Resource International, Inc.

FRA-70-13.10 PHASE 6A
RETAINING WALL E3
PID NO. 89464
FRANKLIN COUNTY, OHIO

STRUCTURE FOUNDATION EXPLORATION REPORT (REV. 1)

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

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Planning

Engineering

Construction Management

Technology

June 19, 2015 (Revised March 10, 2021)

Mr. Thomas P. Mosure, P.E.
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2221 Schrock Road
Columbus, OH 43229-1547

Re: Structure Foundation Exploration Report (Rev. 1)
FRA-70-13.10 Phase 6A
Retaining Wall E3
PID No. 89464
Rii Project No. W-13-072

Mr. Mosure:

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 E3 as part of the FRA-70-13.10 Phase 6A (PID 89464) 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 Services

Jonathan P. Sterenberg, P.E.
Vice President – Geotechnical Services

Enclosure: Structure Foundation Exploration Report (Rev. 1)

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EXECUTIVE SUMMARY

Resource International, Inc. (Rii) has completed a structure foundation exploration for the design and construction of the proposed Retaining Wall E3. Based on plan information provided by the Rii design team and ms consultants, Retaining Wall E3 will be located within the median of I-70, west of Short Street, and will provide the required grade separation between I-70 eastbound and westbound where the profile grades deviate to accommodate the profile geometrics for the roadway alignments. The wall begins at Sta. 172+62 (BL I-70 WB) and extends east along the median of I-70 where it connects to the rear abutment of the proposed FRA-70-1373L structure at Sta. 176+01 (BL I-70 WB). It is understood that a mechanically stabilized earth (MSE) wall type is being considered as the preferred wall type for the entire alignment of Retaining Wall E3. The wall heights along the wall alignment will range from 8.3 feet at Sta. 307+00 (BL Wall E3) to 12.9 feet at Sta. 310+36 (BL Wall E3), and the total wall length is approximately 336 lineal feet.

Exploration and Findings

Between February 18, 2014, and January 29, 2015, four (4) structural borings, designated as B-018-4-13, B-018-5-13, B-019-2-13, and B-020-5-13, were drilled to completion depths ranging from 50.0 to 90.0 feet below the existing ground surface at the locations shown on the boring plan provided in Appendix I of the full report.

All of the borings for this exploration were drilled within the existing shoulder of I-70 westbound, just west of Short Street. Borings B-018-4-13, B-018-5-13 and B-019-2-13 encountered 11.0 to 14.0 inches of asphalt overlying 8.0 to 12.0 inches of concrete at the existing ground surface. Boring B-020-5-13 encountered 6.0 inches of concrete overlying 6.0 inches of aggregate base at the ground surface.

Beneath the surface materials in all four borings, material identified as existing embankment fill was encountered extending to depths ranging from 22.0 feet to 32.0 feet below the ground surface, which corresponds to elevations ranging from 707.9 to 714.0 feet msl. The embankment fill material generally consisted of cohesive soils comprised of brown, dark brown and dark gray silt and clay and silty clay (ODOT A-6a, A-6b) with isolated seams of granular soils comprised of brown gravel, gravel and sand and gravel with sand, silt and clay (ODOT A-1-a, A-1-b, A-2-6). Underlying the embankment fill, material identified as existing fill was encountered in borings B-018-4-13, B-018-5-13 and B-019-2-13 extending to depths ranging from 37.0 to 47.0 feet below existing grade, which corresponds to elevations ranging from 692.1 to 699.0 feet msl. The existing fill material consisted of both cohesive and granular soils described as gray, black and brown gravel and sand, gravel with sand and silt, sandy silt, silt and clay and clay. The fill contained debris consisting of brick and slag fragments, cinders and organics.

A layer of black clay (ODOT A-7-6) was encountered in boring B-018-4-13 at a depth of 37.0 feet and extending to 45.0 feet below existing grade, which was described as highly organic. The organic content in this layer was 18.9 percent at depth of 38.5 feet below existing grade. A petroleum odor was noted within this layer.

Underlying the surficial materials and existing fill, natural soils were encountered consisting primarily of granular soils with intermittent seams of cohesive material in boring B-020-5-13. The granular soils were generally described as brown and gray gravel, gravel and sand and gravel with sand and silt (ODOT A-1-a, A-1-b, A-2-4). The cohesive soils encountered in boring B-020-5-13 were generally described as brown and gray sandy silt, silt and clay and silty clay (ODOT A-4a, A-6a, A-6b).

Top of bedrock in boring B-020-5-13 was encountered at a depth of 76.6 feet below the existing ground surface, which corresponds to an elevation of 656.8 feet msl. The upper 3.4 feet of bedrock was described as weathered, augerable shale. More competent bedrock consisting of mudstone and shale was encountered at a depth of 80.0 feet, which required rock coring techniques to advance the borings.

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 E3 is proposed to be a MSE wall type, which will support I-70 westbound just west of the FRA-70-1373L bridge over Short Street. Based on the proposed plan and profile information, wall heights along the alignment of the proposed structure are anticipated to range from 8.3 feet to a maximum height of 12.9 feet where the wall will connect to the rear abutment of the proposed FRA-70-1373L structure.

MSE Wall Recommendations

Based on the proposed plan and profile information, the proposed retaining wall will have a maximum height of 12.9 feet, as measured from the top of the leveling pad to the top of the coping. Approximately 7 to 12 feet of embankment fill will be required to bring the existing profile grade up to the proposed bottom of wall elevation. The bearing materials along the proposed alignment of Retaining Wall E3 will consist of approximately 7 to 12 feet of ODOT Item 203 embankment. MSE wall foundations bearing on engineered fill, placed and compacted in accordance with ODOT Item 203, may be proportioned for a nominal bearing resistance as indicated in the following table. A geotechnical resistance factor of φ_b=0.65 was considered in calculating the factored bearing resistance at the strength limit state. The reinforcement length presented in the following table represents the minimum foundation width required to satisfy external and global stability requirements, expressed as a percentage of the wall height.

Retaining Wall E3 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]	
307+00	310+36	12.9	Level	9.0 (0.70H ≥ 8.0)	10.80	7.02	307+00

1. *Station referenced to the baseline of Retaining Wall E3.*
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.*

The settlement analysis 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 7.0 to 12.2 feet of embankment fill that will be required to achieve the proposed bottom of wall elevation. Total settlements of up to 1.86 inches at the center of the reinforced soil mass and 1.22 inches at the facing of the wall are anticipated along the alignment of Retaining Wall E3. Based on the results of the analysis, 90 percent of the total settlement at the facing of the wall is anticipated to occur within 75 to 120 days following the completion of construction of the wall.

Based on the results of the external and global stability analysis performed for the MSE walls, the recommended controlling strap length is 0.70 times the maximum height of the MSE wall (measured from the top of the leveling pad to the proposed profile grade of the roadway). All of the external and global stability calculations indicate that adequate resistance is available for support of the MSE wall.

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

1.0 INTRODUCTION

The overall purpose of this project is to provide detailed subsurface information and recommendations for the design and construction of the FRA-70/71-13.10/14.36 (Projects 6A/6R) project in Columbus, Ohio. The projects represent the central portion of the 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-70-13.10 (Project 6A) phase will consist of all work associated with the construction of I-70 westbound from SR 315 to Front Street, including Ramps D3 and D7. This project includes the construction of one (1) new bridge structure for Ramp D3 over the Scioto River (FRA-70-1323C) and the reconstruction of three (3) bridges, including I-70 westbound over the Scioto River (FRA-70-1322L), I-70 westbound over CSX and Norfolk Southern (NS) Railroad (FRA-70-1358L) and I-70 westbound over Short Street (FRA-70-1373L), as well as the construction of five (5) new retaining walls (Walls E2, E3, E4, E7 and E9) 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 E3, as shown on the vicinity map and boring plan presented in Appendix I. Retaining Wall E3 will be located within the median of I-70, west of Short Street, and will provide the required grade separation between I-70 eastbound and westbound where the profile grades deviate to accommodate the profile geometrics for the roadway alignments. The wall begins at Sta. 172+62 (BL I-70 WB) and extends east along the median of I-70 where it connects to the rear abutment of the proposed FRA-70-1373L structure at Sta. 176+01 (BL I-70 WB). It is understood that a mechanically stabilized earth (MSE) wall type is being considered as the preferred wall type for the entire alignment of Retaining Wall E3. The wall heights along the wall alignment will range from 8.3 feet at Sta. 307+00 (BL Wall E3) to 12.9 feet at Sta. 310+36 (BL Wall E3), and the total wall length is approximately 336 lineal feet.

2.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1 Site Geology

Both the Illinoian and Wisconsinan glaciers advanced over two-thirds of the State of Ohio, leaving behind glacial features such as moraines, kame deposits, lacustrine deposits and outwash terraces. The glacial and non-glacial regions comprise five physiographic sections based on geological age, depositional process and geomorphic occurrence (physical features or landforms). The project area lies within the Columbus Lowland District of the Till Plains Section. This area is characterized by flat to gently rolling ground moraine deposits from the Late Wisconsinan age. The site topography exhibits moderate to high relief. The ground moraine deposits are composed primarily of silty loam till (Darby, Bellefontaine, Centerburg, Grand Lake, Arcanum, Knightstown Tills), with smaller alluvium and outwash deposits bordering the Scioto River, its tributaries and floodplain areas. A ground moraine is the sheet of debris left after the steady retreat of glacial ice.

The debris left behind ranges in composition from clay size particles to boulders (including silt, sand, and gravel). Outwash deposits consist of undifferentiated sand and gravel deposited by meltwater in front of glacial ice, and often occurs as valley terraces or low plains. Alluvium and alluvial terrace deposits range in composition from silty clay size particles to cobbles, usually deposited in present and former floodplain areas.

According to the bedrock geology and topography maps obtained from the Ohio Department of Natural Resources (ODNR), the underlying bedrock west of the Scioto River consists predominantly of the Middle to Lower Devonian-aged Columbus Limestone. This formation is further subdivided into two members in the central portion of the state, known as the Delhi and Bellepoint Members. The Delhi Member consists of light gray, finely to coarsely crystalline, irregularly bedded, fossiliferous limestone. The Bellepoint Member consists of variable brown, finely crystalline, massively bedded limey dolomite. Both of these members contain chert nodules. East of the Scioto River, the underlying bedrock consists of the Upper Devonian Ohio Shale Formation overlying the Middle Devonian-aged Delaware Limestone Formation. The Ohio Shale formation consists of brownish black to greenish gray, thinly bedded, fissile, carbonaceous shale. The Delaware Limestone consists of bluish gray, thin to medium bedded dolomitic limestone with nodules and layers of chert. Regionally, the bedrock surface forms a broad valley aligned roughly north-to-south beneath the Scioto River. According to bedrock topography mapping, the elevation of the bedrock surface ranges from approximately 600 feet mean sea level (msl) in the valley to approximately 625 feet msl near the project limits.

2.2 Existing Conditions

The proposed Retaining Wall E3 structure will be situated along the median of I-70, from the west end of the proposed FRA-70-1373L structure to approximately 400 feet west of Short Street. 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 26 feet above the Short Street profile grade. An asphalt/gravel access road is situated at the toe of the existing embankment which provides access to the railroad tracks from Short Street. There is a commercial property situated along the south side of the access road. The terrain along I-70 slopes gently to the west and the surrounding area is relatively flat-lying, and dense vegetation covers the existing I-70 embankment slope.

3.0 EXPLORATION

Between February 18, 2014, and January 29, 2015, four (4) structural borings, designated as B-018-4-13, B-018-5-13, B-019-2-13, and B-020-5-13, were drilled at the locations shown on the boring plan provided in Appendix I of this report and summarized in Table 1. The borings were advanced to completion depths ranging from 50.0 to 90.0 feet below the existing ground surface within the existing inside shoulder pavement of I-70 westbound. On February 23, 2014, auger refusal was encountered in boring B-020-5-13 at a depth of 75.5 feet below the ground surface, and a 1.1-foot rock core run recovered 9.0-inches of granite from a boulder. The boring could not be advanced beyond this depth using the hollow-stem augers, and due to time restrictions for the traffic control, the boring was terminated at this depth. On January 22, 2015, boring B-020-5-13 was extended to bedrock and cored to the final completion depth of 90.0 feet in accordance with ODOT SGE requirements.

Table 1. Test Boring Summary

Boring Number	Station [1]	Offset [1]	Latitude	Longitude	Ground Elevation (feet msl)	Boring Depth (feet)
B-018-4-13	171+35.06	40.2' Rt.	39.953396027	-83.006271115	743.1	50.0
B-018-5-13	173+01.62	30.1' Rt.	39.953431007	-83.005683369	739.1	50.0
B-019-2-13	174+39.64	20.3' Rt.	39.953448237	-83.005193743	736.0	50.0
B-020-5-13	175+57.74	11.8' Rt.	39.953452196	-83.004773258	733.4	90.0

1. Station and offset reference to the proposed baseline of I-70 WB.

The boring locations were determined and located in the field by Rii representatives. Rii utilized a handheld GPS unit to obtain northing and easting coordinates of the boring locations. Ground surface elevations at the boring locations were interpolated using topographic mapping information provided by ms consultants.

The borings were drilled using a truck or an all-terrain vehicle (ATV) mounted rotary drilling machine, utilizing a 3.25 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 20.0 or 30.0 feet, and at 5.0-foot intervals thereafter to the boring termination depth or top of bedrock. The SPT, per the American Society for Testing and Materials (ASTM) designation D1586, is conducted using a 140-pound hammer falling 30.0 inches to drive a 2.0-inch outside diameter split spoon sampler 18.0 inches. Rii utilized a calibrated automatic drop hammer to generate consistent energy transfer to the sampler. Driving resistance is recorded on the boring logs in terms of blow per 6.0-inch interval of the driving distance. The second and third intervals are added to obtain the number of blows per foot (N). Standard penetration blow counts aid in determining soil properties applicable in foundation system design. Measured blow count (N) values are corrected to an equivalent (60%) energy ratio, N_{60}, by the following equation. Both values are represented on boring logs in Appendix III.

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

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

The hammers for the Mobile B-53 and CME 750 drill rigs were calibrated on April 26, 2013, and have drill rod energy ratios of 77.7 and 82.6 percent, respectively.

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

The depth to bedrock in boring B-020-5-13 was determined by visual inspection of the weathered shale samples and based on the blow counts obtained from the SPT testing. An HQ-sized double-tube diamond bit core barrel (utilizing wire line equipment) was used to core the bedrock in boring B-020-5-13. Coring produced 2.5-inch diameter cores, from which the type of rock and geological characteristics were determined.

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

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

During drilling, field logs were prepared by Rii personnel showing the encountered subsurface conditions. Soil and rock samples obtained from the drilling operation were preserved and sealed in glass jars or rock core boxes and delivered to the soil laboratory. In the laboratory, the soil and rock 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	69
Plastic and Liquid Limits	AASHTO T89, T90	25
Gradation – Sieve/Hydrometer	AASHTO T88	25
Loss on Ignition	ASTM D2974	1

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

4.0 FINDINGS

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

4.1 Surface Materials

All of the borings for this exploration were drilled within the existing shoulder of I-70 westbound, just west of Short Street. Borings B-018-4-13, B-018-5-13 and B-019-2-13 encountered 11.0 to 14.0 inches of asphalt overlying 8.0 to 12.0 inches of concrete at the existing ground surface. Boring B-020-5-13 encountered 6.0 inches of concrete overlying 6.0 inches of aggregate base at the ground surface.

4.2 Subsurface Soils

Beneath the surface materials in all four borings, material identified as existing embankment fill was encountered extending to depths ranging from 22.0 feet to 32.0 feet below the ground surface, which corresponds to elevations ranging from 707.9 to 714.0 feet msl. The embankment fill material generally consisted of cohesive soils comprised of brown, dark brown and dark gray silt and clay and silty clay (ODOT A-6a, A-6b) with isolated seams of granular soils comprised of brown gravel, gravel and sand and gravel with sand, silt and clay (ODOT A-1-a, A-1-b, A-2-6). Underlying the embankment fill, material identified as existing fill was encountered in borings B-018-4-13, B-018-5-13 and B-019-2-13 extending to depths ranging from 37.0 to 47.0 feet below existing grade, which corresponds to elevations ranging from 692.1 to 699.0 feet msl. The existing fill material consisted of both cohesive and granular soils described as gray, black and brown gravel and sand, gravel with sand and silt, sandy silt, silt and clay and clay. The fill contained debris consisting of brick and slag fragments, cinders and organics.

A layer of black clay (ODOT A-7-6) was encountered in boring B-018-4-13 at a depth of 37.0 feet and extending to 45.0 feet below existing grade, which was described as highly organic. The organic content in this layer was 18.9 percent at depth of 38.5 feet below existing grade. A petroleum odor was noted within this layer.

Underlying the surficial materials and existing fill, natural soils were encountered consisting primarily of granular soils with intermittent seams of cohesive material in boring B-020-5-13. The granular soils were generally described as brown and gray gravel, gravel and sand and gravel with sand and silt (ODOT A-1-a, A-1-b, A-2-4). The cohesive soils encountered in boring B-020-5-13 were generally described as brown and gray sandy silt, silt and clay and silty clay (ODOT A-4a, A-6a, A-6b).

The relative density of granular soils is primarily derived from SPT blow counts (N_{60}). Based on the SPT blow counts obtained, the granular soil encountered ranged from medium dense ($11 \leq N_{60} \leq 30$ blows per foot [bpf]) to very dense ($N_{60} > 50$ bpf). Overall blow counts recorded from the SPT sampling ranged from 12 bpf to 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. The shear strength and consistency of the cohesive soils are primarily derived from the hand penetrometer values (HP). The cohesive soil encountered ranged from stiff (1.0 < HP ≤ 2.0 tsf) to hard (HP > 4.0 tsf). The unconfined compressive strength of the cohesive soil samples tested, obtained from the hand penetrometer, ranged from 1.75 to over 4.5 tsf (limit of instrument).

Natural moisture contents of the soil samples tested ranged from 4 to 23 percent. The natural moisture content of the cohesive soil samples tested for plasticity index ranged from 11 percent below to 2 percent above their corresponding plastic limits. In general, the soil exhibited natural moisture contents considered to be significantly below to slightly above optimum moisture levels.

4.3 Bedrock

Bedrock was encountered in boring B-020-5-13 as presented in Table 3.

Table 3. Top of Bedrock Elevations

Boring Number	Ground Surface Elevation (feet msl)	Top of Bedrock		Top of Bedrock Core	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-020-5-13	733.4	76.6	656.8	80.0	653.4

Top of bedrock in boring B-020-5-13 was encountered at a depth of 76.6 feet below the existing ground surface, which corresponds to an elevation of 656.8 feet msl. The upper 3.4 feet of bedrock was described as weathered, augerable shale. More competent bedrock consisting of mudstone and shale was encountered at a depth of 80.0 feet, which required rock coring techniques to advance the borings. The mudstone is described as gray, slightly weathered, very weak to weak, very thin to thick bedded, arenaceous, calcareous, friable, fissile, pyritic and slightly to highly fractured with tight to open, rough to very rough apertures. The shale is described as gray and black, unweathered to slightly weathered, very weak to slightly strong, laminated to thick bedded, calcareous, arenaceous, friable, fissile and moderately to highly fractured with tight to open, rough to very rough apertures.

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

Table 4. Rock Core Summary

Boring	Core No.	Elevation (feet msl)	Recovery (%)	RQD (%)	Unconfined Compressive Strength
B-020-5-13	RC-2	653.4 to 648.4	80	63	N/A
	RC-3	648.4 to 643.4	100	72	N/A

It should be noted that bedrock experiences mechanical breaks during the drilling and coring processes. Rii attempted to account for fresh, manmade breaks during tabulation of the RQD analysis. Please note that core run RC-1 was not included in the RQD tabulation above as a granite boulder was recovered from this run, which is not indicative of the actual bedrock at this site. The quality of the bedrock, according to the RQD values, was fair (50% < RQD ≤ 75%).

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-018-4-13	743.1	48.5	694.6	45.0	698.1
B-018-5-13	739.1	43.5	699.1	40.0	695.6
B-019-2-13	736.0	43.5	692.5	40.0	694.0
B-020-5-13	733.4	43.5	689.9	N/A [1]	N/A

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

Groundwater was encountered initially during drilling in all four borings at depths ranging from 43.5 to 48.5 feet below the ground surface, which corresponds to elevations ranging from 689.9 to 699.1 feet msl. At the completion of drilling and prior to removing the augers, groundwater accumulated in the auger stems in borings B-018-4-13, B-018-5-13 and B-019-2-13 ranged to depths ranging from 40.0 to 45.0 feet below existing grade, which corresponds to elevations ranging from 694.0 to 698.1 feet msl. The groundwater level in boring B-020-5-13 could not be measured due to the addition of mud during to counteract heaving sands during drilling as well as water as a circulating fluid during the rock coring process.

Please note that short-term water level readings, especially in cohesive soils, are not necessarily an accurate indication of the actual groundwater level. In addition, groundwater levels or the presence of groundwater are considered to be dependent on seasonal fluctuations in precipitation.

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

5.0 ANALYSES AND RECOMMENDATIONS

Data obtained from the subsurface exploration has been used to determine the foundation support capabilities and the settlement potential for the soil encountered at the site. These parameters have been used to provide guidelines for the design of foundation systems for the subject structure, as well as the construction specifications related to the placement of foundation systems and general earthwork recommendations, which are discussed in the following paragraphs.

Design details of the proposed retaining wall were provided by the Rii design team and ms consultants. It is understood that Retaining Wall E3 is proposed to be a MSE wall type, which will support I-70 westbound just west of the FRA-70-1373L bridge over Short Street. Based on the proposed plan and profile information, wall heights along the alignment of the proposed structure are anticipated to range from 8.3 feet to a maximum height of 12.9 feet where the wall will connect to the rear abutment of the proposed FRA-70-1373L structure.

5.1 MSE Wall Recommendations

MSE walls are constructed on earthen foundations at a minimum depth of 3.0 feet below grade, as defined by the top of the leveling pad to the ground surface located 4.0 feet from the face of the wall. Per Section 840.04.A of ODOT Supplemental Specification 840 (SS 840) and Section 3.11.5.8.1 of the 2020 AASHTO LRFD BDS, 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 307.4.A of the 2020 ODOT BDM and 840.04.A.2 of ODOT 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 12.9 feet, as measured from the top of the leveling pad to the top of the coping. Approximately 7 to 12 feet of embankment fill will be required to bring the existing profile grade up to the proposed bottom of wall elevation. 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. As previously stated, it is understood that the proposed wall will be constructed on approximately 7 to 12 feet of new embankment fill above the existing I-70 westbound grade. Additionally, as stated in Section 4.2, existing embankment fill was encountered in all of the borings performed for this structure extending to elevations ranging from 707.9 to 714.0 feet msl, which is approximately 36 feet below the proposed bottom of wall elevation. The existing embankment fill generally consisted of very stiff to hard silt and clay and silty clay (ODOT A-6a, A-6b) with isolated seams of medium dense to dense gravel, gravel and sand and gravel with sand, silt and clay (ODOT A-1-a, A-1-b, A-2-6). Based on the condition of the existing embankment fill encountered in the borings, it is anticipated that the embankment fill was placed and compacted in a controlled manner. Therefore, this soil in its current condition is considered adequate for support of the new fill material and the proposed MSE wall.

Per Section 307.4.C of the 2020 AASHTO LRFD BDS and Section 840.06.D of ODOT SS 840, following foundation subgrade inspection and acceptance, a minimum of 12.0 inches of ODOT Item 703.16.C, Granular Material Type C, should be placed and compacted in accordance with ODOT Item 204.07.

5.1.1 Strength Parameters Utilized in External and Global Stability Analyses

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

Table 6. Shear Strength Parameters Utilized in Stability Analyses

Material Type	γ (pcf)	φ' [1] (°)	c' [2] (psf)	S_u [3] (psf)
MSE Wall Backfill (Select granular backfill)	120	34	0	N/A
Item 203 Embankment Fill (Retained soil and new embankment)	120	30	0	2,000
Existing Embankment Fill: Very Stiff to Hard Silt and Clay (ODOT A-6a)	125	29	0	3,500
Hard Silty Clay (ODOT A-6b)	125	28	0	3,750
Medium Dense to Very Dense Gravel and Sand, Gravel with Sand and Silt (ODOT A-1-b, A-2-4)	125 to 135	35 to 41	0	N/A
Hard Sand Silt (ODOT A-4a)	130	33	0	8,000
Hard Silt and Clay (ODOT A-6a)	130	29	0	8,000

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

Shear strength parameters for the reinforced soil backfill and retained embankment are provided in Table 307-1 of the 2020 ODOT BDM and Section 840.04.A.3 of ODOT SS 840. Per these specifications, the select granular backfill in the reinforced zone and the retained embankment must meet the shear strength requirements provided in Table 6. The shear strength parameters for the natural soils were assigned using correlations provided in FHWA Geotechnical Engineering Circular (GEC) No. 5 (FHWA-NHI-16-072) Evaluation of Soil and Rock Properties and based on past experience in the vicinity of the site with projects performed in similar subsurface profiles.

5.1.2 Bearing Stability

The bearing materials along the proposed alignment of Retaining Wall E3 will consist of approximately 7 to 12 feet of ODOT Item 203 embankment. MSE wall foundations bearing on engineered fill, placed and compacted in accordance with ODOT Item 203, may be proportioned for a nominal 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. Given that the bearing soils will consist of new embankment fill material, the bearing resistance was evaluated under both drained and undrained conditions. The reinforcement length presented in the following table represents the minimum foundation width required to satisfy external and global stability requirements, expressed as a percentage of the wall height.

Table 7. Retaining Wall E3 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]	
307+00	310+36	12.9	Level	9.0 (0.70H ≥ 8.0)	10.80	7.02	3.58

1. *Station referenced to the baseline of Retaining Wall E3.*
2. *The required foundation width is expressed as a percentage of the wall height, H.*
3. *A geotechnical resistance factor of φ_b=0.65 was considered in calculating the factored bearing resistance at the strength limit state.*
4. *The strength limit equivalent bearing pressure is the uniformly distributed pressure asserted by the wall over an effective base width based on the eccentricity of the wall system at the strength limit state.*

Rii performed a verification of the bearing pressure exerted on the subgrade material for the maximum specified wall height indicated in Table 7. Based on the minimum length of reinforced soil mass presented, the factored equivalent bearing pressure exerted below the wall **will not exceed** the factored bearing resistance at the strength limit state.

5.1.3 Settlement Evaluation

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

Table 8. Compressibility Parameters Utilized in Settlement Analysis

Material Type	γ (pcf)	LL (%)	C_c (1)	C_r (2)	e_o (3)	C_v (4) (ft²/yr)	N_{60}	C' (5)
Existing Embankment Fill: Very Stiff to Hard Silt and Clay (ODOT A-6a)	125	30 to 32	0.180 to 0.198	0.014 to 0.015	0.507 to 0.522	700	N/A	N/A
Existing Embankment Fill: Stiff to Hard Silty Clay (ODOT A-6b)	120 to 125	36 to 38	0.234 to 0.252	0.018 to 0.019	0.553 to 0.569	400	N/A	N/A
Existing Embankment Fill: Medium Dense to Dense Gravel with Sand, Silt and Clay (ODOT A-2-6)	130	N/A	N/A	N/A	N/A	N/A	29	35 to 40
Medium Dense to Very Dense Natural Granular Soils (ODOT A-1-b, A-2-4, A-4a)	125 to 135	N/A	N/A	N/A	N/A	N/A	18 to 62	46 to 157
Hard Sand Silt (ODOT A-4a)	130	25	0.135	0.014	0.467	1,000	N/A	N/A

Material Type	γ (pcf)	LL (%)	C_c (1)	C_r (2)	e_o (3)	C_v (4) (ft^2/yr)	N_{60}	C' (5)
Hard Silt and Clay (ODOT A-6a)	120 to 130	25 to 30	0.135 to 0.180	0.014 to 0.018	0.467 to 0.507	600	N/A	N/A
Hard Silty Clay (ODOT A-6b)	125	33	0.207	0.021	0.530	300	N/A	N/A

1. *Per Table 6-9, Section 6.14.1 of FHWA GEC 5.*
2. *Estimated at 10% of C_c per Section 8.11 of Holtz and Kovacs (1981).*
3. *Per Table 8-2 of Holtz and Kovacs (1981).*
4. *Per Figure 6-37, Section 6.14.2 of FHWA GEC 5.*
5. *Per Figure 10.6.2.4.2b-1 of 2020 AASHTO LRFD BDS.*

The settlement analysis 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 7.0 to 12.2 feet of embankment fill that will be required to achieve the proposed bottom of wall elevation. Results of the settlement analysis are tabulated in Table 9. Total settlements of up to 1.86 inches at the center of the reinforced soil mass and 1.22 inches at the facing of the wall are anticipated along the alignment of Retaining Wall E3. Based on the results of the analysis, 90 percent of the total settlement at the facing of the wall is anticipated to occur within 75 to 120 days following the completion of construction of the wall. Please note that the consolidation settlement and time rate of consolidation are based on estimates using correlated compressibility parameters provided in Table 8 for the underlying soils. Actual settlement and time rate of consolidation should be determined by monitoring the settlement of the wall using settlement platforms.

Table 9. Retaining Wall E3 MSE Wall Settlement Values

From Station [1]	To Station [1]	Service Limit Equivalent Bearing Pressure [2] (ksf)	Total Settlement Values (inches)		Time for 90% Consolidation (Days)
			Center of Wall Mass	Facing of Wall	
307+00	310+36	1.47 to 2.43	1.07 to 1.86	0.88 to 1.22	75 to 115

1. *Station referenced to the baseline of Retaining Wall E3.*
2. *The service limit equivalent bearing pressure is the uniformly distributed pressure asserted by the wall over an effective base width based on the eccentricity of the wall system at the service limit state.*

Per Section 307.1.6 of the 2020 ODOT BDM, the maximum allowable differential settlement in the longitudinal direction (regardless of the size of panels) is one (1) percent (1/100). Based on the total anticipated settlement at the facing of the walls, maximum differential settlements in the longitudinal directions are anticipated to be less than 1/350, which is within the tolerable limit of 1/100. If the total or differential settlement values predicted for the proposed walls present an issue with respect to the deformation tolerances that the walls can withstand, then measures should be taken to minimize the amount of settlement that will occur. This can be achieved by preloading the site and consolidating the underlying soils prior to constructing the walls. If preloading the site is not a desired option, then consideration could be given to ground improvement through the use of stone columns. Settlement calculations are provided in Appendix IV.

5.1.4 Eccentricity (Overturning Stability)

The resistance of the MSE wall to overturning will be dependent on the location of the resultant force at the bottom of the wall due to the overturning and resisting moments acting on the wall. For MSE walls, overturning stability is determined by calculating the eccentricity of the resultant force from the midpoint of the base of the wall and comparing this value to a limiting eccentricity value. Per Section 11.10.5.5 of the 2020 AASHTO LRFD BDS, for foundations bearing on soil, the location of the resultant of the reaction forces shall be within the middle two-thirds ($^{2}/_{3}$) of the base width. Therefore, the limiting eccentricity is one-third ($^{1}/_{3}$) of the base width of the wall. Rii performed a verification of the eccentricity of the resultant force for the specified wall height indicated in Table 7. Based on the minimum length of reinforced soil mass presented in Table 7 and utilizing the soil parameters listed in Section 5.1.1 for the retained embankment material, the calculated eccentricity of the resultant force **will not exceed** the limiting eccentricity at the strength limit state.

5.1.5 Sliding Stability

The resistance of the MSE wall to sliding was evaluated per Section 11.10.5.3 of the 2020 AASHTO LRFD BDS. Given that the bearing soils will consist of new embankment fill material, the sliding resistance was evaluated under both drained and undrained conditions. For drained conditions, the sliding resistance is determined by multiplying a coefficient of sliding friction "f" times the total vertical force at the base of the wall. The coefficient of sliding friction is determined based on the limiting friction angle between the foundation 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 0.58 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 new embankment material is estimated to be 2.0 ksf.

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

5.1.6 Overall (Global) Stability

A slope stability analysis was performed to check the global stability of the wall. As per Section 11.6.2.3 of the 2020 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, manufactured by Rocscience Inc., was utilized to perform the analyses.

Per Section 307.1.2 of the 2020 ODOT BDM and Section 11.6.2.3 of the 2020 AASHTO LRFD BDS, overall (global) stability for MSE walls that are not integrated with or supporting structural foundations or elements, global stability is satisfied if the product of the factor of safety from the slope stability output multiplied by the resistance factor φ=0.75 is greater than 1.0. Therefore, global stability is satisfied when a minimum factor of safety of 1.3 is obtained. For an MSE wall designed with the minimum strap length listed in Table 7, the resulting factor of safety under drained conditions (long-term stability) was greater than 1.3. The wall was also evaluated under undrained conditions (short-term stability) to verify the stability of the wall during and immediately following construction. The resulting factor of safety under undrained conditions was also greater 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 walls, the recommended controlling strap length is 0.70 times the maximum height of the MSE wall (measured from the top of the leveling pad to the proposed profile grade of the roadway). All of the external and global stability calculations indicate that adequate resistance is available for support of the MSE wall.

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

5.2 Lateral Earth Pressure

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

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

Soil Type	γ (pcf) [1]	c (psf)	φ	k_a	k_o	k_p
Soft to Stiff Cohesive Soil	115	1,500	0°	N/A	N/A	N/A
Very Stiff to Hard Cohesive Soil	125	3,000	0°	N/A	N/A	N/A
Very Loose to Loose Granular Soil	120	0	28°	0.32	0.53	5.07
Medium Dense Granular Soil	130	0	32°	0.27	0.47	6.82
Dense to Very Dense Granular Soil	135	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	50	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	130	0	32°	0.27	0.47	6.82
Dense to Very Dense Granular Soil	135	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. Active earth pressure is developed as the structure moves away from the backfill or retained soil, while passive pressure is developed as the structure moves towards the backfill. A relatively small amount of lateral movement is needed to reach the active condition (≥ 0.1 percent of the height), whereas the movements required to engage the passive condition are approximately ten times greater than those required to develop active earth pressure. 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 assumed). Earth pressures on excavation support systems will be dependent on the type of sheeting and method of bracing or anchorage. Surcharge loads, such as that imposed by traffic loading, will create additional lateral loading on the subsurface structures and excavation support systems. The resulting lateral earth pressure should be evaluated based on active (k_a) and at-rest (k_o) conditions and the anticipated magnitude of the loading.

Where necessary, temporary retaining structures such as sheet pile system should be designed using the undrained soil parameters provided in Table 10, and the design should follow all applicable guidelines for the type of retaining structure utilized. Permanent retaining and subsurface structures should be designed using the drained soil parameters provided in Table 11. Regardless of whether the retaining structure is temporary or permanent, the effective unit weight ($\gamma' = \gamma$ - 62.4 pcf) plus the hydrostatic water pressure ($\gamma_w * h_w$, where h_w is the height of water behind the wall above the base of the wall) should be utilized below the design groundwater level. The lateral earth pressure coefficients should only be applied to the horizontal pressure resulting from the effective overburden pressure, and should not be applied to the hydrostatic water pressure.

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

Table 12. Excavation Back Slopes

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

5.3.2 Groundwater Considerations

Based on the groundwater observations made during drilling, groundwater is not anticipated to be encountered during construction of the proposed retaining wall. However, 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.

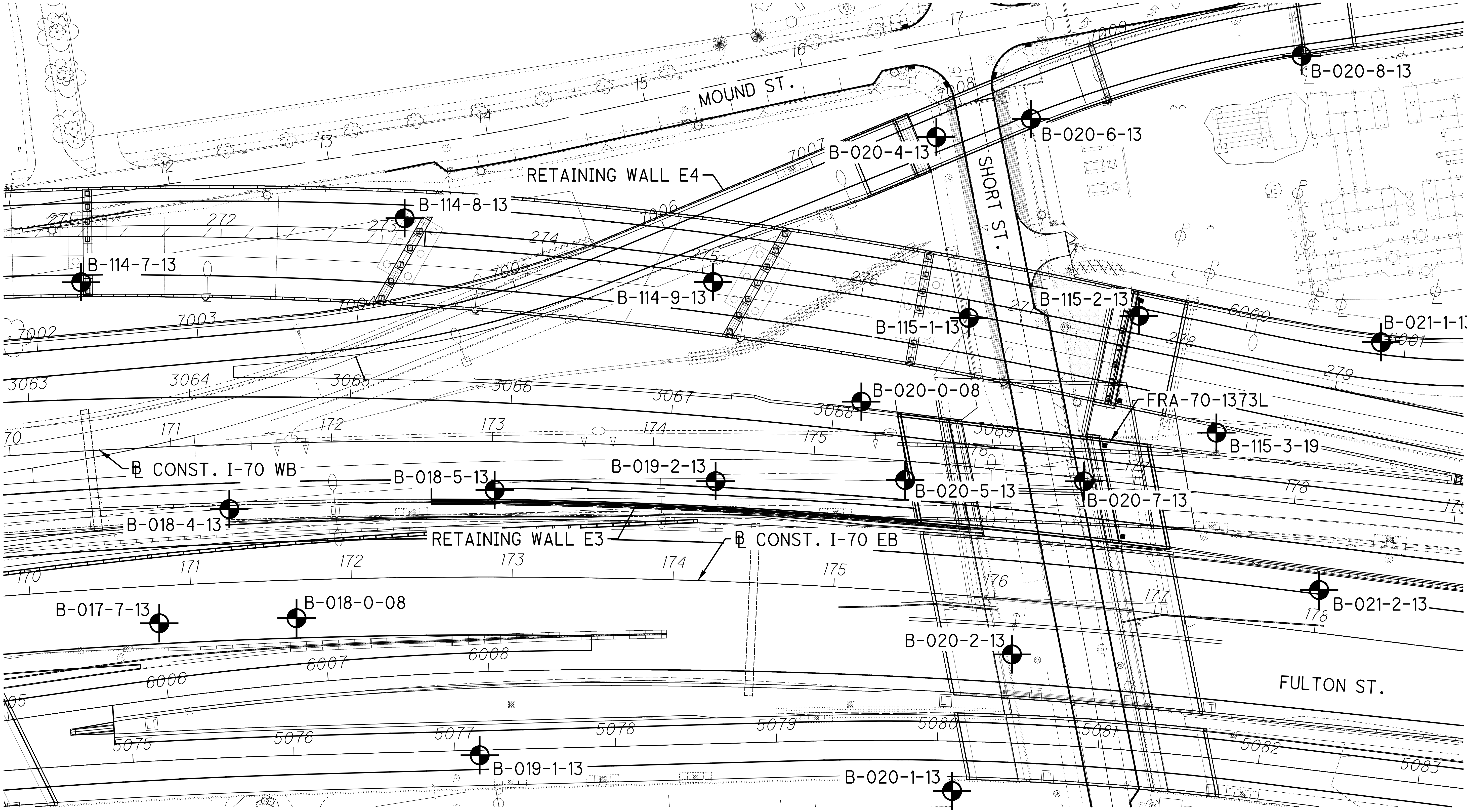
The conclusions and recommendations herein have been based upon the available soil and bedrock information and the design details furnished by a representative of the owner of the proposed project. Any revision in the plans for the proposed construction from those anticipated in this report should be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

FRA-70-13.10 - RETAINING WALL E3

FRANKLIN COUNTY, OHIO

PROJECT NO. Rii W-13-072		DRAWN RRM
SCALE: 1"=60' 0 30 60	N	REVIEWED BRT
		DATE 3/8/2021

RESOURCE INTERNATIONAL, INC.

APPENDIX II

DESCRIPTION OF SOIL TERMS

DESCRIPTION OF SOIL TERMS

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

DESCRIPTION OF ROCK TERMS

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

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

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

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

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

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

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

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

Degree of Fracturing

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

Aperture Width

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

Surface Roughness

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

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

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

CLASSIFICATION OF SOILS

Ohio Department of Transportation

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

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

MATERIAL CLASSIFIED BY VISUAL INSPECTION

- Sod and Topsoil
- Pavement or Base
- Uncontrolled Fill (Describe)
- Bouldery Zone
- P Peat

* 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-018-4-13, B-018-5-13, B-019-2-13 and B-020-5-13

BORING LOGS

Definitions of Abbreviations

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

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

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

<u>Classification Test Data</u>

Gradation (as defined on Description of Soil Terms):

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

Atterberg Limits:

LL	=	Liquid limit
PL	=	Plastic limit
PI	=	Plasticity Index
WC	=	Water content (%)

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-13.10 - PHASE 6A	DRILLING FIRM / OPERATOR: RII / J.K.	DRILL RIG: MOBILE B-53 (SN 624400)	STATION / OFFSET: 171+35.06 / 40.2' RT	EXPLORATION ID **B-018-4-13**	
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / S.B.	HAMMER: AUTOMATIC	ALIGNMENT: BL I-70 WB		
PID: 89464 BR ID: N/A	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 4/26/13	ELEVATION: 743.1 (MSL) EOB: 50.0 ft.	PAGE 1 OF 2	
START: 3/26/14 END: 3/27/14	SAMPLING METHOD: SPT	ENERGY RATIO (%): 77.7	LAT / LONG: 39.953396, -83.006271		

MATERIAL DESCRIPTION AND NOTES	ELEV. 743.1	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
0.9' - ASPHALT (11.0")	742.2																	
0.8' - CONCRETE (9.0")	741.4	1																
FILL: VERY STIFF TO HARD, DARK GRAY AND BROWN **SILTY CLAY**, LITTLE TO SOME FINE GRAVEL, LITTLE TO SOME COARSE TO FINE SAND, DAMP TO MOIST.		2, 3	5 5 7	16	67	SS-1	4.25	-	-	-	-	-	-	-	-	21	A-6b (V)	
		4, 5	11 11 12	30	67	SS-2	4.00	27	15	8	26	24	38	18	20	7	A-6b (7)	
-ROCK FRAGMENTS PRESENT THROUGHOUT		6, 7, 8	8 8 8	21	44	SS-3	2.25	-	-	-	-	-	-	-	-	17	A-6b (V)	
		9, 10	15 14 10	31	67	SS-4	4.5+	12	6	10	41	31	34	17	17	14	A-6b (10)	
	730.1	11, 12	15 12 10	28	44	SS-5	3.00	-	-	-	-	-	-	-	-	12	A-6b (V)	
LIMESTONE FRAGMENTS.		13, 14	8 11 14	32	6	SS-6	-	-	-	-	-	-	-	-	-	-	A-1-a (V)	
	727.6	15	15	-	0	2S-6A	-	-	-	-	-	-	-	-	-	-		
FILL: VERY STIFF TO HARD, DARK GRAY AND BROWN **SILTY CLAY**, SOME FINE GRAVEL, LITTLE COARSE TO FINE SAND, DAMP.		16, 17	10 15 10	32	56	SS-7	-	-	-	-	-	-	-	-	-	12	A-6b (V)	
-ROCK FRAGMENTS PRESENT THROUGHOUT	723.1	18, 19	8 11 8	25	67	SS-8	3.00	21	10	6	29	34	38	18	20	17	A-6b (10)	
FILL: DARK BROWN **GRAVEL WITH SAND**, TRACE SILT, TRACE CLAY, MOIST.	720.1	20, 21, 22			29	ST-9	-	-	-	-	-	-	-	-	-	7	A-1-b (V)	
FILL: VERY STIFF TO HARD, DARK GRAY AND BROWN **SILTY CLAY**, SOME FINE GRAVEL, LITTLE COARSE TO FINE SAND, DAMP.		23, 24, 25	7 11 12	30	33	SS-10	4.00	-	-	-	-	-	-	-	-	15	A-6b (V)	
-ROCK FRAGMENTS PRESENT THROUGHOUT		26, 27, 28																
		29	10 8 15	30	67	SS-11	4.5+	29	10	9	20	32	38	18	20	12	A-6b (7)	

PID: 89464	BR ID: N/A	PROJECT: FRA-70-13.10 - PHASE 6A	STATION / OFFSET: 171+35.06 / 40.2 RT	START: 3/26/14	END: 3/27/14	PG 2 OF 2	B-018-4-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 713.1	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
FILL: VERY STIFF TO HARD, DARK GRAY AND BROWN **SILTY CLAY**, SOME FINE GRAVEL, LITTLE COARSE TO FINE SAND, DAMP. *(same as above)*	711.1	31 – 32															
FILL: VERY DENSE, GRAY **GRAVEL WITH SAND**, TRACE SILT, TRACE CLAY, DAMP. -LIMESTONE FRAGMENTS PRESENT IN SS-12		33 – 34	40 50/1"	-	100	SS-12	-	-	-	-	-	-	-	-	-	4	A-1-b (V)
	706.1	35 – 37															
FILL: HARD, BLACK **CLAY**, "AND" SILT, LITTLE COARSE TO FINE SAND, DAMP TO MOIST, HIGHLY ORGANIC. -LLo @ 38.5' = 37% -LOI @ 38.5' = 18.9% -ASPHALT FRAGMENTS PRESENT IN SS-13 -PETROLEUM ODOR PRESENT IN SS-13 AND SS-14		38 – 40	7 7 9	21	78	SS-13	4.50	0	4	11	48	37	41	23	18	23	A-7-6 (11)
		41 – 43															
	698.1	44 – 45	17 33 25	75	11	SS-14	4.50	-	-	-	-	-	-	-	-	20	A-7-6 (V)
DENSE, BROWN **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, MOIST.		45	11	-	50	SS-14A	-	-	-	-	-	-	-	-	-	9	A-1-b (V)
		46 – 48															
	693.1	49 – 50 EOB	7 16 14	39	72	2S-15	-	62	9	11	14	4	NP	NP	NP	9	A-1-b (0)

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 48.5' AND AT COMPLETION @ 45.0'; CAVE-IN DEPTH @ 20.0'

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

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: 173+01.62 / 30.1' RT	EXPLORATION ID **B-018-5-13**
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / S.B.	HAMMER: AUTOMATIC	ALIGNMENT: BL I-70 WB	
PID: 89464 BR ID: N/A	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 4/26/13	ELEVATION: 739.1 (MSL) EOB: 50.0 ft.	PAGE 1 OF 2
START: 3/25/14 END: 3/26/14	SAMPLING METHOD: SPT	ENERGY RATIO (%): 77.7	LAT / LONG: 39.953431, -83.005683	

MATERIAL DESCRIPTION AND NOTES	ELEV. 739.1	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
1.0' - ASPHALT (12.0")	738.1	1																
1.0' - CONCRETE (12.0")	737.1	2																
FILL: VERY STIFF, DARK BROWN **SILTY CLAY**, SOME COARSE TO FINE SAND, LITTLE FINE GRAVEL, MOIST.	735.6	3	6 6 5	14	83	SS-1	3.75	-	-	-	-	-	-	-	-	21	A-6b (V)	
FILL: MEDIUM DENSE TO DENSE, BROWN TO DARK GRAY **GRAVEL WITH SAND, SILT, AND CLAY**, DAMP TO WET. -CONCRETE FRAGMENTS PRESENT IN SS-2		4 5	6 8 12	26	33	SS-2	-	36	20	9	22	13	30	17	13	12	A-2-6 (1)	
-ROCK FRAGMENTS PRESENT THROUGHOUT		6 7 8	13 8 11	25	78	SS-3	-	-	-	-	-	-	-	-	-	20	A-2-6 (V)	
-COBBLES ENCOUNTERED @ 8.5'	728.6	9 10	20 15 13	36	67	SS-4	-	45	11	9	21	14	30	17	13	7	A-2-6 (1)	
FILL: STIFF TO HARD, BROWN **SILTY CLAY**, LITTLE TO SOME COARSE TO FINE SAND, LITTLE FINE GRAVEL, DAMP TO MOIST. -BRICK FRAGMENTS PRESENT IN SS-5		11 12 13	10 12 14	34	89	SS-5	4.5+	-	-	-	-	-	-	-	-	17	A-6b (V)	
		14 15	6 8 14	28	44	SS-6	4.5+	-	-	-	-	-	-	-	-	17	A-6b (V)	
		16 17 18	7 11 14	32	56	SS-7	4.00	14	9	15	31	31	36	16	20	17	A-6b (9)	
-ROCK FRAGMENTS PRESENT THROUGHOUT		19 20	5 6 5	14	67	SS-8	1.75	-	-	-	-	-	-	-	-	20	A-6b (V)	
		21 22 23			38	ST-9	2.00	19	8	6	35	32	38	19	19	19	A-6b (10)	
	712.1	24 25 26	6 5 4	12	50	SS-10	-	-	-	-	-	-	-	-	-	12	A-6b (V)	
FILL: MEDIUM DENSE TO VERY DENSE, BLACK AND BROWN TO BROWN **GRAVEL WITH SAND AND SILT**, TRACE CLAY, MOIST.		27 28 29	43 35 24	76	100	SS-11	-	32	19	18	21	10	NP	NP	NP	16	A-2-4 (0)	

PID: 89464	BR ID: N/A	PROJECT: FRA-70-13.10 - PHASE 6A	STATION / OFFSET: 173+01.62 / 30.1 RT	START: 3/25/14	END: 3/26/14	PG 2 OF 2	B-018-5-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 709.1	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
FILL: MEDIUM DENSE TO VERY DENSE, BLACK AND BROWN TO BROWN **GRAVEL WITH SAND AND SILT**, TRACE CLAY, MOIST. *(same as above)*		31 – 33																
		34	7 14 11	32	33	SS-12	-	-	-	-	-	-	-	-	-	11	A-2-4 (V)	
-BRICK, CINDER, AND LIMESTONE FRAGMENTS PRESENT THROUGHOUT		35 – 38																
		39	4 30 20	65	56	SS-13	-	46	19	11	16	8	32	25	7	13	A-2-4 (0)	
		▼ 40 – 43																
		W 44	5 7 7	18	61	SS-14	-	-	-	-	-	-	-	-	-	11	A-2-4 (V)	
	692.1	45 – 46																
DENSE, BROWN **GRAVEL**, SOME COARSE TO FINE SAND, LITTLE SILT, TRACE CLAY, WET.		47 – 48																
		49	10 16 17	43	100	SS-15	-	63	16	7	11	3	23	19	4	21	A-1-a (0)	
	689.1	EOB 50																

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 43.5' AND AT COMPLETION @ 40.0'

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

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: 174+39.64 / 20.3' RT
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / S.B.	HAMMER: AUTOMATIC	ALIGNMENT: BL I-70 WB
PID: 89464 BR ID: N/A	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 4/26/13	ELEVATION: 736.0 (MSL) EOB: 50.0 ft.
START: 3/24/14 END: 3/24/14	SAMPLING METHOD: SPT	ENERGY RATIO (%): 77.7	LAT / LONG: 39.953448, -83.005194

EXPLORATION ID B-019-2-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 736.0	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
1.2' - ASPHALT (14.0")	734.8																	
0.7' - CONCRETE (8.0")	734.1																	
FILL: VERY STIFF TO HARD, BROWN TO DARK BROWN **SILTY CLAY**, SOME COARSE TO FINE SAND, SOME FINE GRAVEL, DAMP TO MOIST.			6 7 7	18	83	SS-1	2.50	-	-	-	-	-	-	-	-	22	A-6b (V)	
			8 8 13	27	83	SS-2	3.75	22	20	7	25	26	38	18	20	17	A-6b (7)	
-ROCK FRAGMENTS AND ORGANICS PRESENT THROUGHOUT			13 13 13	34	33	SS-3	4.25	-	-	-	-	-	-	-	-	16	A-6b (V)	
	725.5		6 10 12	28	44	SS-4	3.50	-	-	-	-	-	-	-	-	17	A-6b (V)	
FILL: VERY STIFF, BROWN **SILT AND CLAY**, SOME FINE GRAVEL, SOME COARSE TO FINE SAND, DAMP. -ROCK FRAGMENTS PRESENT IN SS-5	723.0		15 11 12	30	78	SS-5	4.00	33	11	12	23	21	30	17	13	13	A-6a (3)	
FILL: HARD, DARK BROWN TO BROWN **SILTY CLAY**, LITTLE COARSE TO FINE SAND, LITTLE FINE GRAVEL, DAMP TO MOIST. -ORGANICS PRESENT IN SS-6			16 14 12	34	72	SS-6	4.25	-	-	-	-	-	-	-	-	19	A-6b (V)	
-ROCK FRAGMENTS PRESENT THROUGHOUT			8 12 20	41	72	SS-7	4.50	-	-	-	-	-	-	-	-	16	A-6b (V)	
	714.0		9 12 17	38	67	SS-8	4.50	14	7	8	35	36	37	18	19	17	A-6b (11)	
FILL: MEDIUM DENSE, LIGHT BROWN **GRAVEL WITH SAND AND SILT**, TRACE CLAY, DAMP.			12 13 9	28	17	SS-9	-	-	-	-	-	-	-	-	-	9	A-2-4 (V)	
-ROCK FRAGMENTS PRESENT IN 2S-9A	709.0		7	-	100	2S-9A	-	-	-	-	-	-	-	-	-	7	A-2-4 (V)	
FILL: MEDIUM DENSE, BLACK **SANDY SILT**, LITTLE FINE GRAVEL, LITTLE CLAY, MOIST.			17 13 10	30	0	SS-10	-	-	-	-	-	-	-	-	-	-		

PID: 89464 | BR ID: N/A | PROJECT: FRA-70-13.10 - PHASE 6A | STATION / OFFSET: 174+39.64 / 20.3 RT | START: 3/24/14 | END: 3/24/14 | PG 2 OF 2 | B-019-2-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 706.0	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
FILL: MEDIUM DENSE, BLACK **SANDY SILT**, LITTLE FINE GRAVEL, LITTLE CLAY, MOIST. *(same as above)* -CHLORINE ODOR, ORGANICS, AND BRICK, CINDER, AND SLAG FRAGMENTS PRESENT IN 2S-10A	704.0	31 – 32	8	-	67	2S-10A	-	17	21	24	22	16	27	24	3	16	A-4a (1)	
FILL: HARD, DARK BROWN **SILT AND CLAY**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST. -BRICK FRAGMENTS PRESENT IN SS-11	699.0	33 – 37	3 / 6 / 6	16	89	SS-11	4.25	-	-	-	-	-	-	-	-	21	A-6a (V)	
MEDIUM DENSE, BROWN **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, MOIST. -ROCK FRAGMENTS PRESENT IN SS-12	694.0	38 – 42 (▼ 40)	8 / 9 / 8	22	72	SS-12	-	50	14	13	18	5	NP	NP	NP	9	A-1-b (0)	
MEDIUM DENSE, BROWN **GRAVEL WITH SAND AND SILT**, TRACE CLAY, MOIST. -ROCK FRAGMENTS PRESENT THROUGHOUT		43 – 45 (W 43.5)	7 / 10 / 6	21	6	SS-13	-	-	-	-	-	-	-	-	-	14	A-2-4 (V)	
	689.0	45 – 47	11	-	100	2S-13A	-	39	14	20	19	8	NP	NP	NP	16	A-2-4 (0)	
DENSE, BROWN **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, MOIST. -ROCK FRAGMENTS PRESENT IN SS-14	686.0	48 – 50 EOB	27 / 20 / 10	39	33	SS-14	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 43.5' AND AT COMPLETION @ 40.0'; CAVE-IN DEPTH @ 20.0'

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

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-13.10 - PHASE 6A	DRILLING FIRM / OPERATOR: RII / J.B./J.K.	DRILL RIG: CME-750 (SN 98048)	STATION / OFFSET: 175+57.74 / 11.8' RT	EXPLORATION ID **B-020-5-13**
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / S.B./N.A.	HAMMER: CME AUTOMATIC	ALIGNMENT: BL I-70 WB	
PID: 89464 BR ID: FRA-70-1373L	DRILLING METHOD: 4.25" HSA / HQ	CALIBRATION DATE: 4/26/13	ELEVATION: 733.4 (MSL) EOB: 90.0 ft.	PAGE 1 OF 3
START: 2/18/14 END: 1/29/15	SAMPLING METHOD: SPT / RC	ENERGY RATIO (%): 82.6	LAT / LONG: 39.953452, -83.004773	

MATERIAL DESCRIPTION AND NOTES	ELEV. 733.4	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
0.5' - CONCRETE (6.0")	732.9																	
0.5' - AGGREGATE BASE (6.0")	732.4																	
FILL: VERY STIFF TO HARD, BROWN **SILT AND CLAY**, LITTLE COARSE TO FINE SAND, LITTLE FINE GRAVEL, DAMP TO MOIST.																		
-WOOD FRAGMENTS PRESENT IN SS-1		4 - 5	8 9 10	26	50	SS-1	3.50	-	-	-	-	-	-	-	-	17	A-6a (V)	
-IRON STAINING PRESENT IN SS-2		9 - 10	4 7 8	21	44	SS-2	3.75	-	-	-	-	-	-	-	-	20	A-6a (V)	
-ROCK FRAGMENTS PRESENT THROUGHOUT		14 - 15	5 7 11	25	67	SS-3	4.25	11	8	16	36	29	32	17	15	15	A-6a (8)	
-IRON STAINING PRESENT IN SS-4		19 - 20	4 9 12	29	78	SS-4	4.00	-	-	-	-	-	-	-	-	17	A-6a (V)	
-BRICK FRAGMENTS PRESENT IN SS-5		21 - 22	12 15 15	41	89	SS-5	4.50	-	-	-	-	-	-	-	-	17	A-6a (V)	
-LIMESTONE FRAGMENTS PRESENT IN SS-6	707.9	24 - 25	15 8 13	29	44	SS-6	4.50	-	-	-	-	-	-	-	-	15	A-6a (V)	
HARD, BROWN **SILTY CLAY**, LITTLE COARSE TO FINE SAND, LITTLE FINE GRAVEL, MOIST. -IRON STAINING PRESENT IN SS-7		26 - 27	6 8 12	28	56	SS-7	4.50	13	13	15	29	30	33	16	17	18	A-6b (8)	
		29 - 30	4 6 10	22	89	SS-8	4.50	-	-	-	-	-	-	-	-	23	A-6b (V)	

PID: 89464	BR ID: FRA-70-1373L	PROJECT: FRA-70-13.10 - PHASE 6A	STATION / OFFSET: 175+57.74 / 11.8 RT	START: 2/18/14	END: 1/29/15	PG 2 OF 3	B-020-5-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 703.4	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
HARD, BROWN **SILTY CLAY**, LITTLE COARSE TO FINE SAND, LITTLE FINE GRAVEL, MOIST. *(same as above)*		31 – 32	9 14 15	40	72	SS-9	4.50	-	-	-	-	-	-	-	-	19	A-6b (V)	
	700.4	33																
MEDIUM DENSE TO VERY DENSE, BROWN TO REDDISH BROWN **GRAVEL WITH SAND AND SILT**, LITTLE CLAY, MOIST.		34 – 35	5 6 6	17	56	SS-10	-	-	-	-	-	-	-	-	-	14	A-2-4 (V)	
		36 – 37	10 8 10	25	100	SS-11	-	33	21	12	25	9	24	19	5	13	A-2-4 (0)	
		38 – 40	15 20 30	69	83	SS-12	-	-	-	-	-	-	-	-	-	13	A-2-4 (V)	
	692.9																	
MEDIUM DENSE TO VERY DENSE, BROWN TO GRAY **GRAVEL WITH SAND**, TRACE SILT, TRACE CLAY, DAMP TO MOIST.		41 – 42	19 19 17	50	78	SS-13	-	-	-	-	-	-	-	-	-	6	A-1-b (V)	
-INTRODUCED MUD @ 43.0'		W 43 – 45	18 19 24	59	56	SS-14	-	-	-	-	-	-	-	-	-	8	A-1-b (V)	
		46 – 47	11 7 10	23	100	SS-15	-	23	46	18	9	4	NP	NP	NP	14	A-1-b (0)	
-ROCK FRAGMENTS PRESENT THROUGHOUT		48 – 50	15 23 27	69	56	SS-16	-	-	-	-	-	-	-	-	-	8	A-1-b (V)	
-STRONG PETROLEUM ODOR PRESENT IN SS-17		51 – 54	50/5"	-	100	SS-17	-	-	-	-	-	-	-	-	-	13	A-1-b (V)	
	676.4	55 – 57																
HARD, GRAY **SANDY SILT**, SOME CLAY, TRACE FINE GRAVEL, MOIST.		58 – 60	15 30 50/5"	-	59	SS-18	4.50	5	9	18	45	23	25	15	10	17	A-4a (7)	
	671.4	61																

PID: 89464	BR ID: FRA-70-1373L	PROJECT: FRA-70-13.10 - PHASE 6A	STATION / OFFSET: 175+57.74 / 11.8 RT	START: 2/18/14	END: 1/29/15	PG 3 OF 3	B-020-5-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 671.3	DEPTHS
VERY DENSE, BROWN **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, DAMP. *(same as above)*	669.2	63–64
HARD, BROWN **SILT AND CLAY**, "AND" COARSE TO FINE SAND, LITTLE FINE GRAVEL, DAMP. -ROCK FRAGMENTS PRESENT IN SS-20	661.4	64–72
HARD, BROWN **SILTY CLAY**, LITTLE FINE GRAVEL, TRACE COARSE TO FINE SAND, DAMP. AUGER REFUSAL @ 75.5'	657.9	72–75.5
GRANITE BOULDER -BORING TERMINATED @ 76.6' ON 2-23-14. RESUMED DRILLING ON 1-22-15 AND CONTINUED SAMPLING @ 78.5'.	656.8	75.5–76.6 (TR)
SHALE : GRAY, HIGHLY WEATHERED, VERY WEAK.	653.4	76.6–80
MUDSTONE : GRAY, SLIGHTLY WEATHERED, VERY WEAK TO WEAK, VERY THIN TO THICK BEDDED, ARENACEOUS, CALCAREOUS, FRIABLE, FISSILE, PYRITIC, SLIGHTLY TO HIGHLY FRACTURED, TIGHT TO OPEN APERTURES, ROUGH TO VERY ROUGH; RQD 73%, REC 88%. -0.3' GRANITE BOULDER @ 80.0' -0.3' LIMESTONE SEAM @ 82.6'	645.4	80–88
SHALE : BLACK AND GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, VERY WEAK TO SLIGHTLY STRONG, LAMINATED TO THICK BEDDED, ARENACEOUS, CALCAREOUS, FRIABLE, FISSILE, MODERATELY TO HIGHLY FRACTURED, TIGHT TO OPEN APERTURES, ROUGH TO VERY ROUGH; RQD 46%, REC 100%.	643.4	88–90 (EOB)

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
8 / 30 / 45	103	61	SS-19	-	-	-	-	-	-	-	-	-	9	A-1-b (V)
				4.50	-	-	-	-	-	-	-	-	9	A-6a (V)
25 / 30 / 50	110	78	SS-20	4.50	20	13	23	24	20	25	12	13	10	A-6a (3)
24 / 17 / 30	65	89	SS-21	4.50	-	-	-	-	-	-	-	-	17	A-6b (V)
		62	RC-1	-	-	-	-	-	-	-	-	-	-	
49 / 29 / 37	91	17	SS-22	-	-	-	-	-	-	-	-	-	13	Rock (V)
63		80	RC-2											CORE
72		100	RC-3											CORE

HOLE SEALED

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 43.5'

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

B-020-5-13 – RC-1 – Depth from 75.5 to 76.6 feet

B-020-5-13 – RC-2 and RC-3 – Depth from 80.0 to 90.0 feet

APPENDIX IV

MSE WALL CALCULATIONS

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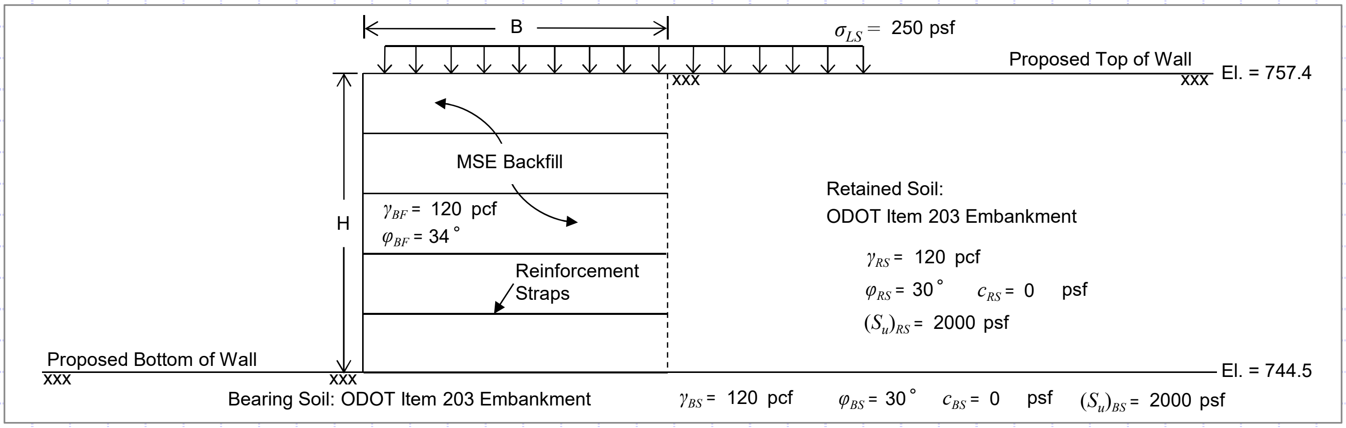
Retaining Wall E3 - Sta. 307+00 to 310+36 - B-018-4-13, B-018-5-13, B-019-2-13 and B-020-5-13 - 12.9 ft. Wall Height



MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	12.9	ft
MSE Wall Width (Reinforcement Length), (B) =	9.0	ft
MSE Wall Length, (L) =	336	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}) =	30	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(S_u)_{BS}]$ =	2000	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	33.0	ft

LRFD Load Factors

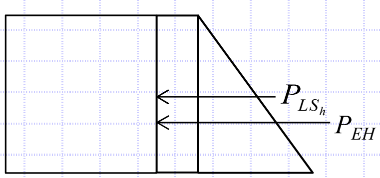
	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

(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:

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



$$P_{EH} = ½\gamma_{RS} H^2 K_a \gamma_{EH} = \text{½(120 pcf)(12.9 ft)²(0.297)(1.5)} = 4.45 \text{ kip/ft}$$

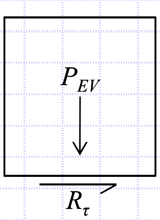
$$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS} = \text{(250 psf)(12.9 ft)(0.297)(1.75)} = 1.68 \text{ kip/ft}$$

$$P_H = \text{4.45 kip/ft + 1.68 kip/ft} = 6.13 \text{ kip/ft}$$

Check Sliding Resistance - Drained Condition

Nominal Sliding Resistance:

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



$$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV} = \text{(120 pcf)(12.9 ft)(9.0 ft)(1.00)} = 13.93 \text{ kip/ft}$$

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

$$\tan\delta = \tan(30) \le \tan(34) \longrightarrow 0.58 \le 0.67 \longrightarrow \tan\delta = 0.58$$

$$R_\tau = \text{(13.93 kip/ft)(0.58)} = 8.08 \text{ kip/ft}$$

Verify Sliding Force Less Than Factored Sliding Resistance - Drained Condition

$$P_H \le R_\tau \cdot \phi_\tau \longrightarrow \text{6.13 kip/ft} \le \text{(8.08 kip/ft)(1.0) = 8.08 kip/ft} \longrightarrow \text{6.13 kip/ft} \le \text{8.08 kip/ft}$$

OK

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

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MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	12.9	ft
MSE Wall Width (Reinforcement Length), (B) =	9.0	ft
MSE Wall Length, (L) =	336	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}) =	30	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	2000	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	33.0	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

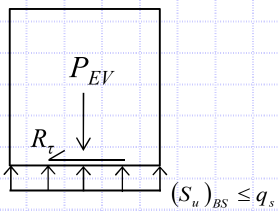
(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

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

Check Sliding Resistance - Undrained Condition

Nominal Sliding Resisting:

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



$$(S_u)_{BS} = \text{2.00 ksf}$$

$$q_s = \sigma_v / 2 = \text{(1.55 ksf) / 2} = \text{0.78 ksf}$$

$$\sigma_v = P_{EV} / B = \text{(13.93 kip/ft) / (9 ft)} = \text{1.55 ksf}$$

$$R_\tau = \text{(2.00 ksf} \le \text{0.78 ksf)(9.0 ft)} = \text{7.02 kip/ft}$$

Verify Sliding Force Less Than Factored Sliding Resistance - Undrained Condition

$P_H \le R_\tau \cdot \phi_\tau$ $\longrightarrow$ 6.13 kip/ft ≤ (7.02 kip/ft)(1.0) = 7.02 kip/ft $\longrightarrow$ 6.13 kip/ft ≤ 7.02 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) =	12.9	ft
MSE Wall Width (Reinforcement Length), (B) =	9.0	ft
MSE Wall Length, (L) =	336	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:

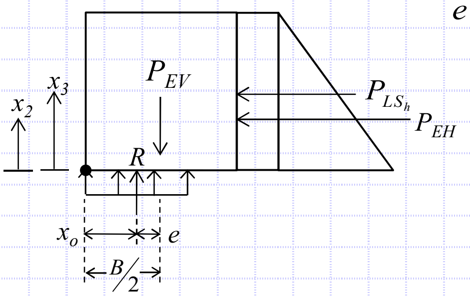
Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	30	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	2000	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	33.0	ft

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



$$e = B/2 - x_o$$

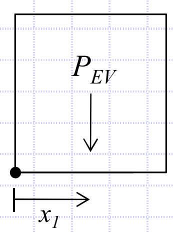
$$x_o = \frac{M_{EV} - M_H}{P_{EV}}$$ = (62.69 kip·ft/ft - 29.97 kip·ft/ft) / (13.93 kip/ft) = 2.35 ft

M_{EV} = 62.69 kip·ft/ft
M_H = 29.97 kip·ft/ft
P_{EV} = 13.93 kip/ft
Defined below

$e =$ (9 ft)/2 - 2.35 ft = 2.15 ft

Resisting Moment, M_{EV}:

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



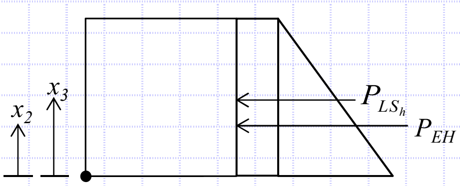
$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV}$ = (120 pcf)(12.9 ft)(9.0 ft)(1.00) = 13.93 kip/ft

$x_1 = B/2$ = (9.0 ft) / 2 = 4.50 ft

$M_{EV} =$ (13.93 kip/ft)(4.50 ft) = 62.69 kip·ft/ft

Overturning Moment, M_H:

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



$P_{EH} = ½\gamma_{RS} H^2 K_a \gamma_{EH}$ = ½(120 pcf)(12.9 ft)²(0.297)(1.5) = 4.45 kip/ft

$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS}$ = (250 psf)(12.9 ft)(0.297)(1.75) = 1.68 kip/ft

$x_2 = H/3$ = (12.9 ft) / 3 = 4.30 ft

$x_3 = H/2$ = (12.9 ft) / 2 = 6.45 ft

$M_H =$ (4.45 kip/ft)(4.3 ft) + (1.68 kip/ft)(6.45 ft) = 29.97 kip·ft/ft

Check Eccentricity

$e < e_{max} \longrightarrow$ 2.15 ft < 3.00 ft **OK**

Limiting Eccentricity: $e_{max} = B/3 \longrightarrow e_{max} =$ (9.0 ft) / 3 = 3.00 ft

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JOB	FRA-70-13.10	NO.	W-13-072
SHEET NO.	4	OF	6
CALCULATED BY	BRT	DATE	12/27/2020
CHECKED BY	JPS	DATE	3/5/2021

Retaining Wall E3 - Sta. 307+00 to 310+36

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	12.9	ft
MSE Wall Width (Reinforcement Length), (B) =	9.0	ft
MSE Wall Length, (L) =	336	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:

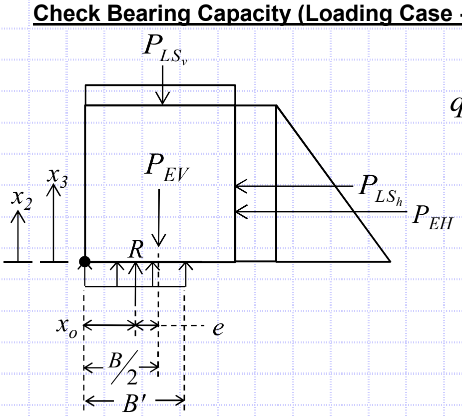
Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	30	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	2000	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	33.0	ft

LRFD Load Factors

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

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

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



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

$B' = B - 2e$ = 9.0 ft - 2(1.32 ft) = 6.36 ft

$e = B/2 - x_o$ = (9.0 ft) / 2 - 3.18 ft = 1.32 ft

$x_o = \dfrac{M_V - M_H}{P_V}$ = (102.36 kip·ft/ft - 29.94 kip·ft/ft) / 22.75 kip/ft = 3.18 ft

$q_{eq} =$ (22.75 kip/ft) / (6.36 ft) = 3.58 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)(12.9 ft)(9.0 ft)(1.35)](4.5 ft) + [(250 psf)(9.0 ft)(1.75)](4.5 ft) = 102.36 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)(12.9 ft)²(0.297)(1.5)](4.3 ft) + [(250 psf)(12.9 ft)(0.297)(1.75)](6.45 ft) = 29.94 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)(12.9 ft)(9.0 ft)(1.35) + (250 psf)(9.0 ft)(1.75) = 22.75 kip/ft

Check Bearing Resistance - Drained Condition

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

$N_{cm} = N_c s_c i_c$ = 30.50

N_c = 30.14
s_c = 1+(6.36 ft/336 ft)(18.4/30.14)
= 1.012
i_c = 1.000 (Assumed)

$N_{qm} = N_q s_q d_q i_q$ = 21.62

N_q = 18.40
s_q = 1.011
d_q = 1+2tan(30°)[1-sin(30°)]²tan⁻¹(4.0 ft/6.36 ft)
1.162
i_q = 1.000 (Assumed)
C_{wq} = 33.0 ft > 4.0 ft = 1.000

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 22.22

N_γ = 22.4
s_γ = 0.992
i_γ = 1.000 (Assumed)
$C_{w\gamma}$ = 33.0 ft > 1.5(6.36 ft) + 33.0 ft = 1.000

$q_n =$ (0 psf)(30.502) + (120 pcf)(4.0 ft)(21.616)(1.000) + ½(120 pcf)(6.4 ft)(22.221)(1.000) = 18.86 ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

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

$q_{eq} \le q_n \cdot \phi_b$ $\longrightarrow$ 3.58 ksf ≤ (18.86 ksf)(0.65) = 12.26 ksf $\longrightarrow$ 3.58 ksf ≤ 12.26 ksf **OK**

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MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	12.9	ft
MSE Wall Width (Reinforcement Length), (B) =	9.0	ft
MSE Wall Length, (L) =	336	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}) =	30	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	2000	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	33.0	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: $q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \frac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$

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

N_c = 5.140

s_c = 1+(6.36 ft/[(5)(336 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/6.36 ft)

1.000

i_q = 1.000 (Assumed)

C_{wq} = 33.0 ft > 4.0 ft = 1.000

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

N_γ = 0.000

s_γ = 1.000

i_γ = 1.000 (Assumed)

$C_{w\gamma}$ = 33.0 ft > 1.5(6.36 ft) + 33.0 ft = 1.000

q_n = (2000 psf)(5.160) + (120 pcf)(4.0 ft)(1.000)(1.000) + ½(120 pcf)(6.4 ft)(0.000)(1.000) = 10.80 ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

$q_{eq} \leq q_n \cdot \phi_b \longrightarrow$ 3.58 ksf ≤ (10.80 ksf)(0.65) = 7.02 ksf $\longrightarrow$ 3.58 ksf ≤ 7.02 ksf **OK**

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

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	12.9	ft
MSE Wall Width (Reinforcement Length), (B) =	9.0	ft
MSE Wall Length, (L) =	336	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:

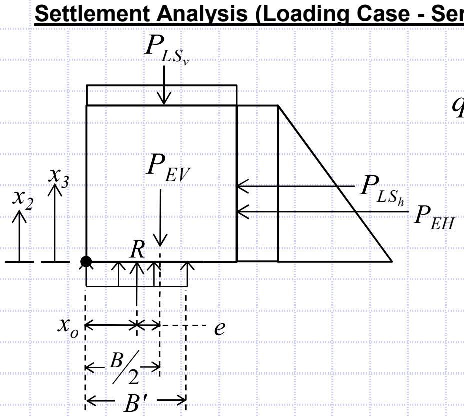
Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	30	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	2000	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	33.0	ft

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



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

$$B' = B - 2e \quad = \quad 9.0\text{ ft} - 2(1.17\text{ ft}) \quad = \quad 6.66 \quad \text{ft}$$

$$e = B/2 - x_o \quad = \quad (9.0\text{ ft}) / 2 - 3.33\text{ ft} \quad = \quad 1.17 \quad \text{ft}$$

$$x_o = \frac{M_V - M_H}{P_V} \quad = \quad (72.82\text{ kip·ft/ft} - 18.93\text{ kip·ft/ft}) / 16.18\text{ kip/ft} \quad = \quad 3.33 \quad \text{ft}$$

$$q_{eq} = \quad (16.18\text{ kip/ft}) / (6.66\text{ ft}) \quad = \quad 2.43 \quad \text{ksf}$$

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

$$M_V = \quad [(120\text{ pcf})(12.9\text{ ft})(9.0\text{ ft})(1.00)](4.5\text{ ft}) + [(250\text{ psf})(9.0\text{ ft})(1.00)](4.5\text{ ft}) \quad = \quad 72.82 \quad \text{kip·ft/ft}$$

$$M_H = P_{EH}(x_2) + P_{LS_h}(x_3) = (½\gamma_{RS} H^2 K_a \gamma_{EH})(x_2) + (\sigma_{LS} H K_a \gamma_{LS})(x_3)$$

$$M_H = \quad [½(120\text{ pcf})(12.9\text{ ft})^2(0.297)(1.00)](4.3\text{ ft}) + [(250\text{ psf})(12.9\text{ ft})(0.297)(1.00)](6.45\text{ ft}) \quad = \quad 18.93 \quad \text{kip·ft/ft}$$

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

$$P_V = \quad (120\text{ pcf})(12.9\text{ ft})(9.0\text{ ft})(1.00) + (250\text{ psf})(9.0\text{ ft})(1.00) \quad = \quad 16.18 \quad \text{kip/ft}$$

Settlement, Time Rate of Consolidation and Differential Settlement:

Boring	Total Settlement at Center of Reinforced Soil Mass	Total Settlement at Wall Facing	Time for 100% Consolidation	Distance Between Borings Along Wall Facing	Differential Settlement Along Wall Facing
B-018-5-13	1.067 in	0.880 in	75 days		
B-019-2-13	1.820 in	1.370 in	85 days	170 ft	1 in / 350 ft
B-020-5-13	1.860 in	1.222 in	115 days	166 ft	1 in / 1,120 ft

W-13-072 - FRA-70-13.10 - Retaining Wall E3
MSE Wall Settlement

Calculated By: BRT Date: 12/27/2020
Checked By: JPS Date: 3/5/2021

Boring B-018-5-13

H=	8.3	ft	Total wall height
B'=	6.8	ft	Effective footing width due to eccentricity
D_w =	35.5	ft	Depth below bottom of footing
q_e =	1,470	psf	Equivalent bearing pressure at bottom of wall
γ_{emb} =	120	pcf	Unit weight of embankment fill below wall
q_{emb} =	840	psf	Overburden pressure from embankment fill below wall (7.0-foot fill height)
q_{tot} =	2,310	psf	Total pressure from embankment fill and retaining 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-6b	C	0.0	3.5	3.5	1.8	120	420	210	210	3,210	36	0.234	0.018	0.553				0.26	0.956	2,209	2,419	0.042	0.504	0.497	1,147	1,357	0.032	0.385
2	A-2-6	G	3.5	7.0	3.5	5.3	130	875	648	648	3,648					29	40	131	0.77	0.656	1,516	2,164	0.014	0.169	0.445	1,027	1,675	0.011	0.133
	A-2-6	G	7.0	10.5	3.5	8.8	130	1,330	1,103	1,103	4,103					29	35	113	1.29	0.451	1,041	2,144	0.009	0.107	0.365	842	1,945	0.008	0.091
3	A-6b	C	10.5	13.5	3.0	12.0	125	1,705	1,518	1,518	4,518	36	0.234	0.018	0.553				1.76	0.343	792	2,309	0.006	0.074	0.301	694	2,212	0.006	0.067
	A-6b	C	13.5	17.0	3.5	15.3	125	2,143	1,924	1,924	4,924	36	0.234	0.018	0.553				2.24	0.275	635	2,559	0.005	0.059	0.252	582	2,506	0.005	0.054
4	A-6b	C	17.0	20.0	3.0	18.5	120	2,503	2,323	2,323	5,323	38	0.252	0.019	0.569				2.72	0.229	529	2,851	0.003	0.039	0.215	497	2,820	0.003	0.037
	A-6b	C	20.0	23.0	3.0	21.5	120	2,863	2,683	2,683	5,683	38	0.252	0.019	0.569				3.16	0.198	458	3,140	0.002	0.030	0.189	437	3,119	0.002	0.028
	A-6b	C	23.0	26.0	3.0	24.5	120	3,223	3,043	3,043	6,043	38	0.252	0.019	0.569				3.60	0.174	403	3,446	0.002	0.023	0.168	389	3,431	0.002	0.023
5	A-2-4	G	26.0	31.0	5.0	28.5	135	3,898	3,560	3,560	6,560					58	47	157	4.19	0.150	348	3,908	0.001	0.015	0.146	338	3,898	0.001	0.015
	A-2-4	G	31.0	36.0	5.0	33.5	135	4,573	4,235	4,235	7,235					58	44	144	4.93	0.128	296	4,531	0.001	0.012	0.126	291	4,526	0.001	0.012
	A-2-4	G	36.0	41.0	5.0	38.5	135	5,248	4,910	4,723	7,723					58	41	136	5.66	0.112	258	4,981	0.001	0.010	0.110	254	4,977	0.001	0.010
6	A-2-4	G	41.0	46.0	5.0	43.5	125	5,873	5,560	5,061	8,061					18	12	60	6.40	0.099	229	5,290	0.002	0.019	0.098	226	5,287	0.002	0.019
7	A-1-b	G	46.0	49.0	3.0	47.5	135	6,278	6,075	5,326	8,326					43	29	96	6.99	0.091	210	5,536	0.001	0.006	0.090	208	5,534	0.001	0.006
																						Total Settlement:		1.067 in			Total Settlement:		0.880 in

1. $\sigma_p' = \sigma_{vo}'+\sigma_m$; Estimate σ_m of 3,000 psf for moderately overconsolidated soil deposit; Ref. Table 11.2, Coduto 2003
2. C_c = 0.009(LL-10); Ref. Table 6-9, FHWA GEC 5
3. C_r = 0.075(C_c) for the existing embankment fill and 0.10(C_c) for the natural soil; Ref. Section 5.4.2.5 of FHWA GEC 5
4. e_o = (C_c/1.15)+0.35; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_n N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \leq \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (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 E3
MSE Wall Settlement

Calculated By: BRT — Date: 12/27/2020
Checked By: JPS — Date: 03/05/2021

Boring B-018-5-13

H=	8.3	ft	Total wall height
B'=	6.8	ft	Effective footing width due to eccentricity
D_w =	35.5	ft	Depth below bottom of footing
q_e =	1,470	psf	Equivalent bearing pressure at bottom of wall
γ_{emb} =	120	pcf	Unit weight of embankment fill below wall
q_{emb} =	840	psf	Overburden pressure from embankment fill below wall (7.0-foot fill height)
q_{tot} =	2,310	psf	Total pressure from embankment fill and retaining wall

	A-6b (Emb.)		
c_v =	400	ft^2/yr	Coefficient of consolidation
t =	75	days	Time following completion of construction
H_{dr} =	11	ft	Length of longest drainage path considered
T_v =	0.679		Time factor
U =	85	%	Degree of consolidation

$(S_c)_t$ = 0.791 in Settlement complete at 90% of primary consolidation

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)	Total Settlement at Facing of Wall: S_c (9,10) (ft)	S_c (in)	Layer Settlement (in)	Settlement Complete at 90% of Primary Consolidation: $(S_c)_t$ (11) (in)	Layer Settlement (in)
1	A-6b	C	0.0	3.5	3.5	1.8	120	420	210	210	3,210	36	0.234	0.018	0.553				0.26	0.497	1,147	1,357	0.032	0.385	0.385	0.327	0.327
2	A-2-6	G	3.5	7.0	3.5	5.3	130	875	648	648	3,648					29	40	131	0.77	0.445	1,027	1,675	0.011	0.133	0.224	0.133	0.224
	A-2-6	G	7.0	10.5	3.5	8.8	130	1,330	1,103	1,103	4,103					29	35	113	1.29	0.365	842	1,945	0.008	0.091		0.091	
3	A-6b	C	10.5	13.5	3.0	12.0	125	1,705	1,518	1,518	4,518	36	0.234	0.018	0.553				1.76	0.301	694	2,212	0.006	0.067	0.121	0.057	0.103
	A-6b	C	13.5	17.0	3.5	15.3	125	2,143	1,924	1,924	4,924	36	0.234	0.018	0.553				2.24	0.252	582	2,506	0.005	0.054		0.046	
4	A-6b	C	17.0	20.0	3.0	18.5	120	2,503	2,323	2,323	5,323	38	0.252	0.019	0.569				2.72	0.215	497	2,820	0.003	0.037	0.088	0.031	0.074
	A-6b	C	20.0	23.0	3.0	21.5	120	2,863	2,683	2,683	5,683	38	0.252	0.019	0.569				3.16	0.189	437	3,119	0.002	0.028		0.024	
	A-6b	C	23.0	26.0	3.0	24.5	120	3,223	3,043	3,043	6,043	38	0.252	0.019	0.569				3.60	0.168	389	3,431	0.002	0.023		0.019	
5	A-2-4	G	26.0	31.0	5.0	28.5	135	3,898	3,560	3,560	6,560					58	47	157	4.19	0.146	338	3,898	0.001	0.015	0.037	0.015	0.037
	A-2-4	G	31.0	36.0	5.0	33.5	135	4,573	4,235	4,235	7,235					58	44	144	4.93	0.126	291	4,526	0.001	0.012		0.012	
	A-2-4	G	36.0	41.0	5.0	38.5	135	5,248	4,910	4,723	7,723					58	41	136	5.66	0.110	254	4,977	0.001	0.010		0.010	
6	A-2-4	G	41.0	46.0	5.0	43.5	125	5,873	5,560	5,061	8,061					18	12	60	6.40	0.098	226	5,287	0.002	0.019	0.019	0.019	0.019
7	A-1-b	G	46.0	49.0	3.0	47.5	135	6,278	6,075	5,326	8,326					43	29	96	6.99	0.090	208	5,534	0.001	0.006	0.006	0.006	0.006

Settlement Remaining After Hold Period: 0.089 in

1. $\sigma_p' = \sigma_{vo}' + \sigma_m$; Estimate σ_m of 3,000 psf for moderately overconsolidated soil deposit; Ref. Table 11.2, Coduto 2003
2. C_c = 0.009(LL-10); Ref. Table 6-9, FHWA GEC 5
3. C_r = 0.075(C_c) for the existing embankment fill and 0.10(C_c) for the natural soil; Ref. Section 5.4.2.5 of FHWA GEC 5
4. e_o = (C_c/1.15)+0.35; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_N N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \le 2.0$ 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 $\sigma_p' \le \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ 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 E3
MSE Wall Settlement

Calculated By: BRT   Date: 12/27/2020
Checked By: JPS   Date: 3/5/2021

Boring B-019-2-13

H=	12.7	ft	Total wall height
B'=	6.6	ft	Effective footing width due to eccentricity
D_w =	33.0	ft	Depth below bottom of footing
q_e =	2,390	psf	Equivalent bearing pressure at bottom of wall
γ_{emb} =	120	pcf	Unit weight of embankment fill below wall
q_{emb} =	1,044	psf	Overburden pressure from embankment fill below wall (8.7-foot fill height)
q_{tot} =	3,434	psf	Total pressure from embankment fill and retaining 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-6b	C	0.0	2.5	2.5	1.3	125	313	156	156	3,156	38	0.252	0.019	0.569				0.19	0.980	3,367	3,523	0.058	0.702	0.499	1,712	1,869	0.032	0.389
	A-6b	C	2.5	5.0	2.5	3.8	125	625	469	469	3,469	38	0.252	0.019	0.569				0.57	0.775	2,662	3,131	0.025	0.298	0.472	1,622	2,090	0.020	0.235
	A-6b	C	5.0	7.5	2.5	6.3	125	938	781	781	3,781	38	0.252	0.019	0.569				0.95	0.572	1,965	2,746	0.016	0.197	0.418	1,434	2,215	0.014	0.164
	A-6b	C	7.5	10.5	3.0	9.0	125	1,313	1,125	1,125	4,125	38	0.252	0.019	0.569				1.36	0.430	1,475	2,600	0.013	0.158	0.353	1,213	2,338	0.011	0.138
2	A-6a	C	10.5	13.0	2.5	11.8	125	1,625	1,469	1,469	4,469	30	0.180	0.014	0.507				1.78	0.340	1,168	2,636	0.006	0.068	0.299	1,026	2,495	0.005	0.062
3	A-6b	C	13.0	17.5	4.5	15.3	125	2,188	1,906	1,906	4,906	37	0.243	0.018	0.561				2.31	0.267	918	2,824	0.009	0.108	0.246	845	2,751	0.008	0.100
	A-6b	C	17.5	22.0	4.5	19.8	125	2,750	2,469	2,469	5,469	37	0.243	0.018	0.561				2.99	0.209	717	3,186	0.006	0.070	0.198	681	3,150	0.006	0.067
4	A-2-4	G	22.0	27.0	5.0	24.5	130	3,400	3,075	3,075	6,075					28	24	83	3.71	0.169	582	3,657	0.005	0.054	0.164	562	3,637	0.004	0.053
5	A-4a	G	27.0	32.0	5.0	29.5	125	4,025	3,713	3,713	6,713					30	24	46	4.47	0.141	485	4,198	0.006	0.070	0.138	473	4,186	0.006	0.069
6	A-6a	C	32.0	37.0	5.0	34.5	120	4,625	4,325	4,231	7,231	30	0.180	0.018	0.507				5.23	0.121	416	4,647	0.002	0.029	0.119	408	4,640	0.002	0.029
7	A-1-b	G	37.0	42.0	5.0	39.5	125	5,250	4,938	4,532	7,532					22	16	66	5.98	0.106	364	4,895	0.003	0.030	0.104	359	4,891	0.002	0.030
8	A-2-4	G	42.0	47.0	5.0	44.5	125	5,875	5,563	4,845	7,845					21	15	64	6.74	0.094	323	5,168	0.002	0.026	0.093	320	5,164	0.002	0.026
9	A-1-b	G	47.0	50.0	3.0	48.5	130	6,265	6,070	5,103	8,103					39	27	90	7.35	0.086	297	5,399	0.001	0.010	0.086	294	5,397	0.001	0.010
																					Total Settlement:			1.820 in		Total Settlement:			1.370 in

1. $\sigma_p' = \sigma_{vo}' + \sigma_m$; Estimate σ_m of 3,000 psf for moderately overconsolidated soil deposit; Ref. Table 11.2, Coduto 2003
2. C_c = 0.009(LL-10); Ref. Table 6-9, FHWA GEC 5
3. $C_r = 0.075(C_c)$ for the existing embankment fill and $0.10(C_c)$ for the natural soil; Ref. Section 5.4.2.5 of FHWA GEC 5
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_N N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \leq \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (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 E3

MSE Wall Settlement

Calculated By: BRT Date: 12/27/2020

Checked By: JPS Date: 03/05/2021

Boring B-020-5-13

H=	12.7	ft	Total wall height
B'=	6.6	ft	Effective footing width due to eccentricity
D_w =	33.0	ft	Depth below bottom of footing
q_e =	2,390	psf	Equivalent bearing pressure at bottom of wall
γ_{emb} =	120	pcf	Unit weight of embankment fill below wall
q_{emb} =	1,044	psf	Overburden pressure from embankment fill below wall (8.7-foot fill height)
q_{tot} =	3,434	psf	Total pressure from embankment fill and retaining wall

	A-6b (Emb.)	A-6a (Emb.)	A-6a (Nat.)		
c_v =	400	700	600	ft^2/yr	Coefficient of consolidation
t =	85	85	85	days	Time following completion of construction
H_{dr} =	11	11	2.5	ft	Length of longest drainage path considered
T_v =	0.770	1.347	22.356		Time factor
U =	88	97	100	%	Degree of consolidation

$(S_c)_t$ = 1.230 in Settlement complete at 90% of primary consolidation

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)	Total Settlement at Facing of Wall: S_c (9,10) (ft)	S_c (in)	Layer Settlement (in)	Settlement Complete at 90% of Primary Consolidation: $(S_c)_t$ (11) (in)	Layer Settlement (in)
1	A-6b	C	0.0	2.5	2.5	1.3	125	313	156	156	3,156	38	0.252	0.019	0.569				0.19	0.499	1,712	1,869	0.032	0.389	0.925	0.343	0.814
	A-6b	C	2.5	5.0	2.5	3.8	125	625	469	469	3,469	38	0.252	0.019	0.569				0.57	0.472	1,622	2,090	0.020	0.235		0.206	
	A-6b	C	5.0	7.5	2.5	6.3	125	938	781	781	3,781	38	0.252	0.019	0.569				0.95	0.418	1,434	2,215	0.014	0.164		0.144	
	A-6b	C	7.5	10.5	3.0	9.0	125	1,313	1,125	1,125	4,125	38	0.252	0.019	0.569				1.36	0.353	1,213	2,338	0.011	0.138		0.121	
2	A-6a	C	10.5	13.0	2.5	11.8	125	1,625	1,469	1,469	4,469	30	0.180	0.014	0.507				1.78	0.299	1,026	2,495	0.005	0.062	0.062	0.060	0.060
3	A-6b	C	13.0	17.5	4.5	15.3	125	2,188	1,906	1,906	4,906	37	0.243	0.018	0.561				2.31	0.246	845	2,751	0.008	0.100	0.167	0.088	0.147
	A-6b	C	17.5	22.0	4.5	19.8	125	2,750	2,469	2,469	5,469	37	0.243	0.018	0.561				2.99	0.198	681	3,150	0.006	0.067		0.059	
4	A-2-4	G	22.0	27.0	5.0	24.5	130	3,400	3,075	3,075	6,075					28	24	83	3.71	0.164	562	3,637	0.004	0.053	0.053	0.046	0.046
5	A-4a	G	27.0	32.0	5.0	29.5	125	4,025	3,713	3,713	6,713					30	24	46	4.47	0.138	473	4,186	0.006	0.069	0.069	0.069	0.069
6	A-6a	C	32.0	37.0	5.0	34.5	120	4,625	4,325	4,231	7,231	30	0.180	0.018	0.507				5.23	0.119	408	4,640	0.002	0.029	0.029	0.029	0.029
7	A-1-b	G	37.0	42.0	5.0	39.5	125	5,250	4,938	4,532	7,532					22	16	66	5.98	0.104	359	4,891	0.002	0.030	0.030	0.030	0.030
8	A-2-4	G	42.0	47.0	5.0	44.5	125	5,875	5,563	4,845	7,845					21	15	64	6.74	0.093	320	5,164	0.002	0.026	0.026	0.026	0.026
9	A-1-b	G	47.0	50.0	3.0	48.5	130	6,265	6,070	5,103	8,103					39	27	90	7.35	0.086	294	5,397	0.001	0.010	0.010	0.010	0.010

Settlement Remaining After Hold Period: 0.139 in

1. $\sigma_p' = \sigma_{vo}' + \sigma_m$; Estimate σ_m of 3,000 psf for moderately overconsolidated soil deposit; Ref. Table 11.2, Coduto 2003
2. C_c = 0.009(LL-10); Ref. Table 6-9, FHWA GEC 5
3. C_r = 0.075(C_c) for the existing embankment fill and 0.10(C_c) for the natural soil; Ref. Section 5.4.2.5 of FHWA GEC 5
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_N N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \leq \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (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 E3
MSE Wall Settlement

Calculated By: BRT Date: 12/27/2020
Checked By: JPS Date: 3/5/2021

Boring B-020-5-13

H=	12.9	ft	Total wall height
B'=	6.7	ft	Effective footing width due to eccentricity
D_w =	33.0	ft	Depth below bottom of footing
q_e =	2,430	psf	Equivalent bearing pressure at bottom of wall
γ_{emb} =	120	pcf	Unit weight of embankment fill below wall
q_{emb} =	1,464	psf	Overburden pressure from embankment fill below wall (12.2-foot fill height)
q_{tot} =	3,894	psf	Total pressure from embankment fill and retaining 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	Total Settlement at Center of Reinforced Soil Mass: I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	Total Settlement at Facing of Wall: 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	5.0	5.0	2.5	125	625	313	313	3,313	32	0.198	0.015	0.522				0.37	0.897	3,493	3,805	0.089	1.070	0.491	1,910	2,223	0.042	0.499
	A-6a	C	5.0	10.0	5.0	7.5	125	1,250	938	938	3,938	32	0.198	0.015	0.522				1.12	0.504	1,964	2,902	0.024	0.287	0.390	1,520	2,457	0.020	0.245
	A-6a	C	10.0	15.0	5.0	12.5	125	1,875	1,563	1,563	4,563	32	0.198	0.015	0.522				1.87	0.326	1,269	2,831	0.013	0.151	0.289	1,126	2,688	0.011	0.138
	A-6a	C	15.0	20.0	5.0	17.5	125	2,500	2,188	2,188	5,188	32	0.198	0.015	0.522				2.61	0.238	927	3,114	0.007	0.090	0.223	867	3,055	0.007	0.085
	A-6a	C	20.0	25.5	5.5	22.8	125	3,188	2,844	2,844	5,844	32	0.198	0.015	0.522				3.40	0.185	720	3,563	0.005	0.063	0.177	691	3,535	0.005	0.061
2	A-6b	C	25.5	33.0	7.5	29.3	125	4,125	3,656	3,656	6,656	33	0.207	0.021	0.530				4.37	0.145	563	4,219	0.006	0.076	0.141	549	4,205	0.006	0.074
3	A-2-4	G	33.0	38.0	5.0	35.5	125	4,750	4,438	4,282	7,282					21	16	66	5.30	0.119	465	4,747	0.003	0.041	0.117	457	4,739	0.003	0.040
4	A-1-b	G	38.0	42.0	4.0	40.0	135	5,290	5,020	4,583	7,583					62	45	149	5.97	0.106	413	4,997	0.001	0.012	0.105	408	4,991	0.001	0.012
	A-1-b	G	42.0	47.0	5.0	44.5	135	5,965	5,628	4,910	7,910					62	43	143	6.64	0.095	372	5,282	0.001	0.013	0.094	368	5,278	0.001	0.013
	A-1-b	G	47.0	52.0	5.0	49.5	135	6,640	6,303	5,273	8,273					62	42	138	7.39	0.086	335	5,607	0.001	0.012	0.085	332	5,604	0.001	0.012
	A-1-b	G	52.0	57.0	5.0	54.5	135	7,315	6,978	5,636	8,636					62	41	133	8.13	0.078	304	5,940	0.001	0.010	0.077	302	5,938	0.001	0.010
5	A-4a	C	57.0	64.0	7.0	60.5	130	8,225	7,770	6,054	9,054	25	0.135	0.014	0.467				9.03	0.070	274	6,328	0.001	0.015	0.070	272	6,326	0.001	0.015
6	A-6a	C	64.0	69.5	5.5	66.8	130	8,940	8,583	6,477	9,477	25	0.135	0.014	0.467				9.96	0.064	248	6,725	0.001	0.010	0.063	247	6,724	0.001	0.010
	A-6a	C	69.5	75.5	6.0	72.5	130	9,720	9,330	6,865	9,865	25	0.135	0.014	0.467				10.82	0.059	229	7,094	0.001	0.009	0.059	228	7,093	0.001	0.009
																					Total Settlement:			1.860 in		Total Settlement:			1.222 in

1. $\sigma_p' = \sigma_{vo}'+\sigma_m$; Estimate σ_m of 3,000 psf for moderately overconsolidated soil deposit; Ref. Table 11.2, Coduto 2003
2. C_c = 0.009(LL-10); Ref. Table 6-9, FHWA GEC 5
3. C_r = 0.075(C_c) for the existing embankment fill and 0.10(C_c) for the natural soil; Ref. Section 5.4.2.5 of FHWA GEC 5
4. e_o = (C_c/1.15)+0.35; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_N N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \leq \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (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 E3
MSE Wall Settlement

Calculated By: BRT Date: 12/27/2020
Checked By: JPS Date: 03/05/2021

Boring B-020-5-13

H=	12.9	ft	Total wall height
B'=	6.7	ft	Effective footing width due to eccentricity
D_w =	33.0	ft	Depth below bottom of footing
q_e =	2,430	psf	Equivalent bearing pressure at bottom of wall
γ_{emb} =	120	pcf	Unit weight of embankment fill below wall
q_{emb} =	1,464	psf	Overburden pressure from embankment fill below wall (12.2-foot fill height)
q_{tot} =	3,894	psf	Total pressure from embankment fill and retaining wall

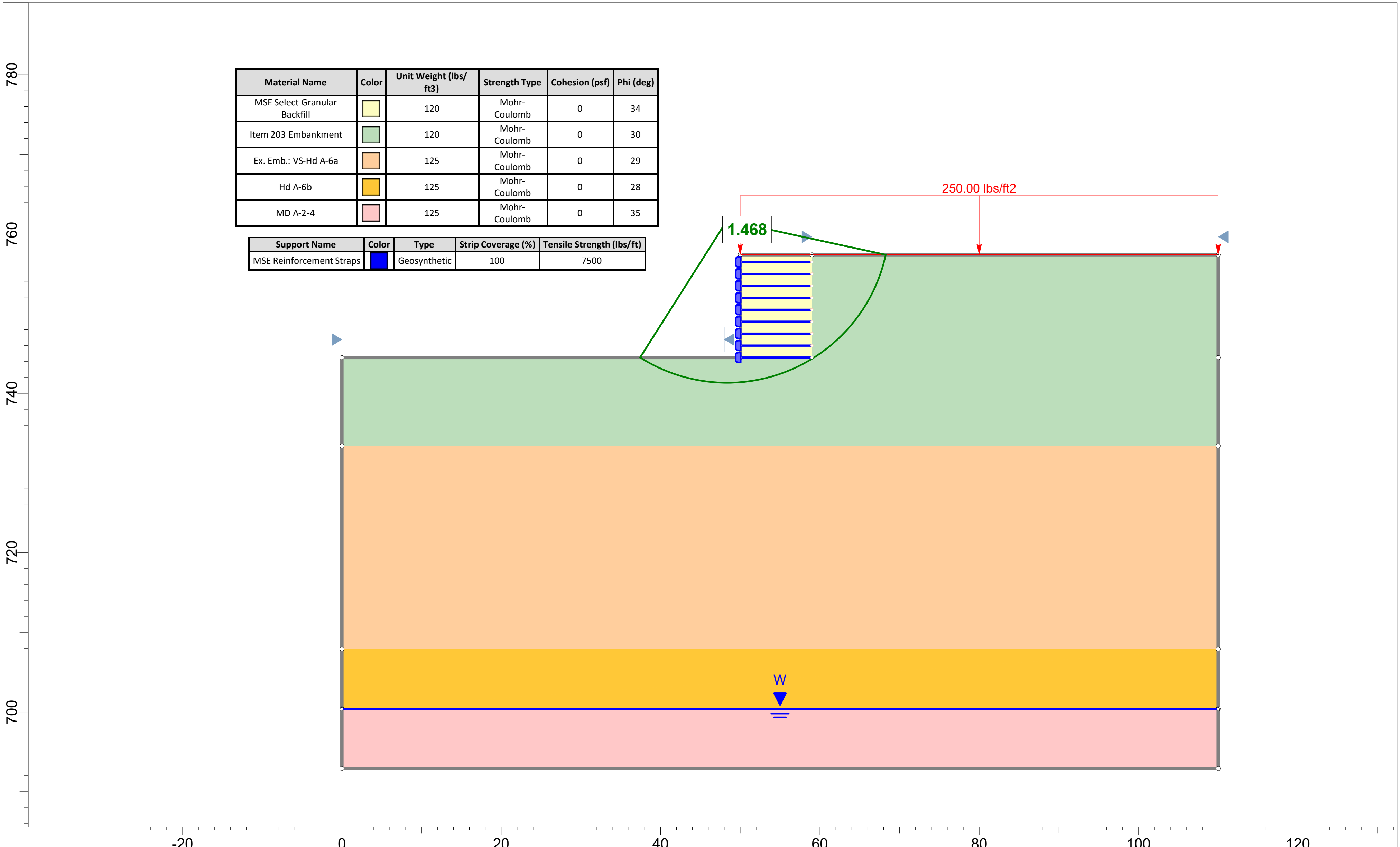
	A-6a (Emb.)	A-6b (Nat.)	A-6a (Nat.)	A-4a		
c_v =	700	300	600	1000	ft^2/yr	Coefficient of consolidation
t =	115	115	115	115	days	Time following completion of construction
H_{dr} =	16.5	7.5	18.5	7	ft	Length of longest drainage path considered
T_v =	0.810	1.680	0.552	6.430		Time factor
U =	89	99	79	100	%	Degree of consolidation

$(S_c)_t$ = 1.105 in Settlement complete at 90% of primary consolidation

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)	Total Settlement at Facing of Wall: S_c (9,10) (ft)	S_c (in)	Layer Settlement (in)	Settlement Complete at 90% of Primary Consolidation: $(S_c)_t$ (11) (in)	Layer Settlement (in)
	A-6a	C	0.0	5.0	5.0	2.5	125	625	313	313	3,313	32	0.198	0.015	0.522				0.37	0.491	1,910	2,223	0.042	0.499		0.444	
	A-6a	C	5.0	10.0	5.0	7.5	125	1,250	938	938	3,938	32	0.198	0.015	0.522				1.12	0.390	1,520	2,457	0.020	0.245		0.218	
1	A-6a	C	10.0	15.0	5.0	12.5	125	1,875	1,563	1,563	4,563	32	0.198	0.015	0.522				1.87	0.289	1,126	2,688	0.011	0.138	1.027	0.123	0.914
	A-6a	C	15.0	20.0	5.0	17.5	125	2,500	2,188	2,188	5,188	32	0.198	0.015	0.522				2.61	0.223	867	3,055	0.007	0.085		0.076	
	A-6a	C	20.0	25.5	5.5	22.8	125	3,188	2,844	2,844	5,844	32	0.198	0.015	0.522				3.40	0.177	691	3,535	0.005	0.061		0.054	
2	A-6b	C	25.5	33.0	7.5	29.3	125	4,125	3,656	3,656	6,656	33	0.207	0.021	0.530				4.37	0.141	549	4,205	0.006	0.074	0.074	0.073	0.073
3	A-2-4	G	33.0	38.0	5.0	35.5	125	4,750	4,438	4,282	7,282					21	16	66	5.30	0.117	457	4,739	0.003	0.040	0.040	0.040	0.040
	A-1-b	G	38.0	42.0	4.0	40.0	135	5,290	5,020	4,583	7,583					62	45	149	5.97	0.105	408	4,991	0.001	0.012		0.012	
4	A-1-b	G	42.0	47.0	5.0	44.5	135	5,965	5,628	4,910	7,910					62	43	143	6.64	0.094	368	5,278	0.001	0.013	0.047	0.013	0.047
	A-1-b	G	47.0	52.0	5.0	49.5	135	6,640	6,303	5,273	8,273					62	42	138	7.39	0.085	332	5,604	0.001	0.012		0.012	
	A-1-b	G	52.0	57.0	5.0	54.5	135	7,315	6,978	5,636	8,636					62	41	133	8.13	0.077	302	5,938	0.001	0.010		0.010	
5	A-4a	C	57.0	64.0	7.0	60.5	130	8,225	7,770	6,054	9,054	25	0.135	0.014	0.467				9.03	0.070	272	6,326	0.001	0.015	0.015	0.015	0.015
6	A-6a	C	64.0	69.5	5.5	66.8	130	8,940	8,583	6,477	9,477	25	0.135	0.014	0.467				9.96	0.063	247	6,724	0.001	0.010	0.019	0.008	0.015
	A-6a	C	69.5	75.5	6.0	72.5	130	9,720	9,330	6,865	9,865	25	0.135	0.014	0.467				10.82	0.059	228	7,093	0.001	0.009		0.007	

Settlement Remaining After Hold Period: 0.118 in

1. $\sigma_p' = \sigma_{vo}' + \sigma_m$; Estimate σ_m of 3,000 psf for moderately overconsolidated soil deposit; Ref. Table 11.2, Coduto 2003
2. $C_c = 0.009(LL-10)$; Ref. Table 6-9, FHWA GEC 5
3. $C_r = 0.075(C_c)$ for the existing embankment fill and $0.10(C_c)$ for the natural soil; Ref. Section 5.4.2.5 of FHWA GEC 5
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_N N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \leq \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (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 Embankment		120	Mohr-Coulomb	0	30
Ex. Emb.: VS-Hd A-6a		125	Mohr-Coulomb	0	29
Hd A-6b		125	Mohr-Coulomb	0	28
MD A-2-4		125	Mohr-Coulomb	0	35

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

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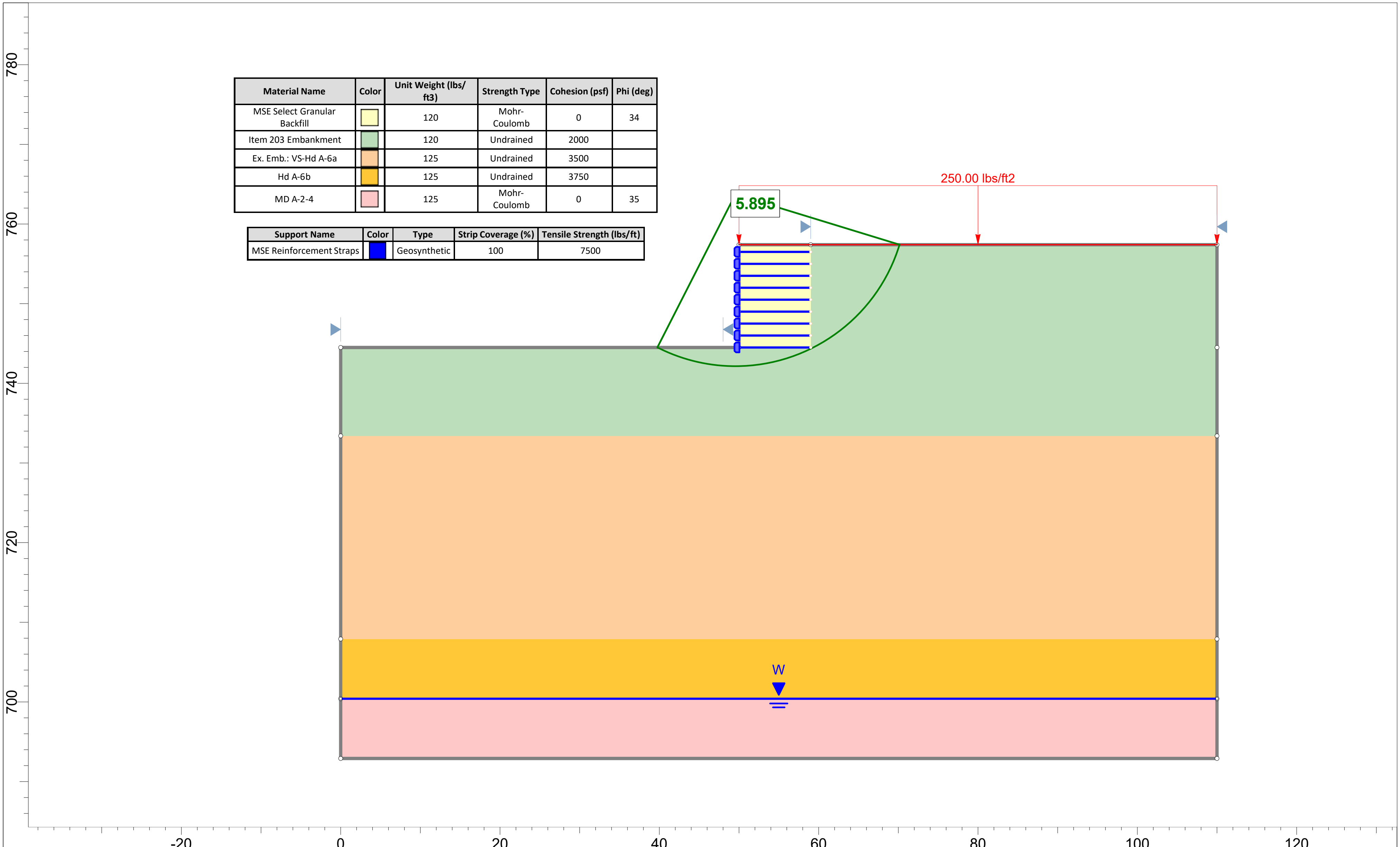
Project	Retaining Wall E3 - Sta. 307+00 to 310+36 - MSE Wall Global Stability
Analysis Description	12.9 ft Wall Height - Drained - Circular - Spencer
Drawn By	BRT
Scale	1:200
Company	Resource International, Inc.
Date	01/13/2021
File Name	Retaining Wall E3 - Global Stability.slim

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
MSE Select Granular Backfill		120	Mohr-Coulomb	0	34
Item 203 Embankment		120	Undrained	2000	
Ex. Emb.: VS-Hd A-6a		125	Undrained	3500	
Hd A-6b		125	Undrained	3750	
MD A-2-4		125	Mohr-Coulomb	0	35

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



Project	Retaining Wall E3 - Sta. 307+00 to 310+36 - MSE Wall Global Stability		
Analysis Description	12.9 ft Wall Height - Undrained - Circular - Spencer		
Drawn By BRT	*Scale* 1:200	*Company*	Resource International, Inc.
Date 01/13/2021		*File Name*	Retaining Wall E3 - Global Stability.slim

SLIDEINTERPRET 9.012