Resource International, Inc.

FRA-71-14.36 PHASE 6R
RETAINING WALL E10
PID NO. 105588
FRANKLIN COUNTY, OHIO

DRAFT STRUCTURE FOUNDATION EXPLORATION REPORT

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

Prepared By:
Resource International, Inc.
6350 Presidential Gateway
Columbus, Ohio 43231

Rii Project No. W-13-072

April 2020

Planning, Engineering, Construction Management, Technology
6350 Presidential Gateway, Columbus, Ohio 43231
P 614.823.4949 F 614.823.4990

RESOURCE INTERNATIONAL, INC.
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April 06, 2020

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

Re: Draft Structure Foundation Exploration Report
FRA-71-14.36 Phase 6R
Retaining Wall E10
PID No. 105588
Rii Project No. W-13-072

Mr. Antonios:

Resource International, Inc. (Rii) is pleased to submit this draft 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 E10 as part of the FRA-71-14.36 Phase 6R project in Columbus, Ohio.

We sincerely appreciate the opportunity to be of service to you on this project. If you have any questions regarding the structure foundation exploration or this report, please contact us.

Sincerely,

RESOURCE INTERNATIONAL, INC.

Hanumanth S. Kulkarni, Ph.D., P.E.
Project Engineer – Geotechnical Services

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

Enclosure: Draft Structure Foundation Exploration Report

6350 Presidential Gateway
Columbus, Ohio 43231
Phone: 614.823.4949
Fax: 614.823.4990

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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 E10. Based on plan information provided by the Rii design group and ms consultants, retaining Wall E10 will be located along the south side of I-71 southbound, between South Front Street and Short Street, and will provide the required grade separation between the ramp and I-70 westbound where the alignments diverge. The wall begins at Sta. 277+55.02 (BL I-71 SB) and extends west along the south side of I-71 southbound to Sta. 383+35.41 (BL I-71 SB Transition). The total wall length for Retaining Wall E10 is approximately 555 lineal feet. **Please note that the design of the MSE wall between Sta. 277+55.02 and 277+97.19 (BL I-71 SB) overlaps with Retaining Wall E7 where is crosses in front of the forward abutment with the FRA-71-1503L bridge structure, and as such will be governed by the recommendations in that bridge structure report, which is presented under separate covers.** The wall heights along the portion of the wall alignment that is considered for this exploration report will range from 4.5 feet at Sta. 383+35.41 (BL I-71 SB Transition) to 27.2 feet at Sta. 277+97.19 (BL I-71 SB). The Wall height along the portion of the wall not being considered for this report is 51.5 feet.

Exploration and Findings

Between January 20, and February 18, 2020, five (5) structural borings, designated as B-115-3-19 through B-115-7-19, were drilled at the locations shown on the boring plan provided in Appendix I of the full report. The borings were advanced to completion depths ranging from 20.0 to 45.0 feet below the existing ground surface along the existing mainline driving lanes of I-70 westbound.

Borings B-115-3-19, B-115-6-19 and B-115-7-19 were drilled through the existing mainline pavement along the southbound I-71 and encountered a composite pavement section consisting of 8.0 to 9.0 inches of asphalt overlying 10.0 inches of concrete. Borings B-115-4-19 and B-115-5-19, which were performed in the existing outside shoulder encountered full depth asphalt section with 8.0 to 9.0 inches of asphalt. Aggregate base was encountered in all of the borings with thicknesses varying from 6.0 to 12.0 inches.

Beneath the surface materials in borings B-115-3-19, B-115-4-19 and B-115-5-19, material identified as existing fill or possible fill was encountered extending to a depth of 32.0, 18.0 and 15.5 feet, respectively, below existing grade, which corresponds to an elevation of 701.9, 718.7 and 723.3 feet msl. The existing fill material was generally described as dark brown, brown, black and gray gravel with sand, gravel with sand, silt and clay, sandy silt, silt and clay and silty clay (ODOT A-1-b, A-2-6, A-4a, A-6a, A-6b). The fill contained debris consisting of cinders, red tile/brick fragments, broken rock fragments and possible concrete fragments in boring B-115-3-19 between the depths of 18.0 and 32.0 feet below the existing ground surface. Based on the site topography, the fill material is likely embankment fill that was placed during the original construction of the roadway.

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

Analyses and Recommendations

It is understood that a standard MSE wall type is being utilized between Sta. 380+20.00 to 383+35.41 (BL I-71 SB Transition), with a wall height ranging from 4.5 to 18.4 feet. A lightweight cellular concrete modified MSE wall system will be utilized between Sta. 379+50.82 to 380+20.00 (BL I-71 SB Transition), where the wall will span over two existing electrical duct banks, with a wall height ranging from 18.4 to 21.7 feet. A modified MSE wall system consisting of geofoam is being utilized between Sta. 277+97.19 (BL I-71 SB) to Sta. 379+50.82 (BL I-71 SB Transition), where the alignment spans over the influence zone of the Franklin Main, with a wall height ranging from 21.7 to 27.2 feet. It is understood that precast facing panels will be utilized, which will be supported on pedestal footings and anchored into the distribution slab at the top of the wall for the geofoam and cellular concrete modified MSE wall segments.

MSE Wall Recommendations

Based on the proposed plan and profile information, the proposed standard MSE wall section will have wall heights ranging from 4.5 to 18.4 feet, as measured from the top of the leveling pad to the top of the coping. The anticipated bearing materials along the proposed alignment of Retaining Wall E10 between Sta. 380+20 to 383+35.41 (BL I-71 SB Transition) consists of existing fill comprised of stiff to very stiff silty clay (ODOT A-6b) and loose to medium dense gravel with sand (ODOT A-1-b) which transitions to natural very stiff sandy silt (ODOT A-4a) along the east end of the wall. MSE wall foundations bearing on these soils may be proportioned for a factored bearing resistance as indicated in the following table. A geotechnical resistance factor of φ_b=0.65 was considered in calculating the factored bearing resistance at the strength limit state.

Retaining Wall E10 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]	
380+20.00	382+00.00	18.4	Level	12.9 (0.70H ≥ 8.0)	8.24	5.36	4.70
382+00.00	383+35.41	9.0	Level	8.0	12.76	8.25	2.35

1. *Stationing referenced to the baseline of I-71 SB Transition.*
2. *The required foundation width is expressed as a percentage of the wall height, H.*
3. *A geotechnical resistance factor of φ_b=0.65 was considered in calculating the factored bearing resistance at the strength limit state.*
4. *The strength limit equivalent bearing pressure is the uniformly distributed pressure asserted by the wall over an effective base width based on the eccentricity of the wall system at the strength limit state.*

Total settlements of up to 2.057 inches at the center of the reinforced soil mass and 1.550 inches at the facing of the wall are anticipated along the alignment of Retaining Wall E10 between Sta. 380+20.00 through 383+35.41 (BL I-71 SB Transition). Based on the results of the analysis, 90 percent of the total settlement at the facing of the wall is anticipated to occur within 9 to 80 days following the completion of construction of the wall.

Based on the results of the external and global stability analysis performed for the MSE wall, the recommended controlling strap length is 0.70 times the height of the MSE wall (measured from the top of the leveling pad to the proposed profile grade of the roadway) between Sta. 380+20.00 through 383+35.41 (BL I-71 SB Transition). All of the external and global stability calculations indicate that adequate resistance is available for support of the MSE wall.

Cellular Concrete Wall Recommendations

It is understood that a lightweight cellular concrete modified MSE system wall will be utilized between Sta. 702+50 and 703+00 (BL Wall E5), where the wall will span over two existing electrical duct banks, with a wall height ranging from 34.2 to 39.3 feet. Based on information provided by the Rii design team, two types of lightweight cellular concrete will be utilized in lieu of typical embankment fill and select granular fill, which is typically used for MSE wall applications. The wall facing will be connected to geosynthetic straps that are embedded into the cellular concrete and supported on a leveling pad, similar to traditional MSE walls.

Based on the plan information provided, it is understood that the cellular concrete fill will be placed the full height of the embankment and full width of both Ramp D6 and I-71 southbound. Provided that all backslopes cut into the existing I-70 embankment are graded no steeper than 2H:1V, external and global stability calculations will not be required for this section of Retaining Wall E10. However, if bearing resistance must be checked, then a factored bearing resistance of 4.49 ksf should be utilized for design at the strength limit state.

Total settlements of 0.926 to 1.097 inches at the center of the wall mass and 0.667 to 0.771 at the facing of the wall is anticipated along Retaining Wall E10 between Sta. 379+50.82 through 380+20.00 (BL I-71 SB Transition). Based on the results of the analysis, 90 percent of the total settlement at the facing of the wall is anticipated to occur during or immediately following construction of the wall or within 35 to 37 days following the completion of construction of the wall.

Geofoam Wall Recommendations

A modified MSE wall system consisting of geofoam blocking is being utilized between Sta. 277+97.19 (BL I-71 SB) through Sta. 379+50.82 (BL I-71 SB Transition), where the alignment spans over the influence zone of the Franklin Main, with a wall height ranging from 21.7 to 27.2 feet. Based on the information provided, the typical section for this segment will consist of an approximate 5.0-foot thick pavement section, including asphalt and/or concrete and aggregate base on top of a concrete distribution slab, overlying geofoam blocking (ASTM D6817, Type 19) to the bottom of wall elevation. Where the Franklin Main crosses the wall alignment, at approximately Sta. 378+80 (BL I-71 SB Transition), the wall height is approximately 24.8 feet and the bottom of wall (top of leveling pad) is at El. 730.5 feet msl. Considering a unit weight of 1.5 pcf for the geofoam blocking and 130 pcf for the pavement/aggregate base/distribution slab, the required depth embedment depth of the geofoam blocking below existing grade to provide zero net loading is 5.5 feet based on the maximum wall height of 27.2 feet (23.0 feet above existing grade).

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

1.0 INTRODUCTION

The overall purpose of this project is to provide detailed subsurface information and recommendations for the design and construction of the FRA-70/71-13.10/14.36 (Projects 6A/6R) project in Columbus, Ohio. The projects represent the central portion of FRA-70-8.93 (PID 77369) I-70/71 south innerbelt improvements project, which includes all improvements along I-70 westbound from the I-71/SR-315 interchange to Front Street and along I-71 southbound from I-70 to Greenlawn Avenue. The FRA-71-14.36 (Project 6R) phase will consist of all work associated with the reconfiguration and construction of I-71 southbound from downtown (Front Street) to Greenlawn Avenue, including Ramps C3, D6 and D7. This project includes the construction of two (2) new bridge structures, one (1) for I-71 southbound over Short Street, NS/CXS Railroad and the Scioto River (FRA-71-1503L) and one (1) for Ramp D7 over Short Street (FRA-70-1373B), as well as the construction of five (5) new retaining walls (Walls E4, E5, E7, W2 and W5) and one (1) temporary retaining wall (Wall E10) 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 E10 as shown on the vicinity map and boring plan presented in Appendix I. Retaining Wall E10 will be located along the south side of I-71 southbound, between South Front Street and Short Street, and will provide the required grade separation between the ramp and I-70 westbound where the alignments diverge. The wall begins at Sta. 277+55.02 (BL I-71 SB) and extends west along the south side of I-71 southbound to Sta. 383+35.41 (BL I-71 SB Transition). The total wall length for Retaining Wall E10 is approximately 555 lineal feet. **Please note that the design of the MSE wall between Sta. 277+55.02 and 277+97.19 (BL I-71 SB) overlaps with Retaining Wall E7 where is crosses in front of the forward abutment with the FRA-71-1503L bridge structure, and as such will be governed by the recommendations in that bridge structure report, which is presented under separate covers.** The wall heights along the portion of the wall alignment that is considered for this exploration report will range from 4.5 feet at Sta. 383+35.41 (BL I-71 SB Transition) to 27.2 feet at Sta. 277+97.19 (BL I-71 SB). The Wall height along the portion of the wall not being considered for this report is 51.5 feet.

It is understood that no net loading is permitted to be applied over the Franklin Main, which is a 60-inch brick sewer that crosses under the wall alignment at approximately Sta. 378+81 (BL I-71 SB Transition). It is understood that the sewer has been previously lined and subsequently loaded to the maximum permissible overburden pressure. Therefore, geofoam blocking in conjunction with undercut of the existing soil will be utilized within the zone of influence of the existing sewer pipe. In Addition, lightweight cellular concrete will be utilized to the east and west of the geofoam section of the wall, which will transition to a traditional mechanically stabilized earth (MSE) wall type along the eastern half of the wall alignment. The various sections of the wall will line up with walls of similar composition that will be constructed as part of Retaining Wall E7 on the north side of Ramp D6 and I-71 southbound.

Based on the plan information provided, it is understood that a standard mechanically stabilized earth (MSE) wall type is being utilized between Sta. 380+20.00 and 383+35.41 (BL I-71 SB Transition). The geofoam section of the wall is being utilized within the zone of influence of the Franklin Main, between Sta. 277+97.19 (BL I-70 SB) and Sta. 379+50.82 (BL I-71 SB Transition), in order to eliminate the loading imparted on the sewer line. The lightweight cellular concrete will be utilized along the reminder of the wall alignment to the west of the geofoam section of wall between Sta. 277+55.02 and 277+97.16 (BL I-71 SB) and to the east of the geofoam section of the wall between Sta. 379+50.82 and 380+20.00 (BL I-71 SB Transition).

2.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1 Site Geology

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

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

2.2 Existing Conditions

The proposed Retaining Wall E10 structure will be situated along the north side of the existing I-70 westbound lanes from approximately 470 feet east of the existing Front Street overpass to just west of the existing Short Street bridge. The existing I-70 westbound in the vicinity of the structure is a three-lane, asphalt paved roadway that is aligned east-to-west. The existing I-70 roadway profile grade is elevated approximately 26 feet above the Short Street profile grade. There is an existing single lane entrance ramp from Mount Street which merges with I-70 westbound just west of the Short Street bridge. There is an existing electrical substation located along the north side of the existing ramp, which is owned and operated by American Electric Power (AEP). The terrain along I-70 slopes down gently to the east and the surrounding area is relatively flat-lying, and the area between I-70 westbound and the Mound Street Entrance Ramp is grass covered and dense vegetation covers the existing embankment slope that supports the entrance ramp.

3.0 EXPLORATION

Between January 20, and February 18, 2020, five (5) structural borings, designated as B-115-3-19 through B-115-7-19, 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 20.0 to 45.0 feet below the existing ground surface along the existing mainline driving lanes of I-70 westbound.

Table 1. Test Boring Summary

Boring Number	Reference Alignment	Station [1]	Offset [1]	Latitude	Longitude	Ground Elevation (feet msl)	Boring Depth (feet)
B-115-3-19	BL I-71 SB	278+33.33	41.1' Rt.	39.953536	-83.004084	733.9	45.0
B-115-4-19	BL I-71 SB	279+95.95	19.2' Rt.	39.953507	-83.003500	736.7	40.0
B-115-5-19	BL I-71 SB	281+08.27	4.6' Rt.	39.953510	-83.003090	738.8	30.0
B-115-6-19	BL I-71 SB	282+49.27	23.3' Rt.	39.953362	-83.002614	740.9	20.0
B-115-7-19	BL I-71 SB	283+89.84	22.1' Rt.	39.953303	-83.002116	741.0	20.5

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 CME 750X all-terrain vehicle (ATV) mounted rotary drilling machine, utilizing a 3.25-inch inside diameter, hollow-stem auger to advance the holes. Standard penetration test (SPT) and split spoon were performed in the borings at 2.5-foot intervals to a depth of 20.0 feet, and at 5.0-foot intervals thereafter to the boring termination depth. The SPT, per the American Society for Testing and Materials (ASTM) designation D1586, is conducted using a 140-pound hammer falling 30.0 inches to drive a 2.0-inch outside diameter split spoon sampler 18.0 inches. Rii utilized a calibrated automatic drop hammer to generate consistent energy transfer to the sampler. Driving resistance is recorded on the boring logs in terms of blow per 6.0-inch interval of the driving distance. The second and third intervals are added to obtain the number of blows per foot (N). Standard penetration blow counts aid in determining soil properties applicable in foundation system design. Measured blow count (N) values are corrected to an equivalent (60%) energy ratio, N_{60}, by the following equation. Both values are represented on boring logs in Appendix III.

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

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

The hammer for the CME 750X drill rig was calibrated on September 3, 2018, and has a drill rod energy ratio of 79.5 percent.

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.

At the completion of drilling, the borings were sealed with the cement-bentonite grout and the pavement was patched within an equivalent thickness of cold patch asphalt or quick set concrete.

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

Table 2. Laboratory Test Schedule

Laboratory Test	Test Designation	Number of Tests Performed
Natural Moisture Content	ASTM D 2216	54
Plastic and Liquid Limits	AASHTO T89, T90	17
Gradation – Sieve/Hydrometer	AASHTO T88	17

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

Borings B-115-3-19, B-115-6-19 and B-115-7-19 were drilled through the existing mainline pavement along the southbound I-71 and encountered a composite pavement section consisting of 8.0 to 9.0 inches of asphalt overlying 10.0 inches of concrete. Borings B-115-4-19 and B-115-5-19, which were performed in the existing outside shoulder encountered full depth asphalt section with 8.0 to 9.0 inches of asphalt. Aggregate base was encountered in all of the borings with thicknesses varying from 6.0 to 12.0 inches.

4.2 Subsurface Soils

Beneath the surface materials in borings B-115-3-19, B-115-4-19 and B-115-5-19, material identified as existing fill or possible fill was encountered extending to a depth of 32.0, 18.0 and 15.5 feet, respectively, below existing grade, which corresponds to an elevation of 701.9, 718.7 and 723.3 feet msl. The existing fill material was generally described as dark brown, brown, black and gray gravel with sand, gravel with sand, silt and clay, sandy silt, silt and clay and silty clay (ODOT A-1-b, A-2-6, A-4a, A-6a, A-6b). The fill contained debris consisting of cinders, red tile/brick fragments, broken rock fragments and possible concrete fragments in boring B-115-3-19 between the depths of

18.0 and 32.0 feet below the existing ground surface. Based on the site topography, the fill material is likely embankment fill that was placed during the original construction of the roadway.

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

The relative density of granular soils is primarily derived from SPT blow counts (N_{60}). Based on the SPT blow counts obtained, the granular soil encountered ranged from loose ($5 < N_{60} \leq 10$ blows per foot [bpf]) to very dense ($N_{60} > 50$ bpf). Overall blow counts recorded from the SPT sampling ranged from 8 bpf to split spoon sampler refusal. The shear strength and consistency of the cohesive soils are primarily derived from the hand penetrometer values (HP). The cohesive soil encountered ranged from stiff ($1.0 < HP \leq 2.0$ tsf) to hard ($HP > 4.0$ tsf). The unconfined compressive strength of the cohesive soil samples tested, obtained from the hand penetrometer, ranged from 1.5 to over 4.5 tsf (limit of instrument).

Natural moisture contents of the soil samples tested ranged from 5 to 28 percent. The natural moisture content of the cohesive soil samples tested for plasticity index ranged from 5 percent below to 3 percent above their corresponding plastic limits. In general, the soil exhibited natural moisture contents considered to be moderately below to slightly above optimum moisture levels.

4.3 Bedrock

Bedrock was not encountered in any of the borings performed in this exploration.

4.4 Groundwater

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

Table 3. Groundwater Levels in Borings

Boring Number	Ground Elevation (feet msl)	Initial Groundwater		Upon Completion	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-115-3-19	733.9	43.5	690.4	40.0	693.9
B-115-4-19	736.7	37.2	699.5	N/A [1]	N/A
B-115-5-19	738.8	24.6	714.2	N/A [1]	N/A
B-115-6-19	740.9	13.5	727.4	16.0	724.9
B-115-7-19	741.0	Dry	-	Dry	-

1. Groundwater at completion not obtained prior to grouting the boreholes.

Groundwater was encountered initially during drilling in all of the borings with the exception of boring B-115-7-19, at depths ranging from 13.5 to 43.5 feet below the ground surface, which corresponds to elevations ranging from 690.4 to 727.4 feet msl. At the completion of drilling in borings B-115-3-19 and B-115-6-19, groundwater was measured at a depth of 40.0 and 16.0 feet below grade, respectively, corresponding to elevations of 693.9 and 724.9 feet msl. The groundwater level at completion in borings B-115-4-19 and B-115-5-19 was not obtained prior to grouting the boreholes. Boring B-115-7-19 was observed to be dry, meaning that no measurable amount of groundwater was observed in the borehole during or at the completion of drilling.

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

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

5.0 ANALYSES AND RECOMMENDATIONS

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

Design details of the proposed retaining wall were provided by the Rii design team and ms consultants. It is understood that a standard MSE wall type is being utilized between Sta. 380+20.00 to 383+35.41 (BL I-71 SB Transition), with a wall height ranging from 4.5 to 18.4 feet. A lightweight cellular concrete modified MSE wall system will be utilized between Sta. 379+50.82 to 380+20.00 (BL I-71 SB Transition), where the wall will span over two existing electrical duct banks, with a wall height ranging from 18.4 to 21.7 feet. A modified MSE wall system consisting of geofoam is being utilized between Sta. 277+97.19 (BL I-71 SB) to Sta. 379+50.82 (BL I-71 SB Transition), where the alignment spans over the influence zone of the Franklin Main, with a wall height ranging from 21.7 to 27.2 feet. It is understood that precast facing panels will be utilized, which will be supported on pedestal footings and anchored into the distribution slab at the top of the wall for the geofoam and cellular concrete modified MSE wall segments.

Additionally, it is understood that no net loading is permitted to be applied over the Franklin Main, which has been previously lined and subsequently loaded to the maximum permissible overburden pressure. Therefore, the geofoam modified MSE wall will be utilized to span I-71 southbound over the existing Franklin Main, where additional undercut of the existing soil and replacement with geofoam blocking will be provided to eliminate the net loading on the existing 60-inch brick sanitary sewer.

The design of retaining wall systems that incorporate geofoam and lightweight cellular concrete fill is considered proprietary. Therefore, only calculations for settlement and bearing capacity for these segments of the wall are provided in this report. If additional analyses for internal stability or external sliding, overturning or global stability are required, they should be performed by a specialty contractor that is qualified to design these systems.

5.1 MSE Wall Recommendations

It is understood that a standard MSE wall type is being utilized between Sta. 380+20.00 to 383+35.41 (BL I-71 SB Transition). MSE walls are constructed on earthen foundations at a minimum depth of 3.0 feet below grade, as defined by the top of the leveling pad to the ground surface located 4.0 feet from the face of the wall. Per Section 204.6.2.1 of the 2019 ODOT BDM, the height of the MSE wall is defined as the elevation difference between the top of coping and the top of the leveling pad. However, it is noted that the reinforced soil mass only extends from the foundation bearing elevation (top of leveling pad) to the roadway subgrade elevation where the roadway is supported on the top of the wall, and the reinforced soil mass extends to the top of the coping where the roadway is not supported on top of the wall. The width of the MSE wall foundation (B) is defined by the length of the reinforced soil mass. Per the Section 204.6.2.1 of the 2019 ODOT BDM and Supplemental Specification (SS) 840, the minimum length of the reinforced soil mass is equal to 70 percent of the height of the MSE wall or 8.0 feet, whichever is greater. A non-structural bearing leveling pad consisting of a minimum of 6.0-inches of unreinforced concrete should be placed at the base of the wall facing for constructability purposes. Please note that the leveling pad is not a structural foundation.

Based on the proposed plan and profile information, the proposed standard MSE wall section will have wall heights ranging from 4.5 to 18.4 feet, as measured from the top of the leveling pad to the top of the coping. For the analysis, the foundation width was set at 70 percent of the wall height and the foundation width was increased, if required, until external and global stability requirements were satisfied.

Per Section 840.06.D of ODOT SS 840, the foundation subgrade should be inspected to verify that the subsurface conditions are the same as those anticipated in this report. The anticipated soils at the proposed bearing elevation along the wall alignment between Sta. 380+20.00 to 383+35.41 (BL I-71 SB Transition) consists of existing fill material described as stiff to very stiff silty clay (ODOT A-6b) and loose to medium dense gravel with sand (ODOT A-1-b) which transitions to natural very stiff sandy silt (ODOT A-4a) along the east end of the wall. Based on the condition of the existing embankment fill encountered in borings B-115-4-16 and B-115-5-16, 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 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 wall are provided in Table 4.

Table 4. Shear Strength Parameters Utilized in Stability Analyses

Material Type	**γ (pcf)**	**φ' [1] (°)**	**c' [2] (psf)**	**S_u [3] (psf)**
MSE Wall Backfill (Select granular backfill)	120	34	0	N/A
Item 203 Embankment Fill (Retained soil)	120	30	0	2,000
Medium Dense to Very Dense Granular Soils (ODOT A-1-a, A-1-b, A-2-4, A-3a)	125 to 135	33 to 42	0	N/A
Very Stiff to Hard Sandy Silt and Silt and Clay (ODOT A-4a, A-6a)	125	28 to 30	0	2,375
Stiff to Very Stiff Silty Clay and Clay (ODOT A-6b, A-7-6)	120 to 130	25 to 26	0	1,125 to 3,875

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

Shear strength parameters for the reinforced soil backfill and retained embankment are provided in ODOT SS 840. Per SS 840, the select granular backfill in the reinforced zone and the retained embankment must meet the shear strength requirements provided in Table 4. The shear strength parameters for the natural soils and existing fill were assigned using correlations provided in FHWA Geotechnical Engineering Circular (GEC) No. 5 (FHWA-NHI-16-072) Evaluation of Soil and Rock Properties and based on past experience in the vicinity of the site with projects performed in similar subsurface profiles.

5.1.2 Bearing Stability

The anticipated bearing materials along the proposed alignment of Retaining Wall E10 between Sta. 380+20 to 383+35.41 (BL I-71 SB Transition) consists of existing fill comprised of stiff to very stiff silty clay (ODOT A-6b) and loose to medium dense gravel with sand (ODOT A-1-b) which transitions to natural very stiff sandy silt (ODOT A-4a) along the east end of the wall. MSE wall foundations bearing on these soils may be proportioned for a factored bearing resistance as indicated in Table 5. A geotechnical resistance factor of φ_b=0.65 was considered in calculating the factored bearing resistance at the strength limit state. The reinforcement length presented in the following table represents the minimum foundation width required to satisfy external and global stability requirements, expressed as a percentage of the wall height.

Table 5. Retaining Wall E10 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]	
380+20.00	382+00.00	18.4	Level	12.9 (0.70H ≥ 8.0)	8.24	5.36	4.70
382+00.00	383+35.41	9.0	Level	8.0	12.76	8.25	2.35

1. *Stationing referenced to the baseline of I-71 SB Transition.*
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 5. 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 standard MSE wall section are provided in Table 6.

Table 6. Compressibility Parameters Utilized in Settlement Analysis

Material Type	γ (pcf)	LL (%)	C_c (1)	C_r (2)	e_o (3)	C_v (4) (ft^2/yr)	N_{60}	C' (5)
Medium Dense to Very Dense Granular Soils (ODOT A-1-a, A-1-b, A-24)	125 to 135	N/A	N/A	N/A	N/A	N/A	8 to 93	59 to 543
Very Stiff Sandy Silt (ODOT A-4a)	125	21 to 24	0.099 to 0.126	0.010 to 0.013	0.436 to 0.460	1,000	N/A	N/A
Very Stiff Silt and Clay (ODOT A-6a)	125	34	0.216	0.022	0.538	600	N/A	N/A
Stiff to Hard Silty Clay (ODOT A-6b)	120	35 to 37	0.225 to 0.243	0.023 to 0.024	0.546 to 0.561	300	N/A	N/A
Very Stiff Clay (ODOT A-7-6)	125	44	0.306	0.031	0.616	150	N/A	N/A

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

Results of the settlement analysis are tabulated in Table 7. Total settlements of up to 2.057 inches at the center of the reinforced soil mass and 1.550 inches at the facing of the wall are anticipated along the alignment of Retaining Wall E10 between Sta. 380+20.00 through 383+35.41 (BL I-71 SB Transition). Based on the results of the analysis, 90 percent of the total settlement at the facing of the wall is anticipated to occur within 9 to 80 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 6 for the underlying soils. Actual settlement and time rate of consolidation should be determined by monitoring the settlement of the wall using settlement platforms.

Table 7. Retaining Wall E10 MSE Wall Settlement Values

From Station [1]	To Station [1]	Service Limit Equivalent Bearing Pressure [2] (ksf)	Total Settlement Values (inches)		Time for 100% Consolidation (Days)
			Center of Wall Mass	Facing of Wall	
380+20.00	382+00.00	2.59 to 3.24	1.481 to 2.057	1.240 to 1.550	35 to 45
382+00.00	383+35.41	0.83 to 1.61	0.344 to 0.669	0.266 to 0.575	9 to 80

1. *Stationing referenced to the baseline of I-71 SB Transition.*
2. *The service limit equivalent bearing pressure is the uniformly distributed pressure asserted by the wall over an effective base width based on the eccentricity of the wall system at the service limit state.*

Per Section 204.6.2.1 of the ODOT BDM, "the maximum allowable differential settlement in the longitudinal direction (regardless of the size of panels) is one (1) percent." Based on the total anticipated settlement at the facing of the walls, maximum differential settlements in the longitudinal directions are anticipated to be less than 1/1,000, which is within the tolerable limit of 1/100. If the total or differential settlement values predicted for the proposed walls present an issue with respect to the deformation tolerances that the walls can withstand, then measures should be taken to minimize the amount of settlement that will occur. This can be achieved by preloading the site and consolidating the underlying soils prior to constructing the walls. If preloading the site is not a desired option, then consideration could be given to ground improvement through the use of stone columns. Settlement calculations are provided in Appendix 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 2018 AASHTO LRFD BDS, for foundations bearing on soil, the location of the resultant of the reaction forces shall be within the middle two-thirds ($^2/_3$) of the base width. Therefore, the limiting eccentricity is one-third ($^1/_3$) of the base width of the wall. Rii performed a verification of the eccentricity of the resultant force for the specified wall height indicated in Table 5. Based on the minimum length of reinforced soil mass presented in Table 5 and utilizing the soil parameters listed in Section 5.1.1 for the retained embankment material, the calculated eccentricity of the resultant force **will not exceed** the limiting eccentricity at the strength limit state.

5.1.5 Sliding Stability

The resistance of the MSE wall to sliding was evaluated per Section 11.10.5.3 of the 2018 AASHTO LRFD BDS. For drained conditions, the sliding resistance is determined by multiplying a coefficient of sliding friction "f" times the total vertical force at the base of the wall. The coefficient of sliding friction is determined based on the limiting friction angle between the foundation soil and the reinforced soil backfill. Based on the soil parameters listed in Section 5.1.1 for the foundation and reinforced soil backfill, a coefficient of sliding friction of 0.49 to 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 existing fill material is estimated to be 1,500 to 2,375 psf.

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 5 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 **<u>will not exceed</u>** the factored shear resistance at the strength limit state for drained or undrained conditions.

5.1.6 Overall (Global) Stability

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

Per Section 11.6.2.3 of the 2018 AASHTO LRFD BDS, overall (global) stability for MSE walls that are not integrated with or supporting structural foundations or elements, global stability is satisfied if the product of the factor of safety from the slope stability output multiplied by the resistance factor φ=0.75 is greater than 1.0. Therefore, global stability is satisfied when a minimum factor of safety of 1.3 is obtained. For an MSE wall designed with the minimum strap length listed in Table 5, the resulting factor of safety under drained conditions (long-term stability) was greater than 1.3. Given the cohesive nature of the subsurface profile, an undrained analysis was also performed to consider short-term condition. The resulting factor of safety under undrained conditions was 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 wall, the recommended controlling strap length is 0.70 times the height of the MSE wall (measured from the top of the leveling pad to the proposed profile grade of the roadway) between Sta. 380+20.00 through 383+35.41 (BL I-71 SB Transition). All of the external and global stability calculations indicate that adequate resistance is available for support of the MSE wall.

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

5.2 Cellular Concrete Wall Recommendations

It is understood that a lightweight cellular concrete modified MSE system wall will be utilized between Sta. 379+50.82 through 380+20.00 (BL I-71 SB Transition), where the wall will span over two existing electrical duct banks, with a wall height ranging from 18.4 to 21.7 feet. Based on information provided by the Rii design team, two types of lightweight cellular concrete will be utilized in lieu of typical embankment fill and select granular fill, which is typically used for MSE wall applications. The wall facing will be connected to geosynthetic straps that are embedded into the cellular concrete and supported on a leveling pad, similar to traditional MSE walls.

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

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

Where,

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

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

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

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

The active earth pressure coefficient, and consequently the active pressure on the back of the cellular concrete mass, will greatly reduce as the slope of the backfill soil flattens. Once the slope of the backfill flattens more than the internal friction angle of the backfill soil, the active earth pressure coefficient will go to zero. Therefore, if the backslope of any backfill is reduced to the internal friction angle of the backfill material, analysis of external stability is not required, with the exception of bearing and overall (global) stability. Based on the plan information provided, it is understood that the cellular concrete fill will be placed the full height of the embankment and full width of both Ramp D6 and I-71 southbound. Provided that all backslopes cut into the existing I-70 embankment are graded no steeper than 2H:1V, external and global stability calculations will not be required for this section of Retaining Wall E10. However, if bearing resistance must be checked, then a factored bearing resistance of 4.49 ksf should be utilized for design at the strength limit state.

The compressibility parameters utilized in the settlement analysis of the proposed cellular concrete backfilled areas are provided in Table 6.

Table 8. Compressibility Parameters Utilized in Settlement Analysis

Material Type	γ (pcf)	LL (%)	C_c (1)	C_r (2)	e_o (3)	C_v (4) (ft^2/yr)	N_{60}	C' (5)
Medium Dense to Very Dense Granular Soils (ODOT A-1-a, A-1-b, A-2-4)	120 to 135	N/A	N/A	N/A	N/A	N/A	8 to 120	59 to 942
Stiff to Hard Silt and Clay (ODOT A-6a, A-6b)	120 to 125	34 to 37	0.216 to 0.243	0.022 to 0.024	0.538 to 0.561	300 to 600	N/A	N/A
Hard Clay (ODOT A-7-6)	125	44	0.306	0.031	0.616	150	N/A	N/A

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

Results of the settlement analysis are tabulated in Table 7. Total settlements of 0.926 to 1.097 inches at the center of the wall mass and 0.667 to 0.771 at the facing of the wall is anticipated along Retaining Wall E10 between Sta. 379+50.82 through 380+20.00 (BL I-71 SB Transition). Based on the results of the analysis, 90 percent of the total settlement at the facing of the wall is anticipated to occur during or immediately following construction of the wall or within 35 to 37 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 6 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 E10 Settlement Results

Boring	Wall Height (feet)	Pressure at Bottom of Wall [1] (ksf)	Total Settlement Values (inches)		Time for 90% Consolidation (Days)
			Center of Wall Mass	Facing of Wall	
B-115-4-19	21.7	0.964	1.097	0.771	35
B-115-5-19	18.4	0.864	0.926	0.667	37

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

Per Section 204.6.2.1 of the ODOT BDM, for traditional MSE walls “the maximum allowable differential settlement in the longitudinal direction (regardless of the size of panels) is one (1) percent.” Based on the total anticipated settlement at the facing of the walls, maximum differential settlements in the longitudinal directions are anticipated to be less than 1/1,000, which is within the tolerable limit of 1/100.

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

5.3 Geofoam Wall Recommendations

A modified MSE wall system consisting of geofoam blocking is being utilized between Sta. 277+97.19 (BL I-71 SB) through Sta. 379+50.82 (BL I-71 SB Transition), where the alignment spans over the influence zone of the Franklin Main, with a wall height ranging from 21.7 to 27.2 feet. Based on the information provided, the typical section for this segment will consist of an approximate 5.0-foot thick pavement section, including asphalt and/or concrete and aggregate base on top of a concrete distribution slab, overlying geofoam blocking (ASTM D6817, Type 19) to the bottom of wall elevation. Where the Franklin Main crosses the wall alignment, at approximately Sta. 378+80 (BL I-71 SB Transition), the wall height is approximately 24.8 feet and the bottom of wall (top of leveling pad) is at El. 730.5 feet msl. Considering a unit weight of 1.5 pcf for the geofoam blocking and 130 pcf for the pavement/aggregate base/distribution slab, the required depth embedment depth of the geofoam blocking below existing grade to provide zero net loading is 5.5 feet based on the maximum wall height of 27.2 feet (23.0 feet above existing grade).

5.4 Lateral Earth Pressure

For the soil types encountered in the borings, the "in-situ" unit weight (γ), cohesion (c), effective angle of friction (φ'), and lateral earth pressure coefficients for at-rest conditions (k_o), active conditions (k_a), and passive conditions (k_p) have been estimated and are provided in Table 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	125	0	32°	0.27	0.47	6.82
Dense to Very Dense Granular Soil	130	0	36°	0.23	0.41	9.09
Compacted Cohesive Engineered Fill	120	2,000	0°	N/A	N/A	N/A
Compacted Granular Engineered Fill	120	0	32°	0.27	0.47	6.82

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

Table 11. Estimated Drained (Long-term) Soil Parameters for Design

Soil Type	γ (pcf) [1]	c (psf)	φ'	k_a	k_o	k_p
Soft to Stiff Cohesive Soil	115	0	26°	0.35	0.56	4.53
Very Stiff to Hard Cohesive Soil	125	0	28°	0.32	0.53	5.07
Very Loose to Loose Granular Soil	120	0	28°	0.32	0.53	5.07
Medium Dense Granular Soil	125	0	32°	0.27	0.47	6.82
Dense to Very Dense Granular Soil	130	0	36°	0.23	0.41	9.09
Compacted Cohesive Engineered Fill	120	0	30°	0.30	0.50	5.58
Compacted Granular Engineered Fill	120	0	32°	0.27	0.47	6.82

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

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

5.5 Construction Considerations

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

5.5.1 Excavation Considerations

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

Table 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

5.5.2 Groundwater Considerations

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

6.0 LIMITATIONS OF STUDY

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

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

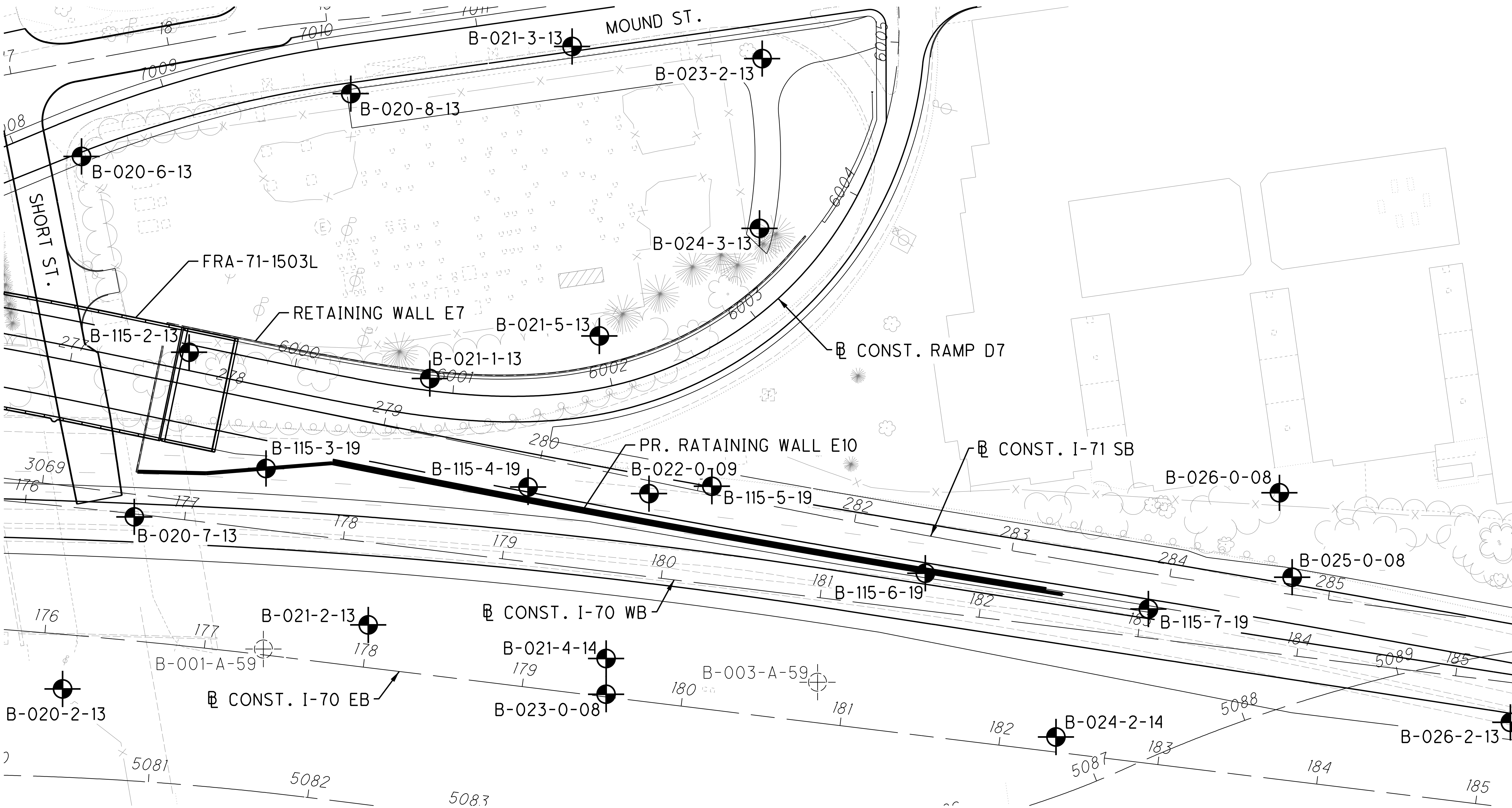
The conclusions and recommendations herein have been based upon the available soil and bedrock information and the design details furnished by a representative of the owner of the proposed project. Any revision in the plans for the proposed construction from those anticipated in this report should be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

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

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

APPENDIX I

VICINITY MAP AND BORING PLAN



BORING PLAN

FRA-70-13.10 - RETAINING WALL E10

FRANKLIN COUNTY, OHIO

RII PROJECT NO. W-13-072	DRAWN RRM
SCALE: 1"=60' (0 30 60)	REVIEWED BRT
N	DATE 3-6-2020

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

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

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

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

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

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

Modifiers of Components - Modifiers of components are as follows:

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

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

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

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

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

Bedrock – The following terms are used to describe bedrock hardness:

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

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

MATERIAL CLASSIFIED BY VISUAL INSPECTION

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

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

APPENDIX III

PROJECT BORING LOGS:

B-115-3-19 through B-115-7-9

BORING LOGS

Definitions of Abbreviations

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

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

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

Classification Test Data

Gradation (as defined on Description of Soil Terms):

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

Atterberg Limits:

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

WC	=	Water content (%)

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-13.10 PHASE 6A	DRILLING FIRM / OPERATOR: RII / S.B.	DRILL RIG: CME 750X (310218)	STATION / OFFSET: 278+33.33 / 41.1' RT	EXPLORATION ID B-115-3-19
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / E.T.	HAMMER: AUTOMATIC	ALIGNMENT: I-71 SB	
PID: 89464 SFN: NA	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 9/3/18	ELEVATION: 733.9 (MSL) EOB: 45.0 ft.	PAGE 1 OF 2
START: 1/20/20 END: 1/20/20	SAMPLING METHOD: SPT	ENERGY RATIO (%): 79.5	LAT / LONG: 39.953536, -83.004084	

MATERIAL DESCRIPTION AND NOTES	ELEV. 733.9	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
0.8' - ASPHALT (9.0")	733.1																	
0.8' - CONCRETE (10.0")	732.3	1																
0.5' - AGGREGATE BASE (6.0")	731.8	2																
FILL: VERY STIFF, BROWN AND DARK BROWN **SANDY SILT**, SOME FINE GRAVEL, LITTLE CLAY, DAMP.		3	5 7 6	17	50	SS-1	3.00	35	18	11	22	14	26	18	8	13	A-4a (0)	
	728.4	4 5	8 7 8	20	56	SS-2	3.00	-	-	-	-	-	-	-	-	14	A-4a (V)	
FILL: VERY STIFF, DARK BROWN AND BROWN **SILTY CLAY**, SOME COARSE TO FINE SAND, LITTLE FINE GRAVEL, MOIST.		6 7	4 10 10	27	56	SS-3	3.00	-	-	-	-	-	-	-	-	19	A-6b (V)	
	724.9	8 9	8 5 7	16	67	SS-4A	3.50	-	-	-	-	-	-	-	-	21	A-6b (V)	
FILL: MEDIUM DENSE, DARK BROWN, BROWN AND GRAY **GRAVEL WITH SAND, SILT, AND CLAY**, DAMP TO MOIST.		10				SS-4B	-	-	-	-	-	-	-	-	-	9	A-2-6 (V)	
-PULVERIZED ROCK FRAGMENTS FROM 11.0-11.5'	722.4	11	10 14 8	29	72	SS-5A	-	-	-	-	-	-	-	-	-	6	A-2-6 (V)	
FILL: VERY STIFF TO HARD, DARK BROWN AND DARK GRAYISH BROWN **SILTY CLAY**, LITTLE COARSE TO FINE SAND, LITTLE FINE GRAVEL, DAMP TO MOIST.		12 13				SS-5B	3.00	-	-	-	-	-	-	-	-	13	A-6b (V)	
		14 15	6 7 14	28	78	SS-6	4.50	15	9	9	32	35	34	18	16	18	A-6b (9)	
	715.9	16 17 18	9 10 12	29	61	SS-7	3.00	-	-	-	-	-	-	-	-	17	A-6b (V)	
FILL: VERY DENSE, DARK BROWN, BLACK AND RED **GRAVEL WITH SAND, SILT, AND CLAY**, MOIST.		19 20	19 19 20	52	67	SS-8	-	-	-	-	-	-	-	-	-	9	A-2-6 (V)	
-CINDERS, RED TILE/BRICK FRAGMENTS, BROKEN ROCK FRAGMENTS, POSSIBLE CONCRETE FRAGMENTS THROUGHOUT	711.9	21 22																
FILL: STIFF, DARK BROWN, BLACK AND RED **SILTY CLAY**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST.		23 24	5 3 3	8	0	SS-9	-	-	-	-	-	-	-	-	-	-		
		25	6	-	100	2S-9A	1.50	-	-	-	-	-	-	-	-	25	A-6b (V)	
		26 27 28																
-CINDERS, RED TILE/BRICK FRAGMENTS, BROKEN ROCK FRAGMENTS, POSSIBLE CONCRETE FRAGMENTS IN SS-10		29	4 5 5	13	78	SS-10	1.50	-	-	-	-	-	-	-	-	22	A-6b (V)	

PID: 89464	SFN: NA	PROJECT: FRA-70-13.10 PHASE 6A	STATION / OFFSET: 27833.33, 41' RT.	START: 1/20/20	END: 1/20/20	PG 2 OF 2	B-115-3-19

MATERIAL DESCRIPTION AND NOTES	ELEV. 703.9	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
FILL: STIFF, DARK BROWN, BLACK AND RED **SILTY CLAY**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST. *(continued)*	701.9	31–32																
MEDIUM DENSE, BROWN AND GRAY **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, MOIST.		34	4 7 10	23	44	SS-11	-	57	10	12	13	8	NP	NP	NP	11	A-1-b (0)	
	696.9																	
DENSE TO VERY DENSE, BROWN AND GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST.		39	8 20 33	70	89	SS-12	-	57	22	11	6	4	NP	NP	NP	10	A-1-a (0)	
-COBBLES PRESENT THROUGHOUT		W 690.4																
	688.9	EOB 45	12 15 14	38	89	SS-13	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	

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

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PUMPED 94 LBS CEMENT / 50 LBS BENTONITE POWDER / 40 GAL WATER; PAVEMENT PATCHED WITH CONCRETE.

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-13.10 PHASE 6A	DRILLING FIRM / OPERATOR: RII / S.B.	DRILL RIG: CME 750X (310218)	STATION / OFFSET: 279+95.95 / 19.2' RT	EXPLORATION ID B-115-4-19
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / J.K.	HAMMER: AUTOMATIC	ALIGNMENT: I-71 SB	
PID: 89464 SFN: NA	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 9/3/18	ELEVATION: 736.7 (MSL) EOB: 40.0 ft.	PAGE 1 OF 2
START: 2/18/20 END: 2/18/20	SAMPLING METHOD: SPT	ENERGY RATIO (%): 79.5	LAT / LONG: 39.953507, -83.003500	

MATERIAL DESCRIPTION AND NOTES	ELEV. 736.7
0.8' - ASPHALT (9.0")	735.9
1.0' - AGGREGATE BASE (12.0")	734.9
POSSIBLE FILL: DENSE, BROWNISH GRAY **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, DAMP. -COBBLES PRESENT @ 4.5'	731.2
POSSIBLE FILL: VERY STIFF, DARK GRAY **SILTY CLAY**, "AND" FINE GRAVEL, LITTLE COARSE TO FINE SAND, MOIST.	723.7
POSSIBLE FILL: VERY STIFF, DARK GRAY **SILT AND CLAY**, SOME COARSE TO FINE SAND, LITTLE FINE GRAVEL, MOIST.	718.7
DENSE TO VERY DENSE, BROWNISH GRAY TO GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST.	

DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
2–3	7 11 11	29	72	SS-1	-	-	-	-	-	-	-	-	-	6	A-1-b (V)
4–5	5 6 26	42	39	SS-2	-	-	-	-	-	-	-	-	-	6	A-1-b (V)
6–7	3 6 5	15	58	SS-3	3.50	47	9	6	21	17	37	17	20	18	A-6b (3)
9–10	8 16 17	44	50	SS-4	3.50	-	-	-	-	-	-	-	-	13	A-6b (V)
11–12	8 10 15	33	33	SS-5	2.50	-	-	-	-	-	-	-	-	12	A-6b (V)
14–15	6 7 6	17	83	SS-6	3.00	-	-	-	-	-	-	-	-	14	A-6a (V)
16–17	5 8 7	20	72	SS-7	3.50	18	13	9	35	25	34	21	13	18	A-6a (6)
19–20	6 10 12	29	67	SS-8	-	-	-	-	-	-	-	-	-	5	A-1-a (V)
24–25	8 10 14	32	61	SS-9	-	59	20	9	9	3	NP	NP	NP	5	A-1-a (0)
29	14 27 34	81	69	SS-10	-	-	-	-	-	-	-	-	-	5	A-1-a (V)

PID: 89464	SFN: NA	PROJECT: FRA-70-13.10 PHASE 6A	STATION / OFFSET: 27995.95, 19' RT.	START: 2/18/20	END: 2/18/20	PG 2 OF 2	B-115-4-19

MATERIAL DESCRIPTION AND NOTES	ELEV. 706.7	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
DENSE TO VERY DENSE, BROWNISH GRAY TO GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST. *(continued)*	704.7	31 – 32																
VERY DENSE, BROWN **GRAVEL WITH SAND AND SILT**, TRACE CLAY, DAMP.		33 – 34	26 50/5"	-	100	SS-11	-	35	30	11	15	9	24	14	10	8	A-2-4 (0)	
		35 – 37 (W 699.5) – 38																
	696.7	39 – 40 EOB	21 35 35	93	89	SS-12	-	-	-	-	-	-	-	-	-	12	A-2-4 (V)	

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 37.2'.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PUMPED 47 LBS CEMENT / 25 LBS BENTONITE POWDER / 20 GAL WATER; PAVEMENT PATCHED WITH CONCRETE.

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-13.10 PHASE 6A	DRILLING FIRM / OPERATOR: RII / S.B.	DRILL RIG: CME 750X (310218)	STATION / OFFSET: 281+08.27 / 4.6' RT
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / J.K.	HAMMER: AUTOMATIC	ALIGNMENT: I-71 SB
PID: 89464 SFN: NA	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 9/3/18	ELEVATION: 738.8 (MSL) EOB: 30.0 ft.
START: 2/17/20 END: 2/17/20	SAMPLING METHOD: SPT	ENERGY RATIO (%): 79.5	LAT / LONG: 39.953510, -83.003090

EXPLORATION ID B-115-5-19

MATERIAL DESCRIPTION AND NOTES	ELEV. 738.8
0.7' - ASPHALT (8.0")	738.1
1.0' - AGGREGATE BASE (12.0")	737.1
FILL: VERY STIFF, BROWN AND DARK BROWN **SILTY CLAY**, SOME COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP TO MOIST.	733.3
POSSIBLE FILL: LOOSE TO MEDIUM DENSE, GRAY **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, MOIST.	728.8
POSSIBLE FILL: STIFF TO VERY STIFF, DARK BROWN AND BROWN **SILTY CLAY**, SOME COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP TO MOIST.	723.3
VERY STIFF, GRAY **CLAY**, SOME SILT, MOIST.	720.8
VERY STIFF, GRAY **SILTY CLAY**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST.	714.2
MEDIUM DENSE, GRAY **GRAVEL WITH SAND AND SILT**, MOIST.	711.8
VERY DENSE, DARK BROWNISH GRAY **GRAVEL WITH SAND**, TRACE SILT, TRACE CLAY, MOIST.	708.8

DEPTHS: W 714.2; EOB

SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
8 10 8	24	39	SS-1	3.50	-	-	-	-	-	-	-	-	7	A-6b (V)
4 4 4	11	64	SS-2	2.50	4	11	16	35	34	35	17	18	19	A-6b (10)
16 9 9	24	44	SS-3	-	-	-	-	-	-	-	-	-	6	A-1-b (V)
3 3 3	8	0	SS-4	-	-	-	-	-	-	-	-	-	-	
4	-	67	2S-4A	2.00	-	-	-	-	-	-	-	-	16	A-6b (V)
2 3 3	8	75	SS-5	2.00	8	15	15	33	29	37	17	20	20	A-6b (9)
3 3 4	9	83	SS-6	2.50	-	-	-	-	-	-	-	-	28	A-6b (V)
6 8 12	27	83	SS-7	4.00	0	0	0	35	65	44	22	22	22	A-7-6 (14)
3 4 5	12	100	SS-8	2.50	-	-	-	-	-	-	-	-	20	A-6b (V)
3 3 6	12	67	SS-9A	2.50	-	-	-	-	-	-	-	-	22	A-6b (V)
			SS-9B	-	-	-	-	-	-	-	-	-	23	A-2-4 (V)
11 21 18	52	83	SS-10	-	-	-	-	-	-	-	-	-	9	A-1-b (V)

NOTES: SEEPAGE ENCOUNTERED @ 23.8'; GROUNDWATER ENCOUNTERED INITIALLY @ 24.6'.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PUMPED 47 LBS CEMENT / 25 LBS BENTONITE POWDER / 20 GAL WATER; PAVEMENT PATCHED WITH CONCRETE.

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-13.10 PHASE 6A	DRILLING FIRM / OPERATOR: RII / S.B.	DRILL RIG: CME 750X (310218)	STATION / OFFSET: 282+49.27 / 23.3' RT	EXPLORATION ID B-115-6-19
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / E.T.	HAMMER: AUTOMATIC	ALIGNMENT: I-71 SB	
PID: 89464 SFN: NA	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 9/3/18	ELEVATION: 740.9 (MSL) EOB: 20.0 ft.	PAGE 1 OF 1
START: 1/23/20 END: 1/23/20	SAMPLING METHOD: SPT	ENERGY RATIO (%): 79.5	LAT / LONG: 39.953362, -83.002614	

MATERIAL DESCRIPTION AND NOTES	ELEV. 740.9	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
0.7' - ASPHALT (8.0")	740.2																	
0.8' - CONCRETE (10.0")	739.4																	
0.5' - AGGREGATE BASE (6.0")	738.9																	
VERY STIFF, BROWN TO GRAYISH BROWN **SANDY SILT**, SOME CLAY, SOME TO LITTLE FINE GRAVEL, DAMP.		2–3.5	6 15 11	34	83	SS-1	3.00	26	15	15	22	22	23	16	7	11	A-4a (2)	
		3.5–5	4 10 9	25	33	SS-2	2.50	-	-	-	-	-	-	-	-	10	A-4a (V)	
		6–7.5	10 10 11	28	100	SS-3	3.00	15	13	16	32	24	24	15	9	11	A-4a (4)	
		8.5–10	5 7 10	23	67	SS-4	2.50	-	-	-	-	-	-	-	-	12	A-4a (V)	
	727.9	11–12.5	4 11 16	36	33	SS-5	2.50	-	-	-	-	-	-	-	-	14	A-4a (V)	
MEDIUM DENSE, GRAY **COARSE AND FINE SAND**, LITTLE FINE GRAVEL, LITTLE SILT, MOIST.	725.9	W 727.4; 13.5–15	4 6 6	16	56	SS-6	-	-	-	-	-	-	-	-	-	13	A-3a (V)	
VERY STIFF TO HARD, GRAY **SILTY CLAY**, TRACE COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST.		16–17.5	7 9 8	23	100	SS-7	4.50	2	2	2	34	60	36	18	18	18	A-6b (11)	
	720.9	18.5–20 EOB	4 6 8	19	89	SS-8	3.00	-	-	-	-	-	-	-	-	21	A-6b (V)	

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 13.5' AND AT COMPLETION @ 16.0'

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PUMPED 47 LBS CEMENT / 25 LBS BENTONITE POWDER / 20 GAL WATER; PAVEMENT PATCHED WITH CONCRETE.

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-13.10 PHASE 6A	DRILLING FIRM / OPERATOR: RII / S.B.	DRILL RIG: CME 750X (310218)	STATION / OFFSET: 283+89.84 / 22.1' RT
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / E.T.	HAMMER: AUTOMATIC	ALIGNMENT: I-71 SB
PID: 89464 SFN: NA	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 9/3/18	ELEVATION: 741.0 (MSL) EOB: 20.5 ft.
START: 1/23/20 END: 1/23/20	SAMPLING METHOD: SPT	ENERGY RATIO (%): 79.5	LAT / LONG: 39.953303, -83.002116

EXPLORATION ID B-115-7-19

MATERIAL DESCRIPTION AND NOTES	ELEV. 741.0
0.7' - ASPHALT (8.0")	740.3
0.8' - CONCRETE (10.0")	739.5
0.5' - AGGREGATE BASE (6.0")	739.0
VERY STIFF TO HARD, GRAYISH BROWN TO GRAY **SANDY SILT**, SOME CLAY, LITTLE TO SOME FINE GRAVEL, DAMP.	720.5

EOB

SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
10 6 7	17	89	SS-1	3.00	-	-	-	-	-	-	-	-	9	A-4a (V)
3 5 5	13	83	SS-2	3.00	17	10	13	32	28	23	14	9	11	A-4a (5)
6 7 8	20	100	SS-3	3.00	-	-	-	-	-	-	-	-	9	A-4a (V)
6 8 8	21	100	SS-4	3.50	23	13	14	26	24	21	14	7	9	A-4a (3)
6 7 9	21	100	SS-5	4.50	-	-	-	-	-	-	-	-	11	A-4a (V)
6 7 8	20	100	SS-6	3.50	-	-	-	-	-	-	-	-	10	A-4a (V)
4 7 7	19	100	SS-7	3.50	20	14	16	27	23	21	13	8	10	A-4a (3)
7 8 10	24	0	SS-8	-	-	-	-	-	-	-	-	-	-	
15	-	100	2S-8A	3.00	-	-	-	-	-	-	-	-	11	A-4a (V)

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING AND AT COMPLETION OF DRILLING

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PUMPED 47 LBS CEMENT / 25 LBS BENTONITE POWDER / 20 GAL WATER; PAVEMENT PATCHED WITH CONCRETE.

02019 NEW STA ODOT BORING LOG (8.5X11) - OH DOT.GDT - 4/6/20 10:38 - U:\GI8\PROJECTS\2013\W-13-072 -2020.GPJ

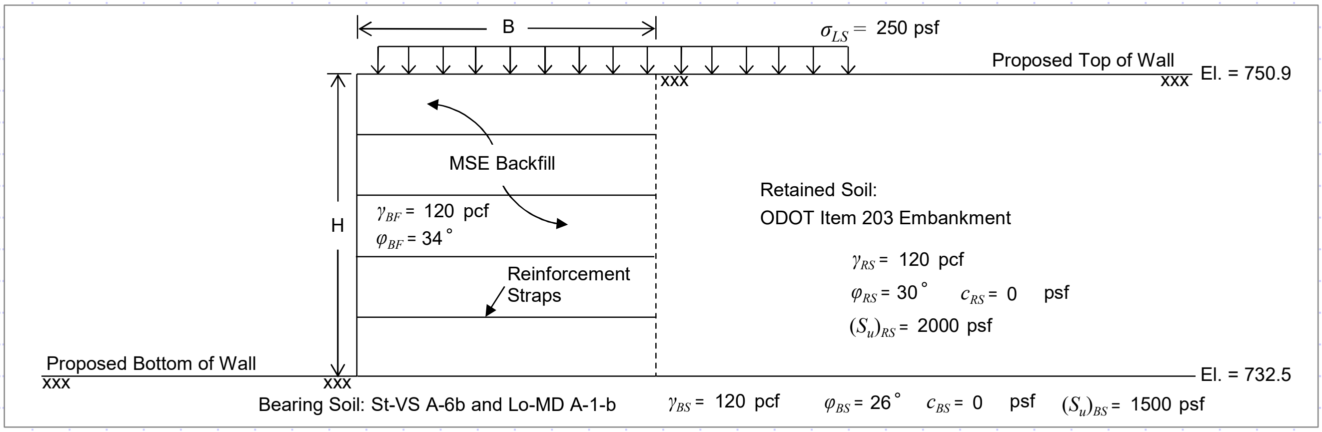
APPENDIX IV

MSE WALL CALCULATIONS

Boring No.	Elevation (feet msl)	Soil Class.	Soil Type	Strata	N_{60}	$N1_{60}$	γ (pcf)	γ' (pcf)	Strength Parameter	k (soil) k_{rm} (rock)	ε_{50} (soil) E_r (rock)	RQD (rock)
B-115-3-19	733.9 to 728.4	A-4a	C	3	19	19	130 psf	130 psf	Su = 2,375 psf	790 pci	0.0058	-
	728.4 to 724.9	A-6b	C	3	27	27	125 psf	125 psf	Su = 3,375 psf	1,125 pci	0.0049	-
	724.9 to 722.4	A-2-4	G	4	16	18	130 psf	130 psf	φ = 36°	160 pci	-	-
	722.4 to 715.9	A-6b	C	3	29	29	135 psf	135 psf	Su = 3,625 psf	1,210 pci	0.0048	-
	715.9 to 711.9	A-2-6	G	4	52	47	130 psf	130 psf	φ = 40°	280 pci	-	-
	711.9 to 701.9	A-6b	C	3	11	11	130 psf	130 psf	Su = 1,375 psf	435 pci	0.0075	-
	701.9 to 696.9	A-1-b	G	4	23	17	135 psf	135 psf	φ = 37°	190 pci	-	-
	696.9 to 688.9	A-1-a	G	4	54	36	135 psf	72.6 psf	φ = 41°	175 pci	-	-
B-115-4-19	736.7 to 731.2	A-1-b	G	4	36	57	130 psf	130 psf	φ = 42°	355 pci	-	-
	731.2 to 723.7	A-6b	C	3	31	31	120 psf	120 psf	Su = 3,875 psf	1,290 pci	0.0047	-
	723.7 to 718.7	A-6a	C	3	19	19	125 psf	125 psf	Su = 2,375 psf	790 pci	0.0058	-
	718.7 to 709.7	A-1-a	G	4	31	27	130 psf	130 psf	φ = 40°	280 pci	-	-
	709.7 to 704.7	A-1-a	G	4	81	64	135 psf	135 psf	φ = 43°	395 pci	-	-
	704.7 to 696.7	A-2-4	G	4	107	77	135 psf	135 psf	φ = 41°	315 pci	-	-
B-115-5-19	738.8 to 733.3	A-6b	C	3	18	18	120 psf	120 psf	Su = 2,250 psf	750 pci	0.0060	-
	733.3 to 728.8	A-1-b	G	4	16	20	125 psf	125 psf	φ = 37°	190 pci	-	-
	728.8 to 723.3	A-6b	C	3	9	9	120 psf	120 psf	Su = 1,125 psf	300 pci	0.0085	-
	723.3 to 720.8	A-7-6	C	3	27	27	125 psf	125 psf	Su = 3,375 psf	1,125 pci	0.0049	-
	720.8 to 714.2	A-6b	C	3	12	12	125 psf	125 psf	Su = 1,500 psf	500 pci	0.0070	-
	714.2 to 711.8	A-2-4	G	4	12	10	125 psf	62.6 psf	φ = 33°	60 pci	-	-
	714.2 to 708.8	A-1-b	G	4	52	44	135 psf	72.6 psf	φ = 41°	175 pci	-	-
B-115-6-19	740.9 to 727.9	A-4a	C	3	19	19	125 psf	125 psf	Su = 2,375 psf	790 pci	0.0058	-
	727.9 to 725.9	A-3a	G	4	16	17	130 psf	67.6 psf	φ = 35°	85 pci	-	-
	725.9 to 720.9	A-6b	C	2	21	21	130 psf	67.6 psf	Su = 2,625 psf	875 pci	0.0055	-
B-115-7-19	741.0 to 720.5	A-4a	C	3	19	19	125 psf	125 psf	Su = 2,375 psf	790 pci	0.0058	-

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

Retaining Wall E10 - Sta. 380+20.00 to 382+00 (BL I-71 SB Transition) - B-115-4-19, B-115-5-19 - 18.4 ft. Wall Height



MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	18.4	ft
MSE Wall Width (Reinforcement Length), (B) =	12.9	ft
MSE Wall Length, (L) =	315	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion [1], (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(S_u)_{BS}$] =	1500	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	20.0	ft

LRFD Load Factors

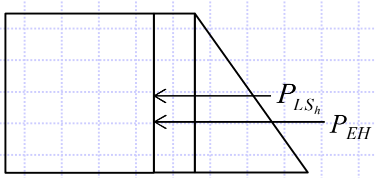
	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

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

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

Sliding Force:

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



$$P_{EH} = \tfrac{1}{2}\gamma_{RS} H^2 K_a \gamma_{EH} = \text{½(120 pcf)(18.4 ft)²(0.297)(1.5)} = 9.05 \text{ kip/ft}$$

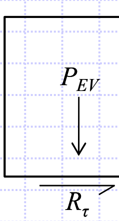
$$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS} = \text{(250 psf)(18.4 ft)(0.297)(1.75)} = 2.39 \text{ kip/ft}$$

$$P_H = 9.05 \text{ kip/ft} + 2.39 \text{ kip/ft} = 11.44 \text{ kip/ft}$$

Check Sliding Resistance - Drained Condition

Nominal Sliding Resistance:

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



$$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV} = \text{(120 pcf)(18.4 ft)(12.9 ft)(1.00)} = 28.48 \text{ kip/ft}$$

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

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

$$R_\tau = \text{(28.48 kip/ft)(0.49)} = 13.96 \text{ kip/ft}$$

Verify Sliding Force Less Than Factored Sliding Resistance - Drained Condition

$$P_H \le R_\tau \cdot \phi_\tau \longrightarrow 11.44 \text{ kip/ft} \le (13.96 \text{ kip/ft})(1.0) = 13.96 \text{ kip/ft} \longrightarrow 11.44 \text{ kip/ft} \le 13.96 \text{ kip/ft}$$

OK

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

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	18.4	ft
MSE Wall Width (Reinforcement Length), (B) =	12.9	ft
MSE Wall Length, (L) =	315	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	1500	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	20.0	ft

LRFD Load Factors

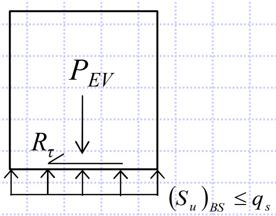
	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

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

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

Check Sliding Resistance - Undrained Condition

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



$(S_u)_{BS} =$ 1.50 ksf

$q_s = \sigma_v / 2$ = (2.21 ksf) / 2 = 1.11 ksf

$\sigma_v = P_{EV} / B$ = (28.48 kip/ft) / (12.9 ft) = 2.21 ksf

R_τ = (1.50 ksf ≤ 1.11 ksf)(12.9 ft) = 14.32 kip/ft

Verify Sliding Force Less Than Factored Sliding Resistance - Undrained Condition

$P_H \le R_\tau \cdot \phi_\tau$ $\longrightarrow$ 11.44 kip/ft ≤ (14.32 kip/ft)(1.0) = 14.32 kip/ft $\longrightarrow$ 11.44 kip/ft ≤ 14.32 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) =	18.4	ft
MSE Wall Width (Reinforcement Length), (B) =	12.9	ft
MSE Wall Length, (L) =	315	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

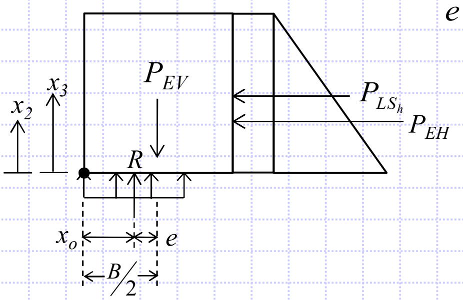
Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	1500	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	20.0	ft

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



$$e = B/2 - x_o$$

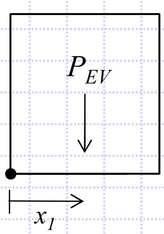
$$x_o = \frac{M_{EV} - M_H}{P_{EV}}$$ = (183.7 kip·ft/ft - 77.46 kip·ft/ft) / (28.48 kip/ft) = 3.73 ft

M_{EV}	=	183.70	kip·ft/ft
M_H	=	77.46	kip·ft/ft
P_{EV}	=	28.48	kip/ft

$\left.\right\}$ Defined below

$e =$ (12.9 ft)/2 - 3.73 ft = 2.72 ft

Resisting Moment, M_{EV}:



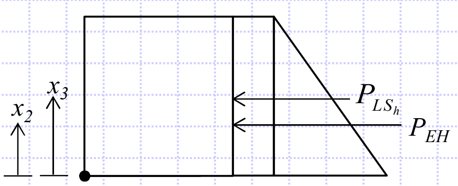
$$M_{EV} = P_{EV}(x_1)$$

$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV}$ = (120 pcf)(18.4 ft)(12.9 ft)(1.00) = 28.48 kip/ft

$x_1 = B/2$ = (12.9 ft) / 2 = 6.45 ft

$M_{EV} =$ (28.48 kip/ft)(6.45 ft) = 183.70 kip·ft/ft

Overturning Moment, M_H:



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

$P_{EH} = \frac{1}{2}\gamma_{RS} H^2 K_a \gamma_{EH}$ = ½(120 pcf)(18.4 ft)²(0.297)(1.5) = 9.05 kip/ft

$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS}$ = (250 psf)(18.4 ft)(0.297)(1.75) = 2.39 kip/ft

$x_2 = H/3$ = (18.4 ft) / 3 = 6.13 ft

$x_3 = H/2$ = (18.4 ft) / 2 = 9.20 ft

$M_H =$ (9.05 kip/ft)(6.13 ft) + (2.39 kip/ft)(9.20 ft) = 77.46 kip·ft/ft

Check Eccentricity

$e < e_{max} \longrightarrow$ 2.72 ft < 4.30 ft **OK**

Limiting Eccentricity: $e_{max} = B/3 \longrightarrow e_{max} =$ (12.9 ft) / 3 = 4.30 ft

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

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	18.4	ft
MSE Wall Width (Reinforcement Length), (B) =	12.9	ft
MSE Wall Length, (L) =	315	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

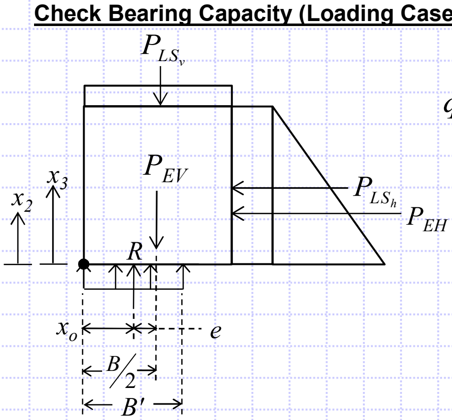
Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	1500	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	20.0	ft

LRFD Load Factors

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

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

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



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

$$B' = B - 2e \quad = \quad 12.9\text{ ft} - 2(1.76\text{ ft}) \quad = \quad 9.38 \quad \text{ft}$$

$$e = B/2 - x_o \quad = \quad (12.9\text{ ft}) / 2 - 4.69\text{ ft} \quad = \quad 1.76 \quad \text{ft}$$

$$x_o = \frac{M_V - M_H}{P_V} \quad = \quad (284.42\text{ kip·ft/ft} - 77.47\text{ kip·ft/ft}) / 44.1\text{ kip/ft} \quad = \quad 4.69 \quad \text{ft}$$

$$q_{eq} = \quad (44.1\text{ kip/ft}) / (9.38\text{ ft}) \quad = \quad 4.70 \quad \text{ksf}$$

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

$$M_V = [(120\text{ pcf})(18.4\text{ ft})(12.9\text{ ft})(1.35)](6.45\text{ ft}) + [(250\text{ psf})(12.9\text{ ft})(1.75)](6.45\text{ ft}) \quad = \quad 284.42 \quad \text{kip·ft/ft}$$

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

$$M_H = [\tfrac{1}{2}(120\text{ pcf})(18.4\text{ ft})^2(0.297)(1.5)](6.13\text{ ft}) + [(250\text{ psf})(18.4\text{ ft})(0.297)(1.75)](9.2\text{ ft}) \quad = \quad 77.47 \quad \text{kip·ft/ft}$$

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

$$P_V = (120\text{ pcf})(18.4\text{ ft})(12.9\text{ ft})(1.35) + (250\text{ psf})(12.9\text{ ft})(1.75) \quad = \quad 44.1 \quad \text{kip/ft}$$

Check Bearing Resistance - Drained Condition

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

$N_{cm} = N_c s_c i_c$ = 22.61

N_c = 22.25
s_c = (9.38 ft/315.410000000003 ft)(11.85/22.
= 1.016
i_c = 1.000 (Assumed)

$N_{qm} = N_q s_q d_q i_q$ = 13.52

N_q = 11.85
s_q = 1.015
d_q = 1+2tan(26°)[1-sin(26°)]²tan⁻¹(4.0 ft/9.38 ft)
1.124
i_q = 1.000 (Assumed)
C_{wq} = 20.0 ft > 4.0 ft = 1.000

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 12.39

N_γ = 12.54
s_γ = 0.988
i_γ = 1.000 (Assumed)
$C_{w\gamma}$ = 20.0 ft > 1.5(9.38 ft) + 20.0 ft = 1.000

$$q_n = (0\text{ psf})(22.606) + (120\text{ pcf})(4.0\text{ ft})(13.519)(1.000) + \tfrac{1}{2}(120\text{ pcf})(9.4\text{ ft})(12.390)(1.000) \quad = \quad 13.46 \quad \text{ksf}$$

Verify Equivalent Pressure Less Than Factored Bearing Resistance

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

$q_{eq} \le q_n \cdot \phi_b \longrightarrow$ 4.70 ksf ≤ (13.46 ksf)(0.65) = 8.75 ksf $\longrightarrow$ 4.70 ksf ≤ 8.75 ksf **OK**

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FAX: (614) 823-4990
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MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	18.4	ft
MSE Wall Width (Reinforcement Length), (B) =	12.9	ft
MSE Wall Length, (L) =	315	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	1500	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	20.0	ft

LRFD Load Factors

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

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

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

Check Bearing Resistance - Undrained Condition

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

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

N_c = 5.140

s_c =).38 ft/[(5)(315.410000000003 = 1.006

i_c = 1.000 (Assumed)

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

N_q = 1.000

s_q = 1.000

d_q = 1+2tan(0°)[1-sin(0°)]²tan⁻¹(4.0 ft/9.38 ft)

1.000

i_q = 1.000 (Assumed)

C_{wq} = 20.0 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)

$C_{w\gamma}$ = 20.0 ft > 1.5(9.38 ft) + 20.0 ft = 1.000

$q_n =$ (1500 psf)(5.170) + (120 pcf)(4.0 ft)(1.000)(1.000) + ½(120 pcf)(9.4 ft)(0.000)(1.000) = 8.24 ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

$q_{eq} \leq q_n \cdot \phi_b$ $\longrightarrow$ 4.70 ksf ≤ (8.24 ksf)(0.65) = 5.36 ksf $\longrightarrow$ 4.70 ksf ≤ 5.36 ksf **OK**

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

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

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	18.4	ft
MSE Wall Width (Reinforcement Length), (B) =	12.9	ft
MSE Wall Length, (L) =	315	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

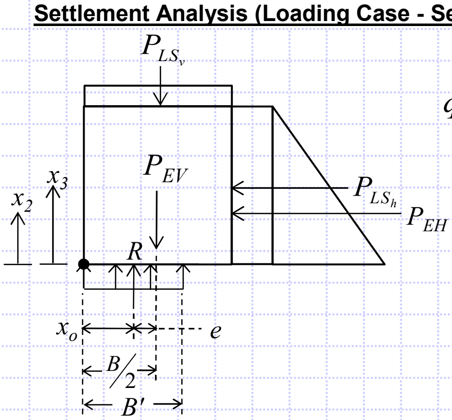
Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	1500	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	20.0	ft

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



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

$$B' = B - 2e \quad = \quad 12.9 \text{ ft} - 2(1.56 \text{ ft}) \quad = \quad 9.78 \quad \text{ft}$$

$$e = B/2 - x_o \quad = \quad (12.9 \text{ ft}) / 2 - 4.89 \text{ ft} \quad = \quad 1.56 \quad \text{ft}$$

$$x_o = \frac{M_V - M_H}{P_V} \quad = \quad (204.52 \text{ kip·ft/ft} - 49.55 \text{ kip·ft/ft}) / 31.71 \text{ kip/ft} \quad = \quad 4.89 \quad \text{ft}$$

$$q_{eq} = \quad (31.71 \text{ kip/ft}) / (9.78 \text{ ft}) \quad = \quad 3.24 \quad \text{ksf}$$

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

$$M_V = \quad [(120 \text{ pcf})(18.4 \text{ ft})(12.9 \text{ ft})(1.00)](6.5 \text{ ft}) + [(250 \text{ psf})(12.9 \text{ ft})(1.00)](6.5 \text{ ft}) \quad = \quad 204.52 \quad \text{kip·ft/ft}$$

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

$$M_H = \quad [\tfrac{1}{2}(120 \text{ pcf})(18.4 \text{ ft})^2(0.297)(1.00)](6.13 \text{ ft}) + [(250 \text{ psf})(18.4 \text{ ft})(0.297)(1.00)](9.2 \text{ ft}) \quad = \quad 49.55 \quad \text{kip·ft/ft}$$

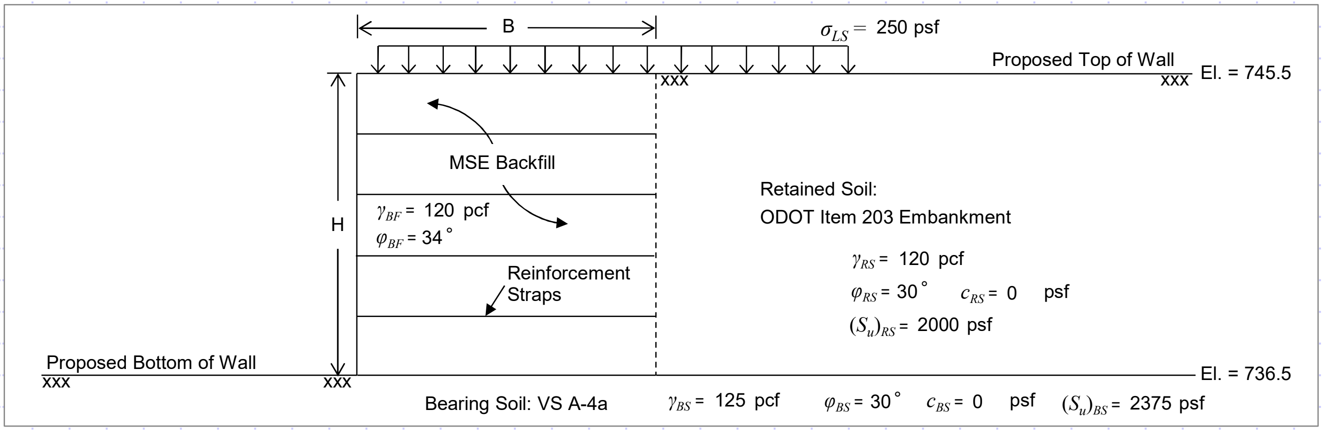
$$P_V = P_{EV} + P_{LS} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV} + \sigma_{LS} \cdot B \cdot \gamma_{LS}$$

$$P_V = \quad (120 \text{ pcf})(18.4 \text{ ft})(12.9 \text{ ft})(1.00) + (250 \text{ psf})(12.9 \text{ ft})(1.00) \quad = \quad 31.71 \quad \text{kip/ft}$$

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-115-4-19	2.057 in	1.550 in	35 days		
B-115-5-19	1.481 in	1.240 in	45 days	88 ft	1/3410
B-115-6-19	0.669 in	0.575 in	9 days	141 ft	1/2540
B-115-7-19	0.344 in	0.266 in	80 days	141 ft	1/5480

Retaining Wall E10 - Sta. 382+00 to 383+35.41 (BL I-71 SB Transition) - B-115-6-19, B-115-7-19 - 9.0 ft. Wall Height



MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	9.0	ft
MSE Wall Width (Reinforcement Length), (B) =	8.0	ft
MSE Wall Length, (L) =	315	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion [1], (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	30	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(S_u)_{BS}$] =	2375	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	9.0	ft

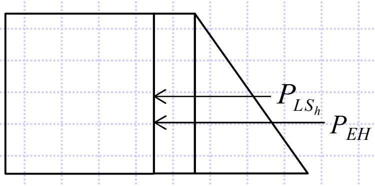
LRFD Load Factors

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

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

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

Sliding Force: $P_H = P_{EH} + P_{LS_h}$



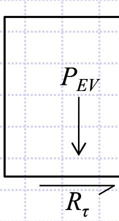
$P_{EH} = \frac{1}{2}\gamma_{RS} H^2 K_a \gamma_{EH}$ = ½(120 pcf)(9 ft)²(0.297)(1.5) = 2.17 kip/ft

$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS}$ = (250 psf)(9 ft)(0.297)(1.75) = 1.17 kip/ft

P_H = 2.17 kip/ft + 1.17 kip/ft = 3.34 kip/ft

Check Sliding Resistance - Drained Condition

Nominal Sliding Resistance: $R_\tau = P_{EV} \cdot \tan\delta$



$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV}$ = (120 pcf)(9 ft)(8.0 ft)(1.00) = 8.64 kip/ft

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

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

R_τ = (8.64 kip/ft)(0.58) = 5.01 kip/ft

Verify Sliding Force Less Than Factored Sliding Resistance - Drained Condition

$P_H \leq R_\tau \cdot \phi_\tau$ $\longrightarrow$ 3.34 kip/ft ≤ (5.01 kip/ft)(1.0) = 5.01 kip/ft $\longrightarrow$ 3.34 kip/ft ≤ 5.01 kip/ft **OK**

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

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

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	9.0	ft
MSE Wall Width (Reinforcement Length), (B) =	8.0	ft
MSE Wall Length, (L) =	315	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	30	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	2375	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	9.0	ft

LRFD Load Factors

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

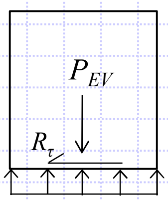
(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

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

Check Sliding Resistance - Undrained Condition

Nominal Sliding Resisting:

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



$(S_u)_{BS}$ = 2.38 ksf

$q_s = \sigma_v / 2$ = (1.08 ksf) / 2 = 0.54 ksf

$\sigma_v = P_{EV} / B$ = (8.64 kip/ft) / (8 ft) = 1.08 ksf

R_τ = (2.38 ksf ≤ 0.54 ksf)(8.0 ft) = 4.32 kip/ft

Verify Sliding Force Less Than Factored Sliding Resistance - Undrained Condition

$P_H \le R_\tau \cdot \phi_\tau$ $\longrightarrow$ 3.34 kip/ft ≤ (4.32 kip/ft)(1.0) = 4.32 kip/ft $\longrightarrow$ 3.34 kip/ft ≤ 4.32 kip/ft **OK**

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

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

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	9.0	ft
MSE Wall Width (Reinforcement Length), (B) =	8.0	ft
MSE Wall Length, (L) =	315	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, $[(S_u)_{RS}]$ =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

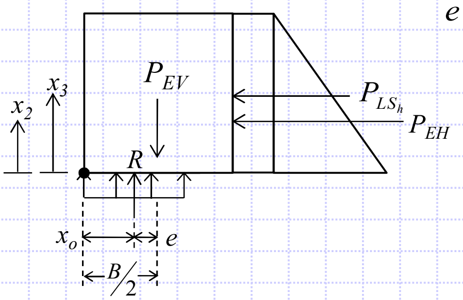
Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	30	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	2375	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	9.0	ft

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



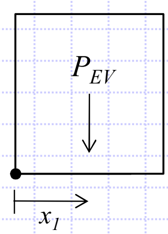
$$e = B/2 - x_o$$

$$x_o = \frac{M_{EV} - M_H}{P_{EV}} = \text{(34.56 kip·ft/ft - 11.78 kip·ft/ft) / (8.64 kip/ft)} = 2.64 \text{ ft}$$

M_{EV}	=	34.56	kip·ft/ft	Defined below
M_H	=	11.78	kip·ft/ft	
P_{EV}	=	8.64	kip/ft	

$$e = \text{(8 ft)/2 - 2.64 ft} = 1.36 \text{ ft}$$

Resisting Moment, M_{EV}:



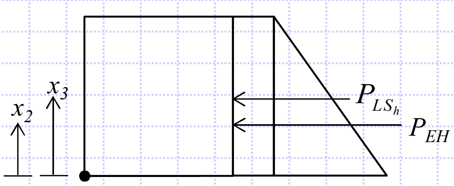
$$M_{EV} = P_{EV}(x_1)$$

$$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV} = \text{(120 pcf)(9 ft)(8.0 ft)(1.00)} = 8.64 \text{ kip/ft}$$

$$x_1 = B/2 = \text{(8.0 ft) / 2} = 4.00 \text{ ft}$$

$$M_{EV} = \text{(8.64 kip/ft)(4.00 ft)} = 34.56 \text{ kip·ft/ft}$$

Overturning Moment, M_H:



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

$$P_{EH} = \tfrac{1}{2}\gamma_{RS} H^2 K_a \gamma_{EH} = \text{½(120 pcf)(9 ft)²(0.297)(1.5)} = 2.17 \text{ kip/ft}$$

$$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS} = \text{(250 psf)(9 ft)(0.297)(1.75)} = 1.17 \text{ kip/ft}$$

$$x_2 = H/3 = \text{(9 ft) / 3} = 3.00 \text{ ft}$$

$$x_3 = H/2 = \text{(9 ft) / 2} = 4.50 \text{ ft}$$

$$M_H = \text{(2.17 kip/ft)(3 ft) + (1.17 kip/ft)(4.50 ft)} = 11.78 \text{ kip·ft/ft}$$

Check Eccentricity

$e < e_{max} \longrightarrow$ 1.36 ft < 2.67 ft **OK**

Limiting Eccentricity: $e_{max} = B/3 \longrightarrow e_{max} =$ (8.0 ft) / 3 = 2.67 ft

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

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	9.0	ft
MSE Wall Width (Reinforcement Length), (B) =	8.0	ft
MSE Wall Length, (L) =	315	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

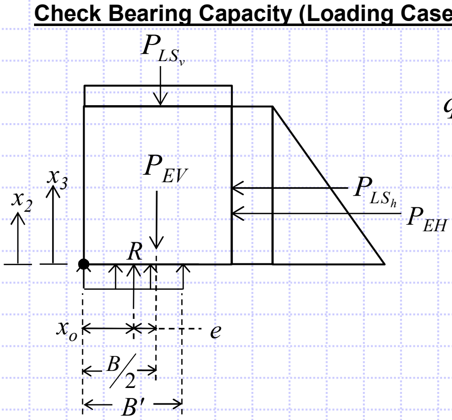
Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	30	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	2375	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	9.0	ft

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



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

$B' = B - 2e$ = 8.0 ft - 2(0.77 ft) = 6.46 ft

$e = B/2 - x_o$ = (8.0 ft) / 2 - 3.23 ft = 0.77 ft

$x_o = \dfrac{M_V - M_H}{P_V}$ = (60.66 kip·ft/ft - 11.76 kip·ft/ft) / 15.16 kip/ft = 3.23 ft

q_{eq} = (15.16 kip/ft) / (6.46 ft) = 2.35 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)(9 ft)(8.0 ft)(1.35)](4 ft) + [(250 psf)(8.0 ft)(1.75)](4 ft) = 60.66 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)(9 ft)²(0.297)(1.5)](3 ft) + [(250 psf)(9 ft)(0.297)(1.75)](4.5 ft) = 11.76 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)(9 ft)(8.0 ft)(1.35) + (250 psf)(8.0 ft)(1.75) = 15.16 kip/ft

Check Bearing Resistance - Drained Condition

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

$N_{cm} = N_c s_c i_c$ = 30.53

N_c = 30.14
s_c = (6.46 ft/315.410000000003 ft)(18.4/30.1
= 1.013
i_c = 1.000 (Assumed)

$N_{qm} = N_q s_q d_q i_q$ = 21.60

N_q = 18.40
s_q = 1.012
d_q = 1+2tan(30°)[1-sin(30°)]²tan⁻¹(4.0 ft/6.46 ft)
1.160
i_q = 1.000 (Assumed)
C_{wq} = 9.0 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)
$C_{w\gamma}$ = 9.0 ft < 1.5(6.46 ft) + 4.0 ft = 0.964

q_n = (0 psf)(30.532) + (125 pcf)(4.0 ft)(21.600)(1.000) + ½(125 pcf)(6.5 ft)(22.221)(0.964) = 19.45 ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

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

$q_{eq} \le q_n \cdot \phi_b$ ⟶ 2.35 ksf ≤ (19.45 ksf)(0.65) = 12.64 ksf ⟶ 2.35 ksf ≤ 12.64 ksf **OK**

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	9.0	ft
MSE Wall Width (Reinforcement Length), (B) =	8.0	ft
MSE Wall Length, (L) =	315	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	30	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	2375	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	9.0	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 = ;.46 ft/[(5)(315.410000000003 = 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/6.46 ft)
1.000
i_q = 1.000 (Assumed)
C_{wq} = 9.0 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)
$C_{w\gamma}$ = 9.0 ft < 1.5(6.46 ft) + 4.0 ft = 0.964

q_n = (2375 psf)(5.160) + (125 pcf)(4.0 ft)(1.000)(1.000) + ½(125 pcf)(6.5 ft)(0.000)(0.964) = 12.76 ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

$q_{eq} \leq q_n \cdot \phi_b$ $\longrightarrow$ 2.35 ksf ≤ (12.76 ksf)(0.65) = 8.29 ksf $\longrightarrow$ 2.35 ksf ≤ 8.29 ksf **OK**

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

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

JOB	FRA-70-13.10	NO.	W-13-072
SHEET NO.	6	OF	6
CALCULATED BY	HSK	DATE	3/19/2020
CHECKED BY	BRT	DATE	4/4/2020

Retaining Wall E10 - Sta. 382+00.00 to 383+35.41

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	9.0	ft
MSE Wall Width (Reinforcement Length), (B) =	8.0	ft
MSE Wall Length, (L) =	315	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

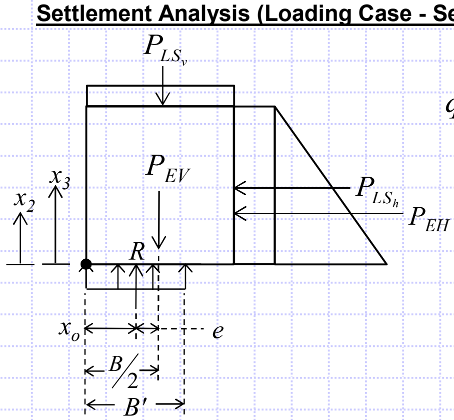
Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	30	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	2375	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	9.0	ft

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



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

$B' = B - 2e$ = 8.0 ft - 2(0.69 ft) = 6.62 ft

$e = B/2 - x_o$ = (8.0 ft) / 2 - 3.31 ft = 0.69 ft

$x_o = \dfrac{M_V - M_H}{P_V}$ = (42.56 kip·ft/ft - 7.34 kip·ft/ft) / 10.64 kip/ft = 3.31 ft

q_{eq} = (10.64 kip/ft) / (6.62 ft) = 1.61 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)(9.0 ft)(8.0 ft)(1.00)](4.0 ft) + [(250 psf)(8.0 ft)(1.00)](4.0 ft) = 42.56 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)(9 ft)²(0.297)(1.00)](3 ft) + [(250 psf)(9 ft)(0.297)(1.00)](4.5 ft) = 7.34 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)(9.0 ft)(8.0 ft)(1.00) + (250 psf)(8.0 ft)(1.00) = 10.64 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-115-4-19	2.057 in	1.550 in	35 days		
B-115-5-19	1.481 in	1.240 in	45 days	88 ft	1/3410
B-115-6-19	0.669 in	0.575 in	9 days	141 ft	1/2540
B-115-7-19	0.344 in	0.266 in	80 days	141 ft	1/5480

W-13-072 - FRA-71-14.36 Phase 6R - Retaining Wall E10

MSE Wall Settlement - Sta. 380+20.00 to 382+00 (BL I-71 SB Transition)

Calculated By: HSK Date: 3/19/2020

Checked By: BRT Date: 4/4/2020

Boring B-115-4-19

H=	18.4	ft	Total wall height
B'=	9.8	ft	Effective footing width due to eccentricity
D_w =	20.0	ft	Depth below bottom of footing
q_e =	3,240	psf	Equivalent bearing pressure at bottom of wall

																				Total Settlement at Center of Reinforced Soil Mass					Total Settlement at Facing of Wall				
Layer	Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
1	A-6b	C	0.0	1.5	1.5	0.8	120	180	90	90	3,090	37	0.243	0.024	0.561				0.08	0.999	3,235	3,325	0.043	0.519	0.500	1,620	1,710	0.030	0.358
	A-6b	C	1.5	3.5	2.0	2.5	120	420	300	300	3,300	37	0.243	0.024	0.561				0.26	0.957	3,101	3,401	0.036	0.438	0.497	1,609	1,909	0.025	0.300
	A-6b	C	3.5	5.5	2.0	4.5	120	660	540	540	3,540	37	0.243	0.024	0.561				0.46	0.844	2,733	3,273	0.024	0.292	0.484	1,567	2,107	0.018	0.221
	A-6b	C	5.5	7.5	2.0	6.5	120	900	780	780	3,780	37	0.243	0.024	0.561				0.66	0.716	2,321	3,101	0.019	0.224	0.460	1,490	2,270	0.014	0.173
2	A-6a	C	7.5	10.0	2.5	8.8	125	1,213	1,056	1,056	4,056	34	0.216	0.022	0.538				0.89	0.596	1,930	2,986	0.016	0.190	0.426	1,380	2,436	0.013	0.153
	A-6a	C	10.0	12.5	2.5	11.3	125	1,525	1,369	1,369	4,369	34	0.216	0.022	0.538				1.15	0.494	1,600	2,969	0.012	0.142	0.385	1,249	2,617	0.010	0.119
3	A-1-a	G	12.5	14.5	2.0	13.5	130	1,785	1,655	1,655	4,655					29	31	101	1.38	0.425	1,377	3,032	0.005	0.062	0.351	1,137	2,792	0.004	0.054
	A-1-a	G	14.5	16.5	2.0	15.5	130	2,045	1,915	1,915	4,915					29	29	97	1.58	0.377	1,222	3,137	0.004	0.053	0.323	1,046	2,961	0.004	0.047
	A-1-a	G	16.5	21.5	5.0	19.0	130	2,695	2,370	2,370	5,370					32	30	100	1.94	0.314	1,017	3,387	0.008	0.093	0.281	910	3,280	0.007	0.085
	A-1-a	G	21.5	26.5	5.0	24.0	135	3,370	3,033	2,783	5,783					81	72	280	2.45	0.253	818	3,601	0.002	0.024	0.234	759	3,542	0.002	0.022
4	A-2-4	G	26.5	31.5	5.0	29.0	135	4,045	3,708	3,146	6,146					120	102	484	2.97	0.211	683	3,829	0.001	0.011	0.200	648	3,794	0.001	0.010
	A-2-4	G	31.5	34.5	3.0	33.0	135	4,450	4,248	3,436	6,436					93	76	305	3.37	0.186	603	4,039	0.001	0.008	0.178	578	4,014	0.001	0.008
																						Total Settlement:		2.057 in			Total Settlement:		1.550 in

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

W-13-072 - FRA-71-14.36 Phase 6R - Retaining Wall E10
MSE Wall Settlement - Sta. 380+20.00 to 382+00 (BL I-71 SB Transition)

Calculated By: HSK Date: 3/19/2020
Checked By: BRT Date: 4/4/2020

Boring B-115-4-19

H=	18.4	ft	Total wall height
B'=	9.8	ft	Effective footing width due to eccentricity
D_w =	20.0	ft	Depth below bottom of footing
q_e =	3,240	psf	Equivalent bearing pressure at bottom of wall

	A-6b	A-6a			
c_v =	300	600		ft^2/yr	Coefficient of consolidation
t =	35	35		days	Time following completion of construction
H_{dr} =	6.3	5		ft	Length of longest drainage path considered
T_v =	0.725	2.301			Time factor
U =	86	100		%	Degree of consolidation

$(S_c)_t$ = 1.403 in Settlement complete at 90% of primary consolidation

Layer	Soil Type	Soil Type	Layer Depth (ft)		Layer Thickness (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	Total Settlement at Facing of Wall: S_c (9,10) (ft)	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	1.5	1.5	0.8	120	180	90	90	3,090	37	0.243	0.024	0.561				0.08	0.500	1,620	1,710	0.030	0.358	1.053	0.308	0.905
	A-6b	C	1.5	3.5	2.0	2.5	120	420	300	300	3,300	37	0.243	0.024	0.561				0.26	0.497	1,609	1,909	0.025	0.300		0.258	
	A-6b	C	3.5	5.5	2.0	4.5	120	660	540	540	3,540	37	0.243	0.024	0.561				0.46	0.484	1,567	2,107	0.018	0.221		0.190	
	A-6b	C	5.5	7.5	2.0	6.5	120	900	780	780	3,780	37	0.243	0.024	0.561				0.66	0.460	1,490	2,270	0.014	0.173		0.149	
2	A-6a	C	7.5	10.0	2.5	8.8	125	1,213	1,056	1,056	4,056	34	0.216	0.022	0.538				0.89	0.426	1,380	2,436	0.013	0.153	0.272	0.153	0.272
	A-6a	C	10.0	12.5	2.5	11.3	125	1,525	1,369	1,369	4,369	34	0.216	0.022	0.538				1.15	0.385	1,249	2,617	0.010	0.119		0.119	
3	A-1-a	G	12.5	14.5	2.0	13.5	130	1,785	1,655	1,655	4,655					29	31	101	1.38	0.351	1,137	2,792	0.004	0.054	0.208	0.054	0.208
	A-1-a	G	14.5	16.5	2.0	15.5	130	2,045	1,915	1,915	4,915					29	29	97	1.58	0.323	1,046	2,961	0.004	0.047		0.047	
	A-1-a	G	16.5	21.5	5.0	19.0	130	2,695	2,370	2,370	5,370					32	30	100	1.94	0.281	910	3,280	0.007	0.085		0.085	
	A-1-a	G	21.5	26.5	5.0	24.0	135	3,370	3,033	2,783	5,783					81	72	280	2.45	0.234	759	3,542	0.002	0.022		0.022	
4	A-2-4	G	26.5	31.5	5.0	29.0	135	4,045	3,708	3,146	6,146					120	102	484	2.97	0.200	648	3,794	0.001	0.010	0.018	0.010	0.018
	A-2-4	G	31.5	34.5	3.0	33.0	135	4,450	4,248	3,436	6,436					93	76	305	3.37	0.178	578	4,014	0.001	0.008		0.008	

Settlement Remaining After Hold Period: 0.147 in

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

W-13-072 - FRA-71-14.36 Phase 6R - Retaining Wall E10
MSE Wall Settlement - Sta. 380+20.00 to 382+00.00 (BL I-71 SB Transition)

Calculated By: HSK Date: 3/20/2020
Checked By: BRT Date: 4/4/2020

Boring B-115-5-19

H=	14.0	ft	Total wall height
B'=	7.3	ft	Effective footing width due to eccentricity
D_w=	20.0	ft	Depth below bottom of footing
q_e =	2,590	psf	Equivalent bearing pressure at bottom of wall

																				Total Settlement at Center of Reinforced Soil Mass					Total Settlement at Facing of Wall				
Layer	Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' [1] (psf)	LL	C_c [2]	C_r [3]	e_o [4]	N_{60}	$(N1)_{60}$ [5]	C' [6]	Z_f/B	I [7]	$\Delta\sigma_v$ [8] (psf)	σ_{vf}' Midpoint (psf)	S_c [9,10] (ft)	S_c (in)	I [7]	$\Delta\sigma_v$ [8] (psf)	σ_{vf}' Midpoint (psf)	S_c [9,10] (ft)	S_c (in)
1	A-6b	C	0.0	1.0	1.0	0.5	120	120	60	60	3,060	35	0.225	0.023	0.546				0.07	0.999	2,587	2,647	0.024	0.287	0.500	1,295	1,355	0.020	0.236
2	A-1-b	G	1.0	3.5	2.5	2.3	125	433	276	276	3,276					24	40	130	0.31	0.933	2,416	2,692	0.019	0.228	0.494	1,281	1,557	0.014	0.173
	A-1-b	G	3.5	5.5	2.0	4.5	120	673	553	553	3,553					8	11	59	0.62	0.745	1,930	2,483	0.022	0.266	0.466	1,208	1,760	0.017	0.205
3	A-6b	C	5.5	8.0	2.5	6.8	120	973	823	823	3,823	37	0.243	0.024	0.561				0.92	0.582	1,507	2,330	0.018	0.211	0.421	1,091	1,913	0.014	0.171
	A-6b	C	8.0	11.0	3.0	9.5	120	1,333	1,153	1,153	4,153	37	0.243	0.024	0.561				1.30	0.447	1,157	2,309	0.014	0.169	0.362	938	2,091	0.012	0.145
4	A-7-6	C	11.0	13.5	2.5	12.3	125	1,645	1,489	1,489	4,489	44	0.306	0.031	0.616				1.68	0.359	929	2,417	0.010	0.120	0.311	806	2,294	0.009	0.107
5	A-6b	C	13.5	16.5	3.0	15.0	125	2,020	1,833	1,833	4,833	37	0.243	0.024	0.561				2.05	0.298	772	2,605	0.007	0.086	0.269	698	2,530	0.007	0.079
	A-6b	C	16.5	20.0	3.5	18.3	125	2,458	2,239	2,239	5,239	37	0.243	0.024	0.561				2.50	0.248	643	2,881	0.006	0.072	0.231	598	2,837	0.006	0.067
6	A-2-4	G	20.0	22.4	2.4	21.2	125	2,758	2,608	2,533	5,533					12	11	58	2.90	0.215	557	3,089	0.004	0.043	0.204	527	3,060	0.003	0.041
7	A-1-b	G	22.4	25.4	3.0	23.9	135	3,163	2,960	2,717	5,717					52	47	156	3.27	0.000	0	2,717	0.000	0.000	0.183	475	3,191	0.001	0.016
																						Total Settlement:		1.481 in			Total Settlement:		1.240 in

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

W-13-072 - FRA-71-14.36 Phase 6R - Retaining Wall E10

MSE Wall Settlement - Sta. 380+20.00 to 382+00.00 (BL I-71 SB Transition)

Calculated By: HSK Date: 03/20/2020

Checked By: BRT Date: 4/4/2020

Boring B-115-5-19

H=	14.0	ft	Total wall height
B'=	7.3	ft	Effective footing width due to eccentricity
D_w =	20.0	ft	Depth below bottom of footing
q_e =	2,590	psf	Equivalent bearing pressure at bottom of wall

	A-6b (Upper)	A-6b (Lower)	A-7-6		
c_v =	300	300	150	ft^2/yr	Coefficient of consolidation
t =	45	45	45	days	Time following completion of construction
H_{dr} =	0.5	7.3	7.3	ft	Length of longest drainage path considered
T_v =	147.945	0.694	0.347		Time factor
U =	100	85	66	%	Degree of consolidation

$(S_c)_t$ = 1.135 in Settlement complete at 91% of primary consolidation

																							Total Settlement at Facing of Wall			Settlement Complete at 91% of Primary Consolidation	
Layer	Soil Type	Soil Type	Layer Depth (ft)		Layer Thickness (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	Layer Settlement (in)	$(S_c)_t$ (11) (in)	Layer Settlement (in)
1	A-6b	C	0.0	1.0	1.0	0.5	120	120	60	60	3,060	35	0.225	0.023	0.546				0.07	0.500	1,295	1,355	0.020	0.236	0.236	0.236	0.236
2	A-1-b	G	1.0	3.5	2.5	2.3	125	433	276	276	3,276					24	40	130	0.31	0.494	1,281	1,557	0.014	0.173	0.378	0.173	0.378
	A-1-b	G	3.5	5.5	2.0	4.5	120	673	553	553	3,553					8	11	59	0.62	0.466	1,208	1,760	0.017	0.205		0.205	
3	A-6b	C	5.5	8.0	2.5	6.8	120	973	823	823	3,823	37	0.243	0.024	0.561				0.92	0.421	1,091	1,913	0.014	0.171	0.316	0.146	0.269
	A-6b	C	8.0	11.0	3.0	9.5	120	1,333	1,153	1,153	4,153	37	0.243	0.024	0.561				1.30	0.362	938	2,091	0.012	0.145		0.123	
4	A-7-6	C	11.0	13.5	2.5	12.3	125	1,645	1,489	1,489	4,489	44	0.306	0.031	0.616				1.68	0.311	806	2,294	0.009	0.107	0.107	0.070	0.070
5	A-6b	C	13.5	16.5	3.0	15.0	125	2,020	1,833	1,833	4,833	37	0.243	0.024	0.561				2.05	0.269	698	2,530	0.007	0.079	0.146	0.067	0.124
	A-6b	C	16.5	20.0	3.5	18.3	125	2,458	2,239	2,239	5,239	37	0.243	0.024	0.561				2.50	0.231	598	2,837	0.006	0.067		0.057	
6	A-2-4	G	20.0	22.4	2.4	21.2	125	2,758	2,608	2,533	5,533					12	11	58	2.90	0.204	527	3,060	0.003	0.041	0.041	0.041	0.041
7	A-1-b	G	22.4	25.4	3.0	23.9	135	3,163	2,960	2,717	5,717					52	47	156	3.27	0.183	475	3,191	0.001	0.016	0.016	0.016	0.016

Settlement Remaining After Hold Period: 0.106 in

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

W-13-072 - FRA-71-14.36 Phase 6R - Retaining Wall E10

MSE Wall Settlement - Sta. 382+00.00 to 383+35.41 (BL I-71 SB Transition)

Calculated By: HSK Date: 3/20/2020

Checked By: BRT Date: 4/4/2020

Boring B-115-6-19

H=	9.0	ft	Total wall height
B'=	6.6	ft	Effective footing width due to eccentricity
D_w=	9.0	ft	Depth below bottom of footing
q_e =	1,610	psf	Equivalent bearing pressure at bottom of wall

																			Total Settlement at Center of Reinforced Soil Mass					Total Settlement at Facing of Wall				
Layer	Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
1	A-4a	C	0.0	2.5	2.5	1.3	125	313	156	156	3,156	24	0.126	0.013	0.460				0.19	0.981	1,579	1,735	0.023	0.271	0.499	803	959	0.017	0.204
	A-4a	C	2.5	5.5	3.0	4.0	125	688	500	500	3,500	24	0.126	0.013	0.460				0.60	0.753	1,212	1,712	0.014	0.166	0.468	753	1,253	0.010	0.124
	A-4a	C	5.5	8.5	3.0	7.0	125	1,063	875	875	3,875	24	0.126	0.013	0.460				1.06	0.527	849	1,724	0.008	0.092	0.400	644	1,519	0.006	0.074
2	A-3a	G	8.5	10.5	2.0	9.5	130	1,323	1,193	1,161	4,161					16	19	65	1.44	0.411	662	1,823	0.006	0.072	0.343	552	1,714	0.005	0.062
3	A-6b	C	10.5	13.0	2.5	11.8	130	1,648	1,485	1,313	4,313	36	0.234	0.023	0.553				1.77	0.341	549	1,862	0.006	0.069	0.299	482	1,796	0.005	0.061
	A-6b	C	13.0	15.5	2.5	14.3	130	1,973	1,810	1,482	4,482	36	0.234	0.023	0.553				2.15	0.000	0	1,482	0.000	0.000	0.260	419	1,901	0.004	0.049
																					Total Settlement:		0.669 in			Total Settlement:		0.575 in

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

W-13-072 - FRA-71-14.36 Phase 6R - Retaining Wall E10

MSE Wall Settlement - Sta. 382+00.00 to 383+35.41 (BL I-71 SB Transition)

Calculated By: HSK Date: 03/20/2020

Checked By: BRT Date: 04/04/2020

Boring B-115-6-19

H=	9.0	ft	Total wall height
B'=	6.6	ft	Effective footing width due to eccentricity
D_w =	9.0	ft	Depth below bottom of footing
q_e =	1,610	psf	Equivalent bearing pressure at bottom of wall

	A-4a	A-6b			
c_v =	1,000	300		ft^2/yr	Coefficient of consolidation
t =	9	9		days	Time following completion of construction
H_{dr} =	4.25	5		ft	Length of longest drainage path considered
T_v =	1.365	0.296			Time factor
U =	97	61		%	Degree of consolidation

$(S_c)_t$ = 0.520 in Settlement complete at 90% of primary consolidation

																								Total Settlement at Facing of Wall		Settlement Complete at 90% of Primary Consolidation	
Layer	Soil Type	Soil Type	Layer Depth (ft)		Layer Thickness (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	Layer Settlement (in)	$(S_c)_t$ (11) (in)	Layer Settlement (in)
1	A-4a	C	0.0	2.5	2.5	1.3	125	313	156	156	3,156	24	0.126	0.013	0.460				0.19	0.499	803	959	0.017	0.204	0.403	0.198	0.390
	A-4a	C	2.5	5.5	3.0	4.0	125	688	500	500	3,500	24	0.126	0.013	0.460				0.60	0.468	753	1,253	0.010	0.124		0.120	
	A-4a	C	5.5	8.5	3.0	7.0	125	1,063	875	875	3,875	24	0.126	0.013	0.460				1.06	0.400	644	1,519	0.006	0.074		0.072	
2	A-3a	G	8.5	10.5	2.0	9.5	130	1,323	1,193	1,161	4,161					16	19	65	1.44	0.343	552	1,714	0.005	0.062	0.062	0.062	0.062
3	A-6b	C	10.5	13.0	2.5	11.8	130	1,648	1,485	1,313	4,313	36	0.234	0.023	0.553				1.77	0.299	482	1,796	0.005	0.061	0.110	0.037	0.067
	A-6b	C	13.0	15.5	2.5	14.3	130	1,973	1,810	1,482	4,482	36	0.234	0.023	0.553				2.15	0.260	419	1,901	0.004	0.049		0.030	

Settlement Remaining After Hold Period: 0.055 in

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

W-13-072 - FRA-71-14.36 Phase 6R - Retaining Wall E10

MSE Wall Settlement - Sta. 382+00.00 to 383+35.41 (BL I-71 SB Transition)

Calculated By: HSK Date: 3/20/2020

Checked By: BRT Date: 4/4/2020

Boring B-115-7-19

H=	4.5	ft	Total wall height
B'=	7.6	ft	Effective footing width due to eccentricity
D_w=	9.0	ft	Depth below bottom of footing
q_e =	830	psf	Equivalent bearing pressure at bottom of wall

																				Total Settlement at Center of Reinforced Soil Mass					Total Settlement at Facing of Wall				
Layer	Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
1	A-4a	C	0.0	2.5	2.5	1.3	125	313	156	156	3,156	21	0.099	0.010	0.436				0.16	0.987	819	975	0.014	0.164	0.499	414	570	0.010	0.116
	A-4a	C	2.5	5.0	2.5	3.8	125	625	469	469	3,469	21	0.099	0.010	0.436				0.49	0.822	683	1,151	0.007	0.081	0.480	399	867	0.005	0.055
	A-4a	C	5.0	7.5	2.5	6.3	125	938	781	781	3,781	21	0.099	0.010	0.436				0.82	0.630	523	1,304	0.004	0.046	0.437	363	1,144	0.003	0.034
	A-4a	C	7.5	10.0	2.5	8.8	125	1,250	1,094	1,094	4,094	21	0.099	0.010	0.436	16	19	39	1.15	0.493	410	1,503	0.002	0.029	0.385	320	1,413	0.002	0.023
	A-4a	C	10.0	13.0	3.0	11.5	125	1,625	1,438	1,282	4,282	21	0.099	0.010	0.436				1.51	0.393	326	1,608	0.002	0.024	0.332	276	1,557	0.002	0.021
	A-4a	C	13.0	16.0	3.0	14.5	125	2,000	1,813	1,469	4,469	21	0.099	0.010	0.436				1.91	0.000	0	1,469	0.000	0.000	0.285	236	1,705	0.001	0.016
																						Total Settlement:		0.344 in			Total Settlement:		0.266 in

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

W-13-072 - FRA-71-14.36 Phase 6R - Retaining Wall E10
MSE Wall Settlement - Sta. 382+00.00 to 383+35.41 (BL I-71 SB Transition)

Calculated By: HSK Date: 03/20/2020
Checked By: BRT Date: 04/04/2020

Boring B-115-7-19

H=	4.5	ft	Total wall height
B'=	7.6	ft	Effective footing width due to eccentricity
D_w =	9.0	ft	Depth below bottom of footing
q_e =	830	psf	Equivalent bearing pressure at bottom of wall

	A-4a				
c_v =	1,000			ft^2/yr	Coefficient of consolidation
t =	80			days	Time following completion of construction
H_{dr} =	16			ft	Length of longest drainage path considered
T_v =	0.856				Time factor
U =	90			%	Degree of consolidation

$(S_c)_t$ = 0.239 in Settlement complete at 90% of primary consolidation

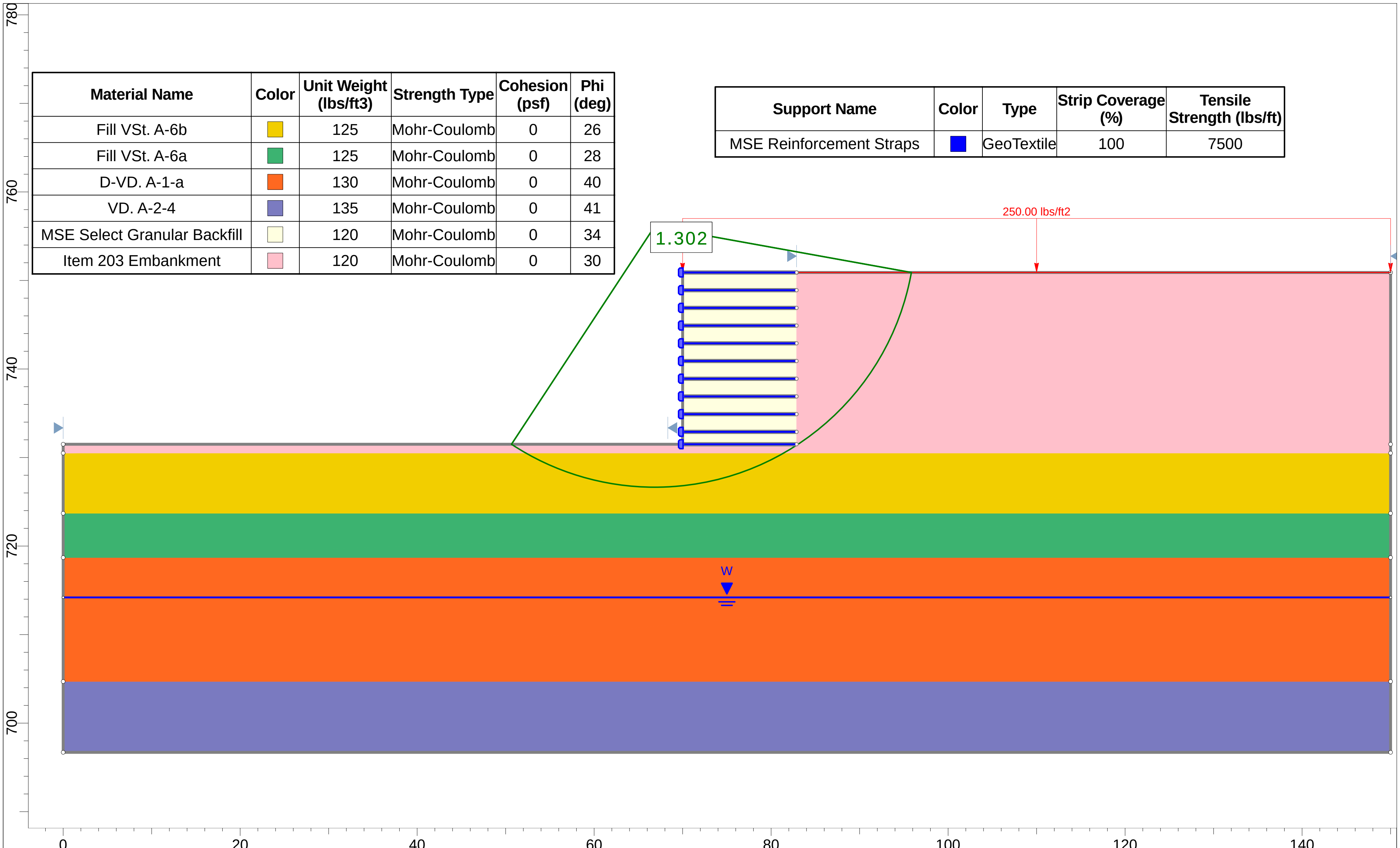
																							Total Settlement at Facing of Wall			Settlement Complete at 90% of Primary Consolidation	
Layer	Soil Type	Soil Type	Layer Depth (ft)		Layer Thickness (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	Layer Settlement (in)	$(S_c)_t$ (11) (in)	Layer Settlement (in)
	A-4a	C	0.0	2.5	2.5	1.3	125	313	156	156	3,156	21	0.099	0.010	0.436				0.16	0.499	414	570	0.010	0.116		0.105	
	A-4a	C	2.5	5.0	2.5	3.8	125	625	469	469	3,469	21	0.099	0.010	0.436				0.49	0.480	399	867	0.005	0.055		0.050	
1	A-4a	C	5.0	7.5	2.5	6.3	125	938	781	781	3,781	21	0.099	0.010	0.436				0.82	0.437	363	1,144	0.003	0.034	0.266	0.031	0.239
	A-4a	C	7.5	10.0	2.5	8.8	125	1,250	1,094	1,094	4,094	21	0.099	0.010	0.436				1.15	0.385	320	1,413	0.002	0.023		0.021	
	A-4a	C	10.0	13.0	3.0	11.5	125	1,625	1,438	1,282	4,282	21	0.099	0.010	0.436				1.51	0.332	276	1,557	0.002	0.021		0.019	
	A-4a	C	13.0	16.0	3.0	14.5	125	2,000	1,813	1,469	4,469	21	0.099	0.010	0.436				1.91	0.285	236	1,705	0.001	0.016		0.014	

Settlement Remaining After Hold Period: 0.027 in

1. $\sigma_p' = \sigma_{vo}' + \sigma_m$; Estimate σ_m of 3,000 psf (slightly to moderately overconsolidated) for all soil deposits; Ref. Table 11.2, Coduto 2003
2. C_c = 0.009(LL-10); Ref. Table 6-9, FHWA GEC 5
3. C_r = 0.10(Cc) for natural soil deposits; Ref. Section 8.11, Holtz and Kovacs 1981
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_N N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \le 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \le \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \le \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (Cohesive soil layers)
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)
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)
Fill VSt. A-6b		125	Mohr-Coulomb	0	26
Fill VSt. A-6a		125	Mohr-Coulomb	0	28
D-VD. A-1-a		130	Mohr-Coulomb	0	40
VD. A-2-4		135	Mohr-Coulomb	0	41
MSE Select Granular Backfill		120	Mohr-Coulomb	0	34
Item 203 Embankment		120	Mohr-Coulomb	0	30

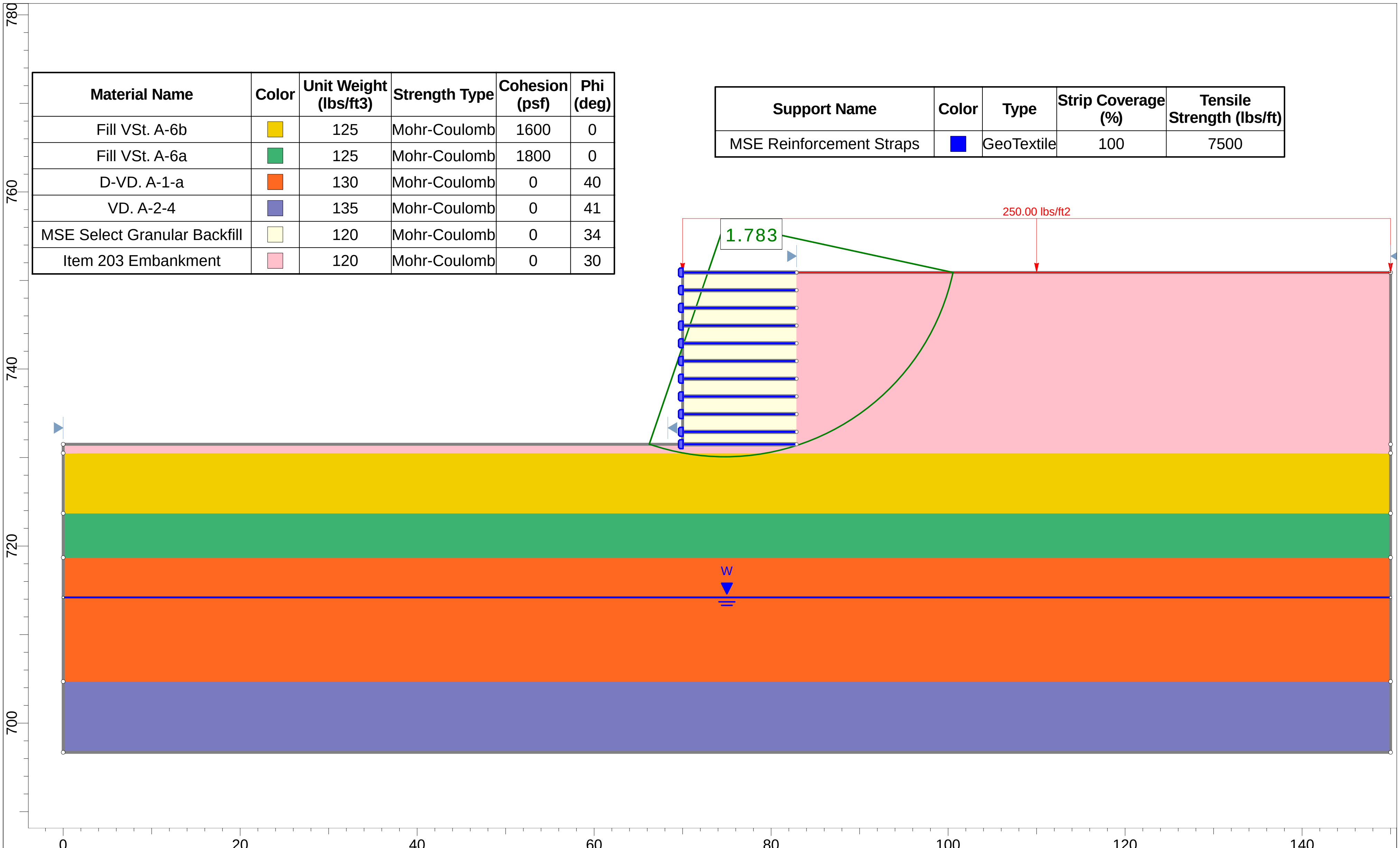
Support Name	Color	Type	Strip Coverage (%)	Tensile Strength (lbs/ft)
MSE Reinforcement Straps		GeoTextile	100	7500



Resource International, Inc. Planning I Engineering I Construction Management I Technology	*Project* Retaining Wall E10 - Sta 380+20.00 to 383+35.41 - MSE Wall Global Stability		
	Analysis Description 18.4 ft Wall Height - Drained Circular - Spencer		
	Drawn By HSK/BRT	*Scale* 1:180	*Company* Resource International, Inc.
SLIDEINTERPRET 8.020	*Date* 04/03/2020		*File Name* Retaining Wall E10 - Global Stability.slmd

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill VSt. A-6b		125	Mohr-Coulomb	1600	0
Fill VSt. A-6a		125	Mohr-Coulomb	1800	0
D-VD. A-1-a		130	Mohr-Coulomb	0	40
VD. A-2-4		135	Mohr-Coulomb	0	41
MSE Select Granular Backfill		120	Mohr-Coulomb	0	34
Item 203 Embankment		120	Mohr-Coulomb	0	30

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



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

SLIDEINTERPRET 8.020

Project Retaining Wall E10 - Sta 380+20.00 to 383+35.41 - MSE Wall Global Stability

Analysis Description 18.4 ft Wall Height - Undrained Circular - Spencer

Drawn By HSK/BRT | *Scale* 1:180 | *Company* Resource International, Inc.

Date 04/03/2020 | *File Name* Retaining Wall E10 - Global Stability - Undrained.slmd

APPENDIX V

CELLULAR CONCRETE WALL CALCULATIONS

W-13-072 - FRA-71-14.36 Phase 6R - Retaining Wall E10

MSE Wall with Cellular Concrete Backfill Settlement - Sta. 379+50.82 to 380+20.00 (BL I-71 SB Transition)

Boring	Boring Elevation	Top of Wall Elevation (ft msl)	Bottom of Wall Elevation (ft msl)	Wall Height (ft)	Pressure at Bottom of Wall [1] (psf)	Total Settlement at Center of Wall (in)	Total Settlement at Wall Facing (in)	Time for 90% Consolidation (Days)
B-115-4-19	736.7	753.2	731.5	21.7	963	1.097	0.771	35
B-115-5-19	738.8	750.9	732.5	18.4	864	0.926	0.667	37

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

W-13-072 - FRA-71-14.36 Phase 6R - Retaining Wall E10

MSE Wall with Cellular Concrete Backfill Settlement - Sta. 379+50.82 to 380+20.00 (BL I-71 SB Transition)

Calculated By: HSK Date: 3/9/2020

Checked By: BRT Date: 4/2/2020

Boring B-115-4-19

H =	21.7	ft	Total wall height from profile grade to top of leveling pad
B =	15.2	ft	Wall width considered in analysis, equal to 70% of the wall height
D_w =	30.0	ft	Depth below bottom of wall
q =	963	psf	Bearing pressure at bottom of wall (see summary sheet)

																				Total Settlement at Center of Reinforced Soil Mass					Total Settlement at Facing of Wall				
Layer	Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
1	A-6b	C	0.0	1.5	1.5	0.8	120	180	90	90	3,090	37	0.243	0.024	0.561				0.05	1.000	963	1,053	0.025	0.299	0.500	481	571	0.019	0.225
	A-6b	C	1.5	3.5	2.0	2.5	120	420	300	300	3,300	37	0.243	0.024	0.561				0.16	0.987	950	1,250	0.019	0.232	0.499	481	781	0.013	0.155
	A-6b	C	3.5	5.5	2.0	4.5	120	660	540	540	3,540	37	0.243	0.024	0.561				0.30	0.939	904	1,444	0.013	0.160	0.495	477	1,017	0.009	0.103
	A-6b	C	5.5	7.5	2.0	6.5	120	900	780	780	3,780	37	0.243	0.024	0.561				0.43	0.864	832	1,612	0.010	0.118	0.486	468	1,248	0.006	0.076
2	A-6a	C	7.5	10.0	2.5	8.8	125	1,213	1,056	1,056	4,056	34	0.216	0.022	0.538				0.58	0.770	742	1,798	0.008	0.097	0.471	454	1,510	0.005	0.065
	A-6a	C	10.0	12.5	2.5	11.3	125	1,525	1,369	1,369	4,369	34	0.216	0.022	0.538				0.74	0.674	649	2,017	0.006	0.071	0.449	433	1,802	0.004	0.050
3	A-1-a	G	12.5	14.5	2.0	13.5	130	1,785	1,655	1,655	4,655					29	31	101	0.89	0.599	576	2,231	0.003	0.031	0.427	411	2,066	0.002	0.023
	A-1-a	G	14.5	16.5	2.0	15.5	130	2,045	1,915	1,915	4,915					29	29	97	1.02	0.542	522	2,437	0.002	0.026	0.406	391	2,306	0.002	0.020
	A-1-a	G	16.5	21.5	5.0	19.0	130	2,695	2,370	2,370	5,370					32	30	100	1.25	0.462	445	2,815	0.004	0.045	0.370	356	2,726	0.003	0.037
	A-1-a	G	21.5	26.5	5.0	24.0	135	3,370	3,033	3,033	6,033					81	70	267	1.58	0.378	364	3,397	0.001	0.011	0.324	312	3,344	0.001	0.010
4	A-2-4	G	26.5	31.5	5.0	29.0	135	4,045	3,708	3,708	6,708					120	95	434	1.91	0.319	307	4,015	0.000	0.005	0.285	274	3,982	0.000	0.004
	A-2-4	G	31.5	34.5	3.0	33.0	135	4,450	4,248	4,060	7,060					93	71	274	2.17	0.283	273	4,333	0.000	0.004	0.258	249	4,309	0.000	0.003
																						Total Settlement:		1.097 in			Total Settlement:		0.771 in

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

W-13-072 - FRA-71-14.36 Phase 6R - Retaining Wall E10

MSE Wall with Cellular Concrete Backfill Settlement - Sta. 379+50.82 to 380+20.00 (BL I-71 SB Transition)

Calculated By: HSK Date: 3/9/2020

Checked By: BRT Date: 4/2/2020

Boring B-115-4-19

H =	21.7	ft	Total wall height from profile grade to top of leveling pad
B =	15.2	ft	Wall width considered in analysis, equal to 70% of the wall height
D_w =	30.0	ft	Depth below bottom of wall
q =	963	psf	Bearing pressure at bottom of wall (see summary sheet)

	A-6b	A-6a			
c_v =	300	600		ft^2/yr	Coefficient of consolidation
t =	35	35		days	Time following completion of construction
H_{dr} =	6.3	5		ft	Length of longest drainage path considered
T_v =	0.725	2.301			Time factor
U =	86	100		%	Degree of consolidation

$(S_c)_t$ = 0.693 in Settlement complete at 90% of primary consolidation

Layer	Soil Type	Soil Type	Layer Depth (ft)		Layer Thickness (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	Total Settlement at Facing of Wall: S_c (9,10) (ft)	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	1.5	1.5	0.8	120	180	90	90	3,090	37	0.243	0.024	0.561				0.05	0.500	481	571	0.019	0.225	0.559	0.193	0.481
	A-6b	C	1.5	3.5	2.0	2.5	120	420	300	300	3,300	37	0.243	0.024	0.561				0.16	0.499	481	781	0.013	0.155		0.133	
	A-6b	C	3.5	5.5	2.0	4.5	120	660	540	540	3,540	37	0.243	0.024	0.561				0.30	0.495	477	1,017	0.009	0.103		0.088	
	A-6b	C	5.5	7.5	2.0	6.5	120	900	780	780	3,780	37	0.243	0.024	0.561				0.43	0.486	468	1,248	0.006	0.076		0.066	
2	A-6a	C	7.5	10.0	2.5	8.8	125	1,213	1,056	1,056	4,056	34	0.216	0.022	0.538				0.58	0.471	454	1,510	0.005	0.065	0.116	0.065	0.116
	A-6a	C	10.0	12.5	2.5	11.3	125	1,525	1,369	1,369	4,369	34	0.216	0.022	0.538				0.74	0.449	433	1,802	0.004	0.050		0.050	
3	A-1-a	G	12.5	14.5	2.0	13.5	130	1,785	1,655	1,655	4,655					29	31	101	0.89	0.427	411	2,066	0.002	0.023	0.089	0.023	0.089
	A-1-a	G	14.5	16.5	2.0	15.5	130	2,045	1,915	1,915	4,915					29	29	97	1.02	0.406	391	2,306	0.002	0.020		0.020	
	A-1-a		16.5	21.5	5.0	19.0	130	2,695	2,370	2,370	5,370					32	30	100	1.25	0.370	356	2,726	0.003	0.037		0.037	
	A-1-a	G	21.5	26.5	5.0	24.0	135	3,370	3,033	3,033	6,033					81	70	267	1.58	0.324	312	3,344	0.001	0.010		0.010	
4	A-2-4	G	26.5	31.5	5.0	29.0	135	4,045	3,708	3,708	6,708					120	95	434	1.91	0.285	274	3,982	0.000	0.004	0.008	0.004	0.008
	A-2-4	G	31.5	34.5	3.0	33.0	135	4,450	4,248	4,060	7,060					93	71	274	2.17	0.258	249	4,309	0.000	0.003		0.003	

Settlement Remaining After Hold Period: 0.078 in

1. $\sigma_p' = \sigma_{vo}' + \sigma_m$; Estimate σ_m of 3,000 psf (slightly to moderately overconsolidated) for all soil deposits; Ref. Table 11.2, Coduto 2003
2. $C_c = 0.009(LL-10)$; Ref. Table 26, FHWA GEC 5
3. $C_r = 0.15(C_c)$ for medium stiff to stiff natural soil deposits and existing fill material, 0.075 to 0.10(C_c) for very stiff to hard natural soil deposits, and 0.05(C_c) for new embankment fill; Ref. Section 5.4.2.5 of FHWA GEC 5
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_N N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \leq \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (Cohesive soil layers)
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)
11. $(S_c)_t = S_c(U/100)$; U = 100 for all granular soils at time t = 0

W-13-072 - FRA-71-14.36 Phase 6R - Retaining Wall E10 | Calculated By: HSK | Date: 3/9/2020

MSE Wall with Cellular Concrete Backfill Settlement - Sta. 379+50.82 to 380+20.00 (BL I-71 SB Transition) | Checked By: BRT | Date: 4/4/2020

Boring B-115-5-19

H =	18.4	ft	Total wall height from profile grade to top of leveling pad
B =	12.9	ft	Wall width considered in analysis, equal to 70% of the wall height
D_w =	30.0	ft	Depth below bottom of wall
q =	864	psf	Bearing pressure at bottom of wall (see summary sheet)

																				Total Settlement at Center of Reinforced Soil Mass					Total Settlement at Facing of Wall				
Layer	Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
1	A-6b	C	0.0	1.0	1.0	0.5	120	120	60	60	3,060	35	0.225	0.023	0.546				0.04	1.000	864	924	0.017	0.207	0.500	432	492	0.013	0.160
2	A-1-b	G	1.0	3.5	2.5	2.3	125	433	276	276	3,276					24	40	130	0.17	0.984	850	1,127	0.012	0.141	0.499	431	707	0.008	0.094
	A-1-b	G	3.5	5.5	2.0	4.5	120	673	553	553	3,553					8	11	59	0.35	0.911	787	1,340	0.013	0.157	0.492	425	978	0.008	0.101
3	A-6b	C	5.5	8.0	2.5	6.8	120	973	823	823	3,823	37	0.243	0.024	0.561				0.52	0.804	694	1,517	0.010	0.124	0.477	412	1,235	0.007	0.082
	A-6b	C	8.0	11.0	3.0	9.5	120	1,333	1,153	1,153	4,153	37	0.243	0.024	0.561				0.74	0.676	584	1,736	0.008	0.100	0.450	389	1,541	0.006	0.071
4	A-7-6	C	11.0	13.5	2.5	12.3	125	1,645	1,489	1,489	4,489	44	0.306	0.031	0.616				0.95	0.571	493	1,982	0.006	0.071	0.417	360	1,849	0.004	0.053
5	A-6b	C	13.5	16.5	3.0	15.0	125	2,020	1,833	1,833	4,833	37	0.243	0.024	0.561				1.16	0.490	423	2,255	0.004	0.051	0.383	331	2,164	0.003	0.040
	A-6b	C	16.5	20.0	3.5	18.3	125	2,458	2,239	2,239	5,239	37	0.243	0.024	0.561				1.41	0.416	360	2,598	0.004	0.042	0.346	299	2,538	0.003	0.036
6	A-2-4	G	20.0	22.4	2.4	21.2	125	2,758	2,608	2,608	5,608					12	11	58	1.64	0.365	316	2,923	0.002	0.025	0.315	272	2,880	0.002	0.021
7	A-1-b	G	22.4	25.4	3.0	23.9	135	3,163	2,960	2,960	5,960					52	45	150	1.85	0.328	283	3,243	0.001	0.010	0.291	251	3,211	0.001	0.008
																						Total Settlement:		0.926 in			Total Settlement:		0.667 in

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

W-13-072 - FRA-71-14.36 Phase 6R - Retaining Wall E10

MSE Wall with Cellular Concrete Backfill Settlement - Sta. 379+50.82 to 380+20.00 (BL I-71 SB Transition)

Calculated By: HSK Date: 3/9/2020

Checked By: BRT Date: 4/4/2020

Boring B-115-5-19

H =	18.4	ft	Total wall height from profile grade to top of leveling pad
B =	12.9	ft	Wall width considered in analysis, equal to 70% of the wall height
D_w =	30.0	ft	Depth below bottom of wall
q =	864	psf	Bearing pressure at bottom of wall (see summary sheet)

	A-6b (Upper)	A-6b (Lower)	A-7-6		
c_v =	300	300	150	ft^2/yr	Coefficient of consolidation
t =	37	37	37	days	Time following completion of construction
H_{dr} =	0.5	7.3	7.3	ft	Length of longest drainage path considered
T_v =	121.644	0.571	0.285		Time factor
U =	100	80	60	%	Degree of consolidation

$(S_c)_t$ = 0.600 in Settlement complete at 90% of primary consolidation

Layer	Soil Type	Soil Type	Layer Depth (ft)		Layer Thickness (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	Total Settlement at Facing of Wall: S_c (9,10) (ft)	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	1.0	1.0	0.5	120	120	60	60	3,060	35	0.225	0.023	0.546				0.04	0.500	432	492	0.013	0.160	0.160	0.160	0.160
2	A-1-b	G	1.0	3.5	2.5	2.3	125	433	276	276	3,276					24	40	130	0.17	0.499	431	707	0.008	0.094	0.195	0.094	0.195
	A-1-b	G	3.5	5.5	2.0	4.5	120	673	553	553	3,553					8	11	59	0.35	0.492	425	978	0.008	0.101		0.101	
3	A-6b	C	5.5	8.0	2.5	6.8	120	973	823	823	3,823	37	0.243	0.024	0.561				0.52	0.477	412	1,235	0.007	0.082	0.153	0.066	0.123
	A-6b	C	8.0	11.0	3.0	9.5	120	1,333	1,153	1,153	4,153	37	0.243	0.024	0.561				0.74	0.450	389	1,541	0.006	0.071		0.057	
4	A-7-6	C	11.0	13.5	2.5	12.3	125	1,645	1,489	1,489	4,489	44	0.306	0.031	0.616				0.95	0.417	360	1,849	0.004	0.053	0.053	0.032	0.032
5	A-6b	C	13.5	16.5	3.0	15.0	125	2,020	1,833	1,833	4,833	37	0.243	0.024	0.561				1.16	0.383	331	2,164	0.003	0.040	0.076	0.032	0.061
	A-6b	C	16.5	20.0	3.5	18.3	125	2,458	2,239	2,239	5,239	37	0.243	0.024	0.561				1.41	0.346	299	2,538	0.003	0.036		0.028	
6	A-2-4		20.0	22.4	2.4	21.2	125	2,758	2,608	2,608	5,608					12	11	58	1.64	0.315	272	2,880	0.002	0.021	0.021	0.021	0.021
7	A-1-b	G	22.4	25.4	3.0	23.9	135	3,163	2,960	2,960	5,960					52	45	150	1.85	0.291	251	3,211	0.001	0.008	0.008	0.008	0.008

Settlement Remaining After Hold Period: 0.067 in

1. $\sigma_p' = \sigma_{vo}' + \sigma_m$; Estimate σ_m of 3,000 psf (slightly to moderately overconsolidated) for all soil deposits; Ref. Table 11.2, Coduto 2003
2. $C_c = 0.009(LL-10)$; Ref. Table 26, FHWA GEC 5
3. $C_r = 0.15(C_c)$ for medium stiff to stiff natural soil deposits and existing fill material, 0.075 to $0.10(C_c)$ for very stiff to hard natural soil deposits, and $0.05(C_c)$ for new embankment fill; Ref. Section 5.4.2.5 of FHWA GEC 5
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_N N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \leq \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (Cohesive soil layers)
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)
11. $(S_c)_t = S_c(U/100)$; U = 100 for all granular soils at time t = 0

Calculated By:	HSK
Date:	3/9/2020
Checked By:	BRT
Date:	4/2/2020

$B =$	15.2	ft	
$L =$	70	ft	
$c =$	0	psf	
$\gamma =$	125	pcf	
$D_f =$	4.0	ft	
$\varphi =$	26	deg	
$D_w =$	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma} = 8.97 \text{ ksf}$$

$$N_{cm} = N_c s_c i_c = 24.82$$

$$N_{qm} = N_q s_q d_q i_q = 14.14$$

$$N_{\gamma m} = N_\gamma s_\gamma i_\gamma = 11.45$$

$N_c =$	22.25	$s_c =$	1+(15.2 ft/70 ft)(11.85/22.25) =	1.116	$i_c =$	1.000	$d_q =$	1+2tan(26°)[1-sin(26°)]²tan⁻¹(4 ft/15.2 ft) =	1.079
$N_q =$	11.85	$s_q =$	1+(15.2 ft/70 ft)tan(26°) =	1.106	$i_q =$	1.000	$C_{wq} =$	0.0 ft < 4.0 ft =	0.500
$N_\gamma =$	12.54	$s_\gamma =$	1-0.4(15.2 ft/70 ft) =	0.913	$i_\gamma =$	1.000	$C_{w\gamma} =$	0.0 ft < 1.5(15.2 ft) + 4 ft =	0.500

$$q_R = q_n \cdot \phi_b = 4.49 \text{ ksf}$$

$\varphi_b =$ 0.5