Resource International, Inc.

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FRA-71-14.36 PHASE 6R
FRA-70-1373B
RAMP D7 OVER SHORT STREET
PID NO. 105588
FRANKLIN COUNTY, OHIO

STRUCTURE FOUNDATION EXPLORATION REPORT (REV. 2)

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

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

April 2020

RESOURCE INTERNATIONAL, INC.
ISO | ISO 9001:2015 Certified QMS
An ISO 9001:2008 QMS Certified Firm

June 18, 2015 (Revised April 6, 2020)

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

Re: Structure Foundation Exploration Report (Rev. 2)
FRA-71-14.36 Phase 6R
FRA-70-1373B – Ramp D7 over Short Street
PID No. 105588
Rii Project No. W-13-072

Mr. Antonios:

Resource International, Inc. (Rii) is pleased to submit this revised structure foundation exploration report for the above referenced project. Engineering logs have been prepared and are attached to this report along with the results of laboratory testing. This report includes recommendations for the design and construction of the proposed FRA-70-1373B bridge structure carrying Ramp D7 over Short Street as part of the FRA-71-14.36 Phase 6R in Columbus, Ohio.

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

Sincerely,

RESOURCE INTERNATIONAL, INC.

Brian Trenner

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

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Enclosure: Structure Foundation Exploration Report (Rev. 2)

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TABLE OF CONTENTS

Section **Page**

APPENDICIES

Appendix I	**Vicinity Map and Boring Plan**
Appendix II	**Description of Soil Terms**
Appendix III	**Project Boring Logs: B-020-4-13, B-020-6-13 and B-020-8-13**
Appendix IV	**Laboratory Test Results**
Appendix V	**Drilled Shaft Calculations**
Appendix VI	**GRLWEAP Driveability Analysis Outputs**
Appendix VII	**Lateral Design Parameters**
Appendix VIII	**MSE Wall Calculations**

EXECUTIVE SUMMARY

Resource International, Inc. (Rii) has completed a structure foundation exploration for the design and construction of the proposed FRA-70-1373B bridge structure carrying the proposed Ramp D7 over Short Street. The proposed Ramp D7 will be a two-lane ramp that will carry traffic from Mound Street to I-70 westbound. Based on information provided by CH2M HILL and ms consultants, it is understood that the proposed bridge consist of a two-span composite steel girder structure with a reinforced concrete deck and semi-integral rear abutment behind a mechanically stabilized earth (MSE) wall, cap and column pier and stub-type forward abutment. The proposed structure will have a total length of approximately 262 feet and width of approximately 37 feet. In addition, the roadway profile will be elevated approximately 26 feet above the existing ground surface grade at the rear abutment and approximately 15 feet above existing grade at the forward abutment. Please note that the analysis and recommendations for Retaining Wall E4, between Sta. 400+98 and 402+02 (BL Wall E4), at the rear abutment is presented under this report cover. In addition, Retaining Wall E5 connects to both ends of the forward abutment and extend west from both ends of the abutment foundation. Design recommendations for the remaining alignment of Retaining Wall E4 and Retaining Wall E5 are under separate covers.

Exploration and Findings

Between March 5, 2014, and January 12, 2015, three (3) borings, designated as B-020-4-13, B-020-6-13, and B-020-8-13, were drilled to completion depths ranging from 94.7 to 102.0 feet below the existing ground surface at the locations shown on the boring plan provided in Appendix I of the full report.

Boring B-020-4-13 was performed in a grass area between the sidewalk and pine trees at the southwest corner of the intersection of Mound Street and Short Street and encountered 2.0 inches of topsoil at the ground surface. Borings B-020-6-13 and B-020-8-13 were drilled through the existing sidewalk that runs along the south side of Mound Street, east of Short Street, and encountered 2.0 and 8.0 inches of concrete overlying 7.0 and 4.0 inches of aggregate base, respectively, at the ground surface.

Beneath the surface materials in borings B-020-4-13, B-020-6-13 and B-020-8-13, material identified as existing fill was encountered extending to depths ranging from 5.5 to 10.5 feet below the existing ground surface. The fill material consisted of dark brown and brown gravel, gravel with sand and silt and silt and clay (ODOT A-1-a, A-2-4, A-6a) and contained organics, wood fibers and slag.

Underlying the existing fill, natural granular soils were encountered with intermittent seams of cohesive material. The granular soils were generally described as brown and gray gravel, gravel with sand, gravel with sand and silt and coarse and fine sand (ODOT A-1-a, A-1-b, A-2-4, A-3a). The cohesive soils were described as brown and gray silt and clay, silty clay and clay (ODOT A-6a, A-6b, A-7-6). Cobbles and boulders were generally encountered above the bedrock in borings B-020-4-13, B-020-6-13 and B-020-8-13 starting at elevations ranging from 686 to 696 feet msl. Due to the significant presence of large boulders in borings B-020-4-13 and B-020-8-13 at 55.5 and 37.0 feet below existing grade, respectively, mud rotary drilling techniques with casing advancer or tricone bit were utilized until bedrock was encountered.

Top of bedrock in the borings was encountered at elevations ranging from 649.6 to 656.0 feet msl. The upper portion of the bedrock encountered consists of shale with interbedded seams of mudstone and claystone overlying competent limestone bedrock. Shale bedrock was encountered in the borings at elevations ranging from 649.6 to 656.0 feet msl, and limestone bedrock was encountered at elevations ranging from 628.6 to 630.3 feet msl.

Analyses and Recommendations

Design details of the proposed structure were provided by CH2M HILL and ms consultants. Based on the information provided, it is understood that the proposed bridge consist of a two-span composite steel girder structure with a reinforced concrete deck and semi-integral rear abutment behind a mechanically stabilized earth (MSE) wall, cap and column pier and stub-type forward abutment. The roadway profile will be elevated approximately 26 feet above the existing ground surface grade at the rear abutment and approximately 15 feet above existing grade at the forward abutment.

Driven piles were considered for support of all of the substructure units for this structure. However, due to the significant presence of underground utilities within the roadway of both Mound Street and Short Street and the proximity of the proposed substructure units to these facilities, pile driving at the pier and forward abutment substructure locations was considered unfavorable due to the potential for excessive vibration and the potential for damage or need for relocation. Therefore, drilled shafts are recommended for foundation support at these locations.

Drilled Shafts Extended to Bedrock

It is understood that drilled shaft foundations are being utilized to support the pier and forward abutment substructure units due to the significant presence of underground utilities within the footprint of the substructure units. It is recommended that the drilled shafts extend through the surficial soils, and be socketed a minimum of 1.5 shaft diameters into the underlying shale, mudstone, claystone or limestone bedrock, as specified in Section 10.8.3.5.4c of the 2014 AASHTO LRDF BDS, and end bear on or within the competent limestone bedrock encountered below the shale, mudstone and claystone. Based on plan information provided by CH2M HILL and ms consultants, the shaft diameter within the overburden soils will be 3.5 feet, and the shaft diameter within the rock socket will be 3.0 feet. The following table lists the estimated elevation of the top of limestone bedrock and corresponding embedment depth to the top of limestone bedrock at the pier and forward abutment substructure units.

Drilled Shaft Recommendations

Substructure Unit (Boring)	Top of Shaft Elevation [1] (feet msl)	Top of Bedrock Elevation [2] (feet msl)	Top of Limestone Elevation (feet msl)	Shaft Tip Elevation [3] (feet msl)	Shaft Length [4] (feet)
Pier (B-020-6-13)	713.8	653.7	628.6	628.6	85.2
Forward Abutment (B-020-8-13)	720.0	656.0	629.0	629.0	91.0

1. *Top of shaft elevation based on structure information provided by CH2M HILL and ms consultants.*
2. *Top of bedrock elevation indicates the top of shale, mudstone or claystone bedrock elevation where coring techniques will likely be required to advance the shaft excavations.*
3. *Shaft tip elevation based on a minimum rock socket length of 1.5 shaft diameters into the shale, mudstone, claystone or limestone bedrock, and the shaft tip end bearing on or within the competent limestone bedrock encountered below the shale, mudstone and claystone.*
4. *Shaft length represents the overall length of shaft through the overburden soil and bedrock socket within the shale, mudstone, claystone or limestone bedrock.*

Based on unconfined compression tests performed on limestone rock cores obtained from borings B-020-4-13, B-020-6-13 and B-020-8-13, the unconfined compressive strength of the limestone bedrock ranges from 4,737 to 16,173 psi. It is recommended that drilled shaft foundations socketed into competent limestone bedrock be proportioned for a nominal end bearing resistance of 1,705 ksf at the strength limit state. A resistance factor of $\varphi = 0.5$ at the strength limit state should be utilized for design.

If the drilled shaft lengths required to end bear in the limestone bedrock not economically feasible, then consideration can be given to extending the drilled shafts through the surficial soils to bear in the shale/mudstone/claystone bedrock encountered at an elevation of 653.7 and 656.0 feet msl in borings B-020-6-13 and B-020-8-13, respectively. Given the weak nature of this type of rock encountered in the borings, it is recommended that the drilled shafts be designed using a combination of end bearing and side resistance within the bedrock socket. The shafts should be socketed a minimum of 2.0 shaft diameters into the shale/mudstone/claystone bedrock, and extended beyond this minimum socket length depending on the vertical and lateral load demands on the drilled shafts.

Based on unconfined compression tests performed on shale rock cores obtained from borings B-020-6-13 and B-020-8-13, the unconfined compressive strength of the shale bedrock ranges from 177 to 318 psi. It is recommended that drilled shaft foundations socketed into the shale/mudstone/claystone bedrock be proportioned for a nominal end bearing resistance of 60 ksf at the strength limit state. A resistance factor of $\varphi = 0.5$ at the strength limit state should be utilized for design. The nominal side resistance, q_s, for drilled shafts socketed into rock was calculated using equation 10.8.3.5.4b-1 of the AASHTO LRFD BDS. A nominal side resistance of 7.0 ksf and resistance factor of $\varphi = 0.55$ at the strength limit state should be utilized for the portion of the shafts that will extend into the shale/mudstone/claystone.

Due to the difference in the required displacement to mobilize side friction in the overburden soil, the calculated resistance should only consider end bearing and side resistance within the bedrock socket. Side resistance within the overburden soils should be neglected.

Drilled Shafts Bearing Above Bedrock

If it is not desired to extend the drilled shafts into the underlying shale or limestone bedrock due to the depth or effort required, it is recommended that the drilled shafts be designed using the axial design parameters provided in the following table. To achieve the most economical design, the drilled shafts should extend to bear in the very dense gravel, gravel and sand (ODOT A-1-a, A-1-b) or hard clay (ODOT A-7-6) at the corresponding elevations noted below in order to maximize the end bearing resistance. The drilled shafts should be proportioned for a nominal bearing resistance as follows:

Drilled Shaft Axial Design Parameters

Substructure Unit	Elevation [1] (feet msl)	Shaft Length (feet)	Soil Type	Nominal Resistance (ksf)		Resistance Factor	
				End	Side	End	Side
Pier (B-020-6-13)	714.1-708.6	0.0-5.5	A-6a	9	0.68 [2]	0.40	0.45
	708.6-703.6	5.5-10.5	A-1-a	15	0.71	0.50	0.55
	703.6-693.6	10.5-20.5	A-7-6	18	1.10	0.40	0.45
	693.6-688.6	20.5-25.5	A-1-a	31	1.66	0.50	0.55
	688.6-654.1	25.5-60.0	A-1-a	60	5.18	0.50	0.55
Forward Abutment (B-020-8-13)	720.0-715.5	0.0-4.5	A-2-4	9	0.32	0.50	0.55
	715.5-700.5	4.5-19.5	A-1-a	48	1.91	0.50	0.55
	700.5-689.0	19.5-31.0	A-3a	54	1.91	0.50	0.55
	689.0-669.0	31.0-51.0	A-1-b	60	5.02	0.50	0.55
	669.0-656.0	51.0-64.0	A-7-6	36	2.04	0.40	0.45

1. *Top of shaft elevation based on structure information provided by CH2M HILL and ms consultants.*
2. *Side resistance should be neglected for the upper 5.0 feet of the shaft length where cohesive soils (ODOT A-4a, A-4b, A-6a, A-6b, A-7-6) are present below the bottom of footing/top of shaft elevation.*

Per Section 10.8.3.5.3 of the AASHTO LRFD BDS, where drilled shafts are extended to end bear in a strong soil layer overlying a weaker soil layer, the end bearing resistance shall be reduced if the tip elevation is within 1.5 times the diameter of the drilled shaft above the top of the weaker soil layer. A weighted average that varies linearly from the full end bearing resistance in the overlying strong soil layer at a distance of 1.5 times the diameter of the drilled shaft above the top of the weak soil layer to the end bearing resistance of the weak soil layer at the top of the weak soil layer should be used to determine the end bearing resistance utilized in the design. Therefore, the end bearing resistance utilized in the design will need to be adjusted accordingly if the tip elevation of the drilled shafts will be within 1.5 times the diameter of the drilled shaft above the underlying weaker soil layer.

Per Section 10.8.3.5.1b of the AASHTO LRFD BDS, side resistance should be neglected for the upper 5.0 feet of the shaft length where cohesive soils (ODOT A-4a, A-4b, A-6a, A-6b, A-7-6) are present below the bottom of footing/top of shaft elevation. Additionally, based on the estimated settlement noted above, it is anticipated that 100 percent of the side friction resistance will be mobilized and approximately 70 percent of the end bearing resistance will be mobilized based on the displacement of the shaft. Therefore, the nominal end bearing resistance noted in the table above should be reduced to 70 percent of the values provided for the respective tip elevation in the determination of the design shaft resistance.

Downdrag Considerations for Drilled Shafts

Based on information provided by CH2M HILL, it is understood that the forward abutment wall will be constructed prior to the construction of the adjacent Retaining Wall E5. Retaining Wall E5 will consist of two independent MSE walls that will be constructed parallel to each other and will support Ramp D7. The walls will connect to the back of the forward abutment wall and will have a maximum height of 19.3 feet at the connection with the forward abutment wall. The anticipated settlement below the foundation at the forward abutment due to the weight of the MSE wall backfill behind the abutment is 0.57 inches. Based on the results of the settlement analysis, downdrag loads will develop along the drilled shafts supporting the proposed forward abutment. Using the traditional method criterion, the depth of downdrag at the forward abutment for 100 percent of primary consolidation is calculated to be 18.5 feet below the bottom of footing elevation. The unfactored downdrag load induced on the shafts was calculated using static analysis and is equal to the magnitude of the side resistance over the length of the shaft within the downdrag zone provided above. The side resistance values provided in the table above for the forward abutment (boring B-020-8-13) were utilized in the calculation of the downdrag load. Considering a 3.5-foot shaft diameter within the overburden soils, the unfactored downdrag load is 310 kips. A load factor of 1.25 should be utilized in accounting for the factored downdrag load.

Driven Pile Recommendations

It is understood that driven piles will be utilized at the rear abutment substructure unit. Given the depth of bedrock encountered in boring B-020-4-13, it is recommended that steel H-piles (ODOT Item 507.06) driven to refusal on bedrock be employed for foundation support. Per Section 202.2.3.2a of the 2019 ODOT Bridge Design Manual, refusal is met during driving when the pile penetration is an inch or less after receiving at least 20 blows from the pile hammer. The following table shows recommended pile lengths and the factored structural axial resistance ($R_{R\,max}$) of steel H-piles and associated resistance factors (φ):

Driven Pile Recommendations

Substructure Unit	Ground Elevation [1] (feet msl)	Pile Size	Pile Elevation		Pile Length [4] (feet)	$R_{R\ max}$ [5] (kips/pile)	Sleeve Length [6]	φ [7]
			Top [2]	Tip [3]				
Rear Abutment (B-020-4-13)	714.0	HP 10x42 [8]	731.0	646.0	85	310	18.0	N/A

1. *Ground elevation listed is the ground elevation at the respective boring locations.*
2. *The top of pile elevation corresponds to the pile cutoff elevation, which is assumed to be 1.0-foot above the proposed bottom of footing elevation.*
3. *The pile tip elevation for steel H-piles driven to refusal on bedrock is based on a penetration of 4.0 feet into the weathered shale bedrock.*
4. *Per Section 202.3.2 of the 2019 ODOT BDM, the estimated pile length was determined as the pile cutoff elevation (top) minus the pile tip elevation, rounded up to the nearest 5.0 feet.*
5. *The factored structural axial resistance for H-piles is based on the structural limit state of the steel H-pile section per Section 202.2.3.2.a of the 2019 ODOT BDM.*
6. *Sleeve length represents the required length of pile that should be sleeved within the MSE wall backfill.*
7. *For H-piles driven to refusal on bedrock, no geotechnical resistance factor should be applied to the factored structural axial resistance values presented, as the values presented account for the structural resistance factor, φ_c = 0.50, for H-piles subject to damage due to severe driving conditions.*
8. *A steel pile point is recommended to protect the tips of the steel H-piles during pile installation.*

The anticipated total settlement at the facing of the MSE wall at the rear abutment is 2.19 inches. Results of the settlement analysis indicate that approximately 90 percent of the primary consolidation of the cohesive layers at the rear will be complete within 15 days following the placement if the surcharge load. Therefore, if the above noted waiting period is specified following completion of construction of the MSE wall at the rear abutment, downdrag forces along the piles will be eliminated.

MSE Wall Recommendations

It is proposed to construct an MSE wall at the rear abutment (Retaining Wall E4 between Sta. 400+98 and 402+02, BL Wall E4) of the proposed bridge structure. Based upon the proposed plan information provided by CH2M HILL and ms consultants, the wall height at the rear abutment is anticipated to be 29.5 feet from the top of the leveling pad to the proposed profile grade of the roadway. The anticipated bearing materials at the rear abutment are anticipated to consist of existing fill comprised of stiff silt and clay (ODOT A-6a) that contained organics and wood fibers overlying very stiff clay (ODOT A-7-6). As noted in Section 5.4 of the full report, it is understood that ground improvement techniques will be implemented along the alignment of Retaining Wall E4, including where the wall crosses the rear abutment. 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 factored bearing resistance as indicated in the following table.

FRA-70-1373B MSE Wall Design Parameters

Substructure Element	Wall Height Analyzed (feet)	Backslope Behind Wall	Minimum Required Reinforcement Length [1] (feet)	Bearing Resistance at Strength Limit (ksf)		Service Limit Equivalent Bearing Pressure [3] (ksf)
				Nominal	Factored [2]	
Rear Abutment / Retaining Wall E4 (Sta. 400+98 to 402+02) (B-020-4-13)	29.5	Level	20.7 (0.70H)	3.71	2.41	7.00

1. *The required foundation width is expressed as a percentage of the wall height, H.*
2. *A geotechnical resistance factor of φ_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 service limit state.*

Total settlements of up to 10.91 inches at the center of the reinforced soil mass and 5.66 inches at the facing of the wall are anticipated at the rear abutment. Based on the results of the analysis, 90 percent of the total settlement at the rear abutment is anticipated to occur over a period of approximately 30 days.

Based on the results of the external and global stability, sliding under undrained conditions as well as bearing and global stability under both drained and undrained conditions were not satisfied at a strap length equal to 0.7 times the wall height. Increasing the width of the wall up to 1.3 times the wall height still did not satisfy all of the external and global stability requirements. As noted in Section 5.4 of the full report, consideration was given to over excavating these soils and replacing it with granular embankment; however, similar conditions are anticipated along the remainder of the alignment of Retaining Wall E4, which makes this a very expensive and uneconomical option. Recommendations have been provided in the structure foundation exploration report for Retaining Wall E4 to incorporate the use of ground improvement techniques to stabilize the existing fill and underlying cohesive soils. 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-70/71-13.10/14.36 (Projects 6A/6R) project in Columbus, Ohio. The projects represent the central portion of FRA-70-8.93 (PID 77369) I-70/71 south innerbelt improvements project, which includes all improvements along I-70 westbound from the I-71/SR-315 interchange to Front Street and along I-71 southbound from I-70 to Greenlawn Avenue. The FRA-71-14.36 (Project 6R) phase will consist of all work associated with the reconfiguration and construction of I-71 southbound from downtown (Front Street) to Greenlawn Avenue, including Ramps C3, D6 and D7. This project includes the construction of two (2) new bridge structures, one (1) for I-71 southbound over Short Street, NS/CXS Railroad and the Scioto River (FRA-71-1503L) and one (1) for Ramp D7 over Short Street (FRA-70-1373B), as well as the construction of five (5) new retaining walls (Walls E4, E5, E7, W2 and W5) to accommodate the new configuration.

This report is a presentation of the structure foundation exploration performed for the design and construction of the proposed FRA-70-1373B bridge structure carrying the proposed Ramp D7 over Short Street, as shown on the vicinity map and boring plan presented in Appendix I. The proposed Ramp D7 will be a two-lane ramp that will carry traffic from Mound Street to I-70 westbound. Based on information provided by CH2M HILL and ms consultants, it is understood that the proposed bridge consist of a two-span composite steel girder structure with a reinforced concrete deck and semi-integral rear abutment behind a mechanically stabilized earth (MSE) wall, cap and column pier and stub-type forward abutment. The proposed structure will have a total length of approximately 262 feet and width of approximately 37 feet. In addition, the roadway profile will be elevated approximately 26 feet above the existing ground surface grade at the rear abutment and approximately 15 feet above existing grade at the forward abutment. Please note that the analysis and recommendations for Retaining Wall E4 between Sta. 400+98 and 402+02 (BL Wall E4), at the rear abutment is presented under this report cover. In addition, Retaining Wall E5 connects to both ends of the forward abutment and extend west from both ends of the abutment foundation. Design recommendations for the remaining alignment of Retaining Wall E4 and Retaining Wall E5 are under separate covers.

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. 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 consisting of shale overlying limestone was encountered in the borings performed for this structure at elevations ranging from 649.6 to 656.0 feet msl.

2.2 Existing Conditions

Mound Street in the vicinity of the proposed structure is currently a three-lane, asphalt paved roadway that is aligned east-to-west, and Short Street is a two-lane, asphalt paved roadway that is aligned north-to-south. There is an existing electrical substation at the southeast corner of the intersection of Mound Street and Short Street that is owned and operated by American Electric Power (AEP), and there is an existing tennis court at the southwest corner of the intersection. The terrain along both roadways and the surrounding area is relatively flat-lying, and the existing Mound Street entrance ramp and I-70 roadway are elevated above the surrounding terrain on engineered embankments. Based on utility plans provided by ms consultants, there are many underground utilities with both roadways and also beneath the surrounding sidewalks, including the Olentangy Scioto Interceptor Sewer (OSIS), which runs north-to-south within the roadway of Short Street.

3.0 EXPLORATION

Between March 5, 2014, and January 12, 2015, three (3) borings, designated as B-020-4-13, B-020-6-13, and B-020-8-13, were drilled along the proposed bridge alignment at the locations shown on the boring plan provided in Appendix I of this report and summarized in Table 1. The borings were advanced to completion depths ranging from 94.7 to 102.0 feet below the existing ground surface at the respective boring location. Borings B-020-6-13 and B-020-8-13 were performed following completion of Subsurface Utility Engineering (SUE) Level A locating, in order to identify the physical location of existing electrical duct banks in the vicinity of the boring locations, which was required by AEP prior to performing the borings.

Table 1. Test Boring Summary

Boring Number	Station [1]	Offset [1]	Latitude	Longitude	Ground Elevation (feet msl)	Boring Depth (feet)
B-020-4-13	7007+77.61	12.0' Rt.	39.954037298	-83.004708660	714.0	94.7
B-020-6-13	7008+36.29	24.1' Rt.	39.954069154	-83.004498924	714.1	95.0
B-020-8-13	7010+15.89	30.7' Rt.	39.954179475	-83.003899896	721.0	102.0

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

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

The borings was drilled using a truck mounted rotary drilling machine, utilizing a 3.25-inch or 4.25-inch inside diameter, continuous hollow-stem auger to advance the holes. Upon encountering auger refusal on bounders in boring B-020-4-13, a 6.0-inch driven casing was used to advance the hole through and below the boulder zone. Standard penetration testing (SPT) and split spoon sampling were performed in the borings at 2.5-foot increments of depth to 30.0 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 hammer for the Mobile B-53 drill rig used for boring B-020-4-13 was calibrated on April 26, 2013, and has a drill rod energy ratio of 77.7 percent, and the hammer for the CME 55 drill rig used for borings B-020-6-13 and B-020-8-13 was calibrated on October 20, 2014, and has a drill rod energy ratio of 92.0 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	53
Plastic and Liquid Limits	AASHTO T89, T90	20
Gradation – Sieve/Hydrometer	AASHTO T88	20
Unconfined Compressive Strength of Intact Rock	ASTM D7012	7

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

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

The depth to bedrock was determined by split spoon sampler refusal. Split spoon sampler refusal is defined as exceeding 50 blows from the hammer with less than 6.0 inches of penetration by the split spoon sampler.

Where borings were extended into the bedrock (after encountering auger refusal), an HQ-sized double-tube diamond bit core barrel (utilizing wire line equipment) was used to core the bedrock. Coring produced 2.5 inch diameter cores, from which the type of rock and geological characteristics were determined.

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

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

4.0 FINDINGS

Interpreted engineering logs have been prepared based on the field logs, visual examination of samples and laboratory test results. Classification follows the 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

Boring B-020-4-13 was performed in a grass area between the sidewalk and pine trees at the southwest corner of the intersection of Mound Street and Short Street and encountered 2.0 inches of topsoil at the ground surface. Borings B-020-6-13 and B-020-8-13 were drilled through the existing sidewalk that runs along the south side of Mound Street, east of Short Street, and encountered 2.0 and 8.0 inches of concrete overlying 7.0 and 4.0 inches of aggregate base, respectively, at the ground surface.

4.2 Subsurface Soils

Beneath the surface materials in borings B-020-4-13, B-020-6-13 and B-020-8-13, material identified as existing fill was encountered extending to depths ranging from 5.5 to 10.5 feet below the existing ground surface. The fill material consisted of dark brown and brown gravel, gravel with sand and silt and silt and clay (ODOT A-1-a, A-2-4, A-6a) and contained organics, wood fibers and slag.

Underlying the existing fill, natural granular soils were encountered with intermittent seams of cohesive material. The granular soils were generally described as brown and gray gravel, gravel with sand, gravel with sand and silt and coarse and fine sand (ODOT A-1-a, A-1-b, A-2-4, A-3a). The cohesive soils were described as brown and gray silt and clay, silty clay and clay (ODOT A-6a, A-6b, A-7-6). Cobbles and boulders were generally encountered above the bedrock in borings B-020-4-13, B-020-6-13 and B-020-8-13 starting at elevations ranging from 686 to 696 feet msl. Due to the significant presence of large boulders in borings B-020-4-13 and B-020-8-13 at 55.5 and 37.0 feet below existing grade, respectively, mud rotary drilling techniques with casing advancer or tricone bit were utilized until bedrock was encountered.

The relative density of granular soils is primarily derived from SPT blow counts (N_{60}). Based on the SPT blow counts obtained, the granular soil encountered ranged from very loose (N_{60} < 5 blows per foot [bpf]) to very dense (N_{60} > 50 bpf). Overall blow counts recorded from the SPT sampling ranged from 3 bpf to split spoon sampler refusal. The shear strength and consistency of the cohesive soils are primarily derived from the hand penetrometer values (HP). The cohesive soil encountered ranged from very soft (HP ≤ 0.25 tsf) to hard (HP > 4.0 tsf). The unconfined compressive strength of the cohesive soil samples tested, obtained from the hand penetrometer, ranged from 0.25 to over 4.5 tsf (limit of instrument).

Natural moisture contents of the soil samples tested ranged from 1 to 29 percent. The natural moisture content of the cohesive soil samples tested for plasticity index ranged from 5 percent below to 10 percent above their corresponding plastic limits. In general, the soil exhibited natural moisture contents considered to be moderately below to significantly above optimum moisture levels.

4.3 Bedrock

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

Table 3. Top of Bedrock Elevations

Boring Number	Ground Surface Elevation (feet msl)	Top of Bedrock		Top of Bedrock Core (Auger Refusal)	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-020-4-13	714.0	64.4	649.6	74.7	639.3
B-020-6-13	714.1	60.4	653.7	60.0	654.1
B-020-8-13	721.0	65.0	656.0	65.0	656.0

Top of bedrock in the borings was encountered at elevations ranging from 649.6 to 656.0 feet msl. The upper portion of the bedrock encountered consists of shale with interbedded seams of mudstone and claystone overlying competent limestone bedrock.

Table 4 tabulates the depth and elevation that the surficial shale bedrock was encountered as well as the top of competent limestone bedrock. Shale bedrock was encountered in the borings at elevations ranging from 649.6 to 656.0 feet msl, and limestone bedrock was encountered at elevations ranging from 628.6 to 630.3 feet msl.

Table 4. Bedrock Types

Boring Number	Ground Surface Elevation (feet msl)	Top of Shale, Mudstone and Claystone		Top of Limestone	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-020-4-13	714.0	64.4	649.6	83.7	630.3
B-020-6-13	714.1	60.4	653.7	85.5	628.6
B-020-8-13	721.0	65.0	656.0	92.0	629.0

The upper 10.3 feet of shale bedrock in boring B-020-4-13 was able to be augered and sampled. The cored bedrock consists of shale, mudstone, claystone and limestone. The mudstone is described as gray, slightly weathered to unweathered, very weak to weak, medium to thick bedded, calcareous, fissile, friable, argillaceous, and moderately fractured to intact with tight to open, slickensided to very rough apertures. The claystone is described as dark gray, slightly weathered, very weak, thin bedded, calcareous and fractured with tight, slickensided to slightly rough apertures. The shale is described as gray and dark gray, highly weathered to unweathered, very weak to strong, laminated to very thick bedded, calcareous, argillaceous, arenaceous, fissile, crystalline, pyritic and slightly to highly fractured with tight to open, slickensided to very rough apertures. The limestone is described as gray, brown, dark brownish gray, unweathered, moderately strong to very strong, thin to very thick bedded, arenaceous, siliceous, crystalline, dolomitic, cherty, pyritic, ferriferous and slightly to moderately fractured with intact to open, slightly rough apertures.

The percent recovery, RQD values and unconfined compressive strengths of the bedrock core runs from the current exploration borings are summarized in Table 5.

Table 5. Rock Core Summary

Boring	Core No.	Depth (feet)	Recovery (%)	RQD (%)	Unconfined Compressive Strength
B-020-4-13	RC-1	74.7 to 79.7	92	75	N/A
	RC-2	79.7 to 84.7	100	100	N/A
	RC-3	84.7 to 89.7	95	90	q_u @ 86.5’ = 13,130 psi
	RC-4	89.7 to 94.7	95	85	q_u @ 90.7’ = 16,178 psi

Boring	Core No.	Depth (feet)	Recovery (%)	RQD (%)	Unconfined Compressive Strength
B-020-6-13	RC-1	60.0 to 65.0	90	60	N/A
	RC-2	65.0 to 70.0	33	0	N/A
	RC-3	70.0 to 75.0	100	64	N/A
	RC-4	75.0 to 80.0	97	63	q_u @ 77.4’ = 177 psi
	RC-5	80.0 to 85.0	83	57	N/A
	RC-6	85.0 to 90.0	100	89	N/A
	RC-7	90.0 to 95.0	100	100	q_u @ 91.5’ = 12,531 psi
B-020-8-13	RC-1	65.0 to 67.0	52	33	N/A
	RC-2	67.0 to 72.0	97	72	q_u @ 71.2’ = 275 psi
	RC-3	72.0 to 77.0	100	83	N/A
	RC-4	77.0 to 82.0	90	68	N/A
	RC-5	82.0 to 87.0	95	78	q_u @ 85.9’ = 318 psi
	RC-6	87.0 to 92.0	52	8	N/A
	RC-7	92.0 to 97.0	100	100	N/A
	RC-8	97.0 to 102.0	100	100	q_u @ 97.1’ = 4,737 psi

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

4.4 Groundwater

Groundwater was encountered in the borings as presented in Table 6.

Table 6. Groundwater Levels

Boring Number	Ground Elevation (feet msl)	Initial Groundwater		Upon Completion	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-020-4-13	714.0	18.5	695.5	N/A [1]	N/A
B-020-6-13	714.1	26.0	688.1	N/A [1]	N/A
B-020-8-13	721.0	18.5	702.5	N/A [1]	N/A

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

Groundwater was encountered initially during drilling in the borings at depths ranging from 18.5 to 56.0 feet below existing grade. The groundwater levels at the completion of drilling could not be measured due to the addition of mud to counteract heaving sands as well as water as a circulating fluid during the rock coring process. Please note that short-term water level readings, especially in cohesive soils, are not necessarily an accurate indication of the actual groundwater level. In addition, groundwater levels or the presence of groundwater are considered to be dependent on seasonal fluctuations in precipitation.

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

5.0 ANALYSES AND RECOMMENDATIONS

Data obtained from the subsurface exploration 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 bridge, 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 structure were provided by CH2M HILL and ms consultants. Based on the information provided, it is understood that the proposed bridge consist of a two-span composite steel girder structure with a reinforced concrete deck and semi-integral rear abutment behind a mechanically stabilized earth (MSE) wall, cap and column pier and stub-type forward abutment. The roadway profile will be elevated approximately 26 feet above the existing ground surface grade at the rear abutment and approximately 15 feet above existing grade at the forward abutment. As previously noted, the analysis and recommendations for Retaining Wall E4, between Sta. 400+98 and 402+02 (BL Wall E4), at the rear abutment is presented in this report.

Driven piles were considered for support of all of the substructure units for this structure. However, due to the significant presence of underground utilities within the roadway of both Mound Street and Short Street and the proximity of the proposed substructure units to these facilities, pile driving at the pier and forward abutment substructure locations was considered unfavorable due to the potential for excessive vibration and the potential for damage or need for relocation. Therefore, drilled shafts are recommended for foundation support at these locations. Proposed structural data was obtained from design details provided by ms consultants and are included in Table 7.

Table 7. Structure and Bridge Design Elevations

Substructure Unit	Structure Component [1]	Elevation [1,2] (feet msl)	Design Maximum Factored Load
Rear Abutment / Retaining Wall E4 (Sta. 401+98 to 402+02) (B-020-4-13)	Profile Grade	741.5	250 kips/pile
	Bottom of Footing	730.0	
	Bottom of Wall (Top of Leveling Pad)	712.0	
Pier (B-020-6-13)	Top of Shaft	713.8	1,600 kips/shaft
Forward Abutment (B-020-8-13)	Bottom of Footing / Top of Shaft	720.0	1,100 kips/shaft

1. *Proposed foundation elevations and structural loading based on structure information provided by CH2M HILL and ms consultants.*

5.1 Drilled Shaft Recommendations

5.1.1 Drilled Shafts Extended to Limestone Bedrock

It is understood that drilled shaft foundations are being utilized to support the pier and forward abutment substructure units due to the significant presence of underground utilities within the footprint of the substructure units. It is recommended that the drilled shafts extend through the surficial soils, and be socketed a minimum of 1.5 shaft diameters into the underlying shale, mudstone, claystone or limestone bedrock, as specified in Section 10.8.3.5.4c of the 2014 AASHTO LRDF BDS, and end bear on or within the competent limestone bedrock encountered below the shale, mudstone and claystone. Based on plan information provided by CH2M HILL and ms consultants, the shaft diameter within the overburden soils will be 3.5 feet, and the shaft diameter within the rock socket will be 3.0 feet. Table 8 lists the estimated elevation of the top of limestone bedrock and corresponding embedment depth to the top of limestone bedrock at the pier and forward abutment substructure units.

Table 8. Drilled Shaft Recommendations

Substructure Unit	Top of Shaft Elevation [1] (feet msl)	Top of Bedrock Elevation [2] (feet msl)	Top of Limestone Elevation (feet msl)	Shaft Tip Elevation [3] (feet msl)	Shaft Length [4] (feet)
Pier (B-020-6-13)	713.8	653.7	628.6	628.6	85.2
Forward Abutment (B-020-8-13)	720.0	656.0	629.0	629.0	91.0

1. *Top of shaft elevation based on structure information provided by CH2M HILL and ms consultants.*
2. *Top of bedrock elevation indicates the top of shale, mudstone or claystone bedrock elevation where coring techniques will likely be required to advance the shaft excavations.*
3. *Shaft tip elevation based on a minimum rock socket length of 1.5 shaft diameters into the shale, mudstone, claystone or limestone bedrock, and the shaft tip end bearing on or within the competent limestone bedrock encountered below the shale, mudstone and claystone.*
4. *Shaft length represents the overall length of shaft through the overburden soil and bedrock socket within the shale, mudstone, claystone or limestone bedrock.*

Using equation 10.8.3.5.4c-1 of the AASHTO LRFD BDS, the nominal end bearing resistance for intact rock is 2.5 times the unconfined compressive strength of the rock. Based on unconfined compression tests performed on limestone rock cores obtained from borings B-020-4-13, B-020-6-13 and B-020-8-13, the unconfined compressive strength of the limestone bedrock ranges from 4,737 to 16,173 psi. Using the above noted equation and the limiting unconfined compressive strength from the given range for the limestone bedrock, it is recommended that drilled shaft foundations socketed into competent limestone bedrock be proportioned for a maximum nominal end bearing resistance of 1,705 ksf at the strength limit state. A resistance factor of $\varphi = 0.50$ at the strength limit state should be utilized for design.

Given the factored end bearing resistance noted above for drilled shafts extended to bear on or within the limestone bedrock, it is anticipated that the axial resistance will be governed by structural resistance of the drilled shaft. Group efficiency of the shafts, socketed into bedrock, is considered negligible for a minimum spacing of 2.0 shaft diameters center-to-center. Drilled shafts designed in accordance with the requirements presented above should experience a maximum settlement estimated to be less than 0.5 inches.

5.1.2 Drilled Shafts Extended to Shale Bedrock

If the drilled shaft lengths required to end bear in the limestone bedrock are not economically feasible, then consideration can be given to extending the drilled shafts through the surficial soils to bear in the shale/mudstone/claystone bedrock encountered at an elevation of 653.7 and 656.0 feet msl in borings B-020-6-13 and B-020-8-13, respectively. Given the weak nature of this type of rock encountered in the borings, it is recommended that the drilled shafts be designed using a combination of end bearing and side resistance within the bedrock socket. The shafts should be socketed a minimum of 2.0 shaft diameters into the shale/mudstone/claystone bedrock, and extended beyond this minimum socket length depending on the vertical and lateral load demands on the drilled shaft. Based on plan information provided by CH2M HILL and ms consultants, the shaft diameter within the overburden soils will be 3.5 feet, and the shaft diameter within the rock socket will be 3.0 feet.

Based on unconfined compression tests performed on shale rock cores obtained from borings B-020-6-13 and B-020-8-13, the unconfined compressive strength of the shale bedrock ranges from 177 to 318 psi. Using equation 10.8.3.5.4c-1 of the AASHTO LRFD BDS and the limiting unconfined compressive strength from the given range for the shale bedrock, it is recommended that drilled shaft foundations socketed into the shale/mudstone/claystone bedrock be proportioned for a nominal end bearing resistance of 60 ksf at the strength limit state. A resistance factor of φ = 0.50 at the strength limit state should be utilized for design. The nominal side resistance, q_s, for drilled shafts socketed into rock was calculated using equation 10.8.3.5.4b-1 of the AASHTO LRFD BDS as follows:

$$\frac{q_s}{p_a} = C\sqrt{\frac{q_u}{p_a}}$$

Where:
p_a = atmospheric pressure (2.12 ksf)
C = regression coefficient taken as 1.0 for normal conditions
q_u = unconfined compressive strength of rock (ksf)

Applying the above noted equation and utilizing a regression coefficient of 1.0 and the limiting unconfined compressive strength from the given range above for the shale/mudstone/claystone bedrock, a nominal side resistance of 7.0 ksf and resistance factor of φ = 0.55 at the strength limit state should be utilized for the portion of the shafts that will extend into the shale/mudstone/claystone.

Due to the difference in the required displacement to mobilize side friction in the overburden soil, the calculated resistance should only consider end bearing and side resistance within the bedrock socket. Side resistance within the overburden soils should be neglected.

Drilled shafts designed in accordance with the requirements presented above should experience a maximum settlement estimated to be less than 1.0 inch.

5.1.3 Drilled Shafts Bearing Above Bedrock

If it is not desired to extend the drilled shafts into the underlying shale or limestone bedrock due to the depth or effort required, it is recommended that the drilled shafts be designed using the axial design parameters provided in Table 9. To achieve the most economical design, the drilled shafts should extend to bear in the very dense gravel, gravel and sand (ODOT A-1-a, A-1-b) or hard clay (ODOT A-7-6) at the corresponding elevations noted below in order to maximize the end bearing resistance. The drilled shafts should be proportioned for a nominal end bearing and side resistance as follows:

Table 9. Drilled Shaft Axial Design Parameters

Substructure Unit	Elevation [1] (feet msl)	Shaft Length (feet)	Soil Type	Nominal Resistance (ksf)		Resistance Factor	
				End	Side	End	Side
Pier (B-020-6-13)	714.1-708.6	0.0-5.5	A-6a	9	0.68 [2]	0.40	0.45
	708.6-703.6	5.5-10.5	A-1-a	15	0.71	0.50	0.55
	703.6-693.6	10.5-20.5	A-7-6	18	1.10	0.40	0.45
	693.6-688.6	20.5-25.5	A-1-a	31	1.66	0.50	0.55
	688.6-654.1	25.5-60.0	A-1-a	60	5.18	0.50	0.55
Forward Abutment (B-020-8-13)	720.0-715.5	0.0-4.5	A-2-4	9	0.32	0.50	0.55
	715.5-700.5	4.5-19.5	A-1-a	48	1.91	0.50	0.55
	700.5-689.0	19.5-31.0	A-3a	54	1.91	0.50	0.55
	689.0-669.0	31.0-51.0	A-1-b	60	5.02	0.50	0.55
	669.0-656.0	51.0-64.0	A-7-6	36	2.04	0.40	0.45

1. *Top of shaft elevation based on structure information provided by CH2M HILL and ms consultants.*
2. *Side resistance should be neglected for the upper 5.0 feet of the shaft length where cohesive soils (ODOT A-4a, A-4b, A-6a, A-6b, A-7-6) are present below the bottom of footing/top of shaft elevation.*

It is recommended with this option that the drilled shafts be extended to bear at or below an elevation of 689.0 feet msl. Drilled shaft lengths should measure a minimum of three (3) times the shaft diameter. Total settlement of the drilled shafts is estimated to be less than 1.0 inch for shafts bearing at or below this elevation.

Per Section 10.8.3.5.3 of the AASHTO LRFD BDS, where drilled shafts are extended to end bear in a strong soil layer overlying a weaker soil layer, the end bearing resistance shall be reduced if the tip elevation is within 1.5 times the diameter of the drilled shaft above the top of the weaker soil layer. A weighted average that varies linearly from the full end bearing resistance in the overlying strong soil layer at a distance of 1.5 times the diameter of the drilled shaft above the top of the weak soil layer to the end bearing resistance of the weak soil layer at the top of the weak soil layer should be used to determine the end bearing resistance utilized in the design. Therefore, the end bearing resistance utilized in the design will need to be adjusted accordingly if the tip elevation of the drilled shafts will be within 1.5 times the diameter of the drilled shaft above the underlying weaker soil layer.

Per Section 10.8.3.5.1b of the AASHTO LRFD BDS, side resistance should be neglected for the upper 5.0 feet of the shaft length where cohesive soils (ODOT A-4a, A-4b, A-6a, A-6b, A-7-6) are present below the bottom of footing/top of shaft elevation. Additionally, based on the estimated settlement noted above, it is anticipated that 100 percent of the side friction resistance will be mobilized and approximately 70 percent of the end bearing resistance will be mobilized based on the displacement of the shaft. Therefore, the nominal end bearing resistance noted Table 9 should be reduced to 70 percent of the values provided for the respective tip elevation in the determination of the design shaft resistance. Drilled shaft calculations are provided in Appendix V.

5.1.3.1 Group Resistance

The axial resistance of a group of shafts may be less than the sum of the individual shaft resistance within a group of shafts. Per Section 10.8.3.6.3 of the AASHTO LRFD BDS, for soil profiles that consist of primarily granular soils, the individual nominal resistance of each drilled shaft shall be reduced by applying an adjustment factor, η, as defined in Table 10.8.3.6.1-1 of the AASHTO LRFD BDS. The following criteria are recommended for the group resistance of any shaft groups:

- η = 0.9 for a center-to-center spacing of 2.0 diameters,
- η = 1.0 for a center-to-center spacing of 3.0 diameters or greater,
- For intermediate spacing, the value of η may be determined by liner interpolation.

Please note that the adjustment factor should be applied to the total individual nominal shaft resistance (including both end bearing side resistance along the shaft length). It is not recommended to use a center-to-center spacing less than 2.0 diameters of the shaft for drilled shafts bearing above bedrock.

5.1.4 Downdrag Considerations

Based on information provided by CH2M HILL, it is understood that the forward abutment wall will be constructed prior to the construction of the adjacent Retaining Wall E5. Retaining Wall E5 will consist of two independent MSE walls that will be constructed parallel to each other and will support Ramp D7. The walls will connect to the back of the forward abutment wall and will have a maximum height of 19.3 feet at the connection with the forward abutment wall. Downdrag was evaluated using the traditional method to determine the depth of downdrag. Per the traditional method for calculating the depth of downdrag, downdrag loads will develop along the portion of the shaft above the interface where the relative soil movement from consolidation with respect to the shaft is greater than 0.40 inches. The anticipated settlement below the foundation at the forward abutment due to the weight of the MSE wall backfill behind the abutment is 0.57 inches. Based on the results of the settlement analysis, downdrag loads will develop along the drilled shafts supporting the proposed forward abutment. Using the traditional method criterion, the depth of downdrag at the forward abutment for 100 percent of primary consolidation is calculated to be 18.5 feet below the bottom of footing elevation.

The unfactored downdrag load induced on the shafts was calculated using static analysis and is equal to the magnitude of the side resistance over the length of the shaft within the downdrag zone provided above. The side resistance values provided in Table 9 for the forward abutment (boring B-020-8-13) were utilized in the calculation of the downdrag load. Considering a 3.5-foot shaft diameter within the overburden soils, the unfactored downdrag load is 310 kips. A load factor of 1.25 should be utilized in accounting for the factored downdrag load. For shafts extended to bear in bedrock, the factored downdrag load should be added to the factored structural loading and only the factored end bearing and/or side resistance within the bedrock socket should be considered in calculating the factored shaft resistance. For friction bearing drilled shafts bearing within the overburden soils above the bedrock, the factored downdrag load should be added to the factored structural loading and only the factored ending bearing and side resistance below the downdrag depth should be considered in calculating the factored shaft resistance.

5.1.5 Drilled Shaft Considerations

The minimum requirements for proper inspection of drilled shaft construction are as follows:

- A qualified inspector should record the material types being removed from the hole as excavation proceeds.
- When the bearing material has been encountered and identified and/or the design tip elevation has been reached, the shaft walls and base should be observed for anomalies, unexpected soft soil conditions, obstructions or caving.
- Concrete placed freefall should not be allowed to hit the sidewalls of the excavation or the rebar cage and should not pass through any water.
- Structural stability of the rebar cage should be maintained during the concrete pour to prevent buckling.
- The volume of concrete should be checked to ensure voids did not result during extraction of the casing (if utilized).

- The placement of all concrete for the drilled shafts shall follow the American Concrete Institute's Design and Construction of Drilled Piers (ACI 336.3R-93).

- If concrete is placed by tremie method, it must be done so with an adequate head to displace water or slurry if groundwater has entered the caisson (all tremie procedures shall follow applicable ACI specifications).

- Pulling casing with insufficient concrete inside should be restricted.

- The bottom of drilled shaft excavation should be clean and free of loose material. Any loose material observed should be removed using a clean-out bucket (muck bucket).

The use of casing for drilled shafts is recommended under any of the following conditions:

- Caving material is encountered at any time during the drilling of the shaft.

- Groundwater is encountered at any time during the drilling of the shaft, or groundwater seepage occurs in the drilled shaft.

- Down hole inspection is planned (casing is required for this instance).

In addition, it is recommended that if casing is used, it be pulled immediately after the concrete is placed, allowing for re-use of the casing and eliminating reduction of side resistance (between soil and concrete).

It is anticipated that conventional drilled shaft equipment (with a standard soil bit) will be able to penetrate the surficial soils to the bedrock depths provided in Table 3. However, depending on the conditions encountered, additional effort may be needed at or above this depth. Below the depths noted, it will likely be necessary to employ more specialized drilling techniques, such as the use of rock teeth or a rock bit. The ability to penetrate the bedrock will be entirely dependent on the drilled shaft contractor and the equipment employed. It is the responsibility of the contractor to determine the most effective excavation procedures. The elevation and hardness of bedrock is subject to change within the project area.

As noted in Section 4.2, large boulders were encountered in borings B-020-4-13 and B-020-8-13 at 55.5 and 37.0 feet below existing grade, which corresponds to elevations of 656.5 and 683.0, respectively, which required special drilling techniques to advance the borings below the depths noted in the respective boring. Therefore, boulders should be anticipated to be encountered during installation of the drilled shafts. If boulders are encountered during installation of the drilled shafts, specialized drilling/coring equipment may be required to advance the drilled shaft excavation beyond the obstruction.

5.2 Driven Pile Recommendations

It is understood that driven piles will be utilized at the rear abutment substructure unit. Given the depth of bedrock encountered in boring B-020-4-13, it is recommended that steel H-piles (ODOT Item 507.06) driven to refusal on bedrock be employed for foundation support. Per Section 202.2.3.2a of the 2019 ODOT Bridge Design Manual, refusal is met during driving when the pile penetration is an inch or less after receiving at least 20 blows from the pile hammer. Table 10 shows recommended pile lengths and the corresponding factored structural axial resistance ($R_{R\ max}$) of steel H-piles and associated resistance factors (φ):

Table 10. Driven Pile Recommendations

Substructure Unit	Ground Elevation [1] (feet msl)	Pile Size	Pile Elevation		Pile Length [4] (feet)	$R_{R\ max}$ [5] (kips/pile)	Sleeve Length [6]	φ [7]
			Top [2]	Tip [3]				
Rear Abutment (B-020-4-13)	714.0	HP 10x42 [8]	731.0	646.0	85	310	18.0	N/A

1. *Ground elevation listed is the ground elevation at the respective boring locations.*
2. *The top of pile elevation corresponds to the pile cutoff elevation, which is assumed to be 1.0-foot above the proposed bottom of footing elevation.*
3. *The pile tip elevation for steel H-piles driven to refusal on bedrock is based on a penetration of 4.0 feet into the weathered shale bedrock.*
4. *Per Section 202.3.2 of the 2019 ODOT BDM, the estimated pile length was determined as the pile cutoff elevation (top) minus the pile tip elevation, rounded up to the nearest 5.0 feet.*
5. *The factored structural axial resistance for H-piles is based on the structural limit state of the steel H-pile section per Section 202.2.3.2.a of the 2019 ODOT BDM.*
6. *Sleeve length represents the required length of pile that should be sleeved within the MSE wall backfill.*
7. *For H-piles driven to refusal on bedrock, no geotechnical resistance factor should be applied to the factored structural axial resistance values presented, as the values presented account for the structural resistance factor, φ_c = 0.50, for H-piles subject to damage due to severe driving conditions.*
8. *A steel pile point is recommended to protect the tips of the steel H-piles during pile installation.*

Per Section 202.2.3.2.a of the 2019 ODOT BDM, the factored resistance of H-piles driven to refusal on bedrock is typically governed by the structural resistance of the pile element. The factored structural axial resistances listed in Table 10 consider an axially loaded pile with negligible moment, no appreciable loss of section due to deterioration throughout the life of the structure, a steel yield strength of 50 ksi, a structural resistance factor for H-piles subject to damage due to severe driving conditions (LRFD 6.5.4.2: φ_c = 0.50) and a pile fully braced along its length. **These bearing values should not be used for piles that are subjected to bending moments or are not supported by soil for their entire length.** Static or dynamic load testing is not required for H-piles driven to refusal on bedrock. It is anticipated that the piles will be able to be driven a short distance into the surficial bedrock before satisfying the driving conditions that meet the refusal criterion. Due to the weathered, variable nature of the upper shale bedrock, it is estimated that refusal will be met within the upper 3.0 to 5.0 feet of the surficial bedrock. Therefore, the recommended pile tip elevation is based on a penetration of 4.0 feet into the weathered shale bedrock. Settlement is estimated to be less than 0.5 inches for H-piles driven to refusal on bedrock.

5.2.1 Downdrag Considerations

The anticipated total settlement at the facing of the MSE wall at the rear abutment is 2.19 inches. Given the anticipated amount of settlement at the MSE wall facing, downdrag loads may be induced on the pile elements if installed to the final tip elevation prior to construction of the wall. To reduce the amount of downdrag induced on the piles, it is recommended that the piles be pre-driven into the soil only as far as necessary to remain vertical and that the MSE wall should be constructed around the piles and then allowed to sit for a specified holding period such that a percentage of the consolidation can occur prior to driving the piles the to the design tip elevation and reduce the amount of downdrag on the piles. In order to consolidate the underlying soil to the required settlement, consideration should be given to the placement of a surcharge load in order to preload the site under the full weight of the MSE wall height (from the bottom of wall elevation to the profile grade) and left in place until approximately 90 percent of consolidation of the subsurface soils has occurred to prevent downdrag loads from developing along the pile elements. Results of the settlement analysis indicate that approximately 90 percent of the primary consolidation of the cohesive layers at the rear abutment will be complete within 15 days following the placement if the surcharge load. Therefore, if the above noted waiting period is specified following completion of construction of the MSE wall at the rear abutment, downdrag forces along the piles will be eliminated.

Settlement platforms should be installed once the embankment surcharge has been placed to monitor the settlement of the embankment over time. A shorter or longer hold period than specified may be required based on the settlement platform readings as directed by the geotechnical engineer. The required hold period may be considered complete when survey monitoring of the settlement platforms indicate that the above noted settlement has occurred for the hold period or until the survey shows less than ⅛-inch of total movement per week over a two week period **following placement of the final lifts of surcharge loading.**

5.2.2 Driveability

A drivability analysis was performed in accordance with Section 10.7.8 of the 2018 AASHTO LRFD Bridge Design Specifications (BDS) using the GRLWEAP software program, and the results are provided in Appendix VI. In the driveability analysis, a Delmag 19-42 hammer with a rated energy of approximately 43,000 ft-lbs was used in conjunction with the H-pile sections. Based on the results of this analysis, driving stresses induced on the H-piles **would not exceed** 90 percent of the yield stress of the steel (f_y = 50 ksi, 0.9f_y = 45 ksi) if driven through the overburden soils to the bedrock elevation provided in Table 10. Care should be taken during pile driving operations when approaching the bedrock, and when extending the piles into the surficial bedrock material, to ensure that the driving stresses induced on the pile elements do not exceed the maximum allowable value of 90 percent of the yield stress of the steel, subsequently damaging the pile elements. Pile driving should be terminated upon achieving the required 20 blows from the pile hammer with an inch or less of penetration to reduce the possibility of damaging the pile element.

Per Section 202.2.3.2.a of the 2019 ODOT BDM, steel pile points should not be used when the piles are driven to bear on shale bedrock. However, due to the presence of cobbles and boulders encountered in the majority of the borings performed within the project corridor, it is recommend that the steel H-piles piles be driven with pile points in order to reduce damage to the piles during driving. With the use of pile points, the piles will likely penetrate further into the weathered shale bedrock than the anticipated 4.0 feet recommended in Table 10 prior to satisfying the refusal criterion. Given the condition of the shale bedrock, it is estimated that the piles will be able to penetrate an additional 4.0 feet into the shale bedrock (4.0 feet below the pile tip elevation provided in Table 10) prior to satisfying the refusal criterion if steel pile points are utilized.

5.3 Lateral Design

If lateral loads or moments are expected to be applied on the foundation elements, they should be analyzed to verify the shaft or pile has enough lateral and bending resistance against these loads. A boring-by-boring tabulation of parameters that should be used for lateral loading design is provided in Appendix VII. In order to evaluate the lateral capacity, it is recommended that a derivation of COM624, such as LPILE, be utilized to determine the proper embedment depth and cross section (for drilled shafts) required to resist the lateral load for a given end condition and deflection. Table 11 lists the different soil types internal to the LPILE program. These strata were utilized to define the soil strata in the soil profile for each boring provided in Appendix VII.

Table 11. Subsurface Strata Description

Strata	Description
1	Soft Clay
2	Stiff Clay with Water
3	Stiff Clay without Free Water
4	Sand (Reese)
5	User Defined
6	Vuggy Limestone (Strong Rock)
7	Silt (with cohesion and internal friction angle)
8	API Sand
9	Weak Rock
10	Liquefiable Sand (Rollins)
11	Stiff Clay without free water with a specified initial K (Brown)

5.4 MSE Wall Recommendations

It is proposed to construct an MSE wall at the rear abutment (Retaining Wall E4 between Sta. 400+98 and 402+02, BL Wall E4) of the proposed bridge structure. MSE walls are constructed on earthen foundations at a minimum depth of 3.0 feet below grade, as defined by the top of the leveling pad to the ground surface located 4.0 feet from the face of the wall. Per Section 204.6.2.1 of the 2019 ODOT BDM, the height of the MSE wall at the bridge abutments 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 bottom of footing elevation. Additionally, per Section 303.5.1 of the 2019 ODOT BDM, a minimum of one row of soil reinforcement straps should be attached to the backside of the abutment footing to resist horizontal forces from the bridge structure and lateral pressures along the back wall of the abutment footing, and prevent any load transfer from these forces to the coping and facing panels. The width of the MSE wall foundation (B) is defined by the length of the reinforced soil mass. Per the Section 204.6.2.1 of the 2019 ODOT BDM and Supplemental Specification (SS) 840, the minimum length of the reinforced soil mass is equal to 70 percent of the height of the MSE wall or 8.0 feet whichever is greater. A non-structural bearing leveling pad consisting of a minimum of 6.0-inches of unreinforced concrete should be placed at the base of the wall facing for constructability purposes. Please note that the leveling pad is not a structural foundation.

Based on the proposed plan information provided by CH2M HILL and ms consultants, the wall height at the rear abutment is anticipated to be 29.5 feet from the top of the leveling pad to the proposed profile grade of the roadway. For the analysis, the foundation width was set at 70 percent of the wall height and the foundation width was increased, if required, until external and global stability requirements were satisfied.

Per Section 840.06.D of ODOT SS 840, the foundation subgrade should be inspected to verify that the subsurface conditions are the same as those anticipated in this report.

Based on the conditions encountered in boring B-020-4-13, existing fill material was encountered at the proposed bearing elevation at the rear abutment (Wall E4), which extends to a depth of 6.0 feet below the proposed bottom of wall elevation (El. 706 feet msl). The fill material consisted of stiff silt and clay (ODOT A-6a) and contained organic material and wood fibers. Underlying the fill material, very stiff clay (ODOT A-7-6) overlying very loose to loose gravel with sand and silt (ODOT A-2-4) was encountered extending to a depth of 16.0 feet below the proposed bearing elevation (El. 696 feet msl). 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, similar conditions are anticipated along the remainder of the alignment of Retaining Wall E4 where the alignment overlies the surrounding grade outside of the existing I-70 embankment, which makes this a very expensive and uneconomical option. Recommendations have been provided in the structure foundation exploration report for Retaining Wall E4 to incorporate the use of ground improvement techniques to stabilize the existing fill and underlying cohesive soils. The recommendations for this alternative should govern the design of this portion of the wall as well. For this report, the analysis this section of Wall E4 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 wall is located within an existing floodplain, the analyses were performed using a design groundwater level at the ground surface.

5.4.1 Strength Parameters Utilized in External and Global Stability Analyses

The shear strength parameters utilized in the external and global stability analyses for the MSE wall at the rear abutment are provided in Table 12.

Table 12. Shear Strength Parameters Utilized in MSE Wall Stability Analyses

Material Type	γ (pcf)	φ' (1) (°)	c' (2) (psf)	S_u (3) (psf)
MSE Wall Backfill (Select granular fill)	120	34	0	N/A
Item 203 Embankment Fill (Retained soil)	120	30	0	2,000
Ex. Fill: Stiff Silt and Clay (ODOT A-6a)	115	26	0	625
Very Stiff Clay (ODOT A-7-6)	120	25	0	1,750
Loose to Dense Gravel and Sand, Gravel with Sand and Silt (A-1-b, A-2-4)	120 to 130	32 to 39	0	N/A
Very Dense Gravel (A-1-a)	135	43	0	N/A
Hard Silty Clay (ODOT A-6b)	130	28	100	8,000

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

Shear strength parameters for the reinforced soil backfill and retained embankment are provided in ODOT SS 840. Per SS 840, the select granular backfill in the reinforced zone and the retained embankment must meet the shear strength requirements provided in Table 12. The shear strength parameters for the natural soils were assigned using correlations provided in FHWA Geotechnical Engineering Circular (GEC) No. 5 (FHWA-NHI-16-072) Evaluation of Soil and Rock Properties and based on past experience in the vicinity of the site with projects performed in similar subsurface profiles.

5.4.2 Bearing Stability

The anticipated bearing materials at the rear abutment are anticipated to consist of existing fill comprised of stiff silt and clay (ODOT A-6a) that contained organics and wood fibers overlying very stiff clay (ODOT A-7-6). As noted in Section 5.4, it is understood that ground improvement techniques will be implemented along the alignment of Retaining Wall E4, including where the wall crosses the rear abutment. 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 factored bearing resistance as indicated in Table 13. A geotechnical resistance factor of φ_b=0.65 was considered in calculating the factored bearing resistance at the strength limit state.

Table 13. FRA-70-1373B MSE Wall Design Parameters

Substructure Element	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]	
Rear Abutment / Retaining Wall E4 (Sta. 400+98 to 402+02) (B-020-4-13)	29.5	Level	20.7 (0.70H)	3.71	2.41	7.00

1. *The required foundation width is expressed as a percentage of the wall height, H.*
2. *A geotechnical resistance factor of φ_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 service limit state.*

Rii performed a verification of the bearing pressure exerted on the subgrade soils for the maximum specified wall height indicated in Table 13. Based on the minimum length of reinforced soil mass presented, the factored equivalent bearing pressure exerted below the wall **will exceed** the factored bearing resistance at the strength limit state, considering the wall will bear on the existing fill material.

5.4.3 Settlement Evaluation

The compressibility parameters utilized in the settlement analysis of the proposed MSE walls are provided in Table 14.

Table 14. Compressibility Parameters Utilized in Settlement Analysis

Material Type	γ (pcf)	LL (%)	C_c (1)	C_r (2)	e_o (3)	C_v (4) (ft²/yr)	N_{60}	C' (5)
Ex. Fill: Stiff Silt and Clay (ODOT A-6a)	115	32	0.198	0.030	0.522	600	N/A	N/A
Very Stiff Clay (ODOT A-7-6)	120	48	0.342	0.034	0.647	150	N/A	N/A
Loose Gravel with Sand and Silt (ODOT A-2-4)	120	N/A	N/A	N/A	N/A	N/A	7	55 to 56
Medium Dense to Very Dense Gravel and Gravel with Sand (ODOT A-1-a, A-1-b)	135	N/A	N/A	N/A	N/A	N/A	18 to 88	71 to 338
Hard Silty Clay (ODOT A-6b)	130	40	0.270	0.027	0.585	300	N/A	N/A

1. *Per Table 6-9, Section 6.14.1 of FHWA GEC 5.*
2. *Estimated at 10% of C_c 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 15. Total settlements of up to 10.91 inches at the center of the reinforced soil mass and 5.66 inches at the facing of the wall are anticipated at the rear abutment. Based on the results of the analysis, 90 percent of the total settlement at the rear abutment is anticipated to occur over a period of approximately 30 days. Please note that the consolidation settlement and time rate of consolidation are based on estimates using correlated compressibility parameters provided in Table 14 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 15. FRA-70-1373B MSE Wall Settlement Results

Substructure Unit [1] (Boring)	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	
Rear Abutment / Retaining Wall E4 (Sta. 400+98 to 402+02) (B-020-4-13)	29.5	Level	4.90	10.91	5.66	30

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." Given the amount of settlement anticipated at the facing along Wall E4, 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 and underlying high plasticity clay 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 VIII.

5.4.4 Eccentricity (Overturning Stability)

The resistance of the MSE wall to overturning will be dependent on the location of the resultant force at the bottom of the wall due to the overturning and resisting moments acting on the wall. For MSE walls, overturning stability is determined by calculating the eccentricity of the resultant force from the midpoint of the base of the wall and comparing this value to a limiting eccentricity value. Per Section 11.10.5.5 of the 2018 AASHTO LRFD BDS, for foundations bearing on soil, the location of the resultant of the reaction forces shall be within the middle two-thirds ($^2/_3$) of the base width. Therefore, the limiting eccentricity is one-third ($^1/_3$) of the base width of the wall. Rii performed a verification of the eccentricity of the resultant force for the maximum specified wall height indicated in Table 13. Based on the minimum length of reinforced soil mass presented in Table 13 and utilizing the soil parameters listed in Section 5.4.1 for the retained embankment material, the calculated eccentricity of the resultant force **will not exceed** the limiting eccentricity at the strength limit state.

5.4.5 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 5.4.1 for the foundation and reinforced soil backfill, a coefficient of sliding friction of 0.49 was utilized for design. The sliding resistance was also evaluated under undrained conditions as well. For undrained conditions, the sliding resistance is taken as the limiting value between the undrained shear strength of the bearing soil and half of the vertical stress applied by the wall multiplied by the width of the MSE wall. Based on the soil parameters listed in Section 5.4.1, the undrained shear strength of the existing silt and clay fill material encountered at the proposed bearing elevation is estimated to be 0.625 ksf.

A geotechnical resistance factor of φ_τ=1.0 was considered when calculating the factored shear resistance between the select granular backfill and foundation soil for sliding. Based on the minimum length of reinforced soil mass presented in Table 13 and utilizing the soil parameters listed in Section 5.4.1 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. However, the resultant horizontal forces on the back of the MSE wall **will exceed** the factored shear resistance at the strength limit state under undrained conditions.

5.4.6 Overall (Global) Stability

A slope stability analysis was performed to check the global stability of the wall. As per the AASHTO LRFD BDS, safety against soil failure shall be evaluated at the service limit state by assuming the reinforced soil mass to be a rigid body. Soil parameters utilized in external stability analyses are presented in Section 5.4.1. For the global stability condition, it was considered that the failure plane will not cross through the reinforced soil mass. The computer software program Slide, manufactured by Rocscience Inc., was utilized to perform the analysis.

Per Section 11.6.2.3 of the 2018 AASHTO LRFD BDS, overall (global) stability for MSE walls that are integrated with or supporting structural foundations or elements is satisfied if the product of the factor of safety from the slope stability output multiplied by the resistance factor φ=0.65 is greater than 1.0. Therefore, global stability is satisfied when a minimum factor of safety of 1.5 is obtained. For an MSE wall designed with the minimum strap lengths listed in Table 13, the resulting factor of safety under drained conditions (long-term stability) and undrained conditions (short-term stability) along the alignment using the Spencer's analysis method was less than 1.5.

5.4.7 Final MSE Wall Considerations

Based on the results of the external and global stability analysis performed for the MSE wall at the rear abutment, sliding under undrained conditions as well as bearing and global stability under both drained and undrained conditions were not satisfied at a strap length equal to 0.7 times the wall height. Increasing the width of the wall up to 1.3 times the wall height still did not satisfy all of the external and global stability requirements. As noted in Section 5.4, consideration was given to over excavating the existing fill and underlying high plasticity clay soils and replacing them with granular embankment; however, similar conditions are anticipated along the remainder of the alignment of Retaining Wall E4, which makes this a very expensive and uneconomical option. Recommendations have been provided in the structure foundation exploration report for Retaining Wall E4 to incorporate the use of ground improvement techniques to stabilize the existing fill and underlying cohesive soils. 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 walls are provided in Appendix VIII.

5.5 Lateral Earth Pressure

For the soil types encountered in the borings, the "in-situ" unit weight (γ), cohesion (c), effective angle of friction (φ'), and lateral earth pressure coefficients for at-rest conditions (k_o), active conditions (k_a), and passive conditions (k_p) have been estimated and are provided in Table 16 and Table 17.

Table 16. Estimated Undrained (Short-term) Soil Parameters for Design

Soil Type	γ (pcf) [1]	c (psf)	φ	k_a	k_o	k_p
Soft to Stiff Cohesive Soil	115	1,500	0°	N/A	N/A	N/A
Very Stiff to Hard Cohesive Soil	125	3,000	0°	N/A	N/A	N/A
Very Loose to Loose Granular Soil	120	0	28°	0.32	0.53	5.07
Medium Dense Granular Soil	125	0	32°	0.27	0.47	6.82
Dense to Very Dense Granular Soil	130	0	36°	0.23	0.41	9.09
Compacted Cohesive Engineered Fill	120	2,000	0°	N/A	N/A	N/A
Compacted Granular Engineered Fill	120	0	32°	0.27	0.47	6.82
MSE Wall Select Granular Fill	120	0	34°	0.28	0.44	3.54

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

Table 17. Estimated Drained (Long-term) Soil Parameters for Design

Soil Type	γ (pcf) [1]	c (psf)	φ'	k_a	k_o	k_p
Soft to Stiff Cohesive Soil	115	0	26°	0.35	0.56	4.53
Very Stiff to Hard Cohesive Soil	125	50	28°	0.32	0.53	5.07
Very Loose to Loose Granular Soil	120	0	28°	0.32	0.53	5.07
Medium Dense Granular Soil	125	0	32°	0.27	0.47	6.82
Dense to Very Dense Granular Soil	130	0	36°	0.23	0.41	9.09
Compacted Cohesive Engineered Fill	120	0	30°	0.30	0.50	5.58
Compacted Granular Engineered Fill	120	0	32°	0.27	0.47	6.82
MSE Wall Select Granular Fill	120	0	34°	0.28	0.44	3.54

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.

For the evaluation of the drilled shafts at the forward abutment, if the analysis will be performed considering the end condition at the top of the drilled shaft as unrestrained, then the resultant shear and moment forces applied at the top of the drilled shaft from the lateral loading along the back of the abutment footing/wall should be determined using active earth pressure distribution (k_a) for the MSE wall select granular fill. For design of the reinforcement within the abutment footing/wall, the footing/wall should be assumed to be restrained at the top of the footing and the lateral loading along the back of the abutment footing/wall should be determined using at-rest earth pressure distribution (k_o) for the MSE wall select granular fill. Please note that the recommendations noted above consider that the MSE wall select granular fill will be placed the entire width of the abutment foundation/wall.

5.6 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).

5.6.1 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 18. 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
Rock to 3.0' +/- below Auger Refusal	0.75 : 1.0	Above Ground Water Table and No Seepage
Stable Rock	Vertical	Above Ground Water Table and No Seepage

5.6.2 Groundwater Considerations

Based on the groundwater observations made during drilling, groundwater may be encountered during construction of the drilled shafts. Where groundwater is encountered, proper groundwater control should be employed and maintained to prevent disturbance to excavation bottoms consisting of cohesive soil, and to prevent the possible development of a quick or "boiling" condition where soft silts and/or fine sands are encountered. It is preferable that the groundwater level, if encountered, be maintained at least 36 inches below the deepest excavation. In the case of drilled shafts, the utilization of casing will be required below the water table to maintain an open hole and prevent the sidewalls from collapse. In addition, concrete placed below the water table should be placed by tremie method. Any seepage or groundwater encountered at this site should be able to be controlled by pumping from temporary sumps. Additional measures may be required depending on seasonal fluctuations of the groundwater level. Note that determining and maintaining actual groundwater levels during construction is the responsibility of the contractor.

6.0 LIMITATIONS OF STUDY

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

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

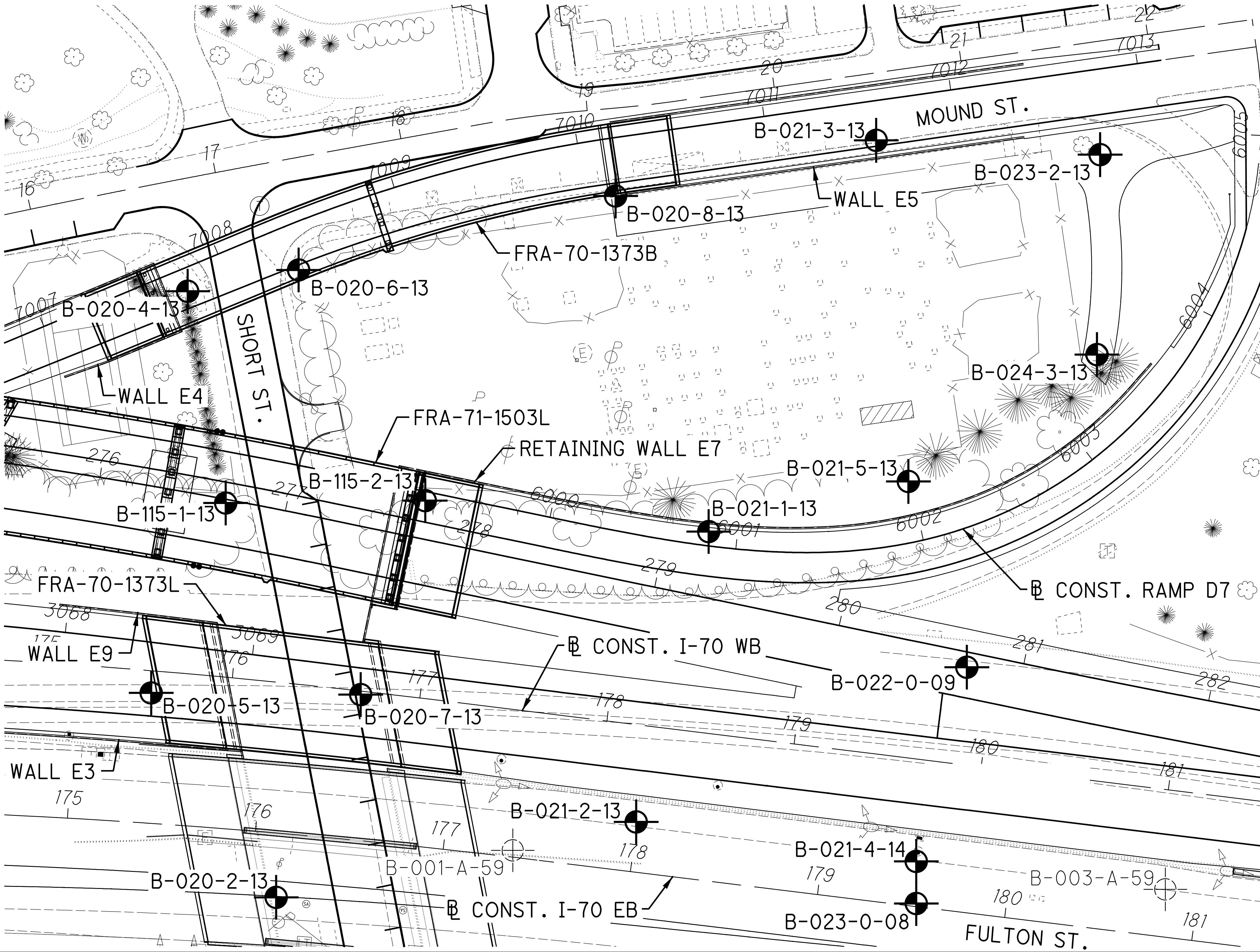
The conclusions and recommendations herein have been based upon the available soil and bedrock information and the design details furnished by a representative of the owner of the proposed project. Any revision in the plans for the proposed construction from those anticipated in this report should be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

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

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

APPENDIX I

VICINITY MAP AND BORING PLAN



BORINGS B-021-1-13, B-021-5-13 AND B-024-3-13 WERE PEFORMED FOR WALL E7.

BORINGS B-020-4-13, B-020-6-13 AND B-020-8-13 WERE PERFORMED FOR THE FRA-70-1373B STRUCTURE.

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

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

BORINGS B-115-1-13 AND B-115-2-13 WERE PERFORMED FOR THE FRA-71-1503L STRUCTURE.

BORINGS B-020-2-13, B-021-2-13 AND B-021-4-14 WERE PERFORMED AS PART OF THE FRA-70-12.68 PROJECT 4A/4R.

BORINGS B-022-0-09, B-023-0-08, B-025-0-08 AND B-026-0-08 WERE PERFORMED AS PART OF THE FRA-70-8.93 PRELIMINARY EXPLORATION.

LEGEND

- PROJECT BORING
- HISTORIC BORING

BORING PLAN

FRA-70-13.10 - RETAINING WALL E7

FRANKLIN COUNTY, OHIO

PROJECT NO. Rii W-13-072

SCALE: 1"=60'
0 30 60

N

DRAWN RRM

REVIEWED BRT

DATE 7-17-19

RESOURCE INTERNATIONAL Rii

APPENDIX II

DESCRIPTION OF SOIL TERMS

DESCRIPTION OF SOIL TERMS

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

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

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

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

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

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

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

Modifiers of Components - Modifiers of components are as follows:

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

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

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

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

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

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

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

DESCRIPTION OF ROCK TERMS

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

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

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

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

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

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

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

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

Degree of Fracturing

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

Condition of Fractures

Aperature Width

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

Surface Roughness

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

RQD – Rock Quality Designation:

RQD %	Rock Index Property Classification
0 – 25%	Very Poor
26 – 50%	Poor
51 – 70%	Fair
71 – 85%	Good
86 – 100%	Very Good

CLASSIFICATION OF SOILS

Ohio Department of Transportation

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

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

MATERIAL CLASSIFIED BY VISUAL INSPECTION

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

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

APPENDIX III

PROJECT BORING LOGS:

B-020-4-13, B-020-6-13 and B-020-8-13

BORING LOGS

Definitions of Abbreviations

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

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

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

Classification Test Data

Gradation (as defined on Description of Soil Terms):

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

Atterberg Limits:

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

WC	=	Water content (%)

PROJECT: FRA-70-13.10 - PHASE 6A	DRILLING FIRM / OPERATOR: RII / J.B./T.F.	DRILL RIG: MOBILE B-53 (SN 624400)	STATION / OFFSET: 7007+77.61 / 12.0' RT	EXPLORATION ID B-020-4-13
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / S.B./S.M.	HAMMER: AUTOMATIC	ALIGNMENT: BL RAMP D7	PAGE
PID: 89464 BR ID: FRA-70-1373B	DRILLING METHOD: 3.25" HSA / HQ	CALIBRATION DATE: 4/26/13	ELEVATION: 714.0 (MSL) EOB: 94.7 ft.	1 OF 4
START: 3/5/14 END: 3/11/14	SAMPLING METHOD: SPT / RC	ENERGY RATIO (%): 77.7	LAT / LONG: 39.954037, -83.004709	

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

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

MATERIAL DESCRIPTION AND NOTES	ELEV. 684.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
MEDIUM DENSE TO DENSE, BROWN TO GRAY **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, MOIST TO WET. *(same as above)*	682.0	31 – 32																
MEDIUM DENSE, GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST.		33 – 34	1 1 17	23	44	SS-13	-	62	22	5	7	4	NP	NP	NP	11	A-1-a (0)	
	677.0	35 – 37																
VERY DENSE, GRAY **GRAVEL**, LITTLE COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST.		38 – 39	8 20 25	58	67	SS-14	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	
		40 – 44	18 26 27	69	83	SS-15	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	
		45 – 49	8 31 50	105	72	SS-16	-	83	10	3	3	1	NP	NP	NP	9	A-1-a (0)	
		50 – 54	50 50/3"	-	100	SS-17	-	-	-	-	-	-	-	-	-	12	A-1-a (V)	
-AUGER REFUSAL ON BOULDER @ 55.5'; SWITCHED TO MUD ROTARY DRILLING WITH CASING ADVANCER.	657.0	55 – 57																
HARD, GRAY **SILTY CLAY**, TRACE COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP.		58 – 61	10 27 35	80	42	SS-18	4.5+	7	4	4	41	44	40	19	21	14	A-6b (12)	

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

MATERIAL DESCRIPTION AND NOTES	ELEV. 651.9	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
HARD, GRAY **SILTY CLAY**, TRACE COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP. *(same as above)*	649.6	63, 64	31 50/5"	-	91	SS-19	4.5+	-	-	-	-	-	-	-	-	16	A-6b (V)	
SHALE : GRAY, HIGHLY WEATHERED, VERY WEAK.		TR; 65 – 69	50/5"	-	100	SS-20	-	-	-	-	-	-	-	-	-	19	Rock (V)	
	639.3	70 – 74	50/5"	-	20	SS-21	-	-	-	-	-	-	-	-	-	23	Rock (V)	
SHALE : DARK GRAY, SLIGHTLY WEATHERED, WEAK, VERY THIN TO THIN BEDDED, ARGILLACEOUS, MODERATELY FRACTURED, NARROW APERTURES, SLICKENSIDED TO SLIGHTLY ROUGH; RQD 66%, REC 89%.	635.6	75 – 79	75		92	RC-1											CORE	
CLAYSTONE : DARK GRAY, SLIGHTLY WEATHERED, VERY WEAK, THIN BEDDED, CALCAREOUS, FRACTURED, TIGHT APERTURES, SLICKENSIDED TO SLIGHTLY ROUGH; RQD 100%, REC 100%. -SHALE SEAM PRESENT FROM 79.7' TO 80.7' -PYRITIC FROM 80.7' TO 83.7'	630.3	80 – 84	100		100	RC-2											CORE	
LIMESTONE : DARK BROWNISH GRAY, UNWEATHERED, MODERATELY STRONG TO STRONG, THIN TO MEDIUM BEDDED, CHERTY, DOLOMITIC, MODERATELY TO SLIGHTLY FRACTURED, NARROW TO OPEN APERTURES, SLIGHTLY ROUGH; RQD 87%, REC 95%. -QU @ 86.5' = 13,130 PSI		85 – 89	90		95	RC-3											CORE	
-QU @ 90.7' = 16,178 PSI		90 – 94	85		95	RC-4											CORE	

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

MATERIAL DESCRIPTION AND NOTES	ELEV. 619.7	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
	619.3	EOB																

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

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

B-020-4-13 – RC-1 and RC-2 – Depth from 74.7 to 84.7 feet

B-020-4-13 – RC-3 and RC-4 – Depth from 84.7 to 94.7 feet

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-13.10 - PHASE 6A	DRILLING FIRM / OPERATOR: RII / J.K.	DRILL RIG: CME-55 (SN 386345)	STATION / OFFSET: 7008+36.29 / 24.1' RT	EXPLORATION ID **B-020-6-13**
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / N.A.	HAMMER: AUTOMATIC	ALIGNMENT: BL RAMP D7	PAGE
PID: 89464 BR ID: FRA-70-1373B	DRILLING METHOD: 4.25" HSA / HQ	CALIBRATION DATE: 10/20/14	ELEVATION: 714.1 (MSL) EOB: 95.0 ft.	1 OF 4
START: 1/5/15 END: 1/12/15	SAMPLING METHOD: SPT / RC	ENERGY RATIO (%): 92	LAT / LONG: 39.954069, -83.004499	

MATERIAL DESCRIPTION AND NOTES	ELEV. 714.1	DEPTHS
0.2' - CONCRETE (2.0")	713.9	
0.6' - AGGREGATE BASE (7.0")	713.5	
FILL: STIFF, BROWN **SILT AND CLAY**, SOME COARSE TO FINE SAND, LITTLE FINE GRAVEL, MOIST.	708.6	
FILL: LOOSE TO MEDIUM DENSE, BROWN **GRAVEL**, TRACE COARSE SAND, TRACE SILT, DAMP TO MOIST. -CINDERS PRESENT IN SS-3 -SLAG PRESENT IN SS-3 AND SS-4	703.6	
STIFF, GRAY **CLAY**, AND SILT, TRACE COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST. -IRON STAINING PRESENT IN SS-5 THROUGH SS-7	696.1	W (16)
VERY STIFF, BROWN **SILT AND CLAY**, SOME COARSE TO FINE SAND, SOME FINE GRAVEL, DAMP.	693.6	
MEDIUM DENSE TO DENSE, GRAY AND BLACK **GRAVEL**, TRACE COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST.	688.6	W (26)
VERY DENSE, BROWN **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, WET. -ROCK FRAGMENTS PRESENT IN SS-11 -INTRODUCED MUD @26.0'	686.1	
VERY DENSE, GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST.		

DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
1-2	3 4 4	12	50	SS-1	1.50	14	15	16	32	23	33	20	13	21	A-6a (5)	
4-5	2 3 3	9	33	SS-2	1.25	-	-	-	-	-	-	-	-	24	A-6a (V)	
6-7	7 7 5	18	44	SS-3	-	98	1	0	1	0	NP	NP	NP	6	A-1-a (0)	
9-10	2 2 3	8	39	SS-4	-	-	-	-	-	-	-	-	-	14	A-1-a (V)	
11-12	4 4 5	14	100	SS-5	1.75	-	-	-	-	-	-	-	-	25	A-7-6 (V)	
14-15	4 4 5	14	100	SS-6	2.00	1	1	9	46	43	43	19	24	24	A-7-6 (14)	
16-17	2 1 2	5	44	SS-7	2.00	-	-	-	-	-	-	-	-	22	A-7-6 (V)	
19-20	9 8 10	27	100	SS-8	-	-	-	-	-	-	-	-	-	15	A-6a (V)	
21-22	10 13 11	36	39	SS-9	-	91	3	3	2	1	NP	NP	NP	12	A-1-a (0)	
24-25	5 5 5	15	50	SS-10	-	-	-	-	-	-	-	-	-	13	A-1-a (V)	
26-27	13 16 18	51	67	SS-11	-	59	18	6	11	6	29	23	6	25	A-1-b (0)	
29	35 34 30	96	100	SS-12	-	-	-	-	-	-	-	-	-	8	A-1-a (V)	

MATERIAL DESCRIPTION AND NOTES (ELEV. 684.1)

VERY DENSE, GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST. *(same as above)*

-ROCK FRAGMENTS PRESENT IN SS-13

-COBBLES AND BOULDERS PRESENT THROUGHOUT

654.1

PINK AND BLACK GRANITE BOULDER — 653.7 — TR

MUDSTONE : GRAY, UNWEATHERED, VERY WEAK, THICK BEDDED, CALCAREOUS, FRIABLE, FISSILE, — 652.1

DEPTHS	SPT/RQD	N60	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
34–35	44 / 23 / 27	75	100	SS-13	-	66	15	6	9	4	NP	NP	NP	11	A-1-a (0)
38.5	50/1"	-	0	SS-14	-	-	-	-	-	-	-	-	-	-	
44–45	38 / 23 / 40	95	100	SS-15	-	-	-	-	-	-	-	-	-	11	A-1-a (V)
49–50	40 / 38 / 38	114	100	SS-16	-	70	16	6	5	3	20	15	5	10	A-1-a (0)
53.5	50/6"	-	100	SS-17	-	-	-	-	-	-	-	-	-	8	A-1-a (V)
58.5	50/1"	-	100	SS-18	-	-	-	-	-	-	-	-	-	3	A-1-a (V)

PID: 89464	BR ID: FRA-70-1373B	PROJECT: FRA-70-13.10 - PHASE 6A	STATION / OFFSET: 7008+36.29 / 24.1 RT	START: 1/5/15	END: 1/12/15	PG 3 OF 4	B-020-6-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 652.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
INTACT, TIGHT APERTURES, SLICKENSIDED; RQD 46%, REC 100%.		63–65	60		90	RC-1											CORE	
SHALE : GRAY, SLIGHTLY WEATHERED TO UNWEATHERED, VERY WEAK TO STRONG, LAMINATED TO THICK BEDDED, ARENACEOUS, CALCAREOUS, PYRITIC, CRYSTALLINE, FISSILE, SLIGHTLY TO HIGHLY FRACTURED, TIGHT TO OPEN APERTURES, SLIGHTLY TO VERY ROUGH; RQD 49%, REC 81%. *(same as above)* -0.5' MUDSTONE SEAM @ 65.0' -1.1' LIMESTONE SEAM @ 66.0'		65–70	0		33	RC-2											CORE	
-0.4' MUDSTONE SEAM @ 71.0' -0.4' LIMESTONE SEAM @ 71.4' -0.8' MUDSTONE SEAM @ 71.8' -0.5' MUDSTONE SEAM @ 74.5'		70–75	64		100	RC-3											CORE	
-QU @ 77.4' = 177 PSI -0.3' CLAY SEAM @ 78.6'		75–80	63		97	RC-4											CORE	
-0.2' CLAY SEAM @ 80.9'	628.6	80–85	57		83	RC-5											CORE	
LIMESTONE : GRAY AND BROWN, UNWEATHERED, MODERATELY STRONG TO STRONG, MEDIUM TO THICK BEDDED, CHERTY, CRYSTALLINE, PYRITIC, DOLOMITIC, SLIGHTLY FRACTURED, NARROW TO OPEN APERTURES, SLIGHTLY TO VERY ROUGH; RQD 94%, REC 100%.		85–90	89		100	RC-6											CORE	
-QU @ 91.5' = 12,531 PSI		90–94	100		100	RC-7											CORE	

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

PID: 89464	BR ID: FRA-70-1373B	PROJECT: FRA-70-13.10 - PHASE 6A	STATION / OFFSET: 7008+36.29 / 24.1 RT	START: 1/5/15	END: 1/12/15	PG 4 OF 4	B-020-6-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 619.8	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
	619.1	EOB 95																

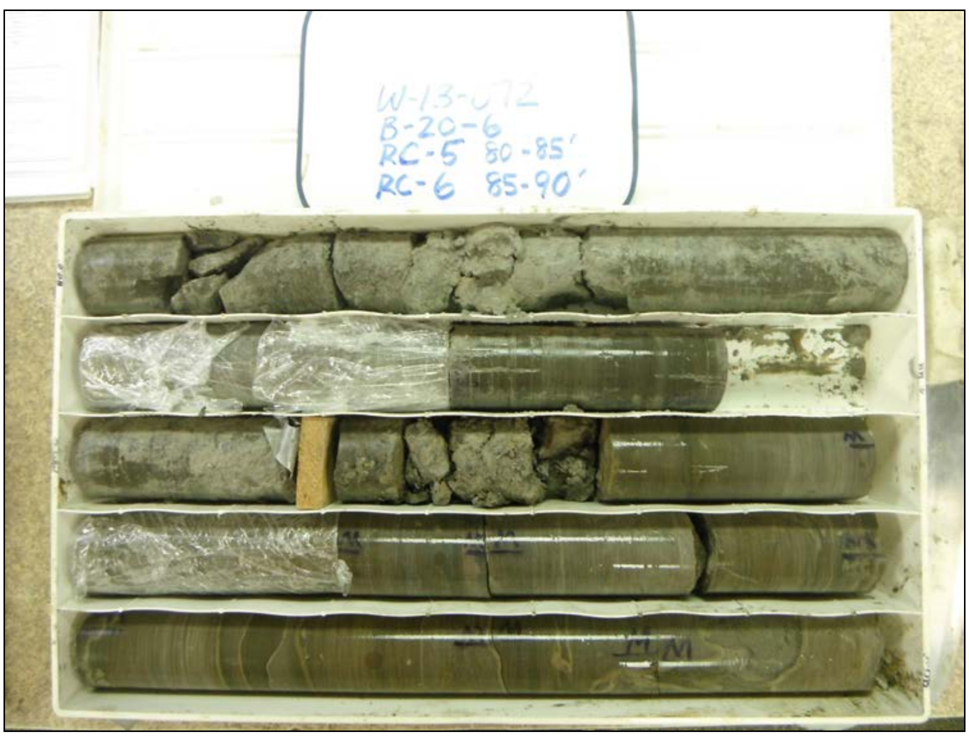
NOTES: SEEPAGE ENCOUNTERED @ 16.0'; GROUNDWATER ENCOUNTERED INITIALLY @ 26.0'

ABANDONMENT METHODS, MATERIALS, QUANTITIES: COMPACTED WITH THE AUGER 150 LBS BENTONITE CHIPS AND SOIL CUTTINGS; PUMPED 47 LBS CEMENT / 100 LBS BENTONITE POWDER / 40 GAL WATER

B-020-6-13 – RC-1 – Depth from 60.0 to 65.0 feet

B-020-6-13 – RC-2 and RC-3 – Depth from 65.0 to 75.0 feet

B-020-6-13 – RC-4 – Depth from 75.0 to 80.0 feet



B-020-6-13 – RC-5 and RC-6 – Depth from 80.0 to 90.0 feet

B-020-6-13 – RC-7 – Depth from 90.0 to 95.0 feet

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-13.10 - PHASE 6A	DRILLING FIRM / OPERATOR: RII / J.K.	DRILL RIG: CME-55 (SN 386345)	STATION / OFFSET: 7010+15.89 / 30.7' RT	EXPLORATION ID B-020-8-13
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / N.A.	HAMMER: AUTOMATIC	ALIGNMENT: BL RAMP D7	PAGE 1 OF 4
PID: 89464 BR ID: FRA-70-1373B	DRILLING METHOD: 4.25" HSA / HQ	CALIBRATION DATE: 10/20/14	ELEVATION: 721.0 (MSL) EOB: 102.0 ft.	
START: 12/15/14 END: 12/19/14	SAMPLING METHOD: SPT / RC	ENERGY RATIO (%): 92	LAT / LONG: 39.954179, -83.003900	

MATERIAL DESCRIPTION AND NOTES	ELEV. 721.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
0.7' - CONCRETE (8.0")	720.3																
0.3' - AGGREGATE BASE (4.0")	720.0																
FILL: LOOSE, BROWN **GRAVEL WITH SAND AND SILT**, LITTLE CLAY, DAMP TO MOIST.		1–2	2 3 2	8	56	SS-1	-	-	-	-	-	-	-	-	-	8	A-2-4 (V)
-ROCK FRAGMENTS PRESENT THROUGHOUT	715.5	4–5	2 2 3	8	56	SS-2	-	40	19	12	13	16	28	18	10	15	A-2-4 (0)
MEDIUM DENSE, GRAY **GRAVEL**, DRY. -LIMESTONE FRAGMENTS PRESENT IN SS-3	713.0	6–7	5 7 8	23	33	SS-3	-	-	-	-	-	-	-	-	-	1	A-1-a (V)
DENSE TO VERY DENSE, BROWN **GRAVEL**, LITTLE COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, DAMP TO MOIST.		8.5–9	50/6"	-	0	SS-4	-	-	-	-	-	-	-	-	-	-	
		11–12	26 16 13	44	67	SS-5	-	77	12	4	3	4	20	19	1	6	A-1-a (0)
		13.5–15	13 24 23	71	33	SS-6	-	-	-	-	-	-	-	-	-	6	A-1-a (V)
		16 (W)	50/6"	-	100	SS-7	-	-	-	-	-	-	-	-	-	9	A-1-a (V)
-INTRODUCED MUD @ 20.0'	700.5	18.5–20 (W at 18.5)	15 16 16	48	0	SS-8	-	-	-	-	-	-	-	-	-	-	
DENSE TO VERY DENSE, BROWN **COARSE AND FINE SAND**, TRACE TO LITTLE CLAY, TRACE SILT, TRACE FINE GRAVEL, WET.		21–22	25 26 26	78	100	SS-9	-	1	10	70	9	10	NP	NP	NP	21	A-3a (0)
		23.5–25	10 13 18	47	100	SS-10	-	-	-	-	-	-	-	-	-	21	A-3a (V)
		26–27	8 11 15	39	100	SS-11	-	1	14	61	10	14	NP	NP	NP	22	A-3a (0)
		28.5–30	11 16 16	48	100	SS-12	-	-	-	-	-	-	-	-	-	20	A-3a (V)

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

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

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

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

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

MATERIAL DESCRIPTION AND NOTES	ELEV. 626.7	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
LIMESTONE : GRAY, UNWEATHERED, VERY STRONG, VERY THICK BEDDED, ARENACEOUS, FERRIFEROUS, SILICEOUS, PYRITIC, INTACT, NARROW APERTURES, SLIGHTLY ROUGH; RQD 100%, REC 100%. *(same as above)* -QU @ 97.1' = 4,737 PSI		95, 96, 97	100		100	RC-7											CORE	
	619.0	98, 99, 100, 101, EOB 102	100		100	RC-8											CORE	

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

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

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

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

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

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

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

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

APPENDIX IV

LABORATORY TEST RESULTS

RESOURCE INTERNATIONAL, INC.
Engineering Consultants

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

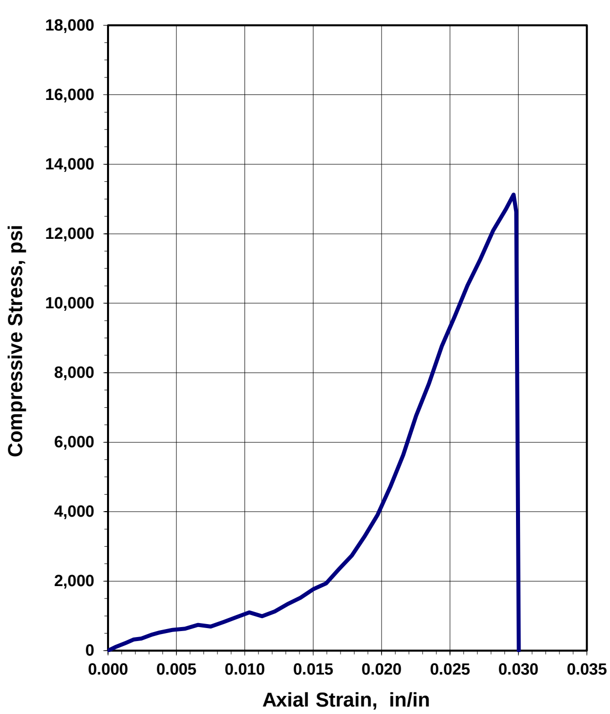
6350 Presidential Gatew.	9885 Rockside Road	4480 Lake Forest Drive
Columbus, OH 43231	Cleveland, OH 44125	Cincinnati, Ohio 45242
Phone (614) 823-4949	Phone (216) 573-0955	Phone (513) 769-6998

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

Rock Description: Limestone

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

Unconfined Compression Test



Before Testing

After Failure

REMARKS:

RESOURCE INTERNATIONAL, INC.
Engineering Consultants

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

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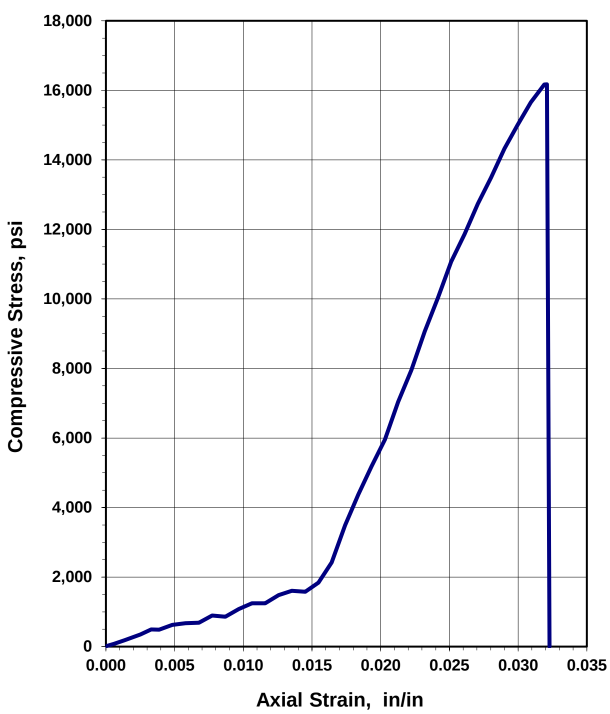
Phone (614) 823-4949	Phone (216) 573-0955	Phone (513) 769-6998

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

Rock Description: Limestone

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

Unconfined Compression Test



Before Testing

After Failure

REMARKS:

RESOURCE INTERNATIONAL, INC.

Engineering Consultants

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Cleveland, OH 44125
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4480 Lake Forest Drive
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Phone (513) 769-6998

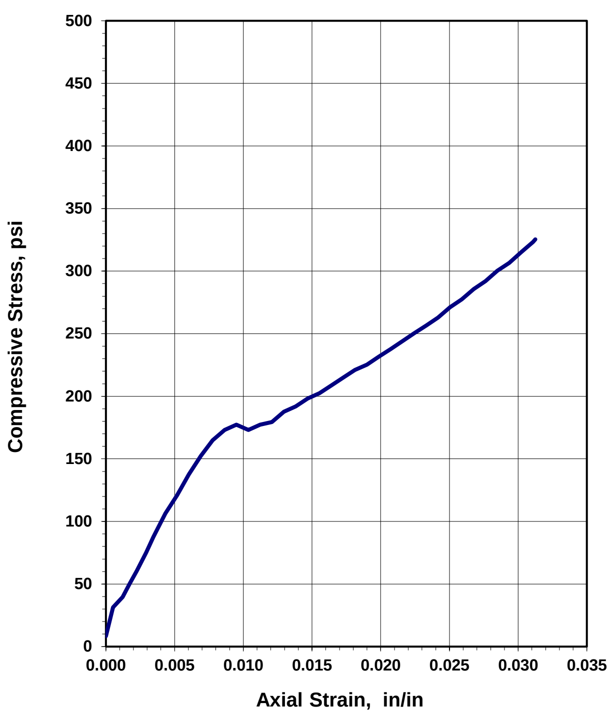
Unconfined Compressive Strength
of Intact Rock Core Specimens (ASTM D 7012-04)

Project: FRA-70-13.10 - Project 6A
Project No.: W-13-072
Date of Testing: 1/19/2015
Test Performed by: C.S./T.K.

Rock Description: Shale

Boring No.:	B-20-6	Average Length:	5.792 in
Sample No:	RC-4	Average Diameter:	2.471 in
Depth (ft):	77.4	Length to diameter ratio:	2.344
Moisture condition:	As received	Cross Sectional Area:	4.793 in^2
Rate of Loading:	2.1 lbs/sec	Failure Load:	850 lbs
Testing Time:	413 sec	Axial Strain at Failure:	0.0095 in/in
(Rate 2-15 minutes to failure)		Stress:	177 psi

Unconfined Compression Test



Before Testing

After Failure

REMARKS:

RESOURCE INTERNATIONAL, INC.
Engineering Consultants

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

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

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

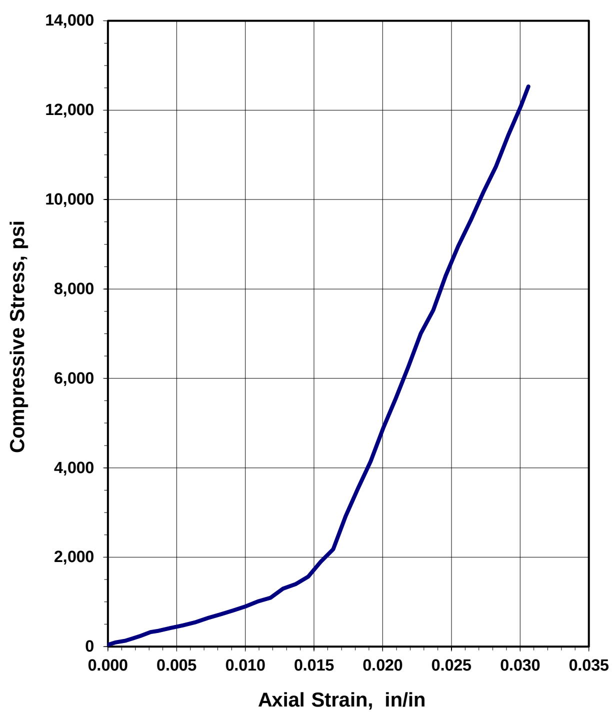
4480 Lake Forest Drive
Cincinnati, Ohio 45242
Phone (513) 769-6998

Project: FRA-70-13.10 - Project 6A
Project No.: W-13-072
Date of Testing: 1/19/2015
Test Performed by: C.S./T.K.

Rock Description: Limestone with Chert nodules

Boring No.:	B-20-6	Average Length:	5.49 in
Sample No:	RC-7	Average Diameter:	2.488 in
Depth (ft):	91.5	Length to diameter ratio:	2.207
Moisture condition:	As received	Cross Sectional Area:	4.859 in^2
Rate of Loading:	82.5 lbs/sec	Failure Load:	60,910 lbs
Testing Time:	738 sec	Axial Strain at Failure:	0.0306 in/in
(Rate 2-15 minutes to failure)		Stress:	12,531 psi

Unconfined Compression Test



Before Testing

After Failure

REMARKS:

RESOURCE INTERNATIONAL, INC.
Engineering Consultants

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

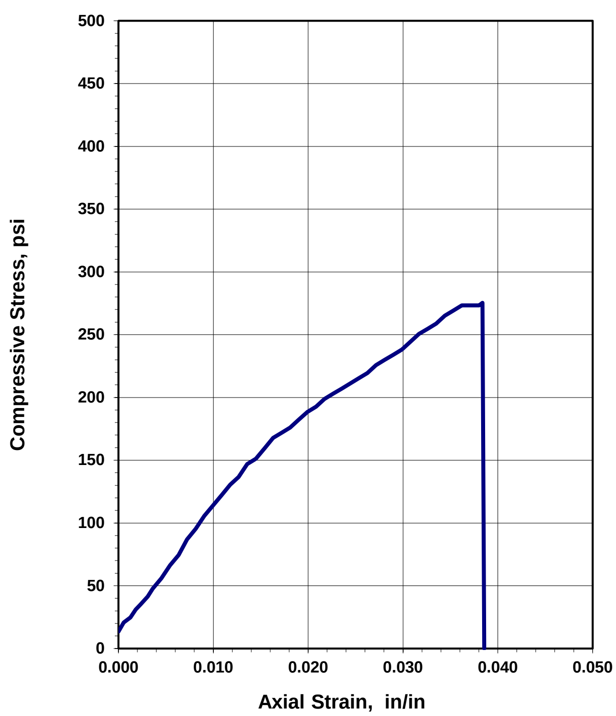
Unconfined Compressive Strength
of Intact Rock Core Specimens (ASTM D 7012-04)

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

Rock Description: Shale

Boring No.:	B-20-8	Average Length:	5.523 in
Sample No:	RC-2	Average Diameter:	2.48 in
Depth (ft):	71.2	Length to diameter ratio:	2.227
Moisture condition:	As received	Cross Sectional Area:	4.828 in^2
Rate of Loading:	2.9 lbs/sec	Failure Load:	1,330 lbs
Testing Time:	454 sec	Axial Strain at Failure:	0.0384 in/in
(Rate 2-15 minutes to failure)		Stress:	275 psi

Unconfined Compression Test



Before Testing

After Failure

REMARKS:

RESOURCE INTERNATIONAL, INC.
Engineering Consultants

6350 Presidential Gatew.
Columbus, OH 43231
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9885 Rockside Road
Cleveland, OH 44125
Phone (216) 573-0955

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

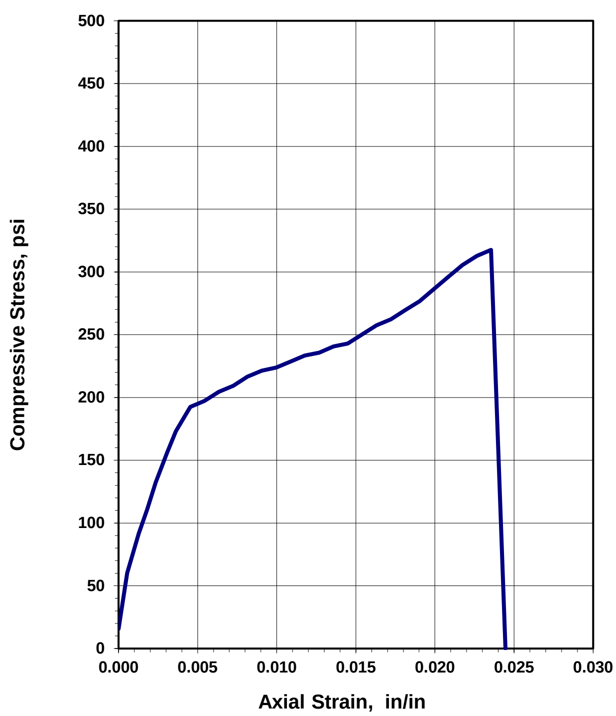
Unconfined Compressive Strength
of Intact Rock Core Specimens (ASTM D 7012-04)

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

Rock Description: Shale

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

Unconfined Compression Test



Before Testing

After Failure

REMARKS:

RESOURCE INTERNATIONAL, INC.

Engineering Consultants

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

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

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

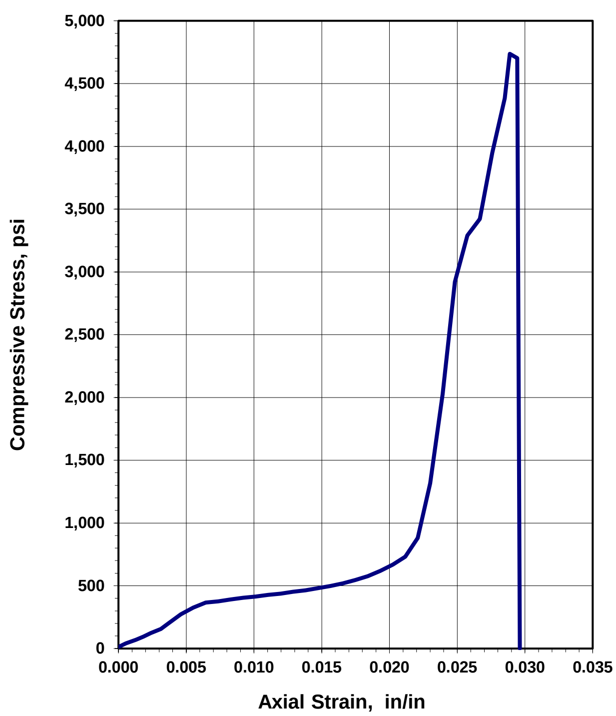
4480 Lake Forest Drive
Cincinnati, Ohio 45242
Phone (513) 769-6998

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

Rock Description: Limestone

Boring No.:	B-20-8	Average Length:	5.435 in
Sample No:	RC-8	Average Diameter:	2.491 in
Depth (ft):	97.1	Length to diameter ratio:	2.182
Moisture condition:	As received	Cross Sectional Area:	4.871 in^2
Rate of Loading:	65.4 lbs/sec	Failure Load:	23,080 lbs
Testing Time:	353 sec	Axial Strain at Failure:	0.0289 in/in
(Rate 2-15 minutes to failure)		Stress:	4,737 psi

Unconfined Compression Test



Before Testing

After Failure

REMARKS:

APPENDIX V

DRILLED SHAFT CALCULATIONS

Boring	Proposed Top of Shaft Elevation (ft msl)	D_w (ft)	Shaft Diameter, D (ft)	Soil Class.	Material Type [1]	Stratum Depth, z (ft)	Stratum Thickness (ft)	Bottom Elevation (ft msl)	γ (pcf)	σ_v' (Midpoint) (psf)	σ_v (Bottom) (psf)	S_u [2] (psf)	N_c [3]	α [4]	N_{60} [5]	$(N_1)_{60}$ [6]	φ'_f [7]	σ_p' [8] (psf)	β [9]	Boring	Elevation (ft msl)	Shaft Length (ft)	Nominal Unit Tip Resistance, q_p [10,11] (ksf)	Nominal Unit Side Resistance, q_s [12,13] (ksf)	φ_{qp} [14]	φ_{qs} [15]
B-020-6-13	714.1	26.0	3.5	A-6a	C	5.5	5.5	708.6	115	316	633	1,250	7.9	0.55						B-020-6-13	714.1-708.6	0.0-5.5	9	0.68	0.40	0.45
				A-1-a	G	10.5	5.0	703.6	125	945	1,258				13	16	39	4,134	0.76		708.6-703.6	5.5-10.5	15	0.71	0.50	0.55
				A-7-6	C	20.5	10.0	693.6	115	1,833	2,408	2,000	9.0	0.55							703.6-693.6	10.5-20.5	18	1.10	0.40	0.45
				A-1-a	G	25.5	5.0	688.6	125	2,720	3,033				26	23	40	8,268	0.61		693.6-688.6	20.5-25.5	31	1.66	0.50	0.55
				A-1-a	G	60.0	34.5	654.1	135	4,316	7,690				100	74	45	31,800	1.20		688.6-654.1	25.5-60.0	60	5.18	0.50	0.55
B-020-8-13	720.0	17.5	3.5	A-2-4	G	4.5	4.5	715.5	120	270	540				8	13	38	2,544	1.19	B-020-8-13	720.0-715.5	0.0-4.5	9	0.32	0.50	0.55
				A-1-a	G	19.5	15.0	700.5	130	1,515	2,490				40	44	43	12,720	1.27		715.5-700.5	4.5-19.5	48	1.91	0.50	0.55
				A-3a	G	31.0	11.5	689.0	130	2,754	3,985				45	40	42	9,781	0.70		700.5-689.0	19.5-31.0	54	1.91	0.50	0.55
				A-1-b	G	51.0	20.0	669.0	135	3,869	6,685				100	78	45	31,800	1.30		689.0-669.0	31.0-51.0	60	5.02	0.50	0.55
				A-7-6	C	64.0	13.0	656.0	130	5,034	8,375	4,000	9.0	0.51							669.0-656.0	51.0-64.0	36	2.04	0.40	0.45

1. C = cohesive soil stratum; G = granular soil stratum
2. S_u = average shear strength over stratum thickness (cohesive soil layers)
3. $N_C = 6[1+0.2(Z/D)] \leq 9$; Ref. Section 10.8.3.5.1c, AASHTO LRFD BDS (cohesive soil layers)
4. $\alpha = 0.55$ for $S_u/P_a \leq 1.5$; $\alpha = 0.55-0.1(S_u/P_a-1.5)$ for $1.5 \leq S_u/P_a \leq 2.5$, where P_a = 2.12 ksf = 2,120 psf; Ref. Section 10.8.3.5.1b AASHTO LRFD BDS (cohesive soil layers)
5. N_{60} = average energy corrected N-values over stratum thickness (granular soil layers)
6. $(N_1)_{60} = C_nN_{60}$, where $C_N = [0.77\log(40/\sigma_v')] \leq 2.0$ ksf, where σ_v' = vetical effective stress at midpoint of soil layer; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS (granular soil layers)
7. $\varphi'_f = 27.5+9.2\log[(N_1)_{60}]$; Ref. Section 10.8.3.5.2b, AASHTO LRFD BDS (granular soil layers)
8. $\sigma_p' = n(N_{60})^m(P_{a)}$, where n = 0.15 and m = 1.0 for A-1-a/1-b and A-2-4/2-6, n = 0.47 and m = 0.6 for A-3/3a, n = 0.47 and m = 0.8 for A-4a/4b soils, and P_a = 2.12 ksf = 2,120 psf; Ref. Section 10.8.3.5.2b, AASHTO LRFD BDS (granular soil layers)
9. $\beta = \tan\varphi'_f(1-\sin\varphi'_f)(\sigma_p'/\sigma_v')\text{^}(\sin\varphi'_f)$, where σ_v' = vetical effective stress at midpoint of soil layer; Ref. Section 10.8.3.5.2b, AASHTO LRFD BDS (granular soil layers)
10. $q_p = N_CS_u \leq 80.0$ ksf; Ref. Section 10.8.3.5.1c, AASHTO LRFD BDS (cohesive soil layers)
11. $q_p = 1.2N_{60} \leq 60$ ksf; Ref. Section 10.8.3.5.2c, AASHTO LRFD BDS (granular soil layers)
12. $q_s = \alpha S_u$; Ref. Section 10.8.3.5.1b, AASHTO LRFD BDS (cohesive soil layers)
13. $q_s = \beta\sigma_v'$, where σ_v' = vetical effective stress at midpoint of soil layer; Ref. Section 10.8.3.5.2b, AASHTO LRFD BDS (granular soil layers)
14. φ_{qp} = 0.50 for granular soils layers and 0.40 for cohesive soil layers; Ref. Table 10.5.5.2.4-1, AASHTO LRFD BDS
15. φ_{qs} = 0.55 for granular soils layers and 0.45 for cohesive soil layers; Ref. Table 10.5.5.2.4-1, AASHTO LRFD BDS

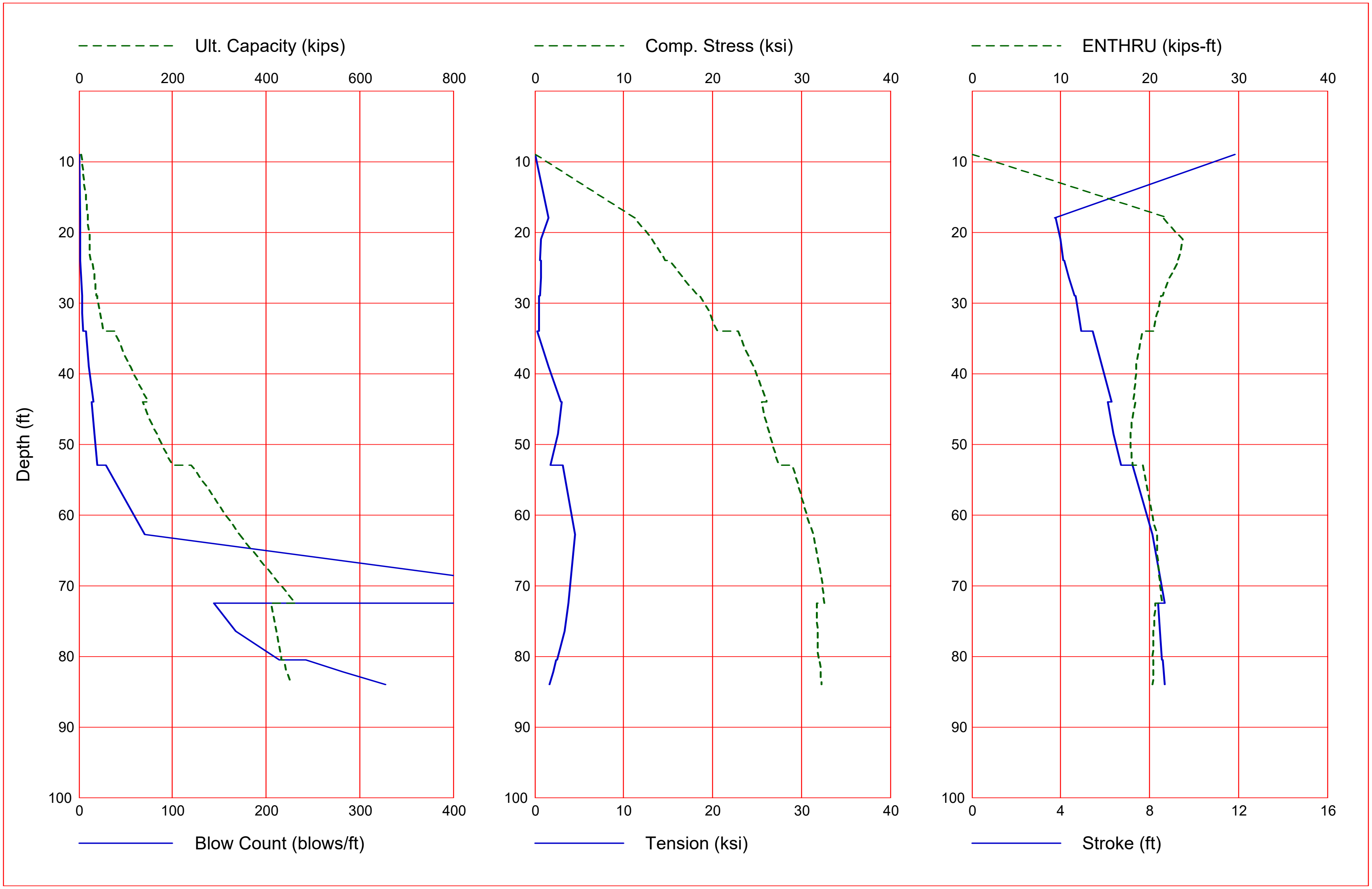
APPENDIX VI

GRLWEAP DRIVEABILITY ANALYSIS OUTPUTS

Resource International Inc
FRA-70-1373B - RA - B-020-4-13 - HP10x42

Apr 06 2020
GRLWEAP Version 2010

Gain/Loss 3 at Shaft and Toe 0.500 / 1.000



Gain/Loss 3 at Shaft and Toe 0.500 / 1.000

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
9.0	5.7	4.6	1.1	0.0	0.000	0.000	11.86	0.0
18.0	19.4	18.3	1.1	1.4	11.266	-1.525	3.75	22.1
18.0	18.8	18.3	0.5	1.4	11.244	-1.552	3.76	21.5
21.0	22.5	22.1	0.5	1.6	13.200	-0.737	3.97	23.7
24.0	26.3	25.9	0.5	1.9	14.661	-0.588	4.10	23.2
24.0	27.3	25.9	1.4	1.9	15.128	-0.719	4.16	23.3
26.5	32.8	31.4	1.4	2.6	16.708	-0.642	4.37	22.2
29.0	38.3	37.0	1.4	3.3	18.304	-0.535	4.62	21.5
29.0	39.9	37.1	2.8	3.5	18.636	-0.457	4.67	21.3
31.5	46.3	43.5	2.8	4.0	19.679	-0.523	4.81	20.9
34.0	53.1	50.3	2.8	4.6	20.509	-0.520	4.94	20.4
34.0	76.1	50.4	25.6	7.5	22.832	-0.314	5.43	19.2
39.0	110.8	83.2	27.6	11.0	24.544	-1.547	5.85	18.5
44.0	148.6	119.5	29.2	15.5	26.078	-2.955	6.28	18.2
44.0	137.5	119.7	17.7	13.8	25.588	-3.021	6.14	18.4
48.5	167.3	149.5	17.7	16.8	26.421	-2.643	6.39	17.9
53.0	199.2	181.5	17.7	20.1	27.427	-1.743	6.72	18.1
53.0	240.2	181.8	58.3	28.9	28.939	-3.102	7.22	19.2
62.8	343.3	285.0	58.3	70.1	31.274	-4.504	8.11	20.8
72.5	462.4	404.0	58.3	621.8	32.550	-3.820	8.69	21.4
72.5	410.5	404.3	6.2	144.7	31.744	-3.791	8.37	20.6
76.5	421.6	415.4	6.2	167.5	31.844	-3.391	8.45	20.4
80.5	432.7	426.5	6.2	214.2	31.943	-2.529	8.54	20.3
80.5	437.8	426.6	11.2	242.4	32.076	-2.406	8.59	20.4
82.2	444.9	433.8	11.2	282.3	32.128	-2.101	8.63	20.4
84.0	452.3	441.1	11.2	327.3	32.300	-1.701	8.68	20.3

Total Continuous Driving Time 161.00 minutes; Total Number of Blows 6553

GRLWEAP - Version 2010
WAVE EQUATION ANALYSIS OF PILE FOUNDATIONS

written by GRL Engineers, Inc. (formerly Goble Rausche Likins and Associates, Inc.) with cooperation from Pile Dynamics, Inc.

ABOUT THE WAVE EQUATION ANALYSIS RESULTS

The GRLWEAP program simulates the behavior of a preformed pile driven by either an impact hammer or a vibratory hammer. The program is based on mathematical models, which describe motion and forces of hammer, driving system, pile and soil under the hammer action. Under certain conditions, the models only crudely approximate, often complex, dynamic situations.

A wave equation analysis generally relies on input data, which represents normal situations. In particular, the hammer data file supplied with the program assumes that the hammer is in good working order. All of the input data selected by the user may be the best available information at the time when the analysis is performed. However, input data and therefore results may significantly differ from actual field conditions.

Therefore, the program authors recommend prudent use of the GRLWEAP results. Soil response and hammer performance should be verified by static and/or dynamic testing and measurements. Estimates of bending or other local stresses (e.g., helmet or clamp contact, uneven rock surfaces etc.), prestress effects and others must also be accounted for by the user.

The calculated capacity - blow count relationship, i.e. the bearing graph, should be used in conjunction with observed blow counts for the capacity assessment of a driven pile. Soil setup occurring after pile installation may produce bearing capacity values that differ substantially from those expected from a wave equation analysis due to soil setup or relaxation. This is particularly true for pile driven with vibratory hammers. The GRLWEAP user must estimate such effects and should also use proper care when applying blow counts from restrike because of the variability of hammer energy, soil resistance and blow count during early restriking.

Finally, the GRLWEAP capacities are ultimate values. They MUST be reduced by means of an appropriate factor of safety to yield a design or working load. The selection of a factor of safety should consider the quality of the construction control, the variability of the site conditions, uncertainties in the loads, the importance of building and other factors.

```
 Input  File: J:\GEOTECH\PROJECTS\2013\W-13-072 FRA-70-13.10 PROJECT 6A\ANALYSIS\FRA-70-1373B\DRIVEABILITY\REAR
ABUTMENT\B-20-4.GWW
 Hammer File: C:\ProgramData\PDI\GRLWEAP\2010\Resource\HAMMER2003.GW
 Hammer File Version:  2003 (2/22/2013)

                       Input File Contents
     FRA-70-1373B - RA - B-020-4-13 - HP10x42
 OUT OSG HAM STR FUL PEL  N SPL N-U P-D %SK ISM  0 PHI RSA ITR H-D MXT     DEx
-100   0  41   0   0   0  0   0   0   1   0   0  0   0   0   0   0   0   0.000
    Pile g  Hammer g  Toe Area Pile Size          Pile Type
    32.170    32.170   144.000    10.000            Unknown
      W Cp      A Cp      E Cp      T Cp       CoR      ROut      StCp
     1.900   227.000     530.0     2.000     0.800     0.010       0.0
      A Cu      E Cu      T Cu       CoR      ROut      StCu
     0.000       0.0     0.000     0.000     0.000       0.0
      LPle      APle      EPle      WPle      Peri        CI       CoR      ROut
    84.000     12.40   29000.0   492.000     3.300         0     0.850     0.010
 Manufac  Hmr Name  HmrType No Seg-s
 DELMAG   D 19-42         1        5
    Ram Wt     Ram L  Ram Dia   MaxStrk   RtdStrk    Efficy
      4.00    129.10    12.60     11.86     10.81      0.80
    IB. Wt     IB. L   IB.Dia    IB CoR     IB RO
      0.75     25.30    12.60     0.900     0.010
  CompStrk A Chamber V Chamber   C Delay  C Duratn Exp Coeff VolCStart  Vol CEnd
     16.65    124.70    157.70     0.002     0.002     1.250      0.00      0.00
     P atm        P1        P2        P3        P4        P5
     14.70   1520.00   1368.00   1231.00   1108.00      0.00
    Stroke    Effic.  Pressure  R-Weight   T-Delay Exp-Coeff   Eps-Str  Total-AW
```

10.8100	0.8000	1520.0000	0.0000	0.0000	0.0000	0.0100	0.0000
Qs	Qt	Js	Jt	Qx	Jx	Rati	Dept
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Research Soil Model: Atoe, Plug, Gap, Q-fac
0.000 0.000 0.000 0.000
Research Soil Model: RD-skn: m, d, toe: m, d
0.000 0.000 0.000 0.000
Res. Distribution

Dpth	Rskn	Rtoe	Qs	Qt	Js	Jt	SU F	LimD	SU T
0.01	0.00	0.00	0.10	0.10	0.05	0.15	1.00	0.00	0.0
9.01	0.31	1.15	0.10	0.10	0.05	0.15	1.00	0.00	0.0
16.49	0.57	1.15	0.10	0.10	0.05	0.15	1.00	0.00	0.0
16.51	0.57	1.15	0.10	0.10	0.05	0.15	1.00	0.00	0.0
17.99	0.59	1.15	0.10	0.10	0.05	0.15	1.00	0.00	0.0
18.01	0.56	0.48	0.10	0.10	0.20	0.15	1.49	0.00	0.0
23.99	0.58	0.48	0.10	0.10	0.20	0.15	1.49	0.00	0.0
24.01	1.32	1.36	0.10	0.10	0.20	0.15	2.00	0.00	0.0
28.99	1.38	1.36	0.10	0.10	0.20	0.15	2.00	0.00	0.0
29.01	0.92	2.84	0.10	0.10	0.05	0.15	1.21	0.00	0.0
33.99	1.02	2.84	0.10	0.10	0.05	0.15	1.21	0.00	0.0
34.01	1.89	25.61	0.10	0.10	0.05	0.15	1.00	0.00	0.0
43.01	2.27	29.17	0.10	0.10	0.05	0.15	1.00	0.00	0.0
43.99	2.31	29.17	0.10	0.10	0.05	0.15	1.00	0.00	0.0
44.01	1.94	17.74	0.10	0.10	0.05	0.15	1.00	0.00	0.0
52.99	2.24	17.74	0.10	0.10	0.05	0.15	1.00	0.00	0.0
53.01	2.96	58.35	0.10	0.10	0.05	0.15	1.00	0.00	0.0
62.01	3.42	58.35	0.10	0.10	0.05	0.15	1.00	0.00	0.0
71.01	3.88	58.35	0.10	0.10	0.05	0.15	1.00	0.00	0.0
72.49	3.95	58.35	0.10	0.10	0.05	0.15	1.00	0.00	0.0
72.51	1.26	6.20	0.10	0.10	0.20	0.15	1.49	0.00	0.0
80.49	1.26	6.20	0.10	0.10	0.20	0.15	1.49	0.00	0.0
80.51	1.26	11.16	0.10	0.10	0.20	0.15	1.00	0.00	0.0
84.00	1.26	11.16	0.10	0.10	0.20	0.15	1.00	0.00	0.0

Gain/Loss factors: shaft and toe
0.40000 0.45000 0.50000 0.55000 0.60000
1.00000 1.00000 1.00000 1.00000 1.00000

Dpth	L	Wait	Strk	Pmx%	Eff.	Stff	CoR
9.00	0.00	0.00	0.000	0.000	0.000	0.000	0.000
17.98	0.00	0.00	0.000	0.000	0.000	0.000	0.000
18.02	0.00	0.00	0.000	0.000	0.000	0.000	0.000
21.00	0.00	0.00	0.000	0.000	0.000	0.000	0.000
23.98	0.00	0.00	0.000	0.000	0.000	0.000	0.000
24.02	0.00	0.00	0.000	0.000	0.000	0.000	0.000
26.50	0.00	0.00	0.000	0.000	0.000	0.000	0.000
28.98	0.00	0.00	0.000	0.000	0.000	0.000	0.000
29.02	0.00	0.00	0.000	0.000	0.000	0.000	0.000
31.50	0.00	0.00	0.000	0.000	0.000	0.000	0.000
33.98	0.00	0.00	0.000	0.000	0.000	0.000	0.000
34.02	0.00	0.00	0.000	0.000	0.000	0.000	0.000
39.00	0.00	0.00	0.000	0.000	0.000	0.000	0.000
43.98	0.00	0.00	0.000	0.000	0.000	0.000	0.000
44.02	0.00	0.00	0.000	0.000	0.000	0.000	0.000
48.50	0.00	0.00	0.000	0.000	0.000	0.000	0.000
52.98	0.00	0.00	0.000	0.000	0.000	0.000	0.000
53.02	0.00	0.00	0.000	0.000	0.000	0.000	0.000
62.75	0.00	0.00	0.000	0.000	0.000	0.000	0.000
72.48	0.00	0.00	0.000	0.000	0.000	0.000	0.000
72.52	0.00	0.00	0.000	0.000	0.000	0.000	0.000
76.50	0.00	0.00	0.000	0.000	0.000	0.000	0.000
80.48	0.00	0.00	0.000	0.000	0.000	0.000	0.000
80.52	0.00	0.00	0.000	0.000	0.000	0.000	0.000
82.24	0.00	0.00	0.000	0.000	0.000	0.000	0.000
84.00	0.00	0.00	0.000	0.000	0.000	0.000	0.000
0.00	0.00	0.00	0.000	0.000	0.000	0.000	0.000

1 0 10.81000 11.86000

GRLWEAP: WAVE EQUATION ANALYSIS OF PILE FOUNDATIONS
Version 2010
English Units

FRA-70-1373B - RA - B-020-4-13 - HP10x42

Hammer Model: D 19-42 Made by: DELMAG

No. Weight Stiffn CoR C-Slk Dampg

	kips	k/inch		ft	k/ft/s
1	0.800				
2	0.800	140046.7	1.000	0.0100	
3	0.800	140046.7	1.000	0.0100	
4	0.800	140046.7	1.000	0.0100	
5	0.800	140046.7	1.000	0.0100	
Imp Block	0.753	70735.6	0.900	0.0100	
Helmet	1.900	60155.0	0.800	0.0100	5.8
Combined Pile Top		8918.7			

HAMMER OPTIONS:

Hammer File ID No.	41	Hammer Type	OE Diesel
Stroke Option	FxdP-VarS	Stroke Convergence Crit.	0.010
Fuel Pump Setting	Maximum		

HAMMER DATA:

Ram Weight	(kips)	4.00	Ram Length	(inch)	129.10
Maximum Stroke	(ft)	11.86			
Rated Stroke	(ft)	10.81	Efficiency		0.800
Maximum Pressure	(psi)	1520.00	Actual Pressure	(psi)	1520.00
Compression Exponent		1.350	Expansion Exponent		1.250
Ram Diameter	(inch)	12.60			
Combustion Delay	(s)	0.00200	Ignition Duration	(s)	0.00200

The Hammer Data Includes Estimated (NON-MEASURED) Quantities

HAMMER CUSHION			PILE CUSHION		
Cross Sect. Area	(in2)	227.00	Cross Sect. Area	(in2)	0.00
Elastic-Modulus	(ksi)	530.0	Elastic-Modulus	(ksi)	0.0
Thickness	(inch)	2.00	Thickness	(inch)	0.00
Coeff of Restitution		0.8	Coeff of Restitution		1.0
RoundOut	(ft)	0.0	RoundOut	(ft)	0.0
Stiffness	(kips/in)	60155.0	Stiffness	(kips/in)	0.0

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Depth	(ft)	9.0		
Shaft Gain/Loss Factor		0.400	Toe Gain/Loss Factor	1.000

PILE PROFILE:

Toe Area	(in2)	144.000	Pile Type	Unknown
Pile Size	(inch)	10.000		

L b Top	Area	E-Mod	Spec Wt	Perim	C Index	Wave Sp	EA/c
ft	in2	ksi	lb/ft3	ft		ft/s	k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model Total Capacity Rut (kips) 5.7

No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Perim	Area
	kips	k/in	ft	ft		kips	s/ft	inch	ft	ft	in2
1	0.142	8919	0.010	0.000	0.85	0.0	0.000	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	0.0	0.000	0.100	6.72	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	0.3	0.050	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	1.5	0.050	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	2.8	0.050	0.100	84.00	3.3	12.4
Toe						1.1	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
3.559 kips total reduced pile weight (g= 32.17 ft/s2)

PILE, SOIL, ANALYSIS OPTIONS:

Uniform pile		Pile Segments: Automatic		
No. of Slacks/Splices	0	Pile Damping	(%)	1
		Pile Damping Fact.(k/ft/s)		0.435
Driveability Analysis				
Soil Damping Option	Smith			

```
 Max No Analysis Iterations         0  Time Increment/Critical          160
 Output Time Interval               1  Analysis Time-Input   (ms)         0
 Output Level: Normal
 Gravity Mass, Pile, Hammer:   32.170    32.170    32.170
 Output Segment Generation: Automatic

    Depth    Stroke  Pressure    Efficy
       ft        ft     Ratio
     9.00     10.81      1.00     0.800


INITIAL STATIC ANALYSIS: Total Wt, Sum(R)     6.2     5.7
 Hammer+Pile Weight > Rult: Pile Runs


INITIAL STATIC ANALYSIS: Total Wt, Sum(R)     6.2     5.7
 Hammer+Pile Weight > Rult: Pile Runs


INITIAL STATIC ANALYSIS: Total Wt, Sum(R)     6.2     5.7
 Hammer+Pile Weight > Rult: Pile Runs


INITIAL STATIC ANALYSIS: Total Wt, Sum(R)     6.2     5.7
 Hammer+Pile Weight > Rult: Pile Runs


INITIAL STATIC ANALYSIS: Total Wt, Sum(R)     6.2     5.7
 Hammer+Pile Weight > Rult: Pile Runs
```

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut kips	Bl Ct b/ft	Stroke (ft) down	up	Ten Str ksi	i	t	Comp Str ksi	i	t	ENTHRU kip-ft	Bl Rt b/min
5.7	0.0	10.81	0.00	0.00	1	0	0.00	1	0	0.0	78.4
5.7	0.0	11.86	0.00	0.00	1	0	0.00	1	0	0.0	74.4
5.7	0.0	11.86	0.00	0.00	1	0	0.00	1	0	0.0	74.4
5.7	0.0	11.86	0.00	0.00	1	0	0.00	1	0	0.0	74.4
5.7	0.0	11.86	0.00	0.00	1	0	0.00	1	0	0.0	74.4

```
        1           0   10.81000       11.86000
```

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

```
 Depth                 (ft)      18.0
 Shaft Gain/Loss Factor         0.400  Toe Gain/Loss Factor          1.000

 PILE PROFILE:
 Toe Area            (in2)   144.000  Pile Type                    Unknown
 Pile Size          (inch)    10.000
```

L b Top ft	Area in2	E-Mod ksi	Spec Wt lb/ft3	Perim ft	C Index	Wave Sp ft/s	EA/c k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model — Total Capacity Rut (kips) 19.4

No.	Weight kips	Stiffn k/in	C-Slk ft	T-Slk ft	CoR	Soil-S kips	Soil-D s/ft	Quake inch	LbTop ft	Perim ft	Area in2
1	0.142	8919	0.010	0.000	0.85	0.0	0.000	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	0.0	0.000	0.100	6.72	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	0.1	0.050	0.100	67.20	3.3	12.4
21	0.142	8919	0.000	0.000	1.00	1.1	0.050	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	2.4	0.050	0.100	73.92	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	3.7	0.050	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	4.9	0.050	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	6.1	0.050	0.100	84.00	3.3	12.4
Toe						1.1	0.150	0.100			

```
     3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
     3.559 kips total reduced pile weight   (g= 32.17 ft/s2)
```

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
17.98	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut	Bl Ct	Stroke (ft)		Ten Str	i	t	Comp Str	i	t	ENTHRU	Bl Rt
kips	b/ft	down	up	ksi			ksi			kip-ft	b/min
19.4	1.4	3.75	3.78	-1.52	9	15	11.27	1	7	22.1	61.0
19.4	1.4	3.75	3.78	-1.52	9	15	11.27	1	7	22.1	61.0
19.4	1.4	3.75	3.78	-1.52	9	15	11.27	1	7	22.1	61.0
19.4	1.4	3.75	3.78	-1.52	9	15	11.27	1	7	22.1	61.0
19.4	1.4	3.75	3.78	-1.52	9	15	11.27	1	7	22.1	61.0

1 0 10.81000 11.86000

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Depth (ft) 18.0
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 10.000

L b Top	Area	E-Mod	Spec Wt	Perim	C Index	Wave Sp	EA/c
ft	in2	ksi	lb/ft3	ft		ft/s	k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model — Total Capacity Rut (kips) 18.8

No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Perim	Area
	kips	k/in	ft	ft		kips	s/ft	inch	ft	ft	in2
1	0.142	8919	0.010	0.000	0.85	0.0	0.000	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	0.0	0.000	0.100	6.72	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	0.1	0.050	0.100	67.20	3.3	12.4
21	0.142	8919	0.000	0.000	1.00	1.1	0.050	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	2.4	0.050	0.100	73.92	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	3.7	0.050	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	4.9	0.050	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	6.1	0.051	0.100	84.00	3.3	12.4
Toe						0.5	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
3.559 kips total reduced pile weight (g= 32.17 ft/s2)

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
18.02	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut	Bl Ct	Stroke (ft)		Ten Str	i	t	Comp Str	i	t	ENTHRU	Bl Rt
kips	b/ft	down	up	ksi			ksi			kip-ft	b/min
18.8	1.4	3.76	3.74	-1.58	9	15	11.19	1	7	21.6	61.1
18.8	1.4	3.72	3.75	-1.64	9	15	11.16	1	7	21.6	61.2
18.8	1.4	3.76	3.74	-1.55	9	15	11.24	1	7	21.5	61.1
18.8	1.4	3.76	3.74	-1.56	9	15	11.22	1	7	21.5	61.1
18.8	1.4	3.72	3.75	-1.65	9	15	11.14	1	7	21.6	61.2

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FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Depth (ft) 21.0
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown

Pile Size (inch) 10.000

L b Top	Area	E-Mod	Spec Wt	Perim	C Index	Wave Sp	EA/c
ft	in2	ksi	lb/ft3	ft		ft/s	k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model Total Capacity Rut (kips) 22.2

No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Perim	Area
	kips	k/in	ft	ft		kips	s/ft	inch	ft	ft	in2
1	0.142	8919	0.010	0.000	0.85	0.0	0.000	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	0.0	0.000	0.100	6.72	3.3	12.4
19	0.142	8919	0.000	0.000	1.00	0.0	0.050	0.100	63.84	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	1.0	0.050	0.100	67.20	3.3	12.4
21	0.142	8919	0.000	0.000	1.00	2.2	0.050	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	3.5	0.050	0.100	73.92	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	4.8	0.050	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	6.0	0.050	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	4.1	0.183	0.100	84.00	3.3	12.4
Toe						0.5	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
3.559 kips total reduced pile weight (g= 32.17 ft/s2)

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
21.00	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut	Bl Ct	Stroke (ft) down	up	Ten Str	i	t	Comp Str	i	t	ENTHRU	Bl Rt
kips	b/ft	down	up	ksi			ksi			kip-ft	b/min
22.2	1.6	3.96	3.93	-0.76	4	14	13.05	1	2	23.7	59.8
22.4	1.6	3.96	3.93	-0.75	4	14	13.12	1	2	23.7	59.7
22.5	1.6	3.97	3.94	-0.74	4	14	13.20	1	2	23.7	59.7
22.7	1.6	3.97	3.95	-0.72	4	15	13.27	1	2	23.7	59.6
22.9	1.6	3.98	3.95	-0.71	4	14	13.34	1	2	23.6	59.6

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FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Depth (ft) 24.0
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 10.000

L b Top	Area	E-Mod	Spec Wt	Perim	C Index	Wave Sp	EA/c
ft	in2	ksi	lb/ft3	ft		ft/s	k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model Total Capacity Rut (kips) 25.6

No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Perim	Area
	kips	k/in	ft	ft		kips	s/ft	inch	ft	ft	in2
1	0.142	8919	0.010	0.000	0.85	0.0	0.000	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	0.0	0.000	0.100	6.72	3.3	12.4
18	0.142	8919	0.000	0.000	1.00	0.0	0.050	0.100	60.48	3.3	12.4
19	0.142	8919	0.000	0.000	1.00	0.8	0.050	0.100	63.84	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	2.1	0.050	0.100	67.20	3.3	12.4
21	0.142	8919	0.000	0.000	1.00	3.4	0.050	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	4.7	0.050	0.100	73.92	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	5.9	0.050	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	4.4	0.166	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	3.9	0.200	0.100	84.00	3.3	12.4
Toe						0.5	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
3.559 kips total reduced pile weight (g= 32.17 ft/s2)

Depth ft	Stroke ft	Pressure Ratio	Efficy
23.98	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut kips	Bl Ct b/ft	Stroke (ft) down	up	Ten Str ksi	i	t	Comp Str ksi	i	t	ENTHRU kip-ft	Bl Rt b/min
25.6	1.8	4.06	4.09	-0.58	4	14	14.31	1	2	23.1	58.6
26.0	1.8	4.08	4.10	-0.55	2	12	14.46	1	2	23.1	58.5
26.3	1.9	4.10	4.12	-0.59	2	12	14.66	1	2	23.2	58.4
26.7	1.9	4.16	4.13	-0.73	2	12	15.04	1	2	23.3	58.2
27.1	1.9	4.16	4.15	-0.69	2	12	15.09	1	2	23.3	58.1

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FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Depth (ft) 24.0
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 10.000

L b Top ft	Area in2	E-Mod ksi	Spec Wt lb/ft3	Perim ft	C Index	Wave Sp ft/s	EA/c k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model — Total Capacity Rut (kips) 26.5

No.	Weight kips	Stiffn k/in	C-Slk ft	T-Slk ft	CoR	Soil-S kips	Soil-D s/ft	Quake inch	LbTop ft	Perim ft	Area in2
1	0.142	8919	0.010	0.000	0.85	0.0	0.000	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	0.0	0.000	0.100	6.72	3.3	12.4
18	0.142	8919	0.000	0.000	1.00	0.0	0.050	0.100	60.48	3.3	12.4
19	0.142	8919	0.000	0.000	1.00	0.8	0.050	0.100	63.84	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	2.1	0.050	0.100	67.20	3.3	12.4
21	0.142	8919	0.000	0.000	1.00	3.4	0.050	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	4.7	0.050	0.100	73.92	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	5.9	0.050	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	4.3	0.168	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	3.9	0.200	0.100	84.00	3.3	12.4
Toe						1.4	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
3.559 kips total reduced pile weight (g= 32.17 ft/s2)

Depth ft	Stroke ft	Pressure Ratio	Efficy
24.02	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut kips	Bl Ct b/ft	Stroke (ft) down	up	Ten Str ksi	i	t	Comp Str ksi	i	t	ENTHRU kip-ft	Bl Rt b/min
26.5	1.9	4.11	4.13	-0.60	2	12	14.70	1	2	23.2	58.3
26.9	1.9	4.16	4.13	-0.72	2	12	15.05	1	2	23.3	58.2
27.3	1.9	4.16	4.16	-0.72	2	12	15.13	1	2	23.3	58.1
27.7	2.0	4.16	4.18	-0.67	2	12	15.17	1	2	23.2	58.0
28.0	2.0	4.21	4.18	-0.76	2	12	15.46	1	2	23.3	57.8

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FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Depth (ft) 26.5
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 10.000

L b Top	Area	E-Mod	Spec Wt	Perim	C Index	Wave Sp	EA/c
ft	in2	ksi	lb/ft3	ft		ft/s	k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model Total Capacity Rut (kips) 30.9

No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Perim	Area
	kips	k/in	ft	ft		kips	s/ft	inch	ft	ft	in2
1	0.142	8919	0.010	0.000	0.85	0.0	0.000	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	0.0	0.000	0.100	6.72	3.3	12.4
18	0.142	8919	0.000	0.000	1.00	0.5	0.050	0.100	60.48	3.3	12.4
19	0.142	8919	0.000	0.000	1.00	1.8	0.050	0.100	63.84	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	3.1	0.050	0.100	67.20	3.3	12.4
21	0.142	8919	0.000	0.000	1.00	4.3	0.050	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	5.6	0.050	0.100	73.92	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	5.0	0.128	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	3.8	0.200	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	5.4	0.200	0.100	84.00	3.3	12.4
Toe						1.4	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
3.559 kips total reduced pile weight (g= 32.17 ft/s2)

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
26.50	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut	Bl Ct	Stroke (ft)		Ten Str	i	t	Comp Str	i	t	ENTHRU	Bl Rt
kips	b/ft	down	up	ksi			ksi			kip-ft	b/min
30.9	2.4	4.30	4.33	-0.71	2	12	16.24	3	3	22.5	57.0
31.8	2.5	4.34	4.37	-0.69	2	12	16.50	5	3	22.4	56.8
32.8	2.6	4.37	4.40	-0.64	2	12	16.71	6	3	22.2	56.5
33.7	2.7	4.40	4.44	-0.60	2	12	16.94	7	3	22.0	56.3
34.6	2.8	4.48	4.46	-0.66	2	12	17.38	8	4	22.1	56.0

1 0 10.81000 11.86000

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Depth (ft) 29.0
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 10.000

L b Top	Area	E-Mod	Spec Wt	Perim	C Index	Wave Sp	EA/c
ft	in2	ksi	lb/ft3	ft		ft/s	k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model Total Capacity Rut (kips) 35.4

No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Perim	Area
	kips	k/in	ft	ft		kips	s/ft	inch	ft	ft	in2
1	0.142	8919	0.010	0.000	0.85	0.0	0.000	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	0.0	0.000	0.100	6.72	3.3	12.4
17	0.142	8919	0.000	0.000	1.00	0.2	0.050	0.100	57.12	3.3	12.4
18	0.142	8919	0.000	0.000	1.00	1.4	0.050	0.100	60.48	3.3	12.4
19	0.142	8919	0.000	0.000	1.00	2.7	0.050	0.100	63.84	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	4.0	0.050	0.100	67.20	3.3	12.4

21	0.142	8919	0.000	0.000	1.00	5.3	0.050	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	5.6	0.090	0.100	73.92	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	3.8	0.200	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	4.9	0.200	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	6.0	0.200	0.100	84.00	3.3	12.4
Toe						1.4	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
3.559 kips total reduced pile weight (g= 32.17 ft/s2)

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
28.98	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut	Bl Ct	Stroke (ft)		Ten Str	i	t	Comp Str	i	t	ENTHRU	Bl Rt
kips	b/ft	down	up	ksi			ksi			kip-ft	b/min
35.4	2.9	4.51	4.49	-0.71	2	12	17.60	9	4	21.9	55.8
36.8	3.1	4.57	4.55	-0.64	2	12	17.99	10	4	21.7	55.5
38.3	3.3	4.62	4.60	-0.54	2	12	18.30	12	4	21.5	55.1
39.8	3.5	4.67	4.65	-0.45	2	12	18.65	12	4	21.3	54.8
41.3	3.7	4.73	4.70	-0.36	3	14	18.97	14	5	21.2	54.5

1 0 10.81000 11.86000

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Depth (ft) 29.0
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 10.000

L b Top	Area	E-Mod	Spec Wt	Perim	C Index	Wave Sp	EA/c
ft	in2	ksi	lb/ft3	ft		ft/s	k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model Total Capacity Rut (kips) 36.9

No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Perim	Area
	kips	k/in	ft	ft		kips	s/ft	inch	ft	ft	in2
1	0.142	8919	0.010	0.000	0.85	0.0	0.000	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	0.0	0.000	0.100	6.72	3.3	12.4
17	0.142	8919	0.000	0.000	1.00	0.3	0.050	0.100	57.12	3.3	12.4
18	0.142	8919	0.000	0.000	1.00	1.5	0.050	0.100	60.48	3.3	12.4
19	0.142	8919	0.000	0.000	1.00	2.7	0.050	0.100	63.84	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	4.0	0.050	0.100	67.20	3.3	12.4
21	0.142	8919	0.000	0.000	1.00	5.3	0.050	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	5.6	0.092	0.100	73.92	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	3.8	0.200	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	4.9	0.200	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	6.0	0.199	0.100	84.00	3.3	12.4
Toe						2.8	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
3.559 kips total reduced pile weight (g= 32.17 ft/s2)

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
29.02	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut	Bl Ct	Stroke (ft)		Ten Str	i	t	Comp Str	i	t	ENTHRU	Bl Rt
kips	b/ft	down	up	ksi			ksi			kip-ft	b/min
36.9	3.1	4.57	4.55	-0.64	2	12	17.96	11	4	21.7	55.5
38.4	3.3	4.62	4.60	-0.57	2	12	18.32	12	4	21.5	55.2

39.9	3.5	4.67	4.64	-0.46	2	12	18.64	13	4	21.3	54.8
41.4	3.7	4.72	4.70	-0.36	3	14	18.96	14	5	21.2	54.5
42.9	3.9	4.77	4.75	-0.34	3	15	19.26	14	5	21.0	54.3

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FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Depth (ft) 31.5
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 10.000

L b Top	Area	E-Mod	Spec Wt	Perim	C Index	Wave Sp	EA/c
ft	in2	ksi	lb/ft3	ft		ft/s	k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model Total Capacity Rut (kips) 43.1

No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Perim	Area
	kips	k/in	ft	ft		kips	s/ft	inch	ft	ft	in2
1	0.142	8919	0.010	0.000	0.85	0.0	0.000	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	0.0	0.000	0.100	6.72	3.3	12.4
16	0.142	8919	0.000	0.000	1.00	0.1	0.050	0.100	53.76	3.3	12.4
17	0.142	8919	0.000	0.000	1.00	1.1	0.050	0.100	57.12	3.3	12.4
18	0.142	8919	0.000	0.000	1.00	2.4	0.050	0.100	60.48	3.3	12.4
19	0.142	8919	0.000	0.000	1.00	3.7	0.050	0.100	63.84	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	5.0	0.050	0.100	67.20	3.3	12.4
21	0.142	8919	0.000	0.000	1.00	6.1	0.053	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	3.8	0.200	0.100	73.92	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	4.3	0.200	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	6.0	0.200	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	7.8	0.100	0.100	84.00	3.3	12.4
Toe						2.8	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
3.559 kips total reduced pile weight (g= 32.17 ft/s2)

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
31.50	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut	Bl Ct	Stroke (ft)		Ten Str	i	t	Comp Str	i	t	ENTHRU	Bl Rt
kips	b/ft	down	up	ksi			ksi			kip-ft	b/min
43.1	3.6	4.71	4.68	-0.75	2	12	19.01	14	5	21.3	54.6
44.7	3.8	4.76	4.74	-0.64	2	12	19.33	15	5	21.0	54.3
46.3	4.0	4.81	4.78	-0.52	2	12	19.68	17	5	20.9	54.0
48.0	4.2	4.86	4.83	-0.41	2	12	19.99	17	5	20.8	53.8
49.6	4.4	4.90	4.87	-0.39	3	14	20.26	17	5	20.6	53.5

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FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Depth (ft) 34.0
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 10.000

L b Top	Area	E-Mod	Spec Wt	Perim	C Index	Wave Sp	EA/c
ft	in2	ksi	lb/ft3	ft		ft/s	k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model — Total Capacity Rut (kips) 49.6

No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Perim	Area
	kips	k/in	ft	ft		kips	s/ft	inch	ft	ft	in2
1	0.142	8919	0.010	0.000	0.85	0.0	0.000	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	0.0	0.000	0.100	6.72	3.3	12.4
15	0.142	8919	0.000	0.000	1.00	0.0	0.050	0.100	50.40	3.3	12.4
16	0.142	8919	0.000	0.000	1.00	0.8	0.050	0.100	53.76	3.3	12.4
17	0.142	8919	0.000	0.000	1.00	2.1	0.050	0.100	57.12	3.3	12.4
18	0.142	8919	0.000	0.000	1.00	3.3	0.050	0.100	60.48	3.3	12.4
19	0.142	8919	0.000	0.000	1.00	4.6	0.050	0.100	63.84	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	5.9	0.050	0.100	67.20	3.3	12.4
21	0.142	8919	0.000	0.000	1.00	4.4	0.162	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	3.9	0.200	0.100	73.92	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	5.9	0.200	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	7.1	0.141	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	8.7	0.050	0.100	84.00	3.3	12.4
Toe						2.8	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
3.559 kips total reduced pile weight (g= 32.17 ft/s2)

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
33.98	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut	Bl Ct	Stroke (ft) down	Stroke (ft) up	Ten Str	i	t	Comp Str	i	t	ENTHRU	Bl Rt
kips	b/ft			ksi			ksi			kip-ft	b/min
49.6	4.2	4.85	4.82	-0.79	2	12	20.00	16	5	20.8	53.8
51.4	4.4	4.89	4.87	-0.66	2	12	20.25	17	5	20.6	53.5
53.1	4.6	4.94	4.90	-0.52	2	12	20.51	17	5	20.4	53.2
54.9	4.8	4.99	4.95	-0.42	3	14	20.76	17	5	20.3	53.0
56.6	5.0	5.03	4.99	-0.42	3	14	20.98	17	5	20.2	52.7

1 0 10.81000 11.86000

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Depth (ft) 34.0
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 10.000

L b Top	Area	E-Mod	Spec Wt	Perim	C Index	Wave Sp	EA/c
ft	in2	ksi	lb/ft3	ft		ft/s	k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model — Total Capacity Rut (kips) 72.5

No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Perim	Area
	kips	k/in	ft	ft		kips	s/ft	inch	ft	ft	in2
1	0.142	8919	0.010	0.000	0.85	0.0	0.000	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	0.0	0.000	0.100	6.72	3.3	12.4
15	0.142	8919	0.000	0.000	1.00	0.0	0.050	0.100	50.40	3.3	12.4
16	0.142	8919	0.000	0.000	1.00	0.8	0.050	0.100	53.76	3.3	12.4
17	0.142	8919	0.000	0.000	1.00	2.1	0.050	0.100	57.12	3.3	12.4
18	0.142	8919	0.000	0.000	1.00	3.4	0.050	0.100	60.48	3.3	12.4
19	0.142	8919	0.000	0.000	1.00	4.6	0.050	0.100	63.84	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	5.9	0.050	0.100	67.20	3.3	12.4
21	0.142	8919	0.000	0.000	1.00	4.4	0.164	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	3.9	0.200	0.100	73.92	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	5.9	0.200	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	7.2	0.140	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	8.8	0.050	0.100	84.00	3.3	12.4
Toe						25.6	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)

3.559 kips total reduced pile weight (g= 32.17 ft/s2)

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
34.02	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut	Bl Ct	Stroke (ft)		Ten Str	i	t	Comp Str	i	t	ENTHRU	Bl Rt
kips	b/ft	down	up	ksi			ksi			kip-ft	b/min
72.5	7.1	5.36	5.33	0.00	1	0	22.50	17	5	19.4	50.9
74.3	7.3	5.39	5.37	-0.07	15	50	22.66	17	5	19.2	50.7
76.1	7.5	5.43	5.41	-0.31	16	50	22.83	17	5	19.2	50.6
77.8	7.7	5.46	5.44	-0.53	16	50	22.97	17	5	19.1	50.4
79.6	7.9	5.50	5.48	-0.70	15	50	23.13	17	5	19.0	50.2

1 0 10.81000 11.86000

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Depth (ft) 39.0
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 10.000

L b Top	Area	E-Mod	Spec Wt	Perim	C Index	Wave Sp	EA/c
ft	in2	ksi	lb/ft3	ft		ft/s	k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model Total Capacity Rut (kips) 107.3

No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Perim	Area
	kips	k/in	ft	ft		kips	s/ft	inch	ft	ft	in2
1	0.142	8919	0.010	0.000	0.85	0.0	0.000	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	0.0	0.000	0.100	6.72	3.3	12.4
14	0.142	8919	0.000	0.000	1.00	0.2	0.050	0.100	47.04	3.3	12.4
15	0.142	8919	0.000	0.000	1.00	1.4	0.050	0.100	50.40	3.3	12.4
16	0.142	8919	0.000	0.000	1.00	2.7	0.050	0.100	53.76	3.3	12.4
17	0.142	8919	0.000	0.000	1.00	4.0	0.050	0.100	57.12	3.3	12.4
18	0.142	8919	0.000	0.000	1.00	5.3	0.050	0.100	60.48	3.3	12.4
19	0.142	8919	0.000	0.000	1.00	5.7	0.087	0.100	63.84	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	3.8	0.200	0.100	67.20	3.3	12.4
21	0.142	8919	0.000	0.000	1.00	4.8	0.200	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	6.0	0.200	0.100	73.92	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	8.4	0.055	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	14.9	0.050	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	22.5	0.050	0.100	84.00	3.3	12.4
Toe						27.6	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
3.559 kips total reduced pile weight (g= 32.17 ft/s2)

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
39.00	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut	Bl Ct	Stroke (ft)		Ten Str	i	t	Comp Str	i	t	ENTHRU	Bl Rt
kips	b/ft	down	up	ksi			ksi			kip-ft	b/min
107.3	10.4	5.77	5.81	-1.62	16	43	24.22	16	5	18.5	48.9
109.0	10.7	5.81	5.85	-1.59	16	43	24.37	16	5	18.5	48.7
110.8	11.0	5.85	5.88	-1.55	16	43	24.54	16	5	18.5	48.6
112.6	11.4	5.88	5.92	-1.53	16	43	24.68	16	5	18.4	48.4
114.3	11.7	5.93	5.95	-1.54	16	43	24.84	16	5	18.4	48.2

1 0 10.81000 11.86000

Depth	(ft)	44.0		
Shaft Gain/Loss Factor		0.400	Toe Gain/Loss Factor	1.000

PILE PROFILE:

Toe Area	(in2)	144.000	Pile Type	Unknown
Pile Size	(inch)	10.000		

L b Top	Area	E-Mod	Spec Wt	Perim	C Index	Wave Sp	EA/c
ft	in2	ksi	lb/ft3	ft		ft/s	k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model — Total Capacity Rut (kips) 145.1

No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Perim	Area
	kips	k/in	ft	ft		kips	s/ft	inch	ft	ft	in2
1	0.142	8919	0.010	0.000	0.85	0.0	0.000	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	0.0	0.000	0.100	6.72	3.3	12.4
12	0.142	8919	0.000	0.000	1.00	0.0	0.050	0.100	40.32	3.3	12.4
13	0.142	8919	0.000	0.000	1.00	0.8	0.050	0.100	43.68	3.3	12.4
14	0.142	8919	0.000	0.000	1.00	2.0	0.050	0.100	47.04	3.3	12.4
15	0.142	8919	0.000	0.000	1.00	3.3	0.050	0.100	50.40	3.3	12.4
16	0.142	8919	0.000	0.000	1.00	4.6	0.050	0.100	53.76	3.3	12.4
17	0.142	8919	0.000	0.000	1.00	5.9	0.050	0.100	57.12	3.3	12.4
18	0.142	8919	0.000	0.000	1.00	4.5	0.159	0.100	60.48	3.3	12.4
19	0.142	8919	0.000	0.000	1.00	3.9	0.200	0.100	63.84	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	5.8	0.200	0.100	67.20	3.3	12.4
21	0.142	8919	0.000	0.000	1.00	7.1	0.145	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	8.7	0.050	0.100	73.92	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	21.3	0.050	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	23.3	0.050	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	24.8	0.050	0.100	84.00	3.3	12.4
Toe						29.2	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
3.559 kips total reduced pile weight (g= 32.17 ft/s2)

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
43.98	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut	Bl Ct	Stroke (ft) down	up	Ten Str	i	t	Comp Str	i	t	ENTHRU	Bl Rt
kips	b/ft	down	up	ksi			ksi			kip-ft	b/min
145.1	14.9	6.24	6.26	-2.94	14	37	25.89	15	5	18.2	47.0
146.9	15.2	6.26	6.28	-2.95	14	37	25.98	15	5	18.2	46.9
148.6	15.5	6.28	6.29	-2.96	14	37	26.08	15	5	18.2	46.9
150.4	15.8	6.31	6.32	-2.93	14	37	26.15	14	5	18.1	46.8
152.2	16.0	6.33	6.33	-2.87	14	37	26.23	15	5	18.1	46.8

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FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Depth	(ft)	44.0		
Shaft Gain/Loss Factor		0.400	Toe Gain/Loss Factor	1.000

PILE PROFILE:

Toe Area	(in2)	144.000	Pile Type	Unknown
Pile Size	(inch)	10.000		

L b Top	Area	E-Mod	Spec Wt	Perim	C Index	Wave Sp	EA/c
ft	in2	ksi	lb/ft3	ft		ft/s	k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

B-20-4

Pile and Soil Model						Total Capacity Rut	(kips)	134.0		
No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Perim	Area
	kips	k/in	ft	ft		kips	s/ft	inch	ft	ft	in2
1	0.142	8919	0.010	0.000	0.85	0.0	0.000	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	0.0	0.000	0.100	6.72	3.3	12.4
12	0.142	8919	0.000	0.000	1.00	0.0	0.050	0.100	40.32	3.3	12.4
13	0.142	8919	0.000	0.000	1.00	0.8	0.050	0.100	43.68	3.3	12.4
14	0.142	8919	0.000	0.000	1.00	2.1	0.050	0.100	47.04	3.3	12.4
15	0.142	8919	0.000	0.000	1.00	3.3	0.050	0.100	50.40	3.3	12.4
16	0.142	8919	0.000	0.000	1.00	4.6	0.050	0.100	53.76	3.3	12.4
17	0.142	8919	0.000	0.000	1.00	5.9	0.050	0.100	57.12	3.3	12.4
18	0.142	8919	0.000	0.000	1.00	4.5	0.161	0.100	60.48	3.3	12.4
19	0.142	8919	0.000	0.000	1.00	3.9	0.200	0.100	63.84	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	5.9	0.200	0.100	67.20	3.3	12.4
21	0.142	8919	0.000	0.000	1.00	7.1	0.143	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	8.7	0.050	0.100	73.92	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	21.5	0.050	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	23.3	0.050	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	24.8	0.050	0.100	84.00	3.3	12.4
Toe						17.7	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
3.559 kips total reduced pile weight (g= 32.17 ft/s2)

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
44.02	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut	Bl Ct	Stroke (ft)		Ten Str	i	t	Comp Str	i	t	ENTHRU	Bl Rt
kips	b/ft	down	up	ksi			ksi			kip-ft	b/min
134.0	13.2	6.09	6.11	-2.86	14	37	25.35	15	5	18.4	47.6
135.7	13.5	6.11	6.14	-2.92	14	37	25.46	14	5	18.3	47.5
137.5	13.8	6.14	6.16	-3.02	14	37	25.59	15	5	18.4	47.4
139.2	14.1	6.17	6.19	-3.06	14	37	25.68	14	5	18.3	47.3
141.0	14.4	6.19	6.21	-3.10	14	37	25.78	15	5	18.3	47.2

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FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Depth (ft) 48.5
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 10.000

L b Top	Area	E-Mod	Spec Wt	Perim	C Index	Wave Sp	EA/c
ft	in2	ksi	lb/ft3	ft		ft/s	k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model						Total Capacity Rut	(kips)	163.8			
No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Perim	Area
	kips	k/in	ft	ft		kips	s/ft	inch	ft	ft	in2
1	0.142	8919	0.010	0.000	0.85	0.0	0.000	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	0.0	0.000	0.100	6.72	3.3	12.4
11	0.142	8919	0.000	0.000	1.00	0.1	0.050	0.100	36.96	3.3	12.4
12	0.142	8919	0.000	0.000	1.00	1.2	0.050	0.100	40.32	3.3	12.4
13	0.142	8919	0.000	0.000	1.00	2.5	0.050	0.100	43.68	3.3	12.4
14	0.142	8919	0.000	0.000	1.00	3.8	0.050	0.100	47.04	3.3	12.4
15	0.142	8919	0.000	0.000	1.00	5.0	0.050	0.100	50.40	3.3	12.4
16	0.142	8919	0.000	0.000	1.00	6.0	0.062	0.100	53.76	3.3	12.4
17	0.142	8919	0.000	0.000	1.00	3.8	0.200	0.100	57.12	3.3	12.4
18	0.142	8919	0.000	0.000	1.00	4.5	0.200	0.100	60.48	3.3	12.4
19	0.142	8919	0.000	0.000	1.00	6.0	0.200	0.100	63.84	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	7.9	0.089	0.100	67.20	3.3	12.4
21	0.142	8919	0.000	0.000	1.00	12.7	0.050	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	22.2	0.050	0.100	73.92	3.3	12.4

No.	Weight kips	Stiffn k/in	C-Slk ft	T-Slk ft	CoR	Soil-S kips	Soil-D s/ft	Quake inch	LbTop ft	Perim ft	Area in2
23	0.142	8919	0.000	0.000	1.00	23.8	0.050	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	24.0	0.050	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	22.5	0.050	0.100	84.00	3.3	12.4
Toe						17.7	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
3.559 kips total reduced pile weight (g= 32.17 ft/s2)

Depth ft	Stroke ft	Pressure Ratio	Efficy
48.50	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut kips	Bl Ct b/ft	Stroke (ft) down	up	Ten Str ksi	i	t	Comp Str ksi	i	t	ENTHRU kip-ft	Bl Rt b/min
163.8	16.3	6.36	6.36	-2.74	10	36	26.30	13	4	18.0	46.6
165.5	16.5	6.37	6.38	-2.69	10	36	26.34	13	4	17.9	46.6
167.3	16.8	6.39	6.40	-2.64	9	36	26.42	13	4	17.9	46.5
169.0	17.1	6.40	6.43	-2.59	9	36	26.46	13	4	17.9	46.5
170.8	17.2	6.43	6.44	-2.54	9	36	26.54	13	4	17.9	46.4

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FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Depth (ft) 53.0
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 10.000

L b Top ft	Area in2	E-Mod ksi	Spec Wt lb/ft3	Perim ft	C Index	Wave Sp ft/s	EA/c k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model Total Capacity Rut (kips) 195.7

No.	Weight kips	Stiffn k/in	C-Slk ft	T-Slk ft	CoR	Soil-S kips	Soil-D s/ft	Quake inch	LbTop ft	Perim ft	Area in2
1	0.142	8919	0.010	0.000	0.85	0.0	0.000	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	0.0	0.000	0.100	6.72	3.3	12.4
10	0.142	8919	0.000	0.000	1.00	0.4	0.050	0.100	33.60	3.3	12.4
11	0.142	8919	0.000	0.000	1.00	1.6	0.050	0.100	36.96	3.3	12.4
12	0.142	8919	0.000	0.000	1.00	2.9	0.050	0.100	40.32	3.3	12.4
13	0.142	8919	0.000	0.000	1.00	4.2	0.050	0.100	43.68	3.3	12.4
14	0.142	8919	0.000	0.000	1.00	5.5	0.050	0.100	47.04	3.3	12.4
15	0.142	8919	0.000	0.000	1.00	5.3	0.111	0.100	50.40	3.3	12.4
16	0.142	8919	0.000	0.000	1.00	3.8	0.200	0.100	53.76	3.3	12.4
17	0.142	8919	0.000	0.000	1.00	5.2	0.200	0.100	57.12	3.3	12.4
18	0.142	8919	0.000	0.000	1.00	6.3	0.185	0.100	60.48	3.3	12.4
19	0.142	8919	0.000	0.000	1.00	8.5	0.050	0.100	63.84	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	17.0	0.050	0.100	67.20	3.3	12.4
21	0.142	8919	0.000	0.000	1.00	22.7	0.050	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	24.3	0.050	0.100	73.92	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	23.1	0.050	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	23.0	0.050	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	24.2	0.050	0.100	84.00	3.3	12.4
Toe						17.7	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
3.559 kips total reduced pile weight (g= 32.17 ft/s2)

Depth ft	Stroke ft	Pressure Ratio	Efficy
52.98	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut kips	Bl Ct b/ft	Stroke (ft) down	up	Ten Str ksi	i	t	Comp Str ksi	i	t	ENTHRU kip-ft	Bl Rt b/min
195.7	19.7	6.60	6.61	-1.86	7	35	27.07	12	4	17.9	45.8
197.5	19.8	6.70	6.63	-1.81	7	35	27.35	12	4	18.1	45.6
199.2	20.1	6.72	6.66	-1.74	7	35	27.43	12	4	18.1	45.5
201.0	20.4	6.74	6.68	-1.67	7	35	27.51	12	4	18.1	45.4
202.8	20.9	6.70	6.71	-1.69	11	31	27.41	12	4	18.0	45.4

1 0 10.81000 11.86000

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Depth (ft) 53.0
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 10.000

L b Top ft	Area in2	E-Mod ksi	Spec Wt lb/ft3	Perim ft	C Index	Wave Sp ft/s	EA/c k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model Total Capacity Rut (kips) 236.7

No.	Weight kips	Stiffn k/in	C-Slk ft	T-Slk ft	CoR	Soil-S kips	Soil-D s/ft	Quake inch	LbTop ft	Perim ft	Area in2
1	0.142	8919	0.010	0.000	0.85	0.0	0.000	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	0.0	0.000	0.100	6.72	3.3	12.4
10	0.142	8919	0.000	0.000	1.00	0.4	0.050	0.100	33.60	3.3	12.4
11	0.142	8919	0.000	0.000	1.00	1.6	0.050	0.100	36.96	3.3	12.4
12	0.142	8919	0.000	0.000	1.00	2.9	0.050	0.100	40.32	3.3	12.4
13	0.142	8919	0.000	0.000	1.00	4.2	0.050	0.100	43.68	3.3	12.4
14	0.142	8919	0.000	0.000	1.00	5.5	0.050	0.100	47.04	3.3	12.4
15	0.142	8919	0.000	0.000	1.00	5.3	0.113	0.100	50.40	3.3	12.4
16	0.142	8919	0.000	0.000	1.00	3.8	0.200	0.100	53.76	3.3	12.4
17	0.142	8919	0.000	0.000	1.00	5.2	0.200	0.100	57.12	3.3	12.4
18	0.142	8919	0.000	0.000	1.00	6.4	0.184	0.100	60.48	3.3	12.4
19	0.142	8919	0.000	0.000	1.00	8.5	0.050	0.100	63.84	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	17.2	0.050	0.100	67.20	3.3	12.4
21	0.142	8919	0.000	0.000	1.00	22.8	0.050	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	24.4	0.050	0.100	73.92	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	23.0	0.050	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	23.0	0.050	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	24.2	0.050	0.100	84.00	3.3	12.4
Toe						58.3	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
3.559 kips total reduced pile weight (g= 32.17 ft/s2)

Depth ft	Stroke ft	Pressure Ratio	Efficy
53.02	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut kips	Bl Ct b/ft	Stroke (ft) down	up	Ten Str ksi	i	t	Comp Str ksi	i	t	ENTHRU kip-ft	Bl Rt b/min
236.7	28.1	7.17	7.13	-3.01	12	28	28.77	12	4	19.1	44.0
238.4	28.4	7.20	7.16	-3.05	12	28	28.90	12	4	19.2	43.9
240.2	28.9	7.22	7.18	-3.10	12	28	28.94	12	4	19.2	43.8
242.0	29.4	7.24	7.21	-3.14	12	27	29.02	12	4	19.2	43.7
243.7	29.8	7.27	7.23	-3.18	12	27	29.11	12	4	19.3	43.7

1 0 10.81000 11.86000

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Depth (ft) 62.8
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:

Toe Area	(in2)	144.000	Pile Type	Unknown
Pile Size	(inch)	10.000		

L b Top	Area	E-Mod	Spec Wt	Perim	C Index	Wave Sp	EA/c
ft	in2	ksi	lb/ft3	ft		ft/s	k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model — Total Capacity Rut (kips) 339.8

No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Perim	Area
	kips	k/in	ft	ft		kips	s/ft	inch	ft	ft	in2
1	0.142	8919	0.010	0.000	0.85	0.0	0.000	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	0.0	0.000	0.100	6.72	3.3	12.4
7	0.142	8919	0.000	0.000	1.00	0.3	0.050	0.100	23.52	3.3	12.4
8	0.142	8919	0.000	0.000	1.00	1.5	0.050	0.100	26.88	3.3	12.4
9	0.142	8919	0.000	0.000	1.00	2.8	0.050	0.100	30.24	3.3	12.4
10	0.142	8919	0.000	0.000	1.00	4.1	0.050	0.100	33.60	3.3	12.4
11	0.142	8919	0.000	0.000	1.00	5.3	0.050	0.100	36.96	3.3	12.4
12	0.142	8919	0.000	0.000	1.00	5.5	0.097	0.100	40.32	3.3	12.4
13	0.142	8919	0.000	0.000	1.00	3.8	0.200	0.100	43.68	3.3	12.4
14	0.142	8919	0.000	0.000	1.00	5.0	0.200	0.100	47.04	3.3	12.4
15	0.142	8919	0.000	0.000	1.00	6.1	0.195	0.100	50.40	3.3	12.4
16	0.142	8919	0.000	0.000	1.00	8.5	0.050	0.100	53.76	3.3	12.4
17	0.142	8919	0.000	0.000	1.00	15.8	0.050	0.100	57.12	3.3	12.4
18	0.142	8919	0.000	0.000	1.00	22.6	0.050	0.100	60.48	3.3	12.4
19	0.142	8919	0.000	0.000	1.00	24.2	0.050	0.100	63.84	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	23.3	0.050	0.100	67.20	3.3	12.4
21	0.142	8919	0.000	0.000	1.00	22.8	0.050	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	24.1	0.050	0.100	73.92	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	32.8	0.050	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	35.5	0.050	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	37.4	0.050	0.100	84.00	3.3	12.4
Toe						58.3	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
3.559 kips total reduced pile weight (g= 32.17 ft/s2)

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
62.75	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut	Bl Ct	Stroke (ft) down	up	Ten Str	i	t	Comp Str	i	t	ENTHRU	Bl Rt
kips	b/ft	down	up	ksi			ksi			kip-ft	b/min
339.8	66.4	8.08	8.07	-4.50	11	24	31.18	10	4	20.8	41.5
341.6	68.6	8.10	8.11	-4.48	11	24	31.26	10	4	20.7	41.4
343.3	70.1	8.11	8.12	-4.50	11	24	31.27	10	4	20.8	41.4
345.1	72.4	8.14	8.15	-4.49	11	24	31.39	10	4	20.7	41.3
346.8	72.8	8.17	8.15	-4.53	11	24	31.48	10	4	20.9	41.2

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FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Depth	(ft)	72.5		
Shaft Gain/Loss Factor		0.400	Toe Gain/Loss Factor	1.000

PILE PROFILE:

Toe Area	(in2)	144.000	Pile Type	Unknown
Pile Size	(inch)	10.000		

L b Top	Area	E-Mod	Spec Wt	Perim	C Index	Wave Sp	EA/c
ft	in2	ksi	lb/ft3	ft		ft/s	k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model Total Capacity Rut (kips) 458.8

No.	Weight kips	Stiffn k/in	C-Slk ft	T-Slk ft	CoR	Soil-S kips	Soil-D s/ft	Quake inch	LbTop ft	Perim ft	Area in2
1	0.142	8919	0.010	0.000	0.85	0.0	0.000	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	0.0	0.000	0.100	6.72	3.3	12.4
4	0.142	8919	0.000	0.000	1.00	0.2	0.050	0.100	13.44	3.3	12.4
5	0.142	8919	0.000	0.000	1.00	1.4	0.050	0.100	16.80	3.3	12.4
6	0.142	8919	0.000	0.000	1.00	2.7	0.050	0.100	20.16	3.3	12.4
7	0.142	8919	0.000	0.000	1.00	3.9	0.050	0.100	23.52	3.3	12.4
8	0.142	8919	0.000	0.000	1.00	5.2	0.050	0.100	26.88	3.3	12.4
9	0.142	8919	0.000	0.000	1.00	5.7	0.082	0.100	30.24	3.3	12.4
10	0.142	8919	0.000	0.000	1.00	3.8	0.200	0.100	33.60	3.3	12.4
11	0.142	8919	0.000	0.000	1.00	4.7	0.200	0.100	36.96	3.3	12.4
12	0.142	8919	0.000	0.000	1.00	6.0	0.200	0.100	40.32	3.3	12.4
13	0.142	8919	0.000	0.000	1.00	8.3	0.062	0.100	43.68	3.3	12.4
14	0.142	8919	0.000	0.000	1.00	14.5	0.050	0.100	47.04	3.3	12.4
15	0.142	8919	0.000	0.000	1.00	22.4	0.050	0.100	50.40	3.3	12.4
16	0.142	8919	0.000	0.000	1.00	24.0	0.050	0.100	53.76	3.3	12.4
17	0.142	8919	0.000	0.000	1.00	23.6	0.050	0.100	57.12	3.3	12.4
18	0.142	8919	0.000	0.000	1.00	22.7	0.050	0.100	60.48	3.3	12.4
19	0.142	8919	0.000	0.000	1.00	23.9	0.050	0.100	63.84	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	31.8	0.050	0.100	67.20	3.3	12.4
21	0.142	8919	0.000	0.000	1.00	35.3	0.050	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	37.2	0.050	0.100	73.92	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	39.1	0.050	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	41.0	0.050	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	42.9	0.050	0.100	84.00	3.3	12.4
Toe						58.3	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
3.559 kips total reduced pile weight (g= 32.17 ft/s2)

Depth ft	Stroke ft	Pressure Ratio	Efficy
72.48	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut kips	Bl Ct b/ft	Stroke (ft) down	up	Ten Str ksi	i	t	Comp Str ksi	i	t	ENTHRU kip-ft	Bl Rt b/min
458.8	520.6	8.69	8.68	-4.12	12	23	32.52	7	3	21.5	40.1
460.6	561.8	8.69	8.69	-3.96	12	23	32.55	7	3	21.4	40.0
462.4	621.8	8.69	8.69	-3.82	11	23	32.55	7	3	21.4	40.0
464.1	684.9	8.68	8.68	-3.75	8	21	32.56	7	3	21.4	40.1
465.9	750.8	8.69	8.69	-3.73	8	21	32.58	7	3	21.3	40.0

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FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Depth (ft) 72.5
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 10.000

L b Top ft	Area in2	E-Mod ksi	Spec Wt lb/ft3	Perim ft	C Index	Wave Sp ft/s	EA/c k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model Total Capacity Rut (kips) 407.0

No.	Weight kips	Stiffn k/in	C-Slk ft	T-Slk ft	CoR	Soil-S kips	Soil-D s/ft	Quake inch	LbTop ft	Perim ft	Area in2
1	0.142	8919	0.010	0.000	0.85	0.0	0.000	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	0.0	0.000	0.100	6.72	3.3	12.4
4	0.142	8919	0.000	0.000	1.00	0.2	0.050	0.100	13.44	3.3	12.4
5	0.142	8919	0.000	0.000	1.00	1.4	0.050	0.100	16.80	3.3	12.4
6	0.142	8919	0.000	0.000	1.00	2.7	0.050	0.100	20.16	3.3	12.4
7	0.142	8919	0.000	0.000	1.00	4.0	0.050	0.100	23.52	3.3	12.4

8	0.142	8919	0.000	0.000	1.00	5.2	0.050	0.100	26.88	3.3	12.4
9	0.142	8919	0.000	0.000	1.00	5.7	0.084	0.100	30.24	3.3	12.4
10	0.142	8919	0.000	0.000	1.00	3.8	0.200	0.100	33.60	3.3	12.4
11	0.142	8919	0.000	0.000	1.00	4.8	0.200	0.100	36.96	3.3	12.4
12	0.142	8919	0.000	0.000	1.00	6.0	0.200	0.100	40.32	3.3	12.4
13	0.142	8919	0.000	0.000	1.00	8.3	0.060	0.100	43.68	3.3	12.4
14	0.142	8919	0.000	0.000	1.00	14.6	0.050	0.100	47.04	3.3	12.4
15	0.142	8919	0.000	0.000	1.00	22.5	0.050	0.100	50.40	3.3	12.4
16	0.142	8919	0.000	0.000	1.00	24.0	0.050	0.100	53.76	3.3	12.4
17	0.142	8919	0.000	0.000	1.00	23.6	0.050	0.100	57.12	3.3	12.4
18	0.142	8919	0.000	0.000	1.00	22.7	0.050	0.100	60.48	3.3	12.4
19	0.142	8919	0.000	0.000	1.00	24.0	0.050	0.100	63.84	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	31.9	0.050	0.100	67.20	3.3	12.4
21	0.142	8919	0.000	0.000	1.00	35.3	0.050	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	37.2	0.050	0.100	73.92	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	39.1	0.050	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	41.0	0.050	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	42.7	0.050	0.100	84.00	3.3	12.4
Toe						6.2	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
3.559 kips total reduced pile weight (g= 32.17 ft/s2)

Depth ft	Stroke ft	Pressure Ratio	Efficy
72.52	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut kips	Bl Ct b/ft	Stroke (ft) down	up	Ten Str ksi	i	t	Comp Str ksi	i	t	ENTHRU kip-ft	Bl Rt b/min
407.0	130.9	8.33	8.40	-3.81	8	22	31.63	7	3	20.6	40.8
408.8	137.4	8.35	8.41	-3.81	9	22	31.69	7	3	20.6	40.7
410.5	144.7	8.37	8.42	-3.79	9	22	31.74	7	3	20.6	40.7
412.3	152.5	8.39	8.43	-3.78	9	21	31.81	7	3	20.6	40.7
414.0	160.6	8.41	8.44	-3.76	9	21	31.87	7	3	20.6	40.7

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FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Depth (ft) 76.5
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 10.000

L b Top ft	Area in2	E-Mod ksi	Spec Wt lb/ft3	Perim ft	C Index	Wave Sp ft/s	EA/c k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model — Total Capacity Rut (kips) 417.0

No.	Weight kips	Stiffn k/in	C-Slk ft	T-Slk ft	CoR	Soil-S kips	Soil-D s/ft	Quake inch	LbTop ft	Perim ft	Area in2
1	0.142	8919	0.010	0.000	0.85	0.0	0.000	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	0.0	0.000	0.100	6.72	3.3	12.4
3	0.142	8919	0.000	0.000	1.00	0.4	0.050	0.100	10.08	3.3	12.4
4	0.142	8919	0.000	0.000	1.00	1.6	0.050	0.100	13.44	3.3	12.4
5	0.142	8919	0.000	0.000	1.00	2.9	0.050	0.100	16.80	3.3	12.4
6	0.142	8919	0.000	0.000	1.00	4.2	0.050	0.100	20.16	3.3	12.4
7	0.142	8919	0.000	0.000	1.00	5.5	0.050	0.100	23.52	3.3	12.4
8	0.142	8919	0.000	0.000	1.00	5.3	0.111	0.100	26.88	3.3	12.4
9	0.142	8919	0.000	0.000	1.00	3.8	0.200	0.100	30.24	3.3	12.4
10	0.142	8919	0.000	0.000	1.00	5.2	0.200	0.100	33.60	3.3	12.4
11	0.142	8919	0.000	0.000	1.00	6.3	0.185	0.100	36.96	3.3	12.4
12	0.142	8919	0.000	0.000	1.00	8.5	0.050	0.100	40.32	3.3	12.4
13	0.142	8919	0.000	0.000	1.00	17.0	0.050	0.100	43.68	3.3	12.4
14	0.142	8919	0.000	0.000	1.00	22.7	0.050	0.100	47.04	3.3	12.4
15	0.142	8919	0.000	0.000	1.00	24.3	0.050	0.100	50.40	3.3	12.4

B-20-4

16	0.142	8919	0.000	0.000	1.00	23.1	0.050	0.100	53.76	3.3	12.4
17	0.142	8919	0.000	0.000	1.00	23.0	0.050	0.100	57.12	3.3	12.4
18	0.142	8919	0.000	0.000	1.00	24.2	0.050	0.100	60.48	3.3	12.4
19	0.142	8919	0.000	0.000	1.00	33.7	0.050	0.100	63.84	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	35.7	0.050	0.100	67.20	3.3	12.4
21	0.142	8919	0.000	0.000	1.00	37.6	0.050	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	39.5	0.050	0.100	73.92	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	41.4	0.050	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	36.5	0.061	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	8.4	0.200	0.100	84.00	3.3	12.4
Toe						6.2	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
3.559 kips total reduced pile weight (g= 32.17 ft/s2)

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
76.50	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut	Bl Ct	Stroke (ft)		Ten Str	i	t	Comp Str	i	t	ENTHRU	Bl Rt
kips	b/ft	down	up	ksi			ksi			kip-ft	b/min
417.0	143.3	8.42	8.44	-3.57	10	21	31.72	6	3	20.4	40.6
419.3	152.0	8.43	8.45	-3.50	10	21	31.79	6	3	20.5	40.6
421.6	167.5	8.45	8.48	-3.39	10	21	31.84	6	3	20.4	40.6
423.9	175.0	8.47	8.48	-3.32	10	21	31.89	6	3	20.5	40.5
426.2	189.1	8.49	8.50	-3.23	10	21	31.94	6	3	20.5	40.5

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FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Depth (ft) 80.5
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 10.000

L b Top	Area	E-Mod	Spec Wt	Perim	C Index	Wave Sp	EA/c
ft	in2	ksi	lb/ft3	ft		ft/s	k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model Total Capacity Rut (kips) 427.0

No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Perim	Area
	kips	k/in	ft	ft		kips	s/ft	inch	ft	ft	in2
1	0.142	8919	0.010	0.000	0.85	0.0	0.000	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	0.6	0.050	0.100	6.72	3.3	12.4
3	0.142	8919	0.000	0.000	1.00	1.9	0.050	0.100	10.08	3.3	12.4
4	0.142	8919	0.000	0.000	1.00	3.1	0.050	0.100	13.44	3.3	12.4
5	0.142	8919	0.000	0.000	1.00	4.4	0.050	0.100	16.80	3.3	12.4
6	0.142	8919	0.000	0.000	1.00	5.7	0.050	0.100	20.16	3.3	12.4
7	0.142	8919	0.000	0.000	1.00	4.8	0.138	0.100	23.52	3.3	12.4
8	0.142	8919	0.000	0.000	1.00	3.8	0.200	0.100	26.88	3.3	12.4
9	0.142	8919	0.000	0.000	1.00	5.5	0.200	0.100	30.24	3.3	12.4
10	0.142	8919	0.000	0.000	1.00	6.8	0.163	0.100	33.60	3.3	12.4
11	0.142	8919	0.000	0.000	1.00	8.6	0.050	0.100	36.96	3.3	12.4
12	0.142	8919	0.000	0.000	1.00	19.5	0.050	0.100	40.32	3.3	12.4
13	0.142	8919	0.000	0.000	1.00	23.0	0.050	0.100	43.68	3.3	12.4
14	0.142	8919	0.000	0.000	1.00	24.6	0.050	0.100	47.04	3.3	12.4
15	0.142	8919	0.000	0.000	1.00	22.5	0.050	0.100	50.40	3.3	12.4
16	0.142	8919	0.000	0.000	1.00	23.2	0.050	0.100	53.76	3.3	12.4
17	0.142	8919	0.000	0.000	1.00	25.9	0.050	0.100	57.12	3.3	12.4
18	0.142	8919	0.000	0.000	1.00	34.1	0.050	0.100	60.48	3.3	12.4
19	0.142	8919	0.000	0.000	1.00	36.0	0.050	0.100	63.84	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	37.9	0.050	0.100	67.20	3.3	12.4
21	0.142	8919	0.000	0.000	1.00	39.8	0.050	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	41.7	0.050	0.100	73.92	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	30.2	0.075	0.100	77.28	3.3	12.4

24	0.142	8919	0.000	0.000	1.00	8.4	0.200	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	8.4	0.200	0.100	84.00	3.3	12.4
Toe						6.2	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
3.559 kips total reduced pile weight (g= 32.17 ft/s2)

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
80.48	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut	Bl Ct	Stroke (ft)		Ten Str	i	t	Comp Str	i	t	ENTHRU	Bl Rt
kips	b/ft	down	up	ksi			ksi			kip-ft	b/min
427.0	176.7	8.50	8.50	-2.70	5	22	31.84	5	3	20.3	40.5
429.8	195.3	8.52	8.52	-2.62	5	22	31.91	5	3	20.3	40.4
432.7	214.2	8.54	8.54	-2.53	5	22	31.94	5	3	20.3	40.4
435.5	237.1	8.56	8.55	-2.44	5	22	32.03	5	3	20.3	40.4
438.4	258.5	8.58	8.56	-2.36	5	22	32.10	5	3	20.3	40.3

1 0 10.81000 11.86000

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Depth (ft) 80.5
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 10.000

L b Top	Area	E-Mod	Spec Wt	Perim	C Index	Wave Sp	EA/c
ft	in2	ksi	lb/ft3	ft		ft/s	k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model Total Capacity Rut (kips) 432.1

No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Perim	Area
	kips	k/in	ft	ft		kips	s/ft	inch	ft	ft	in2
1	0.142	8919	0.010	0.000	0.85	0.0	0.000	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	0.6	0.050	0.100	6.72	3.3	12.4
3	0.142	8919	0.000	0.000	1.00	1.9	0.050	0.100	10.08	3.3	12.4
4	0.142	8919	0.000	0.000	1.00	3.2	0.050	0.100	13.44	3.3	12.4
5	0.142	8919	0.000	0.000	1.00	4.4	0.050	0.100	16.80	3.3	12.4
6	0.142	8919	0.000	0.000	1.00	5.7	0.050	0.100	20.16	3.3	12.4
7	0.142	8919	0.000	0.000	1.00	4.8	0.140	0.100	23.52	3.3	12.4
8	0.142	8919	0.000	0.000	1.00	3.8	0.200	0.100	26.88	3.3	12.4
9	0.142	8919	0.000	0.000	1.00	5.6	0.200	0.100	30.24	3.3	12.4
10	0.142	8919	0.000	0.000	1.00	6.8	0.162	0.100	33.60	3.3	12.4
11	0.142	8919	0.000	0.000	1.00	8.6	0.050	0.100	36.96	3.3	12.4
12	0.142	8919	0.000	0.000	1.00	19.6	0.050	0.100	40.32	3.3	12.4
13	0.142	8919	0.000	0.000	1.00	23.1	0.050	0.100	43.68	3.3	12.4
14	0.142	8919	0.000	0.000	1.00	24.6	0.050	0.100	47.04	3.3	12.4
15	0.142	8919	0.000	0.000	1.00	22.5	0.050	0.100	50.40	3.3	12.4
16	0.142	8919	0.000	0.000	1.00	23.2	0.050	0.100	53.76	3.3	12.4
17	0.142	8919	0.000	0.000	1.00	26.0	0.050	0.100	57.12	3.3	12.4
18	0.142	8919	0.000	0.000	1.00	34.2	0.050	0.100	60.48	3.3	12.4
19	0.142	8919	0.000	0.000	1.00	36.1	0.050	0.100	63.84	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	38.0	0.050	0.100	67.20	3.3	12.4
21	0.142	8919	0.000	0.000	1.00	39.9	0.050	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	41.8	0.050	0.100	73.92	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	29.8	0.076	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	8.4	0.200	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	8.5	0.200	0.100	84.00	3.3	12.4
Toe						11.2	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
3.559 kips total reduced pile weight (g= 32.17 ft/s2)

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
80.52	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut	Bl Ct	Stroke (ft)		Ten Str	i	t	Comp Str	i	t	ENTHRU	Bl Rt
kips	b/ft	down	up	ksi			ksi			kip-ft	b/min
432.1	201.6	8.54	8.53	-2.67	10	21	31.92	5	3	20.4	40.4
434.9	222.4	8.56	8.55	-2.51	10	21	32.00	5	3	20.4	40.4
437.8	242.4	8.59	8.56	-2.41	5	22	32.08	5	3	20.4	40.3
440.6	269.5	8.61	8.57	-2.33	5	22	32.13	5	3	20.5	40.3
443.5	290.0	8.62	8.59	-2.25	5	22	32.19	7	3	20.5	40.2

1 0 10.81000 11.86000

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Depth (ft) 82.2
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 10.000

L b Top	Area	E-Mod	Spec Wt	Perim	C Index	Wave Sp	EA/c
ft	in2	ksi	lb/ft3	ft		ft/s	k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model Total Capacity Rut (kips) 439.2

No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Perim	Area
	kips	k/in	ft	ft		kips	s/ft	inch	ft	ft	in2
1	0.142	8919	0.010	0.000	0.85	0.1	0.050	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	1.2	0.050	0.100	6.72	3.3	12.4
3	0.142	8919	0.000	0.000	1.00	2.5	0.050	0.100	10.08	3.3	12.4
4	0.142	8919	0.000	0.000	1.00	3.8	0.050	0.100	13.44	3.3	12.4
5	0.142	8919	0.000	0.000	1.00	5.1	0.050	0.100	16.80	3.3	12.4
6	0.142	8919	0.000	0.000	1.00	5.9	0.068	0.100	20.16	3.3	12.4
7	0.142	8919	0.000	0.000	1.00	3.8	0.200	0.100	23.52	3.3	12.4
8	0.142	8919	0.000	0.000	1.00	4.5	0.200	0.100	26.88	3.3	12.4
9	0.142	8919	0.000	0.000	1.00	6.0	0.200	0.100	30.24	3.3	12.4
10	0.142	8919	0.000	0.000	1.00	8.0	0.081	0.100	33.60	3.3	12.4
11	0.142	8919	0.000	0.000	1.00	13.2	0.050	0.100	36.96	3.3	12.4
12	0.142	8919	0.000	0.000	1.00	22.3	0.050	0.100	40.32	3.3	12.4
13	0.142	8919	0.000	0.000	1.00	23.9	0.050	0.100	43.68	3.3	12.4
14	0.142	8919	0.000	0.000	1.00	23.9	0.050	0.100	47.04	3.3	12.4
15	0.142	8919	0.000	0.000	1.00	22.6	0.050	0.100	50.40	3.3	12.4
16	0.142	8919	0.000	0.000	1.00	23.8	0.050	0.100	53.76	3.3	12.4
17	0.142	8919	0.000	0.000	1.00	30.9	0.050	0.100	57.12	3.3	12.4
18	0.142	8919	0.000	0.000	1.00	35.1	0.050	0.100	60.48	3.3	12.4
19	0.142	8919	0.000	0.000	1.00	37.0	0.050	0.100	63.84	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	38.9	0.050	0.100	67.20	3.3	12.4
21	0.142	8919	0.000	0.000	1.00	40.8	0.050	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	42.7	0.050	0.100	73.92	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	12.0	0.161	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	8.4	0.200	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	11.3	0.200	0.100	84.00	3.3	12.4
Toe						11.2	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
3.559 kips total reduced pile weight (g= 32.17 ft/s2)

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
82.24	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut Bl Ct Stroke (ft) Ten Str i t Comp Str i t ENTHRU Bl Rt

kips	b/ft	down	up	ksi			ksi			kip-ft	b/min
439.2	239.7	8.60	8.57	-2.27	5	22	32.01	4	2	20.3	40.3
442.1	264.0	8.61	8.58	-2.18	5	22	32.07	4	2	20.3	40.3
444.9	282.3	8.63	8.59	-2.10	5	22	32.13	4	2	20.4	40.2
447.8	303.6	8.65	8.60	-2.02	5	22	32.17	4	2	20.4	40.2
450.7	326.0	8.66	8.61	-1.94	5	22	32.23	4	2	20.4	40.2

1 0 10.81000 11.86000

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Depth (ft) 84.0
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 10.000

L b Top	Area	E-Mod	Spec Wt	Perim	C Index	Wave Sp	EA/c
ft	in2	ksi	lb/ft3	ft		ft/s	k/ft/s
0.0	12.40	29000.	492.0	3.3	0	16524.	21.8
84.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 10.167

Pile and Soil Model Total Capacity Rut (kips) 446.5

No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Perim	Area
	kips	k/in	ft	ft		kips	s/ft	inch	ft	ft	in2
1	0.142	8919	0.010	0.000	0.85	0.6	0.050	0.100	3.36	3.3	12.4
2	0.142	8919	0.000	0.000	1.00	1.9	0.050	0.100	6.72	3.3	12.4
3	0.142	8919	0.000	0.000	1.00	3.2	0.050	0.100	10.08	3.3	12.4
4	0.142	8919	0.000	0.000	1.00	4.5	0.050	0.100	13.44	3.3	12.4
5	0.142	8919	0.000	0.000	1.00	5.8	0.050	0.100	16.80	3.3	12.4
6	0.142	8919	0.000	0.000	1.00	4.7	0.145	0.100	20.16	3.3	12.4
7	0.142	8919	0.000	0.000	1.00	3.8	0.200	0.100	23.52	3.3	12.4
8	0.142	8919	0.000	0.000	1.00	5.6	0.200	0.100	26.88	3.3	12.4
9	0.142	8919	0.000	0.000	1.00	6.9	0.157	0.100	30.24	3.3	12.4
10	0.142	8919	0.000	0.000	1.00	8.7	0.050	0.100	33.60	3.3	12.4
11	0.142	8919	0.000	0.000	1.00	20.1	0.050	0.100	36.96	3.3	12.4
12	0.142	8919	0.000	0.000	1.00	23.1	0.050	0.100	40.32	3.3	12.4
13	0.142	8919	0.000	0.000	1.00	24.7	0.050	0.100	43.68	3.3	12.4
14	0.142	8919	0.000	0.000	1.00	22.4	0.050	0.100	47.04	3.3	12.4
15	0.142	8919	0.000	0.000	1.00	23.2	0.050	0.100	50.40	3.3	12.4
16	0.142	8919	0.000	0.000	1.00	26.3	0.050	0.100	53.76	3.3	12.4
17	0.142	8919	0.000	0.000	1.00	34.2	0.050	0.100	57.12	3.3	12.4
18	0.142	8919	0.000	0.000	1.00	36.1	0.050	0.100	60.48	3.3	12.4
19	0.142	8919	0.000	0.000	1.00	38.0	0.050	0.100	63.84	3.3	12.4
20	0.142	8919	0.000	0.000	1.00	39.9	0.050	0.100	67.20	3.3	12.4
21	0.142	8919	0.000	0.000	1.00	41.8	0.050	0.100	70.56	3.3	12.4
22	0.142	8919	0.000	0.000	1.00	28.6	0.079	0.100	73.92	3.3	12.4
23	0.142	8919	0.000	0.000	1.00	8.4	0.200	0.100	77.28	3.3	12.4
24	0.142	8919	0.000	0.000	1.00	8.7	0.200	0.100	80.64	3.3	12.4
25	0.142	8919	0.000	0.000	1.00	14.0	0.200	0.100	84.00	3.3	12.4
Toe						11.2	0.150	0.100			

3.559 kips total unreduced pile weight (g= 32.17 ft/s2)
3.559 kips total reduced pile weight (g= 32.17 ft/s2)

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
84.00	10.81	1.00	0.800

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Rut	Bl Ct	Stroke (ft)		Ten Str	i	t	Comp Str	i	t	ENTHRU	Bl Rt
kips	b/ft	down	up	ksi			ksi			kip-ft	b/min
446.5	285.2	8.65	8.60	-1.85	5	22	32.20	2	2	20.3	40.2
449.4	304.8	8.67	8.61	-1.77	4	22	32.26	2	2	20.3	40.2
452.3	327.3	8.68	8.62	-1.70	4	22	32.30	2	2	20.3	40.2
455.1	353.9	8.70	8.62	-1.69	6	19	32.34	2	2	20.3	40.1
458.0	383.7	8.70	8.63	-1.66	6	19	32.36	2	2	20.3	40.1

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020

SUMMARY OVER DEPTHS

G/L at Shaft and Toe: 0.400 1.000

Depth	Rut	Frictn	End Bg	Bl Ct	Com Str	Ten Str	Stroke	ENTHRU
ft	kips	kips	kips	bl/ft	ksi	ksi	ft	kip-ft
9.0	5.7	4.6	1.1	0.0	0.000	0.000	10.81	0.0
18.0	19.4	18.3	1.1	1.4	11.266	-1.525	3.75	22.1
18.0	18.8	18.3	0.5	1.4	11.195	-1.584	3.76	21.6
21.0	22.2	21.7	0.5	1.6	13.052	-0.763	3.96	23.7
24.0	25.6	25.1	0.5	1.8	14.315	-0.579	4.06	23.1
24.0	26.5	25.2	1.4	1.9	14.702	-0.601	4.11	23.2
26.5	30.9	29.5	1.4	2.4	16.237	-0.712	4.30	22.5
29.0	35.4	34.0	1.4	2.9	17.599	-0.709	4.51	21.9
29.0	36.9	34.1	2.8	3.1	17.963	-0.641	4.57	21.7
31.5	43.1	40.3	2.8	3.6	19.013	-0.753	4.71	21.3
34.0	49.6	46.8	2.8	4.2	19.998	-0.793	4.85	20.8
34.0	72.5	46.9	25.6	7.1	22.501	0.000	5.36	19.4
39.0	107.3	79.7	27.6	10.4	24.217	-1.617	5.77	18.5
44.0	145.1	116.0	29.2	14.9	25.894	-2.943	6.24	18.2
44.0	134.0	116.2	17.7	13.2	25.352	-2.857	6.09	18.4
48.5	163.8	146.0	17.7	16.3	26.300	-2.743	6.36	18.0
53.0	195.7	178.0	17.7	19.7	27.066	-1.857	6.60	17.9
53.0	236.7	178.3	58.3	28.1	28.769	-3.008	7.17	19.1
62.8	339.8	281.5	58.3	66.4	31.182	-4.495	8.08	20.8
72.5	458.8	400.5	58.3	520.6	32.520	-4.122	8.69	21.5
72.5	407.0	400.8	6.2	130.9	31.632	-3.812	8.33	20.6
76.5	417.0	410.8	6.2	143.3	31.719	-3.571	8.42	20.4
80.5	427.0	420.8	6.2	176.7	31.839	-2.703	8.50	20.3
80.5	432.1	420.9	11.2	201.6	31.923	-2.666	8.54	20.4
82.2	439.2	428.1	11.2	239.7	32.009	-2.267	8.60	20.3
84.0	446.5	435.4	11.2	285.2	32.196	-1.853	8.65	20.3

Total Driving Time 139 minutes; Total No. of Blows 5662

G/L at Shaft and Toe: 0.450 1.000

Depth	Rut	Frictn	End Bg	Bl Ct	Com Str	Ten Str	Stroke	ENTHRU
ft	kips	kips	kips	bl/ft	ksi	ksi	ft	kip-ft
9.0	5.7	4.6	1.1	0.0	0.000	0.000	11.86	0.0
18.0	19.4	18.3	1.1	1.4	11.266	-1.525	3.75	22.1
18.0	18.8	18.3	0.5	1.4	11.164	-1.641	3.72	21.6
21.0	22.4	21.9	0.5	1.6	13.122	-0.751	3.96	23.7
24.0	26.0	25.5	0.5	1.8	14.456	-0.554	4.08	23.1
24.0	26.9	25.6	1.4	1.9	15.051	-0.724	4.16	23.3
26.5	31.8	30.5	1.4	2.5	16.503	-0.691	4.34	22.4
29.0	36.8	35.5	1.4	3.1	17.990	-0.645	4.57	21.7
29.0	38.4	35.6	2.8	3.3	18.317	-0.569	4.62	21.5
31.5	44.7	41.9	2.8	3.8	19.333	-0.636	4.76	21.0
34.0	51.4	48.5	2.8	4.4	20.246	-0.655	4.89	20.6
34.0	74.3	48.7	25.6	7.3	22.661	-0.073	5.39	19.2
39.0	109.0	81.5	27.6	10.7	24.369	-1.586	5.81	18.5
44.0	146.9	117.7	29.2	15.2	25.984	-2.949	6.26	18.2
44.0	135.7	118.0	17.7	13.5	25.457	-2.924	6.11	18.3
48.5	165.5	147.8	17.7	16.5	26.341	-2.690	6.37	17.9
53.0	197.5	179.7	17.7	19.8	27.351	-1.807	6.70	18.1
53.0	238.4	180.1	58.3	28.4	28.897	-3.054	7.20	19.2
62.8	341.6	283.2	58.3	68.6	31.262	-4.476	8.10	20.7
72.5	460.6	402.3	58.3	561.8	32.549	-3.961	8.69	21.4
72.5	408.8	402.6	6.2	137.4	31.687	-3.807	8.35	20.6
76.5	419.3	413.1	6.2	152.0	31.787	-3.497	8.43	20.5
80.5	429.8	423.6	6.2	195.3	31.907	-2.622	8.52	20.3
80.5	434.9	423.8	11.2	222.4	31.996	-2.511	8.56	20.4
82.2	442.1	430.9	11.2	264.0	32.068	-2.176	8.61	20.3
84.0	449.4	438.2	11.2	304.8	32.260	-1.774	8.67	20.3

Total Driving Time 148 minutes; Total No. of Blows 6053

SUMMARY OVER DEPTHS

G/L at Shaft and Toe: 0.500 1.000

Depth	Rut	Frictn	End Bg	Bl Ct	Com Str	Ten Str	Stroke	ENTHRU

ft	kips	kips	kips	bl/ft	ksi	ksi	ft	kip-ft
9.0	5.7	4.6	1.1	0.0	0.000	0.000	11.86	0.0
18.0	19.4	18.3	1.1	1.4	11.266	-1.525	3.75	22.1
18.0	18.8	18.3	0.5	1.4	11.244	-1.552	3.76	21.5
21.0	22.5	22.1	0.5	1.6	13.200	-0.737	3.97	23.7
24.0	26.3	25.9	0.5	1.9	14.661	-0.588	4.10	23.2
24.0	27.3	25.9	1.4	1.9	15.128	-0.719	4.16	23.3
26.5	32.8	31.4	1.4	2.6	16.708	-0.642	4.37	22.2
29.0	38.3	37.0	1.4	3.3	18.304	-0.535	4.62	21.5
29.0	39.9	37.1	2.8	3.5	18.636	-0.457	4.67	21.3
31.5	46.3	43.5	2.8	4.0	19.679	-0.523	4.81	20.9
34.0	53.1	50.3	2.8	4.6	20.509	-0.520	4.94	20.4
34.0	76.1	50.4	25.6	7.5	22.832	-0.314	5.43	19.2
39.0	110.8	83.2	27.6	11.0	24.544	-1.547	5.85	18.5
44.0	148.6	119.5	29.2	15.5	26.078	-2.955	6.28	18.2
44.0	137.5	119.7	17.7	13.8	25.588	-3.021	6.14	18.4
48.5	167.3	149.5	17.7	16.8	26.421	-2.643	6.39	17.9
53.0	199.2	181.5	17.7	20.1	27.427	-1.743	6.72	18.1
53.0	240.2	181.8	58.3	28.9	28.939	-3.102	7.22	19.2
62.8	343.3	285.0	58.3	70.1	31.274	-4.504	8.11	20.8
72.5	462.4	404.0	58.3	621.8	32.550	-3.820	8.69	21.4
72.5	410.5	404.3	6.2	144.7	31.744	-3.791	8.37	20.6
76.5	421.6	415.4	6.2	167.5	31.844	-3.391	8.45	20.4
80.5	432.7	426.5	6.2	214.2	31.943	-2.529	8.54	20.3
80.5	437.8	426.6	11.2	242.4	32.076	-2.406	8.59	20.4
82.2	444.9	433.8	11.2	282.3	32.128	-2.101	8.63	20.4
84.0	452.3	441.1	11.2	327.3	32.300	-1.701	8.68	20.3

Total Driving Time 161 minutes; Total No. of Blows 6553

G/L at Shaft and Toe: 0.550 1.000

Depth	Rut	Frictn	End Bg	Bl Ct	Com Str	Ten Str	Stroke	ENTHRU
ft	kips	kips	kips	bl/ft	ksi	ksi	ft	kip-ft
9.0	5.7	4.6	1.1	0.0	0.000	0.000	11.86	0.0
18.0	19.4	18.3	1.1	1.4	11.266	-1.525	3.75	22.1
18.0	18.8	18.3	0.5	1.4	11.225	-1.561	3.76	21.5
21.0	22.7	22.2	0.5	1.6	13.272	-0.723	3.97	23.7
24.0	26.7	26.2	0.5	1.9	15.035	-0.729	4.16	23.3
24.0	27.7	26.3	1.4	2.0	15.173	-0.670	4.16	23.2
26.5	33.7	32.3	1.4	2.7	16.938	-0.604	4.40	22.0
29.0	39.8	38.4	1.4	3.5	18.651	-0.448	4.67	21.3
29.0	41.4	38.6	2.8	3.7	18.965	-0.360	4.72	21.2
31.5	48.0	45.1	2.8	4.2	19.987	-0.408	4.86	20.8
34.0	54.9	52.0	2.8	4.8	20.763	-0.421	4.99	20.3
34.0	77.8	52.2	25.6	7.7	22.966	-0.533	5.46	19.1
39.0	112.6	85.0	27.6	11.4	24.678	-1.525	5.88	18.4
44.0	150.4	121.2	29.2	15.8	26.152	-2.925	6.31	18.1
44.0	139.2	121.5	17.7	14.1	25.678	-3.060	6.17	18.3
48.5	169.0	151.3	17.7	17.1	26.462	-2.593	6.40	17.9
53.0	201.0	183.3	17.7	20.4	27.507	-1.673	6.74	18.1
53.0	242.0	183.6	58.3	29.4	29.024	-3.142	7.24	19.2
62.8	345.1	286.7	58.3	72.4	31.387	-4.487	8.14	20.7
72.5	464.1	405.8	58.3	684.9	32.560	-3.747	8.68	21.4
72.5	412.3	406.1	6.2	152.5	31.814	-3.776	8.39	20.6
76.5	423.9	417.7	6.2	175.0	31.892	-3.322	8.47	20.5
80.5	435.5	429.3	6.2	237.1	32.026	-2.443	8.56	20.3
80.5	440.6	429.5	11.2	269.5	32.132	-2.327	8.61	20.5
82.2	447.8	436.6	11.2	303.6	32.174	-2.020	8.65	20.4
84.0	455.1	444.0	11.2	353.9	32.342	-1.693	8.70	20.3

Total Driving Time 174 minutes; Total No. of Blows 7072

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

SUMMARY OVER DEPTHS

G/L at Shaft and Toe: 0.600 1.000

Depth	Rut	Frictn	End Bg	Bl Ct	Com Str	Ten Str	Stroke	ENTHRU
ft	kips	kips	kips	bl/ft	ksi	ksi	ft	kip-ft
9.0	5.7	4.6	1.1	0.0	0.000	0.000	11.86	0.0
18.0	19.4	18.3	1.1	1.4	11.266	-1.525	3.75	22.1
18.0	18.8	18.3	0.5	1.4	11.144	-1.651	3.72	21.6
21.0	22.9	22.4	0.5	1.6	13.340	-0.714	3.98	23.6
24.0	27.1	26.6	0.5	1.9	15.085	-0.689	4.16	23.3

24.0	28.0	26.7	1.4	2.0	15.459	-0.758	4.21	23.3
26.5	34.6	33.2	1.4	2.8	17.378	-0.658	4.48	22.1
29.0	41.3	39.9	1.4	3.7	18.973	-0.358	4.73	21.2
29.0	42.9	40.0	2.8	3.9	19.258	-0.342	4.77	21.0
31.5	49.6	46.7	2.8	4.4	20.256	-0.387	4.90	20.6
34.0	56.6	53.8	2.8	5.0	20.983	-0.418	5.03	20.2
34.0	79.6	54.0	25.6	7.9	23.131	-0.696	5.50	19.0
39.0	114.3	86.7	27.6	11.7	24.835	-1.542	5.93	18.4
44.0	152.2	123.0	29.2	16.0	26.231	-2.872	6.33	18.1
44.0	141.0	123.3	17.7	14.4	25.777	-3.105	6.19	18.3
48.5	170.8	153.0	17.7	17.2	26.540	-2.541	6.43	17.9
53.0	202.8	185.0	17.7	20.9	27.413	-1.689	6.70	18.0
53.0	243.7	185.4	58.3	29.8	29.108	-3.178	7.27	19.3
62.8	346.8	288.5	58.3	72.8	31.484	-4.529	8.17	20.9
72.5	465.9	407.5	58.3	750.8	32.577	-3.734	8.69	21.3
72.5	414.0	407.8	6.2	160.6	31.866	-3.761	8.41	20.6
76.5	426.2	420.0	6.2	189.1	31.938	-3.233	8.49	20.5
80.5	438.4	432.2	6.2	258.5	32.098	-2.362	8.58	20.3
80.5	443.5	432.3	11.2	290.0	32.187	-2.250	8.62	20.5
82.2	450.7	439.5	11.2	326.0	32.230	-1.941	8.66	20.4
84.0	458.0	446.8	11.2	383.7	32.357	-1.659	8.70	20.3

Total Driving Time 187 minutes; Total No. of Blows 7604

FRA-70-1373B - RA - B-020-4-13 - HP10x42 04/06/2020
Resource International Inc GRLWEAP Version 2010

Table of Depths Analyzed with Driving System Modifiers

Depth ft	Temp. Length ft	Wait Time hr	Equivalent Stroke ft	Pressure Ratio	Efficy.	Stiffn. Factor	Cushion CoR
9.00	84.00	0.00	10.81	1.00	0.80	1.00	1.00
17.98	84.00	0.00	10.81	1.00	0.80	1.00	1.00
18.02	84.00	0.00	10.81	1.00	0.80	1.00	1.00
21.00	84.00	0.00	10.81	1.00	0.80	1.00	1.00
23.98	84.00	0.00	10.81	1.00	0.80	1.00	1.00
24.02	84.00	0.00	10.81	1.00	0.80	1.00	1.00
26.50	84.00	0.00	10.81	1.00	0.80	1.00	1.00
28.98	84.00	0.00	10.81	1.00	0.80	1.00	1.00
29.02	84.00	0.00	10.81	1.00	0.80	1.00	1.00
31.50	84.00	0.00	10.81	1.00	0.80	1.00	1.00
33.98	84.00	0.00	10.81	1.00	0.80	1.00	1.00
34.02	84.00	0.00	10.81	1.00	0.80	1.00	1.00
39.00	84.00	0.00	10.81	1.00	0.80	1.00	1.00
43.98	84.00	0.00	10.81	1.00	0.80	1.00	1.00
44.02	84.00	0.00	10.81	1.00	0.80	1.00	1.00
48.50	84.00	0.00	10.81	1.00	0.80	1.00	1.00
52.98	84.00	0.00	10.81	1.00	0.80	1.00	1.00
53.02	84.00	0.00	10.81	1.00	0.80	1.00	1.00
62.75	84.00	0.00	10.81	1.00	0.80	1.00	1.00
72.48	84.00	0.00	10.81	1.00	0.80	1.00	1.00
72.52	84.00	0.00	10.81	1.00	0.80	1.00	1.00
76.50	84.00	0.00	10.81	1.00	0.80	1.00	1.00
80.48	84.00	0.00	10.81	1.00	0.80	1.00	1.00
80.52	84.00	0.00	10.81	1.00	0.80	1.00	1.00
82.24	84.00	0.00	10.81	1.00	0.80	1.00	1.00
84.00	84.00	0.00	10.81	1.00	0.80	1.00	1.00

Soil Layer Resistance Values

Depth ft	Shaft Res. k/ft2	End Bearing kips	Shaft Quake inch	Toe Quake inch	Shaft Damping s/ft	Toe Damping s/ft	Soil Setup Normlzd	Limit Distance ft	Setup Time hrs
0.01	0.00	0.00	0.100	0.100	0.050	0.150	0.000	0.000	0.000
9.01	0.31	1.15	0.100	0.100	0.050	0.150	0.000	0.000	0.000
16.49	0.57	1.15	0.100	0.100	0.050	0.150	0.000	0.000	0.000
16.51	0.57	1.15	0.100	0.100	0.050	0.150	0.000	0.000	0.000
17.99	0.59	1.15	0.100	0.100	0.050	0.150	0.000	0.000	0.000
18.01	0.56	0.48	0.100	0.100	0.200	0.150	0.660	0.000	0.000
23.99	0.58	0.48	0.100	0.100	0.200	0.150	0.660	0.000	0.000
24.01	1.32	1.36	0.100	0.100	0.200	0.150	1.000	0.000	0.000
28.99	1.38	1.36	0.100	0.100	0.200	0.150	1.000	0.000	0.000
29.01	0.92	2.84	0.100	0.100	0.050	0.150	0.340	0.000	0.000

33.99	1.02	2.84	0.100	0.100	0.050	0.150	0.340	0.000	0.000
34.01	1.89	25.61	0.100	0.100	0.050	0.150	0.000	0.000	0.000
43.01	2.27	29.17	0.100	0.100	0.050	0.150	0.000	0.000	0.000
43.99	2.31	29.17	0.100	0.100	0.050	0.150	0.000	0.000	0.000
44.01	1.94	17.74	0.100	0.100	0.050	0.150	0.000	0.000	0.000
52.99	2.24	17.74	0.100	0.100	0.050	0.150	0.000	0.000	0.000
53.01	2.96	58.35	0.100	0.100	0.050	0.150	0.000	0.000	0.000
62.01	3.42	58.35	0.100	0.100	0.050	0.150	0.000	0.000	0.000
71.01	3.88	58.35	0.100	0.100	0.050	0.150	0.000	0.000	0.000
72.49	3.95	58.35	0.100	0.100	0.050	0.150	0.000	0.000	0.000
72.51	1.26	6.20	0.100	0.100	0.200	0.150	0.660	0.000	0.000
80.49	1.26	6.20	0.100	0.100	0.200	0.150	0.660	0.000	0.000
80.51	1.26	11.16	0.100	0.100	0.200	0.150	0.000	0.000	0.000
84.00	1.26	11.16	0.100	0.100	0.200	0.150	0.000	0.000	0.000

APPENDIX VII

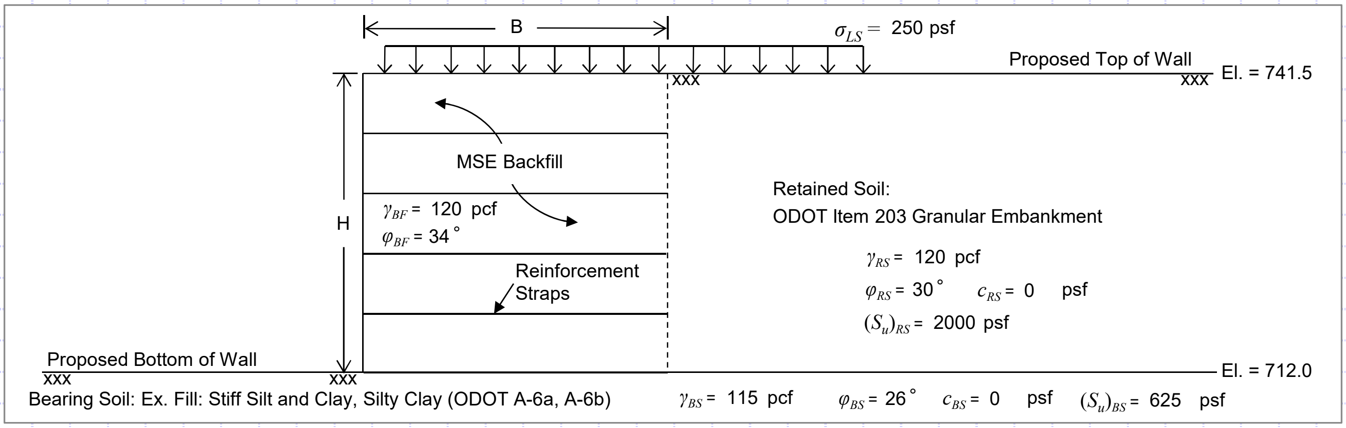
LATERAL DESIGN PARAMETERS

Boring No.	Elevation (feet msl)	Soil Class.	Soil Type	Strata	N_{60}	$N1_{60}$	γ (pcf)	γ' (pcf)	Strength Parameter	k (soil) k_{rm} (rock)	ε_{50} (soil) E_r (rock)	RQD (rock)
B-020-4-13	714.0 to 706.0	A-6a	C	1	5	5	115 psf	115 psf	Su = 625 psf	85 pci	0.0125	-
	706.0 to 701.0	A-7-6	C	3	14	14	120 psf	120 psf	Su = 1,750 psf	585 pci	0.0067	-
	701.0 to 696.0	A-2-4	G	4	7	7	120 psf	120 psf	φ = 32°	75 pci	-	-
	696.0 to 686.0	A-1-b	G	4	34	32	130 psf	67.6 psf	φ = 39°	140 pci	-	-
	686.0 to 677.0	A-1-a	G	4	18	15	125 psf	62.6 psf	φ = 37°	110 pci	-	-
	677.0 to 657.5	A-1-a	G	4	88	67	135 psf	72.6 psf	φ = 43°	215 pci	-	-
	657.5 to 649.6	A-6b	C	2	80	80	130 psf	67.6 psf	Su = 8,000 psf	2,665 pci	0.0033	-
	649.6 to 635.6	Shale	R	9	-	-	150 psf	87.6 psf	Qu = 200 psi	0.0005	20,000 psi	20
	635.6 to 630.3	Claystone	R	9	-	-	150 psf	87.6 psf	Qu = 200 psi	0.0005	20,000 psi	86
	630.3 to 619.3	Limestone	R	9	-	-	165 psf	102.6 psf	Qu = 10,000 psi	0.00005	1,000,000 psi	87
B-020-6-13	714.1 to 708.6	A-6a	C	3	10	10	115 psf	115 psf	Su = 1,250 psf	365 pci	0.0080	-
	708.6 to 703.6	A-1-a	G	4	13	16	125 psf	125 psf	φ = 37°	190 pci	-	-
	703.6 to 696.1	A-7-6	C	3	11	11	115 psf	115 psf	Su = 1,375 psf	435 pci	0.0075	-
	696.1 to 693.6	A-6a	C	3	28	28	125 psf	125 psf	Su = 3,500 psf	1,165 pci	0.0048	-
	693.6 to 688.6	A-1-a	G	4	36	32	125 psf	125 psf	φ = 40°	280 pci	-	-
	688.6 to 686.1	A-1-b	G	4	52	44	135 psf	72.6 psf	φ = 41°	175 pci	-	-
	686.1 to 654.1	A-1-a	G	4	107	79	135 psf	72.6 psf	φ = 43°	215 pci	-	-
	654.1 to 628.6	Shale	R	9	-	-	150 psf	87.6 psf	Qu = 360 psi	0.0005	32,000 psi	49
	628.6 to 619.1	Limestone	R	9	-	-	165 psf	102.6 psf	Qu = 10,000 psi	0.00005	1,000,000 psi	94
B-020-8-13	721.0 to 715.5	A-2-4	G	4	8	13	120 psf	120 psf	φ = 35°	135 pci	-	-
	715.5 to 713.0	A-1-a	G	4	23	30	125 psf	125 psf	φ = 40°	280 pci	-	-
	713.0 to 700.5	A-1-a	G	4	89	92	135 psf	135 psf	φ = 43°	395 pci	-	-
	700.5 to 689.0	A-3a	G	4	48	42	130 psf	67.6 psf	φ = 39°	140 pci	-	-
	689.0 to 669.5	A-1-b	G	4	112	86	135 psf	72.6 psf	φ = 42°	195 pci	-	-
	669.5 to 656.0	A-7-6	C	2	116	116	130 psf	67.6 psf	Su = 8,000 psf	2,665 pci	0.0033	-
	656.0 to 629.0	Shale	R	9	-	-	150 psf	87.6 psf	Qu = 360 psi	0.0005	32,000 psi	63
	629.0 to 619.0	Limestone	R	9	-	-	165 psf	102.6 psf	Qu = 10,000 psi	0.00005	1,000,000 psi	100

APPENDIX VIII

MSE WALL CALCULATIONS

FRA-70-1373B - Retaining Wall E4 - MSE Wall - Rear Abutment - B-020-4-13 - 29.5 ft. Wall Height



MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	29.5	ft
MSE Wall Width (Reinforcement Length), (B) =	20.7	ft
MSE Wall Length, (L) =	37	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion [1], (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	115	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(S_u)_{BS}$] =	625	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

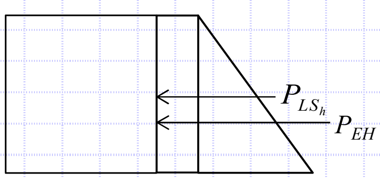
LRFD Load Factors

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

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

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

Sliding Force:



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

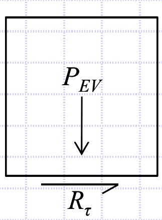
$$P_{EH} = \tfrac{1}{2}\gamma_{RS} H^2 K_a \gamma_{EH} = \text{½(120 pcf)(29.5 ft)²(0.297)(1.5)} = 23.26 \text{ kip/ft}$$

$$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS} = \text{(250 psf)(29.5 ft)(0.297)(1.75)} = 3.83 \text{ kip/ft}$$

$$P_H = \text{23.26 kip/ft + 3.83 kip/ft} = 27.09 \text{ kip/ft}$$

Check Sliding Resistance - Drained Condition

Nominal Sliding Resistance:



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

$$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV} = \text{(120 pcf)(29.5 ft)(20.7 ft)(1.00)} = 73.28 \text{ kip/ft}$$

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

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

$$R_\tau = \text{(73.28 kip/ft)(0.49)} = 35.91 \text{ kip/ft}$$

Verify Sliding Force Less Than Factored Sliding Resistance - Drained Condition

$$P_H \le R_\tau \cdot \phi_\tau \longrightarrow \text{27.09 kip/ft} \le \text{(35.91 kip/ft)(1.0) = 35.91 kip/ft} \longrightarrow \text{27.09 kip/ft} \le \text{35.91 kip/ft}$$ **OK**

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

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MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	29.5	ft
MSE Wall Width (Reinforcement Length), (B) =	20.7	ft
MSE Wall Length, (L) =	37	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, $[(S_u)_{RS}]$ =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	115	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	625	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

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

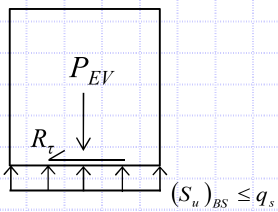
(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

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

Check Sliding Resistance - Undrained Condition

Nominal Sliding Resisting:

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



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

$$q_s = \sigma_v / 2 = (3.54 \text{ ksf}) / 2 = 1.77 \text{ ksf}$$

$$\sigma_v = P_{EV} / B = (73.28 \text{ kip/ft}) / (20.7 \text{ ft}) = 3.54 \text{ ksf}$$

$$R_\tau = (0.63 \text{ ksf} \leq 1.77 \text{ ksf})(20.7 \text{ ft}) = 12.94 \text{ kip/ft}$$

Verify Sliding Force Less Than Factored Sliding Resistance - Undrained Condition

$P_H \leq R_\tau \cdot \phi_\tau$ $\longrightarrow$ 27.09 kip/ft ≤ (12.94 kip/ft)(1.0) = 12.94 kip/ft $\longrightarrow$ 27.09 kip/ft ≤ 12.94 kip/ft **ERROR!!**

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

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MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	29.5	ft
MSE Wall Width (Reinforcement Length), (B) =	20.7	ft
MSE Wall Length, (L) =	37	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, $[(S_u)_{RS}]$ =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

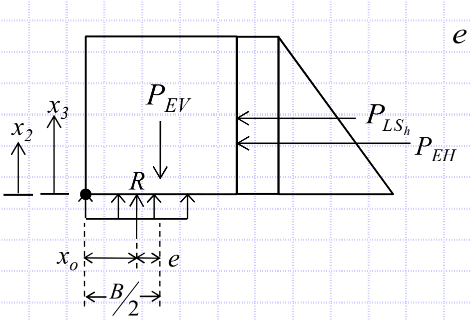
Bearing Soil Unit Weight, (γ_{BS}) =	115	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	625	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

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

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

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



$$e = B/2 - x_o$$

$$x_o = \frac{M_{EV} - M_H}{P_{EV}}$$ = (758.45 kip·ft/ft - 285.14 kip·ft/ft) / (73.28 kip/ft) = 6.46 ft

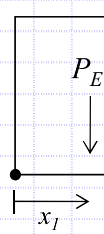
M_{EV}	=	758.45	kip·ft/ft
M_H	=	285.14	kip·ft/ft
P_{EV}	=	73.28	kip/ft

Defined below

$e =$ (20.7 ft)/2 - 6.46 ft = 3.89 ft

Resisting Moment, M_{EV}:

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



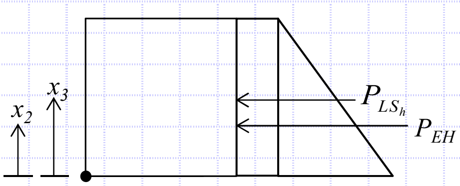
$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV}$ = (120 pcf)(29.5 ft)(20.7 ft)(1.00) = 73.28 kip/ft

$x_1 = B/2$ = (20.7 ft) / 2 = 10.35 ft

$M_{EV} =$ (73.28 kip/ft)(10.35 ft) = 758.45 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)(29.5 ft)²(0.297)(1.5) = 23.26 kip/ft

$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS}$ = (250 psf)(29.5 ft)(0.297)(1.75) = 3.83 kip/ft

$x_2 = H/3$ = (29.5 ft) / 3 = 9.83 ft

$x_3 = H/2$ = (29.5 ft) / 2 = 14.75 ft

$M_H =$ (23.26 kip/ft)(9.83 ft) + (3.83 kip/ft)(14.75 ft) = 285.14 kip·ft/ft

Check Eccentricity

$e < e_{max} \longrightarrow$ 3.89 ft < 6.90 ft **OK**

Limiting Eccentricity: $e_{max} = B/3 \longrightarrow e_{max} =$ (20.7 ft) / 3 = 6.90 ft

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MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	29.5	ft
MSE Wall Width (Reinforcement Length), (B) =	20.7	ft
MSE Wall Length, (L) =	37	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

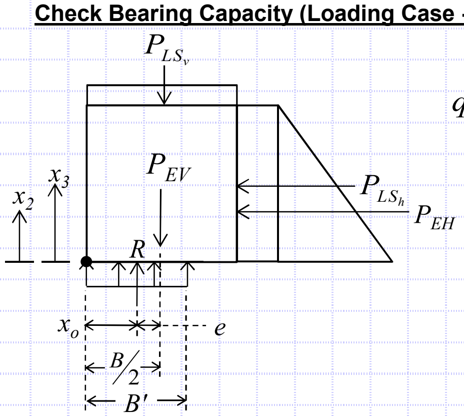
Bearing Soil Unit Weight, (γ_{BS}) =	115	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	625	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

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

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

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



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

$$B' = B - 2e \quad = \quad 20.7 \text{ ft} - 2(2.64 \text{ ft}) \quad = \quad 15.42 \quad \text{ft}$$

$$e = B/2 - x_o \quad = \quad (20.7 \text{ ft}) / 2 - 7.71 \text{ ft} \quad = \quad 2.64 \quad \text{ft}$$

$$x_o = \frac{M_V - M_H}{P_V} \quad = \quad (1117.61 \text{ kip·ft/ft} - 285.2 \text{ kip·ft/ft}) / 107.98 \text{ kip/ft} \quad = \quad 7.71 \quad \text{ft}$$

$$q_{eq} = \quad (107.98 \text{ kip/ft}) / (15.42 \text{ ft}) \quad = \quad 7.00 \quad \text{ksf}$$

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

$$M_V = [(120 \text{ pcf})(29.5 \text{ ft})(20.7 \text{ ft})(1.35)](10.35 \text{ ft}) + [(250 \text{ psf})(20.7 \text{ ft})(1.75)](10.35 \text{ ft}) \quad = \quad 1117.61 \quad \text{kip·ft/ft}$$

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

$$M_H = [\tfrac{1}{2}(120 \text{ pcf})(29.5 \text{ ft})^2(0.297)(1.5)](9.83 \text{ ft}) + [(250 \text{ psf})(29.5 \text{ ft})(0.297)(1.75)](14.75 \text{ ft}) \quad = \quad 285.20 \quad \text{kip·ft/ft}$$

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

$$P_V = (120 \text{ pcf})(29.5 \text{ ft})(20.7 \text{ ft})(1.35) + (250 \text{ psf})(20.7 \text{ ft})(1.75) \quad = \quad 107.98 \quad \text{kip/ft}$$

Check Bearing Resistance - Drained Condition

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

$N_{cm} = N_c s_c i_c$ = 27.19

N_c = 22.25
s_c = 1+(15.42 ft/37 ft)(11.85/22.25)
= 1.222
i_c = 1.000 (Assumed)

$N_{qm} = N_q s_q d_q i_q$ = 15.37

N_q = 11.85
s_q = 1.203
d_q = 1+2tan(26°)[1-sin(26°)]²tan⁻¹(4.0 ft/15.42 ft)
1.078
i_q = 1.000 (Assumed)
C_{wq} = 0.0 ft > 4.0 ft = 0.500

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 10.45

N_γ = 12.54
s_γ = 0.833
i_γ = 1.000 (Assumed)
$C_{w\gamma}$ = 0.0 ft < 1.5(15.42 ft) + 4.0 ft = 0.500

$$q_n = \quad (0 \text{ psf})(27.190) + (115 \text{ pcf})(4.0 \text{ ft})(15.367)(0.500) + \tfrac{1}{2}(115 \text{ pcf})(15.4 \text{ ft})(10.446)(0.500) \quad = \quad 8.17 \quad \text{ksf}$$

Verify Equivalent Pressure Less Than Factored Bearing Resistance

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

$q_{eq} \le q_n \cdot \phi_b$ ⟶ 7.00 ksf ≤ (8.17 ksf)(0.65) = 5.31 ksf ⟶ 7.00 ksf ≤ 5.31 ksf **ERROR!!**

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MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	29.5	ft
MSE Wall Width (Reinforcement Length), (B) =	20.7	ft
MSE Wall Length, (L) =	37	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, $[(S_u)_{RS}]$ =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	115	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	625	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

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

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

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

Check Bearing Resistance - Undrained Condition

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

$N_{cm} = N_c s_c i_c$ = 5.570

N_c = 5.140

s_c = 1+(15.42 ft/[(5)(37 ft)]) = 1.083

i_c = 1.000 (Assumed)

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

N_q = 1.000

s_q = 1.000

d_q = 1+2tan(0°)[1-sin(0°)]²tan⁻¹(4.0 ft/15.42 ft)

1.000

i_q = 1.000 (Assumed)

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

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

N_γ = 0.000

s_γ = 1.000

i_γ = 1.000 (Assumed)

$C_{w\gamma}$ = 0.0 ft < 1.5(15.42 ft) + 4.0 ft = 0.500

$q_n =$ (625 psf)(5.570) + (115 pcf)(4.0 ft)(1.000)(0.500) + ½(115 pcf)(15.4 ft)(0.000)(0.500) = 3.71 ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

$q_{eq} \le q_n \cdot \phi_b$ ⟶ 7.00 ksf ≤ (3.71 ksf)(0.65) = 2.41 ksf ⟶ 7.00 ksf ≤ 2.41 ksf **ERROR!!**

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

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MSE Wall Width (Reinforcement Length), (B) =	20.7	ft
MSE Wall Length, (L) =	37	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

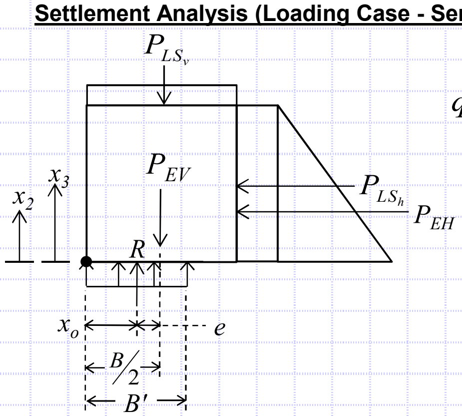
Bearing Soil Unit Weight, (γ_{BS}) =	115	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	625	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

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

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

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



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

$B' = B - 2e$ = 20.7 ft - 2(2.35 ft) = 16.00 ft

$e = B/2 - x_o$ = (20.7 ft) / 2 - 8 ft = 2.35 ft

$x_o = \dfrac{M_V - M_H}{P_V}$ = (811.99 kip·ft/ft - 184.75 kip·ft/ft) / 78.45 kip/ft = 8 ft

$q_{eq} =$ (78.45 kip/ft) / (16 ft) = 4.90 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)(29.5 ft)(20.7 ft)(1.00)](10.4 ft) + [(250 psf)(20.7 ft)(1.00)](10.4 ft) = 811.99 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)(29.5 ft)²(0.297)(1.00)](9.83 ft) + [(250 psf)(29.5 ft)(0.297)(1.00)](14.75 ft) = 184.75 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)(29.5 ft)(20.7 ft)(1.00) + (250 psf)(20.7 ft)(1.00) = 78.45 kip/ft

Settlement (To be calculated at Stage 2 Detailed Design):

Total Settlement at Center of Reinforced Soil Mass:	S_t =	10.909	in
Total Settlement at Wall Facing:	S_t =	5.661	in

Time Rate of Consolidation and Downdrag Depths and Loads:

Hold Period	Degree of Consolidation	Settlement Remaining at Completion of Hold Period	Depth of Downdrag
30 days	90 %	0.539 in	

W-13-072 - FRA-70-13.10 - FRA-70-1373B

MSE Wall Settlement - Rear Abutment - Retaining Wall E4

Calculated By: BRT Date: 6/20/2019

Checked By: JPS Date: 6/20/2019

Boring B-020-4-13

H=	29.5	ft	Total wall height
B'=	16.0	ft	Effective footing width due to eccentricity
D_w =	0.0	ft	Depth below bottom of footing
q_e =	4,900	psf	Equivalent bearing pressure at bottom of wall

																				Total Settlement at Center of Reinforced Soil Mass					Total Settlement at Facing of Wall				
Layer	Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' [1] (psf)	LL	C_c [2]	C_r [3]	e_o [4]	N_{60}	$(N1)_{60}$ [5]	C' [6]	Z_f/B	I [7]	$\Delta\sigma_v$ [8] (psf)	σ_{vf}' Midpoint (psf)	S_c [9,10] (ft)	S_c (in)	I [7]	$\Delta\sigma_v$ [8] (psf)	σ_{vf}' Midpoint (psf)	S_c [9,10] (ft)	S_c (in)
1	A-6a	C	0.0	2.0	2.0	1.0	115	230	115	53	2,053	32	0.198	0.030	0.522				0.06	0.999	4,896	4,949	0.162	1.938	0.500	2,450	2,502	0.084	1.014
	A-6a	C	2.0	4.0	2.0	3.0	115	460	345	158	2,158	32	0.198	0.030	0.522				0.19	0.981	4,806	4,964	0.138	1.662	0.499	2,443	2,601	0.065	0.785
	A-6a	C	4.0	6.0	2.0	5.0	115	690	575	263	2,263	32	0.198	0.030	0.522				0.31	0.931	4,559	4,822	0.122	1.464	0.494	2,422	2,685	0.056	0.669
2	A-7-6	C	6.0	8.5	2.5	7.3	120	990	840	388	2,388	48	0.342	0.034	0.647				0.45	0.848	4,155	4,543	0.186	2.232	0.484	2,373	2,760	0.074	0.884
	A-7-6	C	8.5	11.0	2.5	9.8	120	1,290	1,140	532	2,532	48	0.342	0.034	0.647				0.61	0.750	3,673	4,205	0.150	1.794	0.467	2,289	2,821	0.060	0.715
3	A-2-6	G	11.0	13.5	2.5	12.3	120	1,590	1,440	676	2,676					7	10	56	0.77	0.660	3,233	3,908	0.034	0.408	0.446	2,184	2,859	0.028	0.335
	A-2-6	G	13.5	16.0	2.5	14.8	120	1,890	1,740	820	2,820					7	9	55	0.92	0.583	2,858	3,677	0.029	0.353	0.422	2,066	2,885	0.025	0.296
4	A-1-b	G	16.0	21.0	5.0	18.5	130	2,540	2,215	1,061	5,061					34	41	135	1.16	0.492	2,410	3,470	0.019	0.229	0.384	1,884	2,944	0.016	0.197
	A-1-b	G	21.0	26.0	5.0	23.5	130	3,190	2,865	1,399	5,399					34	38	124	1.47	0.403	1,975	3,374	0.015	0.185	0.338	1,658	3,057	0.014	0.164
5	A-1-a	G	26.0	35.0	9.0	30.5	125	4,315	3,753	1,849	5,849					18	19	71	1.91	0.320	1,566	3,415	0.034	0.404	0.285	1,395	3,244	0.031	0.370
6	A-1-a	G	35.0	45.0	10.0	40.0	135	5,665	4,990	2,494	6,494					88	82	338	2.50	0.248	1,216	3,710	0.005	0.061	0.231	1,131	3,625	0.005	0.058
	A-1-a	G	45.0	55.0	10.0	50.0	135	7,015	6,340	3,220	7,220					88	74	292	3.13	0.200	982	4,202	0.004	0.048	0.191	936	4,156	0.004	0.046
7	A-6b	C	55.0	62.5	7.5	58.8	130	7,990	7,503	3,837	7,837	40	0.270	0.027	0.585				3.67	0.171	839	4,676	0.011	0.132	0.165	810	4,647	0.011	0.128
																					Total Settlement:			10.909 in		Total Settlement:			5.661 in

1. $\sigma_p' = \sigma_{vo}' + \sigma_m$; Estimate σ_m of 2,000 psf in existing fill material and 4,000 psf (moderately overconsolidated) for natural soil deposits; Ref. Table 11.2, Coduto 2003
2. $C_c = 0.009(LL-10)$; Ref. Table 6-9, FHWA GEC 5
3. $C_r = 0.15(Cc)$ for the existing fill and 0.10(Cc) for the natural soil deposits; Ref. Section 8.11, Holtz and Kovacs 1981
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_N N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \le 2.0$ 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-13-072 - FRA-70-13.10 - FRA-70-1373B

MSE Wall Settlement - Rear Abutment - Retaining Wall E4

Calculated By: BRT   Date: 06/20/2019

Checked By: JPS   Date: 06/20/2019

Boring B-020-4-13

H=	29.5	ft	Total wall height
B'=	16.0	ft	Effective footing width due to eccentricity
D_w =	0.0	ft	Depth below bottom of footing
q_e =	4,900	psf	Equivalent bearing pressure at bottom of wall

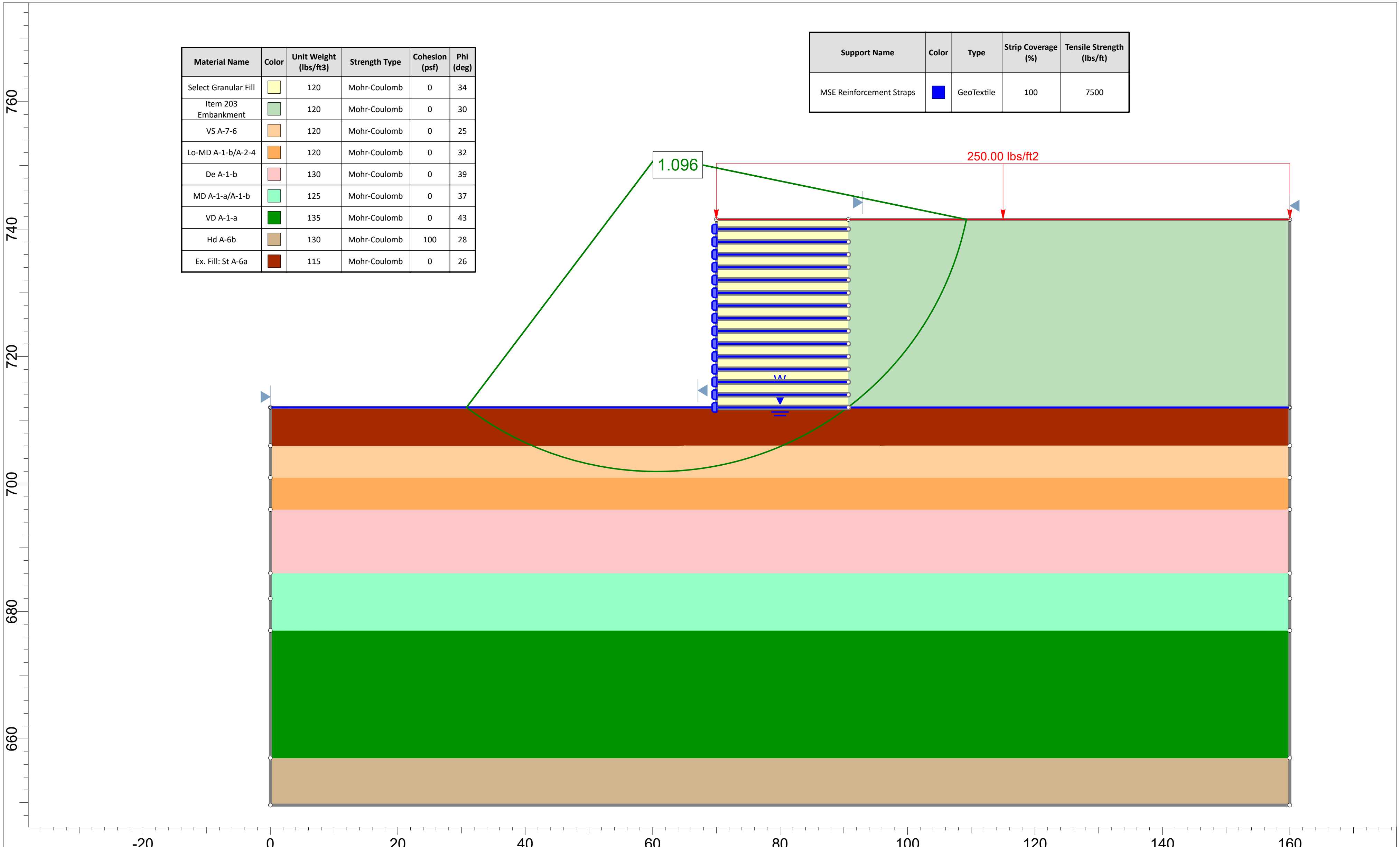
	A-6a	A-7-6	A-6b			
c_v =	600	150	300		ft^2/yr	Coefficient of consolidation
t =	30	30	30		days	Time following completion of construction
H_{dr} =	5.5	5.5	7.5		ft	Length of longest drainage path considered
T_v =	1.630	0.408	0.438			Time factor
U =	99	70	73		%	Degree of consolidation

$(S_c)_t$ = 5.122 in Settlement complete at 90% of primary consolidation

Layer	Soil Type	Soil Type	Layer Depth (ft)		Layer Thickness (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	Total Settlement at Facing of Wall: S_c (9,10) (ft)	S_c (in)	Layer Settlement (in)	Settlement Complete at 90% of Primary Consolidation: $(S_c)_t$ (11) (in)	Layer Settlement (in)
1	A-6a	C	0.0	2.0	2.0	1.0	115	230	115	53	2,053	32	0.198	0.030	0.522				0.06	0.500	2,450	2,502	0.084	1.014	2.468	1.004	2.444
	A-6a	C	2.0	4.0	2.0	3.0	115	460	345	158	2,158	32	0.198	0.030	0.522				0.19	0.499	2,443	2,601	0.065	0.785		0.777	
	A-6a	C	4.0	6.0	2.0	5.0	115	690	575	263	2,263	32	0.198	0.030	0.522				0.31	0.494	2,422	2,685	0.056	0.669		0.663	
2	A-7-6	C	6.0	8.5	2.5	7.3	120	990	840	388	2,388	48	0.342	0.034	0.647				0.45	0.484	2,373	2,760	0.074	0.884	1.599	0.619	1.119
	A-7-6	C	8.5	11.0	2.5	9.8	120	1,290	1,140	532	2,532	48	0.342	0.034	0.647				0.61	0.467	2,289	2,821	0.060	0.715		0.500	
3	A-2-6	G	11.0	13.5	2.5	12.3	120	1,590	1,440	676	4,676					7	10	56	0.77	0.446	2,184	2,859	0.028	0.335	0.631	0.335	0.631
	A-2-6	G	13.5	16.0	2.5	14.8	120	1,890	1,740	820	4,820					7	9	55	0.92	0.422	2,066	2,885	0.025	0.296		0.296	
4	A-1-b	G	16.0	21.0	5.0	18.5	130	2,540	2,215	1,061	5,061					34	41	135	1.16	0.384	1,884	2,944	0.016	0.197	0.361	0.197	0.361
	A-1-b	G	21.0	26.0	5.0	23.5	130	3,190	2,865	1,399	5,399					34	38	124	1.47	0.338	1,658	3,057	0.014	0.164		0.164	
5	A-1-a	G	26.0	35.0	9.0	30.5	125	4,315	3,753	1,849	5,849					18	19	71	1.91	0.285	1,395	3,244	0.031	0.370	0.370	0.370	0.370
6	A-1-a	G	35.0	45.0	10.0	40.0	135	5,665	4,990	2,494	6,494					88	82	338	2.50	0.231	1,131	3,625	0.005	0.058	0.103	0.058	0.103
	A-1-a	G	45.0	55.0	10.0	50.0	135	7,015	6,340	3,220	7,220					88	74	292	3.13	0.191	936	4,156	0.004	0.046		0.046	
7	A-6b	C	55.0	62.5	7.5	58.8	130	7,990	7,503	3,837	7,837	40	0.270	0.027	0.585				3.67	0.165	810	4,647	0.011	0.128	0.128	0.093	0.093

Settlement Remaining After Hold Period: 0.539 in

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



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Select Granular Fill		120	Mohr-Coulomb	0	34
Item 203 Embankment		120	Mohr-Coulomb	0	30
VS A-7-6		120	Mohr-Coulomb	0	25
Lo-MD A-1-b/A-2-4		120	Mohr-Coulomb	0	32
De A-1-b		130	Mohr-Coulomb	0	39
MD A-1-a/A-1-b		125	Mohr-Coulomb	0	37
VD A-1-a		135	Mohr-Coulomb	0	43
Hd A-6b		130	Mohr-Coulomb	100	28
Ex. Fill: St A-6a		115	Mohr-Coulomb	0	26

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

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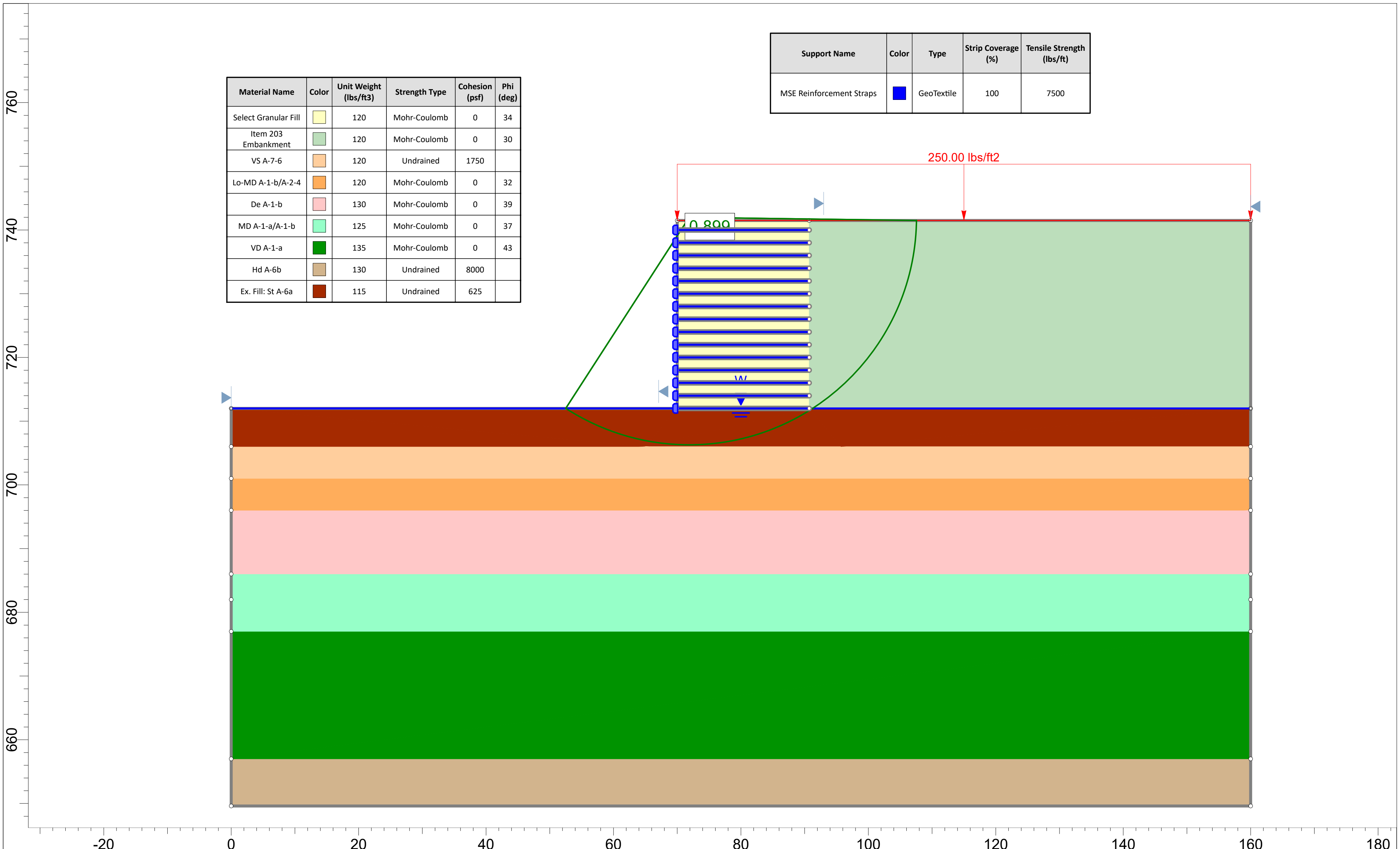
SLIDEINTERPRET 8.020

Project	FRA-70-13.10 - FRA-70-1373B - Rear Abutment - Retaining Wall E4 (Sta. 400+98 to 402+11) - Global Stability
Analysis Description	29.5 ft Wall Height - Drained - Circular - Spencer's
Drawn By	BRT
Scale	1:250
Company	Resource International, Inc.
Date	6/21/2019
File Name	FRA-70-1373B - Rear Abutment - MSE Wall Global Stability.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Select Granular Fill		120	Mohr-Coulomb	0	34
Item 203 Embankment		120	Mohr-Coulomb	0	30
VS A-7-6		120	Undrained	1750	
Lo-MD A-1-b/A-2-4		120	Mohr-Coulomb	0	32
De A-1-b		130	Mohr-Coulomb	0	39
MD A-1-a/A-1-b		125	Mohr-Coulomb	0	37
VD A-1-a		135	Mohr-Coulomb	0	43
Hd A-6b		130	Undrained	8000	
Ex. Fill: St A-6a		115	Undrained	625	

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

Resource International, Inc.
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SLIDEINTERPRET 8.020

Project	FRA-70-13.10 - FRA-70-1373B - Rear Abutment - Retaining Wall E4 (Sta. 400+98 to 402+11) - Global Stability
Analysis Description	29.5 ft Wall Height - Undrained - Circular - Spencer's
Drawn By	BRT
Scale	1:250
Company	Resource International, Inc.
Date	6/21/2019
File Name	FRA-70-1373B - Rear Abutment - MSE Wall Global Stability.slim