FRA-71-14.36 PHASE 6R
RETAINING WALL E4
PID NO. 105588
FRANKLIN COUNTY, OHIO

STRUCTURE FOUNDATION EXPLORATION REPORT (REV. 1)

Prepared For:
ms consultants, inc.
2221 Schrock Road
Columbus, OH 43229-1547

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

July 2019

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

Planning
Engineering
Construction Management
Technology

June 22, 2015 (Revised July 19, 2019)

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

Re: Structure Foundation Exploration Report (Rev. 1)
FRA-71-14.36 Phase 6R
Retaining Wall E4
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 E4 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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TABLE OF CONTENTS

APPENDICIES

EXECUTIVE SUMMARY

Resource International, Inc. (Rii) has completed a structure foundation exploration for the design and construction of the proposed Retaining Wall E4. Based on proposed plan information provided by the Rii, Retaining Wall E4 will support Ramp D7 between the FRA-70-1358L bridge structure over CSX and Norfolk Southern railroads and the FRA-70-1373B bridge structure over Short Street, and will provide the required grade separation such that graded embankments can be utilized up to the existing Mound Street alignment. For the final wall configuration, the wall begins at Sta. 7007+00 (BL Ramp D7) and extends east to Sta. 7007+66 (BL Ramp D7), where the wall wraps around the rear abutment of the proposed FRA-70-1373B structure, and continues back to the west along the north side of Ramp D7 to Sta. 7000+14 (BL Ramp D7), where the wall wraps around the forward abutment of the proposed FRA-70-1358L structure and ends at Sta. 7000+69 (BL Ramp D7). However, for the FRA-71-14.36 project, the wall will only be partially constructed and will end at Sta. 17005+55 (BL Transition Ramp D7), where Ramp D7 will merge with the existing I-70 westbound. The total wall length for Retaining Wall E4 for the final build, including the portion of the wall that crosses in front of the abutments of the proposed bridge structures, is approximately 1,007 lineal feet, and the total length between the abutments of the two crossings is approximately 750 feet. It is understood that a mechanically stabilized earth (MSE) wall is being considered as the preferred wall type for Retaining Wall E4.

Exploration and Findings

Between February 4 and May 13, 2014, five (5) borings, designated as B-018-2-13, B-020-4-13, B-114-7-13, B-114-8-13 and B-114-9-13, were drilled to completion depths ranging from 82.0 to 94.7 feet below the existing ground surface. In addition to the borings performed by Rii as part of the current exploration, one (1) boring, designated as B-017-2-09, was advanced to a completion depth of 54.8 feet below the existing ground surface by DLZ as part of the FRA-70-8.93 preliminary exploration. The project boring locations are shown on the boring plan provided in Appendix I of the full report.

Boring B-017-2-09 was drilled through the existing pavement of Mound Street and encountered 2.0 inches of asphalt overlying 9.0 inches of concrete followed by 3.0 inches of aggregate base. The remaining borings were drilled in the grass area between I-70 and Mound Street and encountered 2.0 to 6.0 inches of topsoil at the ground surface, as identified by the significant presence of vegetation and organic material.

Beneath the surface materials in borings B-017-2-09, B-018-2-13, B-020-4-13, B-114-8-13 and B-114-9-13, material identified as existing fill was encountered extending to depths ranging from 5.5 to 14.5 feet below the existing ground surface, which corresponds to elevations ranging from 699.5 to 708.5 feet msl. The fill material consisted of dark brown, black and brown gravel with sand, gravel with sand and silt, sandy silt, silt and clay and silty clay (ODOT A-1-b, A-2-4, A-4a, A-6a, A-6b). The fill materials contained trash and debris, including organic material, root fibers, cinders, plastic, coal, brick and slag fragments throughout.

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

Top of bedrock was encountered in borings B-018-2-13, B-020-4-13, B-114-7-13, B-114-8-13 and B-114-9-13 at depths ranging from 58.8 to 64.9 feet below the existing ground surface, which corresponds to elevations ranging from 648.4 to 656.9 feet msl. The upper portion of the bedrock consists of weathered shale which was able to be augered to competent shale and claystone bedrock in borings B-020-4-13, B-114-8-13 and B-114-9-13 and competent limestone bedrock in borings B-018-2-13 and B-114-7-13. The cored bedrock consists of shale and claystone, which was encountered in borings B-020-4-13, B-114-8-13 and B-114-9-13 at an elevation of 639.3, 647.0 and 649.0 feet msl, respectively, overly limestone bedrock, which was encountered in all of the borings at elevations ranging from 629.0 fee msl in boring B-114-9-13 to 641.2 feet msl in boring B-018-2-13.

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 E4 is proposed to be a MSE wall type that will support Ramp D7 between the FRA-70-1358L and FRA-70-1373B bridge structures. The wall will support the roadway along Ramp D7 between the FRA-70-1358L and FRA-70-1373B bridge structures, and will wrap around in front of the forward abutment of the proposed FRA-70-1358L bridge structure at the west end of the wall alignment and to the rear abutment of the FRA-70-1373B bridge structure at the east end of the wall alignment. The total wall length for Retaining Wall E4, including the portions of the wall that cross in front of the abutments of the proposed bridge structures, is approximately 1,007 lineal feet, and the total length between the abutments of the two bridges, from Sta. 402+02 to 409+52 (BL Wall E4), is approximately 750 feet.

MSE Wall Recommendations

Based on the proposed plan and profile information, the proposed retaining wall will have a maximum height of 27.7 feet, as measured from the top of the leveling pad to the top of the coping, at the east end of the alignment where it connects to the rear abutment of the FRA-70-1373B structure. The wall will step up along the proposed embankment as the wall continues west, with a minimum wall height of approximately 8.7 feet at Sta. 405+20 (BL Wall E4), and will step back down the proposed embankment to a maximum wall height of 42.1 feet where it connects to the forward abutment of the FRA-70-1358L structure. Fill heights up to approximately 30 feet fill will be required to bring the existing grade up to the proposed bottom of wall elevation along the wall alignment.

Material identified as existing fill or possible fill consisting of loose to medium dense gravel with sand, gravel with sand and silt (ODOT A-1-b, A-2-4) and stiff to very stiff sandy silt, silt and clay and silty clay (ODOT A-4a, A-6a, A-6b), which contained trash and debris, including organic material, root fibers, cinders, plastic, coal, brick and slag fragments, was encountered at the existing grade within the area outside of the existing I-70 embankment. These unsuitable soils extend to a depths ranging from 5.5 to 14.3 feet below the existing ground surface grade (El. 699.5 to 706.0 feet msl). As noted in Section 5.1 of the full report, it is understood that ground improvement techniques will be implemented within the entire footprint of Wall E4 and the proposed embankment in areas outside of the existing I-70 embankment. As this is a proprietary design, the analysis for this wall considers the existing fill material will remain in place and will not be stabilized.

MSE wall foundations bearing on existing fill material or granular embankment, placed and compacted in accordance with ODOT Item 203, may be proportioned for a factored bearing resistance as indicated in the following table. A geotechnical resistance factor of φ_b=0.65 was considered in calculating the factored bearing resistance at the strength limit state.

Retaining Wall E4 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 [3] (ksf)		Strength Limit Equivalent Bearing Pressure [5] (ksf)
					Nominal	Factored [4]	
402+02	404+75	27.7	Level	19.4 (0.70H)	3.47	2.26	6.63
404+75	407+00	15.1	Level	10.6 (0.70H ≥ 8.0)	13.76	5.19 (5)	4.01
407+00	409+50	42.1	Level	29.5 (0.70H)	5.41	3.52	9.64

1. *Stationing referenced to the baseline of Retaining Wall E4.*
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.*
5. *The factored bearing resistance includes a reduction factor applied to the nominal resistance to account for the fore slope in front of the wall per Section 10.6.3.1.2c of the 2018 AASHTO LRFD BDM.*

The settlement analysis for the section of the wall alignment where the wall steps up the proposed embankment fill was performed considering the equivalent bearing pressure at the bottom of the MSW wall at the service limit state, as well as the weight of the embankment fill below the proposed bottom of wall elevation. Results of the settlement analysis are tabulated in Table 9. Total settlements ranging from 7.70 to 15.59 inches at the center of the reinforced soil mass and 4.76 to 8.38 inches at the facing of the wall are anticipated along the alignment of Retaining Wall E4. Based on the results of the analysis, 90 percent of the total settlement is anticipated to occur over a period of approximately 8 to 100 days.

Based on the results of the external and global stability analysis performed for the MSE wall, sliding under undrained conditions as well as bearing and global stability under both drained and undrained conditions were not satisfied at a strap length equal to 0.7 times the wall height. Increasing the width of the wall up to 1.3 times the wall height still did not satisfy all of the external and global stability requirements.

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

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 E4, as shown on the vicinity map and boring plan presented in Appendix I. Based on proposed plan information provided by the Rii design team, Retaining Wall E4 will support Ramp D7 between the FRA-70-1358L bridge structure over CSX and Norfolk Southern railroads and the FRA-70-1373B bridge structure over Short Street, and will provide the required grade separation such that graded embankments can be utilized up to the existing Mound Street alignment. The proposed Ramp D7 will be a two-lane ramp that will carry traffic from Mound Street to I-70 westbound. For the final wall configuration, the wall begins at Sta. 7007+00 (BL Ramp D7) and extends east to Sta. 7007+66 (BL Ramp D7), where the wall wraps around the rear abutment of the proposed FRA-70-1373B structure, and continues back to the west along the north side of Ramp D7 to Sta. 7000+14 (BL Ramp D7), where the wall wraps around the forward abutment of the proposed FRA-70-1358L structure and ends at Sta. 7000+69 (BL Ramp D7). However, for the FRA-71-14.36 project, the wall will only be partially constructed and will end at Sta. 17005+55 (BL Transition Ramp D7), where Ramp D7 will merge with the existing I-70 westbound. The total wall length for Retaining Wall E4 for the final build, including the portion of the wall that crosses in front of the abutments of the proposed bridge structures, is approximately 1,007 lineal feet, and the total length between the abutments of the two crossings is approximately 750 feet. **Please note that the recommendations for the portion of the retaining wall where it crosses in front of the forward abutment of the FRA-70-1358L structure and the rear abutment of the FRA-70-1373B structure have been provided in the respective bridge structure foundation exploration reports, which are presented under separate covers. Additionally, this report provides recommendations for the final alignment and configuration of Retaining Wall E4.** The wall heights along the portion of the wall alignment that is considered for this exploration report will range from 9.2 feet at Sta. 405+20 (BL Wall E4) to 42.1 feet at Sta. 409+30 (BL Wall E4). It is understood that a mechanically stabilized earth (MSE) wall is being considered as the preferred wall type for Retaining Wall E4.

2.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1 Site Geology

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

According to the bedrock geology and topography maps obtained from the Ohio Department of Natural Resources (ODNR), the underlying bedrock consists predominantly of the Middle to Lower Devonian-aged Columbus Limestone. This formation is further subdivided into two members in the central portion of the state, known as the Delhi and Bellepoint Members. The Delhi Member consists of light gray, finely to coarsely crystalline, irregularly bedded, fossiliferous limestone. The Bellepoint Member consists of variable brown, finely crystalline, massively bedded limy dolomite. Both of these members contain chert nodules. Just east of the Scioto River, the underlying bedrock consists of the Upper Devonian Ohio Shale Formation overlying the Middle Devonian-aged Delaware Limestone Formation. The Ohio Shale formation consists of brownish black to greenish gray, thinly bedded, fissile, carbonaceous shale. The Delaware Limestone consists of bluish gray, thin to medium bedded dolomitic limestone with nodules and layers of chert. Regionally, the bedrock surface forms a broad valley aligned roughly north-to-south beneath the Scioto River. According to bedrock topography mapping, the elevation of the bedrock surface ranges from approximately 600 feet mean sea level (msl) in the valley to approximately 625 feet msl near the project limits. Within boring B-020-5-13 performed for this current project, shale and mudstone bedrock was encountered beginning at a depth of 76.6 feet below the existing ground surface, which corresponds to an elevation of 656.8 feet msl.

2.2 Existing Conditions

The proposed Retaining Wall E4 structure will be situated along the south side of the existing Mound Street, west of Short Street, east of the railroads and north of the existing I-70 westbound. Mound Street in the vicinity of the proposed structure is an asphalt paved roadway that narrows from three lanes down to two to the west and ends at a cul-de-sac within the Miranova complex. The existing I-70 westbound in the vicinity of the structure is a four-lane, asphalt paved roadway that is aligned east-to-west. The existing I-70 roadway profile grade is elevated approximately 18 feet above the adjacent grade. There is an existing tennis court at the southwest corner of the intersection of Mound Street and Short Street. The terrain along both roadways and the surrounding area is relatively flat-lying, with dense vegetation covering the existing embankment slope that supports I-70 westbound. The area between the I-70 embankment and Mound Street is grass covered with sparse vegetation.

3.0 EXPLORATION

Between February 4 and May 13, 2014, five (5) borings, designated as B-018-2-13, B-020-4-13, B-114-7-13, B-114-8-13 and B-114-9-13, were drilled to completion depths ranging from 82.0 to 94.7 feet below the existing ground surface. In addition to the borings performed by Rii as part of the current exploration, one (1) boring, designated as B-017-2-09, was performed DLZ in the vicinity of the wall alignment as part of the FRA-70-8.93 preliminary exploration, and the findings were published in the report dated March 18, 2010. The boring was advanced to a completion depth of 54.8 feet below the existing ground surface. The project boring locations are shown on the boring plan provided in Appendix I of this report and summarized in Table 1.

Table 1. Test Boring Summary

Boring Number	Reference Alignment	Station	Offset	Latitude	Longitude	Ground Elevation (feet msl)	Boring Depth (feet)
B-017-2-09	BL I-70 WB	169+62.30	117.3' Lt.	39.953797580	-83.006927454	713.0	54.8
B-018-2-13	BL I-70 WB	168+63.72	22.9' Lt.	39.953512324	-83.007250705	715.7	84.5
B-020-4-13	BL Ramp D7	7007+11.61	12.0’ Rt.	39.954037298	-83.005708660	714.0	94.7
B-114-7-13	BL I-71 SB	271+12.84	28.4’ Rt.	39.953781634	-83.006602443	713.3	84.0
B-114-8-13	BL I-71 SB	273+13.25	16.5’ Lt.	39.953893690	-83.005886092	714.0	82.0
B-114-9-13	BL I-71 SB	275+07.50	4.7' Rt.	39.953788084	-83.005201418	714.0	90.0

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

The borings performed by Rii for the current exploration were drilled using a truck or an all-terrain vehicle (ATV) mounted rotary drilling machine, utilizing a 3.25 or 4.25-inch inside diameter, hollow-stem auger to advance the holes. Standard penetration test (SPT) and split spoon were performed in the borings at 2.5-foot intervals to a depth of 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 hammers for the Mobile B-53 and CME 750 drill rigs operated by Rii were calibrated on April 26, 2013, and have drill rod energy ratios of 77.7 and 82.6 percent, respectively. The hammer for the CME-75 truck rig operated by DLZ has a drill rod energy ratio of 62.0 percent.

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

Table 2. Laboratory Test Schedule

Laboratory Test	Test Designation	Number of Tests Performed
Natural Moisture Content	ASTM D 2216	102
Plastic and Liquid Limits	AASHTO T89, T90	34
Gradation – Sieve/Hydrometer	AASHTO T88	35
Unconfined Compressive Strength of Intact Rock	ASTM D7012	7
Point Load Strength Index of Rock Specimens	ASTM D 5731	1

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

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

The depth to bedrock in borings B-018-2-13, B-020-4-13, B-114-7-13, B-114-8-13 and B-114-9-13 was determined by split spoon sampler refusal. Split spoon sampler refusal is defined as exceeding 50 blows from the hammer with less than 6.0 inches of penetration by the split spoon sampler. Where borings were extended into the bedrock, an NQ or HQ-sized double-tube diamond bit core barrel (utilizing wire line equipment) was used to core the bedrock. Coring produced 1.85 or 2.5-inch diameter cores, respectively, 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:

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

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) at the time the exploration borings were performed. The following is a summary of what was found in the test borings performed as part of the preliminary engineering phase and current exploration and what is represented on the boring logs.

4.1 Surface Materials

Boring B-017-2-09 was drilled through the existing pavement of Mound Street and encountered 2.0 inches of asphalt overlying 9.0 inches of concrete followed by 3.0 inches of aggregate base. The remaining borings were drilled in the grass area between I-70 and Mound Street and encountered 2.0 to 6.0 inches of topsoil at the ground surface, as identified by the significant presence of vegetation and organic material.

4.2 Subsurface Soils

Beneath the surface materials in borings B-017-2-09, B-018-2-13, B-020-4-13, B-114-8-13 and B-114-9-13, material identified as existing fill was encountered extending to depths ranging from 5.5 to 14.5 feet below the existing ground surface, which corresponds to elevations ranging from 699.5 to 708.5 feet msl. The fill material consisted of dark brown, black and brown gravel with sand, gravel with sand and silt, sandy silt, silt and clay and silty clay (ODOT A-1-b, A-2-4, A-4a, A-6a, A-6b). The fill materials contained trash and debris, including organic material, root fibers, cinders, plastic, coal, brick and slag fragments throughout.

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

The relative density of granular soils is primarily derived from SPT blow counts (N_{60}). Based on the SPT blow counts obtained, the granular soil encountered ranged from very loose (N_{60} < 5 blows per foot [bpf]) to very dense (N_{60} > 50 bpf). Overall blow counts recorded from the SPT sampling ranged from 4 bpf to split spoon sampler refusal. The shear strength and consistency of the cohesive soils are primarily derived from the hand penetrometer values (HP). The cohesive soil encountered ranged from soft ($0.25 < HP \leq 0.5$ tsf) to hard (HP > 4.0 tsf). The unconfined compressive strength of the cohesive soil samples tested, obtained from the hand penetrometer, ranged from 0.5 to over 4.5 tsf (limit of instrument).

Natural moisture contents of the soil samples tested ranged from 3 to 30 percent. The natural moisture content of the cohesive soil samples tested for plasticity index ranged from 7 percent below to 10 percent above their corresponding plastic limits. In general, the soil exhibited natural moisture contents considered to be moderately below to significantly above optimum moisture levels.

4.3 Bedrock

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

Table 3. Top of Bedrock Elevations

Boring Number	Ground Surface Elevation (feet msl)	Top of Bedrock		Top of Bedrock Core	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-018-2-13	715.7	58.8	656.9	74.5	641.2
B-020-4-13	714.0	64.4	649.6	74.7	639.3
B-114-7-13	713.3	64.9	648.4	74.0	639.3
B-114-8-13	714.0	63.5	650.5	67.0	647.0
B-114-9-13	714.0	59.0	655.0	65.0	649.0

Top of bedrock was encountered in borings B-018-2-13, B-020-4-13, B-114-7-13, B-114-8-13 and B-114-9-13 at depths ranging from 58.8 to 64.9 feet below the existing ground surface, which corresponds to elevations ranging from 648.4 to 656.9 feet msl. The upper portion of the bedrock consists of weathered shale which was able to be augered to competent shale and claystone bedrock in borings B-020-4-13, B-114-8-13 and B-114-9-13 and competent limestone bedrock in borings B-018-2-13 and B-114-7-13. The cored bedrock consists of shale and claystone, which was encountered in borings B-020-4-13, B-114-8-13 and B-114-9-13 at an elevation of 639.3, 647.0 and 649.0 feet msl, respectively, overly limestone bedrock, which was encountered in all of the borings at elevations ranging from 629.0 fee msl in boring B-114-9-13 to 641.2 feet msl in boring B-018-2-13.

The claystone is described as dark gray, slightly weathered, very weak to weak, thinly laminated to thin bedded, calcareous and fractured to highly fractured with tight to open, slickensided to rough apertures. The shale is described as dark gray, slightly weathered, very weak to weak, thinly laminated to thin bedded, calcareous, argillaceous, fissile, pyritic and moderately to highly fractured with narrow to open, slickensided to slightly rough apertures. The limestone is described as gray and dark gray, unweathered to slightly weathered, moderately strong to very strong, thin to very thick bedded, calcareous, argillaceous, dolomitic, cherty, pyritic, fossiliferous and slightly to highly fractured with narrow to open, slightly rough to rough apertures.

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

Table 4. Rock Core Summary

Boring	Core No.	Elevation (feet msl)	Recovery (%)	RQD (%)	Unconfined Compressive Strength
B-018-2-13	RC-1	641.2 to 636.2	100	72	q_u @ 78.5’ = 10,153 psi
	RC-2	636.2 to 631.2	100	87	N/A
B-020-4-13	RC-1	639.3 to 634.3	92	75	N/A
	RC-2	634.3 to 629.3	100	100	N/A
	RC-3	629.3 to 624.3	95	90	q_u @ 86.5’ = 13,130 psi
	RC-4	624.3 to 619.3	95	85	q_u @ 90.7’ = 16,178 psi
B-114-7-13	RC-1	639.3 to 634.3	92	48	q_u @ 75.1’ = 8,488 psi
	RC-2	634.3 to 629.3	83	52	q_u @ 81.5’ = 9,141 psi
B-114-8-13	RC-1	647.0 to 642.0	82	19	N/A
	RC-2	642.0 to 637.0	97	12	q_u @ 74.8’ = 92 psi
	RC-3	637.0 to 632.0	98	92	q_u @ 78.4’ = 12,567 psi
B-114-9-13	RC-1	649.0 to 644.0	33	26	N/A
	RC-2	644.0 to 639.0	88	58	N/A
	RC-3	639.0 to 634.0	100	83	q_u @ 75.7’ = 338 psi [1]
	RC-4	634.0 to 629.0	93	90	N/A
	RC-5	629.0 to 624.0	93	89	N/A

1. Indicates unconfined compressive strength determined from point load testing.

It should be noted that bedrock experiences mechanical breaks during the drilling and coring processes. Rii attempted to account for fresh, manmade breaks during tabulation of the RQD analysis. The quality of the claystone and shale bedrock, according to the RQD values, ranged from very poor (RQD ≤ 25%) to excellent (90% < RQD ≤ 100%), and the quality of the limestone bedrock ranged from poor (25% < RQD ≤ 50%) excellent (90% < RQD ≤ 100%).

4.4 Groundwater

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

Table 5. Groundwater

Boring Number	Ground Elevation (feet msl)	Initial Groundwater		Upon Completion	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-017-2-09	713.0	16.5	696.5	N/A [1]	N/A
B-018-2-13	715.7	25.0	689.7	N/A [1]	N/A
B-020-4-13	714.0	18.5	695.5	N/A [1]	N/A
B-114-7-13	713.3	17.0	698.3	N/A [1]	N/A
B-114-8-13	714.0	18.5	695.5	N/A [1]	N/A
B-114-9-13	714.0	23.0	691.0	N/A [1]	N/A

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

Groundwater was encountered initially during drilling in all six borings at depths ranging from 16.5 to 25.0 feet below the existing ground surface, which corresponds to elevations ranging from 689.7 to 689.3 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 as well as water as a circulating fluid during the rock coring process. Please note that short-term water level readings, especially in cohesive soils, are not necessarily an accurate indication of the actual groundwater level. In addition, groundwater levels or the presence of groundwater are considered to be dependent on seasonal fluctuations in precipitation.

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

5.0 ANALYSES AND RECOMMENDATIONS

Data obtained from the subsurface exploration programs has been used to determine the foundation support capabilities and the settlement potential for the soil encountered at the site. These parameters have been used to provide guidelines for the design of foundation systems for the subject 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 E4 is proposed to be a MSE wall type that will support Ramp D7 between the FRA-70-1358L and FRA-70-1373B bridge structures. The wall will support the roadway along Ramp D7 between the FRA-70-1358L and FRA-70-1373B bridge structures, and will wrap around in front of the forward abutment of the proposed FRA-70-1358L bridge structure at the west end of the wall alignment and to the rear abutment of the FRA-70-1373B bridge structure at the east end of the wall alignment. The total wall length for Retaining Wall E4, including the portions of the wall that cross in front of the abutments of the proposed bridge structures, is approximately 1,007 lineal feet, and the total length between the abutments of the two bridges, from Sta. 402+02 to 409+52 (BL Wall E4), is approximately 750 feet. Based on the proposed plan and profile information, wall heights along the portion of the wall alignment that is considered for this exploration report will range from 9.2 feet at Sta. 405+20 (BL Wall E4) to 42.1 feet at Sta. 409+30 (BL Wall E4).

5.1 MSE Wall Recommendations

It is understood that a MSE retaining wall is being considered for use in supporting the proposed Ramp D7 alignment. MSE walls are constructed on earthen foundations at a minimum depth of 3.0 feet below grade, as defined by the top of the leveling pad to the ground surface located 4.0 feet from the face of the wall. Per Section 204.6.2.1 of the 2007 ODOT BDM, the height of the MSE wall is defined as the elevation difference between the top of coping and the top of the leveling pad. However, it is noted that the reinforced soil mass only extends from the foundation bearing elevation (top of leveling pad) to the roadway subgrade elevation. The width of the MSE wall foundation (B) is defined by the length of the reinforced soil mass. Per the Section 204.6.2.1 of the 2007 ODOT BDM and Supplemental Specification (SS) 840, the minimum length of the reinforced soil mass is equal to 70 percent of the height of the MSE wall or 8.0 feet whichever is greater. A non-structural bearing leveling pad consisting of a minimum of 6.0-inches of unreinforced concrete should be placed at the base of the wall facing for constructability purposes. Please note that the leveling pad is not a structural foundation.

Based on the proposed plan and profile information, the proposed retaining wall will have a maximum height of 27.7 feet, as measured from the top of the leveling pad to the top of the coping, at the east end of the alignment where it connects to the rear abutment of the FRA-70-1373B structure. The wall will step up along the proposed embankment as the wall continues west, with a minimum wall height of approximately 8.7 feet at Sta. 405+20 (BL Wall E4), and will step back down the proposed embankment to a maximum wall height of 42.1 feet where it connects to the forward abutment of the FRA-70-1358L structure. Fill heights up to approximately 30 feet fill will be required to bring the existing grade up to the proposed bottom of wall elevation along the wall alignment. 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. Material identified as existing fill or possible fill containing soft soils and organic matter was encountered at the proposed bearing elevation in borings B-017-2-09, B-018-2-13, B-020-4-13, B-114-8-13 and B-114-9-13. The unsuitable material extends to a depth of 13.5 feet below existing grade in boring B-017-2-09 (El. 699.5 feet msl), 14.3 feet below existing grade in boring B-018-2-13 (El. 701.4 feet msl), 8.0 feet below existing grade in boring B-020-4-13 (El. 706.0 feet msl), 5.5 feet below existing grade in boring B-114-8-13 (El. 708.5 feet msl) and 8.0 feet below existing grade in boring B-114-9-13 (El. 706.0 feet msl). The fill material consisted of loose to medium dense gravel with sand, gravel with sand and silt (ODOT A-1-b, A-2-4) and stiff to very stiff sandy silt, silt and clay and silty clay (ODOT A-4a, A-6a, A-6b), which contained trash and debris, including organic material, root fibers, cinders, plastic, coal, brick and slag fragments. In addition, stiff to hard silt and clay, silty clay, clay (ODOT A-6a, A-6b, A-7-6) and very loose to loose gravel with sand and silt and coarse and fine sand (ODOT A-2-6, A-3a) was encountered at the ground surface in boring B-114-7-13 and below the existing fill in the borings noted above, which extend to elevations ranging from 694.5 to 701.4 feet msl. Given the low blow counts obtained within the existing fill material and cohesive soil deposits, as well as the presence of organics and debris within the existing fill, these soils are not considered suitable for foundation support for a wall of this size. Consideration was given to over excavation and replacement of the existing fill material and underlying compressible soils, but given the depth of over excavation required, this option is not considered economically feasible.

A study was performed by GPD GROUP as part of the FRA-70-12.68 Project 4R (PID 105523), dated March 2, 2018, to investigate the use of ground improvement techniques (stone columns/rigid inclusions) as well as the use of lightweight fill consisting of cellular concrete to control settlement within the fill material and meet strength requirements. Analyses for both alternatives were provided in the report, as well as a cost comparison between the two alternatives. Based on the results of the study, it was understood that ground improvement techniques will be a cheaper option for that wall. It is anticipated that similar results would be obtained for this wall.

The ground improvement techniques, which will consist of stone columns or rigid inclusions, will increase the bearing resistance of the existing fill and cohesive soil deposits and also reduce settlement potential. The improved soils should also result in an increase of the shear strength within these soils, which in turn will improve the global stability of the wall/embankment system. It is recommended that the ground improvement elements be installed within the entire footprint of Wall E4 and the proposed embankment in areas outside of the existing I-70 embankment. The design of such a system is proprietary and beyond the scope of this investigation. Based on discussions with the ODOT Office of Geotechnical Engineering (OGE), the analysis for the wall was performed assuming that the existing fill and unsuitable soils will remain in place and not be stabilized. Additional considerations for the ground improvement design, including required performance criteria, are provided in Section 5.1.8.

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

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

5.1.1 Strength Parameters Utilized in External and Global Stability Analyses

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

Table 6. Shear Strength Parameters Utilized in Stability Analyses

Material Type	γ **(pcf)**	φ' [(1)] **(°)**	c' [(2)] **(psf)**	S_u [(3)] **(psf)**
MSE Wall Backfill (Select Granular Fill)	120	34	0	N/A
Item 203 Granular Embankment (Retained Soil and Proposed Embankment)	120	32	0	N/A
Ex. Fill: Stiff to Very Stiff Sandy Silt, Silt and Clay, Silty Clay (ODOT A-4a, A-6a, A-6b)	120	25 to 27	0	625 to 1,875
Ex. Fill: Loose to Medium Dense Gravel with Sand and Silt (ODOT A-2-4)	120	26	0	N/A
Loose to Medium Dense Granular Soils (ODOT A-1-a, A-1-b, A-2-4)	120 to 125	32 to 39	0	N/A
Dense to Very Dense Granular Soils (ODOT A-1-a, A-1-b)	130 to 135	39 to 43	0	N/A
Very Stiff Silt and Clay (ODOT A-6a)	125	27	0	4,500
Hard Silty Clay (ODOT A-6b)	125 to 130	26 to 28	50 to 100	4,375 to 8,000
Very Stiff to Hard Clay (ODOT A-7-6)	120 to 130	25 to 26	0 to 100	1,750 to 8,000

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

Shear strength parameters for the reinforced soil backfill are provided in ODOT SS 840. Per SS 840, the select granular backfill in the reinforced zone must meet the shear strength requirements provided in Table 6. Based on the design plans provided by the Rii design team and ms consultants, it is understood that Item 203 granular embankment will be utilized where any new embankment will be placed below the wall and behind the reinforced soil backfill along the wall alignment. Therefore, the shear strength parameters for the retained fill will be modeled using a friction angle of 32 degrees since granular embankment is being specified, instead of using the shear strength parameters provided in ODOT SS 840.

The shear strength parameters for the natural soils were assigned using correlations provided in FHWA Geotechnical Engineering Circular (GEC) No. 5 (FHWA-NHI-16-072) Evaluation of Soil and Rock Properties and based on past experience in the vicinity of the site with projects performed in similar subsurface profiles. However, the friction angle for the existing fill that consisted of medium dense gravel with sand and silt was conservatively assigned since there no records of the material origin or how it was placed.

5.1.2 Bearing Stability

Material identified as existing fill or possible fill consisting of loose to medium dense gravel with sand, gravel with sand and silt (ODOT A-1-b, A-2-4) and stiff to very stiff sandy silt, silt and clay and silty clay (ODOT A-4a, A-6a, A-6b), which contained trash and debris, including organic material, root fibers, cinders, plastic, coal, brick and slag fragments, was encountered at the existing grade within the area outside of the existing I-70 embankment. These unsuitable soils extend to a depths ranging from 5.5 to 14.3 feet below the existing ground surface grade (El. 699.5 to 706.0 feet msl). As noted in Section 5.1, it is understood that ground improvement techniques will be implemented within the entire footprint of Wall E4 and the proposed embankment in areas outside of the existing I-70 embankment. As this is a proprietary design, the analysis for this wall considers the existing fill material will remain in place and will not be stabilized. MSE wall foundations bearing on existing fill material or granular embankment, 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.

Table 7. Retaining Wall E4 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]	
402+02	404+75	27.7	Level	19.4 (0.70H)	3.47	2.26	6.63
404+75	407+00	15.1	Level	10.6 (0.70H ≥ 8.0)	13.76	5.19 (5)	4.01
407+00	409+50	42.1	Level	29.5 (0.70H)	5.41	3.52	9.64

1. *Stationing referenced to the baseline of Retaining Wall E4.*
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.*
5. *The factored bearing resistance includes a reduction factor applied to the nominal resistance to account for the fore slope in front of the wall per Section 10.6.3.1.2c of the 2018 AASHTO LRFD BDM.*

For analysis of the wall section between Sta. 404+75 and 407+00, the calculated factored bearing resistance includes a reduction factor applied to the nominal resistance to account for the fore slope in front of the wall per Section 10.6.3.1.2c of the 2018 AASHTO LRFD BDM.

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

5.1.3 Settlement Evaluation

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

Table 8. Compressibility Parameters Utilized in Settlement Analysis

Material Type	γ (pcf)	LL (%)	C_c (1)	C_r (2)	e_o (3)	C_v (4) (ft^2/yr)	N_{60}	C' (5)
Ex. Fill: Stiff to Hard Sandy Silt (ODOT A-4a)	115 to 120	21 to 32	0.099 to 0.198	0.015 to 0.030	0.436 to 0.522	1,000	N/A	N/A
Ex. Fill: Stiff Silt and Clay (ODOT A-6a)	115	32	0.198	0.030	0.522	600	N/A	N/A
Ex. Fill: Stiff to Very Stiff Silty Clay (ODOT A-6b)	120	40	0.270	0.041	0.585	300	N/A	N/A
Ex. Fill: Very Loose to Medium Dense Granular Soils (ODOT A-1-b, A-2-4)	120	N/A	N/A	N/A	N/A	N/A	13 to 18	73 to 117
Very Loose to Medium Dense Granular Soils (ODOT A-1-a, A-1-b, A-2-6, A-3a)	115 to 125	N/A	N/A	N/A	N/A	N/A	5 to 30	47 to 113
Dense to Very Dense Granular Soils (ODOT A-1-a, A-1-b, A-3a)	130 to 135	N/A	N/A	N/A	N/A	N/A	34 to 100	121 to 379
Hard Sandy Silt (ODOT A-4a)	130	25	0.135	0.014	0.467	1,000	N/A	N/A
Soft to Very Stiff Silt and Clay (ODOT A-6a)	125	33 to 35	0.207 to 0.225	0.031 to 0.023	0.530 to 0.546	600	N/A	N/A
Stiff to Hard Silty Clay (ODOT A-6b)	115 to 130	31 to 40	0.189 to 0.270	0.019 to 0.038	0.514 to 0.585	300	N/A	N/A
Very Stiff to Hard Clay (ODOT A-7-6)	125 to 130	41 to 48	0.279 to 0.342	0.014 to 0.034	0.593 to 0.647	150	N/A	N/A

1. *Per Table 6-9, Section 6.14.1 of FHWA GEC 5.*
2. *Estimated at 10% of C_c for natural soils and 15% C_c for existing fill per Section 8.11 of Holtz and Kovacs (1981).*
3. *Per Table 8-2 of Holtz and Kovacs (1981).*
4. *Per Figure 6-37, Section 6.14.2 of FHWA GEC 5.*
5. *Per Figure 10.6.2.4.2-1 of 2018 AASHTO LRFD BDS.*

The settlement analysis for the section of the wall alignment where the wall steps up the proposed embankment fill was performed considering the equivalent bearing pressure at the bottom of the MSE wall at the service limit state as well as the weight of the embankment fill below the proposed bottom of wall elevation. Results of the settlement analysis are tabulated in Table 9. Total settlements ranging from 7.70 to 15.59 inches at the center of the reinforced soil mass and 4.76 to 8.38 inches at the facing of the wall are anticipated along the alignment of Retaining Wall E4. Based on the results of the analysis, 90 percent of the total settlement is anticipated to occur over the time periods shown in Table 9. 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 E4 MSE Wall Settlement Values

From Station [1]	To Station [1]	Service Limit Bearing Pressure [2] (ksf)	Total Settlement Values (inches)		Time for 90% Consolidation (Days)
			Center of Wall Mass	Facing of Wall	
402+02	404+75	4.64 to 5.05	7.70 to 10.46	4.76 to 5.18	8 to 23
404+75	407+00	5.21 to 5.59	15.45 to 15.59	8.00 to 8.38	90 to 100
407+00	409+50	6.27 to 6.80	9.06 to 12.07	5.79 to 7.60	55 to 100

1. *Stationing referenced to the baseline of Retaining Wall E4.*
2. *The service limit bearing pressure includes the equivalent bearing pressure from the MSE wall at the service limit state as well as the weight of the embankment fill below the proposed bottom of wall elevation.*

Per Section 204.6.2.1 of the 2019 ODOT BDM, “the maximum allowable differential settlement in the longitudinal direction (regardless of the size of panels) is one (1) percent.” Given the amount of settlement anticipated at the facing along the wall alignment, as well as the presence of existing fill material that may vary significantly over the footprint of the wall and proposed embankment fill, differential settlement greater than 1/100 may occur if the fill material is not stabilized or over excavated and replaced with embankment fill. If either the total or differential settlement values predicted present an issue with respect to the deformation tolerances that the wall can withstand, then measures should be taken to minimize the amount of settlement that will occur. This can be achieved by preloading the site and consolidating the underlying soils prior to constructing the walls. If preloading the site is not a desired option, then consideration could be given to ground improvement through the use of stone columns. Settlement calculations are provided in Appendix V.

5.1.4 Eccentricity (Overturning Stability)

The resistance of the MSE wall to overturning will be dependent on the 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 heights indicated in Table 7. Based on the minimum length of reinforced soil mass presented in Table 7 and utilizing the soil parameters listed in Section 5.1.1 for the retained embankment material, the calculated eccentricity of the resultant force **will not exceed** the limiting eccentricity at the strength limit state.

5.1.5 Sliding Stability

The resistance of the MSE wall to sliding was evaluated per Section 11.10.5.3 of the 2018 AASHTO LRFD BDS. For drained conditions, the sliding resistance is determined by multiplying a coefficient of sliding friction "f" times the total vertical force at the base of the wall. The coefficient of sliding friction is determined based on the limiting friction angle between the foundation soil and the reinforced soil backfill. Based on the soil parameters listed in Section 5.1.1 for the foundation and reinforced soil backfill, a coefficient of sliding friction of 0.47 to 0.62 was utilized for design. For undrained conditions, the sliding resistance is taken as the limiting value between the undrained shear strength of the bearing soil and half of the vertical stress applied by the wall multiplied by the width of the MSE wall. Based on the soil parameters listed in Section 5.1.1, the undrained shear strength of the existing fill material is estimated to be 625 to 1,000 psf.

A geotechnical resistance factor of φ_τ=1.0 was considered in calculating the factored shear resistance between the reinforced backfill material and foundation 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 for drained conditions, but **will exceed** the factored shear resistance at the strength limit state for undrained conditions.

5.1.6 Overall (Global) Stability

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

Per Section 11.6.2.3 of the 2018 AASHTO LRFD BDS, overall (global) stability for MSE walls that are not integrated with or supporting structural foundations or elements, global stability is satisfied if the product of the factor of safety from the slope stability output multiplied by the resistance factor φ=0.75 is greater than 1.0. Therefore, global stability is satisfied when a minimum factor of safety of 1.3 is obtained. For an MSE wall designed with the minimum strap lengths listed in Table 7, the resulting factor of safety under drained conditions (long-term stability) and undrained conditions (short-term stability) was less than 1.3.

5.1.7 Final MSE Wall Considerations

Based on the results of the external and global stability analysis performed for the MSE wall, sliding under undrained conditions as well as bearing and global stability under both drained and undrained conditions were not satisfied at a strap length equal to 0.7 times the wall height. Increasing the width of the wall up to 1.3 times the wall height still did not satisfy all of the external and global stability requirements. Calculations for external (bearing and sliding resistance and limiting eccentricity) and overall (global) stability of the MSE wall are provided in Appendix V.

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

5.1.8 Ground Improvement Considerations

The design of the ground improvement should result in the improved soil matrix meeting the design criteria for bearing resistance and compressibility for the MSE wall. The improved soil matrix will need to provide a factored bearing resistance greater than or equal to the factored bearing pressure at the strength limit state of 6.63 to 9.64 ksf. Additionally, the improved soil matrix will need to limit settlement to the required maximum differential settlement of 1/100 along the wall facing and to tolerable limits for maximum settlement of the wall based on the wall manufacturer’s specifications or for constructability of the roadway. In the absence of specific settlement from the wall manufacturer, the ground improvement design should limit total settlement of the embankment and back of the reinforced soil mass to 5.0 inches, and total settlement at the facing of the wall to 2.5 inches.

As noted above, total settlements of approximately 7.7 to 15.5 inches at the center of the reinforced soil mass and 4.75 to 8.4 inches at the facing of the wall are anticipated along the alignment of Retaining Wall E4, based on service limit bearing pressures ranging from 4.64 to 6.80 ksf, without stabilization of the existing fill and unsuitable soils. About 70 to 90 percent of the estimated settlement is occurring within these upper layers. Therefore, it is recommended that the ground improvement elements be extended through the existing fill layers and any underlying compressive (cohesive) layers. Based on the conditions encountered, the ground improvement elements should be extended to an approximate elevation of 695 to 700 feet msl. Additionally, it is recommended that ground improvement elements be located along the length of the leveling pad, where the leveling pad is bearing on new fill with a thickness less than two (2) times the width of the pad, if concentrated loads will be imparted along the pad to ensure that differential settlement does not occur.

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	1,000	0°	N/A	N/A	N/A
Very Stiff to Hard Cohesive Soil	125	3,000	0°	N/A	N/A	N/A
Very Loose to Loose Granular Soil	120	0	28°	0.32	0.53	5.07
Medium Dense Granular Soil	125	0	32°	0.27	0.47	6.82
Dense to Very Dense Granular Soil	130	0	36°	0.23	0.41	9.09
Compacted Cohesive Engineered Fill	120	2,000	0°	N/A	N/A	N/A
Compacted Granular Engineered Fill	120	0	32°	0.27	0.47	6.82

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

Table 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
Very Loose to Loose Granular Soil	120	0	28°	0.32	0.53	5.07
Medium Dense Granular Soil	125	0	32°	0.27	0.47	6.82
Dense to Very Dense Granular Soil	130	0	36°	0.23	0.41	9.09
Compacted Cohesive Engineered Fill	120	0	30°	0.30	0.50	5.58
Compacted Granular Engineered Fill	120	0	32°	0.27	0.47	6.82

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

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

5.3 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, little to no groundwater seepage is anticipated during construction. However, where/if groundwater is encountered, proper groundwater control should be employed and maintained to prevent disturbance to excavation bottoms consisting of cohesive soil, and to prevent the possible development of a quick or "boiling" condition where soft silts and/or fine sands are encountered. It is preferable that the groundwater level, if encountered, be maintained at least 36 inches below the deepest excavation. Any seepage or groundwater encountered at this site should be able to be controlled by pumping from temporary sumps. Additional measures may be required depending on seasonal fluctuations of the groundwater level. Note that determining and maintaining actual groundwater levels during construction is the responsibility of the contractor.

6.0 LIMITATIONS OF STUDY

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

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

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

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

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

APPENDIX I

VICINITY MAP AND BORING PLAN

BORING B-018-2-13 WAS PERFORMED FOR THE FRA-70-1358L STRUCTURE.

BORINGS B-018-4-13, B-018-5-13 AND B-019-2-13 WERE PERFORMED FOR WALL E3.

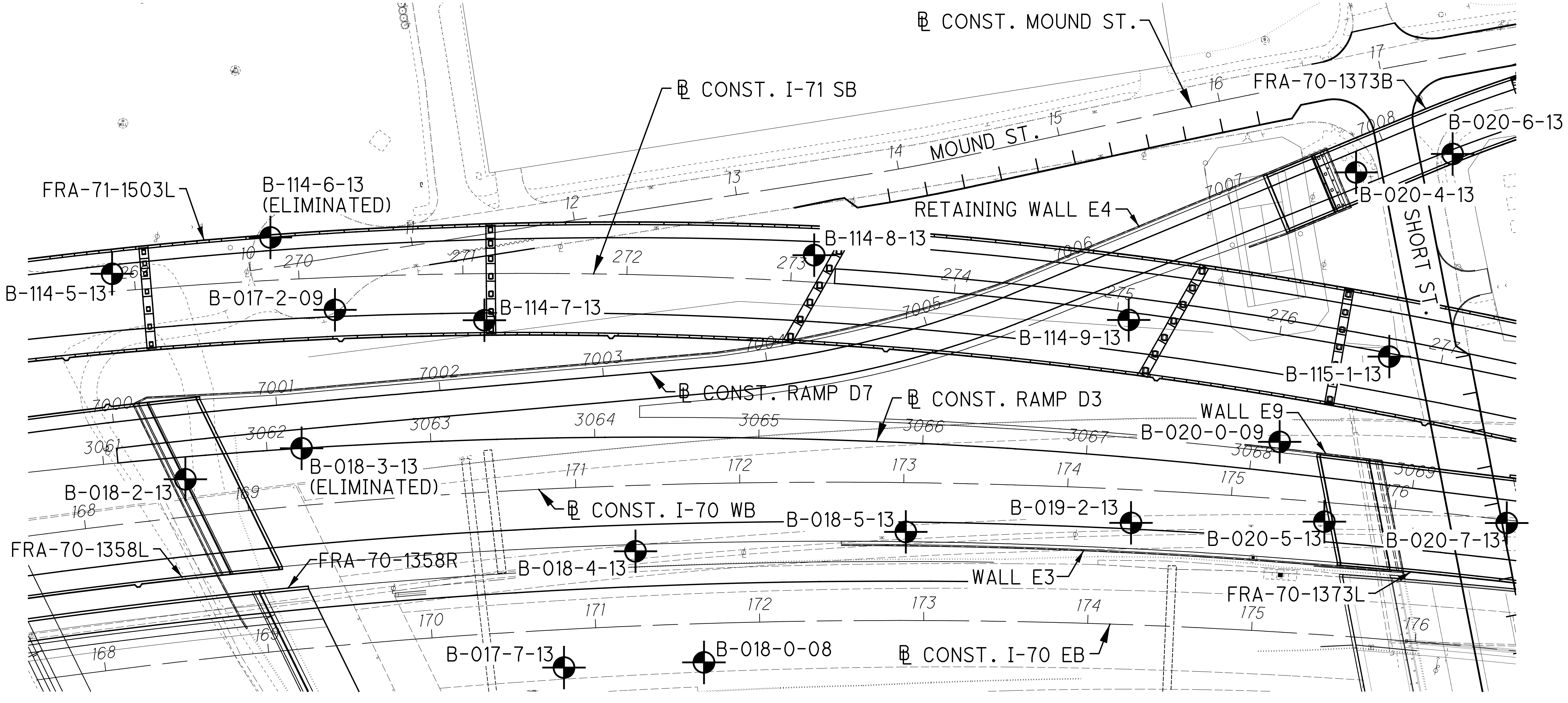
BORINGS B-020-4-13 AND B-020-6-13 WERE PERFORMED FOR THE FRA-70-1373B STRUCTURE.

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

BORINGS B-114-5-13, B-114-7-13, B-114-8-13, B-114-9-13 AND B-115-1-13 WERE PERFORMED FOR THE FRA-71-1503L STRUCTURE.

BORING B-017-7-13 WAS PERFORMED AS PART OF THE FRA-70-12.68 PROJECT 4A/4R.

BORINGS B-017-2-09, B-018-0-08 AND B-020-0-08 WERE DRILLED AS PART OF THE FRA-70-8.93 PRELIMINARY EXPLORATION.



BORING PLAN

FRA-71-14.36 - RETAINING WALL E4

FRANKLIN COUNTY, OHIO

PROJECT NO.
Rii W-13-072

SCALE: 1"=60'
0 30 60

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 x 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	⁂ Uncontrolled Fill (Describe)	▪ Bouldery Zone	● Peat, S-Sedimentary W-Woody F-Fibrous L-Loamy & etc
☒ Pavement or Base			

* 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-017-2-09, B-018-2-13, B-020-4-13 and B-114-7-13 through B-114-9-13

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

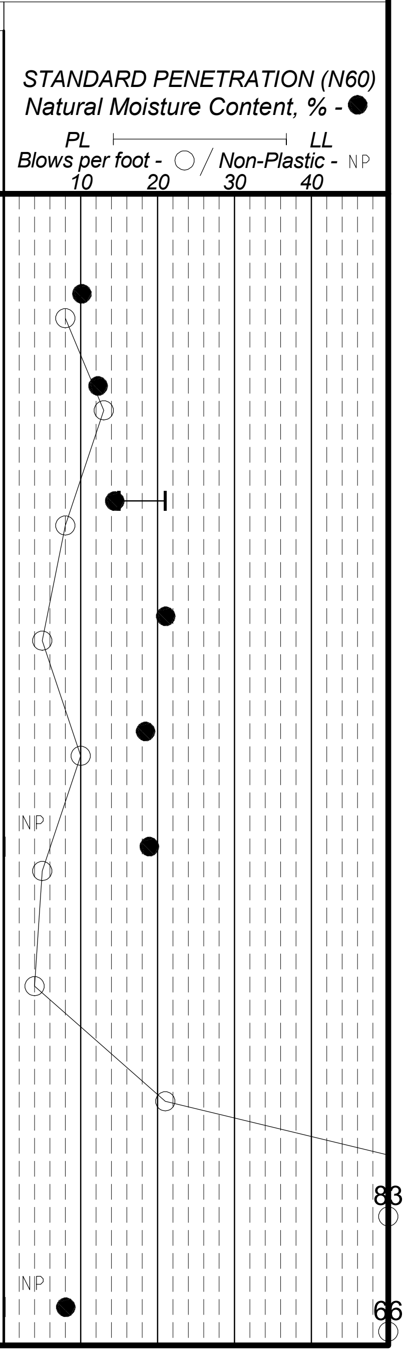
WC = Water content (%)

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

LOG OF: Boring B-017-2-09 | *Location:* Sta. 169+62.3, 117.3' LT., BL I-70 WB | *Date Drilled:* 9/23/2009

WATER OBSERVATIONS:
Water seepage at: 16.5'
Water level at completion: 17.0' (prior to adding water)
45.5' (includes drilling water)
FIELD NOTES:

Depth (ft)	Elev. (ft) 713.0	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	Graphic Log	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
1.2	711.8						Asphalt Concrete - 2" Portland Cement Concrete - 9" Aggregate Base - 3"							
3.5	709.5	3 4 4	10	1		1.0	FILL: Stiff brown SANDY SILT (A-4a), little gravel; moist.							
5 6.0	707.0	3 6 7	5	2		-	FILL: Medium dense brown GRAVEL WITH SAND (A-1-b), some fine to coarse sand, trace to little silt; damp.							
		4 3 5	13	3		2.0	FILL: Stiff brown SANDY SILT (A-4a), trace to little gravel; damp to moist. @ 6.0'-7.5', contains few brick fragments.		10	12	---	19	40	19
10		3 2 3	13	4		1.0								
13.5	699.5	2 3 7	13	5		1.0	@ 11.0'-12.5', little to some gravel;							
15		3 3 2	12	6			Loose brown COARSE TO FINE SAND (A-3a), some silt, little gravel; moist to wet.		15	17	---	42	--26--	
18.5	694.5	1 2 2	18	7										
20		7 9 11	18	8			Medium dense brown GRAVEL WITH SAND, SILT, AND CLAY (A-2-6); wet.							
23.5	689.5	18 36 44	14	9			@ 21.0'-22.5', very dense.							
25	688.0	16 24 40	18	10			Very dense brown GRAVEL (A-1-a), some fine to coarse sand, trace silt; wet.		65	18	---	8	--9--	



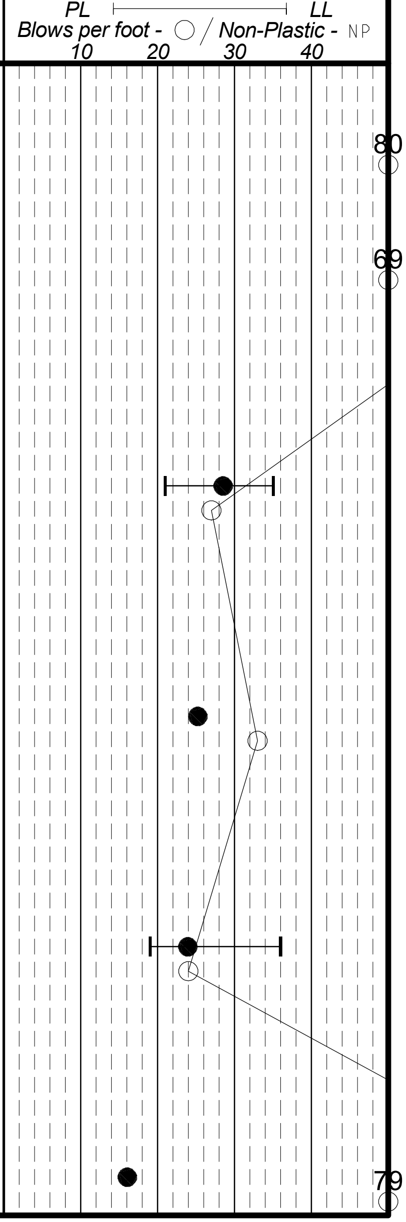
Client: ms consultants | *Project:* FRA-70-8.93 | *Job No.* 0221-1004.01

LOG OF: Boring B-017-2-09 | *Location:* Sta. 169+62.3, 117.3' LT., BL I-70 WB | *Date Drilled:* 9/23/2009

WATER OBSERVATIONS:
Water seepage at: *16.5'*
Water level at completion: *17.0' (prior to adding water)*
45.5' (includes drilling water)
FIELD NOTES:

Depth (ft)	Elev. (ft) 688.0	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	Graphic Log	GRADATION % Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
							Very dense brown GRAVEL (A-1-a), some fine to coarse sand, trace silt; wet.							
		38 44 33	11	11										
30		19 34 33	11	12										
32.0	681.0						Soft to medium stiff gray SILT AND CLAY (A-6a), trace to little fine to coarse sand, trace gravel; moist.							
35		8 12 14	12	13		0.5			7	1	---	1	41	50
37.0	676.0						Very stiff gray SILTY CLAY (A-6b); trace to little fine to coarse sand; damp to moist.							
40		11 8 24	14	14		2.75								
45		8 9 14	18	15		2.75			9	7	---	6	34	44
							@ 48.5'-50.0', possible decomposed shale.							
50	663.0	24 36 40	18	16		3.75								

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



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

LOG OF: Boring B-017-2-09 | *Location:* Sta. 169+62.3, 117.3' LT., BL I-70 WB | *Date Drilled:* 9/23/2009

WATER OBSERVATIONS:
Water seepage at: 16.5'
Water level at completion: 17.0' (prior to adding water)
45.5' (includes drilling water)
FIELD NOTES:

Depth (ft)	Elev. (ft) 663.0	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	Graphic Log	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N60) Natural Moisture Content, % - ● PL ├──┤ LL Blows per foot - ○ / Non-Plastic - NP 10 20 30 40
							Very stiff gray SILTY CLAY (A-6b); trace to little fine to coarse sand; damp to moist.								
54.8	658.2	22 39 50/4	16	17		2.5	@ 53.5'-54.8', possible decomposed shale.								● 50+ ○
55							Bottom of Boring - 54.8'								
60															
65															
70															
75															

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-13.10 - PHASE 6A
TYPE: STRUCTURE
PID: 89464 BR ID: FRA-70-1358L
START: 2/10/14 END: 2/13/14

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

DRILL RIG: CME-750 (SN 98048)
HAMMER: CME AUTOMATIC
CALIBRATION DATE: 4/26/13
ENERGY RATIO (%): 82.6

STATION / OFFSET: 168+63.72 / 22.9' LT
ALIGNMENT: BL I-70 WB
ELEVATION: 715.7 (MSL) EOB: 84.5 ft.
LAT / LONG: 39.953512, -83.007251

EXPLORATION ID
B-018-2-13

PAGE
1 OF 3

MATERIAL DESCRIPTION AND NOTES	ELEV. 715.7	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
0.5' - TOPSOIL (6.0")	715.2																	
FILL: LOOSE TO MEDIUM DENSE, BROWN TO BLACK **GRAVEL WITH SAND AND SILT**, LITTLE CLAY, MOIST TO WET. -TRASH, DEBRIS, PLASTIC, CINDERS AND ROOT FIBERS PRESENT THROUGHOUT			6 4 3	10	67	SS-1	-	-	-	-	-	-	-	-	-	12	A-2-4 (V)	
	710.2		3 5 8	18	72	SS-2	-	26	25	17	22	10	NP	NP	NP	28	A-2-4 (0)	
FILL: STIFF TO VERY STIFF, BROWN **SILTY CLAY**, SOME COARSE TO FINE SAND, LITTLE FINE GRAVEL, MOIST. -TRACE CINDERS AND BRICK FRAGMENTS PRESENT THROUGHOUT			7 7 8	21	83	SS-3	2.25	-	-	-	-	-	-	-	-	26	A-6b (V)	
	705.2		4 4 3	10	61	SS-4	1.75	-	-	-	-	-	-	-	-	23	A-6b (V)	
FILL: STIFF TO VERY STIFF, BROWN TO DARK BROWN **SANDY SILT**, SOME CLAY, TRACE FINE GRAVEL, DAMP. -TRACE CINDERS PRESENT IN SS-5			6 6 8	19	78	SS-5	2.75	9	16	8	39	28	28	19	9	16	A-4a (6)	
	701.4		3 3 5	11	11	SS-6	1.50 -	- -	- -	- -	- -	- -	- -	- -	- -	18 11	A-4a (V) A-1-b (V)	
MEDIUM DENSE TO DENSE, BROWN **GRAVEL WITH SAND**, TRACE CLAY, TRACE SILT, DAMP TO MOIST. -TRACE ROCK FRAGMENTS PRESENT IN SS-6B			5 9 7	22	72	SS-7	-	42	37	13	1	7	NP	NP	NP	9	A-1-b (0)	
	695.2		7 9 19	39	78	SS-8	-	-	-	-	-	-	-	-	-	8	A-1-b (V)	
DENSE TO VERY DENSE, GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, DAMP TO MOIST.			21 25 28	73	56	SS-9	-	-	-	-	-	-	-	-	-	6	A-1-a (V)	
		W	15 20 23	59	83	SS-10	-	54	29	6	6	5	NP	NP	NP	8	A-1-a (0)	
-IRON STAINING PRESENT IN SS-11			25 21 15	50	78	SS-11	-	-	-	-	-	-	-	-	-	9	A-1-a (V)	
-LIMESTONE FRAGMENTS PRESENT THROUGHOUT			23 20 23	59	83	SS-12	-	-	-	-	-	-	-	-	-	9	A-1-a (V)	

PID: 89464	BR ID: FRA-70-1358L	PROJECT: FRA-70-13.10 - PHASE 6A	STATION / OFFSET: 168+63.72 / 22.9 LT	START: 2/10/14	END: 2/13/14	PG 2 OF 3	B-018-2-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 685.7	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			
DENSE TO VERY DENSE, GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, DAMP TO MOIST. *(same as above)* -COBBLES PRESENT @ 31.0'		31 - 33																
		34 - 35	48 15 18	45	56	SS-13	-	-	-	-	-	-	-	-	-	8	A-1-a (V)	
-HEAVING SAND ENCOUNTERED @ 35.0' -INTRODUCED MUD @ 35.0'	678.7	35 - 37																
VERY STIFF TO HARD, GRAY **CLAY**, SOME SILT, TRACE FINE GRAVEL, TRACE COARSE SAND, DRY TO DAMP.		37 - 38																
		39 - 40	7 9 18	37	39	SS-14	3.50	-	-	-	-	-	-	-	-	23	A-7-6 (V)	
		41 - 43																
		44 - 45	5 8 11	26	78	SS-15	3.50	-	-	-	-	-	-	-	-	25	A-7-6 (V)	
		46 - 48																
		49 - 50	8 11 13	33	72	SS-16	4.00	1	1	0	20	78	45	26	19	22	A-7-6 (13)	
		51 - 53																
		54 - 55	8 11 15	36	83	SS-17	4.50	-	-	-	-	-	-	-	-	24	A-7-6 (V)	
	656.9	56 - 58																
SHALE : GRAY, HIGHLY WEATHERED, VERY WEAK.		TR 59	50/3"	-	100	SS-18	4.5+	-	-	-	-	-	-	-	-	11	A-7-6 (V)	
		60 - 61																

2014 ODOT BORING LOG-RII NE BRIDGE ID - OH DOT.GDT - 7/12/19 12:57 - U:\GI8\PROJECTS\2013\W-13-072.GPJ

PID: 89464	BR ID: FRA-70-1358L	PROJECT: FRA-70-13.10 - PHASE 6A	STATION / OFFSET: 168+63.72 / 22.9 LT	START: 2/10/14	END: 2/13/14	PG 3 OF 3	B-018-2-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 653.6	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
SHALE : GRAY, HIGHLY WEATHERED, VERY WEAK. *(same as above)*		63																
		64	12 50/5"	-	91	SS-19	-	-	-	-	-	-	-	-	-	11	Rock (V)	
		65 66 67 68																
		69	25 50/3"	-	100	SS-20	-	-	-	-	-	-	-	-	-	11	Rock (V)	
		70 71 72 73																
AUGER REFUSAL @ 74.5'	641.2	74	30 50/2"	-	100	SS-21	-	-	-	-	-	-	-	-	-	13	Rock (V)	
LIMESTONE : GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, STRONG, VERY THICK BEDDED, CALCAREOUS, ARGILLACEOUS, FOSSILIFEROUS, CHERTY, DOLOMITIC, JOINTED, SLIGHTLY TO HIGHLY FRACTURED, OPEN APERTURES, SLIGHTLY ROUGH; RQD 80%, REC 100%. -0.1' TO 0.2' CLAY SEAMS PRESENT FROM 74.5' TO 75.5' -QU @ 78.5' = 10,153 PSI		75 76 77 78 79	72		100	RC-1											CORE	
	631.2	80 81 82 83 84 EOB	87		100	RC-2											CORE	

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 25.0'

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PUMPED 188 LBS CEMENT / 50 LBS BENTONITE POWDER / 40 GAL WATER; COMPACTED WITH THE AUGER 300 LB. BENTONITE CHIPS AND SOIL CUTTINGS

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-13.10 - PHASE 6A
TYPE: STRUCTURE
PID: 89464 BR ID: FRA-70-1373B
START: 3/5/14 END: 3/11/14

DRILLING FIRM / OPERATOR: RII / J.B./T.F.
SAMPLING FIRM / LOGGER: RII / S.B./S.M.
DRILLING METHOD: 3.25" HSA / HQ
SAMPLING METHOD: SPT / RC

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

STATION / OFFSET: 7007+77.61 / 12.0' RT
ALIGNMENT: BL RAMP D7
ELEVATION: 714.0 (MSL) EOB: 94.7 ft.
LAT / LONG: 39.954037, -83.004709

EXPLORATION ID
B-020-4-13

PAGE
1 OF 4

MATERIAL DESCRIPTION AND NOTES	ELEV. 714.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
0.2' - TOPSOIL (2.0")	713.8																	
FILL: STIFF, DARK BROWN **SILT AND CLAY**, SOME COARSE TO FINE SAND, LITTLE FINE GRAVEL, MOIST.		1 2 3	3 2 1	4	44	SS-1	1.25	-	-	-	-	-	-	-	-	25	A-6a (V)	
-TRACE ORGANICS AND WOOD FIBERS PRESENT IN SS-2		4 5	1 1 2	4	72	SS-2	1.25	14	14	16	31	25	32	18	14	28	A-6a (6)	
	706.0	6 7 8	3 3 3	8	72	SS-3	2.50	-	-	-	-	-	-	-	-	25	A-6a (V)	
VERY STIFF TO HARD, BROWN **CLAY**, AND SILT, TRACE FINE SAND, TRACE FINE GRAVEL, MOIST.		9 10	3 5 6	14	72	SS-4	3.50	1	0	5	53	41	48	20	28	29	A-7-6 (17)	
	701.0	11 12 13	4 5 6	14	100	SS-5	3.75	-	-	-	-	-	-	-	-	23	A-7-6 (V)	
LOOSE, BROWN **GRAVEL WITH SAND AND SILT**, TRACE CLAY, MOIST TO WET.		14 15	WOH 2 2	5	56	SS-6	-	-	-	-	-	-	-	-	-	20	A-2-4 (V)	
	696.0	W 16 17 18	2 2 5	9	44	SS-7	-	33	17	16	25	9	NP	NP	NP	17	A-2-4 (0)	
MEDIUM DENSE TO DENSE, BROWN TO GRAY **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, MOIST TO WET.		W 19 20	4 6 11	22	33	SS-8	-	-	-	-	-	-	-	-	-	16	A-1-b (V)	
-INTRODUCED MUD @ 21.0'		21 22 23	6 12 15	35	100	SS-9	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
-COBBLES PRESENT THROUGHOUT		24 25	8 12 18	39	78	SS-10	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
		26 27 28	21 18 12	39	33	SS-11	-	61	15	8	10	6	NP	NP	NP	8	A-1-b (0)	
		29	4 4 5	12	33	SS-12	-	-	-	-	-	-	-	-	-	22	A-1-b (V)	

PID: 89464 | BR ID: FRA-70-1373B | PROJECT: FRA-70-13.10 - PHASE 6A | STATION / OFFSET: 7007+77.61 / 12.0 RT | START: 3/5/14 | END: 3/11/14 | PG 2 OF 4 | B-020-4-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 684.0	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			
MEDIUM DENSE TO DENSE, BROWN TO GRAY **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, MOIST TO WET. *(same as above)*	682.0	31 32																
MEDIUM DENSE, GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST.		33 34 35	1 1 17	23	44	SS-13	-	62	22	5	7	4	NP	NP	NP	11	A-1-a (0)	
	677.0	36 37																
VERY DENSE, GRAY **GRAVEL**, LITTLE COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST.		38 39 40	8 20 25	58	67	SS-14	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	
		41 42 43 44 45	18 26 27	69	83	SS-15	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	
		46 47 48 49 50	8 31 50	105	72	SS-16	-	83	10	3	3	1	NP	NP	NP	9	A-1-a (0)	
		51 52 53 54	50 50/3"	-	100	SS-17	-	-	-	-	-	-	-	-	-	12	A-1-a (V)	
-AUGER REFUSAL ON BOULDER @ 55.5'; SWITCHED TO MUD ROTARY DRILLING WITH CASING ADVANCER.	657.0	55 56 57																
HARD, GRAY **SILTY CLAY**, TRACE COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP.		58 59 60	10 27 35	80	42	SS-18	4.5+	7	4	4	41	44	40	19	21	14	A-6b (12)	
		61																

PID: 89464	BR ID: FRA-70-1373B	PROJECT: FRA-70-13.10 - PHASE 6A	STATION / OFFSET: 7007+77.61 / 12.0 RT	START: 3/5/14	END: 3/11/14	PG 3 OF 4	B-020-4-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 651.9	DEPTHS	SPT/ RQD	N60	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
HARD, GRAY **SILTY CLAY**, TRACE COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP. *(same as above)*	649.6	63 64 TR	31 50/5"	-	91	SS-19	4.5+	-	-	-	-	-	-	-	-	16	A-6b (V)	
SHALE : GRAY, HIGHLY WEATHERED, VERY WEAK.		65 66 67 68 69	50/5"	-	100	SS-20	-	-	-	-	-	-	-	-	-	19	Rock (V)	
	639.3	70 71 72 73 74	50/5"	-	20	SS-21	-	-	-	-	-	-	-	-	-	23	Rock (V)	
SHALE : DARK GRAY, SLIGHTLY WEATHERED, WEAK, VERY THIN TO THIN BEDDED, ARGILLACEOUS, MODERATELY FRACTURED, NARROW APERTURES, SLICKENSIDED TO SLIGHTLY ROUGH; RQD 66%, REC 89%.	635.6	75 76 77 78 79	75		92	RC-1											CORE	
CLAYSTONE : DARK GRAY, SLIGHTLY WEATHERED, VERY WEAK, THIN BEDDED, CALCAREOUS, FRACTURED, TIGHT APERTURES, SLICKENSIDED TO SLIGHTLY ROUGH; RQD 100%, REC 100%. -SHALE SEAM PRESENT FROM 79.7' TO 80.7' -PYRITIC FROM 80.7' TO 83.7'	630.3	80 81 82 83 84	100		100	RC-2											CORE	
LIMESTONE : DARK BROWNISH GRAY, UNWEATHERED, MODERATELY STRONG TO STRONG, THIN TO MEDIUM BEDDED, CHERTY, DOLOMITIC, MODERATELY TO SLIGHTLY FRACTURED, NARROW TO OPEN APERTURES, SLIGHTLY ROUGH; RQD 87%, REC 95%. -QU @ 86.5' = 13,130 PSI		85 86 87 88 89	90		95	RC-3											CORE	
-QU @ 90.7' = 16,178 PSI		90 91 92 93 94	85		95	RC-4											CORE	

2014 ODOT BORING LOG-RII NE BRIDGE ID - OH DOT.GDT - 7/12/19 12:58 - U:\GI8\PROJECTS\2013\W-13-072.GPJ

PID: 89464 | BR ID: FRA-70-1373B | PROJECT: FRA-70-13.10 - PHASE 6A | STATION / OFFSET: 7007+77.61 / 12.0 RT | START: 3/5/14 | END: 3/11/14 | PG 4 OF 4 | B-020-4-13

<table>
<tr><th rowspan="2">MATERIAL DESCRIPTION AND NOTES</th><th rowspan="2">ELEV.
619.7</th><th rowspan="2">DEPTHS</th><th rowspan="2">SPT/ RQD</th><th rowspan="2">N_{60}</th><th rowspan="2">REC (%)</th><th rowspan="2">SAMPLE ID</th><th rowspan="2">HP (tsf)</th><th colspan="5">GRADATION (%)</th><th colspan="3">ATTERBERG</th><th rowspan="2">WC</th><th rowspan="2">ODOT CLASS (GI)</th><th rowspan="2">HOLE SEALED</th></tr>
<tr><th>GR</th><th>CS</th><th>FS</th><th>SI</th><th>CL</th><th>LL</th><th>PL</th><th>PI</th></tr>
<tr><td></td><td>619.3</td><td>EOB</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
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NOTES: SEEPAGE ENCOUNTERED @ 16.0'; GROUNDWATER ENCOUNTERED INITIALLY @ 18.5'

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

2014 ODOT BORING LOG-RII NE BRIDGE ID - OH DOT.GDT - 7/12/19 12:58 - U:\GI8\PROJECTS\2013\W-13-072.GPJ

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-13.10 - PHASE 6A
TYPE: STRUCTURE
PID: 89464 BR ID: FRA-71-1503L
START: 2/4/14 END: 2/14/14

DRILLING FIRM / OPERATOR: RII / J.B.
SAMPLING FIRM / LOGGER: RII / T.F./K.S
DRILLING METHOD: 4.25" HSA / HQ
SAMPLING METHOD: SPT / RC

DRILL RIG: CME-750 (SN 98048)
HAMMER: CME AUTOMATIC
CALIBRATION DATE: 4/26/13
ENERGY RATIO (%): 82.6

STATION / OFFSET: 271+12.84 / 28.4' RT
ALIGNMENT: BL I-71 SB
ELEVATION: 713.3 (MSL) EOB: 84.0 ft.
LAT / LONG: 39.953782, -83.006602

EXPLORATION ID
B-114-7-13

PAGE
1 OF 3

MATERIAL DESCRIPTION AND NOTES	ELEV. 713.3	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			
0.2' - TOPSOIL (2.0")	713.1																	
VERY STIFF, BROWN **SILT AND CLAY**, SOME FINE GRAVEL, SOME COARSE TO FINE SAND, DAMP. -ROCK FRAGMENTS PRESENT THROUGHOUT		1 2	5 7 30	51	83	SS-1	3.25	32	14	13	22	19	33	19	14	12	A-6a (2)	
-IRON STAINING PRESENT IN SS-2	707.8	3 4 5	9 11 4	21	61	SS-2	3.00	-	-	-	-	-	-	-	-	11	A-6a (V)	
VERY STIFF, BROWN **CLAY**, AND SILT, TRACE FINE SAND, TRACE FINE GRAVEL, MOIST.		6 7	1 4 4	11	44	SS-3	3.00	2	0	8	43	47	41	20	21	22	A-7-6 (13)	
-IRON STAINING PRESENT THROUGHOUT		8 9 10	3 5 8	18	67	SS-4	3.25	-	-	-	-	-	-	-	-	25	A-7-6 (V)	
	700.3	11 12	8 9 9	25	89	SS-5	2.75	-	-	-	-	-	-	-	-	24	A-7-6 (V)	
LOOSE, BROWN **GRAVEL WITH SAND, SILT, AND CLAY**, MOIST. -COBBLES PRESENT @ 15.0'	697.8	13 14 15	3 3 3	8	94	SS-6	-	30	19	21	12	18	34	17	17	14	A-2-6 (1)	
MEDIUM DENSE TO DENSE, BROWN **GRAVEL**, LITTLE COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST.		W 16 W 17	5 9 13	30	56	SS-7	-	-	-	-	-	-	-	-	-	7	A-1-a (V)	
		18 19 20	6 8 8	22	67	SS-8	-	-	-	-	-	-	-	-	-	12	A-1-a (V)	
-ROCK FRAGMENTS PRESENT THROUGHOUT		21 22	7 8 7	21	78	SS-9	-	-	-	-	-	-	-	-	-	11	A-1-a (V)	
		23 24 25	8 10 12	30	72	SS-10	-	69	12	6	8	5	NP	NP	NP	14	A-1-a (0)	
	685.3	26 27	7 10 18	39	83	SS-11	-	-	-	-	-	-	-	-	-	8	A-1-a (V)	
VERY DENSE, BROWN **GRAVEL**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST.		28 29	41 50/4"	-	60	SS-12	-	-	-	-	-	-	-	-	-	8	A-1a (V)	

2014 ODOT BORING LOG-RII NE BRIDGE ID - OH DOT.GDT - 7/12/19 13:03 - U:\GI8\PROJECTS\2013\W-13-072.GPJ

PID: 89464 | BR ID: FRA-71-1503L | PROJECT: FRA-70-13.10 - PHASE 6A | STATION / OFFSET: 271+12.84 / 28.4 RT | START: 2/4/14 | END: 2/14/14 | PG 2 OF 3 | B-114-7-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 683.3	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			
VERY DENSE, BROWN **GRAVEL**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST. *(same as above)*	681.3	31 32																
VERY DENSE, GRAY **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, DAMP.	676.3	33 34 35 36 37	33 21 19	55	56	SS-13	-	58	17	8	10	7	18	14	4	8	A-1-b (0)	
MEDIUM DENSE, GRAY **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, MOIST. -COBBLES PRESENT @ 40.0'	671.3	38 39 40 41 42	6 6 9	21	33	SS-14	-	-	-	-	-	-	-	-	-	14	A-1-b (V)	
HARD, BROWN TO GRAY **SILTY CLAY**, TRACE COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST. -INTRODUCED MUD @ 43.5'	661.3	43 44 45 46 47	14 14 16	41	72	SS-15	4.50	4	1	3	46	46	31	15	16	20	A-6b (10)	
		48 49 50 51 52	11 11 11	30	83	SS-16	4.25	-	-	-	-	-	-	-	-	19	A-6b (V)	
HARD, GRAY **CLAY**, SOME TO "AND" SILT, TRACE TO LITTLE FINE GRAVEL, TRACE COARSE TO FINE SAND, DAMP. -ROCK FRAGMENTS PRESENT IN SS-17		53 54 55 56 57	21 32 50/3"	-	100	SS-17	-	9	3	3	38	47	41	18	23	14	A-7-6 (13)	
		58 59 60 61	31 50/5"	-	100	SS-18	-	-	-	-	-	-	-	-	-	15	A-7-6 (V)	

2014 ODOT BORING LOG-RII NE BRIDGE ID - OH DOT.GDT - 7/12/19 13:03 - U:\GI8\PROJECTS\2013\W-13-072.GPJ

PID: 89464 | BR ID: FRA-71-1503L | PROJECT: FRA-70-13.10 - PHASE 6A | STATION / OFFSET: 271+12.84 / 28.4 RT | START: 2/4/14 | END: 2/14/14 | PG 3 OF 3 | B-114-7-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 651.2	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			
HARD, GRAY **CLAY**, SOME TO "AND" SILT, TRACE TO LITTLE FINE GRAVEL, TRACE COARSE TO FINE SAND, DAMP. *(same as above)*	648.4	63 64	21 21 50/5"	-	59	SS-19	-	15	0	1	32	52	50	24	26	17	A-7-6 (16)	
SHALE : GRAY, HIGHLY WEATHERED, VERY WEAK.		TR 65 66 67 68																
		69	18 24 50/3"	-	67	SS-20	-	-	-	-	-	-	-	-	-	12	Rock (V)	
AUGER REFUSAL @ 74.0'	639.3	70 71 72 73																
LIMESTONE : DARK GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, STRONG TO VERY STRONG, VERY THICK BEDDED, CALCAREOUS, DOLOMITIC, ARGILLACEOUS, CALCITE VEINS, CHERT INCLUSIONS, JOINTED, MODERATELY TO HIGHLY FRACTURED, OPEN APERATURES, ROUGH; RQD 50%, REC 87%. -QU @ 75.1' = 8,488 PSI		74 75 76 77 78	48		92	RC-1											CORE	
-QU @ 81.5' = 9,141 PSI	629.3	79 80 81 82 83 EOB 84	52		83	RC-2											CORE	

NOTES: SEEPAGE ENCOUNTERED @ 16.0'; GROUNDWATER ENCOUNTERED INITIALLY @ 17.0'

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PUMPED 188 LBS CEMENT 100 LBS BENTONITE 70 GALLONS WATER; COMPACTED WITH THE AUGER 300 LBS BENTONITE CHIPS AND SOIL CUTTINGS

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-13.10 - PHASE 6A	DRILLING FIRM / OPERATOR: RII / J.K.	DRILL RIG: MOBILE B-53 (SN 624400)	STATION / OFFSET: 273+13.25 / 16.5' LT	**EXPLORATION ID** **B-114-8-13**
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / S.B.	HAMMER: AUTOMATIC	ALIGNMENT: BL I-71 SB	PAGE 1 OF 3
PID: 89464 BR ID: FRA-71-1503L	DRILLING METHOD: 3.25" HSA / NQ	CALIBRATION DATE: 4/26/13	ELEVATION: 714.0 (MSL) EOB: 82.0 ft.	
START: 3/11/14 END: 3/13/14	SAMPLING METHOD: SPT / RC	ENERGY RATIO (%): 77.7	LAT / LONG: 39.953894, -83.005886	

MATERIAL DESCRIPTION AND NOTES	ELEV. 714.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
0.3' - TOPSOIL (3.0") **FILL**: VERY STIFF TO HARD, DARK BROWN **SANDY SILT**, SOME FINE GRAVEL, LITTLE CLAY, DAMP.	713.7	1 2 3	7 8 8	21	50	SS-1	4.50	-	-	-	-	-	-	-	-	14	A-4a (V)	
	708.5	4 5	6 7 9	21	61	SS-2	3.00	21	22	14	25	18	32	22	10	17	A-4a (2)	
STIFF TO VERY STIFF, BROWN TO DARK BROWN **SILTY CLAY**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST.		6 7 8	3 2 1	4	33	SS-3	1.75	-	-	-	-	-	-	-	-	24	A-6b (V)	
		9 10	5 5 8	17	50	SS-4	2.25	-	-	-	-	-	-	-	-	21	A-6b (V)	
		11 12 13	2 3 4	9	89	SS-5	1.25	1	2	12	36	49	38	16	22	22	A-6b (13)	
	698.5	14 15	4 4 4	10	72	SS-6	1.25	-	-	-	-	-	-	-	-	22	A-6b (V)	
MEDIUM DENSE TO DENSE, BROWN **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST.		W 16 17	14 16 10	34	72	SS-7	-	64	13	9	10	4	20	19	1	10	A-1-a (0)	
	693.5	W 18 19 20	8 7 8	19	83	SS-8	-	-	-	-	-	-	-	-	-	11	A-1-a (V)	
MEDIUM DENSE TO DENSE, DARK BROWN TO BROWN **GRAVEL WITH SAND**, TRACE SILT, TRACE CLAY, MOIST.		21 22 23	4 8 16	31	56	SS-9	-	35	42	9	10	4	NP	NP	NP	16	A-1-b (0)	
-COBBLES PRESENT THROUGHOUT -INTRODUCED MUD @ 25.0'		24 25	12 16 18	44	72	SS-10	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	
	686.0	26 27 28	5 3 8	14	33	SS-11	-	-	-	-	-	-	-	-	-	30	A-1-b (V)	
VERY DENSE, BROWN **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, DRY TO MOIST.		29	19 27 28	71	72	SS-12	-	63	18	7	9	3	NP	NP	NP	12	A-1-a (0)	

PID: 89464 | BR ID: FRA-71-1503L | PROJECT: FRA-70-13.10 - PHASE 6A | STATION / OFFSET: 273+13.25 / 16.5 LT | START: 3/11/14 | END: 3/13/14 | PG 2 OF 3 | B-114-8-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 684.0	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			
VERY DENSE, BROWN **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, DRY TO MOIST. *(same as above)* -COBBLES PRESENT THROUGHOUT		31																
		32																
		33																
		34	50/5"	-	20	SS-13	-	-	-	-	-	-	-	-	-	3	A-1-a (V)	
		35																
		36																
	677.0	37																
HARD, GRAY **SILTY CLAY**, TRACE COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST.		38																
		39	5 6 8	18	33	SS-14	4.25	3	2	2	38	55	36	18	18	21	A-6b (11)	
		40																
		41																
	672.0	42																
MEDIUM DENSE, GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, MOIST.		43																
	669.2	44	4 8 6	18	100	SS-15	-	-	-	-	-	-	-	-	-	14	A-1-a (V)	
VERY STIFF TO HARD, BROWN **CLAY**, SOME SILT, SOME COARSE TO FINE SAND, LITTLE FINE GRAVEL, DAMP.		45					3.50	-	-	-	-	-	-	-	-	18	A-7-6 (V)	
		46																
		47																
		48																
		49	8 10 13	30	67	SS-16	3.50	15	11	10	30	34	42	20	22	18	A-7-6 (11)	
		50																
		51																
		52																
		53																
		54	6 42 50/5"	-	65	SS-17	4.5+	-	-	-	-	-	-	-	-	12	A-7-6 (V)	
		55																
		56																
		57																
		58																
		59	10 49 50/3"	-	100	SS-18	4.5+	-	-	-	-	-	-	-	-	15	A-7-6 (V)	
		60																
		61																

2014 ODOT BORING LOG-RII NE BRIDGE ID - OH DOT.GDT - 7/12/19 13:03 - U:\GI8\PROJECTS\2013\W-13-072.GPJ

PID: 89464	BR ID: FRA-71-1503L	PROJECT: FRA-70-13.10 - PHASE 6A	STATION / OFFSET: 273+13.25 / 16.5 LT	START: 3/11/14	END: 3/13/14	PG 3 OF 3	B-114-8-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 651.9	DEPTHS	SPT/ RQD	N60	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
	650.5	63 TR																
SHALE : GRAY, HIGHLY WEATHERED, VERY WEAK. AUGER REFUSAL @ 67.0'	647.0	64 65 66	37 50/4"	-	100	SS-19	-	-	-	-	-	-	-	-	-	15	Rock (V)	
SHALE : DARK GRAY, SLIGHTLY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED TO LAMINATED, ARGILLACEOUS, FRACTURED TO HIGHLY FRACTURED, NARROW TO OPEN APERTURES, SLICKENSIDED TO SLIGHTLY ROUGH; RQD 19%, REC 82%. -0.5' CLAYSTONE SEAM @ 67.7' -CLAYSTONE SEAMS PRESENT FROM 69.5' TO 72.0'	642.0	67 68 69 70 71	19		82	RC-1											CORE	
CLAYSTONE : DARK GRAY, SLIGHTLY WEATHERED, VERY WEAK TO WEAK, THINLY LAMINATED BEDDING TO VERY THIN BEDDED, FRACTURED TO HIGHLY FRACTURED, OPEN APERTURES, ROUGH; RQD 12%, REC 97%. -QU @ 74.8' = 92 PSI	637.0	72 73 74 75 76	12		97	RC-2											CORE	
LIMESTONE : GRAY, UNWEATHERED, STRONG, VERY THICK BEDDED, CALCAREOUS, CHERTY, PYRITIC, JOINTED, SLIGHTLY FRACTURED TO FRACTURED, NARROW TO OPEN APERTURES, SLIGHTLY ROUGH; RQD 92%, REC 98%. -QU @ 78.4' = 12,567 PSI	632.0	77 78 79 80 81 EOB 82	92		98	RC-3											CORE	

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

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PUMPED 188 LBS CEMENT 50 LBS BENTONITE 40 GALLONS WATER AND SOIL CUTTINGS

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-13.10 - PHASE 6A
TYPE: STRUCTURE
PID: 89464 BR ID: FRA-71-1503L
START: 5/9/14 END: 5/13/14

DRILLING FIRM / OPERATOR: RII / T.F.
SAMPLING FIRM / LOGGER: RII / B.Z.
DRILLING METHOD: 3.25" HSA / NQ
SAMPLING METHOD: SPT / RC

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

STATION / OFFSET: 275+07.50 / 4.7' RT
ALIGNMENT: BL I-71 SB
ELEVATION: 714.0 (MSL) EOB: 90.0 ft.
LAT / LONG: 39.953788, -83.005201

EXPLORATION ID
B-114-9-13

PAGE
1 OF 3

MATERIAL DESCRIPTION AND NOTES	ELEV. 714.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
0.4' - TOPSOIL (4.5")	713.6																	
FILL: LOOSE TO MEDIUM DENSE, BROWN AND BLACK **GRAVEL WITH SAND AND SILT**, TRACE CLAY, DAMP TO MOIST. -ROOT FIBERS PRESENT IN SS-1		1 2 3	2 9 9	23	33	SS-1	-	56	15	10	13	6	-	-	-	8	A-2-4 (V)	
-BRICK FRAGMENTS PRESENT IN SS-2 AND SS-3		4 5	3 4 5	12	33	SS-2	-	60	10	7	15	8	33	23	10	23	A-2-4 (0)	
-CINDERS PRESENT IN SS-3	706.0	6 7 8	2 2 2	5	67	SS-3	-	-	-	-	-	-	-	-	-	28	A-2-4 (V)	
STIFF, DARK BROWN **SILTY CLAY**, TRACE COARSE TO FINE SAND, MOIST.		9 10	1 2 4	8	67	SS-4	1.75	-	-	-	-	-	-	-	-	29	A-6b (V)	
-COBBLE PRESENT @ 13.5'	701.0	11 12 13	2 3 4	9	89	SS-5	2.00	0	1	7	53	39	37	20	17	22	A-6b (11)	
MEDIUM DENSE TO DENSE, BROWN **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, MOIST TO WET. -LARGE PIECE OF ROCK RECOVERED IN SS-6		14 15	3 4 7	14	44	SS-6	-	-	-	-	-	-	-	-	-	-		
		16 17 18	3 5 7	16	61	SS-7	-	49	11	17	19	4	NP	NP	NP	11	A-1-b (0)	
-COBBLES PRESENT THROUGHOUT		19 20	16 14 17	40	67	SS-8	-	-	-	-	-	-	-	-	-	8	A-1-b (V)	
		21 22 W 23	3 4 7	14	61	SS-9	-	33	31	16	14	6	26	20	6	18	A-1-b (0)	
		24 25	21 17 20	48	0	SS-10	-	-	-	-	-	-	-	-	-	-		
	688.5		15	-	100	3S-10A	-	-	-	-	-	-	-	-	-	23	A-1-b (V)	
MEDIUM DENSE TO VERY DENSE, BROWN **GRAVEL**, LITTLE COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST.		26 27 28	12 9 9	23	44	SS-11	-	82	10	3	4	1	NP	NP	NP	10	A-1-a (V)	
		29	16 21 21	54	39	SS-12	-	-	-	-	-	-	-	-	-	9	A-1-a (V)	

PID: 89464	BR ID: FRA-71-1503L	PROJECT: FRA-70-13.10 - PHASE 6A	STATION / OFFSET: 275+07.50 / 4.7 RT	START: 5/9/14	END: 5/13/14	PG 2 OF 3	B-114-9-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 684.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
MEDIUM DENSE TO VERY DENSE, BROWN **GRAVEL**, LITTLE COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST. *(same as above)* -INTRODUCED MUD @ 30.0'	682.0	31 32																
VERY DENSE, BROWN **COARSE AND FINE SAND**, LITTLE FINE GRAVEL, LITTLE SILT, TRACE CLAY, WET.	677.0	33 34 35 36 37	30 50/3"	-	100	SS-13	-	11	18	51	15	5	NP	NP	NP	24	A-3a (0)	
HARD, BROWN **SANDY SILT**, LITTLE CLAY, TRACE FINE GRAVEL, DAMP.	672.0	38 39 40 41 42	50/5"	-	100	SS-14	4.50	-	-	-	-	-	-	-	-	10	A-4a (V)	
VERY DENSE, GRAY **COARSE AND FINE SAND**, LITTLE SILT, TRACE FINE GRAVEL, MOIST.	667.0	43 44 45 46 47	17 19 20	51	67	SS-15	-	-	-	-	-	-	-	-	-	13	A-3a (V)	
VERY STIFF TO HARD, GRAY **CLAY**, SOME SILT, TRACE COARSE TO FINE SAND, MOIST.		48 49 50	2 9 12	27	72	SS-16	3.50	0	1	2	28	69	42	19	23	21	A-7-6 (14)	
		51 52 53 54 55	9 11 16	35	78	SS-17	4.50	-	-	-	-	-	-	-	-	23	A-7-6 (V)	
	655.0	56 57 58 TR 59	18 50/4"	-	100	SS-18	4.5+ -	- -	- -	- -	- -	- -	- -	- -	- -	22 10	A-7-6 (V) Rock (V)	
SHALE : GRAY, HIGHLY WEATHERED, VERY WEAK.		60 61																

PID: 89464	BR ID: FRA-71-1503L	PROJECT: FRA-70-13.10 - PHASE 6A	STATION / OFFSET: 275+07.50 / 4.7 RT	START: 5/9/14	END: 5/13/14	PG 3 OF 3	B-114-9-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 651.9	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
SHALE : GRAY, HIGHLY WEATHERED, VERY WEAK. *(same as above)* AUGER REFUSAL @ 65.0'	649.0	63 64	50/5"	-	100	SS-19	-	-	-	-	-	-	-	-	-	16	Rock (V)	
SHALE : DARK GRAY, SLIGHTLY WEATHERED, VERY WEAK TO WEAK, VERY THIN TO THIN BEDDED, CALCAREOUS, FISSILE, PYRITIC, MODERATELY FRACTURED TO FRACTURED, NARROW TO OPEN APERTURES, SLIGHTLY ROUGH; RQD 64%, REC 79%. -ARGILLACEOUS FROM 65.6' TO 75.0'		65 66 67 68 69	26		33	RC-1											CORE	
-0.3' LIMESTONE SEAM @ 71.5'		70 71 72 73 74	58		88	RC-2											CORE	
-ARGILLACEOUS FROM 78.6' TO 85.0' -POINT LOAD STRENGTH @ 78.8' TO 81.3' -MEAN QU = 176 PSI		75 76 77 78 79	83		100	RC-3											CORE	
	629.0	80 81 82 83 84	90		93	RC-4											CORE	
LIMESTONE : GRAY, UNWEATHERED, VERY STRONG, MEDIUM TO THICK BEDDED, CALCAREOUS, DOLOMITIC, PYRITIC, MODERATELY FRACTURED TO FRACTURED, OPEN APERTURES, SLIGHTLY ROUGH; RQD 89%, REC 93%.	624.0	85 86 87 88 89 EOB 90	89		93	RC-5											CORE	

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 23.0'

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

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: 2/13/2014
Test Performed by: J.H./T.K.

Rock Description: Gray Limestone

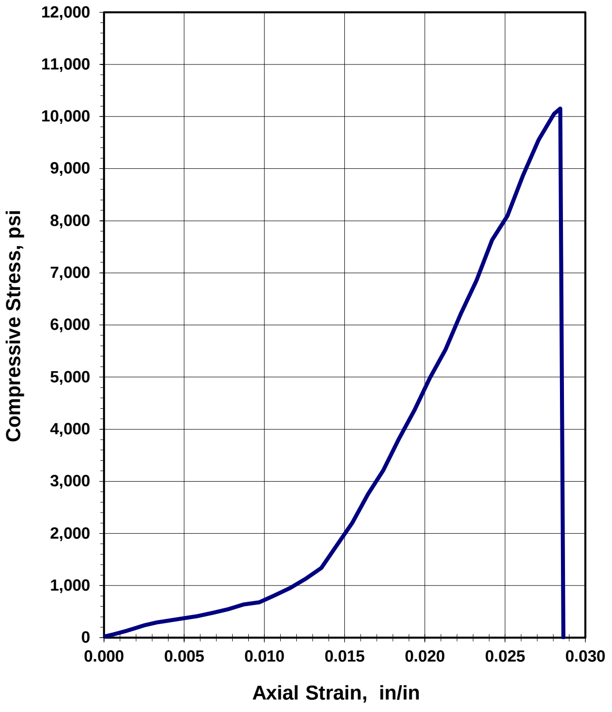
Boring No.: B-018-2-13
Sample No: RC-1
Depth (ft): 78.5
Moisture condition: As received

Average Length: 5.169 in
Average Diameter: 2.4 in
Length to diameter ratio: 2.154
Cross Sectional Area: 4.522 in^2

Rate of Loading: 78.5 lbs/sec
Testing Time: 585 sec
(Rate 2-15 minutes to failure)

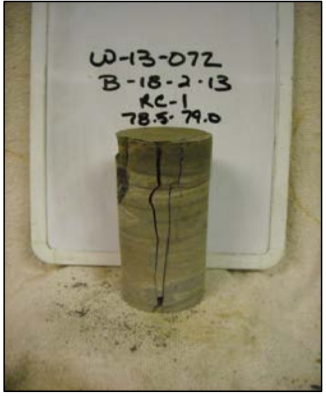
Failure Load: 45,920 lbs
Axial Strain at Failure: 0.0284 in/in
Stress: 10,153 psi

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

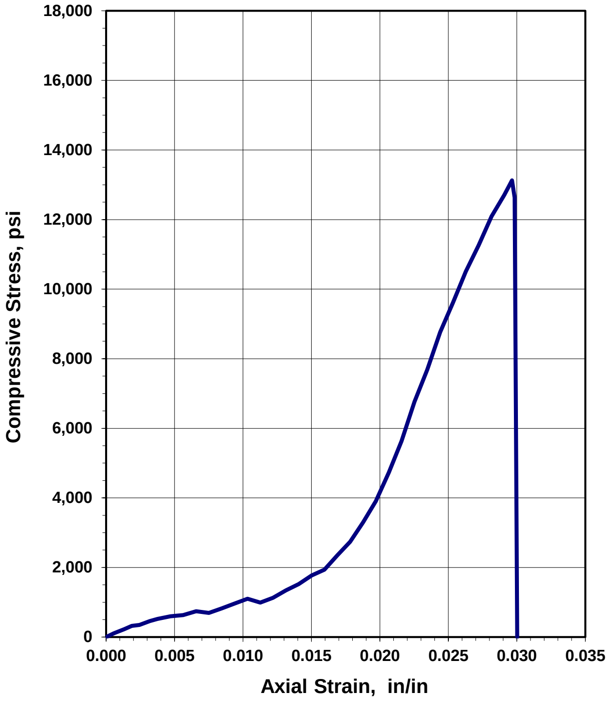
4480 Lake Forest Drive
Cincinnati, Ohio 45242
Phone (513) 769-6998

Project: FRA-70-13.10 - Project 6A
Project No.: W-13-072
Date of Testing: 4/1/2014
Test Performed by: K.R./T.K.

Rock Description: Limestone

Boring No.:	B-20-4	Average Length:	5.329 in
Sample No:	RC-3	Average Diameter:	2.477 in
Depth (ft):	86.5	Length to diameter ratio:	2.151
Moisture condition:	As received	Cross Sectional Area:	4.816 in^2
Rate of Loading:	114.2 lbs/sec	Failure Load:	63,240 lbs
Testing Time:	554 sec	Axial Strain at Failure:	0.0296 in/in
	(Rate 2-15 minutes to failure)	Stress:	13,126 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

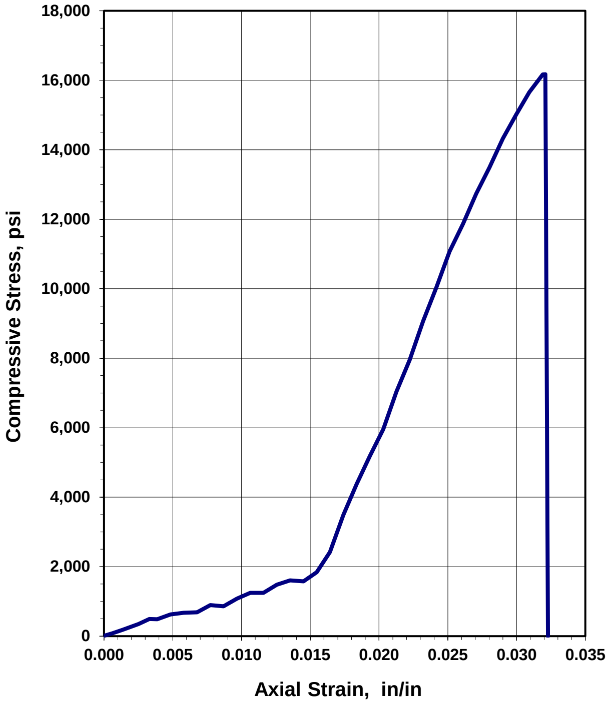
4480 Lake Forest Drive
Cincinnati, Ohio 45242
Phone (513) 769-6998

Project: FRA-70-13.10 - Project 6A
Project No.: W-13-072
Date of Testing: 4/1/2014
Test Performed by: K.R./T.K.

Rock Description: Limestone

Boring No.:	B-20-4	Average Length:	5.173 in
Sample No:	RC-4	Average Diameter:	2.47 in
Depth (ft):	90.7	Length to diameter ratio:	2.094
Moisture condition:	As received	Cross Sectional Area:	4.789 in^2
Rate of Loading:	124.0 lbs/sec	Failure Load:	77,480 lbs
Testing Time:	625 sec	Axial Strain at Failure:	0.0321 in/in
(Rate 2-15 minutes to failure)		Stress:	16,173 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
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9885 Rockside Road
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Phone (216) 573-0955

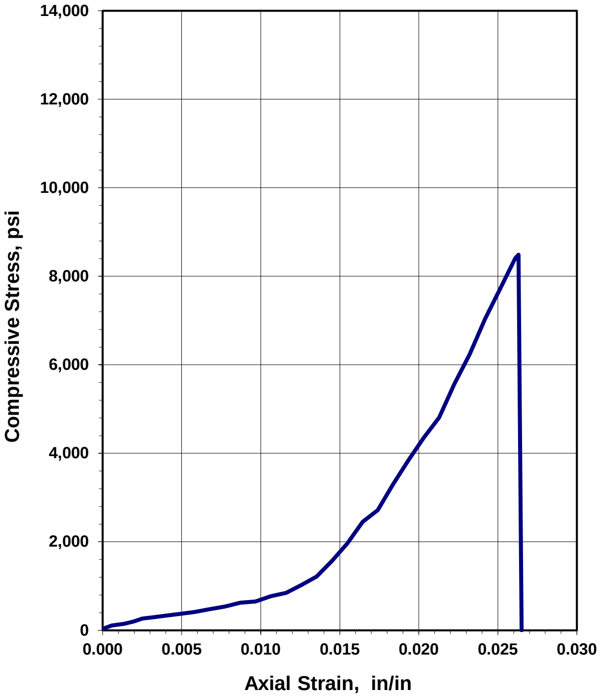
4480 Lake Forest Drive
Cincinnati, Ohio 45242
Phone (513) 769-6998

Project: FRA-70-13.10 - Project 6A
Project No.: W-13-072
Date of Testing: 2/20/2014
Test Performed by: K.R./T.K.

Rock Description: Gray Limestone

Boring No.:	B-114-7-13	Average Length:	5.169 in
Sample No:	RC-1	Average Diameter:	2.4 in
Depth (ft):	75.1	Length to diameter ratio:	2.154
Moisture condition:	As received	Cross Sectional Area:	4.522 in^2
Rate of Loading:	67.1 lbs/sec	Failure Load:	38,390 lbs
Testing Time:	572 sec	Axial Strain at Failure:	0.0263 in/in
(Rate 2-15 minutes to failure)		Stress:	8,488 psi

Unconfined Compression Test



Before Testing

After Failure

REMARKS:

RESOURCE INTERNATIONAL, INC.
Engineering Consultants

6350 Presidential Gatew.
Columbus, OH 43231
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9885 Rockside Road
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Phone (216) 573-0955

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

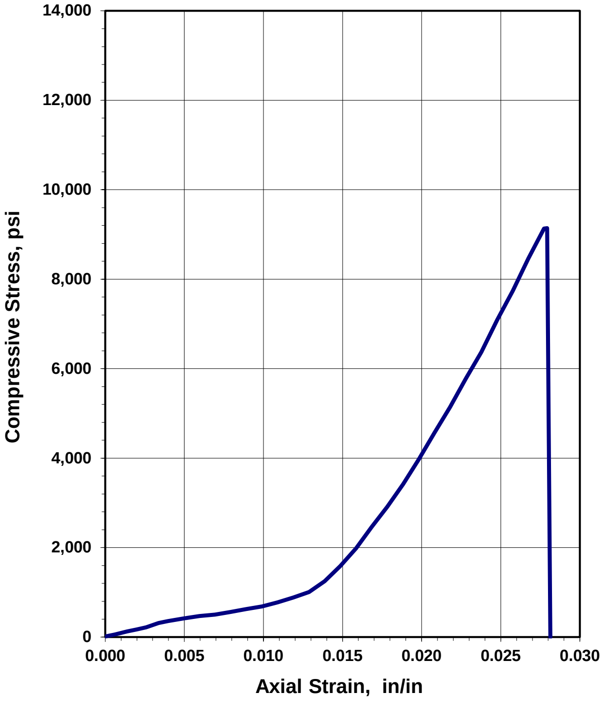
Unconfined Compressive Strength
of Intact Rock Core Specimens (ASTM D 7012-04)

Project: FRA-70-13.10 - Project 6A
Project No.: W-13-072
Date of Testing: 2/20/2014
Test Performed by: K.R./T.K.

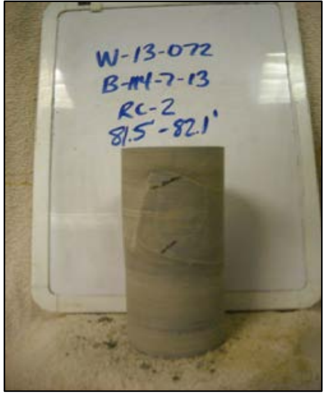
Rock Description: Gray Limestone

Boring No.:	B-114-7-13	Average Length:	5.047 in
Sample No:	RC-2	Average Diameter:	2.397 in
Depth (ft):	81.5	Length to diameter ratio:	2.106
Moisture condition:	As received	Cross Sectional Area:	4.510 in^2
Rate of Loading:	59.8 lbs/sec	Failure Load:	41,240 lbs
Testing Time:	690 sec	Axial Strain at Failure:	0.0279 in/in
(Rate 2-15 minutes to failure)		Stress:	9,141 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

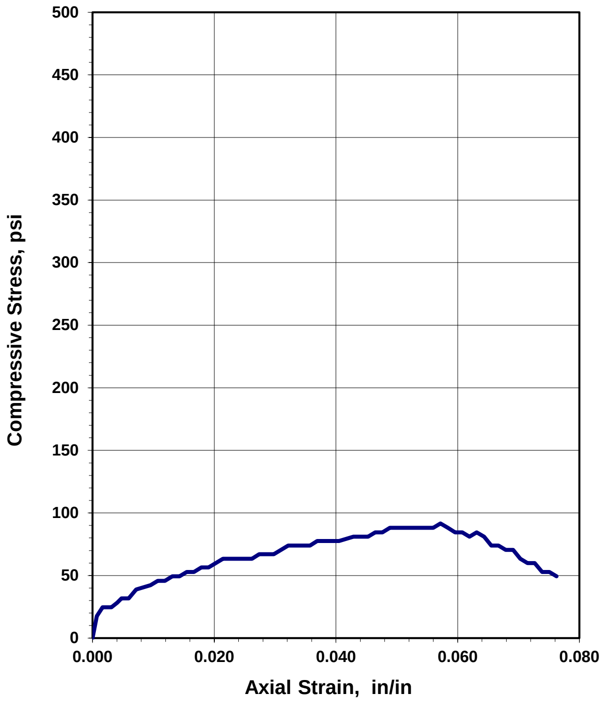
4480 Lake Forest Drive
Cincinnati, Ohio 45242
Phone (513) 769-6998

Project: FRA-70-13.10 - Project 6A
Project No.: W-13-072
Date of Testing: 3/17/2014
Test Performed by: K.R./T.K.

Rock Description: Gray Claystone

Boring No.:	B-114-8-13	Average Length:	4.197 in
Sample No:	RC-2	Average Diameter:	1.901 in
Depth (ft):	74.8	Length to diameter ratio:	2.208
Moisture condition:	As received	Cross Sectional Area:	2.837 in^2
Rate of Loading:	0.3 lbs/sec	Failure Load:	260 lbs
Testing Time:	821 sec	Axial Strain at Failure:	0.0572 in/in
(Rate 2-15 minutes to failure)		Stress:	92 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
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9885 Rockside Road
Cleveland, OH 44125
Phone (216) 573-0955

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

Project: FRA-70-13.10 - Project 6A
Project No.: W-13-072
Date of Testing: 4/1/2014
Test Performed by: K.R./T.K.

Rock Description: Limestone

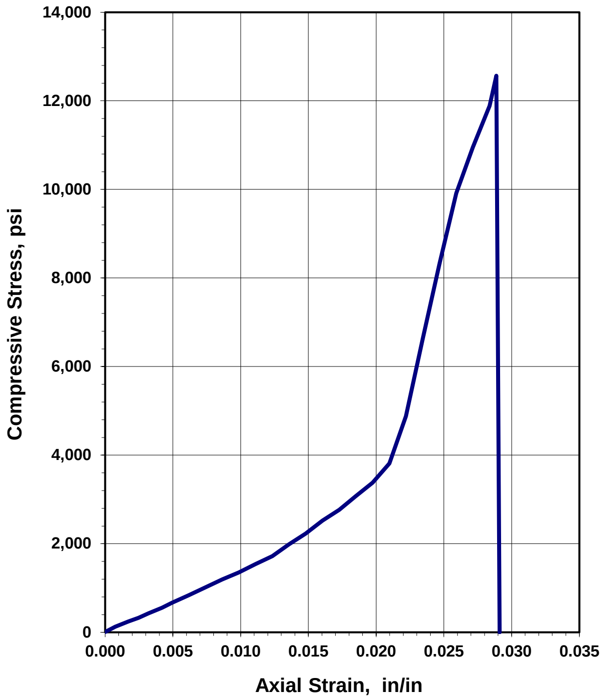
Boring No.: B-114-8-13
Sample No: RC-3
Depth (ft): 78.4
Moisture condition: As received

Average Length: 4.053 in
Average Diameter: 1.869 in
Length to diameter ratio: 2.169
Cross Sectional Area: 2.742 in^2

Rate of Loading: 127.7 lbs/sec
Testing Time: 270 sec
(Rate 2-15 minutes to failure)

Failure Load: 34,470 lbs
Axial Strain at Failure: 0.0289 in/in
Stress: 12,567 psi

Unconfined Compression Test



Before Testing

After Failure

REMARKS:

RESOURCE INTERNATIONAL, INC.

Engineering Consultants

Point Load Strength Index of Rock Specimens

(ASTM D 5731-08)

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 - PHASE 6A
Project No.: W-13-072
Date of Testing: 6/19/2014
Test Performed by: E.M.

Rock Description: SHALE

Boring No.: B-114-9-13
Sample No: RC-3 / RC-4
Moisture condition: As received

Test Apparatus: Forney-LA 0080
Serial Number: A125/AZ/0014
Date of Calibration: 8/12/2013

Sample No.	Test Type	Depth (ft)	Width (mm)	Diameter (mm)	Load (N)	D_e^2 (mm^2)	D_e (mm)	F	Is (MPa)	$Is_{(50)}$ (MPa)	σ_c (MPa)
1	a ⊥	78.8'-81.3'	36.1	46.9	330	2,159	46.5	0.97	0.15	0.15	3.52
2	a ⊥	78.8'-81.3'	33.5	50.5	35	2,157	46.4	0.97	0.02	0.02	0.37
3	a ⊥	78.8'-81.3'	33.4	47.8	25	2,032	45.1	0.95	0.01	0.01	0.28
4	a ⊥	78.8'-81.3'	35.1	47.9	330	2,143	46.3	0.97	0.15	0.15	3.54
5	a ⊥	78.8'-81.3'	35.1	47.9	310	1,808	42.5	0.93	0.17	0.16	3.94

STATISTICS	
Mean $Is_{(50)}$ ⊥	**0.10 MPa (14 psi)**
Mean $Is_{(50)}$ ‖	
$Ia_{(50)}$	

Specific Specimen Shape:

d = diametrical
a = axial
b = block
i = irregular lump
⊥ = perpendicular to bedding plane
‖ = parallel to bedding plane

Estimated Unaxial Compression, σ_c = K*Is

K = 23

*Per Table 1 of ASTM D5731

Mean σ_c = 2.33 MPa (338 psi)

Remarks:

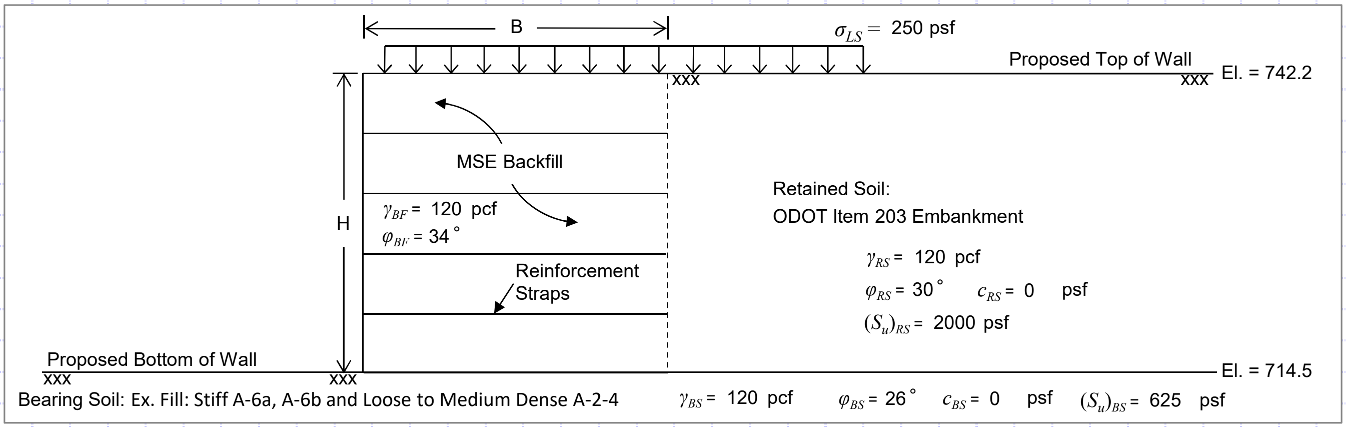
APPENDIX V

MSE WALL CALCULATIONS

JOB	FRA-70-13.10	NO.	W-13-072
SHEET NO.	1	OF	6
CALCULATED BY	BRT	DATE	7/12/2019
CHECKED BY	JPS	DATE	7/12/2019

Retaining Wall E4 - Sta. 402+02 to 404+75

Retaining Wall E4 - Sta. 402+02 to 404+75 - B-020-4-13 and B-114-9-13 - 27.7 ft. Wall Height



MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	27.7	ft
MSE Wall Width (Reinforcement Length), (B) =	19.4	ft
MSE Wall Length, (L) =	750	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}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(S_u)_{BS}]$ =	625	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

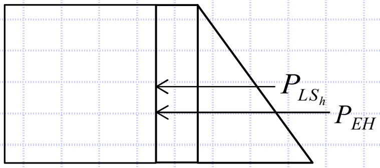
LRFD Load Factors

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

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

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

Sliding Force:



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

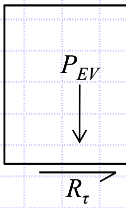
$$P_{EH} = \tfrac{1}{2}\gamma_{RS} H^2 K_a \gamma_{EH} = \text{½(120 pcf)(27.7 ft)²(0.297)(1.5)} = 20.51 \text{ kip/ft}$$

$$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS} = \text{(250 psf)(27.7 ft)(0.297)(1.75)} = 3.6 \text{ kip/ft}$$

$$P_H = 20.51 \text{ kip/ft} + 3.6 \text{ kip/ft} = 24.11 \text{ kip/ft}$$

Check Sliding Resistance - Drained Condition

Nominal Sliding Resistance:



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

$$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV} = \text{(120 pcf)(27.7 ft)(19.4 ft)(1.00)} = 64.49 \text{ kip/ft}$$

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

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

$$R_\tau = \text{(64.49 kip/ft)(0.49)} = 31.60 \text{ kip/ft}$$

Verify Sliding Force Less Than Factored Sliding Resistance - Drained Condition

$$P_H \le R_\tau \cdot \phi_\tau \longrightarrow 24.11 \text{ kip/ft} \le (31.60 \text{ kip/ft})(1.0) = 31.60 \text{ kip/ft} \longrightarrow 24.11 \text{ kip/ft} \le 31.60 \text{ kip/ft}$$

OK

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

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	27.7	ft
MSE Wall Width (Reinforcement Length), (B) =	19.4	ft
MSE Wall Length, (L) =	750	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}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	625	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

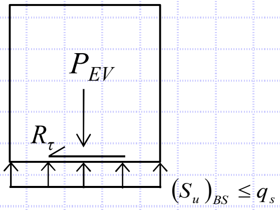
	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

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

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

Check Sliding Resistance - Undrained Condition

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



$(S_u)_{BS}$ = 0.63 ksf

$q_s = \sigma_v / 2$ = (3.32 ksf) / 2 = 1.66 ksf

$\sigma_v = P_{EV} / B$ = (64.49 kip/ft) / (19.4 ft) = 3.32 ksf

R_τ = (0.63 ksf ≤ 1.66 ksf)(19.4 ft) = 12.13 kip/ft

Verify Sliding Force Less Than Factored Sliding Resistance - Undrained Condition

$P_H \leq R_\tau \cdot \phi_\tau$ ⟶ 24.11 kip/ft ≤ (12.13 kip/ft)(1.0) = 12.13 kip/ft ⟶ 24.11 kip/ft ≤ 12.13 kip/ft **ERROR!!**

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

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) = 27.7 ft
MSE Wall Width (Reinforcement Length), (B) = 19.4 ft
MSE Wall Length, (L) = 750 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:

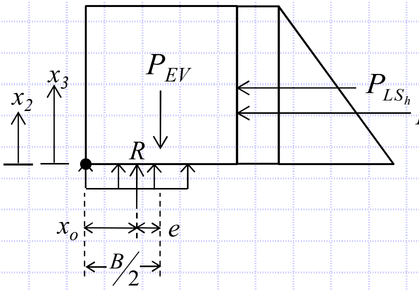
Bearing Soil Unit Weight, (γ_{BS}) = 120 pcf
Bearing Soil Friction Angle, (φ_{BS}) = 26 °
Bearing Soil Drained Cohesion, (c_{BS}) = 0 psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ = 625 psf
Embedment Depth, (D_f) = 4.0 ft
Depth to Grounwater (Below Bot. of Wall), (D_W) = 0.0 ft

LRFD Load Factors

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

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

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



$$e = {}^{B}\!/_{2} - x_o$$

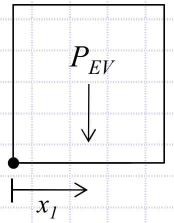
$$x_o = \frac{M_{EV} - M_H}{P_{EV}}$$ = (625.55 kip·ft/ft - 239.17 kip·ft/ft) / (64.49 kip/ft) = 5.99 ft

M_{EV} = 625.55 kip·ft/ft
M_H = 239.17 kip·ft/ft
P_{EV} = 64.49 kip/ft

Defined below

$e =$ (19.4 ft)/2 - 5.99 ft = 3.71 ft

Resisting Moment, M_{EV}:



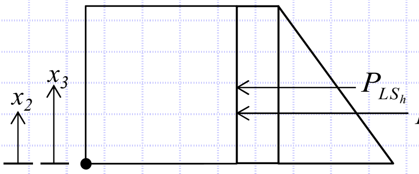
$$M_{EV} = P_{EV}(x_1)$$

$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV}$ = (120 pcf)(27.7 ft)(19.4 ft)(1.00) = 64.49 kip/ft

$x_1 = {}^{B}\!/_{2}$ = (19.4 ft) / 2 = 9.70 ft

$M_{EV} =$ (64.49 kip/ft)(9.70 ft) = 625.55 kip·ft/ft

Overturning Moment, M_H:



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

$P_{EH} = {}^{1}\!/_{2}\gamma_{RS}H^2K_a\gamma_{EH}$ = ½(120 pcf)(27.7 ft)²(0.297)(1.5) = 20.51 kip/ft

$P_{LS_h} = \sigma_{LS}HK_a\gamma_{LS}$ = (250 psf)(27.7 ft)(0.297)(1.75) = 3.6 kip/ft

$x_2 = {}^{H}\!/_{3}$ = (27.7 ft) / 3 = 9.23 ft

$x_3 = {}^{H}\!/_{2}$ = (27.7 ft) / 2 = 13.85 ft

$M_H =$ (20.51 kip/ft)(9.23 ft) + (3.6 kip/ft)(13.85 ft) = 239.17 kip·ft/ft

---

Check Eccentricity

$e < e_{\max} \longrightarrow$ 3.71 ft < 6.47 ft **OK**

Limiting Eccentricity: $e_{\max} = {}^{B}\!/_{3} \longrightarrow e_{\max} =$ (19.4 ft) / 3 = 6.47 ft

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	27.7	ft
MSE Wall Width (Reinforcement Length), (B) =	19.4	ft
MSE Wall Length, (L) =	750	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:

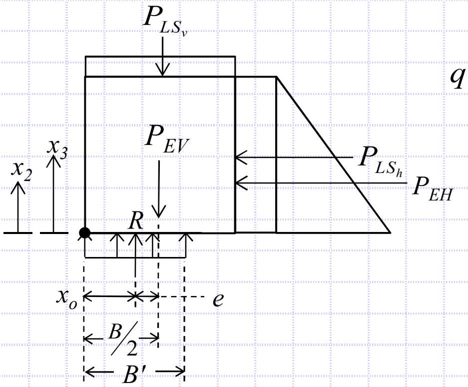
Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	625	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

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

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

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



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

$B' = B - 2e$ = 19.4 ft - 2(2.5 ft) = 14.40 ft

$e = B/2 - x_o$ = (19.4 ft) / 2 - 7.2 ft = 2.50 ft

$x_o = \dfrac{M_V - M_H}{P_V}$ = (926.77 kip·ft/ft - 239.15 kip·ft/ft) / 95.54 kip/ft = 7.2 ft

$q_{eq} =$ (95.54 kip/ft) / (14.4 ft) = 6.63 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)(27.7 ft)(19.4 ft)(1.35)](9.7 ft) + [(250 psf)(19.4 ft)(1.75)](9.7 ft) = 926.77 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)(27.7 ft)²(0.297)(1.5)](9.23 ft) + [(250 psf)(27.7 ft)(0.297)(1.75)](13.85 ft) = 239.15 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)(27.7 ft)(19.4 ft)(1.35) + (250 psf)(19.4 ft)(1.75) = 95.54 kip/ft

Check Bearing Resistance - Drained Condition

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

$N_{cm} = N_c s_c i_c$ = 22.47

N_c = 22.25
s_c = 1+(14.4 ft/750 ft)(11.85/22.25)
= 1.010
i_c = 1.000 (Assumed)

$N_{qm} = N_q s_q d_q i_q$ = 12.95

N_q = 11.85
s_q = 1.009
d_q = 1+2tan(26°)[1-sin(26°)]²tan⁻¹(4.0 ft/14.4 ft)
1.083
i_q = 1.000 (Assumed)
C_{wq} = 0.0 ft > 4.0 ft = 0.500

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 12.44

N_γ = 12.54
s_γ = 0.992
i_γ = 1.000 (Assumed)
$C_{w\gamma}$ = 0.0 ft < 1.5(14.4 ft) + 4.0 ft = 0.500

$q_n =$ (0 psf)(22.473) + (120 pcf)(4.0 ft)(12.949)(0.500) + ½(120 pcf)(14.4 ft)(12.440)(0.500) = 8.48 ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

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

$q_{eq} \le q_n \cdot \phi_b$ $\longrightarrow$ 6.63 ksf ≤ (8.48 ksf)(0.65) = 5.51 ksf $\longrightarrow$ 6.63 ksf ≤ 5.51 ksf **ERROR!!**

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) = 27.7 ft
MSE Wall Width (Reinforcement Length), (B) = 19.4 ft
MSE Wall Length, (L) = 750 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}) = 120 pcf
Bearing Soil Friction Angle, (φ_{BS}) = 26 °
Bearing Soil Drained Cohesion, (c_{BS}) = 0 psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ = 625 psf
Embedment Depth, (D_f) = 4.0 ft
Depth to Grounwater (Below Bot. of Wall), (D_W) = 0.0 ft

LRFD Load Factors

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

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

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

Check Bearing Resistance - Undrained Condition

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

$N_{cm} = N_c s_c i_c$ = 5.160

N_c = 5.140
s_c = 1+(14.4 ft/[(5)(750 ft)]) = 1.004
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/14.4 ft)
1.000
i_q = 1.000 (Assumed)
C_{wq} = 0.0 ft > 4.0 ft = 0.500

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

N_γ = 0.000
s_γ = 1.000
i_γ = 1.000 (Assumed)
$C_{w\gamma}$ = 0.0 ft < 1.5(14.4 ft) + 4.0 ft = 0.500

$q_n =$ (625 psf)(5.160) + (120 pcf)(4.0 ft)(1.000)(0.500) + ½(120 pcf)(14.4 ft)(0.000)(0.500) = 3.47 ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

$q_{eq} \leq q_n \cdot \phi_b$ ⟶ 6.63 ksf ≤ (3.47 ksf)(0.65) = 2.26 ksf ⟶ 6.63 ksf ≤ 2.26 ksf **ERROR!!**

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) =	27.7	ft
MSE Wall Width (Reinforcement Length), (B) =	19.4	ft
MSE Wall Length, (L) =	750	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_{RS}) =	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:

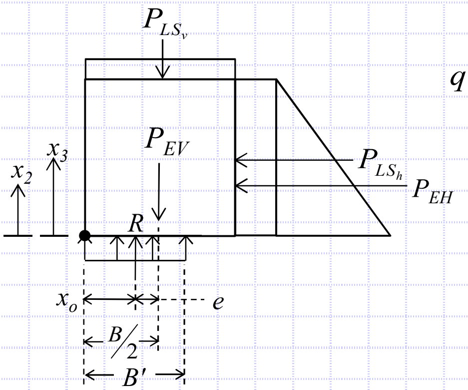
Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	625	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

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

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

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



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

$$B' = B - 2e \quad = \quad 19.4\text{ ft - 2(2.23 ft)} \quad = \quad 14.94 \quad \text{ft}$$

$$e = B/2 - x_o \quad = \quad \text{(19.4 ft) / 2 - 7.47 ft} \quad = \quad 2.23 \quad \text{ft}$$

$$x_o = \frac{M_V - M_H}{P_V} \quad = \quad \text{(672.56 kip·ft/ft - 154.69 kip·ft/ft) / 69.34 kip/ft} \quad = \quad 7.47 \quad \text{ft}$$

$$q_{eq} = \quad \text{(69.34 kip/ft) / (14.94 ft)} \quad = \quad 4.64 \quad \text{ksf}$$

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

$$M_V = \quad \text{[(120 pcf)(27.7 ft)(19.4 ft)(1.00)](9.7 ft) + [(250 psf)(19.4 ft)(1.00)](9.7 ft)} \quad = \quad 672.56 \quad \text{kip·ft/ft}$$

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

$$M_H = \quad \text{[½(120 pcf)(27.7 ft)²(0.297)(1.00)](9.23 ft) + [(250 psf)(27.7 ft)(0.297)(1.00)](13.85 ft)} \quad = \quad 154.69 \quad \text{kip·ft/ft}$$

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

$$P_V = \quad \text{(120 pcf)(27.7 ft)(19.4 ft)(1.00) + (250 psf)(19.4 ft)(1.00)} \quad = \quad 69.34 \quad \text{kip/ft}$$

Settlement, Time Rate of Consolidation and Differential Settlement:

Boring	Total Settlement at Center of Reinforced Soil Mass	Total Settlement at Wall Facing	Time for 90% Consolidation	Distance Between Borings Along Wall Facing	Differential Settlement Along Wall Facing
B-020-4-13	10.464 in	5.175 in	23 days		
B-114-9-13	7.697 in	4.758 in	8 days	160 ft	1/4600
B-114-8-13	15.594 in	8.003 in	90 days	175 ft	1/650
B-114-7-13	15.451 in	8.380 in	100 days	215 ft	1/6840
B-017-2-09	9.057 in	5.790 in	55 days	90 ft	1/420
B-018-2-13	12.067 in	7.604 in	100 days	100 ft	1/660

W-13-072 - FRA-70-13.10 - Retaining Wall E4
MSE Wall Settlement - Sta. 402+02 to 404+75

Calculated By: BRT Date: 7/18/2019
Checked By: JPS Date: 7/18/2019

Boring B-020-4-13

H=	27.7	ft	Total wall height
B'=	14.9	ft	Effective footing width due to eccentricity
D_w =	0.0	ft	Depth below bottom of footing
q_e =	4,640	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)	$\sigma_p'^{(1)}$ (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)	$I^{(7)}$	$\Delta\sigma_v^{(8)}$ (psf)	σ_{vf}' Midpoint (psf)	$S_c^{(9,10)}$ (ft)	S_c (in)
1	A-6a	C	0.0	2.5	2.5	1.3	115	288	144	66	2,066	32	0.198	0.030	0.522				0.08	0.998	4,631	4,697	0.189	2.268	0.500	2,319	2,385	0.093	1.120
	A-6a	C	2.5	5.0	2.5	3.8	115	575	431	197	2,197	32	0.198	0.030	0.522				0.25	0.959	4,449	4,646	0.157	1.882	0.497	2,305	2,503	0.069	0.833
	A-6a	C	5.0	8.0	3.0	6.5	115	920	748	342	2,342	32	0.198	0.030	0.522				0.44	0.859	3,984	4,326	0.153	1.835	0.486	2,254	2,596	0.066	0.796
2	A-7-6	C	8.0	10.5	2.5	9.3	120	1,220	1,070	493	2,493	48	0.342	0.034	0.647				0.62	0.743	3,446	3,939	0.140	1.676	0.466	2,161	2,654	0.051	0.608
	A-7-6	C	10.5	13.0	2.5	11.8	120	1,520	1,370	637	2,637	48	0.342	0.034	0.647				0.79	0.648	3,005	3,642	0.105	1.258	0.442	2,052	2,689	0.036	0.437
3	A-2-6	G	13.0	15.5	2.5	14.3	120	1,820	1,670	781	2,781					7	9	56	0.96	0.568	2,636	3,417	0.029	0.346	0.416	1,931	2,711	0.024	0.292
	A-2-6	G	15.5	18.0	2.5	16.8	120	2,120	1,970	925	2,925					7	9	55	1.12	0.503	2,333	3,258	0.025	0.298	0.389	1,807	2,732	0.021	0.256
4	A-1-b	G	18.0	23.0	5.0	20.5	130	2,770	2,445	1,166	5,166					34	40	131	1.38	0.426	1,978	3,144	0.016	0.197	0.351	1,631	2,796	0.014	0.174
	A-1-b	G	23.0	28.0	5.0	25.5	130	3,420	3,095	1,504	5,504					34	37	121	1.71	0.352	1,635	3,139	0.013	0.158	0.307	1,424	2,928	0.012	0.143
5	A-1-a	G	28.0	37.0	9.0	32.5	125	4,545	3,983	1,955	5,955					18	18	71	2.18	0.282	1,309	3,263	0.028	0.341	0.257	1,194	3,149	0.026	0.317
6	A-1-a	G	37.0	47.0	10.0	42.0	135	5,895	5,220	2,599	6,599					88	80	331	2.82	0.221	1,027	3,626	0.004	0.052	0.209	969	3,568	0.004	0.050
	A-1-a	G	47.0	57.0	10.0	52.0	135	7,245	6,570	3,325	7,325					88	73	286	3.49	0.180	835	4,160	0.003	0.041	0.173	803	4,128	0.003	0.039
7	A-6b	C	57.0	64.5	7.5	60.8	130	8,220	7,733	3,942	7,942	40	0.270	0.027	0.585				4.08	0.155	717	4,659	0.009	0.111	0.150	697	4,639	0.009	0.108
																				Total Settlement:			10.464 in		Total Settlement:			5.175 in	

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

W-13-072 - FRA-70-13.10 - Retaining Wall E4

MSE Wall Settlement - Sta. 402+02 to 404+75

Calculated By: BRT Date: 07/18/2019

Checked By: JPS Date: 07/18/2019

Boring B-020-4-13

H=	27.7	ft	Total wall height
B'=	14.9	ft	Effective footing width due to eccentricity
D_w =	0.0	ft	Depth below bottom of footing
q_e =	4,640	psf	Equivalent bearing pressure at bottom of wall

	A-6a	A-7-6	A-6b			
c_v =	600	150	300		ft^2/yr	Coefficient of consolidation
t =	23	23	23		days	Time following completion of construction
H_{dr} =	5.5	5.5	7.5		ft	Length of longest drainage path considered
T_v =	1.250	0.312	0.336			Time factor
U =	96	63	65		%	Degree of consolidation

$(S_c)_t$ = 4.640 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-6a	C	0.0	2.5	2.5	1.3	115	288	144	66	2,066	32	0.198	0.030	0.522				0.08	0.500	2,319	2,385	0.093	1.120	2.750	1.075	2.640
	A-6a	C	2.5	5.0	2.5	3.8	115	575	431	197	2,197	32	0.198	0.030	0.522				0.25	0.497	2,305	2,503	0.069	0.833		0.800	
	A-6a	C	5.0	8.0	3.0	6.5	115	920	748	342	2,342	32	0.198	0.030	0.522				0.44	0.486	2,254	2,596	0.066	0.796		0.764	
2	A-7-6	C	8.0	10.5	2.5	9.3	120	1,220	1,070	493	2,493	48	0.342	0.034	0.647				0.62	0.466	2,161	2,654	0.051	0.608	1.045	0.383	0.658
	A-7-6	C	10.5	13.0	2.5	11.8	120	1,520	1,370	637	2,637	48	0.342	0.034	0.647				0.79	0.442	2,052	2,689	0.036	0.437		0.275	
3	A-2-6	G	13.0	15.5	2.5	14.3	120	1,820	1,670	781	4,781					7	9	56	0.96	0.416	1,931	2,711	0.024	0.292	0.548	0.292	0.548
	A-2-6	G	15.5	18.0	2.5	16.8	120	2,120	1,970	925	4,925					7	9	55	1.12	0.389	1,807	2,732	0.021	0.256		0.256	
4	A-1-b	G	18.0	23.0	5.0	20.5	130	2,770	2,445	1,166	5,166					34	40	131	1.38	0.351	1,631	2,796	0.014	0.174	0.317	0.174	0.317
	A-1-b	G	23.0	28.0	5.0	25.5	130	3,420	3,095	1,504	5,504					34	37	121	1.71	0.307	1,424	2,928	0.012	0.143		0.143	
5	A-1-a	G	28.0	37.0	9.0	32.5	125	4,545	3,983	1,955	5,955					18	18	71	2.18	0.257	1,194	3,149	0.026	0.317	0.317	0.317	0.317
6	A-1-a	G	37.0	47.0	10.0	42.0	135	5,895	5,220	2,599	6,599					88	80	331	2.82	0.209	969	3,568	0.004	0.050	0.089	0.050	0.089
	A-1-a	G	47.0	57.0	10.0	52.0	135	7,245	6,570	3,325	7,325					88	73	286	3.49	0.173	803	4,128	0.003	0.039		0.039	
7	A-6b	C	57.0	64.5	7.5	60.8	130	8,220	7,733	3,942	7,942	40	0.270	0.027	0.585				4.08	0.150	697	4,639	0.009	0.108	0.108	0.070	0.070

Settlement Remaining After Hold Period: 0.535 in

1. $\sigma_p' = \sigma_{vo}'+\sigma_{m;}$ Estimate σ_m of 2,000 psf in existing fill material and 4,000 psf (moderately overconsolidated) for natural soil deposits; Ref. Table 11.2, Coduto 2003
2. $C_c = 0.009(LL-10)$; Ref. Table 6-9, FHWA GEC 5
3. $C_r = 0.15(Cc)$ for the existing fill and $0.10(Cc)$ for the natural soil deposits; Ref. Section 8.11, Holtz and Kovacs 1981
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_nN_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \le 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$for $\sigma_p' \le \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \le \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (Cohesiv soil layers)
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)
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 E4
MSE Wall Settlement - Sta. 402+02 to 404+75

Calculated By: BRT Date: 7/18/2019
Checked By: JPS Date: 7/18/2019

Boring B-114-9-13

H_{Embank}=	13.0	ft	Height of embankment below wall to existing grade
H_{Wall}=	20.0	ft	Wall height
B=	33.0	ft	Width equal to total height of wall and embankment
D_w =	0.0	ft	Depth below bottom of embankment
q_{Embank} =	1,560	psf	Pressure due to embankment fill below wall
q_{Wall} =	3,490	psf	Equivalent bearing pressure at bottom of wall
q_e =	5,050	psf	Total pressure at bottom of embankment

																			Total Settlement at Center of Reinforced Soil Mass					Total Settlement at Facing of Wall				
Layer	Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
1	A-2-4	G	0.0	2.0	2.0	1.0	125	250	125	63	2,063					13	26	88	0.03	1.000	5,050	5,112	0.043	0.520	0.500	2,525	2,588	0.037	0.440
	A-2-4	G	2.0	4.0	2.0	3.0	125	500	375	188	2,188					13	23	82	0.09	0.998	5,038	5,225	0.035	0.425	0.500	2,524	2,712	0.028	0.341
	A-2-4	G	4.0	6.0	2.0	5.0	125	750	625	313	2,313					13	21	77	0.15	0.989	4,996	5,309	0.032	0.385	0.499	2,521	2,834	0.025	0.300
	A-2-4	G	6.0	8.0	2.0	7.0	125	1,000	875	438	2,438					13	20	73	0.21	0.973	4,916	5,354	0.030	0.355	0.498	2,515	2,953	0.023	0.271
2	A-6b	C	8.0	10.5	2.5	9.3	115	1,288	1,144	567	2,567	37	0.243	0.036	0.561				0.28	0.946	4,779	5,345	0.162	1.947	0.496	2,503	3,070	0.069	0.823
	A-6b	C	10.5	13.0	2.5	11.8	115	1,575	1,431	698	2,698	37	0.243	0.036	0.561				0.36	0.907	4,580	5,278	0.148	1.772	0.492	2,483	3,181	0.062	0.745
3	A-1-b	G	13.0	15.5	2.5	14.3	125	1,888	1,731	842	4,842					16	21	76	0.43	0.861	4,350	5,192	0.026	0.313	0.486	2,455	3,297	0.020	0.235
	A-1-b	G	15.5	18.0	2.5	16.8	125	2,200	2,044	999	4,999					16	20	74	0.51	0.813	4,108	5,107	0.024	0.288	0.479	2,419	3,417	0.018	0.217
	A-1-b	G	18.0	20.5	2.5	19.3	125	2,513	2,356	1,155	5,155					16	19	72	0.58	0.766	3,867	5,022	0.022	0.266	0.470	2,376	3,531	0.017	0.202
	A-1-b	G	20.5	23.0	2.5	21.8	125	2,825	2,669	1,312	5,312					16	18	71	0.66	0.720	3,634	4,946	0.020	0.244	0.461	2,327	3,638	0.016	0.188
4	A-1-a	G	23.0	27.5	4.5	25.3	130	3,410	3,118	1,542	5,542					41	45	148	0.77	0.660	3,333	4,875	0.015	0.183	0.446	2,251	3,793	0.012	0.143
	A-1-a	G	27.5	32.0	4.5	29.8	130	3,995	3,703	1,846	5,846					41	42	138	0.90	0.592	2,992	4,838	0.014	0.163	0.425	2,145	3,991	0.011	0.131
5	A-3a	G	32.0	37.0	5.0	34.5	135	4,670	4,333	2,180	6,180					100	97	338	1.05	0.532	2,686	4,865	0.005	0.062	0.402	2,030	4,209	0.004	0.051
6	A-4a	C	37.0	42.0	5.0	39.5	130	5,320	4,995	2,530	6,530	25	0.135	0.014	0.467				1.20	0.478	2,416	4,946	0.013	0.161	0.378	1,910	4,440	0.011	0.135
7	A-3a	G	42.0	47.0	5.0	44.5	135	5,995	5,658	2,881	6,881					51	45	125	1.35	0.434	2,190	5,070	0.010	0.118	0.355	1,795	4,676	0.008	0.101
8	A-7-6	C	47.0	51.0	4.0	49.0	125	6,495	6,245	3,187	7,187	42	0.288	0.029	0.600				1.48	0.399	2,017	5,204	0.015	0.184	0.336	1,698	4,885	0.013	0.160
	A-7-6	C	51.0	55.0	4.0	53.0	125	6,995	6,745	3,438	7,438	42	0.288	0.029	0.600				1.61	0.373	1,883	5,321	0.014	0.164	0.320	1,616	5,054	0.012	0.145
	A-7-6	C	55.0	59.0	4.0	57.0	125	7,495	7,245	3,688	7,688	42	0.288	0.029	0.600				1.73	0.349	1,765	5,453	0.012	0.147	0.305	1,541	5,229	0.011	0.131
																				Total Settlement:			7.697 in		Total Settlement:			4.758 in	

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

W-13-072 - FRA-70-13.10 - Retaining Wall E4
MSE Wall Settlement - Sta. 402+02 to 404+75

Calculated By: BRT Date: 07/18/2019
Checked By: JPS Date: 07/18/2019

Boring B-114-9-13

H_{Embank}=	13.0	ft	Height of embankment below wall to existing grade
H_{Wall}=	20.0	ft	Wall height
B=	33.0	ft	Width equal to total height of wall and embankment
D_w =	0.0	ft	Depth below bottom of embankment
q_{Embank} =	1,560	psf	Pressure due to embankment fill below wall
q_{Wall} =	3,490	psf	Equivalent bearing pressure at bottom of wall
q_e =	5,050	psf	Total pressure at bottom of embankment

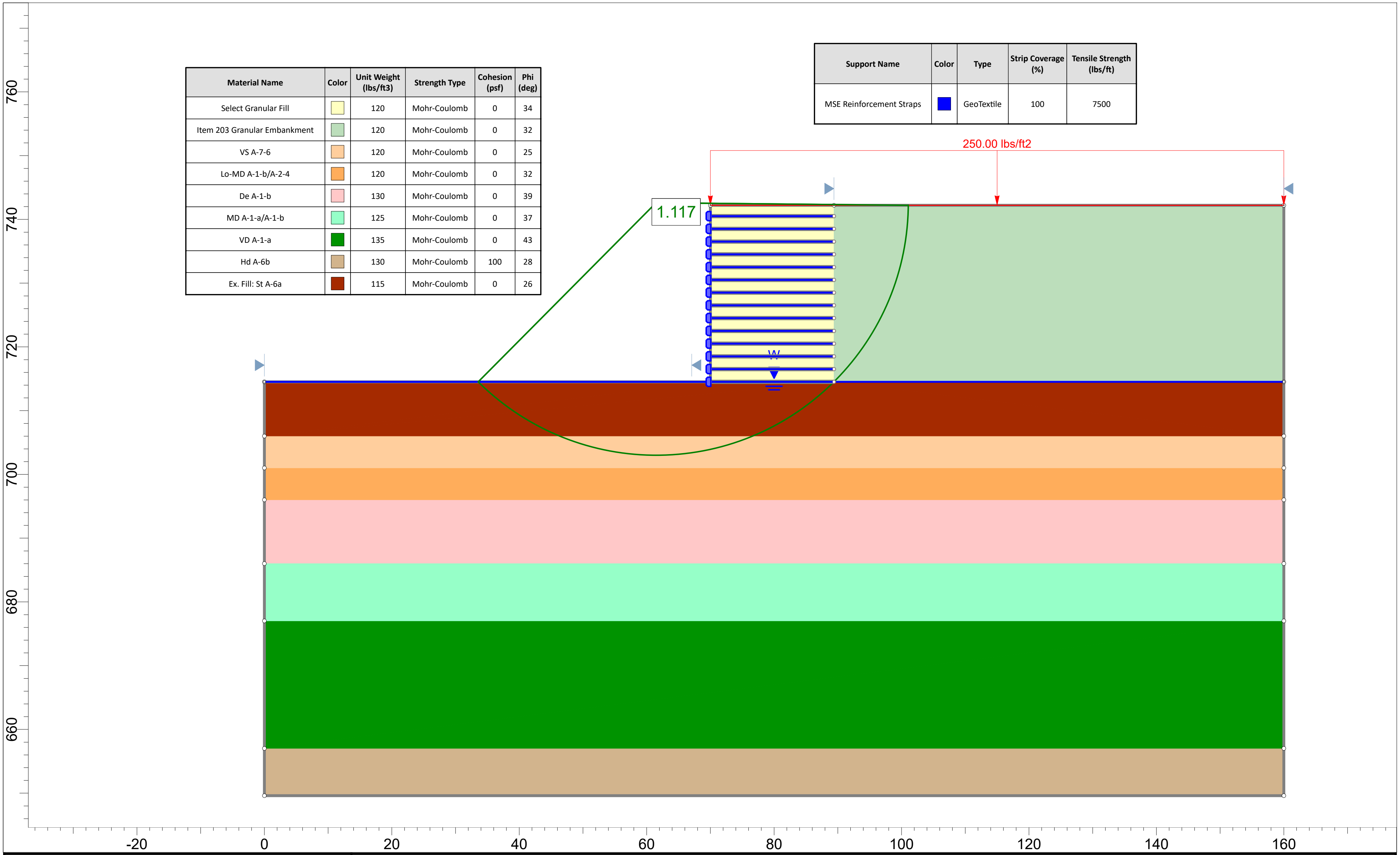
	A-6b	A-4a	A-7-6		
c_v =	300	1000	150	ft^2/yr	Coefficient of consolidation
t =	8	8	8	days	Time following completion of construction
H_{dr} =	2.5	2.5	12	ft	Length of longest drainage path considered
T_v =	1.052	3.507	0.023		Time factor
U =	94	100	17	%	Degree of consolidation

$(S_c)_t$ = 4.302 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-2-4	G	0.0	2.0	2.0	1.0	125	250	125	63	2,063					13	26	88	0.05	0.500	2,525	2,588	0.037	0.440	1.351	0.440	1.351
	A-2-4	G	2.0	4.0	2.0	3.0	125	500	375	188	2,188					13	23	82	0.15	0.500	2,524	2,712	0.028	0.341		0.341	
	A-2-4	G	4.0	6.0	2.0	5.0	125	750	625	313	2,313					13	21	77	0.25	0.499	2,521	2,834	0.025	0.300		0.300	
	A-2-4	G	6.0	8.0	2.0	7.0	125	1,000	875	438	2,438					13	20	73	0.35	0.498	2,515	2,953	0.023	0.271		0.271	
2	A-6b	C	8.0	10.5	2.5	9.3	115	1,288	1,144	567	2,567	37	0.243	0.036	0.561				0.46	0.496	2,503	3,070	0.069	0.823	1.568	0.773	1.474
	A-6b	C	10.5	13.0	2.5	11.8	115	1,575	1,431	698	2,698	37	0.243	0.036	0.561				0.59	0.492	2,483	3,181	0.062	0.745		0.701	
3	A-1-b	G	13.0	15.5	2.5	14.3	125	1,888	1,731	842	4,842					16	21	76	0.71	0.486	2,455	3,297	0.020	0.235	0.842	0.235	0.842
	A-1-b	G	15.5	18.0	2.5	16.8	125	2,200	2,044	999	4,999					16	20	74	0.84	0.479	2,419	3,417	0.018	0.217		0.217	
	A-1-b	G	18.0	20.5	2.5	19.3	125	2,513	2,356	1,155	5,155					16	19	72	0.96	0.470	2,376	3,531	0.017	0.202		0.202	
	A-1-b	G	20.5	23.0	2.5	21.8	125	2,825	2,669	1,312	5,312					16	18	71	1.09	0.461	2,327	3,638	0.016	0.188		0.188	
4	A-1-a	G	23.0	27.5	4.5	25.3	130	3,410	3,118	1,542	5,542					41	45	148	1.26	0.446	2,251	3,793	0.012	0.143	0.274	0.143	0.274
	A-1-a	G	27.5	32.0	4.5	29.8	130	3,995	3,703	1,846	5,846					41	42	138	1.49	0.425	2,145	3,991	0.011	0.131		0.131	
5	A-3a	G	32.0	37.0	5.0	34.5	135	4,670	4,333	2,180	6,180					100	97	338	1.73	0.402	2,030	4,209	0.004	0.051	0.051	0.051	0.051
6	A-4a	C	37.0	42.0	5.0	39.5	130	5,320	4,995	2,530	6,530	25	0.135	0.014	0.467				1.98	0.378	1,910	4,440	0.011	0.135	0.135	0.135	0.135
7	A-3a	G	42.0	47.0	5.0	44.5	135	5,995	5,658	2,881	6,881					51	45	125	2.23	0.355	1,795	4,676	0.008	0.101	0.101	0.101	0.101
8	A-7-6	C	47.0	51.0	4.0	49.0	125	6,495	6,245	3,187	7,187	42	0.288	0.029	0.600				2.45	0.336	1,698	4,885	0.013	0.160	0.436	0.027	0.074
	A-7-6	C	51.0	55.0	4.0	53.0	125	6,995	6,745	3,438	7,438	42	0.288	0.029	0.600				2.65	0.320	1,616	5,054	0.012	0.145		0.025	
	A-7-6	C	55.0	59.0	4.0	57.0	125	7,495	7,245	3,688	7,688	42	0.288	0.029	0.600				2.85	0.305	1,541	5,229	0.011	0.131		0.022	

Settlement Remaining After Hold Period: 0.456 in

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



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Select Granular Fill	🟨	120	Mohr-Coulomb	0	34
Item 203 Granular Embankment	🟩	120	Mohr-Coulomb	0	32
VS A-7-6	🟧	120	Mohr-Coulomb	0	25
Lo-MD A-1-b/A-2-4	🟧	120	Mohr-Coulomb	0	32
De A-1-b	🟥	130	Mohr-Coulomb	0	39
MD A-1-a/A-1-b	🟩	125	Mohr-Coulomb	0	37
VD A-1-a	🟩	135	Mohr-Coulomb	0	43
Hd A-6b	🟫	130	Mohr-Coulomb	100	28
Ex. Fill: St A-6a	🟥	115	Mohr-Coulomb	0	26

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

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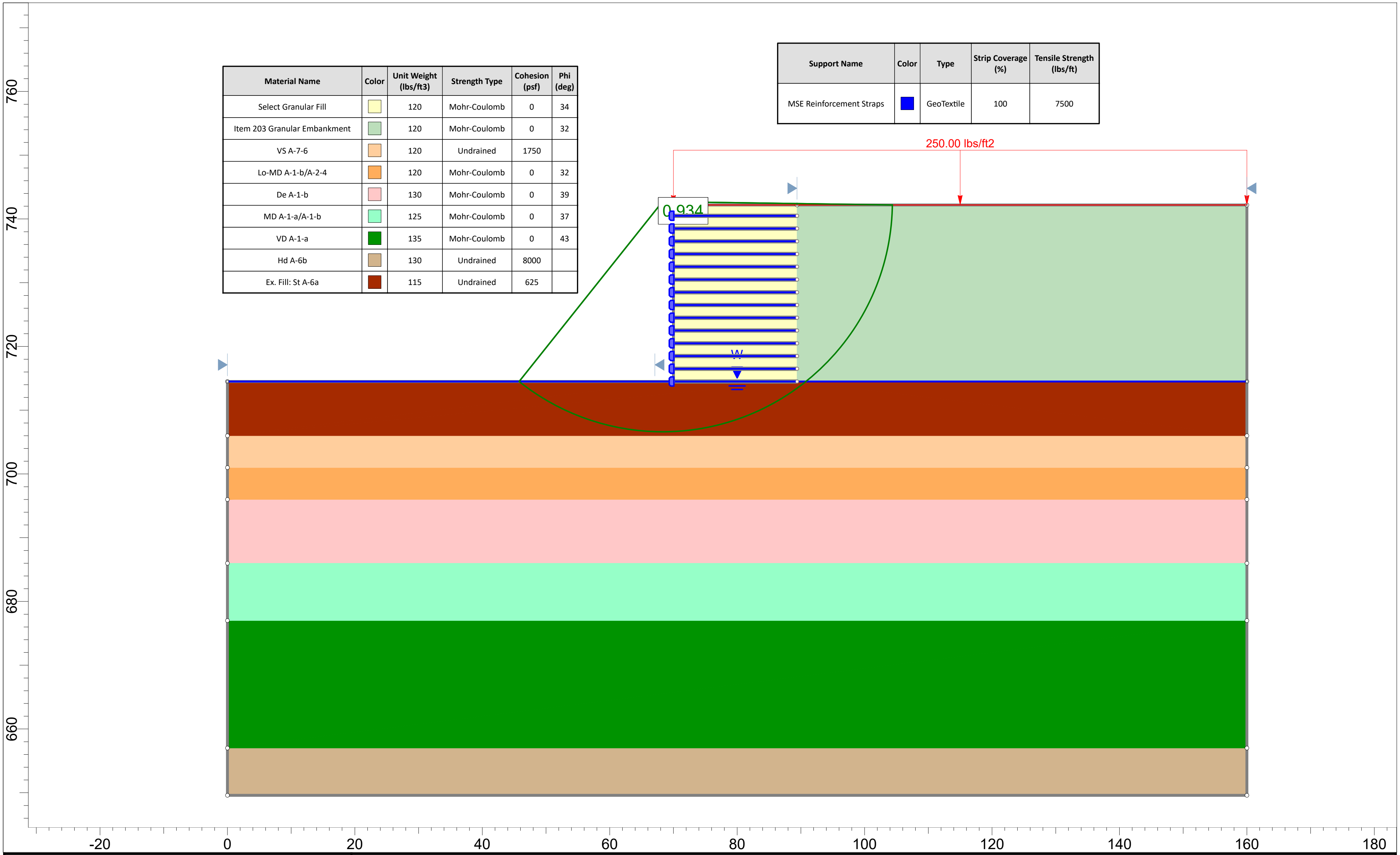
SLIDEINTERPRET 8.020

Project	Retaining Wall E4 - Sta. 402+02 to 404+75 - MSE Wall Global Stability
Analysis Description	27.7 ft Wall Height - Drained - Circular - Spencer's
Drawn By	BRT
Scale	1:250
Company	Resource International, Inc.
Date	7/18/2019
File Name	Retaining Wall E4 - Sta. 402+02 to 404+75 - Global Stability.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Select Granular Fill	🟨	120	Mohr-Coulomb	0	34
Item 203 Granular Embankment	🟩	120	Mohr-Coulomb	0	32
VS A-7-6	🟧	120	Undrained	1750	
Lo-MD A-1-b/A-2-4	🟧	120	Mohr-Coulomb	0	32
De A-1-b	🟥	130	Mohr-Coulomb	0	39
MD A-1-a/A-1-b	🟩	125	Mohr-Coulomb	0	37
VD A-1-a	🟩	135	Mohr-Coulomb	0	43
Hd A-6b	🟫	130	Undrained	8000	
Ex. Fill: St A-6a	🟥	115	Undrained	625	

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

Resource International, Inc.
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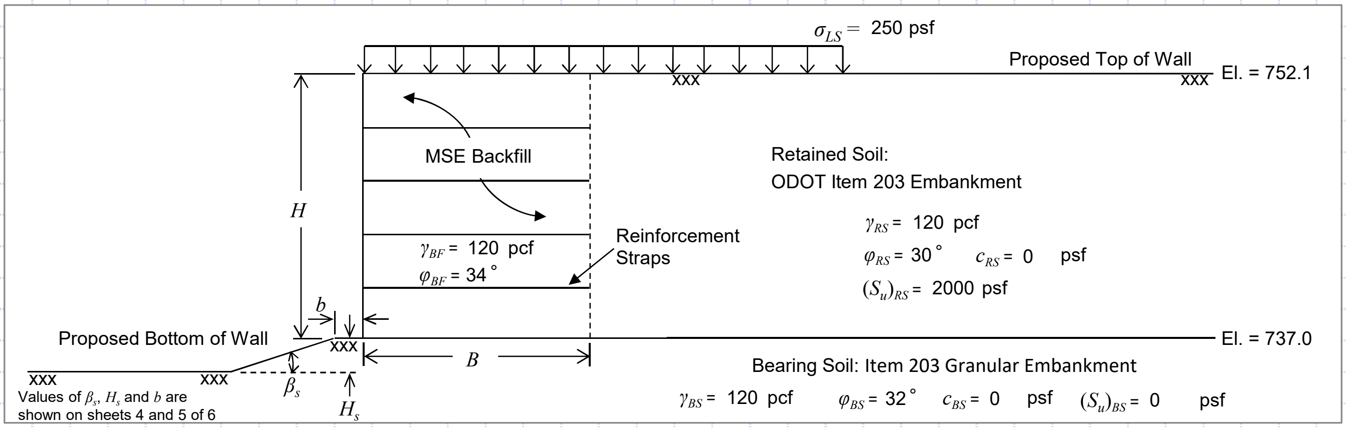
SLIDEINTERPRET 8.020

Project	Retaining Wall E4 - Sta. 402+02 to 404+75 - MSE Wall Global Stability
Analysis Description	27.7 ft Wall Height - Undrained - Circular - Spencer's
Drawn By	BRT
Scale	1:250
Company	Resource International, Inc.
Date	7/18/2019
File Name	Retaining Wall E4 - Sta. 402+02 to 404+75 - Global Stability.slim

Retaining Wall E4 - Sta. 404+75 to 407+00 - B-114-7-13 and B-114-8-13 - 15.1 ft. Wall Height



Values of β_s, H_s and b are shown on sheets 4 and 5 of 6

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) = 15.1 ft
MSE Wall Width (Reinforcement Length), (B) = 10.6 ft
MSE Wall Length, (L) = 750 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}) = 120 pcf
Bearing Soil Friction Angle, (φ_{BS}) = 32 °
Bearing Soil Drained Cohesion, (c_{BS}) = 0 psf
Bearing Soil Undrained Shear Strength, $[(S_u)_{BS}]$ = 0 psf
Embedment Depth, (D_f) = 0.0 ft
Depth to Grounwater (Below Bot. of Wall), (D_W) = 22.5 ft

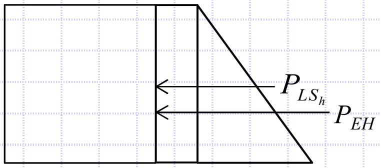
LRFD Load Factors

	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:



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

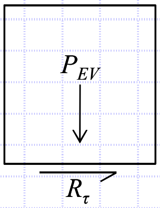
$$P_{EH} = ½\gamma_{RS}H^2K_a\gamma_{EH} = ½(120 \text{ pcf})(15.1 \text{ ft})^2(0.297)(1.5) = 6.09 \text{ kip/ft}$$

$$P_{LS_h} = \sigma_{LS}HK_a\gamma_{LS} = (250 \text{ psf})(15.1 \text{ ft})(0.297)(1.75) = 1.96 \text{ kip/ft}$$

$$P_H = 6.09 \text{ kip/ft} + 1.96 \text{ kip/ft} = 8.05 \text{ kip/ft}$$

Check Sliding Resistance - Drained Condition

Nominal Sliding Resistance:



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

$$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV} = (120 \text{ pcf})(15.1 \text{ ft})(10.6 \text{ ft})(1.00) = 19.21 \text{ kip/ft}$$

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

$$\tan\delta = \tan(32) \le \tan(34) \longrightarrow 0.62 \le 0.67 \longrightarrow \tan\delta = 0.62$$

$$R_\tau = (19.21 \text{ kip/ft})(0.62) = 11.91 \text{ kip/ft}$$

Verify Sliding Force Less Than Factored Sliding Resistance - Drained Condition

$$P_H \le R_\tau \cdot \phi_\tau \longrightarrow 8.05 \text{ kip/ft} \le (11.91 \text{ kip/ft})(1.0) = 11.91 \text{ kip/ft} \longrightarrow 8.05 \text{ kip/ft} \le 11.91 \text{ kip/ft}$$ **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	7/12/2019
CHECKED BY	JPS	DATE	7/12/2019

Retaining Wall E4 - Sta. 404+75 to 407+00

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	15.1	ft
MSE Wall Width (Reinforcement Length), (B) =	10.6	ft
MSE Wall Length, (L) =	750	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}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	32	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	0	psf
Embedment Depth, (D_f) =	0.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	22.5	ft

LRFD Load Factors

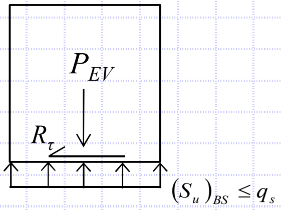
	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

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

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

Check Sliding Resistance - Undrained Condition

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



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

$q_s = \sigma_v/2$ = (1.81 ksf) / 2 = 0.91 ksf

$\sigma_v = P_{EV}/B$ = (19.21 kip/ft) / (10.6 ft) = 1.81 ksf

R_τ = (N/A ksf ≤ 0.91 ksf)(10.6 ft) = N/A kip/ft

Verify Sliding Force Less Than Factored Sliding Resistance - Undrained Condition

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

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

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	15.1	ft
MSE Wall Width (Reinforcement Length), (B) =	10.6	ft
MSE Wall Length, (L) =	750	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:

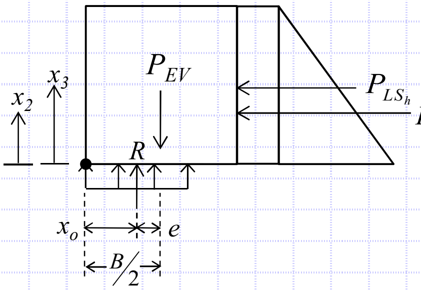
Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	32	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	0	psf
Embedment Depth, (D_f) =	0.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	22.5	ft

LRFD Load Factors

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

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

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



$$e = B/2 - x_o$$

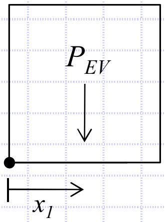
$$x_o = \frac{M_{EV} - M_H}{P_{EV}} = \text{(101.81 kip·ft/ft - 45.43 kip·ft/ft) / (19.21 kip/ft)} = 2.93 \text{ ft}$$

M_{EV} = 101.81 kip·ft/ft
M_H = 45.43 kip·ft/ft
P_{EV} = 19.21 kip/ft

Defined below

$$e = \text{(10.6 ft)/2 - 2.93 ft} = 2.37 \text{ ft}$$

Resisting Moment, M_{EV}:



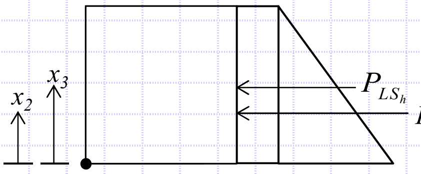
$$M_{EV} = P_{EV}(x_1)$$

$$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV} = \text{(120 pcf)(15.1 ft)(10.6 ft)(1.00)} = 19.21 \text{ kip/ft}$$

$$x_1 = B/2 = \text{(10.6 ft) / 2} = 5.30 \text{ ft}$$

$$M_{EV} = \text{(19.21 kip/ft)(5.30 ft)} = 101.81 \text{ kip·ft/ft}$$

Overturning Moment, M_H:



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

$$P_{EH} = \tfrac{1}{2}\gamma_{RS} H^2 K_a \gamma_{EH} = \text{½(120 pcf)(15.1 ft)²(0.297)(1.5)} = 6.09 \text{ kip/ft}$$

$$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS} = \text{(250 psf)(15.1 ft)(0.297)(1.75)} = 1.96 \text{ kip/ft}$$

$$x_2 = H/3 = \text{(15.1 ft) / 3} = 5.03 \text{ ft}$$

$$x_3 = H/2 = \text{(15.1 ft) / 2} = 7.55 \text{ ft}$$

$$M_H = \text{(6.09 kip/ft)(5.03 ft) + (1.96 kip/ft)(7.55 ft)} = 45.43 \text{ kip·ft/ft}$$

Check Eccentricity

$e < e_{\max} \longrightarrow$ 2.37 ft < 3.53 ft **OK**

Limiting Eccentricity: $e_{\max} = B/3 \longrightarrow e_{\max} =$ (10.6 ft) / 3 = 3.53 ft

JOB	FRA-70-13.10	NO.	W-13-072
SHEET NO.	4	OF	6
CALCULATED BY	BRT	DATE	7/12/2019
CHECKED BY	JPS	DATE	7/12/2019

Retaining Wall E4 - Sta. 404+75 to 407+00

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	15.1	ft
MSE Wall Width (Reinforcement Length), (B) =	10.6	ft
MSE Wall Length, (L) =	750	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:

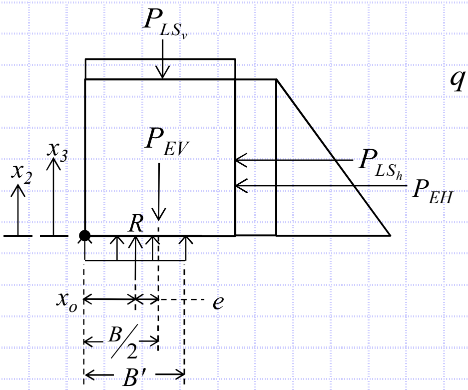
Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	32	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	0	psf
Embedment Depth, (D_f) =	0.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	22.5	ft

LRFD Load Factors

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

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

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



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

$B' = B - 2e$ = 10.6 ft - 2(1.49 ft) = 7.62 ft

$e = B/2 - x_o$ = (10.6 ft) / 2 - 3.81 ft = 1.49 ft

$x_o = \dfrac{M_V - M_H}{P_V}$ = (162.01 kip·ft/ft - 45.47 kip·ft/ft) / 30.57 kip/ft = 3.81 ft

q_{eq} = (30.57 kip/ft) / (7.62 ft) = 4.01 ksf

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

M_V = [(120 pcf)(15.1 ft)(10.6 ft)(1.35)](5.3 ft) + [(250 psf)(10.6 ft)(1.75)](5.3 ft) = 162.01 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)(15.1 ft)²(0.297)(1.5)](5.03 ft) + [(250 psf)(15.1 ft)(0.297)(1.75)](7.55 ft) = 45.47 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)(15.1 ft)(10.6 ft)(1.35) + (250 psf)(10.6 ft)(1.75) = 30.57 kip/ft

Check Bearing Resistance - Drained Condition

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

$N_{cm} = N_c s_c i_c$ = 35.74

N_c = 35.49
s_c = 1+(7.62 ft/750 ft)(23.18/35.49)
= 1.007
i_c = 1.000 (Assumed)

$N_{qm} = N_q s_q d_q i_q$ = 23.32

N_q = 23.18
s_q = 1.006
d_q = 1+2tan(32°)[1-sin(32°)]²tan⁻¹(0.0 ft/7.62 ft)
1.000
i_q = 1.000 (Assumed)
C_{wq} = 22.5 ft > 0.0 ft = 1.000

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 30.09

N_γ = 30.21
s_γ = 0.996
i_γ = 1.000 (Assumed)
$C_{w\gamma}$ = 22.5 ft > 1.5(7.62 ft) + 22.5 ft = 1.000

q_n = (0 psf)(35.738) + (120 pcf)(0.0 ft)(23.319)(1.000) + ½(120 pcf)(7.6 ft)(30.089)(1.000) = 13.76 ksf

Use β_s = 26.6 ° H_s = 22.0 ft b = 15.0 ft

Verify Equivalent Pressure Less Than Factored Bearing Resistance

RC_{BC} = **0.58** *(Per AASHTO LRFD BDM Section 10.6.3.1.2c)*

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

$q_{eq} \le q_n \cdot RC_{BC} \cdot \phi_b \longrightarrow$ 4.01 ksf ≤ (13.76 ksf)(0.58)(0.65) = 5.19 ksf $\longrightarrow$ 4.01 ksf ≤ 5.19 ksf **OK**

JOB FRA-70-13.10 NO. W-13-072
SHEET NO. 5 OF 6
CALCULATED BY BRT DATE 7/12/2019
CHECKED BY JPS DATE 7/12/2019
Retaining Wall E4 - Sta. 404+75 to 407+00

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	15.1	ft
MSE Wall Width (Reinforcement Length), (B) =	10.6	ft
MSE Wall Length, (L) =	750	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}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	32	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [(s_u)$_{BS}$] =	0	psf
Embedment Depth, (D_f) =	0.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	22.5	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: $q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \frac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$

$N_{cm} = N_c s_c i_c$ = 5.150

N_c = 5.140
s_c = 1+(7.62 ft/[(5)(750 ft)]) = 1.002
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(0.0 ft/7.62 ft)
1.000
i_q = 1.000 (Assumed)
C_{wq} = 22.5 ft > 0.0 ft = 1.000

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

N_γ = 0.000
s_γ = 1.000
i_γ = 1.000 (Assumed)
$C_{w\gamma}$ = 22.5 ft > 1.5(7.62 ft) + 22.5 ft = 1.000

$q_n =$ (0 psf)(5.150) + (120 pcf)(0.0 ft)(1.000)(1.000) + ½(120 pcf)(7.6 ft)(0.000)(1.000) = N/A ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

$q_{eq} \leq q_n \cdot RC_{BC} \cdot \phi_b \longrightarrow$ 4.01 ksf ≤ (N/A ksf)(N/A)(0.65) = N/A ksf $\longrightarrow$ N/A

RC_{BC} = **N/A** *(Per AASHTO LRFD BDM Section 10.6.3.1.2c)* $\longrightarrow$ Use β_s = 26.6 ° H_s = 22.0 ft b = 15.0 ft
Use φ_b = **0.65** *(Per AASHTO LRFD BDM Table 11.5.7-1)*

JOB FRA-70-13.10 NO. W-13-072
SHEET NO. 6 OF 6
CALCULATED BY BRT DATE 7/12/2019
CHECKED BY JPS DATE 7/12/2019
Retaining Wall E4 - Sta. 404+75 to 407+00

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	15.1 ft
MSE Wall Width (Reinforcement Length), (B) =	10.6 ft
MSE Wall Length, (L) =	750 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:

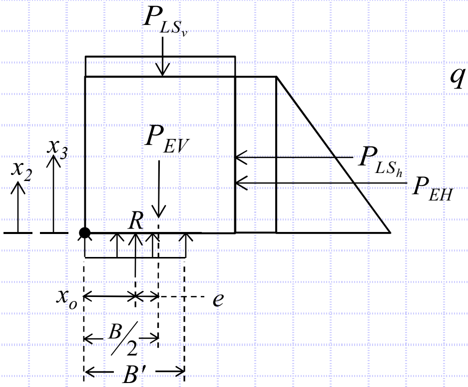
Bearing Soil Unit Weight, (γ_{BS}) =	120 pcf
Bearing Soil Friction Angle, (φ_{BS}) =	32 °
Bearing Soil Drained Cohesion, (c_{BS}) =	0 psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	0 psf
Embedment Depth, (D_f) =	0.0 ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	22.5 ft

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



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

$B' = B - 2e$ = 10.6 ft - 2(1.32 ft) = 7.96 ft

$e = B/2 - x_o$ = (10.6 ft) / 2 - 3.98 ft = 1.32 ft

$x_o = \frac{M_V - M_H}{P_V}$ = (115.84 kip·ft/ft - 28.9 kip·ft/ft) / 21.86 kip/ft = 3.98 ft

q_{eq} = (21.86 kip/ft) / (7.96 ft) = 2.75 ksf

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

M_V = [(120 pcf)(15.1 ft)(10.6 ft)(1.00)](5.3 ft) + [(250 psf)(10.6 ft)(1.00)](5.3 ft) = 115.84 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)(15.1 ft)²(0.297)(1.00)](5.03 ft) + [(250 psf)(15.1 ft)(0.297)(1.00)](7.55 ft) = 28.90 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)(15.1 ft)(10.6 ft)(1.00) + (250 psf)(10.6 ft)(1.00) = 21.86 kip/ft

Settlement, Time Rate of Consolidation and Differential Settlement:

Boring	Total Settlement at Center of Reinforced Soil Mass	Total Settlement at Wall Facing	Time for 90% Consolidation	Distance Between Borings Along Wall Facing	Differential Settlement Along Wall Facing
B-020-4-13	10.464 in	5.175 in	23 days		
B-114-9-13	7.697 in	4.758 in	8 days	160 ft	1/4600
B-114-8-13	15.594 in	8.003 in	90 days	175 ft	1/650
B-114-7-13	15.451 in	8.380 in	100 days	215 ft	1/6840
B-017-2-09	9.057 in	5.790 in	55 days	90 ft	1/420
B-018-2-13	12.067 in	7.604 in	100 days	100 ft	1/660

W-13-072 - FRA-70-13.10 - Retaining Wall E4
MSE Wall Settlement - Sta. 404+75 to 407+00

Calculated By: BRT Date: 7/18/2019
Checked By: JPS Date: 7/18/2019

Boring B-114-7-13

H_{Embank}= 23.7 ft Height of embankment below wall to existing grade
H_{Wall}= 15.1 ft Wall height
B= 38.8 ft Width equal to total height of wall and embankment
D_w = 0.0 ft Depth below bottom of embankment
q_{Embank} = 2,844 psf Pressure due to embankment fill below wall
q_{Wall} = 2,750 psf Equivalent bearing pressure at bottom of wall
q_e = 5,594 psf Total pressure at bottom of embankment

																			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)	$\sigma_p'^{(1)}$ (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)	$I^{(7)}$	$\Delta\sigma_v^{(8)}$ (psf)	σ_{vf}' Midpoint (psf)	$S_c^{(9,10)}$ (ft)	S_c (in)
1	A-6a	C	0.0	2.5	2.5	1.3	125	313	156	78	2,078	33	0.207	0.031	0.530				0.03	1.000	5,593	5,672	0.220	2.637	0.500	2,797	2,875	0.120	1.439
	A-6a	C	2.5	5.5	3.0	4.0	125	688	500	250	2,250	33	0.207	0.031	0.530				0.10	0.996	5,574	5,825	0.226	2.708	0.500	2,796	3,046	0.111	1.337
2	A-7-6	C	5.5	8.3	2.8	6.9	120	1,024	856	425	2,425	41	0.279	0.042	0.593				0.18	0.983	5,501	5,926	0.246	2.952	0.499	2,791	3,216	0.116	1.389
	A-7-6	C	8.3	10.5	2.2	9.4	120	1,288	1,156	569	2,569	41	0.279	0.042	0.593				0.24	0.963	5,385	5,954	0.179	2.142	0.497	2,781	3,350	0.082	0.987
	A-7-6	C	10.5	13.0	2.5	11.8	120	1,588	1,438	704	2,704	41	0.279	0.042	0.593				0.30	0.935	5,233	5,937	0.188	2.255	0.495	2,767	3,472	0.086	1.031
3	A-2-6	G	13.0	15.5	2.5	14.3	120	1,888	1,738	848	2,848					8	10	57	0.37	0.900	5,037	5,885	0.037	0.442	0.491	2,747	3,595	0.027	0.330
4	A-1-a	G	15.5	18.5	3.0	17.0	125	2,263	2,075	1,014	5,014					28	34	112	0.44	0.857	4,797	5,811	0.020	0.244	0.486	2,716	3,730	0.015	0.182
	A-1-a	G	18.5	21.5	3.0	20.0	125	2,638	2,450	1,202	5,202					28	33	107	0.52	0.808	4,523	5,725	0.019	0.228	0.478	2,675	3,877	0.014	0.171
	A-1-a	G	21.5	24.5	3.0	23.0	125	3,013	2,825	1,390	5,390					28	31	103	0.59	0.760	4,250	5,640	0.018	0.212	0.469	2,625	4,015	0.013	0.161
	A-1-a	G	24.5	28.0	3.5	26.3	125	3,450	3,231	1,593	5,593					28	30	99	0.68	0.709	3,969	5,562	0.019	0.229	0.458	2,564	4,158	0.015	0.176
5	A-1-b	G	28.0	32.0	4.0	30.0	135	3,990	3,720	1,848	5,848					77	79	323	0.77	0.656	3,668	5,516	0.006	0.071	0.445	2,487	4,335	0.005	0.055
	A-1-b	G	32.0	37.0	5.0	34.5	135	4,665	4,328	2,175	6,175					77	75	297	0.89	0.598	3,346	5,520	0.007	0.082	0.427	2,387	4,562	0.005	0.065
6	A-1-b	G	37.0	42.0	5.0	39.5	125	5,290	4,978	2,513	6,513					21	19	73	1.02	0.543	3,035	5,548	0.024	0.282	0.406	2,273	4,785	0.019	0.230
7	A-6b	C	42.0	47.0	5.0	44.5	125	5,915	5,603	2,826	6,826	31	0.189	0.019	0.514				1.15	0.495	2,769	5,594	0.019	0.222	0.386	2,159	4,985	0.015	0.185
	A-6b	C	47.0	52.0	5.0	49.5	125	6,540	6,228	3,139	7,139	31	0.189	0.019	0.514				1.28	0.454	2,540	5,679	0.016	0.193	0.366	2,048	5,187	0.014	0.163
8	A-7-6	C	52.0	56.0	4.0	54.0	130	7,060	6,800	3,430	7,430	41	0.279	0.028	0.593				1.39	0.422	2,361	5,792	0.016	0.191	0.349	1,953	5,384	0.014	0.165
	A-7-6	C	56.0	60.0	4.0	58.0	130	7,580	7,320	3,701	7,701	41	0.279	0.028	0.593				1.49	0.397	2,221	5,922	0.014	0.172	0.335	1,873	5,574	0.012	0.150
	A-7-6	C	60.0	64.9	4.9	62.5	130	8,217	7,899	4,002	8,002	41	0.279	0.028	0.593				1.61	0.372	2,082	6,083	0.016	0.187	0.320	1,788	5,790	0.014	0.165
																				Total Settlement:			15.451 in		Total Settlement:			8.380 in	

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

W-13-072 - FRA-70-13.10 - Retaining Wall E4
MSE Wall Settlement - Sta. 404+75 to 407+00

Calculated By: BRT Date: 07/18/2019
Checked By: JPS Date: 07/18/2019

Boring B-114-7-13

H_{Embank}=	23.7	ft	Height of embankment below wall to existing grade
H_{Wall}=	15.1	ft	Wall height
B=	38.8	ft	Width equal to total height of wall and embankment
D_w =	0.0	ft	Depth below bottom of embankment
q_{Embank} =	2,844	psf	Pressure due to embankment fill below wall
q_{Wall} =	2,750	psf	Equivalent bearing pressure at bottom of wall
q_e =	5,594	psf	Total pressure at bottom of embankment

	A-6a	A-7-6 (Upper)	A-6b	A-7-6 (Lower)		
c_v =	600	150	300	150	ft^2/yr	Coefficient of consolidation
t =	100	100	100	100	days	Time following completion of construction
H_{dr} =	5.5	7.5	10	23	ft	Length of longest drainage path considered
T_v =	5.434	0.731	0.822	0.078		Time factor
U =	100	87	89	31	%	Degree of consolidation

$(S_c)_t$ = 7.568 in Settlement complete at 90% of primary consolidation

																						Total Settlement at Facing of Wall			Settlement Complete at 90% of Primary Consolidation	
Layer	Soil Type	Soil Type	Layer Depth (ft)		Layer Thickness (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	Layer Settlement (in)	$(S_c)_t$ (11) (in)	Layer Settlement (in)
1	A-6a	C	0.0	2.5	2.5	1.3	125	313	156	78	2,078	33	0.207	0.031	0.530				0.08	0.500	2,797	2,875	0.120	1.439	2.776	1.439	2.776
	A-6a	C	2.5	5.5	3.0	4.0	125	688	500	250	2,250	33	0.207	0.031	0.530				0.26	0.500	2,796	3,046	0.111	1.337		1.337	
2	A-7-6	C	5.5	8.3	2.8	6.9	120	1,024	856	425	2,425	41	0.279	0.042	0.593				0.46	0.499	2,791	3,216	0.116	1.389	3.407	1.209	2.964
	A-7-6	C	8.3	10.5	2.2	9.4	120	1,288	1,156	569	2,569	41	0.279	0.042	0.593				0.62	0.497	2,781	3,350	0.082	0.987		0.859	
	A-7-6	C	10.5	13.0	2.5	11.8	120	1,588	1,438	704	2,704	41	0.279	0.042	0.593				0.78	0.495	2,767	3,472	0.086	1.031		0.897	
3	A-2-6	G	13.0	15.5	2.5	14.3	120	1,888	1,738	848	2,848					8	10	57	0.94	0.491	2,747	3,595	0.027	0.330	0.330	0.330	0.330
4	A-1-a	G	15.5	18.5	3.0	17.0	125	2,263	2,075	1,014	5,014					28	34	112	1.13	0.486	2,716	3,730	0.015	0.182	0.690	0.182	0.690
	A-1-a	G	18.5	21.5	3.0	20.0	125	2,638	2,450	1,202	5,202					28	33	107	1.32	0.478	2,675	3,877	0.014	0.171		0.171	
	A-1-a	G	21.5	24.5	3.0	23.0	125	3,013	2,825	1,390	5,390					28	31	103	1.52	0.469	2,625	4,015	0.013	0.161		0.161	
	A-1-a	G	24.5	28.0	3.5	26.3	125	3,450	3,231	1,593	5,593					28	30	99	1.74	0.458	2,564	4,158	0.015	0.176		0.176	
5	A-1-b	G	28.0	32.0	4.0	30.0	135	3,990	3,720	1,848	5,848					77	79	323	1.99	0.445	2,487	4,335	0.005	0.055	0.120	0.055	0.120
	A-1-b	G	32.0	37.0	5.0	34.5	135	4,665	4,328	2,175	6,175					77	75	297	2.28	0.427	2,387	4,562	0.005	0.065		0.065	
6	A-1-b	G	37.0	42.0	5.0	39.5	125	5,290	4,978	2,513	6,513					21	19	73	2.62	0.406	2,273	4,785	0.019	0.230	0.230	0.230	0.230
7	A-6b	C	42.0	47.0	5.0	44.5	125	5,915	5,603	2,826	6,826	31	0.189	0.019	0.514				2.95	0.386	2,159	4,985	0.015	0.185	0.348	0.164	0.310
	A-6b	C	47.0	52.0	5.0	49.5	125	6,540	6,228	3,139	7,139	31	0.189	0.019	0.514				3.28	0.366	2,048	5,187	0.014	0.163		0.145	
8	A-7-6	C	52.0	56.0	4.0	54.0	130	7,060	6,800	3,430	7,430	41	0.279	0.028	0.593				3.58	0.349	1,953	5,384	0.014	0.165	0.479	0.051	0.149
	A-7-6	C	56.0	60.0	4.0	58.0	130	7,580	7,320	3,701	7,701	41	0.279	0.028	0.593				3.84	0.335	1,873	5,574	0.012	0.150		0.046	
	A-7-6	C	60.0	64.9	4.9	62.5	130	8,217	7,899	4,002	8,002	41	0.279	0.028	0.593				4.14	0.320	1,788	5,790	0.014	0.165		0.051	

Settlement Remaining After Hold Period: 0.812 in

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

W-13-072 - FRA-70-13.10 - Retaining Wall E4
MSE Wall Settlement - Sta. 404+75 to 407+00

Calculated By: BRT Date: 7/18/2019
Checked By: JPS Date: 7/18/2019

Boring B-114-8-13

H_{Embank}=	30.0	ft	Height of embankment below wall to existing grade
H_{Wall}=	9.0	ft	Wall height
B=	39.0	ft	Width equal to total height of wall and embankment
D_w =	0.0	ft	Depth below bottom of embankment
q_{Embank} =	3,600	psf	Pressure due to embankment fill below wall
q_{Wall} =	1,610	psf	Equivalent bearing pressure at bottom of wall
q_e =	5,210	psf	Total pressure at bottom of embankment

																				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)	$\sigma_p'^{(1)}$ (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)	$I^{(7)}$	$\Delta\sigma_v^{(8)}$ (psf)	σ_{vf}' Midpoint (psf)	$S_c^{(9,10)}$ (ft)	S_c (in)
1	A-4a	C	0.0	2.0	2.0	1.0	120	240	120	58	2,058	32	0.198	0.030	0.522				0.03	1.000	5,210	5,267	0.167	2.002	0.500	2,605	2,663	0.090	1.077
	A-4a	C	2.0	5.5	3.5	3.8	120	660	450	216	2,216	32	0.198	0.030	0.522				0.10	0.997	5,195	5,411	0.246	2.947	0.500	2,604	2,820	0.117	1.401
2	A-6b	C	5.5	9.0	3.5	7.3	115	1,063	861	409	2,409	38	0.252	0.038	0.569				0.19	0.981	5,113	5,522	0.267	3.209	0.499	2,598	3,007	0.119	1.429
	A-6b	C	9.0	11.0	2.0	10.0	115	1,293	1,178	554	2,554	38	0.252	0.038	0.569				0.26	0.957	4,985	5,539	0.140	1.680	0.497	2,588	3,141	0.061	0.731
	A-6b	C	11.0	13.0	2.0	12.0	115	1,523	1,408	659	2,659	38	0.252	0.038	0.569				0.31	0.933	4,861	5,519	0.131	1.573	0.494	2,576	3,235	0.057	0.679
	A-6b	C	13.0	15.5	2.5	14.3	115	1,810	1,666	777	2,777	38	0.252	0.038	0.569				0.37	0.902	4,697	5,474	0.152	1.820	0.491	2,559	3,336	0.065	0.783
3	A-1-a	G	15.5	18.0	2.5	16.8	125	2,123	1,966	921	4,921					26	33	107	0.43	0.863	4,496	5,417	0.018	0.216	0.486	2,534	3,455	0.013	0.161
	A-1-a	G	18.0	20.5	2.5	19.3	125	2,435	2,279	1,078	5,078					26	31	103	0.49	0.822	4,285	5,362	0.017	0.203	0.480	2,503	3,580	0.013	0.152
4	A-1-b	G	20.5	24.0	3.5	22.3	130	2,890	2,663	1,274	5,274					30	35	113	0.57	0.774	4,031	5,305	0.019	0.231	0.472	2,459	3,733	0.015	0.174
	A-1-b	G	24.0	28.0	4.0	26.0	130	3,410	3,150	1,528	5,528					30	33	107	0.67	0.715	3,726	5,254	0.020	0.241	0.460	2,395	3,923	0.015	0.184
5	A-1-a	G	28.0	32.5	4.5	30.3	135	4,018	3,714	1,826	5,826					85	88	379	0.78	0.654	3,410	5,236	0.005	0.065	0.444	2,314	4,140	0.004	0.051
	A-1-a	G	32.5	37.0	4.5	34.8	135	4,625	4,321	2,153	6,153					85	83	347	0.89	0.597	3,112	5,264	0.005	0.060	0.426	2,222	4,375	0.004	0.048
6	A-6b	C	37.0	40.9	3.9	39.0	120	5,093	4,859	2,429	6,429	36	0.234	0.023	0.553				1.00	0.550	2,867	5,296	0.020	0.239	0.409	2,133	4,561	0.016	0.193
	A-6b	C	40.9	44.8	3.9	42.9	120	5,561	5,327	2,653	6,653	36	0.234	0.023	0.553				1.10	0.512	2,667	5,320	0.018	0.213	0.393	2,050	4,703	0.015	0.175
7	A-7-6	C	44.8	48.4	3.6	46.6	125	6,011	5,786	2,878	6,878	42	0.288	0.029	0.600				1.19	0.479	2,496	5,374	0.018	0.211	0.378	1,972	4,850	0.015	0.176
	A-7-6	C	48.4	52.0	3.6	50.2	125	6,461	6,236	3,104	7,104	42	0.288	0.029	0.600				1.29	0.451	2,348	5,452	0.016	0.190	0.364	1,899	5,002	0.013	0.161
8	A-7-6	C	52.0	57.5	5.5	54.8	130	7,176	6,819	3,402	7,402	42	0.288	0.029	0.600				1.40	0.419	2,183	5,585	0.021	0.256	0.347	1,810	5,212	0.018	0.220
	A-7-6	C	57.5	63.5	6.0	60.5	130	7,956	7,566	3,791	7,791	42	0.288	0.029	0.600				1.55	0.384	2,003	5,793	0.020	0.239	0.327	1,705	5,496	0.017	0.209
																						Total Settlement:		15.594 in			Total Settlement:		8.003 in

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

W-13-072 - FRA-70-13.10 - Retaining Wall E4
MSE Wall Settlement - Sta. 404+75 to 407+00

Calculated By: BRT Date: 07/18/2019
Checked By: JPS Date: 07/18/2019

Boring B-114-8-13

H_{Embank}=	30.0	ft	Height of embankment below wall to existing grade
H_{Wall}=	9.0	ft	Wall height
B=	39.0	ft	Width equal to total height of wall and embankment
D_w =	0.0	ft	Depth below bottom of embankment
q_{Embank} =	3,600	psf	Pressure due to embankment fill below wall
q_{Wall} =	1,610	psf	Equivalent bearing pressure at bottom of wall
q_e =	5,210	psf	Total pressure at bottom of embankment

	A-4a (Ex. Fill)	A-6b (Upper)	A-6b (Lower)	A-7-6		
c_v =	1000	300	300	150	ft^2/yr	Coefficient of consolidation
t =	90	90	90	90	days	Time following completion of construction
H_{dr} =	5.5	8	8	26.5	ft	Length of longest drainage path considered
T_v =	8.151	1.156	1.156	0.053		Time factor
U =	100	95	95	26	%	Degree of consolidation

$(S_c)_t$ = 7.236 in Settlement complete at 90% of primary consolidation

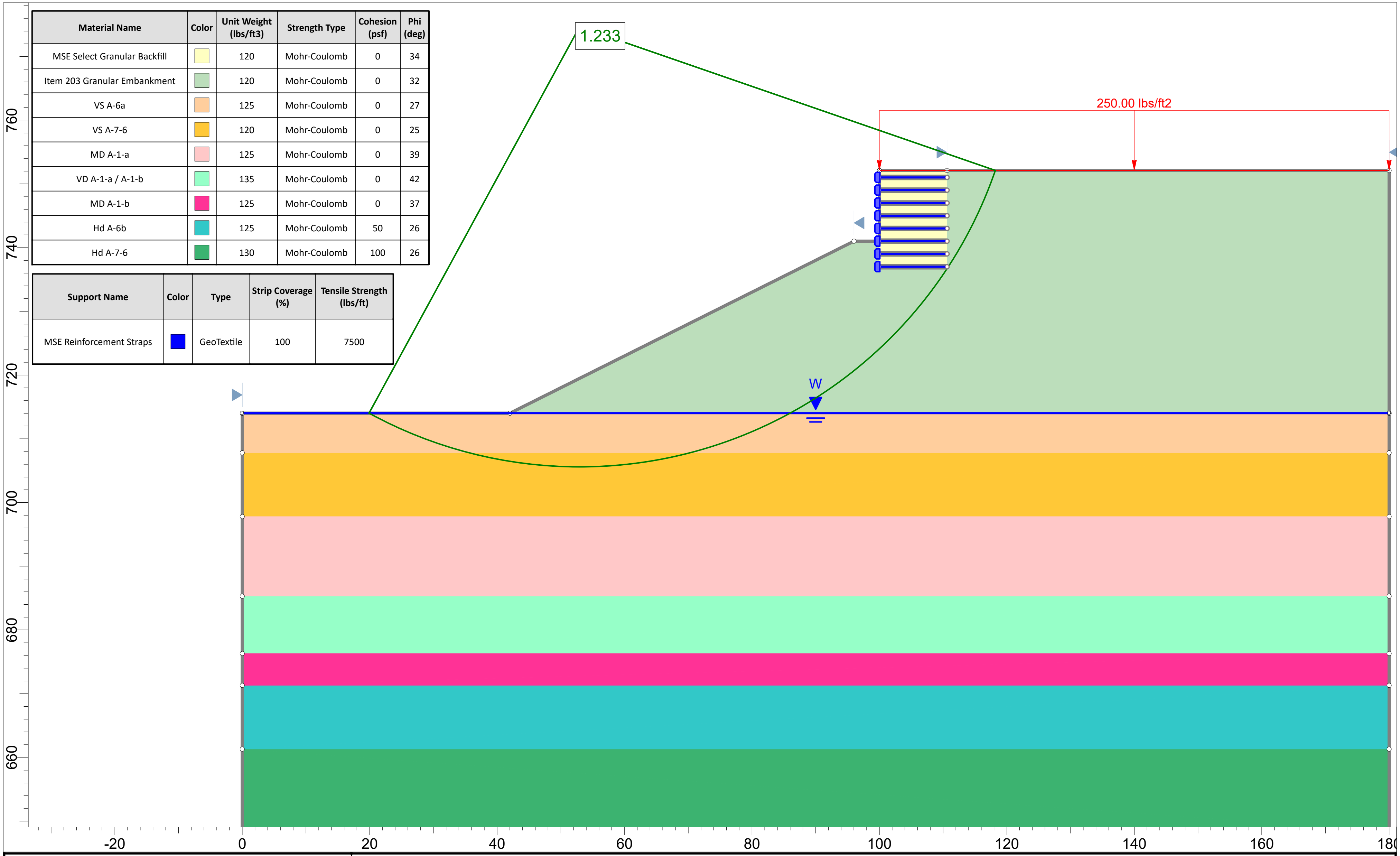
																						Total Settlement at Facing of Wall			Settlement Complete at 90% of Primary Consolidation	
Layer	Soil Type	Soil Type	Layer Depth (ft)		Layer Thickness (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (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-4a	C	0.0	2.0	2.0	1.0	120	240	120	58	2,058	32	0.198	0.030	0.522				0.11	0.500	2,605	2,663	0.090	1.077	2.477	1.077	2.477
	A-4a	C	2.0	5.5	3.5	3.8	120	660	450	216	2,216	32	0.198	0.030	0.522				0.42	0.500	2,604	2,820	0.117	1.401		1.401	
2	A-6b	C	5.5	9.0	3.5	7.3	115	1,063	861	409	2,409	38	0.252	0.038	0.569				0.81	0.499	2,598	3,007	0.119	1.429	3.622	1.358	3.441
	A-6b	C	9.0	11.0	2.0	10.0	115	1,293	1,178	554	2,554	38	0.252	0.038	0.569				1.11	0.497	2,588	3,141	0.061	0.731		0.694	
	A-6b	C	11.0	13.0	2.0	12.0	115	1,523	1,408	659	2,659	38	0.252	0.038	0.569				1.33	0.494	2,576	3,235	0.057	0.679		0.645	
	A-6b	C	13.0	15.5	2.5	14.3	115	1,810	1,666	777	2,777	38	0.252	0.038	0.569				1.58	0.491	2,559	3,336	0.065	0.783		0.744	
3	A-1-a	G	15.5	18.0	2.5	16.8	125	2,123	1,966	921	4,921					26	33	107	1.86	0.486	2,534	3,455	0.013	0.161	0.313	0.161	0.313
	A-1-a	G	18.0	20.5	2.5	19.3	125	2,435	2,279	1,078	5,078					26	31	103	2.14	0.480	2,503	3,580	0.013	0.152		0.152	
4	A-1-b	G	20.5	24.0	3.5	22.3	130	2,890	2,663	1,274	5,274					30	35	113	2.47	0.472	2,459	3,733	0.015	0.174	0.358	0.174	0.358
	A-1-b	G	24.0	28.0	4.0	26.0	130	3,410	3,150	1,528	5,528					30	33	107	2.89	0.460	2,395	3,923	0.015	0.184		0.184	
5	A-1-a	G	28.0	32.5	4.5	30.3	135	4,018	3,714	1,826	5,826					85	88	379	3.36	0.444	2,314	4,140	0.004	0.051	0.099	0.051	0.099
	A-1-a	G	32.5	37.0	4.5	34.8	135	4,625	4,321	2,153	6,153					85	83	347	3.86	0.426	2,222	4,375	0.004	0.048		0.048	
6	A-6b	C	37.0	40.9	3.9	39.0	120	5,093	4,859	2,429	6,429	36	0.234	0.023	0.553				4.33	0.409	2,133	4,561	0.016	0.193	0.368	0.183	0.350
	A-6b	C	40.9	44.8	3.9	42.9	120	5,561	5,327	2,653	6,653	36	0.234	0.023	0.553				4.76	0.393	2,050	4,703	0.015	0.175		0.167	
7	A-7-6	C	44.8	48.4	3.6	46.6	125	6,011	5,786	2,878	6,878	42	0.288	0.029	0.600				5.18	0.378	1,972	4,850	0.015	0.176	0.337	0.046	0.088
	A-7-6	C	48.4	52.0	3.6	50.2	125	6,461	6,236	3,104	7,104	42	0.288	0.029	0.600				5.58	0.364	1,899	5,002	0.013	0.161		0.042	
8	A-7-6	C	52.0	57.5	5.5	54.8	130	7,176	6,819	3,402	7,402	42	0.288	0.029	0.600				6.08	0.347	1,810	5,212	0.018	0.220	0.429	0.057	0.112
	A-7-6	C	57.5	63.5	6.0	60.5	130	7,956	7,566	3,791	7,791	42	0.288	0.029	0.600				6.72	0.327	1,705	5,496	0.017	0.209		0.054	

Settlement Remaining After Hold Period: 0.767 in

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

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
MSE Select Granular Backfill	🟨	120	Mohr-Coulomb	0	34
Item 203 Granular Embankment	🟩	120	Mohr-Coulomb	0	32
VS A-6a	🟧	125	Mohr-Coulomb	0	27
VS A-7-6	🟨	120	Mohr-Coulomb	0	25
MD A-1-a	🟥	125	Mohr-Coulomb	0	39
VD A-1-a / A-1-b	🟩	135	Mohr-Coulomb	0	42
MD A-1-b	🟥	125	Mohr-Coulomb	0	37
Hd A-6b	🟦	125	Mohr-Coulomb	50	26
Hd A-7-6	🟩	130	Mohr-Coulomb	100	26

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



Project
Retaining Wall E4 - Sta. 404+75 to 407+00 - MSE Wall Global Stability

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

Drawn By BRT

Scale 1:250

Company Resource International, Inc.

Date 07/19/2019

File Name Retaining Wall E4 - Sta. 404+75 to 407+00 - Global Stability.slim

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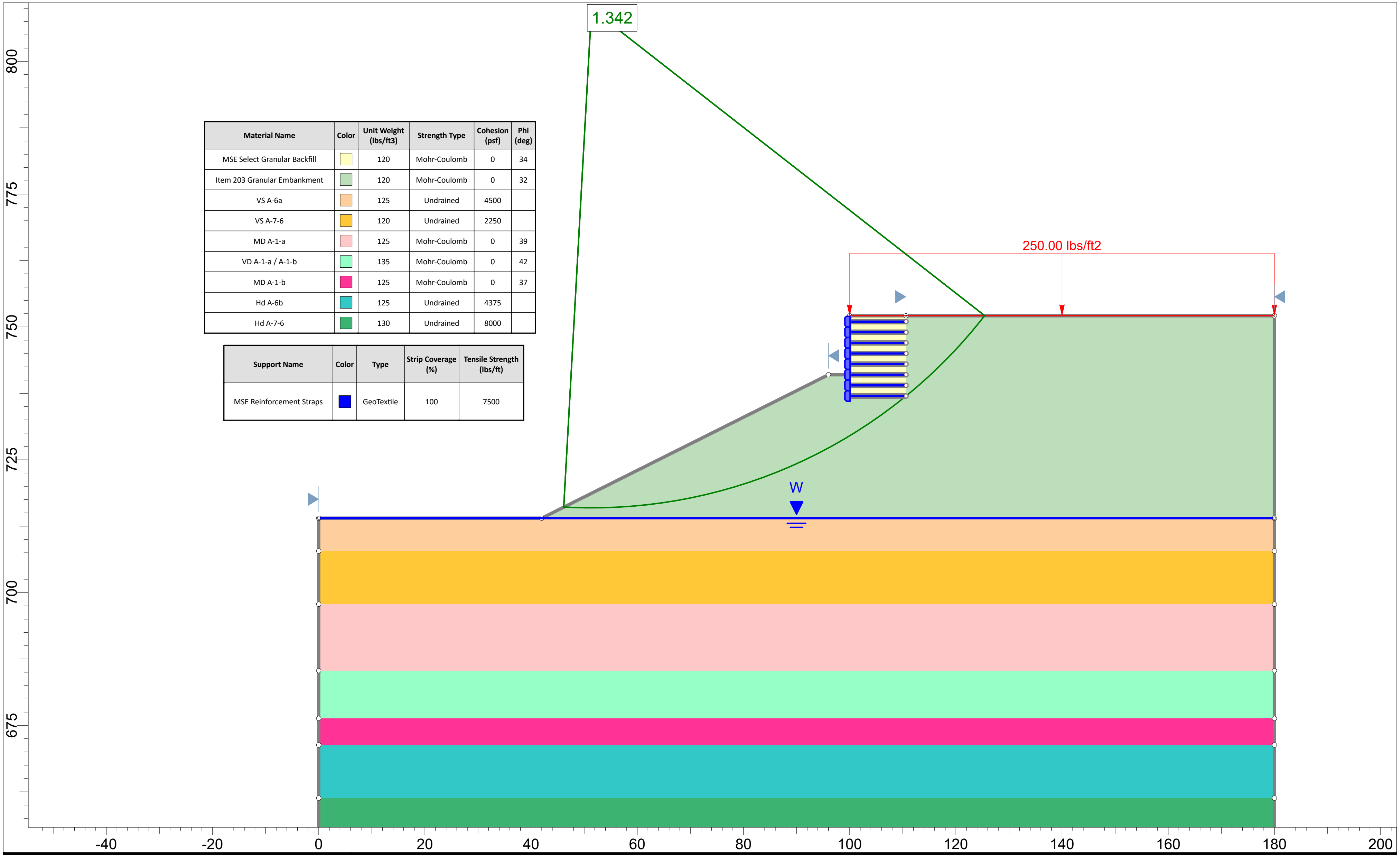
SLIDEINTERPRET 8.020



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
MSE Select Granular Backfill	🟨	120	Mohr-Coulomb	0	34
Item 203 Granular Embankment	🟩	120	Mohr-Coulomb	0	32
VS A-6a	🟧	125	Undrained	4500	
VS A-7-6	🟧	120	Undrained	2250	
MD A-1-a	🟥	125	Mohr-Coulomb	0	39
VD A-1-a / A-1-b	🟩	135	Mohr-Coulomb	0	42
MD A-1-b	🟪	125	Mohr-Coulomb	0	37
Hd A-6b	🟦	125	Undrained	4375	
Hd A-7-6	🟩	130	Undrained	8000	

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

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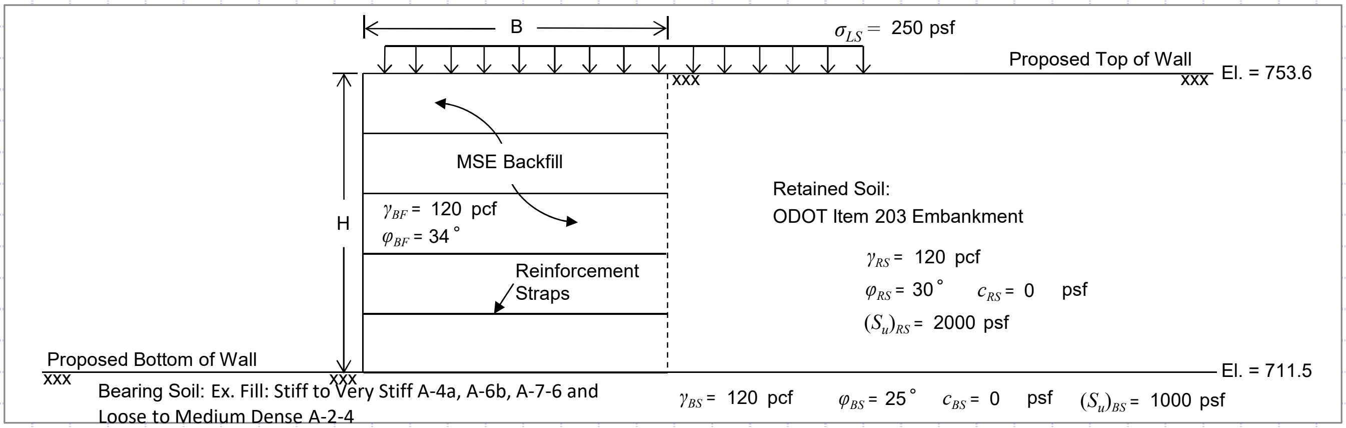
SLIDEINTERPRET 8.020

Project	Retaining Wall E4 - Sta. 404+75 to 407+00 - MSE Wall Global Stability
Analysis Description	15.1 ft Wall Height - Undrained - Circular - Spencer

Drawn By	BRT	*Scale*	1:300	*Company*	Resource International, Inc.
Date	07/19/2019			*File Name*	Retaining Wall E4 - Sta. 404+75 to 407+00 - Global Stability.slim

Retaining Wall E4 - Sta. 407+00 to 409+50 - B-017-2-09, B-018-2-13 and B-114-7-13 - 27.7 ft. Wall Height



MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	42.1	ft
MSE Wall Width (Reinforcement Length), (B) =	29.5	ft
MSE Wall Length, (L) =	750	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}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	25	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(S_u)_{BS}]$ =	1000	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

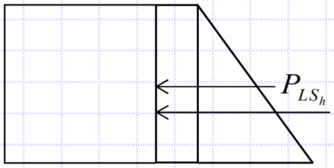
	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

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

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

Sliding Force:

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



$P_{EH} = \frac{1}{2}\gamma_{RS}H^2K_a\gamma_{EH}$ = ½(120 pcf)(42.1 ft)²(0.297)(1.5) = 47.38 kip/ft

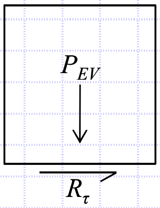
$P_{LS_h} = \sigma_{LS}HK_a\gamma_{LS}$ = (250 psf)(42.1 ft)(0.297)(1.75) = 5.47 kip/ft

P_H = 47.38 kip/ft + 5.47 kip/ft = 52.85 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)(42.1 ft)(29.5 ft)(1.00) = 149.03 kip/ft

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

$\tan\delta =$ tan(25) ≤ tan(34) ⟶ 0.47 ≤ 0.67 ⟶ $\tan\delta =$ 0.47

R_τ = (149.03 kip/ft)(0.47) = 70.04 kip/ft

Verify Sliding Force Less Than Factored Sliding Resistance - Drained Condition

$P_H \le R_\tau \cdot \phi_\tau$ ⟶ 52.85 kip/ft ≤ (70.04 kip/ft)(1.0) = 70.04 kip/ft ⟶ 52.85 kip/ft ≤ 70.04 kip/ft **OK**

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

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) = 42.1 ft
MSE Wall Width (Reinforcement Length), (B) = 29.5 ft
MSE Wall Length, (L) = 750 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}) = 120 pcf
Bearing Soil Friction Angle, (φ_{BS}) = 25 °
Bearing Soil Drained Cohesion, (c_{BS}) = 0 psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ = 1000 psf
Embedment Depth, (D_f) = 4.0 ft
Depth to Grounwater (Below Bot. of Wall), (D_W) = 0.0 ft

LRFD Load Factors

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

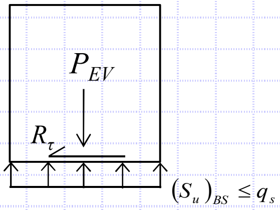
(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

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

Check Sliding Resistance - Undrained Condition

Nominal Sliding Resisting:

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



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

$$q_s = \sigma_v / 2 = (5.05 \text{ ksf}) / 2 = 2.53 \text{ ksf}$$

$$\sigma_v = P_{EV} / B = (149.03 \text{ kip/ft}) / (29.5 \text{ ft}) = 5.05 \text{ ksf}$$

$$R_\tau = (1.00 \text{ ksf} \le 2.53 \text{ ksf})(29.5 \text{ ft}) = 29.50 \text{ kip/ft}$$

Verify Sliding Force Less Than Factored Sliding Resistance - Undrained Condition

$P_H \le R_\tau \cdot \phi_\tau$ ⟶ 52.85 kip/ft ≤ (29.50 kip/ft)(1.0) = 29.50 kip/ft ⟶ 52.85 kip/ft ≤ 29.50 kip/ft **ERROR!!**

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

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	42.1	ft
MSE Wall Width (Reinforcement Length), (B) =	29.5	ft
MSE Wall Length, (L) =	750	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:

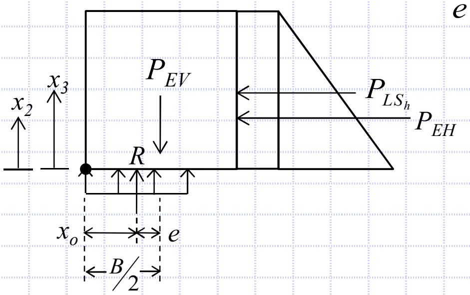
Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	25	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	1000	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

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

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

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



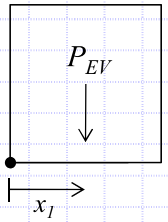
$$e = B/2 - x_o$$

$$x_o = \frac{M_{EV} - M_H}{P_{EV}}$$ = (2198.19 kip·ft/ft - 779.88 kip·ft/ft) / (149.03 kip/ft) = 9.52 ft

M_{EV} = 2198.19 kip·ft/ft
M_H = 779.88 kip·ft/ft
P_{EV} = 149.03 kip/ft
Defined below

$e =$ (29.5 ft)/2 - 9.52 ft = 5.23 ft

Resisting Moment, M_{EV}:



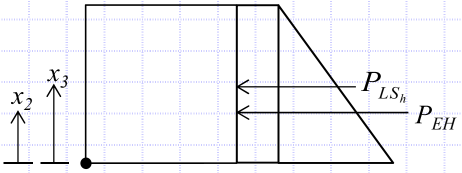
$$M_{EV} = P_{EV}(x_1)$$

$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV}$ = (120 pcf)(42.1 ft)(29.5 ft)(1.00) = 149.03 kip/ft

$x_1 = B/2$ = (29.5 ft) / 2 = 14.75 ft

M_{EV} = (149.03 kip/ft)(14.75 ft) = 2198.19 kip·ft/ft

Overturning Moment, M_H:



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

$P_{EH} = ½\gamma_{RS}H^2K_a\gamma_{EH}$ = ½(120 pcf)(42.1 ft)²(0.297)(1.5) = 47.38 kip/ft

$P_{LS_h} = \sigma_{LS}HK_a\gamma_{LS}$ = (250 psf)(42.1 ft)(0.297)(1.75) = 5.47 kip/ft

$x_2 = H/3$ = (42.1 ft) / 3 = 14.03 ft

$x_3 = H/2$ = (42.1 ft) / 2 = 21.05 ft

M_H = (47.38 kip/ft)(14.03 ft) + (5.47 kip/ft)(21.05 ft) = 779.88 kip·ft/ft

Check Eccentricity

$e < e_{\max} \longrightarrow$ 5.23 ft < 9.83 ft **OK**

Limiting Eccentricity: $e_{\max} = B/3 \longrightarrow e_{\max}$ = (29.5 ft) / 3 = 9.83 ft

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	42.1	ft
MSE Wall Width (Reinforcement Length), (B) =	29.5	ft
MSE Wall Length, (L) =	750	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:

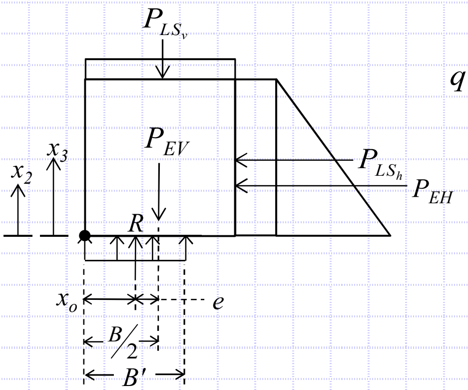
Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	25	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	1000	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

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

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

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



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

$$B' = B - 2e \quad = \quad 29.5\text{ ft - 2(3.64 ft)} \quad = \quad 22.22 \quad \text{ft}$$

$$e = B\big/2 - x_o \quad = \quad \text{(29.5 ft) / 2 - 11.11 ft} \quad = \quad 3.64 \quad \text{ft}$$

$$x_o = \frac{M_V - M_H}{P_V} \quad = \quad \text{(3158.01 kip·ft/ft - 779.84 kip·ft/ft) / 214.1 kip/ft} \quad = \quad 11.11 \quad \text{ft}$$

$$q_{eq} \;=\; \text{(214.1 kip/ft) / (22.22 ft)} \quad = \quad 9.64 \quad \text{ksf}$$

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

$$M_V = \text{[(120 pcf)(42.1 ft)(29.5 ft)(1.35)](14.75 ft) + [(250 psf)(29.5 ft)(1.75)](14.75 ft)} \quad = \quad 3158.01 \quad \text{kip·ft/ft}$$

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

$$M_H = \text{[½(120 pcf)(42.1 ft)²(0.297)(1.5)](14.03 ft) + [(250 psf)(42.1 ft)(0.297)(1.75)](21.05 ft)} \quad = \quad 779.84 \quad \text{kip·ft/ft}$$

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

$$P_V = \text{(120 pcf)(42.1 ft)(29.5 ft)(1.35) + (250 psf)(29.5 ft)(1.75)} \quad = \quad 214.1 \quad \text{kip/ft}$$

Check Bearing Resistance - Drained Condition

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

$N_{cm} = N_c s_c i_c$ = 21.03

N_c = 20.72
s_c = 1+(22.22 ft/750 ft)(10.66/20.72)
= 1.015
i_c = 1.000 (Assumed)

$N_{qm} = N_q s_q d_q i_q$ = 11.40

N_q = 10.66
s_q = 1.014
d_q = 1+2tan(25°)[1-sin(25°)]²tan⁻¹(4.0 ft/22.22 ft)
1.055
i_q = 1.000 (Assumed)
C_{wq} = 0.0 ft > 4.0 ft = 0.500

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 10.75

N_γ = 10.88
s_γ = 0.988
i_γ = 1.000 (Assumed)
$C_{w\gamma}$ = 0.0 ft < 1.5(22.22 ft) + 4.0 ft = 0.500

$$q_n = \text{(0 psf)(21.031) + (120 pcf)(4.0 ft)(11.404)(0.500) + ½(120 pcf)(22.2 ft)(10.749)(0.500)} \quad = \quad 9.90 \quad \text{ksf}$$

Verify Equivalent Pressure Less Than Factored Bearing Resistance

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

$q_{eq} \le q_n \cdot \phi_b$ ⟶ 9.64 ksf ≤ (9.90 ksf)(0.65) = 6.44 ksf ⟶ 9.64 ksf ≤ 6.44 ksf **ERROR!!**

JOB	FRA-70-13.10	NO.	W-13-072
SHEET NO.	5	OF	6
CALCULATED BY	BRT	DATE	7/12/2019
CHECKED BY	JPS	DATE	7/12/2019

Retaining Wall E4 - Sta. 407+00 to 409+50

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	42.1	ft
MSE Wall Width (Reinforcement Length), (B) =	29.5	ft
MSE Wall Length, (L) =	750	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}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	25	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	1000	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

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

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

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

Check Bearing Resistance - Undrained Condition

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

$N_{cm} = N_c s_c i_c$ = 5.170

N_c = 5.140

s_c = 1+(22.22 ft/[(5)(750 ft)]) = 1.006

i_c = 1.000 (Assumed)

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

N_q = 1.000

s_q = 1.000

d_q = 1+2tan(0°)[1-sin(0°)]²tan⁻¹(4.0 ft/22.22 ft)

1.000

i_q = 1.000 (Assumed)

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

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

N_γ = 0.000

s_γ = 1.000

i_γ = 1.000 (Assumed)

$C_{w\gamma}$ = 0.0 ft < 1.5(22.22 ft) + 4.0 ft = 0.500

$q_n =$ (1000 psf)(5.170) + (120 pcf)(4.0 ft)(1.000)(0.500) + ½(120 pcf)(22.2 ft)(0.000)(0.500) = 5.41 ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

$q_{eq} \leq q_n \cdot \phi_b$ $\longrightarrow$ 9.64 ksf ≤ (5.41 ksf)(0.65) = 3.52 ksf $\longrightarrow$ 9.64 ksf ≤ 3.52 ksf **ERROR!!**

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) =	42.1	ft
MSE Wall Width (Reinforcement Length), (B) =	29.5	ft
MSE Wall Length, (L) =	750	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_{RS}) =	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:

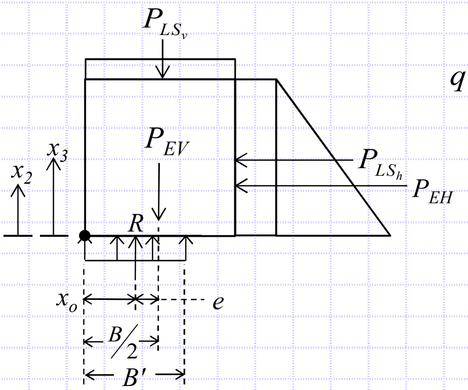
Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	25	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	1000	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

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

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

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



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

$B' = B - 2e$ = 29.5 ft - 2(3.25 ft) = 23.00 ft

$e = B/2 - x_o$ = (29.5 ft) / 2 - 11.5 ft = 3.25 ft

$x_o = \dfrac{M_V - M_H}{P_V}$ = (2307.03 kip·ft/ft - 508.93 kip·ft/ft) / 156.41 kip/ft = 11.5 ft

q_{eq} = (156.41 kip/ft) / (23 ft) = 6.80 ksf

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

M_V = [(120 pcf)(42.1 ft)(29.5 ft)(1.00)](14.8 ft) + [(250 psf)(29.5 ft)(1.00)](14.8 ft) = 2307.03 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)(42.1 ft)²(0.297)(1.00)](14.03 ft) + [(250 psf)(42.1 ft)(0.297)(1.00)](21.05 ft) = 508.93 kip·ft/ft

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

P_V = (120 pcf)(42.1 ft)(29.5 ft)(1.00) + (250 psf)(29.5 ft)(1.00) = 156.41 kip/ft

Settlement, Time Rate of Consolidation and Differential Settlement:

Boring	Total Settlement at Center of Reinforced Soil Mass	Total Settlement at Wall Facing	Time for 90% Consolidation	Distance Between Borings Along Wall Facing	Differential Settlement Along Wall Facing
B-020-4-13	10.464 in	5.175 in	23 days		
B-114-9-13	7.697 in	4.758 in	8 days	160 ft	1/4600
B-114-8-13	15.594 in	8.003 in	90 days	175 ft	1/650
B-114-7-13	15.451 in	8.380 in	100 days	215 ft	1/6840
B-017-2-09	9.057 in	5.790 in	55 days	90 ft	1/420
B-018-2-13	12.067 in	7.604 in	100 days	100 ft	1/660

W-13-072 - FRA-70-13.10 - Retaining Wall E4

MSE Wall Settlement - Sta. 407+00 to 409+50

Calculated By: BRT Date: 7/18/2019

Checked By: JPS Date: 7/18/2019

Boring B-017-2-09

H_{Embank}=	16.0	ft	Height of embankment below wall to existing grade
H_{Wall}=	25.8	ft	Wall height
B=	41.8	ft	Width equal to total height of wall and embankment
D_w =	0.0	ft	Depth below bottom of embankment
q_{Embank} =	1,920	psf	Pressure due to embankment fill below wall
q_{Wall} =	4,350	psf	Equivalent bearing pressure at bottom of wall
q_e =	6,270	psf	Total pressure at bottom of embankment

																			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	3.5	3.5	1.8	115	403	201	92	2,092	21	0.099	0.015	0.436				0.04	1.000	6,268	6,361	0.166	1.987	0.500	3,135	3,227	0.095	1.134
2	A-1-b	G	3.5	6.0	2.5	4.8	120	703	553	256	2,256					13	22	79	0.11	0.995	6,241	6,497	0.045	0.536	0.500	3,133	3,389	0.036	0.429
3	A-4a	C	6.0	8.5	2.5	7.3	115	990	846	394	2,394	21	0.099	0.015	0.436				0.17	0.985	6,173	6,567	0.096	1.150	0.499	3,128	3,522	0.049	0.590
	A-4a	C	8.5	11.0	2.5	9.8	115	1,278	1,134	525	2,525	21	0.099	0.015	0.436				0.23	0.966	6,057	6,582	0.089	1.072	0.497	3,119	3,645	0.045	0.541
	A-4a	C	11.0	13.5	2.5	12.3	115	1,565	1,421	657	2,657	21	0.099	0.015	0.436				0.29	0.940	5,895	6,552	0.083	0.999	0.495	3,105	3,762	0.042	0.501
4	A-3a	G	13.5	16.0	2.5	14.8	115	1,853	1,709	788	2,788					5	7	48	0.35	0.909	5,697	6,486	0.048	0.576	0.492	3,084	3,873	0.036	0.435
	A-3a	G	16.0	18.5	2.5	17.3	115	2,140	1,996	920	2,920					5	6	47	0.41	0.873	5,475	6,395	0.044	0.533	0.488	3,058	3,977	0.034	0.403
5	A-2-6	G	18.5	23.5	5.0	21.0	125	2,765	2,453	1,142	5,142					21	25	86	0.50	0.817	5,121	6,263	0.043	0.518	0.479	3,006	4,149	0.033	0.393
6	A-1-a	G	23.5	27.5	4.0	25.5	135	3,305	3,035	1,444	5,444					72	80	328	0.61	0.749	4,698	6,141	0.008	0.092	0.467	2,929	4,373	0.006	0.071
	A-1-a	G	27.5	32.0	4.5	29.8	135	3,913	3,609	1,752	5,752					72	75	299	0.71	0.689	4,322	6,074	0.008	0.098	0.453	2,843	4,596	0.006	0.076
7	A-6a	C	32.0	37.0	5.0	34.5	125	4,538	4,225	2,072	6,072	35	0.225	0.023	0.546				0.83	0.629	3,943	6,015	0.034	0.404	0.437	2,738	4,810	0.027	0.319
8	A-6b	C	37.0	42.0	5.0	39.5	125	5,163	4,850	2,385	6,385	36	0.234	0.023	0.553				0.94	0.573	3,593	5,978	0.030	0.361	0.418	2,620	5,005	0.024	0.291
	A-6b	C	42.0	47.0	5.0	44.5	125	5,788	5,475	2,698	6,698	36	0.234	0.023	0.553				1.06	0.524	3,289	5,987	0.026	0.313	0.399	2,501	5,199	0.021	0.257
9	A-6b	C	47.0	55.0	8.0	51.0	130	6,828	6,308	3,125	7,125	36	0.234	0.023	0.553				1.22	0.471	2,953	6,078	0.035	0.418	0.375	2,349	5,474	0.029	0.352
																			Total Settlement:				9.057 in		Total Settlement:				5.790 in

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

W-13-072 - FRA-70-13.10 - Retaining Wall E4
MSE Wall Settlement - Sta. 407+00 to 409+50

Calculated By: BRT Date: 07/18/2019
Checked By: JPS Date: 07/18/2019

Boring B-017-2-09

H_{Embank}= 16.0 ft Height of embankment below wall to existing grade
H_{Wall}= 25.8 ft Wall height
B= 41.8 ft Width equal to total height of wall and embankment
D_w = 0.0 ft Depth below bottom of embankment
q_{Embank} = 1,920 psf Pressure due to embankment fill below wall
q_{Wall} = 4,350 psf Equivalent bearing pressure at bottom of wall
q_e = 6,270 psf Total pressure at bottom of embankment

	A-4a (Ex. Fill)	A-6a	A-6b		
c_v =	1000	600	300	ft^2/yr	Coefficient of consolidation
t =	55	55	55	days	Time following completion of construction
H_{dr} =	3.8	5	23	ft	Length of longest drainage path considered
T_v =	10.435	3.616	0.085		Time factor
U =	100	100	33	%	Degree of consolidation

$(S_c)_t$ = 5.187 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	3.5	3.5	1.8	115	403	201	92	2,092	21	0.099	0.015	0.436				0.07	0.500	3,135	3,227	0.095	1.134	1.134	1.134	1.134
2	A-1-b	G	3.5	6.0	2.5	4.8	120	703	553	256	2,256					13	22	79	0.18	0.500	3,133	3,389	0.036	0.429	0.429	0.429	0.429
3	A-4a	C	6.0	8.5	2.5	7.3	115	990	846	394	2,394	21	0.099	0.015	0.436				0.28	0.499	3,128	3,522	0.049	0.590	1.632	0.590	1.632
	A-4a	C	8.5	11.0	2.5	9.8	115	1,278	1,134	525	2,525	21	0.099	0.015	0.436				0.38	0.497	3,119	3,645	0.045	0.541		0.541	
	A-4a	C	11.0	13.5	2.5	12.3	115	1,565	1,421	657	2,657	21	0.099	0.015	0.436				0.47	0.495	3,105	3,762	0.042	0.501		0.501	
4	A-3a	G	13.5	16.0	2.5	14.8	115	1,853	1,709	788	4,788					5	7	48	0.57	0.492	3,084	3,873	0.036	0.435	0.838	0.435	0.838
	A-3a	G	16.0	18.5	2.5	17.3	115	2,140	1,996	920	4,920					5	6	47	0.67	0.488	3,058	3,977	0.034	0.403		0.403	
5	A-2-6	G	18.5	23.5	5.0	21.0	125	2,765	2,453	1,142	5,142					21	25	86	0.81	0.479	3,006	4,149	0.033	0.393	0.393	0.393	0.393
6	A-1-a	G	23.5	27.5	4.0	25.5	135	3,305	3,035	1,444	5,444					72	80	328	0.99	0.467	2,929	4,373	0.006	0.071	0.146	0.071	0.146
	A-1-a	G	27.5	32.0	4.5	29.8	135	3,913	3,609	1,752	5,752					72	75	299	1.15	0.453	2,843	4,596	0.006	0.076		0.076	
7	A-6a	C	32.0	37.0	5.0	34.5	125	4,538	4,225	2,072	6,072	35	0.225	0.023	0.546				1.34	0.437	2,738	4,810	0.027	0.319	0.319	0.319	0.319
8	A-6b	C	37.0	42.0	5.0	39.5	125	5,163	4,850	2,385	6,385	36	0.234	0.023	0.553				1.53	0.418	2,620	5,005	0.024	0.291	0.548	0.096	0.181
	A-6b	C	42.0	47.0	5.0	44.5	125	5,788	5,475	2,698	6,698	36	0.234	0.023	0.553				1.72	0.399	2,501	5,199	0.021	0.257		0.085	
9	A-6b	C	47.0	55.0	8.0	51.0	130	6,828	6,308	3,125	7,125	36	0.234	0.023	0.553				1.98	0.375	2,349	5,474	0.029	0.352	0.352	0.116	0.116

Settlement Remaining After Hold Period: 0.603 in

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

W-13-072 - FRA-70-13.10 - Retaining Wall E4

MSE Wall Settlement - Sta. 407+00 to 409+50

Calculated By: BRT Date: 7/18/2019

Checked By: JPS Date: 7/18/2019

Boring B-018-2-13

H= 42.1 ft Total wall height

B'= 23.0 ft Effective footing width due to eccentricity

D_w = 0.0 ft Depth below bottom of wall

q_e = 6,800 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-2-4	G	0.0	1.5	1.5	0.8	120	180	90	43	2,043					18	36	117	0.03	1.000	6,799	6,842	0.028	0.338	0.500	3,400	3,443	0.024	0.292
2	A-6b	C	1.5	4.0	2.5	2.8	120	480	330	158	2,158	40	0.270	0.041	0.585				0.12	0.995	6,763	6,921	0.288	3.456	0.500	3,398	3,556	0.165	1.978
	A-6b	C	4.0	6.5	2.5	5.3	120	780	630	302	2,302	40	0.270	0.041	0.585				0.23	0.968	6,581	6,884	0.259	3.107	0.498	3,384	3,686	0.143	1.721
3	A-4a	C	6.5	8.5	2.0	7.5	120	1,020	900	432	2,432	28	0.162	0.024	0.491				0.33	0.923	6,279	6,711	0.120	1.443	0.493	3,356	3,788	0.066	0.795
	A-4a	C	8.5	10.5	2.0	9.5	120	1,260	1,140	547	2,547	28	0.162	0.024	0.491				0.41	0.873	5,937	6,484	0.110	1.319	0.488	3,316	3,863	0.061	0.733
4	A-1-b	G	10.5	13.5	3.0	12.0	125	1,635	1,448	699	4,699					22	30	98	0.52	0.804	5,470	6,169	0.029	0.347	0.477	3,247	3,945	0.023	0.275
	A-1-b	G	13.5	16.5	3.0	15.0	125	2,010	1,823	887	4,887					22	28	93	0.65	0.724	4,922	5,808	0.026	0.314	0.462	3,140	4,026	0.021	0.253
5	A-1-a	G	16.5	22.0	5.5	19.3	135	2,753	2,381	1,180	5,180					57	67	252	0.84	0.623	4,237	5,417	0.014	0.173	0.435	2,957	4,137	0.012	0.143
	A-1-a	G	22.0	28.0	6.0	25.0	135	3,563	3,158	1,598	5,598					57	61	222	1.09	0.516	3,510	5,107	0.014	0.164	0.395	2,688	4,286	0.012	0.139
	A-1-a	G	28.0	33.0	5.0	30.5	135	4,238	3,900	1,997	5,997					57	57	201	1.33	0.440	2,990	4,987	0.010	0.119	0.359	2,439	4,436	0.009	0.103
6	A-7-6	C	33.0	38.0	5.0	35.5	125	4,863	4,550	2,335	6,335	45	0.315	0.032	0.624				1.54	0.386	2,625	4,960	0.032	0.381	0.328	2,232	4,567	0.028	0.339
	A-7-6	C	38.0	43.0	5.0	40.5	125	5,488	5,175	2,648	6,648	45	0.315	0.032	0.624				1.76	0.343	2,335	4,983	0.027	0.320	0.301	2,047	4,695	0.024	0.290
	A-7-6	C	43.0	49.0	6.0	46.0	125	6,238	5,863	2,992	6,992	45	0.315	0.032	0.624				2.00	0.306	2,079	5,071	0.027	0.320	0.275	1,869	4,861	0.025	0.294
	A-7-6	C	49.0	55.0	6.0	52.0	125	6,988	6,613	3,368	7,368	45	0.315	0.032	0.624				2.26	0.273	1,855	5,223	0.022	0.266	0.250	1,702	5,070	0.021	0.248
																				Total Settlement:			12.067 in		Total Settlement:			7.604 in	

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

W-13-072 - FRA-70-13.10 - Retaining Wall E4
MSE Wall Settlement - Sta. 407+00 to 409+50

Calculated By: BRT Date: 07/18/2019
Checked By: JPS Date: 07/18/2019

Boring B-018-2-13

H=	42.1	ft	Total wall height
B'=	23.0	ft	Effective footing width due to eccentricity
D_w =	0.0	ft	Depth below bottom of wall
q_e =	6,800	psf	Equivalent bearing pressure at bottom of wall

	A-6b (Ex. Fill)	A-4a (Ex. Fill)	A-7-6		
c_v =	300	1000	150	ft^2/yr	Coefficient of consolidation
t =	100	100	100	days	Time following completion of construction
H_{dr} =	5	4	22	ft	Length of longest drainage path considered
T_v =	3.288	17.123	0.085		Time factor
U =	100	100	33	%	Degree of consolidation

$(S_c)_t$ = 6.819 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-2-4	G	0.0	1.5	1.5	0.8	120	180	90	43	2,043					18	36	117	0.03	0.500	3,400	3,443	0.024	0.292	0.292	0.292	0.292
2	A-6b	C	1.5	4.0	2.5	2.8	120	480	330	158	2,158	40	0.270	0.041	0.585				0.12	0.500	3,398	3,556	0.165	1.978	3.699	1.978	3.699
	A-6b	C	4.0	6.5	2.5	5.3	120	780	630	302	2,302	40	0.270	0.041	0.585				0.23	0.498	3,384	3,686	0.143	1.721		1.721	
3	A-4a	C	6.5	8.5	2.0	7.5	120	1,020	900	432	2,432	28	0.162	0.024	0.491				0.33	0.493	3,356	3,788	0.066	0.795	1.528	0.795	1.528
	A-4a	C	8.5	10.5	2.0	9.5	120	1,260	1,140	547	2,547	28	0.162	0.024	0.491				0.41	0.488	3,316	3,863	0.061	0.733		0.733	
4	A-1-b	G	10.5	13.5	3.0	12.0	125	1,635	1,448	699	4,699					22	30	98	0.52	0.477	3,247	3,945	0.023	0.275	0.528	0.275	0.528
	A-1-b	G	13.5	16.5	3.0	15.0	125	2,010	1,823	887	4,887					22	28	93	0.65	0.462	3,140	4,026	0.021	0.253		0.253	
5	A-1-a	G	16.5	22.0	5.5	19.3	135	2,753	2,381	1,180	5,180					57	67	252	0.84	0.435	2,957	4,137	0.012	0.143	0.385	0.143	0.385
	A-1-a	G	22.0	28.0	6.0	25.0	135	3,563	3,158	1,598	5,598					57	61	222	1.09	0.395	2,688	4,286	0.012	0.139		0.139	
	A-1-a	G	28.0	33.0	5.0	30.5	135	4,238	3,900	1,997	5,997					57	57	201	1.33	0.359	2,439	4,436	0.009	0.103		0.103	
6	A-7-6	C	33.0	38.0	5.0	35.5	125	4,863	4,550	2,335	6,335	45	0.315	0.032	0.624				1.54	0.328	2,232	4,567	0.028	0.339	1.171	0.112	0.386
	A-7-6	C	38.0	43.0	5.0	40.5	125	5,488	5,175	2,648	6,648	45	0.315	0.032	0.624				1.76	0.301	2,047	4,695	0.024	0.290		0.096	
	A-7-6	C	43.0	49.0	6.0	46.0	125	6,238	5,863	2,992	6,992	45	0.315	0.032	0.624				2.00	0.275	1,869	4,861	0.025	0.294		0.097	
	A-7-6	C	49.0	55.0	6.0	52.0	125	6,988	6,613	3,368	7,368	45	0.315	0.032	0.624				2.26	0.250	1,702	5,070	0.021	0.248		0.082	

Settlement Remaining After Hold Period: 0.785 in

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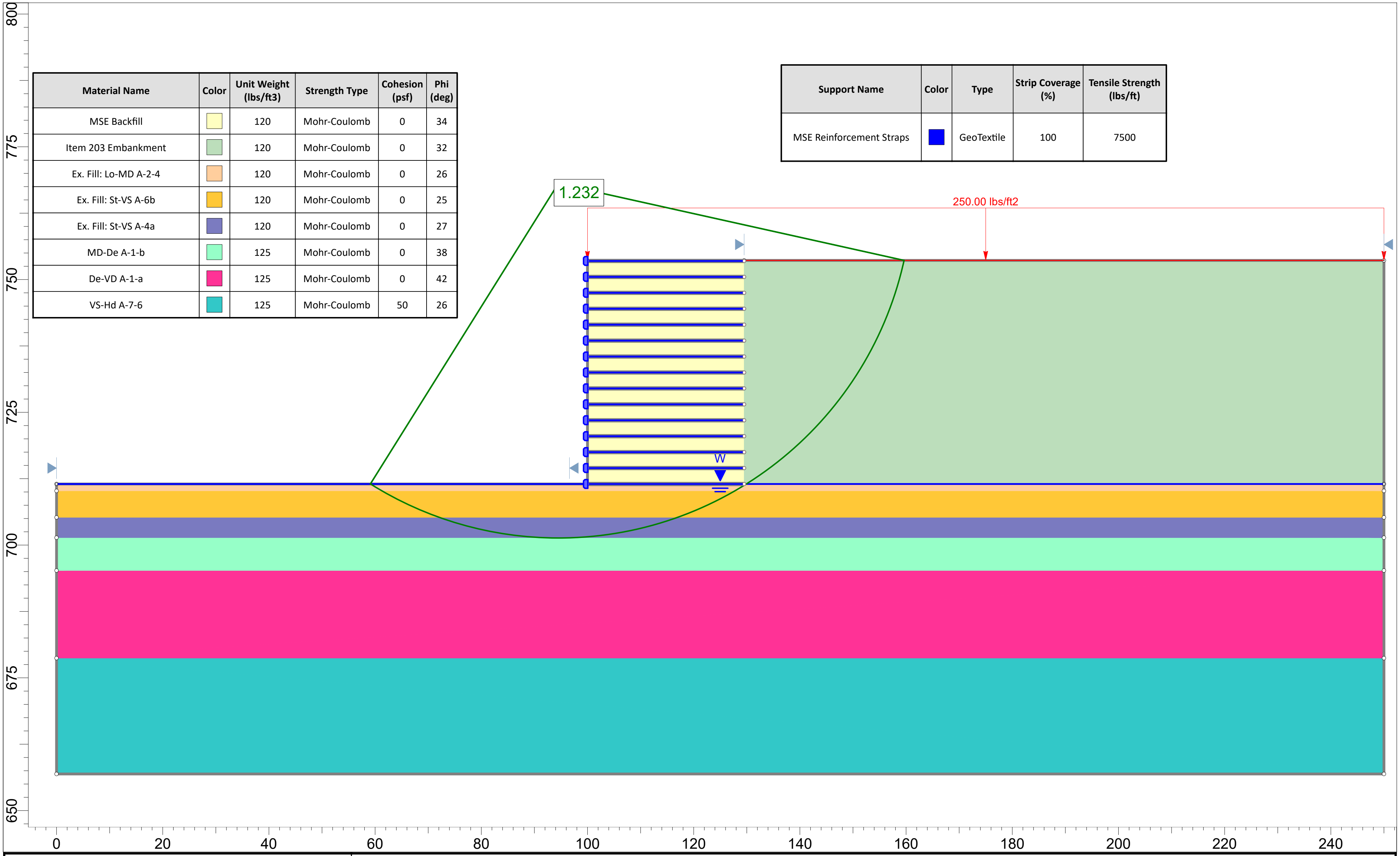
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)
11. $(S_c)_t = S_c(U/100)$; U = 100 for all granular soils at time t = 0



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
MSE Backfill	🟨	120	Mohr-Coulomb	0	34
Item 203 Embankment	🟩	120	Mohr-Coulomb	0	32
Ex. Fill: Lo-MD A-2-4	🟧	120	Mohr-Coulomb	0	26
Ex. Fill: St-VS A-6b	🟧	120	Mohr-Coulomb	0	25
Ex. Fill: St-VS A-4a	🟪	120	Mohr-Coulomb	0	27
MD-De A-1-b	🟩	125	Mohr-Coulomb	0	38
De-VD A-1-a	🟥	125	Mohr-Coulomb	0	42
VS-Hd A-7-6	🟦	125	Mohr-Coulomb	50	26

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

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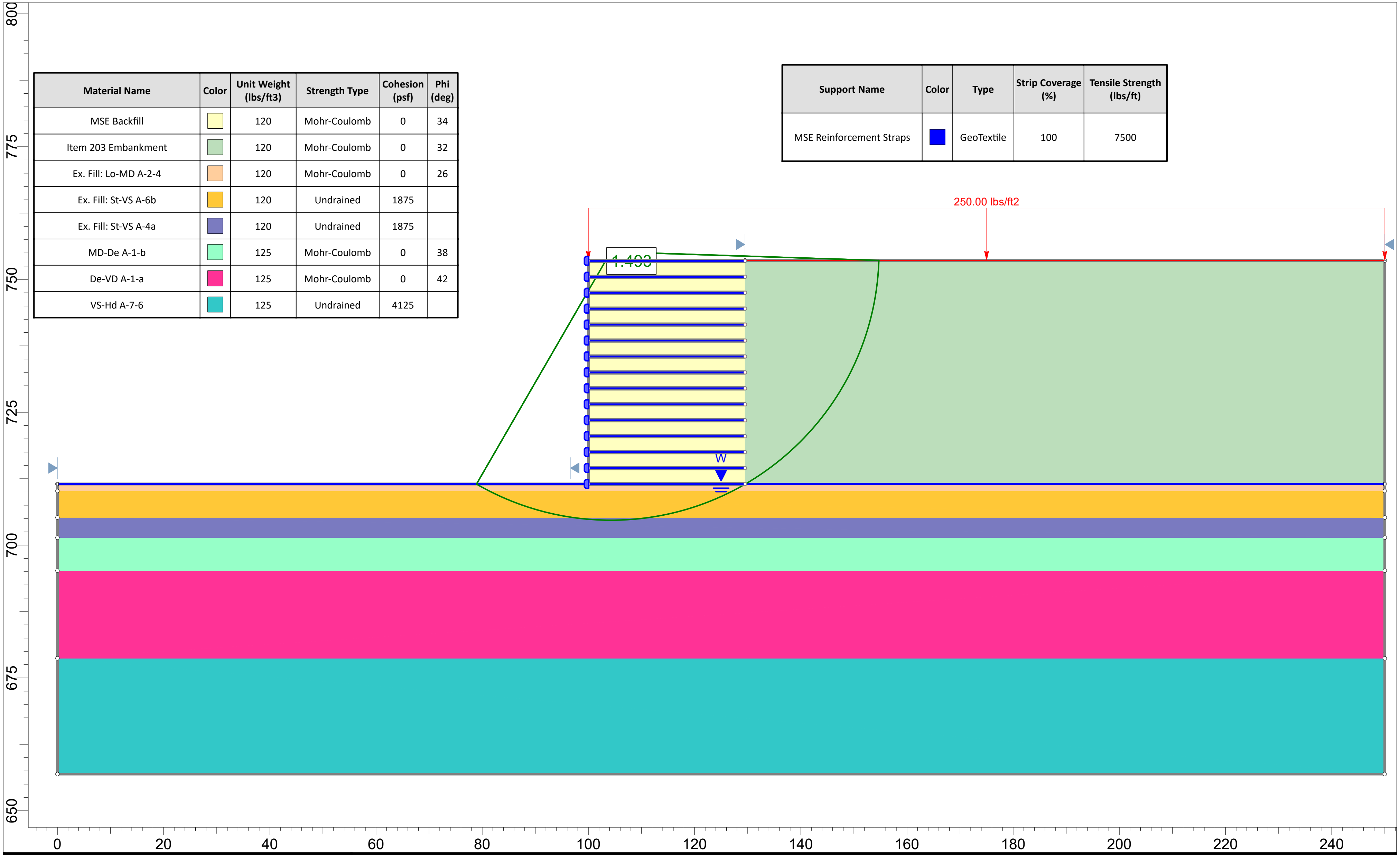
SLIDEINTERPRET 8.020

Project	Retaining Wall E4 - Sta. 407+00 to 409+50 - MSE Wall Global Stability
Analysis Description	42.1 ft Wall Height - Drained - Circular - Spencer's
Drawn By	BRT
Scale	1:300
Company	Resource International, Inc.
Date	07/18/2019
File Name	Retaining Wall E4 - Sta. 407+00 to 409+50 - Global Stability.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
MSE Backfill	🟨	120	Mohr-Coulomb	0	34
Item 203 Embankment	🟩	120	Mohr-Coulomb	0	32
Ex. Fill: Lo-MD A-2-4	🟧	120	Mohr-Coulomb	0	26
Ex. Fill: St-VS A-6b	🟧	120	Undrained	1875	
Ex. Fill: St-VS A-4a	🟪	120	Undrained	1875	
MD-De A-1-b	🟩	125	Mohr-Coulomb	0	38
De-VD A-1-a	🟥	125	Mohr-Coulomb	0	42
VS-Hd A-7-6	🟦	125	Undrained	4125	

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

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