVE SAMPLE FROM AUGER CUTTINGS. -PETROLEUM ODOR PRESENT IN SS-1	727.6		8 9 10	25	33	SS-1	-	-	-	-	-	-	-	-	-	6	A-1-b (V)	
VERY STIFF, BROWN **SANDY SILT**, SOME CLAY, TRACE FINE GRAVEL, MOIST. -IRON STAINING PRESENT IN SS-2	725.1		5 5 8	17	56	SS-2	2.50	4	11	11	44	30	25	16	9	17	A-4a (8)	
MEDIUM DENSE, BROWN **GRAVEL**, LITTLE COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST. -ROCK FRAGMENTS PRESENT IN SS-3	723.1		4 8 11	25	6	SS-3	-	-	-	-	-	-	-	-	-	8	A-1-a (V)	
STIFF, BROWN **SILT AND CLAY**, LITTLE TO SOME COARSE TO FINE SAND, MOIST.	722.1		13	-	100	2S-3A	2.00	-	-	-	-	-	-	-	-	20	A-6a (V)	
MEDIUM DENSE, BROWN **GRAVEL WITH SAND AND SILT**, TRACE CLAY, MOIST.	717.6		6 5 5	13	0	SS-4	-	-	-	-	-	-	-	-	-	-		
			6	-	100	2S-4A	-	42	23	9	18	8	21	16	5	10	A-2-4 (0)	
		W	5 5 4	12	0	SS-5	-	-	-	-	-	-	-	-	-	-		
			8	-	0	2S-5A	-	-	-	-	-	-	-	-	-	-		
MEDIUM STIFF TO STIFF, GRAY **SILT AND CLAY**, TRACE FINE GRAVEL, MOIST.	712.6		2 2 3	6	61	SS-6	0.75	1	0	0	44	55	33	17	16	27	A-6b (10)	
-INTORODUCED WATER @ 18.5'			4 7 11	23	89	SS-7	2.00	-	-	-	-	-	-	-	-	28	A-6b (V)	
MEDIUM DENSE TO VERY DENSE, GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST.	708.1		3 9 14	30	100	SS-8	-	-	-	-	-	-	-	-	-	14	A-1-a (V)	
		EOB 25	19 22 24	60	100	SS-9	-	55	21	10	9	5	NP	NP	NP	8	A-1-a (0)	

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 13.5'

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

2014 ODOT BORING LOG-RII NE BRIDGE ID - OH DOT.GDT - 7/12/19 12:59 - U:\GI8\PROJECTS\2013\W-13-072.GPJ

APPENDIX IV

LABORATORY TEST RESULTS

RESOURCE INTERNATIONAL, INC.
Engineering Consultants

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

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

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

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

Project: FRA-70-13.10 - Project 6A
Project No.: W-13-072
Date of Testing: 12/30/2014
Test Performed by: K.R./T.K.

Rock Description: Shale

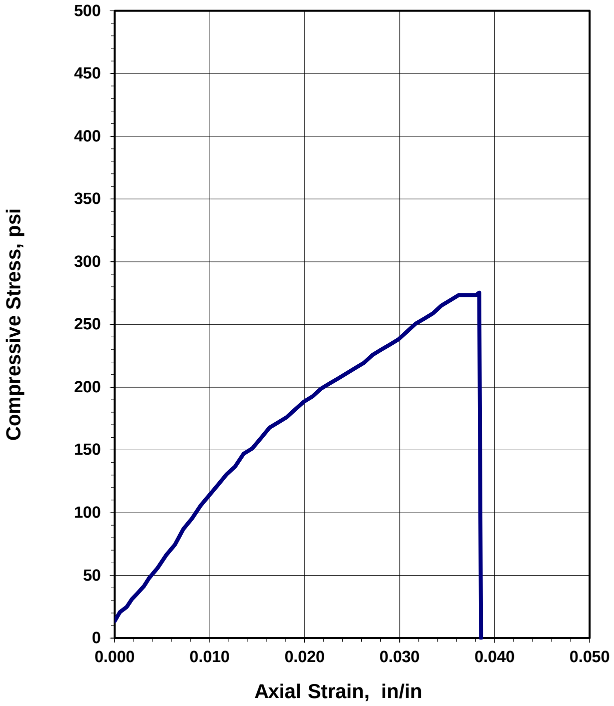
Boring No.: B-20-8
Sample No: RC-2
Depth (ft): 71.2
Moisture condition: As received

Average Length: 5.523 in
Average Diameter: 2.48 in
Length to diameter ratio: 2.227
Cross Sectional Area: 4.828 in^2

Rate of Loading: 2.9 lbs/sec
Testing Time: 454 sec
(Rate 2-15 minutes to failure)

Failure Load: 1,330 lbs
Axial Strain at Failure: 0.0384 in/in
Stress: 275 psi

Unconfined Compression Test



Before Testing

After Failure

REMARKS:

RESOURCE INTERNATIONAL, INC.
Engineering Consultants

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

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

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

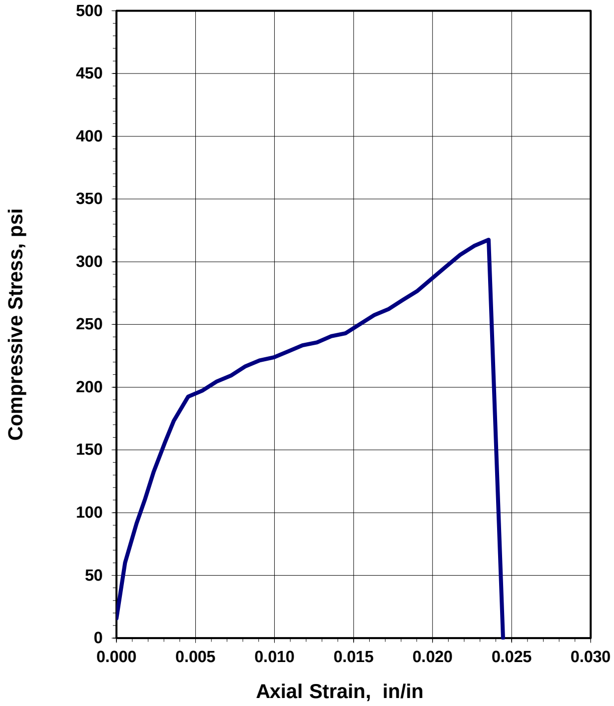
Unconfined Compressive Strength
of Intact Rock Core Specimens (ASTM D 7012-04)

Project: FRA-70-13.10 - Project 6A
Project No.: W-13-072
Date of Testing: 12/30/2014
Test Performed by: K.R./T.K.

Rock Description: Shale

Boring No.:	B-20-8	Average Length:	5.52 in
Sample No:	RC-5	Average Diameter:	2.301 in
Depth (ft):	85.9	Length to diameter ratio:	2.399
Moisture condition:	As received	Cross Sectional Area:	4.156 in^2
Rate of Loading:	4.1 lbs/sec	Failure Load:	1,320 lbs
Testing Time:	322 sec	Axial Strain at Failure:	0.0236 in/in
(Rate 2-15 minutes to failure)		Stress:	318 psi

Unconfined Compression Test



Before Testing

After Failure

REMARKS: ______

RESOURCE INTERNATIONAL, INC.
Engineering Consultants

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

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

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

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

Project: FRA-70-13.10 - Project 6A
Project No.: W-13-072
Date of Testing: 12/30/2014
Test Performed by: K.R./T.K.

Rock Description: Limestone

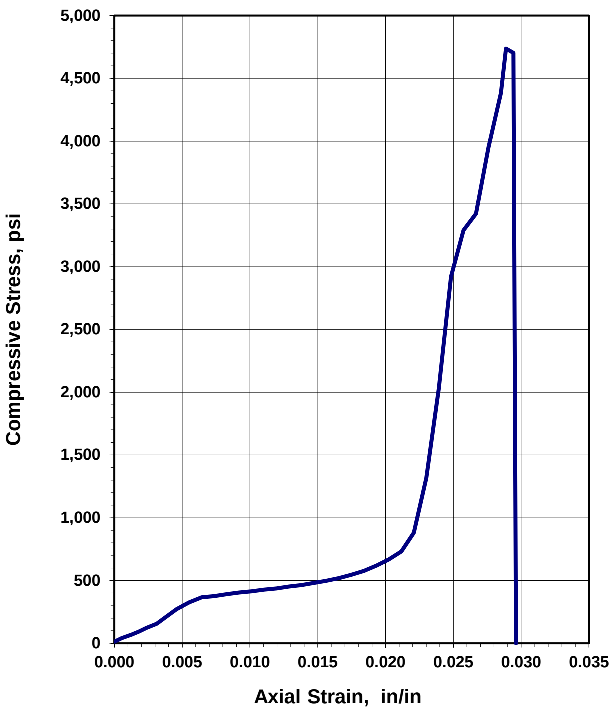
Boring No.: B-20-8
Sample No: RC-8
Depth (ft): 97.1
Moisture condition: As received

Average Length: 5.435 in
Average Diameter: 2.491 in
Length to diameter ratio: 2.182
Cross Sectional Area: 4.871 in^2

Rate of Loading: 65.4 lbs/sec
Testing Time: 353 sec
(Rate 2-15 minutes to failure)

Failure Load: 23,080 lbs
Axial Strain at Failure: 0.0289 in/in
Stress: 4,737 psi

Unconfined Compression Test



Before Testing

After Failure

REMARKS:

APPENDIX V

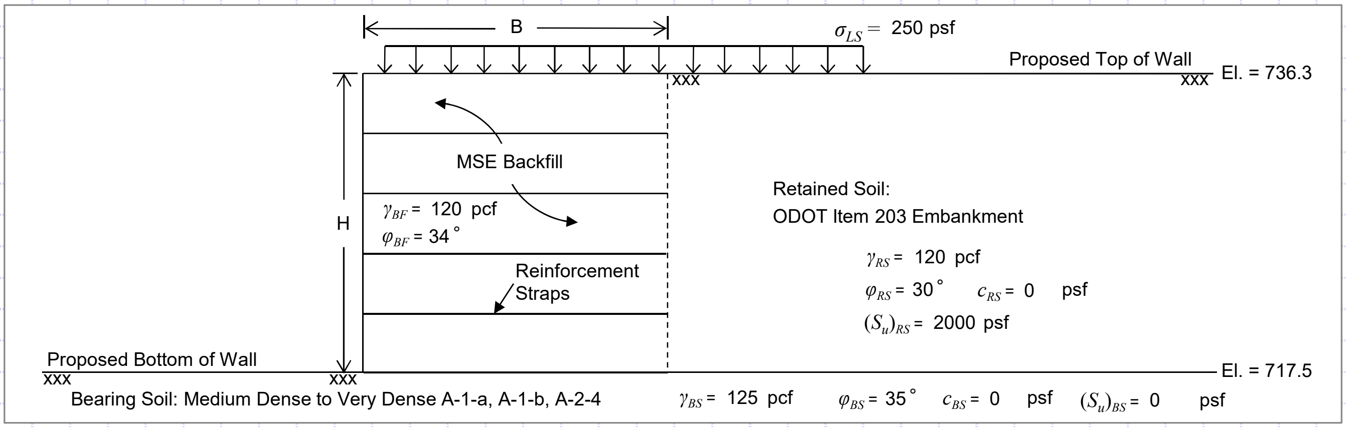
MSE WALL CALCULATIONS

RESOURCE INTERNATIONAL, INC.
6350 PRESIDENTIAL GATEWAY
COLUMBUS, OHIO 43231
PHONE: (614) 823-4949
FAX: (614) 823-4990
WWW.RESOURCEINTERATIONAL.COM

JOB	FRA-70-13.10	NO.	W-13-072
SHEET NO.	1	OF	6
CALCULATED BY	BRT	DATE	7/2/2019
CHECKED BY	JPS	DATE	7/3/2019

Retaining Wall E5 - Sta. 501+18 to 506+12

Retaining Wall E5 - Sta. 501+18 to 506+12 - B-020-8-13, B-021-3-13 and B-023-2-13 - 18.8 ft. Wall Height



MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	18.8	ft
MSE Wall Width (Reinforcement Length), (B) =	13.2	ft
MSE Wall Length, (L) =	222	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}) =	35	°
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) =	15.0	ft

LRFD Load Factors

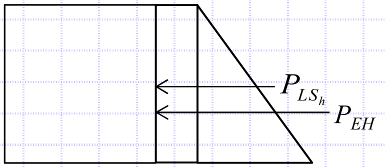
	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

(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:

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



$P_{EH} = \frac{1}{2}\gamma_{RS}H^2K_a\gamma_{EH}$ = ½(120 pcf)(18.8 ft)²(0.297)(1.5) = 9.45 kip/ft

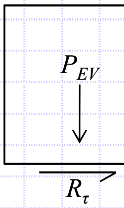
$P_{LS_h} = \sigma_{LS}HK_a\gamma_{LS}$ = (250 psf)(18.8 ft)(0.297)(1.75) = 2.44 kip/ft

P_H = 9.45 kip/ft + 2.44 kip/ft = 11.89 kip/ft

Check Sliding Resistance - Drained Condition

Nominal Sliding Resistance:

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



$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV}$ = (120 pcf)(18.8 ft)(13.2 ft)(1.00) = 29.78 kip/ft

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

$\tan\delta =$ tan(35) ≤ tan(34) ⟶ 0.70 ≤ 0.67 ⟶ $\tan\delta =$ 0.67

R_τ = (29.78 kip/ft)(0.67) = 19.95 kip/ft

Verify Sliding Force Less Than Factored Sliding Resistance - Drained Condition

$P_H \le R_\tau \cdot \phi_\tau$ ⟶ 11.89 kip/ft ≤ (19.95 kip/ft)(1.0) = 19.95 kip/ft ⟶ 11.89 kip/ft ≤ 19.95 kip/ft **OK**

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

JOB	FRA-70-13.10	NO.	W-13-072
SHEET NO.	2	OF	6
CALCULATED BY	BRT	DATE	7/2/2019
CHECKED BY	JPS	DATE	7/3/2019

Retaining Wall E5 - Sta. 501+18 to 506+12

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) = 18.8 ft
MSE Wall Width (Reinforcement Length), (B) = 13.2 ft
MSE Wall Length, (L) = 222 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}) = 35 °
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) = 15.0 ft

LRFD Load Factors

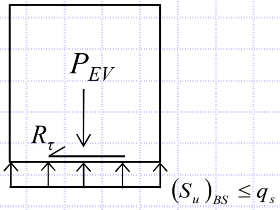
	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

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



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

$q_s = \sigma_v / 2$ = (2.26 ksf) / 2 = 1.13 ksf

$\sigma_v = P_{EV} / B$ = (29.78 kip/ft) / (13.2 ft) = 2.26 ksf

R_τ = (N/A ksf ≤ 1.13 ksf)(13.2 ft) = N/A kip/ft

Verify Sliding Force Less Than Factored Sliding Resistance - Undrained Condition

$P_H \leq R_\tau \cdot \phi_\tau$ → N/A → 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) = 18.8 ft
MSE Wall Width (Reinforcement Length), (B) = 13.2 ft
MSE Wall Length, (L) = 222 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:

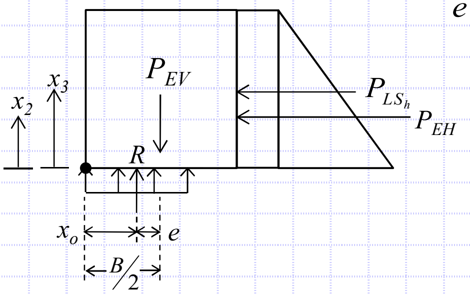
Bearing Soil Unit Weight, (γ_{BS}) = 125 pcf
Bearing Soil Friction Angle, (φ_{BS}) = 35 °
Bearing Soil Drained Cohesion, (c_{BS}) = 0 psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ = 0 psf
Embedment Depth, (D_f) = 4.0 ft
Depth to Grounwater (Below Bot. of Wall), (D_W) = 15.0 ft

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



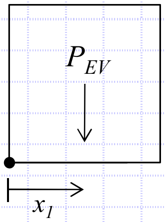
$$e = B/2 - x_o$$

$x_o = \dfrac{M_{EV} - M_H}{P_{EV}}$ = (196.55 kip·ft/ft - 82.19 kip·ft/ft) / (29.78 kip/ft) = 3.84 ft

M_{EV} = 196.55 kip·ft/ft
M_H = 82.19 kip·ft/ft
P_{EV} = 29.78 kip/ft
Defined below

$e =$ (13.2 ft)/2 - 3.84 ft = 2.76 ft

Resisting Moment, M_{EV}:



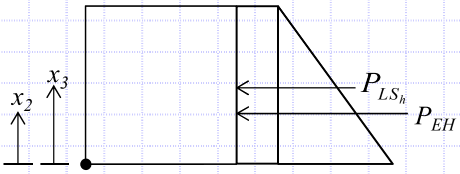
$$M_{EV} = P_{EV}(x_1)$$

$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV}$ = (120 pcf)(18.8 ft)(13.2 ft)(1.00) = 29.78 kip/ft

$x_1 = B/2$ = (13.2 ft) / 2 = 6.60 ft

M_{EV} = (29.78 kip/ft)(6.60 ft) = 196.55 kip·ft/ft

Overturning Moment, M_H:



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

$P_{EH} = ½\gamma_{RS} H^2 K_a \gamma_{EH}$ = ½(120 pcf)(18.8 ft)²(0.297)(1.5) = 9.45 kip/ft

$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS}$ = (250 psf)(18.8 ft)(0.297)(1.75) = 2.44 kip/ft

$x_2 = H/3$ = (18.8 ft) / 3 = 6.27 ft

$x_3 = H/2$ = (18.8 ft) / 2 = 9.40 ft

M_H = (9.45 kip/ft)(6.27 ft) + (2.44 kip/ft)(9.40 ft) = 82.19 kip·ft/ft

Check Eccentricity

$e < e_{\max} \longrightarrow$ 2.76 ft < 4.40 ft **OK**

Limiting Eccentricity: $e_{\max} = B/3 \longrightarrow e_{\max}$ = (13.2 ft) / 3 = 4.40 ft

JOB	FRA-70-13.10	NO.	W-13-072
SHEET NO.	4	OF	6
CALCULATED BY	BRT	DATE	7/2/2019
CHECKED BY	JPS	DATE	7/3/2019

Retaining Wall E5 - Sta. 501+18 to 506+12

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	18.8	ft
MSE Wall Width (Reinforcement Length), (B) =	13.2	ft
MSE Wall Length, (L) =	222	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	°

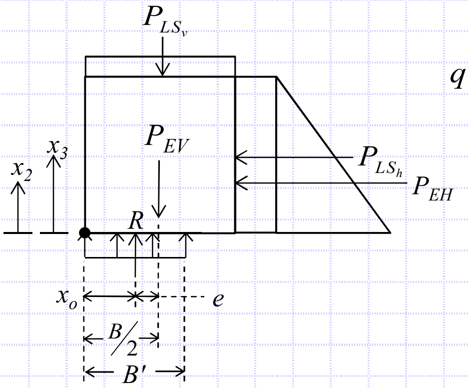
Bearing Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	35	°
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) =	15.0	ft

LRFD Load Factors

	EV	EH	LS	
Strength Ia	1.00	1.50	1.75	*(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)*
Strength Ib	1.35	1.50	1.75	
Service I	1.00	1.00	1.00	

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



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

$$B' = B - 2e$$ = 13.2 ft - 2(1.79 ft) = 9.62 ft

$$e = B/2 - x_o$$ = (13.2 ft) / 2 - 4.81 ft = 1.79 ft

$$x_o = \frac{M_V - M_H}{P_V}$$ = (303.45 kip·ft/ft - 82.2 kip·ft/ft) / 45.98 kip/ft = 4.81 ft

$q_{eq} =$ (45.98 kip/ft) / (9.62 ft) = 4.78 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)(18.8 ft)(13.2 ft)(1.35)](6.6 ft) + [(250 psf)(13.2 ft)(1.75)](6.6 ft) = 303.45 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)(18.8 ft)²(0.297)(1.5)](6.27 ft) + [(250 psf)(18.8 ft)(0.297)(1.75)](9.4 ft) = 82.20 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)(18.8 ft)(13.2 ft)(1.35) + (250 psf)(13.2 ft)(1.75) = 45.98 kip/ft

Check Bearing Resistance - Drained Condition

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

$N_{cm} = N_c s_c i_c$ = 47.55

N_c = 46.12
s_c = 1+(9.62 ft/222 ft)(33.3/46.12)
= 1.031
i_c = 1.000 (Assumed)

$N_{qm} = N_q s_q d_q i_q$ = 37.73

N_q = 33.30
s_q = 1.030
d_q = 1+2tan(35°)[1-sin(35°)]²tan⁻¹(4.0 ft/9.62 ft)
1.100
i_q = 1.000 (Assumed)
C_{wq} = 15.0 ft > 4.0 ft = 1.000

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 47.21

N_γ = 48.03
s_γ = 0.983
i_γ = 1.000 (Assumed)
$C_{w\gamma}$ = 15.0 ft < 1.5(9.62 ft) + 4.0 ft = 1.020

$q_n =$ (0 psf)(47.550) + (125 pcf)(4.0 ft)(37.729)(1.000) + ½(125 pcf)(9.6 ft)(47.213)(1.020) = 47.82 ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

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

$q_{eq} \leq q_n \cdot \phi_b$ ⟶ 4.78 ksf ≤ (47.82 ksf)(0.65) = 31.08 ksf ⟶ 4.78 ksf ≤ 31.08 ksf **OK**

Retaining Wall E5 - Sta. 501+18 to 506+12

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	18.8	ft
MSE Wall Width (Reinforcement Length), (B) =	13.2	ft
MSE Wall Length, (L) =	222	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}) =	35	°
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) =	15.0	ft

LRFD Load Factors

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

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

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

Check Bearing Resistance - Undrained Condition

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

$N_{cm} = N_c s_c i_c$ = 5.190

N_c = 5.140
s_c = 1+(9.62 ft/[(5)(222 ft)]) = 1.009
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/9.62 ft)
1.000
i_q = 1.000 (Assumed)
C_{wq} = 15.0 ft > 4.0 ft = 1.000

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

N_γ = 0.000
s_γ = 1.000
i_γ = 1.000 (Assumed)
$C_{w\gamma}$ = 15.0 ft < 1.5(9.62 ft) + 4.0 ft = 1.020

$q_n =$ (0 psf)(5.190) + (125 pcf)(4.0 ft)(1.000)(1.000) + ½(125 pcf)(9.6 ft)(0.000)(1.020) = N/A ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

$q_{eq} \leq q_n \cdot \phi_b$ ⟶ 4.78 ksf ≤ (N/A ksf)(0.65) = N/A ksf ⟶ 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) = 18.8 ft
MSE Wall Width (Reinforcement Length), (B) = 13.2 ft
MSE Wall Length, (L) = 222 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:

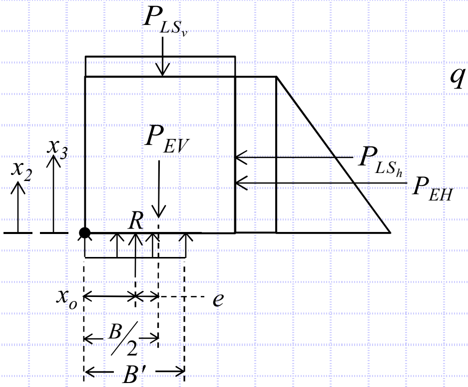
Bearing Soil Unit Weight, (γ_{BS}) = 125 pcf
Bearing Soil Friction Angle, (φ_{BS}) = 35 °
Bearing Soil Drained Cohesion, (c_{BS}) = 0 psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] = 0 psf
Embedment Depth, (D_f) = 4.0 ft
Depth to Grounwater (Below Bot. of Wall), (D_W) = 15.0 ft

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



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

$$B' = B - 2e$$ = 13.2 ft - 2(1.59 ft) = 10.02 ft

$$e = B/2 - x_o$$ = (13.2 ft) / 2 - 5.01 ft = 1.59 ft

$$x_o = \frac{M_V - M_H}{P_V}$$ = (218.32 kip·ft/ft - 52.61 kip·ft/ft) / 33.08 kip/ft = 5.01 ft

q_{eq} = (33.08 kip/ft) / (10.02 ft) = 3.30 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)(18.8 ft)(13.2 ft)(1.00)](6.6 ft) + [(250 psf)(13.2 ft)(1.00)](6.6 ft) = 218.32 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)(18.8 ft)²(0.297)(1.00)](6.27 ft) + [(250 psf)(18.8 ft)(0.297)(1.00)](9.4 ft) = 52.61 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)(18.8 ft)(13.2 ft)(1.00) + (250 psf)(13.2 ft)(1.00) = 33.08 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-020-8-13	0.816 in	0.688 in	30 days		
B-021-3-13	0.569 in	0.473 in	5 days	105 ft	1/5860
B-023-2-13	0.697 in	0.540 in	1 days	105 ft	1/18810

W-13-072 - FRA-70-13.10 - Retaining Wall E5

MSE Wall Settlement - Sta. 501+18 to 506+12

Calculated By: BRT Date: 7/2/2019

Checked By: JPS Date: 7/3/2019

Boring B-020-8-13

H= 18.8 ft Total wall height

B'= 10.0 ft Effective footing width due to eccentricity

D_w = 15.0 ft Depth below bottom of footing

q_e = 3,300 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 (Emb.)	A-1-b	G	0.0	2.0	2.0	1.0	120	240	120	120	4,120					30	58	207	0.10	0.997	3,289	3,409	0.014	0.169	0.500	1,649	1,769	0.011	0.136
2	A-1-a	G	2.0	4.5	2.5	3.3	125	553	396	396	4,396					23	35	115	0.33	0.924	3,049	3,445	0.020	0.244	0.494	1,629	2,025	0.015	0.184
3	A-1-a	G	4.5	7.5	3.0	6.0	135	958	755	755	4,755					89	118	621	0.60	0.755	2,493	3,248	0.003	0.037	0.468	1,546	2,301	0.002	0.028
	A-1-a	G	7.5	10.5	3.0	9.0	135	1,363	1,160	1,160	5,160					89	105	511	0.90	0.593	1,957	3,117	0.003	0.030	0.425	1,403	2,563	0.002	0.024
	A-1-a	G	10.5	13.5	3.0	12.0	135	1,768	1,565	1,565	5,565					89	96	442	1.20	0.477	1,575	3,140	0.002	0.025	0.378	1,246	2,811	0.002	0.021
	A-1-a	G	13.5	17.0	3.5	15.3	135	2,240	2,004	1,988	5,988					89	89	390	1.53	0.390	1,288	3,276	0.002	0.023	0.331	1,091	3,079	0.002	0.020
4	A-3a	G	17.0	22.0	5.0	19.5	130	2,890	2,565	2,284	6,284					48	46	128	1.95	0.313	1,033	3,317	0.006	0.076	0.280	924	3,208	0.006	0.069
	A-3a	G	22.0	28.5	6.5	25.3	130	3,735	3,313	2,673	6,673					48	43	120	2.53	0.246	811	3,484	0.006	0.075	0.229	756	3,429	0.006	0.070
5	A-1-b	G	28.5	38.0	9.5	33.3	135	5,018	4,376	3,237	7,237					112	94	425	3.33	0.189	622	3,860	0.002	0.021	0.181	597	3,834	0.002	0.020
	A-1-b	G	38.0	48.0	10.0	43.0	135	6,368	5,693	3,945	7,945					112	87	372	4.30	0.147	484	4,430	0.001	0.016	0.143	472	4,417	0.001	0.016
6	A-7-6	C	48.0	54.5	6.5	51.3	130	7,213	6,790	4,528	8,528	43	0.297	0.030	0.608				5.13	0.123	407	4,935	0.004	0.054	0.121	400	4,928	0.004	0.053
	A-7-6	C	54.5	61.5	7.0	58.0	130	8,123	7,668	4,984	8,984	43	0.297	0.030	0.608				5.80	0.109	360	5,345	0.004	0.047	0.108	355	5,340	0.004	0.046
																				Total Settlement:			0.816 in		Total Settlement:			0.688 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 E5
MSE Wall Settlement - Sta. 501+18 to 506+12

Calculated By: BRT Date: 07/02/2019
Checked By: JPS Date: 07/03/2019

Boring B-020-8-13

H=	18.8	ft	Total wall height
B'=	10.0	ft	Effective footing width due to eccentricity
D_w =	15.0	ft	Depth below bottom of footing
q_e =	3,300	psf	Equivalent bearing pressure at bottom of wall

	A-7-6		
c_v =	150	ft^2/yr	Coefficient of consolitation
t =	30	days	Time following completion of construction
H_{dr} =	13.5	ft	Length of longest drainage path considered
T_v =	0.068		Time factor
U =	29	%	Degree of consolidation

$(S_c)_t$ = 0.617 in Settlement complete at 90% of primary consolidation

																							Total Settlement at Facing of Wall			Settlement Complete at 90% of Primary Consolidation	
Layer	Soil Type	Soil Type	Layer Depth (ft)		Layer Thickness (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	Layer Settlement (in)	$(S_c)_t$ (11) (in)	Layer Settlement (in)
1 (Emb.)	A-1-b	G	0.0	2.0	2.0	1.0	120	240	120	120	4,120					30	58	207	0.10	0.500	1,649	1,769	0.011	0.136	0.136	0.136	0.136
2	A-1-a	G	2.0	4.5	2.5	3.3	125	553	396	396	4,396					23	35	115	0.33	0.494	1,629	2,025	0.015	0.184	0.184	0.184	0.184
3	A-1-a	G	4.5	7.5	3.0	6.0	135	958	755	755	4,755					89	118	621	0.60	0.468	1,546	2,301	0.002	0.028	0.094	0.028	0.094
	A-1-a	G	7.5	10.5	3.0	9.0	135	1,363	1,160	1,160	5,160					89	105	511	0.90	0.425	1,403	2,563	0.002	0.024		0.024	
	A-1-a	G	10.5	13.5	3.0	12.0	135	1,768	1,565	1,565	5,565					89	96	442	1.20	0.378	1,246	2,811	0.002	0.021		0.021	
	A-1-a	G	13.5	17.0	3.5	15.3	135	2,240	2,004	1,988	5,988					89	89	390	1.53	0.331	1,091	3,079	0.002	0.020		0.020	
4	A-3a	G	17.0	22.0	5.0	19.5	130	2,890	2,565	2,284	6,284					48	46	128	1.95	0.280	924	3,208	0.006	0.069	0.139	0.069	0.139
	A-3a	G	22.0	28.5	6.5	25.3	130	3,735	3,313	2,673	6,673					48	43	120	2.53	0.229	756	3,429	0.006	0.070		0.070	
5	A-1-b	G	28.5	38.0	9.5	33.3	135	5,018	4,376	3,237	7,237					112	94	425	3.33	0.181	597	3,834	0.002	0.020	0.036	0.020	0.036
	A-1-b	G	38.0	48.0	10.0	43.0	135	6,368	5,693	3,945	7,945					112	87	372	4.30	0.143	472	4,417	0.001	0.016		0.016	
6	A-7-6	C	48.0	54.5	6.5	51.3	130	7,213	6,790	4,528	8,528	43	0.297	0.030	0.608				5.13	0.121	400	4,928	0.004	0.053	0.099	0.015	0.029
	A-7-6	C	54.5	61.5	7.0	58.0	130	8,123	7,668	4,984	8,984	43	0.297	0.030	0.608				5.80	0.108	355	5,340	0.004	0.046		0.013	

Settlement Remaining After Hold Period: 0.071 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)
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 E5
MSE Wall Settlement - Sta. 501+18 to 506+12

Calculated By: BRT Date: 7/2/2019
Checked By: JPS Date: 7/3/2019

Boring B-021-3-13

H= 14.2 ft Total wall height
B'= 7.4 ft Effective footing width due to eccentricity
D_w = 14.0 ft Depth below bottom of footing
q_e = 2,620 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	3.0	3.0	1.5	130	390	195	195	4,195					35	62	227	0.20	0.976	2,558	2,753	0.015	0.183	0.498	1,306	1,501	0.012	0.141
	A-1-b	G	3.0	6.0	3.0	4.5	130	780	585	585	4,585					35	49	167	0.61	0.750	1,966	2,551	0.011	0.138	0.467	1,225	1,810	0.009	0.106
2	A-1-b	G	6.0	8.5	2.5	7.3	135	1,118	949	949	4,949					120	150	945	0.98	0.558	1,462	2,411	0.001	0.013	0.412	1,080	2,029	0.001	0.010
	A-1-b	G	8.5	11.0	2.5	9.8	135	1,455	1,286	1,286	5,286					120	138	813	1.32	0.442	1,158	2,444	0.001	0.010	0.360	943	2,229	0.001	0.009
3	A-3a	G	11.0	13.5	2.5	12.3	125	1,768	1,611	1,611	5,611					14	15	59	1.66	0.363	951	2,562	0.009	0.103	0.314	822	2,434	0.008	0.091
4	A-1-b	G	13.5	16.0	2.5	14.8	135	2,105	1,936	1,889	5,889					120	122	661	1.99	0.307	804	2,693	0.001	0.007	0.276	722	2,612	0.001	0.006
5	A-6a	C	16.0	19.5	3.5	17.8	130	2,560	2,333	2,099	6,099	25	0.135	0.014	0.467				2.40	0.258	676	2,774	0.004	0.047	0.239	626	2,724	0.004	0.044
6	A-4a	C	19.5	23.5	4.0	21.5	130	3,080	2,820	2,352	6,352	24	0.126	0.013	0.460				2.91	0.215	563	2,915	0.003	0.039	0.203	533	2,885	0.003	0.037
	A-4a	C	23.5	27.5	4.0	25.5	130	3,600	3,340	2,622	6,622	24	0.126	0.013	0.460				3.45	0.182	477	3,100	0.003	0.030	0.175	459	3,081	0.002	0.029
																				Total Settlement:			0.569 in		Total Settlement:			0.473 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 E5
MSE Wall Settlement - Sta. 501+18 to 506+12

Calculated By: BRT Date: 07/02/2019
Checked By: JPS Date: 07/03/2019

Boring B-021-3-13

H=	14.2	ft	Total wall height
B'=	7.4	ft	Effective footing width due to eccentricity
D_w =	14.0	ft	Depth below bottom of footing
q_e =	2,620	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} =	3.5	11.5		ft	Length of longest drainage path considered
T_v =	0.671	0.104			Time factor
U =	85	36		%	Degree of consolidation

$(S_c)_t$ = 0.424 in Settlement complete at 90% of primary consolidation

																							Total Settlement at Facing of Wall			Settlement Complete at 90% of Primary Consolidation	
Layer	Soil Type	Soil Type	Layer Depth (ft)		Layer Thickness (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	Layer Settlement (in)	$(S_c)_t$ (11) (in)	Layer Settlement (in)
1	A-1-b	G	0.0	3.0	3.0	1.5	130	390	195	195	4,195					35	62	227	0.20	0.498	1,306	1,501	0.012	0.141	0.246	0.141	0.246
	A-1-b	G	3.0	6.0	3.0	4.5	130	780	585	585	4,585					35	49	167	0.61	0.467	1,225	1,810	0.009	0.106		0.106	
2	A-1-b	G	6.0	8.5	2.5	7.3	135	1,118	949	949	4,949					120	150	945	0.98	0.412	1,080	2,029	0.001	0.010	0.019	0.010	0.019
	A-1-b	G	8.5	11.0	2.5	9.8	135	1,455	1,286	1,286	5,286					120	138	813	1.32	0.360	943	2,229	0.001	0.009		0.009	
3	A-3a	G	11.0	13.5	2.5	12.3	125	1,768	1,611	1,611	5,611					14	15	59	1.66	0.314	822	2,434	0.008	0.091	0.091	0.091	0.091
4	A-1-b	G	13.5	16.0	2.5	14.8	135	2,105	1,936	1,889	5,889					120	122	661	1.99	0.276	722	2,612	0.001	0.006	0.006	0.006	0.006
5	A-6a	C	16.0	19.5	3.5	17.8	130	2,560	2,333	2,099	6,099	25	0.135	0.014	0.467				2.40	0.239	626	2,724	0.004	0.044	0.044	0.037	0.037
6	A-4a	C	19.5	23.5	4.0	21.5	130	3,080	2,820	2,352	6,352	24	0.126	0.013	0.460				2.91	0.203	533	2,885	0.003	0.037	0.066	0.013	0.024
	A-4a	C	23.5	27.5	4.0	25.5	130	3,600	3,340	2,622	6,622	24	0.126	0.013	0.460				3.45	0.175	459	3,081	0.002	0.029		0.010	

Settlement Remaining After Hold Period: 0.049 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

W-13-072 - FRA-70-13.10 - Retaining Wall E5
MSE Wall Settlement - Sta. 501+18 to 506+12

Calculated By: BRT Date: 7/2/2019
Checked By: JPS Date: 7/3/2019

Boring B-023-2-13

H=	6.5	ft	Total wall height
B'=	7.2	ft	Effective footing width due to eccentricity
D_w =	8.0	ft	Depth below bottom of footing
q_e =	1,140	psf	Equivalent bearing pressure at bottom of wall

																				Total Settlement at Center of Reinforced Soil Mass					Total Settlement at Facing of Wall				
Layer	Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
1	A-4a	C	0.0	2.5	2.5	1.3	120	300	150	150	4,150	25	0.135	0.014	0.467				0.17	0.985	1,122	1,272	0.021	0.256	0.499	569	719	0.016	0.188
2	A-1-a	G	2.5	5.5	3.0	4.0	125	675	488	488	4,488					25	37	120	0.56	0.783	893	1,380	0.011	0.136	0.474	540	1,028	0.008	0.097
3	A-2-4	G	5.5	7.5	2.0	6.5	125	925	800	800	4,800					12	16	66	0.90	0.592	675	1,475	0.008	0.097	0.425	484	1,284	0.006	0.075
	A-2-4	G	7.5	10.0	2.5	8.8	125	1,238	1,081	1,034	5,034					12	15	64	1.22	0.472	539	1,573	0.007	0.085	0.375	428	1,462	0.006	0.070
4	A-6b	C	10.0	12.5	2.5	11.3	120	1,538	1,388	1,185	5,185	33	0.207	0.021	0.530				1.56	0.382	435	1,620	0.005	0.055	0.326	371	1,556	0.004	0.048
	A-6b	C	12.5	15.0	2.5	13.8	120	1,838	1,688	1,329	5,329	33	0.207	0.021	0.530				1.91	0.319	364	1,692	0.004	0.043	0.284	324	1,653	0.003	0.038
5	A-1-a	G	15.0	19.5	4.5	17.3	130	2,423	2,130	1,553	5,553					45	49	165	2.40	0.258	294	1,847	0.002	0.025	0.239	272	1,825	0.002	0.023
																						Total Settlement:		0.697 in			Total Settlement:		0.540 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 E5
MSE Wall Settlement - Sta. 501+18 to 506+12

Calculated By: BRT Date: 07/02/2019
Checked By: JPS Date: 07/03/2019

Boring B-023-2-13

H=	6.5	ft	Total wall height
B'=	7.2	ft	Effective footing width due to eccentricity
D_w =	8.0	ft	Depth below bottom of footing
q_e =	1,140	psf	Equivalent bearing pressure at bottom of wall

	A-4a	A-6b			
c_v =	1,000	300		ft^2/yr	Coefficient of consolidation
t =	1	1		days	Time following completion of construction
H_{dr} =	1.25	2.5		ft	Length of longest drainage path considered
T_v =	1.753	0.132			Time factor
U =	99	41		%	Degree of consolidation

$(S_c)_t$ = 0.487 in Settlement complete at 90% of primary consolidation

																							Total Settlement at Facing of Wall			Settlement Complete at 90% of Primary Consolidation	
Layer	Soil Type	Soil Type	Layer Depth (ft)		Layer Thickness (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	$\sigma_p'^{(1)}$ (psf)	LL	$C_c^{(2)}$	$C_r^{(3)}$	$e_o^{(4)}$	N_{60}	$(N1)_{60}^{(5)}$	$C'^{(6)}$	Z_f/B	$I^{(7)}$	$\Delta\sigma_v^{(8)}$ (psf)	σ_{vf}' Midpoint (psf)	$S_c^{(9,10)}$ (ft)	S_c (in)	Layer Settlement (in)	$(S_c)_t^{(11)}$ (in)	Layer Settlement (in)
1	A-4a	C	0.0	2.5	2.5	1.3	120	300	150	150	4,150	25	0.135	0.014	0.467				0.17	0.499	569	719	0.016	0.188	0.188	0.186	0.186
2	A-1-a	G	2.5	5.5	3.0	4.0	125	675	488	488	4,488					25	37	120	0.56	0.474	540	1,028	0.008	0.097	0.097	0.097	0.097
3	A-2-4	G	5.5	7.5	2.0	6.5	125	925	800	800	4,800					12	16	66	0.90	0.425	484	1,284	0.006	0.075	0.145	0.075	0.145
	A-2-4	G	7.5	10.0	2.5	8.8	125	1,238	1,081	1,034	5,034					12	15	64	1.22	0.375	428	1,462	0.006	0.070		0.070	
4	A-6b	C	10.0	12.5	2.5	11.3	120	1,538	1,388	1,185	5,185	33	0.207	0.021	0.530				1.56	0.326	371	1,556	0.004	0.048	0.087	0.020	0.035
	A-6b	C	12.5	15.0	2.5	13.8	120	1,838	1,688	1,329	5,329	33	0.207	0.021	0.530				1.91	0.284	324	1,653	0.003	0.038		0.016	
5	A-1-a	G	15.0	19.5	4.5	17.3	130	2,423	2,130	1,553	5,553					45	49	165	2.40	0.239	272	1,825	0.002	0.023	0.023	0.023	0.023

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
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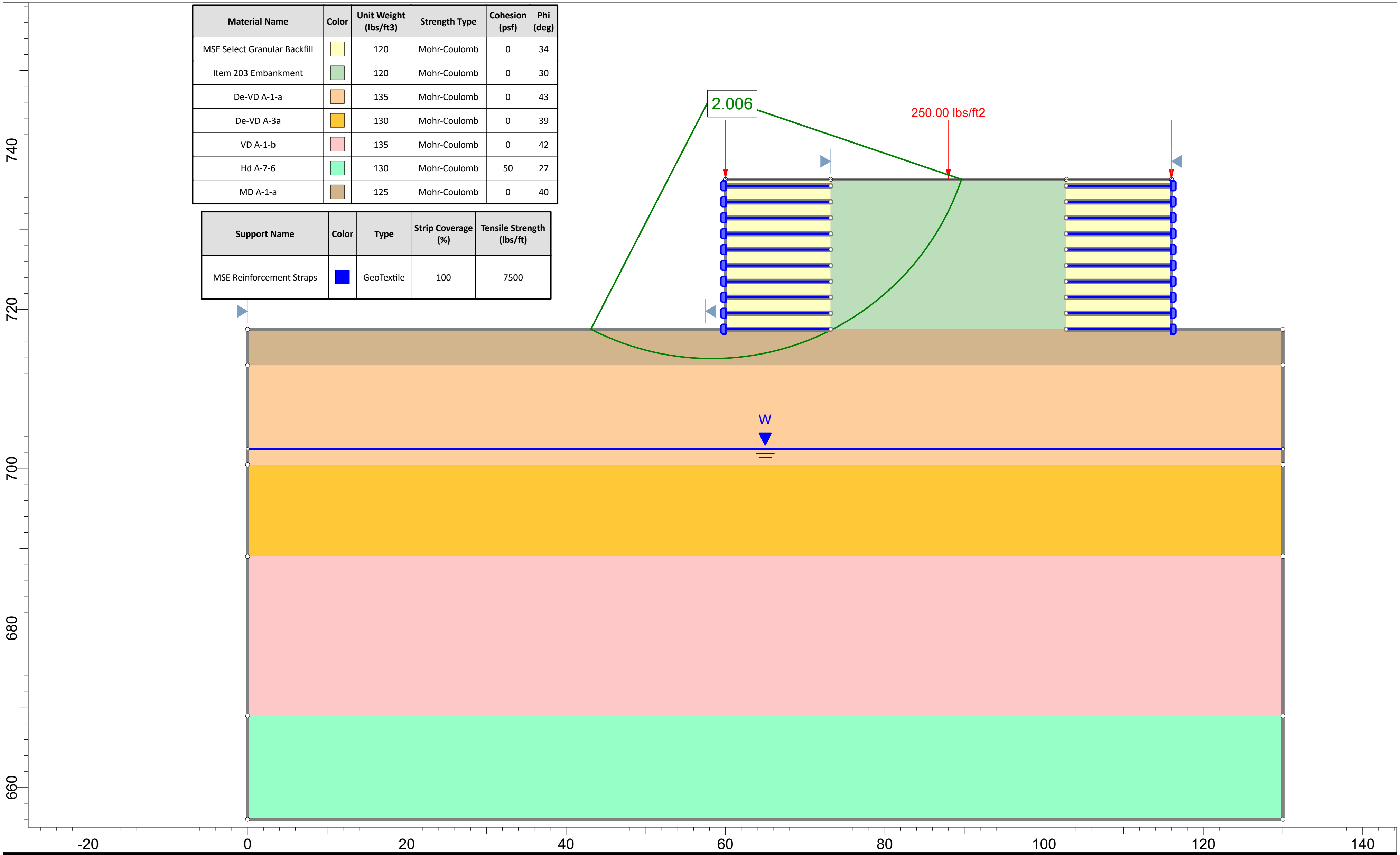
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)
11. $(S_c)_t = S_c(U/100)$; U = 100 for all granular soils at time t = 0

Settlement Remaining After Hold Period: 0.053 in



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-VD A-1-a	🟧	135	Mohr-Coulomb	0	43
De-VD A-3a	🟨	130	Mohr-Coulomb	0	39
VD A-1-b	🟥	135	Mohr-Coulomb	0	42
Hd A-7-6	🟩	130	Mohr-Coulomb	50	27
MD A-1-a	🟫	125	Mohr-Coulomb	0	40

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. 501+18 to 506+12 - MSE Wall Global Stability

Analysis Description
18.8 ft Wall Height - Drained - Circular - Spencer

Drawn By BRT
Scale 1:200
Company Resource International, Inc.

Date 7/3/2019
File Name Retaining Wall E5 - Global Stability.slim