# FRA-70-12.68 PROJECT 4R
# RETAINING WALL 4W20
# PID NO. 105523
# FRANKLIN COUNTY, OHIO

# STRUCTURE FOUNDATION EXPLORATION REPORT

***Prepared For:***
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**Rii Project No. W-13-045**

**July 2018**

RESOURCE INTERNATIONAL, INC.
ISO | ISO 9001:2008 Certified QMS
*An ISO 9001:2008 QMS Certified Firm*



July 16, 2018

Mr. Christopher W. Luzier, P.E.
Project Manager
GPD GROUP
1801 Watermark Drive, Suite 210
Columbus, OH 43215

**Re: Structure Foundation Exploration Report**
**FRA-70-12.68 Project 4R**
**Retaining Wall 4W20**
**PID No. 105523**
**Rii Project No. W-13-045**

Mr. Luzier:

Resource International, Inc. (Rii) is pleased to submit this structure foundation exploration report for the above referenced project. This report includes recommendations for the design and construction of the proposed Retaining Wall 4W20 as part of the FRA-70-12.68 Project 4R 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.
Project Engineer

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

Enclosure: Structure Foundation Exploration Report

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# APPENDICIES

## EXECUTIVE SUMMARY

Resource International, Inc. (Rii) has completed a structure foundation exploration for the Retaining Wall 4W20, located along I-70 eastbound between Scioto River and CSX and Norfolk Southern railroads. Based on information provided by GPD GROUP, it is understood that the proposed Retaining Wall 4W20 will provide the required grade separation to avoid impacts to the adjacent power substation on the south side of I-70. It is also understood that a mechanically stabilized earth (MSE) wall is being considered as the preferred wall type, which will be between Sta. 5063+70.44, 45.12' Rt to Sta. 5066+44.39, 43.09' Rt. (BL Ramp C5). The wall height will range from 9.5 feet at the beginning of the wall alignment to a maximum height of 18.8 feet, and the total wall length is approximately 235 lineal feet.

### Exploration and Findings

On July 18, 2013, one (1) structural boring, designated as B-016-6-09 was drilled to a completion depth of 50.0 feet below the existing ground surface along the proposed alignment of MSE retaining wall 4W20. In addition to the boring performed by Rii as part of the current exploration, one (1) boring, designated as B-016-1-09, from the preliminary engineering exploration were performed by DLZ in the vicinity of the proposed alignment of MSE retaining wall 4W20. Boring B-016-1-09 was advanced to a depth of 45.0 feet below the existing ground surface.

Borings B-016-1-09 and B-016-6-13 were performed on the south side of the I-70 eastbound, just north of the existing electrical substation, and encountered 3.0 inches of topsoil and 3.0 feet of fill material consisting of sandy silt with stone fragments, respectively, at the ground surface.

Beneath the surface materials, existing fill was encountered in borings B-016-1-09 and B-016-6-13 extending to a depth of 8.0 and 18.5 feet below the existing ground surface, respectively. The fill material consisted of brown and black gravel, gravel and sand, gravel with sand and silt, coarse and fine sand, sandy silt and silt and clay (ODOT A-1-a, A-1-b, A-2-4, A-3a, A-4a, A-6b) and contained brick fragments and wood pieces up to a depth of 18.5 feet below the existing grade as indicated in boring B-016-1-09.

Underlying the surficial materials and fill material, natural granular soils were predominantly present in the upper 37 feet of the soil profile overlying cohesive soils as indicated in boring B-016-1-09 and cohesive soils were predominant in the entire boring with seam of granular material indicated in boring B-016-6-13. The granular soils were generally described as brown, gravel, gravel and sand, and silt (ODOT A-1-a, A-1-b, A-4b). The cohesive soils were described as reddish brown, brown, and gray silty clay and sandy silt (ODOT A-4a, A-6b).

## Analyses and Recommendations

### *MSE Wall Recommendations*

Based on proposed plan and profile information provided by GPD GROUP, the maximum wall height is anticipated to be 18.8 feet, from the top of the leveling pad to the proposed profile grade of the roadway. Since the wall is located within an existing floodplain, the analysis was performed using a design groundwater level at the ground surface.

The anticipated bearing materials along the retaining wall 4W20 consist of existing fill comprised of very loose to medium dense gravel, gravel and sand, coarse and fine sand (ODOT A-1-a, A-1-b, A-3a) with brick fragments, wood pieces and organics. However, as noted in Section 5.1 of the full report, it is understood that ground improvement techniques will be implemented along the alignment of Retaining Wall 4W8, which is in close proximity to this wall and would present the most economical method for stabilizing the soil along this wall. As this is a proprietary design, the analysis for this wall considers the existing fill material will remain in place. MSE wall foundations bearing on existing fill material may be proportioned for a nominal bearing resistance as indicated in Table 5. A geotechnical resistance factor of $\varphi_b$=0.65 was considered in calculating the factored nominal bearing resistance at the strength limit state.

**FRA-70-12.58 MSE Retaining Wall 4W20 Design Parameters**

| Structure Reference | Wall Height Analyzed (feet) | Backslope Behind Wall | Minimum Required Reinforcement Length [1] (feet) | Bearing Resistance at Strength Limit (ksf) | | Strength Limit Equivalent Bearing Pressure [3] (ksf) |
|---|---|---|---|---|---|---|
| | | | | Nominal | Factored [2] | |
| Retaining Wall 4W20 (B-016-1-09 / B-016-6-13) | 18.8 | Level | 13.2 (0.7H) | 6.06 | 3.95 | 4.59 |

1. *The required foundation width is expressed as a percentage of the wall height, H.*
2. *A geotechnical resistance factor of $\varphi_b$=0.65 was considered in calculating the factored bearing resistance at the strength limit state.*
3. *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 4.0 to 4.3 inches at the center of the reinforced mass and 3.2 to 3.3 inches at the facing of the wall are anticipated. Based on the results of the analysis, 90 percent of the total settlement is anticipated to occur over a period of approximately 0 to 7 days.

Based on the results of the external and global stability analysis performed for the MSE wall, bearing stability under drained conditions was not satisfied at a strap length equal to 0.7 times the wall height. Increasing the width of the wall up to 80 percent of wall height may satisfy all of the external and global stability requirements. However, this would introduce a significant risk due to the potential for excessive settlement with time if the organic matter and wood within the fill material decompose.

As noted in Section 5.1 of the full report, consideration was given to over excavating these soils and replacing it with granular embankment; however, given the depth of undercut and proximity to the I-70 roadway and adjacent electrical substation, this may be a very expensive and uneconomical option. Recommendations have been provided in the structure foundation exploration report for Retaining Wall 4W8 to incorporate the use of ground improvement techniques to stabilize the existing fill and underlying cohesive soils that were encountered along that wall, which is in close proximity to this wall location. The recommendations for this alternative should govern the design of this portion of the wall as well.

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-70-12.68/13.11/14.05C (Project 4R/4H/4A) projects in Columbus, Ohio. The projects represent the central portion of FRA-70-8.93 (PID 77369) I-70/71 south innerbelt improvements project. The FRA-70-12.68 (Project 4R) phase will consist of all work associated with the construction of Ramp C5, starting at the bridge over Souder Avenue and extending east to Front Street. The proposed Ramp C5 will be a two-lane to four-lane ramp that will collect and direct traffic from I-71 northbound and SR-315 southbound as well as I-70 eastbound to exit in downtown at the intersection of Front Street and W. Fulton Avenue. This project includes the construction of six (6) new bridge structures for the proposed Ramp C5 alignment and replacement of three (3) bridge structures, two along I-70 and the Front Street Structure over I-70, as well as the construction of fourteen (14) new retaining walls and a culvert structure to accommodate the new configuration.

This report is a presentation of the structure foundation exploration performed for the design and construction of proposed Retaining Wall 4W20, located along I-70 eastbound between Scioto River and CSX and Norfolk Southern railroads, as shown on the vicinity map and boring plan presented in Appendix I. Based on information provided by GPD GROUP, it is understood that the proposed Retaining Wall 4W20 will provide the required grade separation to avoid impacts to the adjacent power substation on the south side of I-70. It is also understood that a mechanically stabilized earth (MSE) wall is being considered as the preferred wall type, which will be between Sta. 5063+70.44, 45.12' Rt to Sta. 5066+44.39, 43.09' Rt. (BL Ramp C5). The wall height will range from 9.5 feet at the beginning of the wall alignment to a maximum height of 18.8 feet, and the total wall length is approximately 235 lineal feet.

## 2.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

### 2.1 Site Geology

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

According to the bedrock geology and topography maps obtained from the Ohio Department of Natural Resources (ODNR), the underlying bedrock 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. Bedrock was not encountered in borings B-016-1-09 and B-016-6-13 within the termination depths.

## 2.2 Existing Conditions

The existing site of the proposed MSE retaining wall 4W20 is just north of an existing electrical substation and north of the Scioto Audubon Park. The project is located along the I-70/71 south innerbelt alignment, primarily along I-70 eastbound between Scioto River, and CSX and Norfolk Southern Railroad. The roadway is an eight-lane expressway in the area which continues into downtown Columbus and crosses under Front Street and High Street. The existing I-70 is elevated from the surrounding terrain from east of the Scioto River to just west of Front Street and there are existing overpass bridges where the roadway crosses the existing CSX and Norfolk Southern Railroads and Short Street. The daily traffic volume along the project alignment is very high. The alignment traverses primarily commercial and government properties. The surrounding terrain across the site is relatively flat-lying, with general slope toward the Scioto River.

# 3.0 EXPLORATION

On July 18, 2013, one (1) structural boring, designated as B-016-6-09 was drilled to a completion depth of 50.0 feet below the existing ground surface along the proposed alignment of MSE retaining wall 4W20. In addition to the boring performed by Rii as part of the current exploration, one (1) boring, designated as B-016-1-09, from the preliminary engineering exploration were performed by DLZ in the vicinity of the proposed alignment of MSE retaining wall 4W20. Boring B-016-1-09 was advanced to a depth of 45.0 feet below the existing ground surface. The project boring locations including the current boring and the preliminary boring are shown on the boring plan provided in Appendix I of this report and summarized in Table 1 below.

**Table 1. Test Boring Summary**

| Boring Number | Reference Alignment | Station | Offset | Latitude | Longitude | Ground Elevation (feet msl) | Boring Depth (feet) |
|---|---|---|---|---|---|---|---|
| B-016-1-09 | BL Ramp C5 | 5064+52.34 | 9.1' Rt. | 39.952340044 | -83.010106810 | 717.2 | 45.0 |
| B-016-6-13 | BL Ramp C5 | 5066+43.00 | 2.9' Rt. | 39.952535194 | -83.009477109 | 717.0 | 50.0 |

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

The boring B-016-6-13 performed by Rii for the current exploration was drilled using an all-terrain vehicle (ATV) mounted rotary drilling machine (CME-750), utilizing a 3.25-inch inside diameter, hollow-stem augers to advance the holes. Standard penetration test (SPT) and split spoon sampling were performed in the borings at 2.5-foot increments of depth to 20 feet and at 5.0-foot increments thereafter to the boring termination depth. The boring B-016-1-09 performed by DLZ for during preliminary exploration was drilled using truck mounted rotary drilling machine (CME-55). Standard penetration test (SPT) and split spoon sampling were performed in the borings at 2.5-foot increments of depth to 30 feet and at 5.0-foot increments 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 hammers for the CME 750X drill rig used by Rii was calibrated on April 26th, 2013, and have drill rod energy ratio of 82.6 percent. The hammer for the CME-55 drill rig used by DLZ for the preliminary exploration borings had a drill rod energy ratio of 62.0 percent.

During drilling, Rii personnel prepared field logs 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 | 14 |
| Plastic and Liquid Limits | AASHTO T89, T90 | 5 |
| Gradation – Sieve/Hydrometer | AASHTO T88 | 5 |

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.

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.

## 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 respective version of the ODOT Specifications for Geotechnical Explorations (SGE) at the time the exploration borings were performed. The following is a summary of what was found in the test borings and what is represented on the boring logs.

## 4.1 Surface Materials

Borings B-016-1-09 and B-016-6-13 were performed on the south side of the I-70 eastbound, just north of the existing electrical substation, and encountered 3.0 inches of topsoil and 3.0 feet of fill material consisting of sandy silt with stone fragments, respectively, at the ground surface.

## 4.2 Subsurface Soils

Beneath the surface materials, existing fill was encountered in borings B-016-1-09 and B-016-6-13 extending to a depth of 8.0 and 18.5 feet below the existing ground surface, respectively. The fill material consisted of brown and black gravel, gravel and sand, gravel with sand and silt, coarse and fine sand, sandy silt and silt and clay (ODOT A-1-a, A-1-b, A-2-4, A-3a, A-4a, A-6b) and contained brick fragments and wood pieces up to a depth of 18.5 feet below the existing grade as indicated in boring B-016-1-09.

Underlying the surficial materials and fill material, natural granular soils were predominantly present in the upper 37 feet of the soil profile overlying cohesive soils as indicated in boring B-016-1-09 and cohesive soils were predominant in the entire boring with seam of granular material indicated in boring B-016-6-13. The granular soils were generally described as brown, gravel, gravel and sand, and silt (ODOT A-1-a, A-1-b, A-4b). The cohesive soils were described as reddish brown, brown, and gray silty clay and sandy silt (ODOT A-4a, A-6b).

The relative density of fill material is primarily derived from SPT blow counts ($N_{60}$). Based on the SPT blow counts obtained, the granular fill material encountered ranged from very loose ($N_{60}$ < 5 blows per foot [bpf]) to very dense ($N_{60}$ > 50 bpf). Overall blow counts recorded from the SPT sampling varied greatly from 3 bpf to 56 bpf in the upper 18.5 feet of fill material. The natural granular soil encountered ranged from medium dense ($11 \leq N_{60} \leq 30$ bpf) to very dense ($N_{60}$ > 50 bpf). Overall blow counts recorded from the SPT sampling ranged from 18 bpf to 51 bpf within the natural soils. The shear strength and consistency of the cohesive soils are primarily derived from the hand penetrometer values (HP). The cohesive soil encountered ranged from medium stiff ($0.5 < HP \leq 1.0$ tsf) to hard (HP > 4.0 tsf). The unconfined compressive strength of the cohesive soil samples tested, obtained from the hand penetrometer, ranged from 1.0 to over 4.5 tsf (limit of instrument).

Natural moisture contents of the soil samples tested ranged from 4.0 to 48.0 percent. A moisture content of 52.6 percent was obtained in sample SS-7 from boring B-016-1-09. The high moisture content is likely due to the presence of wood fragments and other organic matter in the sample. The natural moisture content of the cohesive soil samples tested for plasticity index ranged from 0 to 44 percent above their corresponding plastic limits. In general, the soil exhibited natural moisture contents considered to be at to significantly above optimum moisture levels.

### 4.3 Bedrock

Bedrock was not encountered in either of the borings within the termination depts.

### 4.4 Groundwater

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

**Table 3. Groundwater**

| Boring Number | Ground Elevation (feet msl) | Initial Groundwater | | Upon Completion | |
|---|---|---|---|---|---|
| | | Depth (feet) | Elevation (feet msl) | Depth (feet) | Elevation (feet msl) |
| B-016-1-09 | 717.2 | 21.0 | 696.2 | 34.8 | 682.4 |
| B-016-6-13 | 717.0 | 23.5 | 693.5 | 24.0 | 693.0 |

Groundwater was encountered initially during drilling in both the borings at depths ranging from 21.0 to 23.5 feet below the existing ground surface, which corresponds to elevations ranging from 693.5 to 696.2 feet msl. The groundwater level at the completion of drilling was measured at depths ranging from 34.8 to 24.0 feet below the existing ground surface, corresponding to elevations ranging from 682.4 to 693.0 feet msl. 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 MSE retaining wall, 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 GPD GROUP. Based on the information provided, it is understood that Retaining Wall 4W20 will be an MSE wall type with a height ranging from 9.5 feet at the beginning of the wall alignment to a maximum height of 18.8 feet, and a total wall length is approximately 235.6 lineal feet.

## 5.1 MSE Wall Recommendations

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

Based on proposed plan and profile information provided by GPD GROUP, the maximum wall height is anticipated to be 18.8 feet, from the top of the leveling pad to the proposed profile grade of the roadway. Therefore, it is considered that the minimum reinforcement length and the effective foundation width (B) of the MSE wall for external and global stability calculations will be 13.2 feet. For the analysis, the foundation width was set at 70 percent of the wall height, and the foundation width was increased, as necessary, 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. Existing fill consisting of very loose to loose gravel overlying coarse and fine sand (ODOT A-1-a, A-3a) was encountered at the proposed bearing elevation, which extends to a depth ranging from 9.5 to 15.3 feet below the proposed bearing elevation, which corresponds to elevation of 698.7 feet msl in boring B-016-1-09. It is important to note that the fill material contained brick fragments, wood pieces, and organic material. **These soils are not considered suitable for foundation support for a wall of this size.** Consideration was given to over excavating these soils and replacing it with granular embankment; however, given the depth of undercut and proximity to the I-70 roadway and adjacent electrical substation, this may be a very expensive and uneconomical option. Recommendations have been provided in the structure foundation exploration report for Retaining Wall 4W8 to incorporate the use of ground improvement techniques to stabilize the existing fill and underlying soils along that wall, which is just to the east of this wall. The recommendations for this alternative should govern the design of this portion of the wall. For this report, the analysis of Wall 4W20 has been conducted using the soil profile as encountered in the borings.

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

Since the walls are located within an existing floodplain, the analyses were performed using a design groundwater level at the ground surface.

### 5.1.1 *Strength Parameters Utilized in External and Global Stability Analyses*

The shear strength parameters utilized in the external and global stability analyses for the MSE walls at the abutments are provided in Table 4.

**Table 4. Shear Strength Parameters Utilized in MSE Wall Stability Analyses**

| Material Type | $\gamma$ (pcf) | $\varphi'$ [1] (°) | $c'$ [2] (psf) | $S_u$ [3] (psf) |
|---|---|---|---|---|
| MSE Wall Backfill (Select granular fill) | 120 | 34 | 0 | N/A |
| Item 203 Granular Embankment (Retained Soil at 4W20) | 130 | 33 | 0 | N/A |
| Existing Fill: Stiff Silt and Clay (ODOT A-6a) | 115 | 26 [4] | 0 | 1,500 |
| Medium Dense to Very Dense Gravel, Gravel and Sand (ODOT A-1-a, A-1-b) | 125 to 135 | 37 to 42 | 0 | N/A |
| Very Loose to Loose Coarse and Fine Sand (ODOT A-3a) | 120 | 26 | 0 | N/A |
| Loose Gravel with Sand and Silt (ODOT A-2-4) | 125 | 30 | 0 | N/A |
| Hard Silt (ODOT A-4b) | 130 | 32 | 0 | N/A |
| Stiff to Very Stiff Sandy Silt (ODOT A-4a) | 120 | 28 | 0 | 3,125 |
| Very Stiff to Hard Silt and Clay (ODOT A-6b) | 130 | 27 | 50 | 6,500 |

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).*
4. *Friction angle based on the significant presence of debris and deleterious materials present within the existing fill.*

Shear strength parameters for the reinforced soil backfill are provided in ODOT SS 840. Per SS 840, the select granular backfill in the reinforced zone must meet the shear strength requirements provided in Table 4. Based on the design plans provided by GPD GROUP, it is understood that Item 203 granular embankment will be utilized where any new embankment will be placed behind the reinforced soil backfill at both MSE walls. Therefore, the shear strength parameters for the retained fill will be modeled using a friction angle of 33 degrees since granular embankment is being specified, instead of using the shear strength parameters provided in ODOT SS 840.

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

### *5.1.2 Bearing Stability*

The anticipated bearing materials along the retaining wall 4W20 consist of existing fill comprised of very loose to medium dense gravel, gravel and sand, coarse and fine sand (ODOT A-1-a, A-1-b, A-3a) with brick fragments, wood pieces and organics. However, as noted in Section 5.1, it is understood that ground improvement techniques will be implemented along the alignment of Retaining Wall 4W8, which is in close proximity to this wall and would present the most economical method for stabilizing the soil along this wall. As this is a proprietary design, the analysis for this wall considers the existing fill material will remain in place. MSE wall foundations bearing on existing fill material may be proportioned for a nominal bearing resistance as indicated in Table 5. A geotechnical resistance factor of $\varphi_b$=0.65 was considered in calculating the factored nominal bearing resistance at the strength limit state.

**Table 5. FRA-70 MSE Retaining Wall 4W20 Design Parameters**

| Structure Reference | Wall Height Analyzed (feet) | Backslope Behind Wall | Minimum Required Reinforcement Length [1] (feet) | Bearing Resistance at Strength Limit (ksf) | | Strength Limit Equivalent Bearing Pressure [3] (ksf) |
|---|---|---|---|---|---|---|
| | | | | Nominal | Factored [2] | |
| Retaining Wall 4W20 (B-016-1-09 / B-016-6-13) | 18.8 | Level | 13.2 (0.7H) | 6.06 | 3.95 | 4.59 |

1. *The required foundation width is expressed as a percentage of the wall height, H.*
2. *A geotechnical resistance factor of $\varphi_b$=0.65 was considered in calculating the factored bearing resistance at the strength limit state.*
3. *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 soils for the maximum specified wall heights indicated in Table 5. Based on the minimum length of reinforced soil mass presented, the factored equivalent bearing pressure exerted below the wall **will exceed** the factored bearing resistance at the strength limit state for Wall 4W20, considering the wall will bear on the existing fill material.

### *5.1.3 Settlement Evaluation*

The compressibility parameters utilized in the settlement analysis of the proposed MSE walls are provided in Table 6.

**Table 6. Compressibility Parameters Utilized in Settlement Analysis**

| Material Type | $\gamma$ (pcf) | *LL* (%) | $C_c$ (1) | $C_r$ (2) | $e_o$ (3) | $C_v$ (4) (ft$^2$/yr) | $N_{60}$ | *C'* (5) |
|---|---|---|---|---|---|---|---|---|
| Item 203 Granular Embankment | 130 | N/A | N/A | N/A | N/A | N/A | 30 | 161 to 215 |
| Existing Fill: Stiff Silt and Clay (ODOT A-6a) | 115 | 33 | 0.207 | 0.031 | 0.530 | 600 | N/A | N/A |
| Medium Dense to Very Dense Granular Soils (ODOT A-1-a, A-1-b, A-3a) | 125 to 135 | N/A | N/A | N/A | N/A | N/A | 22 to 85 | 76 to 340 |
| Stiff to Very Stiff Sandy Silt (ODOT A-4a) | 120 | 24 | 0.126 | 0.013 | 0.460 | 800 | N/A | N/A |
| Very Stiff to Hard Silty Clay (ODOT A-6b) | 130 | 35 | 0.225 | 0.022 | 0.569 | 400 | N/A | N/A |

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

Results of the settlement analysis are tabulated in Table 7. Total settlements of 4.0 to 4.3 inches at the center of the reinforced mass and 3.2 to 3.3 inches at the facing of the wall are anticipated. Based on the results of the analysis, 90 percent of the total settlement is anticipated to occur over a period of approximately 0 to 7 days. 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. FRA-70-1358A MSE Wall Settlement Results**

| Structure Reference | Wall Height Analyzed (feet) | Backslope Behind Wall in Analysis | Service Limit Equivalent Bearing Pressure [1] (ksf) | Total Settlement Values (inches) | | Time for 90% Consolidation (Days) |
|---|---|---|---|---|---|---|
| | | | | Center of Wall Mass | Facing of Wall | |
| MSE Retaining Wall 4W20 (B-016-1-09 / B-016-6-13) | 18.8 | Level | 3.19 | 4.011 to 4.304 | 3.200 to 3.339 | 0 to 7 |

1. *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 wall, the amount of settlement anticipated at the facing along Wall 4W20, as well as the presence of existing fill material that may vary significantly over the footprint of the wall, differential settlement greater than 1/100 may occur if the fill material is not stabilized or over excavated and replaced with embankment fill.

If either the total or differential settlement predicted presents 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 wall. 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 wall, 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 maximum specified wall heights 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 was utilized for design of the MSE wall. A geotechnical resistance factor of $\varphi_{\tau}$=1.0 was considered in calculating the factored shear resistance between the reinforced backfill material and foundation 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 **will not exceed** the factored shear resistance at the strength limit state under drained conditions for MSE wall.

### *5.1.6 Overall (Global) Stability*

A slope stability analysis was performed to check the global stability of the MSE 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 6.0 manufactured by Rocscience Inc. was utilized to perform the analyses.

Per Section 11.6.2.3 of the 2018 AASHTO LRFD BDS, for MSE walls that are not integrated with or supporting structural foundations or elements, global stability is satisfied if the product of the factor of safety from the slope stability output multiplied by the resistance factor $\varphi$=0.75 is greater than 1.0. Therefore, global stability for the portions of the wall that are adjacent to the abutment substructure is satisfied when a minimum factor of safety of 1.3 is obtained. For an MSE wall designed with the minimum strap lengths listed in Table 5, the resulting factor of safety under drained conditions (long-term stability) using the Spencer's analysis method 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, bearing stability under drained conditions was not satisfied at a strap length equal to 0.7 times the wall height. Increasing the width of the wall up to 80 percent of wall height may satisfy all of the external and global stability requirements. However, this would introduce a significant risk due to the potential for excessive settlement with time if the organic matter and wood within the fill material decompose.

As noted in Section 5.1, consideration was given to over excavating these soils and replacing it with granular embankment; however, given the depth of undercut and proximity to the I-70 roadway and adjacent electrical substation, this may be a very expensive and uneconomical option. Recommendations have been provided in the structure foundation exploration report for Retaining Wall 4W8 to incorporate the use of ground improvement techniques to stabilize the existing fill and underlying cohesive soils that were encountered along that wall, which is in close proximity to this wall location. The recommendations for this alternative should govern the design of this portion of the wall as well.

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 Lateral Earth Pressure

For the soil types encountered in the borings, the “in-situ” unit weight ($\gamma$), cohesion ($c$), effective angle of friction ($\varphi$’), 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 8 and Table 9.

**Table 8. Estimated Undrained (Short-term) Soil Parameters for Design**

| Soil Type | $\gamma$ (pcf) [1] | $c$ (psf) | $\varphi$ | $k_a$ | $k_o$ | $k_p$ |
|---|---|---|---|---|---|---|
| Existing Fill | 115 | 0 | 0° | N/A | N/A | N/A |
| Soft to Stiff Cohesive Soil | 115 | 1,500 | 0° | N/A | N/A | N/A |
| Very Stiff to Hard Cohesive Soil | 125 | 3,000 | 0° | N/A | N/A | N/A |
| Loose Granular Soil | 120 | 0 | 28° | 0.32 | 0.53 | 5.07 |
| Medium Dense Granular Soil | 125 | 0 | 32° | 0.27 | 0.47 | 6.82 |
| Dense to Very Dense Granular Soil | 130 | 0 | 36° | 0.23 | 0.41 | 9.09 |
| Compacted Cohesive Engineered Fill | 120 | 2,000 | 0° | N/A | N/A | N/A |
| Compacted Granular Engineered Fill | 130 | 0 | 33° | 0.30 | 0.46 | 3.39 |

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

**Table 9. Estimated Drained (Long-term) Soil Parameters for Design**

| Soil Type | $\gamma$ (pcf) [1] | $c$ (psf) | $\varphi'$ | $k_a$ | $k_o$ | $k_p$ |
|---|---|---|---|---|---|---|
| Existing Fill | 115 | 0 | 26° | 0.35 | 0.56 | 4.53 |
| Soft to Stiff Cohesive Soil | 115 | 0 | 26° | 0.35 | 0.56 | 4.53 |
| Very Stiff to Hard Cohesive Soil | 125 | 50 | 28° | 0.32 | 0.53 | 5.07 |
| 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 | 130 | 0 | 33° | 0.26 | 0.46 | 7.41 |

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

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

## 5.3 Construction Considerations

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

### *5.3.1 Excavation Considerations*

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

**Table 10. Excavation Back Slopes**

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

### *5.3.2 Groundwater Considerations*

Based on the groundwater observations made during drilling, groundwater is not anticipated to be encountered during construction. 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.

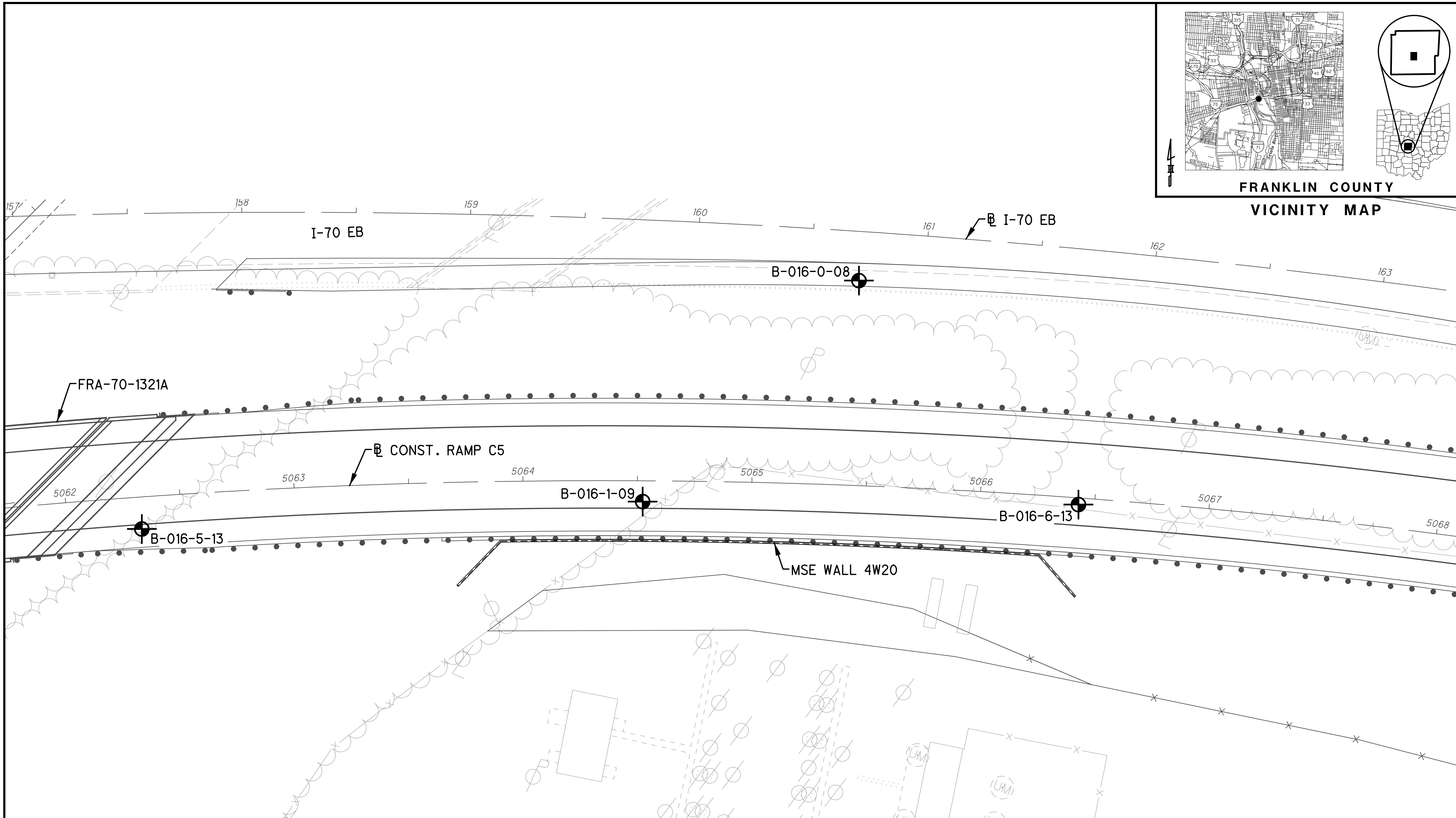
The conclusions and recommendations herein have been based upon the available soil and bedrock information and the design details furnished by a representative of the owner of the proposed project. Any revision in the plans for the proposed construction from those anticipated in this report should be 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-70-12.68 MSE WALL 4W20
## FRANKLIN COUNTY, OHIO

RII PROJECT NO.
W-13-045

SCALE: 1"=40'
0 20 40

DRAWN
RRM

REVIEWED
BRT

DATE
7-12-18

RESOURCE
INTERNATIONAL, INC.

# APPENDIX II

# DESCRIPTION OF SOIL TERMS

## DESCRIPTION OF SOIL TERMS

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

# CLASSIFICATION OF SOILS

Ohio Department of Transportation

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

| SYMBOL | DESCRIPTION | Classifcation AASHTO | Classifcation OHIO | $LL_O/LL \times 100$* | % Pass #40 | % Pass #200 | Liquid Limit (LL) | Plastic Index (PI) | Group Index Max. | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|
| | Gravel and/or Stone Fragments | A-1-a | 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 | A-1-b | | 50 Max. | 25 Max. | | 6 Max. | 0 | |
| F S | Fine Sand | A-3 | A-3 | | 51 Min. | 10 Max. | NON-PLASTIC | 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 | A-2-4 | | | 35 Max. | 40 Max. | 10 Max. | 0 | |
| | | A-2-5 | A-2-5 | | | | 41 Min. | | | |
| | Gravel and/or Stone Fragments with Sand, Silt and Clay | A-2-6 | A-2-6 | | | 35 Max. | 40 Max. | 11 Min. | 4 | |
| | | A-2-7 | 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 | 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 | A-7-5 | 76 Min. | | 36 Min. | 41 Min. | ≦LL-30 | 20 | |
| | Clay | A-7-6 | 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)

Boulder Zone

P

Peat

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

# APPENDIX III

# BORING LOG:

# B-016-1-09 and B-016-6-13

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

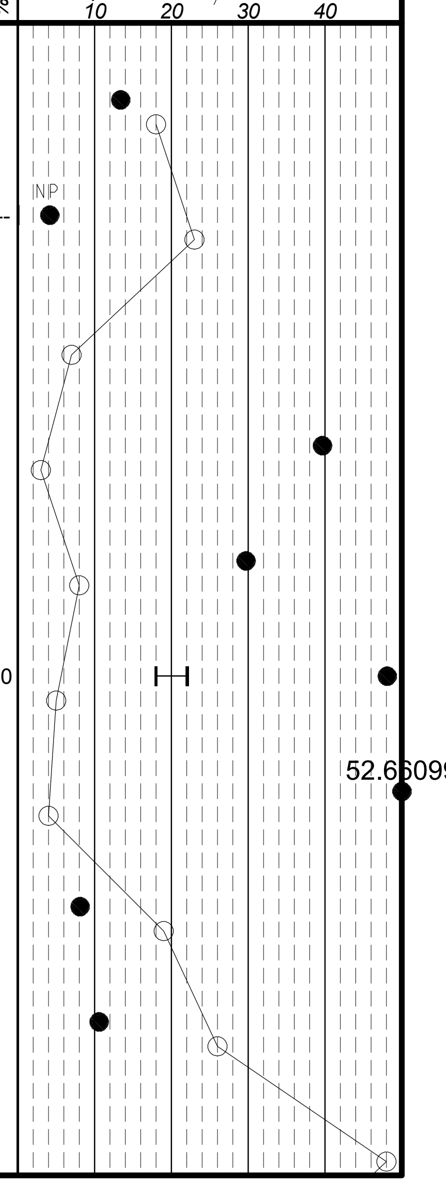
**LOG OF: Boring B-016-1-09** | Location: Sta. 5064+52.34, 9.1' RT., BL RAMP C5 | Date Drilled: 9/24/2009

WATER OBSERVATIONS:
Water seepage at: 21.0'
Water level at completion: 34.8'

FIELD NOTES:

| Depth (ft) | Elev. (ft) 717.2 | Blows per 6" | Recovery | Sample No. Drive | Sample No. Press / Core | Hand Penetrometer (tsf) | DESCRIPTION | Graphic Log | % Aggregate | % C. Sand | % M. Sand | % F. Sand | % Silt | % Clay |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.3 | 716.9 | | | | | | Topsoil - 3" | | | | | | | |
| | | 4 6 11 | 8 | 1 | | 2.0 | FILL: Stiff brown SILT AND CLAY (A-6a), little to some fine to coarse sand, trace to little fine to coarse gravel; contains few brick fragments; damp. | | | | | | | |
| 3.5 | 713.7 | 2 8 14 | 3 | 2 | | | FILL: Loose to medium dense brown GRAVEL (A-1-a), little to some fine to coarse sand, trace to little silt; damp. | | 64 | 18 | --- | 7 | --11-- | |
| 6.0 | 711.2 | 2 4 3 | 2 | 3 | | | FILL: Brick Fragments | | | | | | | |
| 8.5 | 708.7 | 1 2 1 | 6 | 4 | | | FILL: Very loose to loose brown COARSE AND FINE SAND (A-3a), little silt, little gravel; contains brick fragments and wood pieces; damp to moist. | | | | | | | |
| | | 2 4 4 | 2 | 5 | | | | | | | | | | |
| 13.5 | 703.7 | 2 2 3 | 4 | 6 | | | FILL: Loose brown GRAVEL WITH SAND AND SILT (A-2-4), trace to little clay; contains brick fragments and wood pieces; wet. | | 33 | 23 | --- | 17 | 17 | 10 |
| | | 2 2 2 | 2 | 7 | | | | | | | | | | |
| 18.5 | 698.7 | 8 10 8 | 4 | 8 | | | Medium dense to dense brown GRAVEL (A-1-b), some fine to coarse sand, trace to little silt; wet. | | | | | | | |
| | | 10 10 15 | 5 | 9 | | | | | | | | | | |
| 25 | 692.2 | 27 25 21 | 10 | 10 | | | | | | | | | | |

STANDARD PENETRATION (N60)
Natural Moisture Content, % - ●
PL ⊢—⊣ LL
Blows per foot - ○ / Non-Plastic - NP

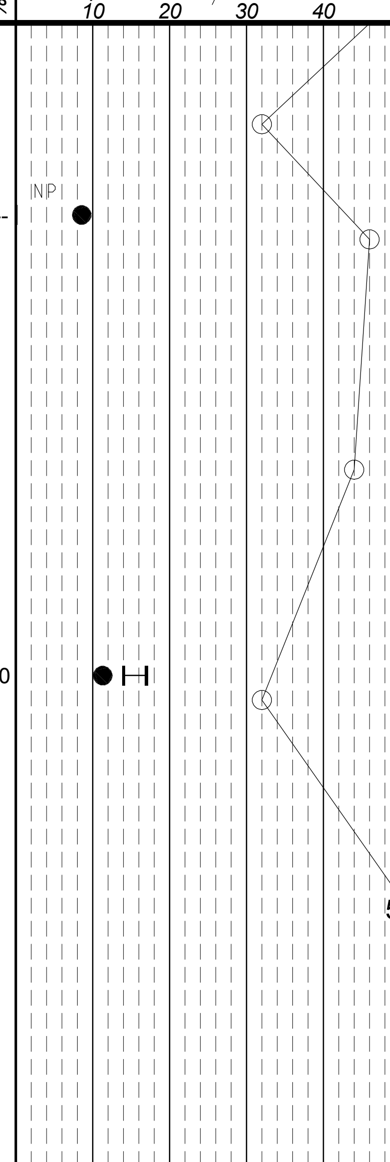


| Client: ms consultants | Project: FRA-70-8.93 | Job No. 0221-1004.01 |
|---|---|---|
| LOG OF: Boring B-016-1-09 | Location: Sta. 5064+52.34, 9.1' RT., BL RAMP C5 | Date Drilled: 9/24/2009 |

WATER OBSERVATIONS:
Water seepage at: 21.0'
Water level at completion: 34.8'
FIELD NOTES:

| Depth (ft) | Elev. (ft) 692.2 | Blows per 6" | Recovery | Sample No. Drive | Sample No. Press / Core | Hand Penetrometer (tsf) | DESCRIPTION | % Aggregate | % C. Sand | % M. Sand | % F. Sand | % Silt | % Clay |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Medium dense to very dense brown GRAVEL (A-1-b), some fine to coarse sand, trace to little silt; wet. | | | | | | |
| | | 20 18 13 | 10 | 11 | | | | | | | | | |
| 30 | | 32 31 14 | 12 | 12 | | | | 38 | 29 | --- | 16 | --17-- | |
| 35 | | 36 25 18 | 3 | 13 | | | | | | | | | |
| 37.0 | 680.2 | | | | | | Hard gray SILT (A-4b), some fine to coarse sand, trace to little clay, trace gravel; damp. | | | | | | |
| 40 | | 8 15 16 | 8 | 14 | | 4.5+ | | 7 | 8 | --- | 22 | 53 | 10 |
| 45.0 (45) | 672.2 | 15 23 28 | 14 | 15 | | 4.5+ | @ 43.5'-45.0', little to some gravel. | | | | | | |
| | | | | | | | Bottom of Boring - 45.0' | | | | | | |
| 50 | | | | | | | | | | | | | |

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



RESOURCE INTERNATIONAL, INC.

| PROJECT: FRA-70-12.68 - PHASE 4A | DRILLING FIRM / OPERATOR: RII / S.M. | DRILL RIG: CME-750 (SN 98048) | STATION / OFFSET: 5066+43.00 / 2.9' RT | EXPLORATION ID B-016-6-13 |
|---|---|---|---|---|
| TYPE: STRUCTURE | SAMPLING FIRM / LOGGER: RII / C.H. | HAMMER: CME AUTOMATIC | ALIGNMENT: BL RAMP C5 | |
| PID: 77372 BR ID: FRA-70-1321A | DRILLING METHOD: 3.25" HSA | CALIBRATION DATE: 4/26/13 | ELEVATION: 717.0 (MSL) EOB: 50.0 ft. | PAGE 1 OF 2 |
| START: 7/18/13 END: 7/18/13 | SAMPLING METHOD: SPT | ENERGY RATIO (%): 82.6 | LAT / LONG: 39.952535194, -83.009477109 | |

| MATERIAL DESCRIPTION AND NOTES | ELEV. 717.0 | DEPTHS | SPT/ RQD | $N_{60}$ | REC (%) | SAMPLE ID | HP (tsf) | GR | CS | FS | SI | CL | LL | PL | PI | WC | ODOT CLASS (GI) | BACK FILL |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **FILL**: HARD, DARK BROWN **SANDY SILT**, SOME FINE GRAVEL, DRY. -STONE FRAGMENTS PRESENT IN SS-1 | 714.0 | 1–3 | 8 13 28 | 56 | 22 | SS-1 | - | - | - | - | - | - | - | - | - | 7 | A-4a (V) | |
| **FILL**: MEDIUM DENSE, BLACK **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, WET. -CINDERS PRESENT IN SS-2 | 711.5 | 4–5 | 8 8 4 | 17 | 44 | SS-2 | - | - | - | - | - | - | - | - | - | 24 | A-1-b (V) | |
| **FILL**: STIFF, BROWN **SANDY SILT**, SOME FINE GRAVEL, LITTLE CLAY, DAMP. -CINDERS AND ROOT FIBERS PRESENT IN SS-3 | 709.0 | 6–8 | 3 5 6 | 15 | 67 | SS-3 | - | 24 | 18 | 17 | 28 | 13 | 27 | 19 | 8 | 13 | A-4a (1) | |
| VERY LOOSE TO LOOSE, BROWN **SILT**, LITTLE COARSE TO FINE SAND, LITTLE CLAY, TRACE FINE GRAVEL, WET. | | 9–10 | 3 1 1 | 3 | 28 | SS-4 | - | - | - | - | - | - | - | - | - | 33 | A-4b (V) | |
| | 704.0 | 11–13 | 3 2 3 | 7 | 44 | SS-5 | - | - | - | - | - | - | - | - | - | 26 | A-4b (V) | |
| MEDIUM STIFF TO STIFF, REDDISH BROWN **SILTY CLAY**, SOME COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST. | | 14–15 | 3 4 4 | 11 | 83 | SS-6 | 1.50 | 1 | 5 | 28 | 43 | 23 | 33 | 17 | 16 | 19 | A-6b (9) | |
| | 699.0 | 16–18 | 3 4 5 | 12 | 61 | SS-7 | 1.00 | - | - | - | - | - | - | - | - | 21 | A-6b (V) | |
| MEDIUM DENSE, BROWN **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST. | 695.0 | 19–22 | 10 10 11 | 29 | 56 | SS-8 | - | - | - | - | - | - | - | - | - | 12 | A-1-b (V) | |
| VERY DENSE, BROWN TO GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST. -HEAVING SAND ENCOUNTERED @ 23.5' | | 23–25 (W) | 11 27 23 | 69 | 67 | SS-9 | - | 54 | 22 | 12 | 10 | 2 | 17 | 17 | NP | 9 | A-1-a (0) | |
| -COBBLES PRESENT @ 26.0' | | 26–29 | 11 27 22 | 67 | 39 | SS-10 | 4.50 | - | - | - | - | - | - | - | - | 9 | A-1-a (V) | |

2014 ODOT BORING LOG-RII NE BRIDGE ID - OH DOT.GDT - 3/14/15 17:34 - U:\GI8\PROJECTS\2013\W-13-045.GPJ

| PID: 77372 | BR ID: FRA-70-1321A | PROJECT: FRA-70-12.68 - PHASE 4A | STATION / OFFSET: 5066+43.00 / 2.9 RT | START: 7/18/13 | END: 7/18/13 | PG 2 OF 2 | B-016-6-13 |
|---|---|---|---|---|---|---|---|

| MATERIAL DESCRIPTION AND NOTES | ELEV. 687.0 | DEPTHS | SPT/ RQD | $N_{60}$ | REC (%) | SAMPLE ID | HP (tsf) | GR | CS | FS | SI | CL | LL | PL | PI | WC | ODOT CLASS (GI) | BACK FILL |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | GRADATION (%) | | | | | ATTERBERG | | | | | |
| VERY DENSE, BROWN TO GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST. *(same as above)* | 685.0 | 31 – 32 | | | | | | | | | | | | | | | | |
| HARD, GRAY **SANDY SILT**, LITTLE CLAY, LITTLE FINE GRAVEL, DAMP. | | 34 – 35 | 19 31 34 | 89 | 39 | SS-11 | 4.50 | 17 | 15 | 21 | 28 | 19 | 22 | 13 | 9 | 10 | A-4a (2) | |
| | 680.0 | 36 – 37 | | | | | | | | | | | | | | | | |
| HARD, GRAY **SILTY CLAY**, TRACE SAND, TRACE FINE GRAVEL, DAMP TO MOIST. | | 39 – 40 | 6 9 13 | 30 | 39 | SS-12 | 4.50 | - | - | - | - | - | - | - | - | 24 | A-6b (V) | |
| | | 44 – 45 | 18 23 28 | 70 | 44 | SS-13 | 4.5+ | 1 | 1 | 1 | 31 | 66 | 38 | 19 | 19 | 19 | A-6b (12) | |
| | 667.0 | 49 – 50 EOB | 20 31 35 | 91 | 50 | SS-14 | 4.5+ | - | - | - | - | - | - | - | - | 16 | A-6b (V) | |

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 23.5' AND AT COMPLETION @ 24.0'; CAVE-IN DEPTH @ 32.0'

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

# APPENDIX IV

# MSE WALL CALCULATIONS

RESOURCE INTERNATIONAL, INC.
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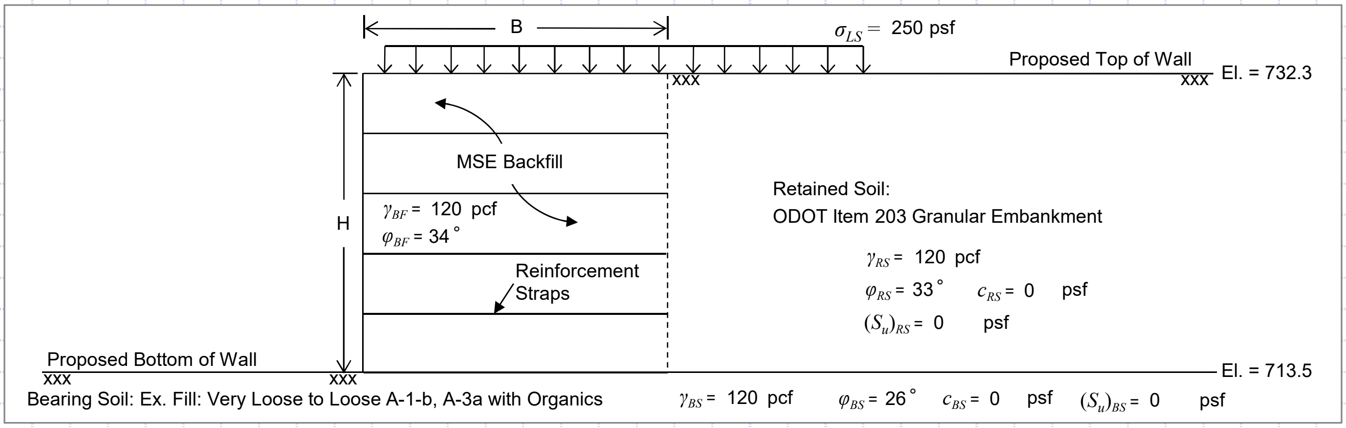
## FRA-70-12.68 - MSE Wall W20 - B-016-1-09 and B-016-6-13 - 18.5 ft. Wall Height




### MSE Wall Dimensions and Retained Soil Parameters

| | | |
|---|---|---|
| MSE Wall Height, ($H$) = | 18.8 | ft |
| MSE Wall Width (Reinforcement Length), ($B$) = | 13.2 | ft |
| MSE Wall Length, ($L$) = | 236 | ft |
| Live Surcharge Load, ($\sigma_{LS}$) = | 250 | psf |
| Retained Soil Unit Weight, ($\gamma_{RS}$) = | 120 | pcf |
| Retained Soil Friction Angle, ($\varphi_{RS}$) = | 33 | ° |
| Retained Soil Drained Cohesion [1], ($c_{BS}$) = | 0 | psf |
| Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] = | 0 | psf |
| Retained Soil Active Earth Pressure Coeff., ($K_a$) = | 0.264 | |
| MSE Backfill Unit Weight, ($\gamma_{BF}$) = | 120 | pcf |
| MSE Backfill Friction Angle, ($\varphi_{BF}$) = | 34 | ° |

### Bearing Soil Properties:

| | | |
|---|---|---|
| Bearing Soil Unit Weight, ($\gamma_{BS}$) = | 120 | pcf |
| Bearing Soil Friction Angle, ($\varphi_{BS}$) = | 26 | ° |
| Bearing Soil Drained Cohesion, ($c_{BS}$) = | 0 | psf |
| Bearing Soil Undrained Shear Strength, [$(S_u)_{BS}$] = | 0 | psf |
| Embedment Depth, ($D_f$) = | 3.0 | ft |
| Depth to Grounwater (Below Bot. of Wall), ($D_W$) = | 0.0 | ft |

### LRFD Load Factors

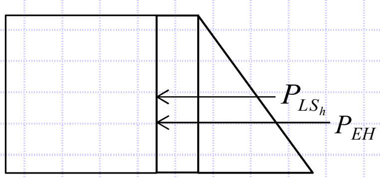
| | EV | EH | LS |
|---|---|---|---|
| Strength Ia | 1.00 | 1.50 | 1.75 |
| Strength Ib | 1.35 | 1.50 | 1.75 |
| Service I | 1.00 | 1.00 | 1.00 |

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

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

Sliding Force:

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




$$P_{EH} = \tfrac{1}{2}\gamma_{RS} H^2 K_a \gamma_{EH} = \text{½(120 pcf)(18.8 ft)²(0.264)(1.5)} = 8.4 \text{ kip/ft}$$

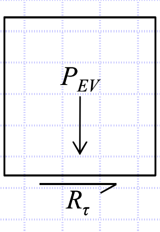
$$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS} = \text{(250 psf)(18.8 ft)(0.264)(1.75)} = 2.17 \text{ kip/ft}$$

$$P_H = \text{8.4 kip/ft + 2.17 kip/ft} = 10.57 \text{ kip/ft}$$

### Check Sliding Resistance - Drained Condition

Nominal Sliding Resistance:

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




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

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

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

$$R_\tau = \text{(29.78 kip/ft)(0.49)} = 14.59 \text{ kip/ft}$$

### Verify Sliding Force Less Than Factored Sliding Resistance - Drained Condition

$$P_H \le R_\tau \cdot \phi_\tau \longrightarrow \text{10.57 kip/ft} \le \text{(14.59 kip/ft)(1.0) = 14.59 kip/ft} \longrightarrow \text{10.57 kip/ft} \le \text{14.59 kip/ft}$$

**OK**

Use $\varphi_\tau$ = **1.0** *(Per AASHTO LRFD BDM Table 11.5.7-1)*

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## MSE Wall Dimensions and Retained Soil Parameters

| | | |
|---|---|---|
| MSE Wall Height, $(H)$ = | 18.8 | ft |
| MSE Wall Width (Reinforcement Length), $(B)$ = | 13.2 | ft |
| MSE Wall Length, $(L)$ = | 236 | ft |
| Live Surcharge Load, $(\sigma_{LS})$ = | 250 | psf |
| Retained Soil Unit Weight, $(\gamma_{RS})$ = | 120 | pcf |
| Retained Soil Friction Angle, $(\varphi_{RS})$ = | 33 | ° |
| Retained Soil Drained Cohesion, $(c_{BS})$ = | 0 | psf |
| Retained Soil Undrained Shear Strength, $[(S_u)_{RS}]$ = | 0 | psf |
| Retained Soil Active Earth Pressure Coeff., $(K_a)$ = | 0.264 | |
| MSE Backfill Unit Weight, $(\gamma_{BF})$ = | 120 | pcf |
| MSE Backfill Friction Angle, $(\varphi_{BF})$ = | 34 | ° |

## Bearing Soil Properties:

| | | |
|---|---|---|
| Bearing Soil Unit Weight, $(\gamma_{BS})$ = | 120 | pcf |
| Bearing Soil Friction Angle, $(\varphi_{BS})$ = | 26 | ° |
| Bearing Soil Drained Cohesion, $(c_{BS})$ = | 0 | psf |
| Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ = | 0 | psf |
| Embedment Depth, $(D_f)$ = | 3.0 | ft |
| Depth to Grounwater (Below Bot. of Wall), $(D_W)$ = | 0.0 | ft |

## LRFD Load Factors

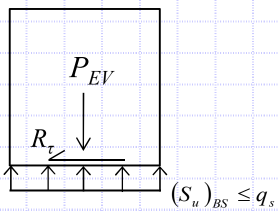
| | EV | EH | LS |
|---|---|---|---|
| Strength Ia | 1.00 | 1.50 | 1.75 |
| Strength Ib | 1.35 | 1.50 | 1.75 |
| Service I | 1.00 | 1.00 | 1.00 |

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

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

## Check Sliding Resistance - Undrained Condition

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




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

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

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

$R_\tau$ = (N/A ksf ≤ 1.13 ksf)(13.2 ft) = N/A kip/ft

## Verify Sliding Force Less Than Factored Sliding Resistance - Undrained Condition

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

Use $\varphi_\tau$ = **1.0** *(Per AASHTO LRFD BDM Table 11.5.7-1)*

**MSE Wall Dimensions and Retained Soil Parameters**

| | | |
|---|---|---|
| MSE Wall Height, $(H)$ = | 18.8 | ft |
| MSE Wall Width (Reinforcement Length), $(B)$ = | 13.2 | ft |
| MSE Wall Length, $(L)$ = | 236 | ft |
| Live Surcharge Load, $(\sigma_{LS})$ = | 250 | psf |
| Retained Soil Unit Weight, $(\gamma_{RS})$ = | 120 | pcf |
| Retained Soil Friction Angle, $(\varphi_{RS})$ = | 33 | ° |
| Retained Soil Drained Cohesion, $(c_{BS})$ = | 0 | psf |
| Retained Soil Undrained Shear Strength, $[(S_u)_{RS}]$ = | 0 | psf |
| Retained Soil Active Earth Pressure Coeff., $(K_a)$ = | 0.264 | |
| MSE Backfill Unit Weight, $(\gamma_{BF})$ = | 120 | pcf |
| MSE Backfill Friction Angle, $(\varphi_{BF})$ = | 34 | ° |

**Bearing Soil Properties:**

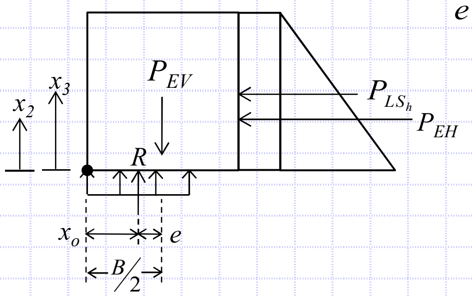
| | | |
|---|---|---|
| Bearing Soil Unit Weight, $(\gamma_{BS})$ = | 120 | pcf |
| Bearing Soil Friction Angle, $(\varphi_{BS})$ = | 26 | ° |
| Bearing Soil Drained Cohesion, $(c_{BS})$ = | 0 | psf |
| Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ = | 0 | psf |
| Embedment Depth, $(D_f)$ = | 3.0 | ft |
| Depth to Grounwater (Below Bot. of Wall), $(D_W)$ = | 0.0 | ft |

**LRFD Load Factors**

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

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

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



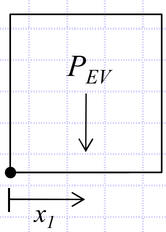

$$e = B/2 - x_o$$

$$x_o = \frac{M_{EV} - M_H}{P_{EV}}$$ = (196.55 kip·ft/ft - 73.07 kip·ft/ft) / (29.78 kip/ft) = 4.15 ft

$M_{EV}$ = 196.55 kip·ft/ft
$M_H$ = 73.07 kip·ft/ft — Defined below
$P_{EV}$ = 29.78 kip/ft

$e =$ (13.2 ft)/2 - 4.15 ft = 2.45 ft

Resisting Moment, $M_{EV}$:



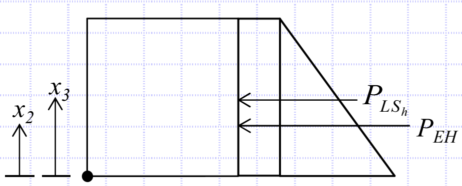

$$M_{EV} = P_{EV}(x_1)$$

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

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

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

Overturning Moment, $M_H$:




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

$P_{EH} = ½\gamma_{RS} H^2 K_a \gamma_{EH}$ = ½(120 pcf)(18.8 ft)²(0.264)(1.5) = 8.40 kip/ft

$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS}$ = (250 psf)(18.8 ft)(0.264)(1.75) = 2.17 kip/ft

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

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

$M_H =$ (8.4 kip/ft)(6.27 ft) + (2.17 kip/ft)(9.40 ft) = 73.07 kip·ft/ft

**Check Eccentricity**

$e < e_{max} \longrightarrow$ 2.45 ft < 4.40 ft **OK**

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

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## MSE Wall Dimensions and Retained Soil Parameters

| | | |
|---|---|---|
| MSE Wall Height, ($H$) = | 18.8 | ft |
| MSE Wall Width (Reinforcement Length), ($B$) = | 13.2 | ft |
| MSE Wall Length, ($L$) = | 236 | ft |
| Live Surcharge Load, ($\sigma_{LS}$) = | 250 | psf |
| Retained Soil Unit Weight, ($\gamma_{RS}$) = | 120 | pcf |
| Retained Soil Friction Angle, ($\varphi_{RS}$) = | 33 | ° |
| Retained Soil Drained Cohesion, ($c_{BS}$) = | 0 | psf |
| Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] = | 0 | psf |
| Retained Soil Active Earth Pressure Coeff., ($K_a$) = | 0.264 | |
| MSE Backfill Unit Weight, ($\gamma_{BF}$) = | 120 | pcf |
| MSE Backfill Friction Angle, ($\varphi_{BF}$) = | 34 | ° |

## Bearing Soil Properties:

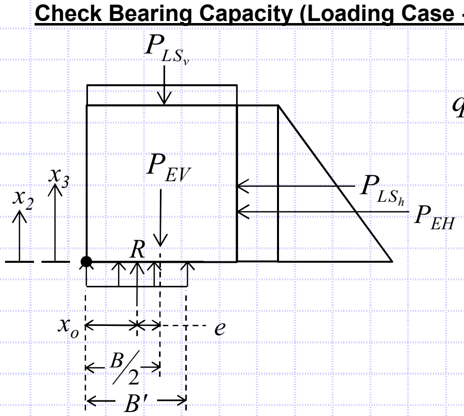
| | | |
|---|---|---|
| Bearing Soil Unit Weight, ($\gamma_{BS}$) = | 120 | pcf |
| Bearing Soil Friction Angle, ($\varphi_{BS}$) = | 26 | ° |
| Bearing Soil Drained Cohesion, ($c_{BS}$) = | 0 | psf |
| Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] = | 0 | psf |
| Embedment Depth, ($D_f$) = | 3.0 | ft |
| Depth to Grounwater (Below Bot. of Wall), ($D_W$) = | 0.0 | ft |

## LRFD Load Factors

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

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

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




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

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

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

$x_o = \dfrac{M_V - M_H}{P_V}$ = (303.45 kip·ft/ft - 73.06 kip·ft/ft) / 45.98 kip/ft = 5.01 ft

$q_{eq}$ = (45.98 kip/ft) / (10.02 ft) = 4.59 ksf

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

$M_V$ = [(120 pcf)(18.8 ft)(13.2 ft)(1.35)](6.6 ft) + [(250 psf)(13.2 ft)(1.75)](6.6 ft) = 303.45 kip·ft/ft

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

$M_H$ = [½(120 pcf)(18.8 ft)²(0.264)(1.5)](6.27 ft) + [(250 psf)(18.8 ft)(0.264)(1.75)](9.4 ft) = 73.06 kip·ft/ft

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

$P_V$ = (120 pcf)(18.8 ft)(13.2 ft)(1.35) + (250 psf)(13.2 ft)(1.75) = 45.98 kip/ft

## Check Bearing Resistance - Drained Condition

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

$N_{cm} = N_c s_c i_c$ = 22.76

$N_c$ = 22.25
$s_c$ = 1+(10.02 ft/235.6 ft)(11.85/22.25)
= 1.023
$i_c$ = 1.000 (Assumed)

$N_{qm} = N_q s_q d_q i_q$ = 13.19

$N_q$ = 11.85
$s_q$ = 1.021
$d_q$ = 1+2tan(26°)[1-sin(26°)]²tan⁻¹(3.0 ft/10.02 ft)
1.090
$i_q$ = 1.000 (Assumed)
$C_{wq}$ = 0.0 ft > 3.0 ft = 0.500

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 12.33

$N_\gamma$ = 12.54
$s_\gamma$ = 0.983
$i_\gamma$ = 1.000 (Assumed)
$C_{w\gamma}$ = 0.0 ft < 1.5(10.02 ft) + 3.0 ft = 0.500

$q_n$ = (0 psf)(22.762) + (120 pcf)(3.0 ft)(13.188)(0.500) + ½(120 pcf)(10.0 ft)(12.327)(0.500) = 6.08 ksf

## Verify Equivalent Pressure Less Than Factored Bearing Resistance

Use $\varphi_b$ = **0.65** *(Per AASHTO LRFD BDM Table 11.5.7-1)*

$q_{eq} \le q_n \cdot \phi_b$ ⟶ 4.59 ksf ≤ (6.08 ksf)(0.65) = 3.95 ksf ⟶ 4.59 ksf ≤ 3.95 ksf **ERROR!!**

**MSE Wall Dimensions and Retained Soil Parameters**

| | | |
|---|---|---|
| MSE Wall Height, ($H$) = | 18.8 | ft |
| MSE Wall Width (Reinforcement Length), ($B$) = | 13.2 | ft |
| MSE Wall Length, ($L$) = | 236 | ft |
| Live Surcharge Load, ($\sigma_{LS}$) = | 250 | psf |
| Retained Soil Unit Weight, ($\gamma_{RS}$) = | 120 | pcf |
| Retained Soil Friction Angle, ($\varphi_{RS}$) = | 33 | ° |
| Retained Soil Drained Cohesion, ($c_{BS}$) = | 0 | psf |
| Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] = | 0 | psf |
| Retained Soil Active Earth Pressure Coeff., ($K_a$) = | 0.264 | |
| MSE Backfill Unit Weight, ($\gamma_{BF}$) = | 120 | pcf |
| MSE Backfill Friction Angle, ($\varphi_{BF}$) = | 34 | ° |

**Bearing Soil Properties:**

| | | |
|---|---|---|
| Bearing Soil Unit Weight, ($\gamma_{BS}$) = | 120 | pcf |
| Bearing Soil Friction Angle, ($\varphi_{BS}$) = | 26 | ° |
| Bearing Soil Drained Cohesion, ($c_{BS}$) = | 0 | psf |
| Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] = | 0 | psf |
| Embedment Depth, ($D_f$) = | 3.0 | ft |
| Depth to Grounwater (Below Bot. of Wall), ($D_W$) = | 0.0 | ft |

**LRFD Load Factors**

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

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

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

**Check Bearing Resistance - Undrained Condition**

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

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

$N_c$ = 5.140

$s_c$ = 1+(10.02 ft/[(5)(235.6 ft)]) = 1.009

$i_c$ = 1.000 (Assumed)

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

$N_q$ = 1.000

$s_q$ = 1.000

$d_q$ = 1+2tan(0°)[1-sin(0°)]²tan⁻¹(3.0 ft/10.02 ft)

1.000

$i_q$ = 1.000 (Assumed)

$C_{wq}$ = 0.0 ft > 3.0 ft = 0.500

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

$N_\gamma$ = 0.000

$s_\gamma$ = 1.000

$i_\gamma$ = 1.000 (Assumed)

$C_{w\gamma}$ = 0.0 ft < 1.5(10.02 ft) + 3.0 ft = 0.500

$q_n =$ (0 psf)(5.190) + (120 pcf)(3.0 ft)(1.000)(0.500) + ½(120 pcf)(10.0 ft)(0.000)(0.500) = N/A ksf

**Verify Equivalent Pressure Less Than Factored Bearing Resistance**

$q_{eq} \leq q_n \cdot \phi_b$ ⟶ 4.59 ksf ≤ (N/A ksf)(0.65) = N/A ksf ⟶ N/A

Use $\varphi_b$ = **0.65** *(Per AASHTO LRFD BDM Table 11.5.7-1)*

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## MSE Wall Dimensions and Retained Soil Parameters

| | | |
|---|---|---|
| MSE Wall Height, $(H)$ = | 18.8 | ft |
| MSE Wall Width (Reinforcement Length), $(B)$ = | 13.2 | ft |
| MSE Wall Length, $(L)$ = | 236 | ft |
| Live Surcharge Load, $(\sigma_{LS})$ = | 250 | psf |
| Retained Soil Unit Weight, $(\gamma_{RS})$ = | 120 | pcf |
| Retained Soil Friction Angle, $(\varphi_{RS})$ = | 33 | ° |
| Retained Soil Drained Cohesion, $(c_{BS})$ = | 0 | psf |
| Retained Soil Undrained Shear Strength, $[(S_u)_{RS}]$ = | 0 | psf |
| Retained Soil Active Earth Pressure Coeff., $(K_a)$ = | 0.264 | |
| MSE Backfill Unit Weight, $(\gamma_{BF})$ = | 120 | pcf |
| MSE Backfill Friction Angle, $(\varphi_{BF})$ = | 34 | ° |

## Bearing Soil Properties:

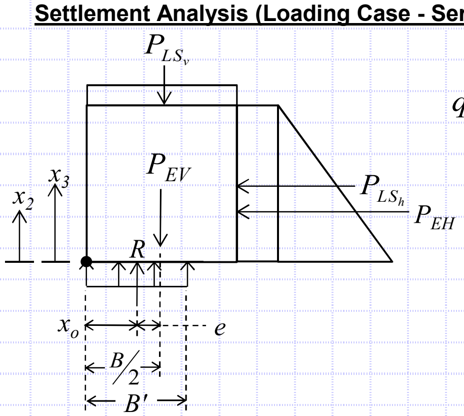
| | | |
|---|---|---|
| Bearing Soil Unit Weight, $(\gamma_{BS})$ = | 120 | pcf |
| Bearing Soil Friction Angle, $(\varphi_{BS})$ = | 26 | ° |
| Bearing Soil Drained Cohesion, $(c_{BS})$ = | 0 | psf |
| Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ = | 0 | psf |
| Embedment Depth, $(D_f)$ = | 3.0 | ft |
| Depth to Grounwater (Below Bot. of Wall), $(D_W)$ = | 0.0 | ft |

## LRFD Load Factors

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

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

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




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

$$B' = B - 2e \quad = \quad 13.2 \text{ ft} - 2(1.41 \text{ ft}) \quad = \quad 10.38 \quad \text{ft}$$

$$e = B/2 - x_o \quad = \quad (13.2 \text{ ft}) / 2 - 5.19 \text{ ft} \quad = \quad 1.41 \quad \text{ft}$$

$$x_o = \frac{M_V - M_H}{P_V} \quad = \quad (218.32 \text{ kip·ft/ft} - 46.77 \text{ kip·ft/ft}) / 33.08 \text{ kip/ft} \quad = \quad 5.19 \quad \text{ft}$$

$$q_{eq} = \quad (33.08 \text{ kip/ft}) / (10.38 \text{ ft}) \quad = \quad 3.19 \quad \text{ksf}$$

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

$$M_V = \quad [(120 \text{ pcf})(18.8 \text{ ft})(13.2 \text{ ft})(1.00)](6.6 \text{ ft}) + [(250 \text{ psf})(13.2 \text{ ft})(1.00)](6.6 \text{ ft}) \quad = \quad 218.32 \quad \text{kip·ft/ft}$$

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

$$M_H = \quad [\tfrac{1}{2}(120 \text{ pcf})(18.8 \text{ ft})^2(0.264)(1.00)](6.27 \text{ ft}) + [(250 \text{ psf})(18.8 \text{ ft})(0.264)(1.00)](9.4 \text{ ft}) \quad = \quad 46.77 \quad \text{kip·ft/ft}$$

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

$$P_V = \quad (120 \text{ pcf})(18.8 \text{ ft})(13.2 \text{ ft})(1.00) + (250 \text{ psf})(13.2 \text{ ft})(1.00) \quad = \quad 33.08 \quad \text{kip/ft}$$

## Settlement, Time Rate of Consolidation and Differential Settlement:

| Boring | Total Settlement at Center of Reinforced Soil Mass | Total Settlement at Wall Facing | Time for 90% Consolidation | Distance Between Borings Along Wall Facing | Differential Settlement Along Wall Facing |
|---|---|---|---|---|---|
| B-016-1-09 | 4.011 in | 3.339 in | 0 days | | |
| B-016-6-13 | 4.304 in | 3.200 in | 7 days | 190 ft | 1/16400 |
| | | | | | |
| | | | | | |

W-13-045 - FRA-70-12.86 - Retaining Wall 4W20
MSE Wall Settlement

Calculated By: HSK Date: 7/12/2018
Checked By: JPS Date: 7/13/2018

Boring B-016-1-09

| | | | |
|---|---|---|---|
| H= | 18.8 | ft | Total wall height |
| B'= | 10.4 | ft | Effective footing width due to eccentricity |
| $D_w$ = | 0.0 | ft | Depth below bottom of footing |
| $q_e$ = | 3,190 | 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) | $\gamma$ (pcf) | $\sigma_{vo}$ Bottom (psf) | $\sigma_{vo}$ Midpoint (psf) | $\sigma_{vo}'$ Midpoint (psf) | $\sigma_p'$ (1) (psf) | LL | $C_c$ (2) | $C_r$ (3) | $e_o$ (4) | $N_{60}$ | $(N1)_{60}$ (5) | C' (6) | $Z_f$/B | I (7) | $\Delta\sigma_v$ (8) (psf) | $\sigma_{vf}'$ Midpoint (psf) | $S_c$ (9,10) (ft) | $S_c$ (in) | I (7) | $\Delta\sigma_v$ (8) (psf) | $\sigma_{vf}'$ Midpoint (psf) | $S_c$ (9,10) (ft) | $S_c$ (in) |
| 1 | Fill | G | 0.0 | 2.5 | 2.5 | 1.3 | 115 | 288 | 144 | 66 | 2,066 | | | | | 8 | 16 | 34 | 0.12 | 0.994 | 3,172 | 3,238 | 0.125 | 1.501 | 0.500 | 1,594 | 1,660 | 0.104 | 1.243 |
| 2 | A-3a | G | 2.5 | 5.0 | 2.5 | 3.8 | 120 | 588 | 438 | 204 | 2,204 | | | | | 5 | 9 | 50 | 0.36 | 0.904 | 2,885 | 3,088 | 0.059 | 0.704 | 0.491 | 1,568 | 1,771 | 0.047 | 0.560 |
| | A-3a | G | 5.0 | 7.5 | 2.5 | 6.3 | 120 | 888 | 738 | 348 | 2,348 | | | | | 5 | 8 | 49 | 0.60 | 0.755 | 2,408 | 2,755 | 0.046 | 0.548 | 0.468 | 1,494 | 1,841 | 0.037 | 0.441 |
| 3 | A-2-4 | G | 7.5 | 10.0 | 2.5 | 8.8 | 120 | 1,188 | 1,038 | 492 | 2,492 | | | | | 4 | 6 | 51 | 0.84 | 0.621 | 1,981 | 2,472 | 0.034 | 0.409 | 0.434 | 1,385 | 1,876 | 0.028 | 0.339 |
| | A-2-4 | G | 10.0 | 12.5 | 2.5 | 11.3 | 120 | 1,488 | 1,338 | 636 | 2,636 | | | | | 4 | 6 | 51 | 1.08 | 0.518 | 1,653 | 2,288 | 0.027 | 0.327 | 0.396 | 1,264 | 1,899 | 0.023 | 0.279 |
| 4 | A-1-b | G | 12.5 | 15.0 | 2.5 | 13.8 | 130 | 1,813 | 1,650 | 792 | 4,792 | | | | | 21 | 28 | 92 | 1.32 | 0.441 | 1,406 | 2,198 | 0.012 | 0.144 | 0.359 | 1,146 | 1,938 | 0.011 | 0.126 |
| | A-1-b | G | 15.0 | 17.5 | 2.5 | 16.3 | 130 | 2,138 | 1,975 | 961 | 4,961 | | | | | 21 | 26 | 89 | 1.56 | 0.382 | 1,218 | 2,179 | 0.010 | 0.120 | 0.326 | 1,039 | 2,000 | 0.009 | 0.108 |
| 5 | A-1-b | G | 17.5 | 24.0 | 6.5 | 20.8 | 130 | 2,983 | 2,560 | 1,265 | 5,265 | | | | | 41 | 47 | 158 | 2.00 | 0.306 | 978 | 2,243 | 0.010 | 0.122 | 0.275 | 879 | 2,144 | 0.009 | 0.113 |
| | A-1-b | G | 24.0 | 31.0 | 7.0 | 27.5 | 135 | 3,928 | 3,455 | 1,739 | 5,739 | | | | | 41 | 43 | 141 | 2.64 | 0.235 | 750 | 2,489 | 0.008 | 0.093 | 0.220 | 703 | 2,442 | 0.007 | 0.088 |
| 6 | A-4b | C | 31.0 | 35.0 | 4.0 | 33.0 | 130 | 4,448 | 4,188 | 2,128 | 6,128 | 17 | 0.063 | 0.006 | 0.405 | | | | 3.17 | 0.197 | 630 | 2,758 | 0.002 | 0.024 | 0.188 | 601 | 2,729 | 0.002 | 0.023 |
| | A-4b | C | 35.0 | 39.0 | 4.0 | 37.0 | 130 | 4,968 | 4,708 | 2,399 | 6,399 | 17 | 0.063 | 0.006 | 0.405 | | | | 3.56 | 0.177 | 563 | 2,962 | 0.002 | 0.020 | 0.170 | 543 | 2,941 | 0.002 | 0.019 |
| | | | | | | | | | | | | | | | | | | | | | | Total Settlement: | | 4.011 in | | | Total Settlement: | | 3.339 in |

1. $\sigma_p' = \sigma_{vo}' + \sigma_m$; Estimate $\sigma_m$ of 2,000 psf in existing fill material and 4,000 psf (moderately overconsolidated) for natural soil deposits; Ref. Table 11.2, Coduto 2003
2. $C_c$ = 0.009(LL-10); Ref. Table 6-9, FHWA GEC 5
3. $C_r$ = 0.10($C_c$); 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-13-045 - FRA-70-12.86 - Retaining Wall 4W20

MSE Wall Settlement

Calculated By: HSK   Date: 07/12/2018

Checked By: JPS   Date: 07/13/2018

Boring B-016-1-09

| | | | |
|---|---|---|---|
| H= | 18.8 | ft | Total wall height |
| B'= | 10.4 | ft | Effective footing width due to eccentricity |
| $D_w$ = | 0.0 | ft | Depth below bottom of footing |
| $q_e$ = | 3,190 | psf | Equivalent bearing pressure at bottom of wall |

| | A-4b | | |
|---|---|---|---|
| $c_v$ = | 800 | $ft^2/yr$ | Coefficient of consolidation |
| t = | 0 | days | Time following completion of construction |
| $H_{dr}$ = | 8 | ft | Length of longest drainage path considered |
| $T_v$ = | 0.000 | | Time factor |
| U = | 0 | % | Degree of consolidation |
| $(S_c)_t$ = | 3.297 | in | Settlement complete at 99% of primary consolidation |

| | | | | | | | | | | | | | | | | | | | | | | | Total Settlement at Facing of Wall | | | Settlement Complete at 99% of Primary Consolidation | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Layer | Soil Type | Soil Type | Layer Depth (ft) | | Layer Thickness (ft) | Depth to Midpoint (ft) | $\gamma$ (pcf) | $\sigma_{vo}$ Bottom (psf) | $\sigma_{vo}$ Midpoint (psf) | $\sigma_{vo}'$ Midpoint (psf) | $\sigma_p'$ (1) (psf) | LL | $C_c$ (2) | $C_r$ (3) | $e_o$ (4) | $N_{60}$ | $(N1)_{60}$ (5) | C' (6) | $Z_f$/B | I (7) | $\Delta\sigma_v$ (8) (psf) | $\sigma_{vf}'$ Midpoint (psf) | $S_c$ (9,10) (ft) | $S_c$ (in) | Layer Settlement (in) | $(S_c)_t$ (11) (in) | Layer Settlement (in) |
| 1 | Fill | G | 0.0 | 2.5 | 2.5 | 1.3 | 115 | 288 | 144 | 66 | 2,066 | | | | | 8 | 16 | 34 | 0.12 | 0.500 | 1,594 | 1,660 | 0.104 | 1.243 | 1.243 | 1.243 | 1.243 |
| 2 | A-3a | G | 2.5 | 5.0 | 2.5 | 3.8 | 120 | 588 | 438 | 204 | 2,204 | | | | | 5 | 9 | 50 | 0.36 | 0.491 | 1,568 | 1,771 | 0.047 | 0.560 | 1.001 | 0.560 | 1.001 |
| | A-3a | G | 5.0 | 7.5 | 2.5 | 6.3 | 120 | 888 | 738 | 348 | 2,348 | | | | | 5 | 8 | 49 | 0.60 | 0.468 | 1,494 | 1,841 | 0.037 | 0.441 | | 0.441 | |
| 3 | A-2-4 | G | 7.5 | 10.0 | 2.5 | 8.8 | 120 | 1,188 | 1,038 | 492 | 2,492 | | | | | 4 | 6 | 51 | 0.84 | 0.434 | 1,385 | 1,876 | 0.028 | 0.339 | 0.619 | 0.339 | 0.619 |
| | A-2-4 | G | 10.0 | 12.5 | 2.5 | 11.3 | 120 | 1,488 | 1,338 | 636 | 2,636 | | | | | 4 | 6 | 51 | 1.08 | 0.396 | 1,264 | 1,899 | 0.023 | 0.279 | | 0.279 | |
| 4 | A-1-b | G | 12.5 | 15.0 | 2.5 | 13.8 | 130 | 1,813 | 1,650 | 792 | 4,792 | | | | | 21 | 28 | 92 | 1.32 | 0.359 | 1,146 | 1,938 | 0.011 | 0.126 | 0.234 | 0.126 | 0.234 |
| | A-1-b | G | 15.0 | 17.5 | 2.5 | 16.3 | 130 | 2,138 | 1,975 | 961 | 4,961 | | | | | 21 | 26 | 89 | 1.56 | 0.326 | 1,039 | 2,000 | 0.009 | 0.108 | | 0.108 | |
| 5 | A-1-b | G | 17.5 | 24.0 | 6.5 | 20.8 | 130 | 2,983 | 2,560 | 1,265 | 5,265 | | | | | 41 | 47 | 158 | 2.00 | 0.275 | 879 | 2,144 | 0.009 | 0.113 | 0.200 | 0.113 | 0.200 |
| | A-1-b | G | 24.0 | 31.0 | 7.0 | 27.5 | 135 | 3,928 | 3,455 | 1,739 | 5,739 | | | | | 41 | 43 | 141 | 2.64 | 0.220 | 703 | 2,442 | 0.007 | 0.088 | | 0.088 | |
| 6 | A-4b | C | 31.0 | 35.0 | 4.0 | 33.0 | 130 | 4,448 | 4,188 | 2,128 | 6,128 | 17 | 0.063 | 0.006 | 0.405 | | | | 3.17 | 0.188 | 601 | 2,729 | 0.002 | 0.023 | 0.042 | 0.000 | 0.000 |
| | A-4b | C | 35.0 | 39.0 | 4.0 | 37.0 | 130 | 4,968 | 4,708 | 2,399 | 6,399 | 17 | 0.063 | 0.006 | 0.405 | | | | 3.56 | 0.170 | 543 | 2,941 | 0.002 | 0.019 | | 0.000 | |

Settlement Remaining After Hold Period: 0.042 in

1. $\sigma_p' = \sigma_{vo}' + \sigma_m$; Estimate $\sigma_m$ of 2,000 psf in existing fill material and 4,000 psf (moderately overconsolidated) for natural soil deposits; Ref. Table 11.2, Coduto 2003
2. $C_c = 0.009(LL-10)$; Ref. Table 6-9, FHWA GEC 5
3. $C_r = 0.10(C_c)$; 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 (Cohesiv soil layers)
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)
11. $(S_c)_t = S_c(U/100)$; U = 100 for all granular soils at time t = 0

W-13-045 - FRA-70-12.86 - Retaining Wall 4W20

MSE Wall Settlement

Calculated By: HSK Date: 7/12/2018

Checked By: JPS Date: 7/13/2018

Boring B-016-6-13

| | | | |
|---|---|---|---|
| H= | 18.8 | ft | Total wall height |
| B'= | 10.4 | ft | Effective footing width due to eccentricity |
| $D_w$ = | 0.0 | ft | Depth below bottom of footing |
| $q_e$ = | 3,190 | 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) | $\gamma$ (pcf) | $\sigma_{vo}$ Bottom (psf) | $\sigma_{vo}$ Midpoint (psf) | $\sigma_{vo}'$ Midpoint (psf) | $\sigma_p'$ [(1)] (psf) | LL | $C_c$ [(2)] | $C_r$ [(3)] | $e_o$ [(4)] | $N_{60}$ | $(N1)_{60}$ [(5)] | C' [(6)] | $Z_f$/B | I [(7)] | $\Delta\sigma_v$ [(8)] (psf) | $\sigma_{vf}'$ Midpoint (psf) | $S_c$ [(9,10)] (ft) | $S_c$ (in) | I [(7)] | $\Delta\sigma_v$ [(8)] (psf) | $\sigma_{vf}'$ Midpoint (psf) | $S_c$ [(9,10)] (ft) | $S_c$ (in) |
| 1 | A-1-b | G | 0.0 | 2.0 | 2.0 | 1.0 | 130 | 260 | 130 | 68 | 2,068 | | | | | 17 | 34 | 111 | 0.10 | 0.997 | 3,181 | 3,248 | 0.030 | 0.364 | 0.500 | 1,594 | 1,662 | 0.025 | 0.301 |
| 2 | A-4a | C | 2.0 | 4.5 | 2.5 | 3.3 | 125 | 573 | 416 | 213 | 2,213 | 27 | 0.153 | 0.015 | 0.483 | | | | 0.31 | 0.931 | 2,968 | 3,182 | 0.067 | 0.802 | 0.494 | 1,577 | 1,790 | 0.024 | 0.286 |
| 3 | A-4b | G | 4.5 | 7.0 | 2.5 | 5.8 | 120 | 873 | 723 | 364 | 2,364 | | | | | 5 | 8 | 22 | 0.55 | 0.785 | 2,503 | 2,867 | 0.103 | 1.240 | 0.474 | 1,512 | 1,876 | 0.082 | 0.986 |
| | A-4b | G | 7.0 | 9.5 | 2.5 | 8.3 | 120 | 1,173 | 1,023 | 508 | 2,508 | | | | | 5 | 7 | 21 | 0.79 | 0.645 | 2,058 | 2,566 | 0.084 | 1.012 | 0.442 | 1,408 | 1,916 | 0.069 | 0.830 |
| 4 | A-6b | C | 9.5 | 12.0 | 2.5 | 10.8 | 125 | 1,485 | 1,329 | 658 | 2,658 | 33 | 0.207 | 0.021 | 0.530 | | | | 1.03 | 0.536 | 1,711 | 2,369 | 0.019 | 0.226 | 0.404 | 1,288 | 1,946 | 0.016 | 0.191 |
| | A-6b | C | 12.0 | 14.5 | 2.5 | 13.3 | 125 | 1,798 | 1,641 | 814 | 2,814 | 33 | 0.207 | 0.021 | 0.530 | | | | 1.27 | 0.455 | 1,450 | 2,265 | 0.015 | 0.180 | 0.366 | 1,169 | 1,983 | 0.013 | 0.157 |
| 5 | A-1-b | G | 14.5 | 18.5 | 4.0 | 16.5 | 130 | 2,318 | 2,058 | 1,028 | 3,028 | | | | | 29 | 36 | 115 | 1.59 | 0.377 | 1,202 | 2,230 | 0.012 | 0.140 | 0.323 | 1,029 | 2,057 | 0.010 | 0.125 |
| 6 | A-1-a | G | 18.5 | 23.5 | 5.0 | 21.0 | 135 | 2,993 | 2,655 | 1,345 | 3,345 | | | | | 68 | 77 | 310 | 2.02 | 0.303 | 967 | 2,311 | 0.004 | 0.046 | 0.273 | 871 | 2,215 | 0.003 | 0.042 |
| | A-1-a | G | 23.5 | 28.5 | 5.0 | 26.0 | 135 | 3,668 | 3,330 | 1,708 | 3,708 | | | | | 68 | 72 | 278 | 2.50 | 0.248 | 791 | 2,499 | 0.003 | 0.036 | 0.231 | 737 | 2,444 | 0.003 | 0.034 |
| 7 | A-4a | C | 28.5 | 31.0 | 2.5 | 29.8 | 125 | 3,980 | 3,824 | 1,967 | 3,967 | 22 | 0.108 | 0.011 | 0.444 | | | | 2.86 | 0.218 | 696 | 2,663 | 0.002 | 0.030 | 0.206 | 658 | 2,625 | 0.002 | 0.028 |
| | A-4a | C | 31.0 | 33.5 | 2.5 | 32.3 | 125 | 4,293 | 4,136 | 2,124 | 4,124 | 22 | 0.108 | 0.011 | 0.444 | | | | 3.10 | 0.202 | 644 | 2,768 | 0.002 | 0.026 | 0.192 | 613 | 2,737 | 0.002 | 0.025 |
| 8 | A-6b | C | 33.5 | 36.5 | 3.0 | 35.0 | 125 | 4,668 | 4,480 | 2,296 | 4,296 | 38 | 0.252 | 0.025 | 0.569 | | | | 3.37 | 0.186 | 595 | 2,891 | 0.005 | 0.058 | 0.179 | 571 | 2,867 | 0.005 | 0.056 |
| | A-6b | C | 36.5 | 41.5 | 5.0 | 39.0 | 125 | 5,293 | 4,980 | 2,546 | 4,546 | 38 | 0.252 | 0.025 | 0.569 | | | | 3.75 | 0.168 | 535 | 3,082 | 0.007 | 0.080 | 0.162 | 517 | 3,064 | 0.006 | 0.077 |
| | A-6b | C | 41.5 | 46.5 | 5.0 | 44.0 | 125 | 5,918 | 5,605 | 2,859 | 4,859 | 38 | 0.252 | 0.025 | 0.569 | | | | 4.23 | 0.149 | 476 | 3,335 | 0.005 | 0.064 | 0.145 | 463 | 3,322 | 0.005 | 0.063 |
| | | | | | | | | | | | | | | | | | | | | | | Total Settlement: | | 4.304 in | | | Total Settlement: | | 3.200 in |

1. $\sigma_p' = \sigma_{vo}' + \sigma_m$; Estimate $\sigma_m$ of 3,000 psf for moderately overconsolidated soil deposit; Ref. Table 11.2, Coduto 2003
2. $C_c$ = 0.009(LL-10); Ref. Table 6-9, FHWA GEC 5
3. $C_r$ = 0.10($C_c$); 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 (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-13-045 - FRA-70-12.86 - Retaining Wall 4W20
MSE Wall Settlement

Calculated By: HSK  Date: 07/12/2018
Checked By: JPS  Date: 07/13/2018

Boring B-016-6-13

| | | | |
|---|---|---|---|
| H= | 18.8 | ft | Total wall height |
| B'= | 10.4 | ft | Effective footing width due to eccentricity |
| $D_w$ = | 0.0 | ft | Depth below bottom of footing |
| $q_e$ = | 3,190 | psf | Equivalent bearing pressure at bottom of wall |

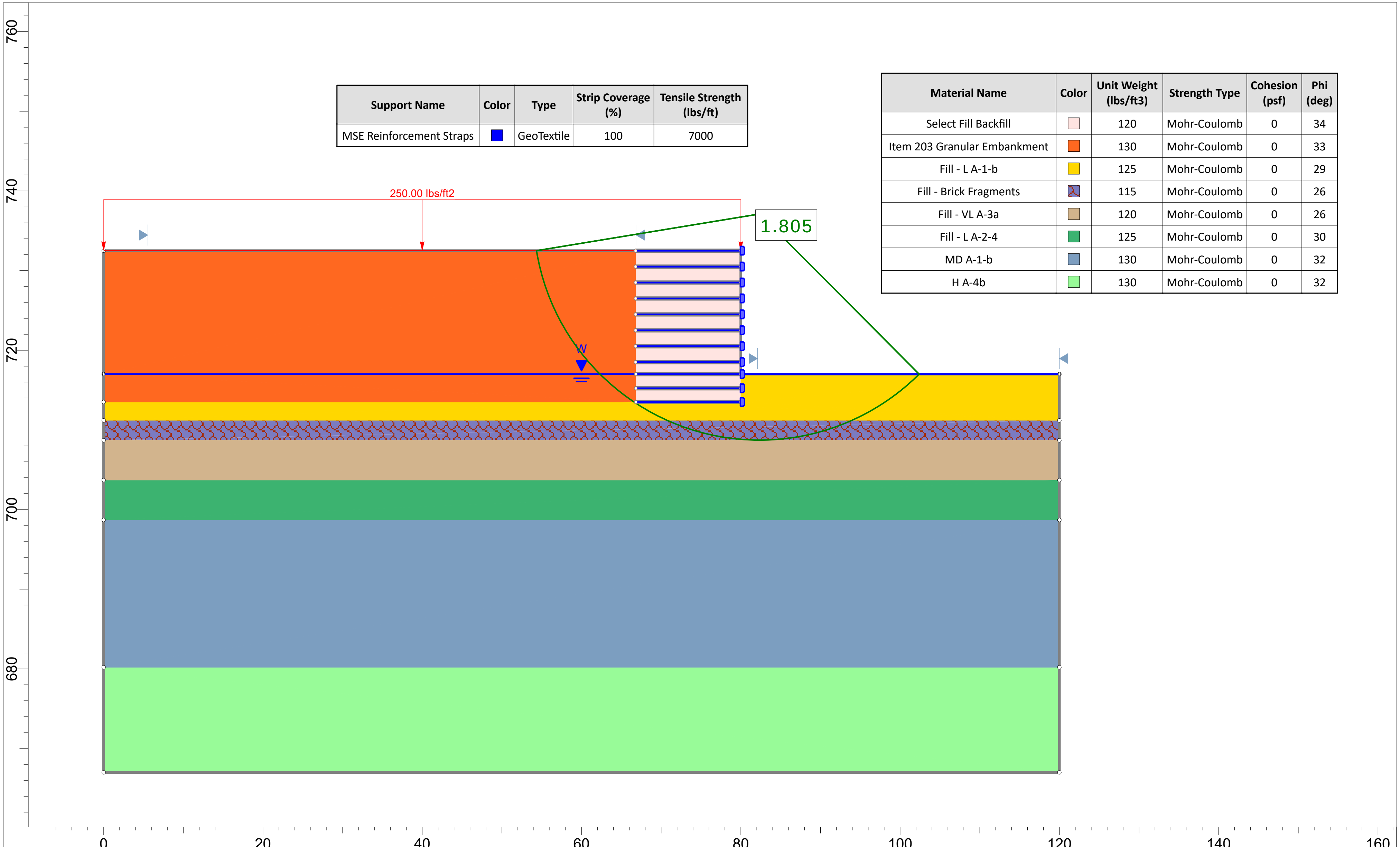
| | A-6b (Upper) | A-4a | A-6b (Lower) | | |
|---|---|---|---|---|---|
| $c_v$ = | 300 | 800 | 300 | $ft^2/yr$ | Coefficient of consolidation |
| t = | 7 | 7 | 7 | days | Time following completion of construction |
| $H_{dr}$ = | 2.5 | 5 | 13 | ft | Length of longest drainage path considered |
| $T_v$ = | 0.921 | 0.614 | 0.034 | | Time factor |
| U = | 92 | 82 | 21 | % | Degree of consolidation |

$(S_c)_t$ = 2.891 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) | $\sigma_{vo}$ Bottom (psf) | $\sigma_{vo}$ Midpoint (psf) | $\sigma_{vo}'$ Midpoint (psf) | $\sigma_p'$ [1] (psf) | LL | $C_c$ [2] | $C_r$ [3] | $e_o$ [4] | $N_{60}$ | $(N1)_{60}$ [5] | C' [6] | $Z_f/B$ | I [7] | $\Delta\sigma_v$ [8] (psf) | $\sigma_{vf}'$ 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-1-b | G | 0.0 | 2.0 | 2.0 | 1.0 | 130 | 260 | 130 | 68 | 4,068 | | | | | 17 | 34 | 111 | 0.10 | 0.500 | 1,594 | 1,662 | 0.025 | 0.301 | 0.301 | 0.301 | 0.301 |
| 2 | A-4a | C | 2.0 | 4.5 | 2.5 | 3.3 | 125 | 573 | 416 | 213 | 4,213 | 27 | 0.153 | 0.015 | 0.483 | | | | 0.31 | 0.494 | 1,577 | 1,790 | 0.024 | 0.286 | 0.286 | 0.286 | 0.286 |
| 3 | A-4b | G | 4.5 | 7.0 | 2.5 | 5.8 | 120 | 873 | 723 | 364 | 4,364 | | | | | 5 | 8 | 22 | 0.55 | 0.474 | 1,512 | 1,876 | 0.082 | 0.986 | 1.815 | 0.907 | 1.670 |
| | A-4b | G | 7.0 | 9.5 | 2.5 | 8.3 | 120 | 1,173 | 1,023 | 508 | 4,508 | | | | | 5 | 7 | 21 | 0.79 | 0.442 | 1,408 | 1,916 | 0.069 | 0.830 | | 0.763 | |
| 4 | A-6b | C | 9.5 | 12.0 | 2.5 | 10.8 | 125 | 1,485 | 1,329 | 658 | 4,658 | 33 | 0.207 | 0.021 | 0.530 | | | | 1.03 | 0.404 | 1,288 | 1,946 | 0.016 | 0.191 | 0.348 | 0.191 | 0.348 |
| | A-6b | C | 12.0 | 14.5 | 2.5 | 13.3 | 125 | 1,798 | 1,641 | 814 | 4,814 | 33 | 0.207 | 0.021 | 0.530 | | | | 1.27 | 0.366 | 1,169 | 1,983 | 0.013 | 0.157 | | 0.157 | |
| 5 | A-1-b | G | 14.5 | 18.5 | 4.0 | 16.5 | 130 | 2,318 | 2,058 | 1,028 | 5,028 | | | | | 29 | 36 | 115 | 1.59 | 0.323 | 1,029 | 2,057 | 0.010 | 0.125 | 0.125 | 0.125 | 0.125 |
| 6 | A-1-a | G | 18.5 | 23.5 | 5.0 | 21.0 | 135 | 2,993 | 2,655 | 1,345 | 5,345 | | | | | 68 | 77 | 310 | 2.02 | 0.273 | 871 | 2,215 | 0.003 | 0.042 | 0.076 | 0.042 | 0.076 |
| | A-1-a | G | 23.5 | 28.5 | 5.0 | 26.0 | 135 | 3,668 | 3,330 | 1,708 | 5,708 | | | | | 68 | 72 | 278 | 2.50 | 0.231 | 737 | 2,444 | 0.003 | 0.034 | | 0.034 | |
| 7 | A-4a | C | 28.5 | 31.0 | 2.5 | 29.8 | 125 | 3,980 | 3,824 | 1,967 | 5,967 | 22 | 0.108 | 0.011 | 0.444 | | | | 2.86 | 0.206 | 658 | 2,625 | 0.002 | 0.028 | 0.053 | 0.023 | 0.043 |
| | A-4a | C | 31.0 | 33.5 | 2.5 | 32.3 | 125 | 4,293 | 4,136 | 2,124 | 6,124 | 22 | 0.108 | 0.011 | 0.444 | | | | 3.10 | 0.192 | 613 | 2,737 | 0.002 | 0.025 | | 0.020 | |
| 8 | A-6b | C | 33.5 | 36.5 | 3.0 | 35.0 | 125 | 4,668 | 4,480 | 2,296 | 6,296 | 38 | 0.252 | 0.025 | 0.569 | | | | 3.37 | 0.179 | 571 | 2,867 | 0.005 | 0.056 | 0.196 | 0.012 | 0.041 |
| | A-6b | C | 36.5 | 41.5 | 5.0 | 39.0 | 125 | 5,293 | 4,980 | 2,546 | 6,546 | 38 | 0.252 | 0.025 | 0.569 | | | | 3.75 | 0.162 | 517 | 3,064 | 0.006 | 0.077 | | 0.016 | |
| | A-6b | C | 41.5 | 46.5 | 5.0 | 44.0 | 125 | 5,918 | 5,605 | 2,859 | 6,859 | 38 | 0.252 | 0.025 | 0.569 | | | | 4.23 | 0.145 | 463 | 3,322 | 0.005 | 0.063 | | 0.013 | |

Settlement Remaining After Hold Period: 0.310 in

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




| Support Name | Color | Type | Strip Coverage (%) | Tensile Strength (lbs/ft) |
|---|---|---|---|---|
| MSE Reinforcement Straps | | GeoTextile | 100 | 7000 |

| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) |
|---|---|---|---|---|---|
| Select Fill Backfill | | 120 | Mohr-Coulomb | 0 | 34 |
| Item 203 Granular Embankment | | 130 | Mohr-Coulomb | 0 | 33 |
| Fill - L A-1-b | | 125 | Mohr-Coulomb | 0 | 29 |
| Fill - Brick Fragments | | 115 | Mohr-Coulomb | 0 | 26 |
| Fill - VL A-3a | | 120 | Mohr-Coulomb | 0 | 26 |
| Fill - L A-2-4 | | 125 | Mohr-Coulomb | 0 | 30 |
| MD A-1-b | | 130 | Mohr-Coulomb | 0 | 32 |
| H A-4b | | 130 | Mohr-Coulomb | 0 | 32 |

| | | | |
|---|---|---|---|
| *Project* | FRA-70 12.68 MSE Retaining Wall 4W20 | | |
| *Analysis Description* | MSE Wall 4W20 - 18.8 ft Wall Height - Drained Spencer | | |
| *Drawn By* HSK | *Scale* 1:200 | *Company* | Resource International Inc |
| *Date* 7/15/2018, 6:22:37 PM | | *File Name* | FRA-70-12.68 - MSE Wall 4W20.slim |

SLIDEINTERPRET 7.020