Resource International, Inc.

FRA-70-12.68 PROJECT 4R
FRA-71-1518A
RAMP A5 OVER THE SCIOTO RIVER
RETAINING WALL 4W11
PID NO. 105523
FRANKLIN COUNTY, OHIO

STRUCTURE FOUNDATION EXPLORATION REPORT

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

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RESOURCE INTERNATIONAL, INC.

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Planning

Engineering

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Technology

April 1, 2015 (Revised July 13, 2018)

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Re: Structure Foundation Exploration Report
FRA-70-12.68 Project 4R
FRA-71-1518A – Ramp A5 over the Scioto River
Retaining Wall 4W11
PID No. 105523
Rii Project No. W-13-045

Mr. Luzier:

Resource International, Inc. (Rii) is pleased to submit this structure foundation exploration report for the above referenced project. 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-71-1518A bridge structure carrying Ramp A5 over the Scioto River as well as Retaining Wall 4W11 as part of the FRA-70-12.68 Project 4R in Columbus, Ohio.

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

Sincerely,

RESOURCE INTERNATIONAL, INC.

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

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

Enclosure: Structure Foundation Exploration Report

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EXECUTIVE SUMMARY

Resource International, Inc. (Rii) has completed a structure foundation exploration for the design and construction of the proposed FRA-71-1518A bridge structure carrying Ramp A5 to connect with the FRA-70-1321A bridge structure over the Scioto River. Based on information provided by Burgess and Niple, it is understood that the proposed bridge will consist of a two-span prestressed concrete I-beam structure with a composite reinforced concrete deck and a pile supported stub abutment behind an MSE wall at the rear abutment and piers supported on drilled shafts. The bridge will have a total length of approximately 280 feet and width of approximately 31.5 feet. The roadway profile will be elevated approximately 35 feet above the existing ground surface grade. In addition, Retaining Wall 4W11 will also be located at the rear abutment of the proposed structure to provide the required grade separation to support the configuration. It is understood that a mechanically stabilized earth (MSE) wall type is being considered as the preferred wall type for the entire alignment of Retaining Wall 4W11. The wall height will range from 4.8 feet at the beginning of the wall alignment to a maximum height of 35.6 feet at the proposed abutment and back down to a height of 3.6 feet at the end of the wall alignment. The total wall length is approximately 328 lineal feet.

Exploration and Findings

Between June 6, 2013, and March 8, 2015, a total of four (4) borings, designated as B-015-7-13, B-108-2-13, B-108-3-13 and B-108-9-15, were drilled to completion depths ranging from 50.0 to 80.5 feet below the existing ground surface along the proposed bridge alignment at the locations shown on the boring plan provided in Appendix I of the full report.

Boring B-108-2-13 was performed in the grass infield along the west side of the existing I-71 northbound ramp and borings B-015-7-13 and B-108-3-13 were performed at the top of the grass covered bank of the Scioto River and encountered 5.0 to 6.0 inches of topsoil at the ground surface. Boring B-108-9-15 was performed within the existing ramp from I-71 northbound to I-70 eastbound and encountered 3.0 inches of asphalt overlying 9.0 inches of concrete followed by 4.0 inches of aggregate base.

Beneath the surface materials in borings B-108-2-13, B-108-3-13 and B-108-9-15, material identified as existing fill was encountered extending to depths ranging from 5.0 to 8.0 feet below existing grade. The fill material consisted of brown, gray, dark brown and brownish gray gravel with sand and silt, and silty clay (ODOT A-2-4, A-6b) and contained trace amounts of cinders, brick and stone fragments in several of the samples recovered.

Underlying the topsoil in boring B-015-7-13 and existing fill in the remaining borings, natural cohesive soils were encountered extending to an approximate elevation of 700 feet msl overlying granular soils. The cohesive soils were generally described as gray, brown, brownish gray, dark brown and dark gray sandy silt, silt and clay, silty clay and clay (ODOT A-4a, A-6a, A-6b, A-7-6). The granular soils were generally described as brown, gray, brownish gray and dark brown gravel, gravel and sand gravel with sand and silt, gravel with sand, silt and clay, coarse and fine sand and sandy silt (ODOT A-1-a, A-1-b, A-2-4, A-2-6, A-3a, A-4a).

Bedrock was encountered in borings B-015-7-13 and B-108-9-15 at a depth of 70.5 and 67.0 feet below the ground surface, respectively, which corresponds to and elevation of 651.3 and 655.4 feet msl. The cored bedrock in boring B-015-7-13 consisted of dolomite, and the cored bedrock in boring B-108-9-15 consisted of limestone.

Analyses and Recommendations

Foundation recommendations for the combined Pier A are provided in the FRA-70-1321A Structure Foundation Exploration report. Therefore, recommendations for this substructure unit are not provided in this report.

Driven Pile Recommendations

It is understood that driven piles are to be utilized at the rear abutment substructure. Given the depth of bedrock encountered in the borings performed, it is recommended that CIP pipe piles (ODOT Item 507.07) driven to the frictional bearing values listed below or 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 2007 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 corresponding ultimate bearing value (R_{ndr}) of CIP pipe piles and factored structural axial resistance ($R_{R\,max}$) of steel H-piles and associated resistance factors (φ):

FRA-71-1518A Driven Pile Recommendations

Substructure Reference	Ground Elevation [1]	Pile Size [2]	Pile Elevation (feet msl)		Pile Length [4] (feet)	R_{ndr} [5] / $R_{R\ max}$ [6] (kips/pile)	Sleeve Length [7] (feet)	φ [8,9]
			Top [3]	Tip				
Rear Abutment (B-108-3-13 and B-108-9-15)	733.4	14" CIP	746.3	690.5	60	390	23.8	0.7
		HP 10x42	746.3	655.5	95	310	23.8	N/A

1. *Ground elevation listed is the ground elevation at the respective boring locations.*
2. *A steel pile point is recommended to protect the tips of the CIP pipe piles during pile installation.*
3. *The top of pile elevation corresponds to the pile cutoff elevation, which is 1.0-foot above the proposed bottom of footing elevation.*
4. *Per Section 202.3.2 of the 2007 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 ultimate bearing value (R_{ndr}) is based on the maximum factored load per pile and was calculated in accordance with Section 202.2.3.2.b of the 2007 ODOT BDM.*
6. *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 2007 ODOT BDM.*
7. *Sleeve length represents the required length of pile that should be sleeved within the MSE wall backfill, including the foundation preparation.*
8. *The resistance factor listed for CIP pipe piles assumes dynamic testing of the pile elements per Section 303.4.2.7 of the 2007 ODOT BDM.*
9. *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.*

The anticipated total settlement at the facing of the MSE wall at the rear abutment is 2.83 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 fifteen (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.

Drilled Shaft Recommendations

It is understood that drilled shaft foundations are planned for support of the proposed Pier 1 due to proximity of the substructure unit to the existing 42-inch storm force main. Given the proposed loading per shaft at Pier 1, friction bearing drilled shafts within the overburden soils are not economically feasible foundation options due to the size and number of shafts that would be required to support the proposed loading. Therefore, it is recommended that the drilled shafts be extended through the surficial soils to bear on or within the underlying limestone bedrock at Pier 1.

Per Section 10.8.3.5.4c of the 2018 AASHTO LRDF Bridge Design Specifications (BDS), a minimum rock socket length of 1.5 times the diameter of the drilled shaft within the rock socket ($1.5B_{RS}$) is required to utilize the full end bearing resistance within the bedrock unit that the shafts are end bearing in/on. Using equation 10.8.3.5.4c-1 and the limiting unconfined compressive strength from the range for the limestone bedrock, it is recommended that drilled shaft foundations socketed a minimum of $1.5B_{RS}$ into the bedrock to bear on or within the competent limestone bedrock be proportioned for a nominal end bearing resistance of 2,804 ksf at the strength limit state.

Where lateral load demands do not require a rock socket length of $1.5B_{RS}$, the socket length can be reduced or the shaft can bear on the bedrock surface with no rock socket. If the rock socket is reduced to a length less than $1.5B_{RS}$, a reduced nominal end bearing resistance should be utilized based on equations 10.8.3.5.4c-2 and 10.8.3.5.4c-3 of the AASHTO LRFD BDS. Using the limiting unconfined compressive strength from the given range for the limestone bedrock, it is recommended that drilled shaft foundations bearing on or within the competent limestone bedrock with a socket length less than $1.5B_{RS}$ into the bedrock be proportioned for a nominal end bearing resistance of 1,188 ksf at the strength limit state.

Based on the plan information provided by Burgess and Niple, the proposed shaft diameter at Pier 1 will be 5.0 feet within the overburden soils and 4.5 feet within the bedrock socket. The following table lists the estimated elevation of the top of bedrock as well as the proposed rock sock diameter and length from the design plans and, corresponding nominal end bearing resistance to be utilized for the design of the drilled shaft foundations. A resistance factor of $\varphi_{qp} = 0.5$ at the strength limit state should be utilized for design.

Drilled Shaft Recommendations

Substructure Unit (Boring)	Top of Bedrock Elevation (feet msl)	Rock Socket Diameter [1] (feet)	Proposed Socket Length [1] (feet)	Nominal End Bearing Resistance [2] (ksf)
Pier 1 (B-108-9-15 / B-015-7-13)	653.5	4.5	4.5	1,188

1. *Proposed rock socket diameter and length determined from proposed plan information provided by Burgess and Niple.*
2. *Nominal end bearing resistance provided is the value that should be utilized in the determination of the end bearing resistance per drilled shaft based on the proposed rock socket length and diameter.*

If lateral analysis of the drilled shafts foundations indicates that the rock socket length can be reduced based on the lateral load demands, then the rock socket length may be reduced from those shown in the current design plans. If the rock socket is reduced to a length less than $1.5B_{RS}$, then the reduced bearing resistance of 1,188 ksf should be utilized for design.

Given the factored end bearing resistances 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. The factored resistance per shaft provided in the design sheets should be the limiting value between the factored geotechnical resistance and the factored axial compressive resistance of the shaft.

MSE Wall Recommendations

Retaining Wall 4W11 will be located at the rear abutment of the proposed structure. Based upon the proposed plan information, the maximum wall height at the rear abutment is anticipated to be 35.7 feet from the top of the leveling pad to the proposed profile grade of the roadway. The wall will be turned back approximately 45 degrees and graded down to the north and south of the abutment, and will then turn back again at approximately 45 degrees at the north side of Ramp A5, just south of the I-71 northbound ramp. It is understood that 2:1 backslopes will be graded up to the proposed Ramp A5 roadway from the top of the wall where it extends away from the rear abutment.

Existing fill material comprised of medium dense gravel with sand and silt (ODOT A-2-4) was encountered at the proposed bearing elevation along the wall alignment, which extends to a depths ranging from 5.0 to 8.0 feet below the proposed bearing elevation, overlying medium stiff to stiff silty clay (ODOT A-6b). Based on the blow counts obtained within fill material and the absence of significant amounts of deleterious materials, the existing fill material is considered suitable to support the proposed retaining wall. Given the relatively shallow thickness of the gravel with sand and silt (existing fill) layer, a two layer bearing stratum should be evaluated for this wall. Therefore, the bearing resistance was calculated considering just the drained and undrained properties of the underlying medium stiff to stiff silty clay (ODOT A-6b). MSE wall foundations bearing on these soils or new embankment fill, placed and compacted in accordance with ODOT Item 203, may be proportioned for a factored bearing resistance as indicated in the following table.

Retaining Wall 4W11 MSE Wall Design Parameters

From Station [1]	To Station [1]	Wall Height Analyzed (feet)	Backslope Behind Wall in Analysis	Minimum Required Reinforcement Length [2] (feet)	Bearing Resistance at Strength Limit (ksf)		Strength Limit Equivalent Bearing Pressure [4] (ksf)
					Nominal	Factored [3]	
0+00	1+78	35.7	Level	28.6 (0.80H)	13.59	8.83	7.70
1+78	3+38	20.0	2:1	17.0 (0.85H)	10.11	6.57	5.77

1. *Station referenced to the baseline of Wall 4W11.*
2. *The required foundation width is expressed as a percentage of the wall height, H.*
3. *A geotechnical resistance factor of φ_b=0.65 was considered in calculating the factored bearing resistance at the strength limit state.*
4. *The strength limit equivalent bearing pressure is the uniformly distributed pressure asserted by the wall over an effective base width based on the eccentricity of the wall system at the strength limit state.*

Total settlements of up to 5.32 inches at the center of the reinforced soil mass and 3.16 inches at the facing of the wall are anticipated along the alignment of retaining wall 4W11. Based on the results of the analysis, 90 percent of the total settlement is anticipated to occur over a period of approximately fifteen (15) days.

Based on the results of the external and global stability analyses performed for Retaining Wall 4W11, the recommended controlling strap length is 0.80 times the maximum height of the MSE wall (measured from the top of the leveling pad to the proposed profile grade of the roadway) between Sta. 0+00 and 1+78 and 1.0 times the wall height from Sta. 1+78 to 3+38 (end of the wall). Global stability under drained and undrained conditions were the controlling factors in the determination of the recommended strap length of 80 percent of the wall height for the section between Sta. 0+00 and 1+78. Bearing stability under drained conditions and global stability under both drained and undrained conditions were the controlling factors in the recommended strap length of 85 percent of the wall height between Sta. 1+78 and 3+38.

As noted in Section 5.4.2 of the full report, bearing stability was not satisfied under undrained conditions for either wall section assuming a uniform bearing stratum with the weakest shear strength parameters within the zone of influence below the bottom of wall. However, if a layered soil profile is considered and the upper gravel with sand and silt layer is modeled a cohesive soil with an undrained shear strength of approximately 4,000 psf, then the nominal bearing resistance increases to the approximate resistance calculated under drained conditions, which satisfies bearing stability requirements.

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

1.0 INTRODUCTION

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

This report is a presentation of the structure foundation exploration performed for the design and construction of the proposed FRA-71-1518A bridge structure carrying Ramp A5 to connect with the FRA-70-1321A bridge structure over the Scioto River, as shown on the vicinity map and boring plan presented in Appendix I. The proposed Ramp A5 will be a single-lane ramp that will carry traffic from I-71 northbound to exit onto the proposed Ramp C5. Based on information provided by Burgess and Niple, it is understood that the proposed bridge will consist of a two-span prestressed concrete I-beam structure with a composite reinforced concrete deck and a pile supported stub abutment behind an MSE wall at the rear abutment and piers supported on drilled shafts. The bridge will have a total length of approximately 280 feet and width of approximately 31.5 feet. The roadway profile will be elevated approximately 35 feet above the existing ground surface grade.

Retaining Wall 4W11 will be located at the rear abutment of the proposed structure to provide the required grade separation to support the configuration. Based on plan information provided by Dynotec, there are several bends in the wall geometry, which are required to minimize the impact of the wall to the existing levee along the Scioto River as well as an existing 48-inch storm force main that is to remain in service. It is understood that a mechanically stabilized earth (MSE) wall is being considered as the preferred wall type for the entire alignment of Retaining Wall 4W11. The wall height will range from 4.8 feet at the beginning of the wall alignment to a maximum height of 35.6 feet at the proposed abutment and back down to a height of 3.6 feet at the end of the wall alignment. The total wall length is approximately 328 lineal feet.

2.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1 Site Geology

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

According to the bedrock geology and topography maps obtained from the Ohio Department of Natural Resources (ODNR), the underlying bedrock consists predominantly of the Middle to Lower Devonian-aged Columbus Limestone. This formation is further subdivided into two members in the central portion of the state, known as the Delhi and Bellepoint Members. The Delhi Member consists of light gray, finely to coarsely crystalline, irregularly bedded, fossiliferous limestone. The Bellepoint Member consists of variable brown, finely crystalline, massively bedded limy dolomite. Both of these members contain chert nodules. Just east of High Street, the underlying bedrock consists of the Upper Devonian Ohio Shale Formation overlying the Middle Devonian-aged Delaware Limestone Formation. The Ohio Shale formation consists of brownish black to greenish gray, thinly bedded, fissile, carbonaceous shale. The Delaware Limestone consists of bluish gray, thin to medium bedded dolomitic limestone with nodules and layers of chert. Regionally, the bedrock surface forms a broad valley aligned roughly north-to-south beneath the Scioto River. According to bedrock topography mapping, the elevation of the bedrock surface ranges from approximately 600 feet mean sea level (msl) in the valley to approximately 625 feet msl near the project limits. Bedrock was encountered in borings B-015-7-13 and B-108-9-15 at an elevation of 651.3 and 655.4 feet msl, respectively.

2.2 Existing Conditions

The proposed FRA-71-1518A structure will be situated over the existing I-71 northbound ramp to I-70 eastbound, approximately 350 feet southwest of the existing bridge over the Scioto River. The Scioto River flows along the east side of I-71 and SR-315 northbound. The existing I-71 northbound ramp is a single-lane, composite asphalt and concrete roadway that is aligned along the top of the existing Scioto River flood protection levee. The western leg of retaining wall 4W11 will be situated over the I-70 eastbound ramp to SR-315 northbound, which is also a single-lane, asphalt roadway. There is also an existing pump station situated within the infield of the loop ramp from I-70 eastbound to SR-315 northbound, which has 42-inch and 48-inch storm force mains that pump storm water to outlets along the Scioto River in the vicinity of the proposed structure. The existing terrain along the top of the level is flat and raised approximately 35 feet above the riverbed elevation of the Scioto River, which provides flood protection for the 500 year flood event. The existing infields between the roadways are generally grass covered, with some dense vegetation along the slope supporting the I-70 eastbound ramp to SR-315 northbound.

3.0 EXPLORATION

Between June 6 and 13, 2013, a total of three (3) borings, designated as B-015-7-13, B-108-2-13 and B-108-3-13, 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 50.0 to 80.5 feet below the existing ground surface at the respective boring location. In compliance with comments provided by the ODOT Office of Geotechnical Engineering (OGE), an additional boring was obtained at the approximate midpoint between the proposed rear abutment and central pier for evaluation of deep foundation alternatives for these substructures. On March 7 and 8, 2015, boring B-108-9-15 was advanced to a completion depth of 80.1 feet below existing grade at that location shown on the boring plan in Appendix I.

Table 1. Test Boring Summary

Boring Number	Station [1]	Offset [1]	Latitude	Longitude	Ground Elevation (feet msl)	Boring Depth (feet)
B-015-7-13	5051+29.66	9.8' Rt.	39.950618516	-83.014254653	721.8	80.5
B-108-2-13	5014+11.97	28.9' Rt.	39.949854823	-83.014658823	722.1	50.0
B-108-3-13	5015+67.23	22.0' Lt.	39.950228123	-83.014459410	722.9	50.0
B-108-9-15	5015+96.50	15.7' Rt.	39.950352963	-83.014507695	722.4	80.1

1. *Station and offset of boring B-015-7-13 referenced to the proposed baseline of Ramp C5, and the station and offset of the remaining borings are referenced to the proposed baseline of Ramp A5.*

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 GPD GROUP.

The borings were drilled using a truck or an all-terrain vehicle (ATV) mounted rotary drilling machine, utilizing a 3.25-inch inside diameter, hollow-stem auger to advance the holes. Standard penetration test (SPT) and split spoon sampling were performed in the borings at 2.5-foot increments of depth to 20.0 feet in borings B-108-2-13 and B-108-3-13 and 30.0 feet in borings B-015-7-13 and B-108-9-15 and 20.0 feet, and at 5.0-foot increments thereafter to the boring termination depth or top of bedrock. The SPT, per the American Society for Testing and Materials (ASTM) designation D1586, is conducted using a 140-pound hammer falling 30.0 inches to drive a 2.0-inch outside diameter split spoon sampler 18.0 inches. Rii utilized a calibrated automatic drop hammer to generate consistent energy transfer to the sampler during the SPT testing. Driving resistance is recorded on the boring logs in terms of blow per 6.0-inch interval of the driving distance. The second and third intervals are added to obtain the number of blows per foot (N). Standard penetration blow counts aid in determining soil properties applicable in foundation system design. Measured blow count (N) values are corrected to an equivalent (60%) energy ratio, N_{60}, by the following equation. Both values are represented on boring logs in Appendix III.

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

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

The hammer for the CME 750 and Mobile B-53 drill rigs used for these structure borings were calibrated on April 26, 2013, and have a drill rod energy ratio of 82.6 and 77.7 percent, respectively.

Borings B-015-7-13 and B-108-3-13 were performed within the flood protection zone along the Scioto River, and were sealed at with a cement-bentonite grout at the completion of drilling. Boring B-108-2-13 was performed outside the limits of the flood protection zone and was also sealed with a cement-bentonite grout at the completion of drilling. Boring B-108-9-15 was performed within the existing I-71 northbound ramp to I-70 eastbound, which is not considered part of the flood protection zone, and was backfilled with a mixture of bentonite chips and soil cuttings.

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 D2216	66
Plastic and Liquid Limits	AASHTO T89, T90	26
Gradation – Sieve/Hydrometer	AASHTO T88	26
Unconfined Compressive Strength of Cohesive Soil	ASTM D2166	1
Unconfined Compressive Strength of Intact Rock	ASTM D7012	3

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 in borings B-015-7-13 and B-108-9-15 was determined by auger refusal. Auger refusal is defined as no or insignificant observable advancement of the augers with the weight of the drill rig driving the augers. Where borings were extended into the bedrock (after encountering auger refusal), an NQ-sized double-tube diamond bit core barrel (utilizing wire line equipment) was used to core the bedrock. Coring produced 1.85 inch diameter cores, from which the type of rock and its geological characteristics were determined.

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

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

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-108-2-13 was performed in the grass infield along the west side of the existing I-71 northbound ramp and borings B-015-7-13 and B-108-3-13 were performed at the top of the grass covered bank of the Scioto River. These borings encountered 5.0 to 6.0 inches of topsoil at the ground surface. Boring B-108-9-15 was performed within the existing ramp from I-71 northbound to I-70 eastbound and encountered 3.0 inches of asphalt overlying 9.0 inches of concrete followed by 4.0 inches of aggregate base.

4.2 Subsurface Soils

Beneath the surface materials in borings B-108-2-13, B-108-3-13 and B-108-9-15, material identified as existing fill was encountered extending to depths ranging from 5.0 to 8.0 feet below existing grade. The fill material consisted of brown, gray, dark brown and brownish gray gravel with sand and silt, and silty clay (ODOT A-2-4, A-6b) and contained trace amounts of cinders, brick and stone fragments in several of the samples recovered.

Underlying the topsoil in boring B-015-7-13 and existing fill in the remaining borings, natural cohesive soils were encountered extending to an approximate elevation of 700 feet msl overlying granular soils. The cohesive soils were generally described as gray, brown, brownish gray, dark brown and dark gray sandy silt, silt and clay, silty clay and clay (ODOT A-4a, A-6a, A-6b, A-7-6). The granular soils were generally described as brown, gray, brownish gray and dark brown gravel, gravel and sand gravel with sand and silt, gravel with sand, silt and clay, coarse and fine sand and sandy silt (ODOT A-1-a, A-1-b, A-2-4, A-2-6, A-3a, A-4a).

The shear strength and consistency of the cohesive soils are primarily derived from the hand penetrometer values (HP). The cohesive soil encountered ranged from soft ($0.25 < HP \leq 0.5$ tsf) to hard ($HP > 4.0$ tsf). The unconfined compressive strength of the cohesive soil samples tested, obtained from the hand penetrometer, ranged from 0.5 to over 4.5 tsf (limit of instrument). The relative density of granular soils is primarily derived from SPT blow counts (N_{60}). Based on the SPT blow counts obtained, the granular soil encountered ranged from loose ($5 \leq N_{60} \leq 10$ blows per foot [bpf]) to very dense ($N_{60} > 50$ bpf). Overall blow counts recorded from the SPT sampling ranged from 9 bpf to split spoon sampler refusal. Split spoon sampler refusal is defined as exceeding 50 blows from the hammer with less than 6.0 inches of penetration by the split spoon sampler.

Natural moisture contents of the soil samples tested ranged from 4 to 26 percent. The natural moisture content of the cohesive soil samples tested for plasticity index ranged from 10 percent below to 5 percent above their corresponding plastic limits. In general, the soil exhibited natural moisture contents considered to be significantly below to moderately 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 (Auger Refusal)	
		Depth (feet)	Elevation (feet msl)
B-015-7-13	721.8	70.5	651.3
B-108-2-13	722.1	N/A	N/A
B-108-3-13	722.9	N/A	N/A
B-108-9-15	722.4	67.0	655.4

Bedrock was encountered in borings B-015-7-13 and B-108-9-15 at a depth of 70.5 and 67.0 feet below the ground surface, respectively, which corresponds to and elevation of 651.3 and 655.4 feet msl. The cored bedrock in boring B-015-7-13 consisted of dolomite, and the cored bedrock in boring B-108-9-15 consisted of limestone. The dolomite bedrock was described as brown and gray, slightly weathered, strong, very thin to medium bedded, cherty, crystalline and siliceous with calcite/pyrite deposits and chert nodules and lenses, and is moderately fractured to fractured with slightly rough to rough, open apertures. The limestone bedrock was described as brown and gray, slightly weathered, moderately strong to strong, medium to thin bedded, cherty, dolomitic, crystalline and slightly fractured with slightly rough, narrow apertures.

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

Table 4. Rock Core Summary

Boring	Core No.	Elevation (feet msl)	Recovery (%)	RQD (%)	Unconfined Compressive Strength
B-015-7-13	RC-1	651.3 to 646.3	97	58	q_u @ 72.1' = 12,300 psi
	RC-2	646.3 to 641.3	95	58	N/A
B-108-9-15	RC-1	655.3 to 652.3	85	69	q_u @ 68.6' = 12,574 psi
	RC-2	652.3 to 647.3	100	86	N/A
	RC-3	647.3 to 642.3	100	95	q_u @ 75.4' = 7,788 psi

It should be noted that bedrock naturally 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 value, was fair (50% < RQD ≤ 75%) to excellent (90% < RQD ≤ 100%).

4.4 Groundwater

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

Table 5. Groundwater Levels

Boring Number	Ground Elevation (feet msl)	Initial Groundwater		Upon Completion	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-015-7-13	721.8	30.0	691.8	N/A [1]	N/A
B-108-2-13	722.1	37.0	685.1	N/A [1]	N/A
B-108-3-13	722.9	37.0	685.9	37.0	685.9
B-108-9-15	722.4	33.5	688.9	N/A [1]	N/A

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

Groundwater was encountered initially during drilling in the borings at depths ranging from 30.0 to 37.0 feet below existing grade, which corresponds to elevations ranging from 685.1 to 691.8 feet msl. At the completion of drilling and after removing the augers, groundwater accumulated in the borehole of boring B-108-3-13 to a depth of 37.0 feet below existing grade, which corresponds to an elevation of 685.9 feet msl. The groundwater level at the completion of drilling in the remaining borings could not be measured due to the addition of either mud to counteract heaving sands that were encountered during drilling or 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 review of existing geotechnical information have 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 bridge structure were provided by Burgess and Niple. Based on the information provided, it is understood that the proposed bridge will consist of a two-span prestressed concrete I-beam with composite reinforced concrete deck structure with pile supported stub abutment behind an MSE wall at the rear abutment and piers supported on drilled shafts and will have a total length of approximately 280 feet and width of approximately 31.5 feet. The roadway profile will be elevated approximately 35 feet above the existing ground surface grade. Foundation recommendations for the combined Pier A are provided in the FRA-70-1321A Structure Foundation Exploration Report. Therefore, recommendations for this substructure unit are not provided in this report. Proposed structural data was obtained from design details provided by Burgess and Niple and are included in Table 6.

Table 6. Structure and Bridge Design Elevations

Substructure Unit	Structure Component [1]	Elevation [1] (feet msl)	Design Maximum Factored Load
Rear Abutment (B-108-3-13 and B-108-9-15)	Profile Grade	758.7	268 kips/pile
	Bottom of Footing	745.3	
	Bottom of Wall (Leveling Pad)	723.0	
Pier 1 (B-108-9-15 and B-015-7-13)	Top of Shaft	721.0	1,317 kips/shaft

1. Proposed foundation elevations and structural loading based on structure information provided by Burgess and Niple.

Retaining Wall 4W11 will be located at the rear abutment of the proposed structure to provide the required grade separation to support the configuration. It is understood that a mechanically stabilized earth (MSE) wall type is being considered as the preferred wall type for the entire alignment of Retaining Wall 4W11. Based on plan information provided by Dynotec, the wall alignment will be turned back approximately 45 degrees at both sides of the abutment and graded down to existing grade on the south side of the proposed structure. On the north side of the proposed structure, the wall will be graded down and turn back again at approximately 45 degrees and graded down adjacent to the proposed I-71 northbound alignment until a transition can be made to graded embankments. The wall height will range from 4.8 feet at the beginning of the wall alignment to a maximum height of 35.6 feet at the proposed abutment and back down to a height of 3.6 feet at the end of the wall alignment. The total wall length is approximately 328 lineal feet.

5.1 Driven Pile Recommendations

It is understood that driven piles are to be utilized at the rear abutment substructure. Given the depth of bedrock encountered in the borings performed, it is recommended that CIP pipe piles (ODOT Item 507.07) driven to the frictional bearing values listed below or 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 2007 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 7 shows recommended pile lengths and the corresponding ultimate bearing value (R_{ndr}) of CIP pipe piles and factored structural axial resistance ($R_{R\,max}$) of steel H-piles and associated resistance factors (φ):

Table 7. FRA-71-1518A Driven Pile Recommendations

Substructure Reference	Ground Elevation [1]	Pile Size [2]	Pile Elevation (feet msl)		Pile Length [4] (feet)	R_{ndr} [5] / $R_{R\,max}$ [6] (kips/pile)	Sleeve Length [7] (feet)	φ [8,9]
			Top [3]	Tip				
Rear Abutment (B-108-3-13 and B-108-9-15)	733.4	14" CIP	746.3	690.5	60	390	23.8	0.7
		HP 10x42	746.3	655.5	95	310	23.8	N/A

1. *Ground elevation listed is the ground elevation at the respective boring locations.*
2. *A steel pile point is recommended to protect the tips of the CIP pipe piles during pile installation.*
3. *The top of pile elevation corresponds to the pile cutoff elevation, which is 1.0-foot above the proposed bottom of footing elevation.*
4. *Per Section 202.3.2 of the 2007 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 ultimate bearing value (R_{ndr}) is based on the maximum factored load per pile and was calculated in accordance with Section 202.2.3.2.b of the 2007 ODOT BDM.*
6. *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 2007 ODOT BDM.*
7. *Sleeve length represents the required length of pile that should be sleeved within the MSE wall backfill, including the foundation preparation.*
8. *The resistance factor listed for CIP pipe piles assumes dynamic testing of the pile elements per Section 303.4.2.7 of the 2007 ODOT BDM.*
9. *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, $\varphi_c = 0.50$, for H-piles subject to damage due to severe driving conditions.*

The CIP pipe piles were analyzed using the DRIVEN software program, and the results are provided in Appendix V. The ultimate bearing value (R_{ndr}) provided in Table 7 for CIP pipe piles is based on the maximum factored load per pile and was calculated in accordance with Section 202.2.3.2.b of the 2007 ODOT BDM. Given that the CIP piles will be driven to end bear in dense to very dense sand and gravel at the elevation noted above, it is anticipated that the capacity will be achieved almost solely through end bearing. However, if results of dynamic pile load testing indicate that the piles have not achieved the required capacity at the end of driving, then it is recommended that restrike of the pile be performed after a hold period which will allow soil setup to occur. A hold period of three (3) days should be specified between the end of driving the pile and the time of restrike to allow adequate soil setup to occur. Settlement is estimated to be less than 1.0 inch for CIP piles driven to the ultimate bearing values listed in Table 7.

Per Section 202.2.3.2.a of the 2007 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 7 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. **The factored structural axial resistance 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. It is estimated that refusal will be met within the upper 6.0 inches of the surficial bedrock. Settlement is estimated to be less than 1.0 inch for H-piles driven to refusal on bedrock.

We emphasize that the pile lengths and ultimate bearing values presented in Table 7 for CIP pipe piles are estimates using empirical equations based on the derived characteristics of the soils encountered in the subject borings drilled. The actual pile capacities should be verified using static or dynamic pile load testing as detailed in Sections 303.4.2.6 and 303.4.2.7 of the 2007 ODOT BDM. The most accurate method for determining pile capacities and lengths is to drive test piling at the site and perform static load testing in accordance with the ASTM D1143 procedure. Dynamic pile load testing should be performed in accordance with ASTM 4945. Static or dynamic load testing is not required for H-piles driven to refusal on bedrock. Further installation considerations are presented in Section 5.1.3.

5.1.1 Downdrag Considerations

The anticipated total settlement at the facing of the MSE wall at the rear abutment is 2.83 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). The surcharge should remain 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 **fifteen (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.**

5.1.2 Driveability

A drivability analysis was performed in accordance with Section 10.7.8 of the 2014 AASHTO LRFD 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 CIP pipe pile and H-pile sections. The minimum wall thickness utilized in the driveability analysis for CIP pipe piles was determined from the following equation per ODOT Item 507.06 for the ultimate bearing values listed in Table 7.

$$t = UBV / 900{,}000$$

Where:
t = pile wall thickness in inches
UBV = design ultimate bearing value in pounds

Based on the results of this analysis and using a wall thickness as determined from ODOT Item 507.06, it appears that the driving stresses induced on the CIP pipe piles **would not exceed** 90 percent of the yield stress for A252, Grade 2 steel (f_y = 35 ksi, $0.9f_y$ = 31.5 ksi) if driven to the depths provided in Table 7. Please note that the pile wall thickness utilized in the driveability analysis was rounded up to the nearest 1/16-inch increment, which resulted in a pile wall thickness of 0.4375 inches.

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 7. 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 2007 ODOT BDM, steel pile points should be used when the piles are driven to bear on strong bedrock.

5.1.3 Driven Pile Considerations

Proper pile installation is as important as pile design in order to obtain a cost effective and safe product. Driven piles must be installed to develop adequate soil resistance without structural damage. Because piles cannot be visually inspected after installation, direct quality control of the finished product is impossible. Consequently, substantial control must be exercised over peripheral operations leading to the pile placement within the foundation. It is essential that installation be considered during the design stage to insure that piles shown on the plans can be installed. Construction monitoring should be employed in (1) pile materials, (2) installation equipment, and (3) the estimation of the static load capacity.

It is recommended that the contractor submit a wave equation analysis (bearing graph) of his driving equipment, or the necessary pile driving and equipment data to perform the wave equation analysis, for hammer approval. A constant capacity wave equation analysis (inspector’s chart) should also be performed to assist field personnel during inspection in accordance with the 2007 ODOT BDM.

5.2 Drilled Shaft Recommendations

It is understood that drilled shaft foundations are planned for support of the proposed Pier 1 due to proximity of the substructure unit to the existing 42-inch storm force main. Given the proposed loading per shaft at Pier 1, friction bearing drilled shafts within the overburden soils are not economically feasible foundation options due to the size and number of shafts that would be required to support the proposed loading. Therefore, it is recommended that the drilled shafts be extended through the surficial soils to bear on or within the underlying limestone bedrock at Pier 1. The elevation of the top of bedrock at Pier 1 was interpolated based on the elevation of the top of bedrock encountered in borings B-015-7-13 and B-108-9-15, and was estimated at El. 653.5 feet msl.

Per Section 10.8.3.5.4c of the 2018 AASHTO LRDF Bridge Design Specifications (BDS), a minimum rock socket length of 1.5 times the diameter of the drilled shaft within the rock socket ($1.5B_{RS}$) is required to utilize the full end bearing resistance within the bedrock unit that the shafts are end bearing in/on. However, based on discussions with the ODOT Office of Geotechnical Engineering (OGE), a reduced tip resistance can be utilized for shafts not extended to the required minimum socket length of $1.5B_{RS}$.

Using equation 10.8.3.5.4c-1 of the AASHTO LRFD BDS, the nominal end bearing resistance for drilled shafts socketed a minimum of $1.5B_{RS}$ into intact rock is 2.5 times the unconfined compressive strength of the bedrock unit that the shaft tip is bearing on or within. Based on unconfined compression tests performed on limestone rock cores obtained from the borings performed at the subject piers, the unconfined compressive strength ranges from 7,788 to 12,574 psi. Using equation 10.8.3.5.4c-1 and the limiting unconfined compressive strength from the given range for the limestone bedrock, it is recommended that **drilled shaft foundations socketed a <u>minimum</u> of $1.5B_{RS}$ into the bedrock to bear on or within the competent limestone bedrock be proportioned for a nominal end bearing resistance of 2,804 ksf at the strength limit state**.

Where lateral load demands do not require a rock socket length of $1.5B_{RS}$, the socket length can be reduced or the shaft can bear on the bedrock surface with no rock socket. If the rock socket is reduced to a length less than $1.5B_{RS}$, a reduced nominal end bearing resistance should be utilized based on equations 10.8.3.5.4c-2 and 10.8.3.5.4c-3 of the AASHTO LRFD BDS, which is as follows:

$$q_p = A + q_u \left[m_b \left(\frac{A}{q_u} \right) + s \right]^a$$

In which:

$$A = \sigma'_{vb} + q_u \left[m_b \left(\frac{\sigma'_{vb}}{q_u} \right) + s \right]^a$$

Where:

σ'_{vb} = vertical effective stress at the socket bearing (tip) elevation (ksf)

s, a and m_b = Hoek-Brown strength parameters for fractured rock mass determined from GSI in accordance with Section 10.4.6.4 of the AASHTO LRFD BDS

q_u = unconfined compressive strength of intact rock (ksf)

Based on discussions with ODOT OGE, the condition of the rock mass for the determination of the GSI rating should consider the limestone to have a "closed" joint condition, a "blocky" structure and a "good" joint surface condition. Using this description for the structure and surface conditions of the rock mass, a GSI rating of 70 was determined from Figure 10.4.6.4-1 of the AASHTO LRFD BDS, and the

Hoek-Brown strength parameters s, a and m_b were calculated as 0.036, 0.50 and 3.08, respectively. The vertical effective stress was estimated considering 68 feet of soil overburden with a buoyant unit weight of 57.6 pcf. Using the above noted equations and the limiting unconfined compressive strength from the given range for the limestone bedrock, it is recommended that **drilled shaft foundations bearing on or within the competent limestone bedrock with a socket length <u>less than</u> 1.5B$_{RS}$ into the bedrock be proportioned for a nominal end bearing resistance of 1,188 ksf at the strength limit state**.

Based on the plan information provided by Burgess and Niple, the proposed shaft diameter at Pier 1 will be 5.0 feet within the overburden soils and 4.5 feet within the bedrock socket. Table 8 lists the estimated elevation of the top of bedrock as well as the proposed rock sock diameter and length from the design plans and, corresponding nominal end bearing resistance to be utilized for the design of the drilled shaft foundations. A resistance factor of $\varphi_{qp} = 0.5$ at the strength limit state should be utilized for design.

Table 8. Drilled Shaft Recommendations

Substructure Unit (Boring)	**Top of Bedrock Elevation (feet msl)**	**Rock Socket Diameter [1] (feet)**	**Proposed Socket Length [1] (feet)**	**Nominal End Bearing Resistance [2] (ksf)**
Pier 1 (B-108-9-15 / B-015-7-13)	653.5	4.5	4.5	1,188

1. *Proposed rock socket diameter and length determined from proposed plan information provided by Burgess and Niple.*
2. *Nominal end bearing resistance provided is the value that should be utilized in the determination of the end bearing resistance per drilled shaft based on the proposed rock socket length and diameter.*

If lateral analysis of the drilled shafts foundations indicates that the rock socket length can be reduced based on the lateral load demands, then the rock socket length may be reduced from those shown in the current design plans. If the rock socket is reduced to a length less than 1.5B$_{RS}$, then the reduced bearing resistance of 1,188 ksf should be utilized for design.

Given the factored end bearing resistances 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. The factored resistance per shaft provided in the design sheets should be the limiting value between the factored geotechnical resistance and the factored axial compressive resistance of the shaft.

Drilled shafts designed in accordance with the requirements presented above should experience a maximum settlement estimated to be less than 0.5 inches. Group settlement of the shafts, socketed into bedrock, is considered negligible for a minimum spacing of 2.0 shaft diameters center-to-center. Drilled shaft calculations are provided in Appendix VII.

5.3 Lateral Design Considerations

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 VIII. 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 9 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 VIII.

Table 9. 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 Retaining Wall 4W11 Recommendations

As previously noted, Retaining Wall 4W11 will be located at the rear abutment of the proposed structure. It is understood that a mechanically stabilized earth (MSE) wall type is being considered as the preferred wall type for the entire alignment of Retaining Wall 4W11. MSE walls are constructed on earthen foundations at a minimum depth of 3.0 feet below grade, as defined by the top of the leveling pad to the ground surface located 4.0 feet from the face of the wall. Per Section 204.6.2.1 of the 2007 ODOT BDM, the height of the MSE wall 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 2007 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 backwall of the abutment footing, and prevent any load transfer from these forces to the coping and facing panels. For portions of the wall outside the limits of the bridge abutments, the straps should be installed the full height of the wall. The width of the MSE wall foundation (B) is defined by the length of the reinforced soil mass. Per the Section 204.6.2.1 of the 2007 ODOT BDM and Supplemental Specification (SS) 840, the minimum length of the reinforced soil mass is equal to 70 percent of the height of the MSE wall or 8.0 feet whichever is greater. A non-structural bearing leveling pad consisting of a minimum of 6.0-inches of unreinforced concrete should be placed at the base of the wall facing for constructability purposes. Please note that the leveling pad is not a structural foundation.

Based upon the proposed plan information, the maximum wall height at the rear abutment is anticipated to be 35.7 feet from the top of the leveling pad to the proposed profile grade of the roadway. The wall will be turned back approximately 45 degrees and graded down to the north and south of the abutment, and will then turn back again at approximately 45 degrees at the north side of Ramp A5, just south of the I-71 northbound ramp. It is understood that 2:1 backslopes will be graded up to the proposed Ramp A5 roadway from the top of the wall where it extends away from the rear abutment. 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. Existing fill material comprised of medium dense gravel with sand and silt (ODOT A-2-4) was encountered at the proposed bearing elevation along the wall alignment, which extends to a depths ranging from 5.0 to 8.0 feet below the proposed bearing elevation, overlying medium stiff to stiff silty clay (ODOT A-6b). Based on the blow counts obtained within fill material and the absence of significant amounts of deleterious materials, the existing fill material is considered suitable to support the proposed retaining wall. Additionally, the final approximately 80 feet of the wall alignment (between Sta. 2+55 and 3+38, BL Wall 4W11) will be bearing partially or completely on new embankment fill that will be placed as part of the construction of the ramps in this area.

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.

Groundwater was encountered in the borings performed in the vicinity of the wall at elevations ranging from 685.1 to 691.8 feet msl. Based on the plan information provided, the normal water elevation for the Scioto River is 696.2 feet msl with a 100-year flood elevation of 715.1 feet msl. For the analysis of the retaining wall, a design groundwater elevation of 715.1 feet msl was utilized, which is 7.9 feet below the bottom of wall (top of leveling pad) elevation.

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 Retaining Wall 4W11 are provided in Table 10.

Table 10. 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
Existing Fill: Medium Dense Gravel with Sand and Silt (ODOT A-2-4)	125	32	0	N/A
Soft to Stiff Silty Clay (ODOT A-6b)	115	26	0	1,500
Medium Dense to Very Dense Natural Granular Soils (ODOT A-1-a, A-1-b, A-2-6, A-3a, A-4a)	125 to 135	34 to 41	0	N/A
Very Stiff Sandy Silt (ODOT A-4a)	120	29	0	2,250

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

5.4.2 Bearing Stability

Existing fill material comprised of medium dense gravel with sand and silt (ODOT A-2-4) was encountered at the proposed bearing elevation along the wall alignment, which extends to a depths ranging from 5.0 to 8.0 feet below the proposed bearing elevation, overlying medium stiff to stiff silty clay (ODOT A-6b). Based on the blow counts obtained within fill material and the absence of significant amounts of deleterious materials, the existing fill material is considered suitable to support the proposed retaining wall. Given the relatively shallow thickness of the gravel with sand and silt (existing fill) layer, a two layer bearing stratum should be evaluated for this wall. Therefore, the bearing resistance was calculated considering just the drained and undrained properties of the underlying medium stiff to stiff silty clay (ODOT A-6b).

MSE wall foundations bearing on these soils or new embankment fill, placed and compacted in accordance with ODOT Item 203, may be proportioned for a factored bearing resistance as indicated in Table 11. A geotechnical resistance factor of φ_b=0.65 was considered in calculating the factored bearing resistance at the strength limit state. The reinforcement length presented in the following table represents the minimum foundation width required to satisfy external and global stability requirements, expressed as a percentage of the wall height.

Table 11. Retaining Wall 4W11 MSE Wall Design Parameters

From Station [1]	To Station [1]	Wall Height Analyzed (feet)	Backslope Behind Wall in Analysis	Minimum Required Reinforcement Length [2] (feet)	Bearing Resistance at Strength Limit (ksf)		Strength Limit Equivalent Bearing Pressure [4] (ksf)
					Nominal	Factored [3]	
0+00	1+78	35.7	Level	28.6 (0.80H)	13.59	8.83	7.70
1+78	3+38	20.0	2:1	17.0 (0.85H)	10.11	6.57	5.77

1. *Station referenced to the baseline of Wall 4W11.*
2. *The required foundation width is expressed as a percentage of the wall height, H.*
3. *A geotechnical resistance factor of φ_b=0.65 was considered in calculating the factored bearing resistance at the strength limit state.*
4. *The strength limit equivalent bearing pressure is the uniformly distributed pressure asserted by the wall over an effective base width based on the eccentricity of the wall system at the strength limit state.*

Rii performed a verification of the bearing pressure exerted on the subgrade material for the maximum specified wall heights indicated in Table 11. Based on the minimum length of reinforced soil mass presented, the factored equivalent bearing pressure exerted below the wall **will not exceed** the factored bearing resistance at the strength limit state under drained conditions. However, using the undrained shear strength noted in Table 10 for the silty clay (ODOT A-6b) material, the analysis indicates that the factored equivalent bearing pressure exerted below the wall **will exceed** the factored bearing resistance at the strength limit state under undrained conditions. If a layered soil profile is considered (as outlined in Section 10.6.3.1.2e of the 2018 AASHTO LRFD BDS) and the upper gravel with sand and silt layer (ODOT A-2-4) is modeled as a cohesive soil with an undrained shear strength of approximately 4,000 psf (based on the SPT N-values obtained within that material), then the nominal bearing resistance increases to the approximate resistance calculated under drained conditions (considering just the silty clay layer properties as provided in Table 11). Therefore, for temporary scenarios where undrained loading of the underlying silty clay soils occurs, the factored equivalent bearing pressure exerted below the wall **will not exceed** the factored bearing resistance at the strength limit state under drained conditions considering a two layer bearing stratum.

5.4.3 Settlement Evaluation

The compressibility parameters utilized in the settlement analyses of the proposed MSE walls are provided in Table 12.

Table 12. Compressibility Parameters Utilized in Settlement Analysis

Material Type	γ (pcf)	LL (%)	C_c (1)	C_r (2)	e_o (3)	C_v (4) (ft^2/yr)	N_{60}	C' (5)
Existing Fill: Medium Dense Gravel with Sand and Silt (ODOT A-2-4)	125	N/A	N/A	N/A	N/A	N/A	14	71 to 83
Soft to Stiff Silty Clay (ODOT A-6b)	115	37	0.243	0.012	0.561	400	N/A	N/A
Medium Dense to Very Dense Natural Granular Soils (ODOT A-1-a, A-1-b, A-2-6, A-3a, A-4a)	125 to 135	N/A	N/A	N/A	N/A	N/A	19 to 37	44 to 91
Very Stiff Sandy Silt (ODOT A-4a)	120	18	0.072	0.007	0.413	1,000	N/A	N/A

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

Results of the settlement analysis are tabulated in Table 13. Total settlements of up to 5.32 inches at the center of the reinforced soil mass and 3.16 inches at the facing of the wall are anticipated along the alignment of retaining wall 4W11. Based on the results of the analysis, 90 percent of the total settlement is anticipated to occur over a period of approximately fifteen (15) days. Please note that the consolidation settlement and time rate of consolidation are based on estimates using correlated compressibility parameters provided in Table 12 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 13. Retaining Wall 4W11 MSE Wall Settlement Results

From Station [1]	To Station [1]	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	
0+00	1+78	35.7	Level	5.47	5.323	3.156	15
1+78	3+38	20.0	2:1	4.08	2.769	2.287	15

1. The service limit equivalent bearing pressure is the uniformly distributed pressure asserted by the wall over an effective base width based on the eccentricity of the wall system at the service limit state.

Per Section 204.6.2.1 of the ODOT BDM, “the maximum allowable differential settlement in the longitudinal direction (regardless of the size of panels) is one (1) percent.” Based on the total anticipated settlement at the facing of the walls, maximum differential settlements in the longitudinal directions are anticipated to be less than 1/500, which is within the tolerable limit of 1/100. If the total or differential settlement values predicted for the proposed walls present an issue with respect to the deformation tolerances that the walls can withstand, then measures should be taken to minimize the amount of settlement that will occur. This can be achieved by preloading the site and consolidating the underlying soils prior to constructing the walls. If preloading the site is not a desired option, then consideration could be given to ground improvement through the use of stone columns. Settlement calculations are provided in Appendix IX.

5.4.4 Eccentricity (Overturning Stability)

The resistance of the MSE wall to overturning will be dependent on the on the location of the resultant force at the bottom of the wall due to the overturning and resisting moments acting on the wall. For MSE walls, overturning stability is determined by calculating the eccentricity of the resultant force from the midpoint of the base of the wall and comparing this value to a limiting eccentricity value. Per Section 11.10.5.5 of the 2018 AASHTO LRFD BDS, for foundations bearing on soil, the location of the resultant of the reaction forces shall be within the middle two-thirds ($^2/_3$) of the base width. Therefore, the limiting eccentricity is one-third ($^1/_3$) of the base width of the wall. Rii performed a verification of the eccentricity of the resultant force for the specified wall heights indicated in Table 11. Based on the minimum length of reinforced soil mass presented in Table 11 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 walls to sliding was evaluated per Section 11.10.5.3 of the 2018 AASHTO LRFD BDS. Given that the bearing soils consist of granular material, the sliding resistance was only evaluated under drained conditions. For drained conditions, the sliding resistance is determined by multiplying a coefficient of sliding friction “f” times the total vertical force at the base of the wall. The coefficient of sliding friction is determined based on the limiting friction angle between the foundation soil and the reinforced soil backfill. Based on the soil parameters listed in Section 5.4.1 for the foundation and select granular backfill material, a coefficient of sliding friction of 0.62 was utilized for design based on the limiting friction angle of 34 degrees for the select granular backfill. A geotechnical resistance factor of φ_τ=1.0 was considered in calculating the factored shear resistance between the backfill material and foundation for sliding. Based on the minimum length of reinforced soil mass presented in Table 11 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.

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 the global 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 6.0 manufactured by Rocscience Inc. was utilized to perform the analyses.

Per Section 11.6.2.3 of the 2018 AASHTO LRFD BDS, overall (global) stability for MSE walls that are 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 for the portion of the wall that crosses the abutment substructure 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 11, the resulting factor of safety under drained conditions (long-term stability) and undrained conditions (short-term stability) for the portion of the wall that crosses the abutment substructure using the Spencer’s analysis method was greater than 1.5.

For MSE walls that are not integrated with or supporting structural foundations or elements, global stability is satisfied if the product of the factor of safety from the slope stability output multiplied by the resistance factor φ=0.75 is greater than 1.0. Therefore, global stability for the portions of the wall that are adjacent to the abutment substructure is satisfied when a minimum factor of safety of 1.3 is obtained. For an MSE wall designed with the minimum strap lengths listed in Table 11, the resulting factor of safety under drained conditions (long-term stability) and undrained conditions (short-term stability) for the portions of the wall that are adjacent to the abutment substructure using the Spencer's analysis method was greater than 1.3.

5.4.7 Final MSE Wall Considerations

Based on the results of the external and global stability analyses performed for Retaining Wall 4W11, the recommended controlling strap length is 0.80 times the maximum height of the MSE wall (measured from the top of the leveling pad to the proposed profile grade of the roadway) between Sta. 0+00 and 1+78 and 1.0 times the wall height from Sta. 1+78 to 3+38 (end of the wall). Global stability under drained and undrained conditions were the controlling factors in the determination of the recommended strap length of 80 percent of the wall height for the section between Sta. 0+00 and 1+78. Bearing stability under drained conditions and global stability under both drained and undrained conditions were the controlling factors in the recommended strap length of 85 percent of the wall height between Sta. 1+78 and 3+38.

As noted in Section 5.4.2, bearing stability was not satisfied under undrained conditions for either wall section assuming a uniform bearing stratum with the weakest shear strength parameters within the zone of influence below the bottom of wall. However, as previously noted, if a layered soil profile is considered and the upper gravel with sand and silt layer is modeled a cohesive soil with an undrained shear strength of approximately 4,000 psf, then the nominal bearing resistance increases to the approximate resistance calculated under drained conditions, which satisfies bearing stability requirements.

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

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 14 and Table 15.

Table 14. 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
Loose Granular Soil	120	0	28°	0.32	0.53	5.07
Medium Dense Granular Soil	125	0	32°	0.27	0.47	6.82
Dense to Very Dense Granular Soil	130	0	36°	0.23	0.41	9.09
Compacted Cohesive Engineered Fill	120	2,000	0°	N/A	N/A	N/A
Compacted Granular Engineered Fill	130	0	33°	0.26	0.46	7.41

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

Table 15. 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
Loose Granular Soil	120	0	28°	0.32	0.53	5.07
Medium Dense Granular Soil	125	0	32°	0.27	0.47	6.82
Dense to Very Dense Granular Soil	130	0	36°	0.23	0.41	9.09
Compacted Cohesive Engineered Fill	120	0	30°	0.30	0.50	5.58
Compacted Granular Engineered Fill	130	0	33°	0.26	0.46	7.41

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

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

5.6 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 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 16. 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

5.7 Groundwater Considerations

Based on the groundwater observations made during drilling, groundwater is anticipated to 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. Any seepage or groundwater encountered at this site should be able to be controlled by pumping from temporary sumps. 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 using a rigid tremie pipe. Additional measures may be required depending on seasonal fluctuations of the groundwater level. Note that determining and maintaining actual groundwater levels during construction is the responsibility of the contractor.

6.0 LIMITATIONS OF STUDY

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

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

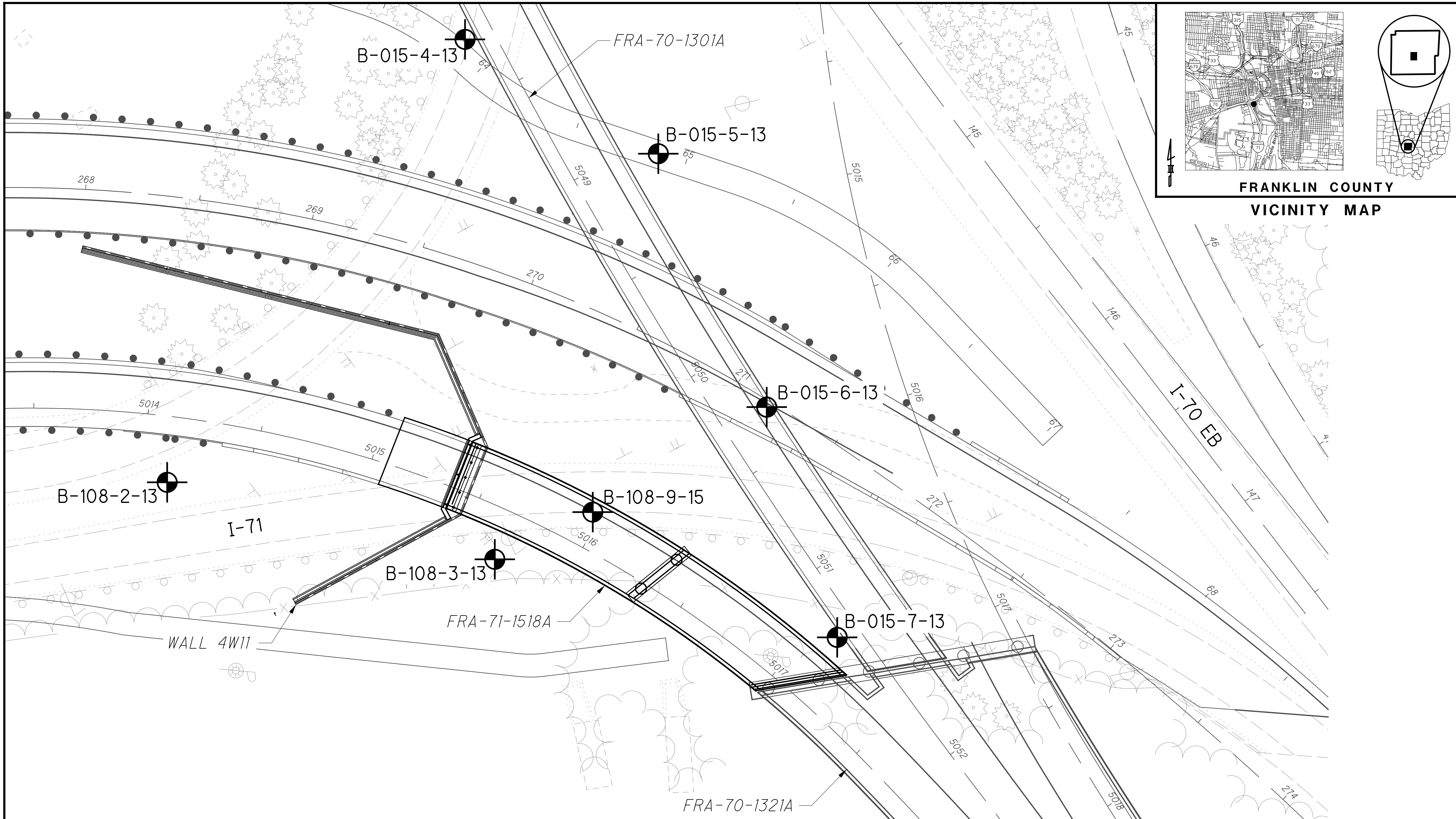
The conclusions and recommendations herein have been based upon the available soil and bedrock information and the design details furnished by a representative of the owner of the proposed project. Any revision in the plans for the proposed construction from those anticipated in this report should be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

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

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

APPENDIX I

VICINITY MAP AND BORING PLAN



BORING PLAN

FRA-71-1518A AND RETAINING WALL 4W11

FRANKLIN COUNTY, OHIO

RII PROJECT NO. W-13-045	DRAWN RRM	RESOURCE INTERNATIONAL, INC.
SCALE: 1"=40' (0 20 40)	REVIEWED BRT	
	DATE 7-11-18	

APPENDIX II

DESCRIPTION OF SOIL TERMS

DESCRIPTION OF SOIL TERMS

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

APPENDIX III

PROJECT BORING LOGS:

B-015-7-13, B-108-2-13, B-108-3-13 and B-108-9-15

DESCRIPTION OF SOIL TERMS

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

CLASSIFICATION OF SOILS

Ohio Department of Transportation

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

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

MATERIAL CLASSIFIED BY VISUAL INSPECTION

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

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

DESCRIPTION OF ROCK TERMS

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

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

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

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

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

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

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

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

Degree of Fracturing

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

Aperture Width

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

Surface Roughness

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

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

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

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-12.68 - PHASE 4A	DRILLING FIRM / OPERATOR: RII / S.M.	DRILL RIG: CME-750 (SN 98048)	STATION / OFFSET: 5051+29.66 / 9.8' RT
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / A.D.	HAMMER: CME AUTOMATIC	ALIGNMENT: BL RAMP C5
PID: 77372 BR ID: FRA-70-1301A	DRILLING METHOD: 3.25" HSA / RC	CALIBRATION DATE: 4/26/13	ELEVATION: 721.8 (MSL) EOB: 80.5 ft.
START: 6/10/13 END: 6/13/13	SAMPLING METHOD: SPT / NQ	ENERGY RATIO (%): 82.6	LAT / LONG: 39.950618516, -83.014254653

EXPLORATION ID **B-015-7-13**

MATERIAL DESCRIPTION AND NOTES	ELEV. 721.8	DEPTHS	SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
0.4' - TOPSOIL (5.0")	721.4																	
HARD, BROWN **CLAY**, SOME FINE GRAVEL, SOME FINE TO COARSE SAND, LITTLE SILT, DRY.		1–2	4 15 12	37	50	SS-1	4.5+	24	14	13	15	34	43	19	24	9	A-7-6 (8)	
	718.8																	
DENSE, GRAY **GRAVEL**, LITTLE FINE TO COARSE SAND, TRACE SILT, TRACE CLAY, DAMP.		3.5–5	9 11 12	32	17	SS-2	-	-	-	-	-	-	-	-	-	6	A-1-a (V)	
	716.3																	
STIFF TO VERY STIFF, DARK BROWN TO BROWNISH GRAY **SILT AND CLAY**, SOME COARSE TO FINE SAND, SOME FINE GRAVEL, DAMP TO MOIST.		6–7.5	3 4 2	8	61	SS-3	2.00	-	-	-	-	-	-	-	-	15	A-6a (V)	
		8.5–10	2 4 4	11	67	SS-4	2.50	26	15	15	16	28	32	17	15	13	A-6a (3)	
		11–12.5	3 6 5	15	56	SS-5	2.00	-	-	-	-	-	-	-	-	20	A-6a (V)	
	708.8																	
LOOSE TO DENSE, BROWN **GRAVEL WITH SAND, SILT, AND CLAY**, DAMP.		13.5–15	2 3 3	8	72	SS-6	-	-	-	-	-	-	-	-	-	17	A-2-6 (V)	
		16–17.5	3 9 5	19	44	SS-7	-	30	20	15	11	24	30	18	12	17	A-2-6 (0)	
	703.8																	
DENSE TO VERY DENSE, GRAY **GRAVEL AND SAND**, LITTLE TO SOME SILT, TRACE CLAY, DAMP TO MOIST.		18.5–20.5			0	ST-8	-	-	-	-	-	-	-	-	-	-		
		21–22.5	5 18 18	50	72	SS-9	-	-	-	-	-	-	-	-	-	7	A-1-b (V)	
		23.5–25	11 21 19	55	67	SS-10	-	49	18	9	23	1	NP	NP	NP	7	A-1-b (0)	
		26–27.5	12 27 22	67	83	SS-11	-	-	-	-	-	-	-	-	-	8	A-1-b (V)	
		28.5–30	8 13 17	41	11	SS-12	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	

2014 ODOT BORING LOG-RII NE BRIDGE ID - OH DOT.GDT - 3/14/15 17:33 - U:\GI8\PROJECTS\2013\W-13-045.GPJ

PID: 77372 | BR ID: FRA-70-1301A | PROJECT: FRA-70-12.68 - PHASE 4A | STATION / OFFSET: 5051+29.66 / 9.8 RT | START: 6/10/13 | END: 6/13/13 | | B-015-7-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 691.8	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
DENSE TO VERY DENSE, GRAY **GRAVEL AND SAND**, LITTLE TO SOME SILT, TRACE CLAY, DAMP TO MOIST. *(same as above)*		31 32 33																
-HEAVING SANDS ENCOUNTERED @ 33.5'		34	21 50/1"	-	100	SS-13	-	34	30	16	19	1	NP	NP	NP	15	A-1-b (0)	
-INTRODUCED MUD @ 33.5'		35 36 37 38																
		39 40	20 36 38	102	56	SS-14	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	
		41 42 43																
		44 45	13 14 24	52	72	SS-15	-	-	-	-	-	-	-	-	-	8	A-1-b (V)	
	674.8	46 47																
HARD, GRAY **SILTY CLAY**, SOME COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST.		48																
		49 50	16 19 25	61	83	SS-16	4.50	7	7	15	46	25	30	14	16	19	A-6b (10)	
	669.8	51 52																
VERY DENSE, GRAY **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST TO WET.		53																
		54	20 50/1"	-	171	SS-17	-	-	-	-	-	-	-	-	-	17	A-1-b (V)	
		55 56 57 58																
		59	30 50/1"	-	100	SS-18	-	-	-	-	-	-	-	-	-	11	A-1-b (V)	
		60 61																

PID: 77372	BR ID: FRA-70-1301A	PROJECT: FRA-70-12.68 - PHASE 4A	STATION / OFFSET: 5051+29.66 / 9.8 RT	START: 6/10/13	END: 6/13/13	PG 3 OF 3	B-015-7-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 659.7	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
VERY DENSE, GRAY **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST TO WET. *(same as above)*		63																
		64	42 50/4"	-	100	SS-19	-	54	17	10	16	3	22	17	5	10	A-1-b (0)	
		65 – 68																
		69	50/5"	-	20	SS-20	-	-	-	-	-	-	-	-	-	24	A-1-b (V)	
	651.3	70 TR																
DOLOMITE : BROWN AND GRAY, SLIGHTLY WEATHERED, STRONG, VERY THIN TO MEDIUM BEDDED, CHERTY, CRYSTALLINE,, SILICEOUS, CALCITE/PYRITE DEPOSITS, CHERT NODULES AND LENSES, MODERATELY FRACTURED TO FRACTURED, OPEN APERTURE, SLIGHTLY ROUGH TO ROUGH; RQD 58%, REC 96%. -CHERT NODULE @ 71.1' -QU @ 72.1' = 12,300 PSI		71 – 75	58		97	RC-1											CORE	
		76 – 80	58		95	RC-2											CORE	
	641.3	EOB																

NOTES: GROUNDWATER INITIALLY ENCOUNTERED @ 30.0'

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

B-015-7-13 – RC-1 – Depth from 70.5 to 75.5 feet

B-015-7-13 – RC-2 – Depth from 75.5 to 80.5 feet

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-12.68 - PHASE 4A	DRILLING FIRM / OPERATOR: RII / S.M.	DRILL RIG: CME-750 (SN 98048)	STATION / OFFSET: 5014+11.97 / 28.9' RT	EXPLORATION ID B-108-2-13
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / K.S.	HAMMER: CME AUTOMATIC	ALIGNMENT: BL RAMP A5	
PID: 77372 BR ID: N/A	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 4/26/13	ELEVATION: 722.1 (MSL) EOB: 50.0 ft.	PAGE
START: 6/6/13 END: 6/6/13	SAMPLING METHOD: SPT	ENERGY RATIO (%): 82.6	LAT / LONG: 39.949854823, -83.014658823	1 OF 2

MATERIAL DESCRIPTION AND NOTES	ELEV. 722.1
0.5' - TOPSOIL (6.0")	721.6
FILL: STIFF TO HARD, BROWNISH GRAY TO DARK GRAY **SILTY CLAY**, SOME TO SOME COARSE TO FINE SAND, LITTLE FINE GRAVEL, DRY TO DAMP. -STONE AND CINDER FRAGMENTS PRESENT THROUGHOUT	716.6
STIFF, DARK GRAY **SILTY CLAY**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP. -COBBLES PRESENT THROUGHOUT	714.1
MEDIUM DENSE, BROWN **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST.	709.1
MEDIUM DENSE, GRAY **GRAVEL WITH SAND, SILT, AND CLAY**, MOIST. -QU @ 13.7' = 0.40 TSF -ATTEMPTED SHELBY TUBE @ 15.5', TUBE CRUSHED @ 17.0'	705.6
SOFT TO STIFF, GRAY **CLAY**, "AND" SILT, SOME COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP TO MOIST. -COBBLES PRESENT @ 18.0'	700.1
MEDIUM DENSE, DARK GRAY TO BROWNISH GRAY **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, DAMP TO MOIST. -COBBLES PRESENT @ 30.0'	

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
4 20 15	48	33	SS-1	4.5+	-	-	-	-	-	-	-	-	7	A-6b (V)
5 7 6	18	67	SS-2	2.00	19	16	12	27	26	36	17	19	16	A-6b (7)
6 6 7	18	78	SS-3	2.00	-	-	-	-	-	-	-	-	16	A-6b (V)
7 9 6	21	39	SS-4	-	-	-	-	-	-	-	-	-	6	A-1-b (V)
9 6 5	15	44	SS-5	-	46	25	8	12	9	24	18	6	6	A-1-b (0)
		54	ST-6	-	55	10	8	15	12	31	19	12	15	A-2-6 (0)
3 4 5	12	56	SS-7	0.50	-	-	-	-	-	-	-	-	24	A-7-6 (V)
2 3 5	11	72	SS-8	1.50	9	10	13	38	30	41	25	16	26	A-7-6 (9)
4 6 5	15	78	SS-9	-	-	-	-	-	-	-	-	-	8	A-1-b (V)
8 9 5	19	33	SS-10	-	62	13	8	11	6	NP	NP	NP	5	A-1-b (0)

PID: 77372	BR ID: N/A	PROJECT: FRA-70-12.68 - PHASE 4A	STATION / OFFSET: 5014+11.97 / 28.9 RT	START: 6/6/13	END: 6/6/13	PG 2 OF 2	B-108-2-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 692.1	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
MEDIUM DENSE, DARK GRAY TO BROWNISH GRAY **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, DAMP TO MOIST. *(same as above)*		31 – 33																
		34 – 35	15 11 11	30	33	SS-11	-	-	-	-	-	-	-	-	-	12	A-1-b (V)	
	685.1	36 – 37 W																
VERY DENSE, BROWNISH GRAY **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST. -HEAVING SANDS ENCOUNTERED @ 38.5' -INTRODUCED MUD @ 38.5' -COBBLES PRESENT @ 40.0'		38																
		39 – 40	10 37 25	85	56	SS-12	-	-	-	-	-	-	-	-	-	11	A-1-b (V)	
		41 – 43																
		44 – 45	15 30 31	84	72	SS-13	-	-	-	-	-	-	-	-	-	8	A-1-b (V)	
	675.1	46 – 47																
HARD, GRAY **SANDY SILT**, SOME CLAY, LITTLE FINE GRAVEL, DAMP.		48																
	672.1	49 – 50 EOB	26 30 22	72	78	SS-14	4.5+	12	9	15	44	20	21	13	8	12	A-4a (6)	

NOTES: GROUNDWATER INITIALLY ENCOUNTERED @ 37.0"

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

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-12.68 - PHASE 4A	DRILLING FIRM / OPERATOR: RII / S.M.	DRILL RIG: CME-750 (SN 98048)	STATION / OFFSET: 5015+67.23 / 22' RT	EXPLORATION ID B-108-3-13
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / K.S.	HAMMER: CME AUTOMATIC	ALIGNMENT: BL RAMP A5	PAGE
PID: 77372 BR ID: FRA-71-1518A	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 4/26/13	ELEVATION: 722.9 (MSL) EOB: 50.0 ft.	1 OF 2
START: 6/7/13 END: 6/7/13	SAMPLING METHOD: SPT	ENERGY RATIO (%): 82.6	LAT / LONG: 39.950228123, -83.014459410	

MATERIAL DESCRIPTION AND NOTES	ELEV. 722.9	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
0.3' - TOPSOIL (4.0")	722.6																	
FILL: MEDIUM DENSE, GRAY TO BROWN **GRAVEL WITH SAND AND SILT**, TRACE CLAY, DAMP.		1–2	10 8 10	25	44	SS-1	-	-	-	-	-	-	-	-	-	7	A-2-4 (V)	
-CINDERS PRESENT IN SS-2 -COBBLES PRESENT @ 4.0'	717.4	4–5	20 12 8	28	56	SS-2	-	-	-	-	-	-	-	-	-	9	A-2-4 (V)	
FILL: STIFF, BROWN **SILTY CLAY**, SOME COARSE TO FINE SAND, SOME FINE GRAVEL, MOIST. -BRICK FRAGMENTS PRESENT IN SS-3	714.9	6–7	4 4 5	12	61	SS-3	1.75	30	15	14	20	21	34	18	16	18	A-6b (3)	
SOFT TO STIFF, BROWN **SILTY CLAY**, LITTLE TO SOME COARSE TO FINE SAND, SOME FINE GRAVEL, DAMP.		9–10	1 5 6	15	67	SS-4	1.50	-	-	-	-	-	-	-	-	17	A-6b (V)	
		11–12	7 3 2	7	44	SS-5	1.50	-	-	-	-	-	-	-	-	14	A-6b (V)	
	707.4	14–15	2 3 3	8	44	SS-6	0.50	-	-	-	-	-	-	-	-	16	A-6b (V)	
LOOSE, BROWN **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST.	704.9	16–17	3 4 3	10	33	SS-7	-	67	13	8	8	4	NP	NP	NP	7	A-1-a (0)	
STIFF, BROWN **SILT AND CLAY**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST.	700.9	19–20	3 4 7	15	44	SS-8	1.75	-	-	-	-	-	-	-	-	23	A-6a (V)	
DENSE, BROWN **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST. -COBBLES PRESENT @ 24.0'	695.9	24–25	11 15 19	47	61	SS-9	-	-	-	-	-	-	-	-	-	7	A-1-a (V)	
MEDIUM DENSE, BROWN **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST.		29	7 11 9	28	56	SS-10	-	31	34	18	14	3	NP	NP	NP	11	A-1-b (0)	

2014 ODOT BORING LOG-RII NE BRIDGE ID - OH DOT.GDT - 3/14/15 17:35 - U:\GI8\PROJECTS\2013\W-13-045.GPJ

PID: 77372	BR ID: FRA-71-1518A	PROJECT: FRA-70-12.68 - PHASE 4A	STATION / OFFSET: 5015+67.23 / 22 RT	START: 6/7/13	END: 6/7/13	PG 2 OF 2	B-108-3-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 692.9	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
MEDIUM DENSE, BROWN **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST. *(same as above)* -COBBLES PRESENT @ 31.0'	690.9	31 – 32																
DENSE TO VERY DENSE, BROWN **GRAVEL**, SOME COARSE TO FINE SAND, LITTLE SILT, TRACE CLAY, MOIST.		33 – 35	14 17 24	56	67	SS-11	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	
		36 – 37 (groundwater @ 37.0')																
-HEAVING SANDS ENCOUNTERED @ 38.5' -COBBLES PRESENT @ 39.0'		38 – 40	5 9 14	32	39	SS-12	-	63	16	7	12	2	NP	NP	NP	12	A-1-a (0)	
		41 – 45	11 31 39	96	17	SS-13	-	-	-	-	-	-	-	-	-	11	A-1-a (V)	
	675.9	46 – 47																
VERY STIFF, GRAY **SILTY CLAY**, SOME FINE GRAVEL, DAMP.	672.9	48 – 50 EOB	5 7 9	22	33	SS-14	3.50	24	21	15	31	9	30	13	17	11	A-6b (3)	

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 37.0' AND AT COMPLETION @ 37.0';

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

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-12.68 - PHASE 4A	DRILLING FIRM / OPERATOR: RII / J.K.	DRILL RIG: MOBILE B-53 (SN 624400)	STATION / OFFSET: 5015+96.50 / 15.7' LT
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / N.A.	HAMMER: AUTOMATIC	ALIGNMENT: BL RAMP A5
PID: 77372 BR ID: FRA-71-1518A	DRILLING METHOD: 3.25" HSA / NQ	CALIBRATION DATE: 4/26/13	ELEVATION: 722.4 (MSL) EOB: 80.1 ft.
START: 3/7/15 END: 3/8/15	SAMPLING METHOD: SPT / RC	ENERGY RATIO (%): 77.7	LAT / LONG: 39.950352963, -83.014507695

EXPLORATION ID B-108-9-15

MATERIAL DESCRIPTION AND NOTES	ELEV. 722.4
0.2' - ASPHALT (3.0")	722.2
0.8' - CONCRETE (9.0")	721.4
0.3' - AGGREGATE BASE (4.0")	721.1
FILL: LOOSE TO MEDIUM DENSE, DARK BROWN **GRAVEL WITH SAND AND SILT**, LITTLE CLAY, MOIST. -LOI = 1.9%	714.4
MEDIUM STIFF TO STIFF, DARK GRAY TO BROWN **SILTY CLAY**, SOME COARSE TO FINE SAND, LITTLE FINE GRAVEL, MOIST.	709.4
MEDIUM DENSE, BROWNISH GRAY **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, MOIST.	706.9
MEDIUM DENSE, GRAY **SANDY SILT**, SOME FINE GRAVEL, TRACE CLAY, MOIST.	704.4
VERY STIFF, BROWNISH GRAY **SILTY CLAY**, SOME COARSE TO FINE SAND, LITTLE FINE GRAVEL, MOIST.	701.9
MEDIUM DENSE TO DENSE, BROWN **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST.	

SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
11 / 8 / 8	21	100	SS-1	-	48	18	9	17	8	25	16	9	8	A-2-4 (0)
6 / 4 / 3	9	100	SS-2	-	-	-	-	-	-	-	-	-	11	A-2-4 (V)
4 / 5 / 4	12	100	SS-3	-	43	20	11	20	6	25	19	6	10	A-2-4 (0)
3 / 3 / 2	6	33	SS-4	1.00	-	-	-	-	-	-	-	-	26	A-6b (V)
		67	ST-5	1.25	14	13	17	32	24	37	19	18	22	A-6b (7)
8 / 7 / 8	19	50	SS-6	-	-	-	-	-	-	-	-	-	7	A-1-b (V)
5 / 8 / 10	23	78	SS-7	-	30	16	15	34	5	NP	NP	NP	14	A-4a (1)
4 / 5 / 7	16	100	SS-8	2.25	-	-	-	-	-	-	-	-	22	A-6b (V)
8 / 14 / 14	36	100	SS-9	-	-	-	-	-	-	-	-	-	7	A-1-b (V)
15 / 12 / 10	28	67	SS-10	-	51	14	18	14	3	NP	NP	NP	7	A-1-b (0)
15 / 16 / 18	44	100	SS-11	-	-	-	-	-	-	-	-	-	8	A-1-b (V)
9 / 6 / 7	17	100	SS-12	-	-	-	-	-	-	-	-	-	9	A-1-b (V)

DEPTHS: 1 – 29 ft (scale); BACK FILL column shown graphically.

PID: 77372	BR ID: FRA-71-1518A	PROJECT: FRA-70-12.68 - PHASE 4A	STATION / OFFSET: 5015+96.50 / 15.7 LT	START: 3/7/15	END: 3/8/15	PG 2 OF 3	B-108-9-15

MATERIAL DESCRIPTION AND NOTES	ELEV. 692.4	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
MEDIUM DENSE TO DENSE, BROWN **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, MOIST. *(same as above)*	690.4	31 – 32																
MEDIUM DENSE TO DENSE, GRAY **GRAVEL**, SOME COARSE TO FINE SAND, LITTLE SILT, TRACE CLAY, MOIST.		33 W																
		34 – 35	19 18 18	47	100	SS-13	-	61	17	7	12	3	NP	NP	NP	10	A-1-a (0)	
		36 – 38																
		39 – 40	5 8 7	19	56	SS-14	-	-	-	-	-	-	-	-	-	15	A-1-a (V)	
		41 – 43																
		44 – 45	15 21 11	41	39	SS-15	-	-	-	-	-	-	-	-	-	16	A-1-a (V)	
-BOULDER PRESENT @ 45.0'		46 – 48																
		49 – 50	24 15 15	39	0	SS-16	-	-	-	-	-	-	-	-	-	-		
		50	30	-	0	2S-16A	-	-	-	-	-	-	-	-	-	-		
	670.4	51																
DENSE, GRAY **GRAVEL WITH SAND, SILT, AND CLAY**, MOIST.		52 – 53																
		54 – 55	14 11 13	31	56	SS-17	-	-	-	-	-	-	-	-	-	15	A-2-6 (V)	
	665.4	56																
VERY STIFF, GRAY **SANDY SILT**, LITTLE FINE GRAVEL, LITTLE CLAY, DAMP.		57 – 58																
		59 – 60	6 6 8	18	100	SS-18	2.50	19	12	19	36	14	18	13	5	11	A-4a (3)	
	660.4	61																

PID: 77372	BR ID: FRA-71-1518A	PROJECT: FRA-70-12.68 - PHASE 4A	STATION / OFFSET: 5015+96.50 / 15.7 LT	START: 3/7/15	END: 3/8/15	PG 3 OF 3	B-108-9-15

MATERIAL DESCRIPTION AND NOTES	ELEV. 660.3	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
MEDIUM DENSE, BROWN **COARSE AND FINE SAND**, LITTLE SILT, TRACE CLAY, TRACE FINE GRAVEL, WET. *(same as above)* -HEAVING SAND ENCOUNTERED @ 63.5' -INTORDUCED WATER @ 63.5'		63–65	6 8 9	22	100	SS-19	-	4	43	32	15	6	NP	NP	NP	19	A-3a (0)	
AUGER REFUSAL @ 67.0'	655.4	67 TR	50/1"	-	100	SS-20	-	-	-	-	-	-	-	-	-	4	Rock (V)	
LIMESTONE : BROWN AND GRAY, SLIGHTLY WEATHERED, MODERATELY STRONG TO STRONG, MEDIUM TO THIN BEDDED, CHERTY, DOLOMITIC, CRYSTALLINE, SLIGHTLY FRACTURED, NARROW APPERTURES, SLIGHTLY ROUGH; RQD 86%, REC 96%. -QU @ 68.6' = 12,574 PSI		67–70	69		85	RC-1											CORE	
-0.1' CLAY SEAM PRESENT @ 73.7' AND 74.1' -LOSS OF CIRCULATION @ 73.9'		70–75	86		100	RC-2											CORE	
-IRON STAINING PRESENT @ 75.5' -QU @ 75.4' = 7,788 PSI -INCREASINGLY DOLOMITIC FROM 78.5' TO 80.1'	642.3	75–80 EOB	95		100	RC-3											CORE	

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 33.5'

ABANDONMENT METHODS, MATERIALS, QUANTITIES: COMPACTED WITH THE AUGER 400 LBS BENTONITE CHIPS AND SOIL CUTTINGS

B-108-9-15 – RC-1 and RC-2 – Depth from 67.1 to 75.1 feet

B-108-9-15 – RC-3 – Depth from 75.1 to 80.1 feet

APPENDIX IV

LABORATORY TEST RESULTS

UNCONFINED COMPRESSION

ASTM D -2166

6350 Presidential Gateway
Columbus, Ohio 43231
Telephone: (614) 823-4949
Fax Number: (614) 823-4990

PROJECT	FRA-70-12.68
JOB No.	W-13-045
BORING	B-108-2-13
STATION / OFFSET	5014+11.97 / 28.9' Rt.
SAMPLE No. / DEPTH	ST-6 / 13.7 ft
DATE OF TESTING	6/14/2013
TESTED BY	JJH

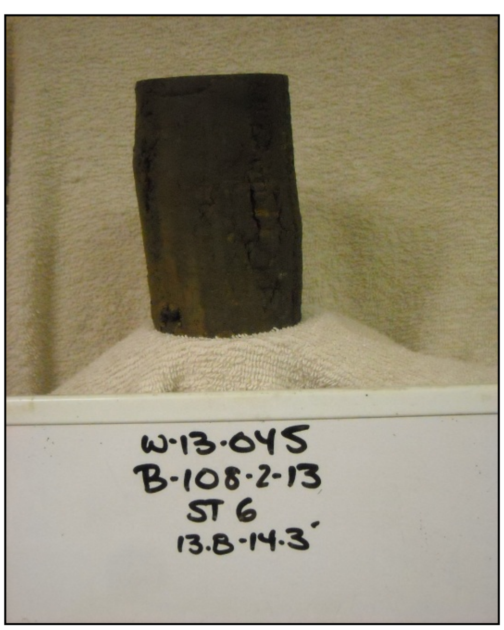
Soil Description: Gray GRAVEL WITH SAND, SILT AND CLAY.
Soil Classification: ODOT A-2-4

Physical Characteristics	L.L.	P.L.	P.I.	Gravel%	C. Sand%	F. Sand%	Silt%	Clay%
	31	19	12	55	10	8	15	12

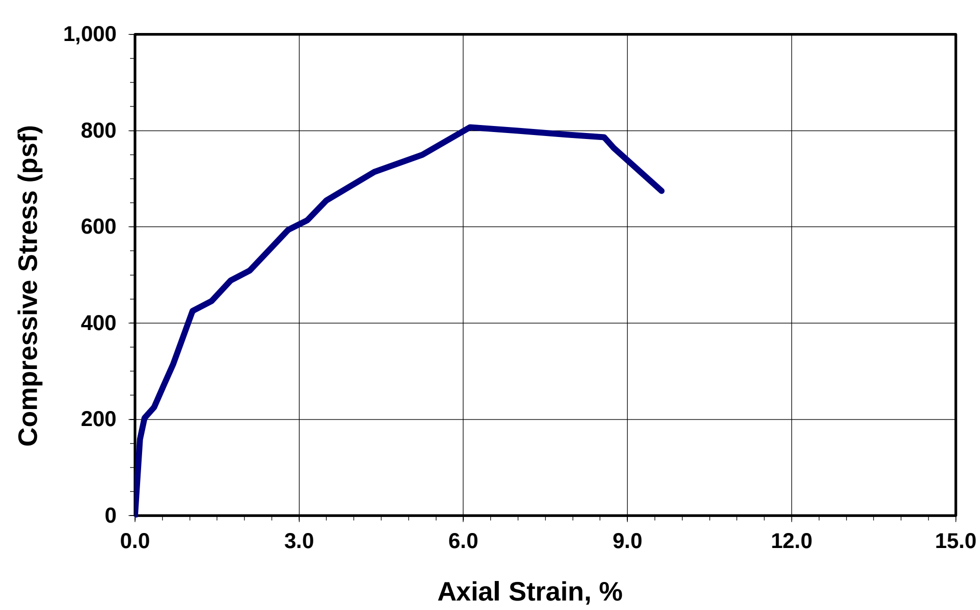
DIAMETER, D_0	2.85	in	72.3	mm	STRAIN RATE	1.00	%/min
AREA, A_0	6.37	in^2	41.1	cm^2	WET SOIL + PAN MASS	1281.5	g
HEIGHT, L_0	5.72	in	145.16	mm	PAN MASS	77.9	g
VOLUME, V_0	36.38	in^3	596.19	cm^3	DRY SOIL + PAN MASS	1062.4	g
MACH. RATE	0.572	in/min			WET DENSITY	126.03	lb/ft^3
WATER CONT.	22.25	%			DRY DENSITY	103.09	lb/ft^3

UNCONFINED COMPRESSION STRESS, q_u	**807**	**psf**	0.40	tsf
AXIAL STRAIN @ FAILURE			6.12	%
HAND PENETROMETER			N/A	tsf

Failure Sketch



Unconfined Compression Test



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-70-12.68

Project No.: W-13-045

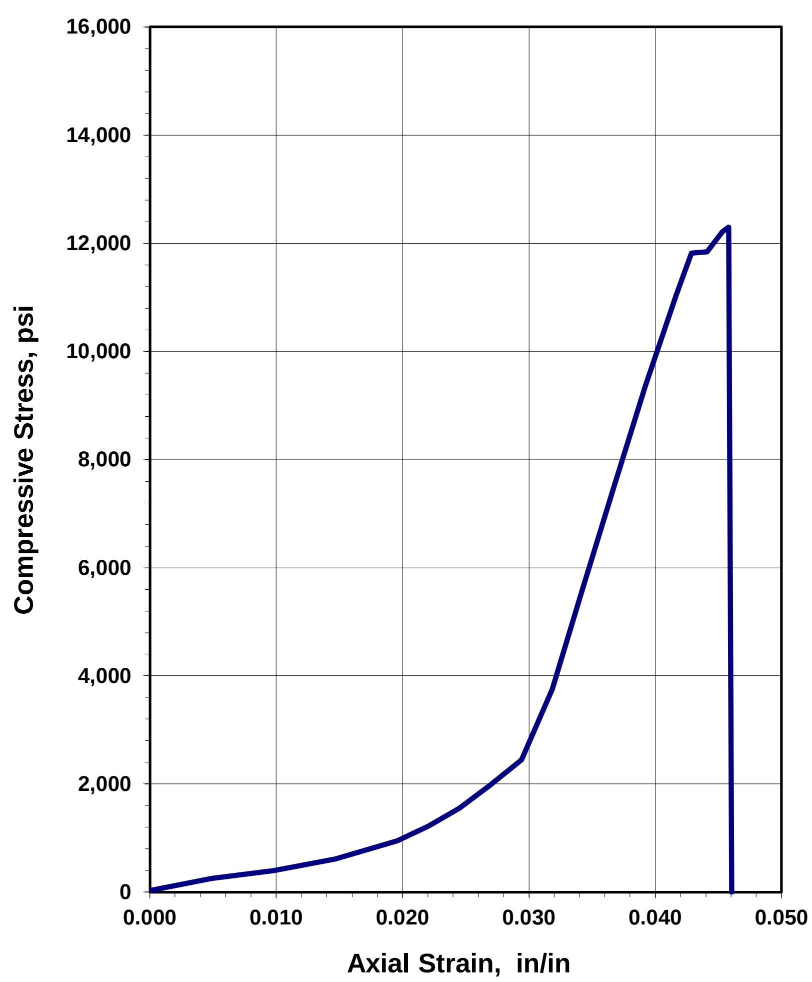
Date of Testing: 7/12/2013

Test Performed by: JJH/TK

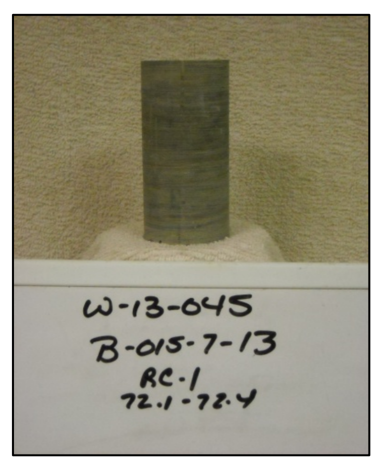
Rock Description: DOLOMITE: Gray and brown, slightly weathered, strong.

Boring No.:	B-015-7-13	Average Length:	4.081 in
Station / Offset:	5051+29.66, 9.8' Rt.	Average Diameter:	1.855 in
Sample No. / Depth:	RC-1 / 72.1 ft.	Length to diameter ratio:	2.200
Moisture condition:	As received	Cross Sectional Area:	2.701 in^2
Rate of Loading:	63.9 lbs/sec	Failure Load:	33,240 lbs
Testing Time:	520 sec	Axial Strain at Failure:	0.0458 in/in
(Rate 2-15 minutes to failure)		Stress:	12,300 psi

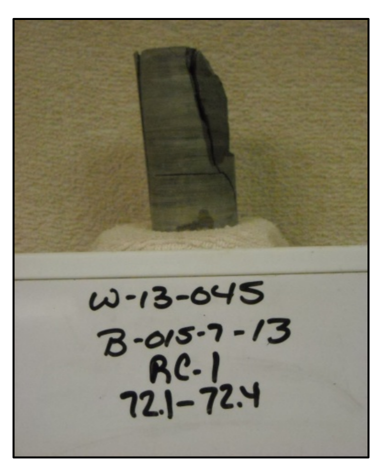
Unconfined Compression Test



Before Testing



After Failure



REMARKS:

RESOURCE INTERNATIONAL, INC.

Engineering Consultants

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

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

Project: FRA-70-12.68

Project No.: W-13-045

Date of Testing: 3/27/2015

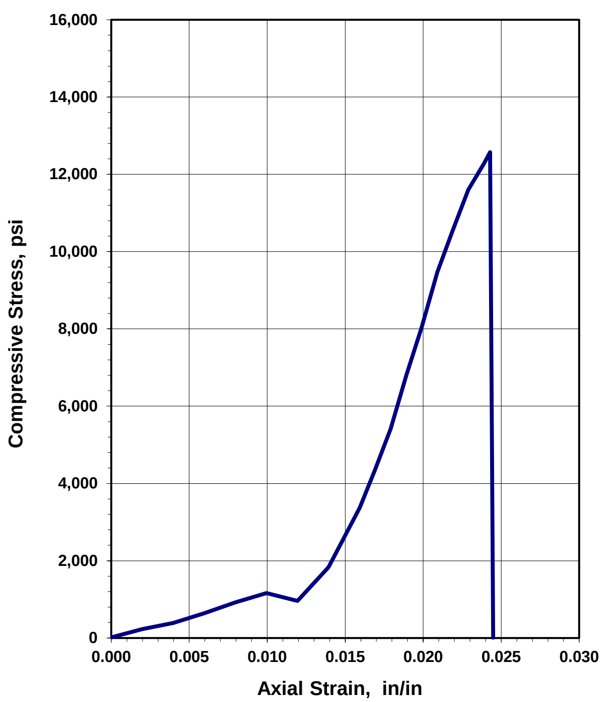
Test Performed by: JH/TK

Rock Description: LIMESTONE: Brown and gray, moderately weathered, strong.

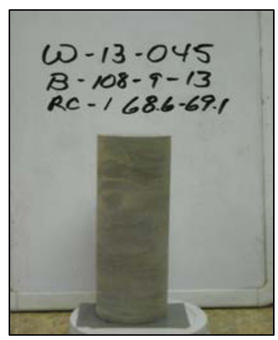
Boring No.:	B-108-9-15	Average Length:	5.023 in
Station / Offset:	5015+96.50, 15.7' Rt.	Average Diameter:	1.862 in
Sample No. / Depth:	RC-1 / 68.6 ft.	Length to diameter ratio:	2.698
Moisture condition:	As received	Cross Sectional Area:	2.722 in^2
		Failure Load:	34,230 lbs
Rate of Loading:	72.7 lbs/sec	Axial Strain at Failure:	0.0243 in/in
Testing Time:	471 sec	Stress:	12,574 psi

(Rate 2-15 minutes to failure)

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)

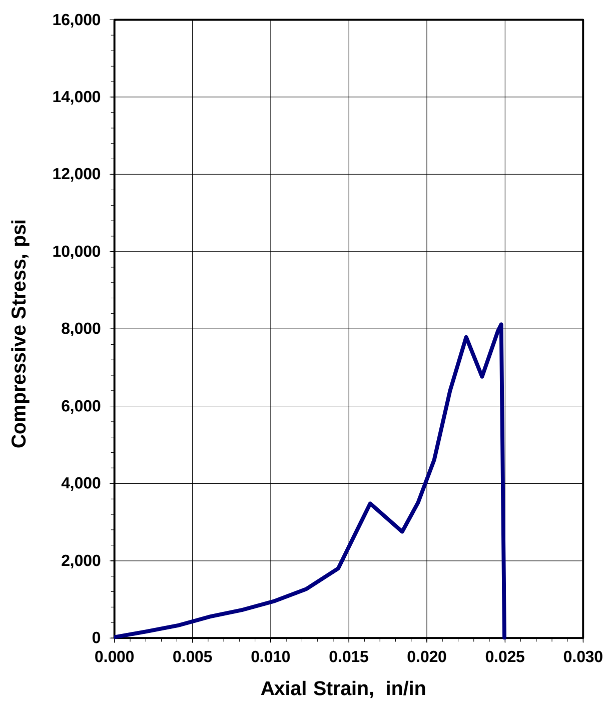
6350 Presidential Gatew.	9885 Rockside Road	4480 Lake Forest Drive
Columbus, OH 43231	Cleveland, OH 44125	Cincinnati, Ohio 45242
Phone (614) 823-4949	Phone (216) 573-0955	Phone (513) 769-6998

Project: FRA-70-12.68
Project No.: W-13-045
Date of Testing: 3/27/2015
Test Performed by: JH/TK

Rock Description: LIMESTONE: Brown and gray, moderately weathered, strong.

Boring No.:	B-108-9-15	Average Length:	4.887 in
Station / Offset:	5015+96.50, 15.7' Rt.	Average Diameter:	1.862 in
Sample No. / Depth:	RC-3 / 75.4 ft.	Length to diameter ratio:	2.625
Moisture condition:	As received	Cross Sectional Area:	2.722 in^2
		Failure Load:	21,200 lbs
Rate of Loading:	38.7 lbs/sec	Axial Strain at Failure:	0.0225 in/in
Testing Time:	571 sec	Stress:	7,788 psi
	(Rate 2-15 minutes to failure)		

Unconfined Compression Test



Before Testing

After Failure

REMARKS: Upper corner of sample sheared off at 3,500 psi. Based on nature of shear, this is not considered failure.

APPENDIX V

DRIVEN ANALYSIS OUTPUTS

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: C:\DOCUME~1\LEGACY\DESKTOP\1518A-RA.DVN
Project Name: FRA-71-1518A-RA-B-108-9 Project Date: 00/00/ 0
Project Client: GPD GROUP
Computed By: BRT
Project Manager: BRT

PILE INFORMATION

Pile Type: Pipe Pile - Closed End
Top of Pile: 0.00 ft
Diameter of Pile: 14.00 in

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	7.90 ft
	- Driving/Restrike	7.90 ft
	- Ultimate:	7.90 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	8.50 ft	17.00%	125.00 pcf	32.0/32.0	Nordlund
2	Cohesive	5.00 ft	33.00%	115.00 pcf	1500.00 psf	T-79 Steel
3	Cohesionless	7.50 ft	17.00%	125.00 pcf	34.0/34.0	Nordlund
4	Cohesionless	11.50 ft	0.00%	130.00 pcf	38.0/38.0	Nordlund
5	Cohesionless	5.00 ft	0.00%	135.00 pcf	41.0/41.0	Nordlund
6	Cohesionless	5.00 ft	0.00%	125.00 pcf	37.0/37.0	Nordlund
7	Cohesionless	10.00 ft	0.00%	130.00 pcf	40.0/40.0	Nordlund
8	Cohesionless	5.00 ft	17.00%	130.00 pcf	36.0/36.0	Nordlund
9	Cohesive	5.00 ft	17.00%	120.00 pcf	2250.00 psf	T-79 Steel
10	Cohesionless	5.00 ft	0.00%	125.00 pcf	34.0/34.0	Nordlund

RESTRIKE - SKIN FRICTION

Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesionless	0.62 psf	21.33	N/A	0.00 Kips
7.89 ft	Cohesionless	493.12 psf	21.33	N/A	6.11 Kips
7.91 ft	Cohesionless	987.81 psf	21.33	N/A	6.14 Kips
8.49 ft	Cohesionless	1005.97 psf	21.33	N/A	7.06 Kips
8.51 ft	Cohesive	N/A	N/A	1060.00 psf	7.11 Kips
13.49 ft	Cohesive	N/A	N/A	1072.24 psf	26.68 Kips
13.51 ft	Cohesionless	1288.37 psf	22.66	N/A	26.75 Kips
20.99 ft	Cohesionless	1522.50 psf	22.66	N/A	48.45 Kips
21.01 ft	Cohesionless	1757.90 psf	25.33	N/A	48.54 Kips
30.01 ft	Cohesionless	2062.10 psf	25.33	N/A	105.95 Kips
32.49 ft	Cohesionless	2145.92 psf	25.33	N/A	124.74 Kips
32.51 ft	Cohesionless	2535.32 psf	27.33	N/A	124.92 Kips
37.49 ft	Cohesionless	2716.10 psf	27.33	N/A	176.24 Kips
37.51 ft	Cohesionless	2898.27 psf	24.66	N/A	176.43 Kips
42.49 ft	Cohesionless	3054.15 psf	24.66	N/A	218.24 Kips
42.51 ft	Cohesionless	3211.30 psf	26.66	N/A	218.45 Kips
51.51 ft	Cohesionless	3515.50 psf	26.66	N/A	337.77 Kips
52.49 ft	Cohesionless	3548.62 psf	26.66	N/A	352.01 Kips
52.51 ft	Cohesionless	3887.30 psf	23.99	N/A	352.25 Kips
57.49 ft	Cohesionless	4055.62 psf	23.99	N/A	400.92 Kips
57.51 ft	Cohesive	N/A	N/A	1570.00 psf	401.08 Kips
62.49 ft	Cohesive	N/A	N/A	1570.00 psf	429.74 Kips
62.51 ft	Cohesionless	4513.27 psf	22.66	N/A	429.88 Kips
67.49 ft	Cohesionless	4669.15 psf	22.66	N/A	474.19 Kips

RESTRIKE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesionless	1.25 psf	40.40	35.28 Kips	0.03 Kips
7.89 ft	Cohesionless	986.25 psf	40.40	35.28 Kips	26.65 Kips
7.91 ft	Cohesionless	988.13 psf	40.40	35.28 Kips	26.70 Kips
8.49 ft	Cohesionless	1024.43 psf	40.40	35.28 Kips	27.68 Kips
8.51 ft	Cohesive	N/A	N/A	N/A	14.43 Kips
13.49 ft	Cohesive	N/A	N/A	N/A	14.43 Kips
13.51 ft	Cohesionless	1288.69 psf	55.60	78.59 Kips	50.77 Kips
20.99 ft	Cohesionless	1756.93 psf	55.60	78.59 Kips	69.21 Kips
21.01 ft	Cohesionless	1758.24 psf	110.40	287.14 Kips	149.82 Kips
30.01 ft	Cohesionless	2366.64 psf	110.40	287.14 Kips	201.66 Kips
32.49 ft	Cohesionless	2534.28 psf	110.40	287.14 Kips	215.95 Kips
32.51 ft	Cohesionless	2535.69 psf	202.00	538.27 Kips	417.24 Kips
37.49 ft	Cohesionless	2897.23 psf	202.00	538.27 Kips	476.73 Kips
37.51 ft	Cohesionless	2898.59 psf	91.20	220.26 Kips	199.94 Kips
42.49 ft	Cohesionless	3210.33 psf	91.20	220.26 Kips	220.26 Kips
42.51 ft	Cohesionless	3211.64 psf	160.00	446.42 Kips	411.99 Kips
51.51 ft	Cohesionless	3820.04 psf	160.00	446.42 Kips	446.42 Kips
52.49 ft	Cohesionless	3886.28 psf	160.00	446.42 Kips	446.42 Kips
52.51 ft	Cohesionless	3887.64 psf	77.60	162.06 Kips	162.06 Kips
57.49 ft	Cohesionless	4224.28 psf	77.60	162.06 Kips	162.06 Kips
57.51 ft	Cohesive	N/A	N/A	N/A	21.65 Kips
62.49 ft	Cohesive	N/A	N/A	N/A	21.65 Kips
62.51 ft	Cohesionless	4513.59 psf	55.60	78.59 Kips	78.59 Kips
67.49 ft	Cohesionless	4825.33 psf	55.60	78.59 Kips	78.59 Kips

RESTRIKE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.03 Kips	0.03 Kips
7.89 ft	6.11 Kips	26.65 Kips	32.75 Kips
7.91 ft	6.14 Kips	26.70 Kips	32.84 Kips
8.49 ft	7.06 Kips	27.68 Kips	34.73 Kips
8.51 ft	7.11 Kips	14.43 Kips	21.54 Kips
13.49 ft	26.68 Kips	14.43 Kips	41.11 Kips
13.51 ft	26.75 Kips	50.77 Kips	77.51 Kips
20.99 ft	48.45 Kips	69.21 Kips	117.66 Kips
21.01 ft	48.54 Kips	149.82 Kips	198.36 Kips
30.01 ft	105.95 Kips	201.66 Kips	307.61 Kips
32.49 ft	124.74 Kips	215.95 Kips	340.69 Kips
32.51 ft	124.92 Kips	417.24 Kips	542.16 Kips
37.49 ft	176.24 Kips	476.73 Kips	652.97 Kips
37.51 ft	176.43 Kips	199.94 Kips	376.37 Kips
42.49 ft	218.24 Kips	220.26 Kips	438.50 Kips
42.51 ft	218.45 Kips	411.99 Kips	630.44 Kips
51.51 ft	337.77 Kips	446.42 Kips	784.19 Kips
52.49 ft	352.01 Kips	446.42 Kips	798.43 Kips
52.51 ft	352.25 Kips	162.06 Kips	514.31 Kips
57.49 ft	400.92 Kips	162.06 Kips	562.98 Kips
57.51 ft	401.08 Kips	21.65 Kips	422.73 Kips
62.49 ft	429.74 Kips	21.65 Kips	451.39 Kips
62.51 ft	429.88 Kips	78.59 Kips	508.48 Kips
67.49 ft	474.19 Kips	78.59 Kips	552.78 Kips

DRIVING - SKIN FRICTION

Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesionless	0.62 psf	21.33	N/A	0.00 Kips
7.89 ft	Cohesionless	493.12 psf	21.33	N/A	5.07 Kips
7.91 ft	Cohesionless	987.81 psf	21.33	N/A	5.10 Kips
8.49 ft	Cohesionless	1005.97 psf	21.33	N/A	5.86 Kips
8.51 ft	Cohesive	N/A	N/A	1060.00 psf	5.89 Kips
13.49 ft	Cohesive	N/A	N/A	1072.24 psf	19.01 Kips
13.51 ft	Cohesionless	1288.37 psf	22.66	N/A	19.06 Kips
20.99 ft	Cohesionless	1522.50 psf	22.66	N/A	37.07 Kips
21.01 ft	Cohesionless	1757.90 psf	25.33	N/A	37.16 Kips
30.01 ft	Cohesionless	2062.10 psf	25.33	N/A	94.57 Kips
32.49 ft	Cohesionless	2145.92 psf	25.33	N/A	113.37 Kips
32.51 ft	Cohesionless	2535.32 psf	27.33	N/A	113.54 Kips
37.49 ft	Cohesionless	2716.10 psf	27.33	N/A	164.87 Kips
37.51 ft	Cohesionless	2898.27 psf	24.66	N/A	165.06 Kips
42.49 ft	Cohesionless	3054.15 psf	24.66	N/A	206.86 Kips
42.51 ft	Cohesionless	3211.30 psf	26.66	N/A	207.07 Kips
51.51 ft	Cohesionless	3515.50 psf	26.66	N/A	326.39 Kips
52.49 ft	Cohesionless	3548.62 psf	26.66	N/A	340.63 Kips
52.51 ft	Cohesionless	3887.30 psf	23.99	N/A	340.83 Kips
57.49 ft	Cohesionless	4055.62 psf	23.99	N/A	381.23 Kips
57.51 ft	Cohesive	N/A	N/A	1570.00 psf	381.36 Kips
62.49 ft	Cohesive	N/A	N/A	1570.00 psf	405.15 Kips
62.51 ft	Cohesionless	4513.27 psf	22.66	N/A	405.29 Kips
67.49 ft	Cohesionless	4669.15 psf	22.66	N/A	449.59 Kips

DRIVING - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesionless	1.25 psf	40.40	35.28 Kips	0.03 Kips
7.89 ft	Cohesionless	986.25 psf	40.40	35.28 Kips	26.65 Kips
7.91 ft	Cohesionless	988.13 psf	40.40	35.28 Kips	26.70 Kips
8.49 ft	Cohesionless	1024.43 psf	40.40	35.28 Kips	27.68 Kips
8.51 ft	Cohesive	N/A	N/A	N/A	14.43 Kips
13.49 ft	Cohesive	N/A	N/A	N/A	14.43 Kips
13.51 ft	Cohesionless	1288.69 psf	55.60	78.59 Kips	50.77 Kips
20.99 ft	Cohesionless	1756.93 psf	55.60	78.59 Kips	69.21 Kips
21.01 ft	Cohesionless	1758.24 psf	110.40	287.14 Kips	149.82 Kips
30.01 ft	Cohesionless	2366.64 psf	110.40	287.14 Kips	201.66 Kips
32.49 ft	Cohesionless	2534.28 psf	110.40	287.14 Kips	215.95 Kips
32.51 ft	Cohesionless	2535.69 psf	202.00	538.27 Kips	417.24 Kips
37.49 ft	Cohesionless	2897.23 psf	202.00	538.27 Kips	476.73 Kips
37.51 ft	Cohesionless	2898.59 psf	91.20	220.26 Kips	199.94 Kips
42.49 ft	Cohesionless	3210.33 psf	91.20	220.26 Kips	220.26 Kips
42.51 ft	Cohesionless	3211.64 psf	160.00	446.42 Kips	411.99 Kips
51.51 ft	Cohesionless	3820.04 psf	160.00	446.42 Kips	446.42 Kips
52.49 ft	Cohesionless	3886.28 psf	160.00	446.42 Kips	446.42 Kips
52.51 ft	Cohesionless	3887.64 psf	77.60	162.06 Kips	162.06 Kips
57.49 ft	Cohesionless	4224.28 psf	77.60	162.06 Kips	162.06 Kips
57.51 ft	Cohesive	N/A	N/A	N/A	21.65 Kips
62.49 ft	Cohesive	N/A	N/A	N/A	21.65 Kips
62.51 ft	Cohesionless	4513.59 psf	55.60	78.59 Kips	78.59 Kips
67.49 ft	Cohesionless	4825.33 psf	55.60	78.59 Kips	78.59 Kips

DRIVING - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.03 Kips	0.03 Kips
7.89 ft	5.07 Kips	26.65 Kips	31.72 Kips
7.91 ft	5.10 Kips	26.70 Kips	31.79 Kips
8.49 ft	5.86 Kips	27.68 Kips	33.53 Kips
8.51 ft	5.89 Kips	14.43 Kips	20.32 Kips
13.49 ft	19.01 Kips	14.43 Kips	33.44 Kips
13.51 ft	19.06 Kips	50.77 Kips	69.83 Kips
20.99 ft	37.07 Kips	69.21 Kips	106.29 Kips
21.01 ft	37.16 Kips	149.82 Kips	186.98 Kips
30.01 ft	94.57 Kips	201.66 Kips	296.23 Kips
32.49 ft	113.37 Kips	215.95 Kips	329.31 Kips
32.51 ft	113.54 Kips	417.24 Kips	530.78 Kips
37.49 ft	164.87 Kips	476.73 Kips	641.60 Kips
37.51 ft	165.06 Kips	199.94 Kips	364.99 Kips
42.49 ft	206.86 Kips	220.26 Kips	427.12 Kips
42.51 ft	207.07 Kips	411.99 Kips	619.07 Kips
51.51 ft	326.39 Kips	446.42 Kips	772.81 Kips
52.49 ft	340.63 Kips	446.42 Kips	787.05 Kips
52.51 ft	340.83 Kips	162.06 Kips	502.89 Kips
57.49 ft	381.23 Kips	162.06 Kips	543.29 Kips
57.51 ft	381.36 Kips	21.65 Kips	403.01 Kips
62.49 ft	405.15 Kips	21.65 Kips	426.79 Kips
62.51 ft	405.29 Kips	78.59 Kips	483.88 Kips
67.49 ft	449.59 Kips	78.59 Kips	528.19 Kips

ULTIMATE - SKIN FRICTION

Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesionless	0.62 psf	21.33	N/A	0.00 Kips
7.89 ft	Cohesionless	493.12 psf	21.33	N/A	6.11 Kips
7.91 ft	Cohesionless	987.81 psf	21.33	N/A	6.14 Kips
8.49 ft	Cohesionless	1005.97 psf	21.33	N/A	7.06 Kips
8.51 ft	Cohesive	N/A	N/A	1060.00 psf	7.11 Kips
13.49 ft	Cohesive	N/A	N/A	1072.24 psf	26.68 Kips
13.51 ft	Cohesionless	1288.37 psf	22.66	N/A	26.75 Kips
20.99 ft	Cohesionless	1522.50 psf	22.66	N/A	48.45 Kips
21.01 ft	Cohesionless	1757.90 psf	25.33	N/A	48.54 Kips
30.01 ft	Cohesionless	2062.10 psf	25.33	N/A	105.95 Kips
32.49 ft	Cohesionless	2145.92 psf	25.33	N/A	124.74 Kips
32.51 ft	Cohesionless	2535.32 psf	27.33	N/A	124.92 Kips
37.49 ft	Cohesionless	2716.10 psf	27.33	N/A	176.24 Kips
37.51 ft	Cohesionless	2898.27 psf	24.66	N/A	176.43 Kips
42.49 ft	Cohesionless	3054.15 psf	24.66	N/A	218.24 Kips
42.51 ft	Cohesionless	3211.30 psf	26.66	N/A	218.45 Kips
51.51 ft	Cohesionless	3515.50 psf	26.66	N/A	337.77 Kips
52.49 ft	Cohesionless	3548.62 psf	26.66	N/A	352.01 Kips
52.51 ft	Cohesionless	3887.30 psf	23.99	N/A	352.25 Kips
57.49 ft	Cohesionless	4055.62 psf	23.99	N/A	400.92 Kips
57.51 ft	Cohesive	N/A	N/A	1570.00 psf	401.08 Kips
62.49 ft	Cohesive	N/A	N/A	1570.00 psf	429.74 Kips
62.51 ft	Cohesionless	4513.27 psf	22.66	N/A	429.88 Kips
67.49 ft	Cohesionless	4669.15 psf	22.66	N/A	474.19 Kips

ULTIMATE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesionless	1.25 psf	40.40	35.28 Kips	0.03 Kips
7.89 ft	Cohesionless	986.25 psf	40.40	35.28 Kips	26.65 Kips
7.91 ft	Cohesionless	988.13 psf	40.40	35.28 Kips	26.70 Kips
8.49 ft	Cohesionless	1024.43 psf	40.40	35.28 Kips	27.68 Kips
8.51 ft	Cohesive	N/A	N/A	N/A	14.43 Kips
13.49 ft	Cohesive	N/A	N/A	N/A	14.43 Kips
13.51 ft	Cohesionless	1288.69 psf	55.60	78.59 Kips	50.77 Kips
20.99 ft	Cohesionless	1756.93 psf	55.60	78.59 Kips	69.21 Kips
21.01 ft	Cohesionless	1758.24 psf	110.40	287.14 Kips	149.82 Kips
30.01 ft	Cohesionless	2366.64 psf	110.40	287.14 Kips	201.66 Kips
32.49 ft	Cohesionless	2534.28 psf	110.40	287.14 Kips	215.95 Kips
32.51 ft	Cohesionless	2535.69 psf	202.00	538.27 Kips	417.24 Kips
37.49 ft	Cohesionless	2897.23 psf	202.00	538.27 Kips	476.73 Kips
37.51 ft	Cohesionless	2898.59 psf	91.20	220.26 Kips	199.94 Kips
42.49 ft	Cohesionless	3210.33 psf	91.20	220.26 Kips	220.26 Kips
42.51 ft	Cohesionless	3211.64 psf	160.00	446.42 Kips	411.99 Kips
51.51 ft	Cohesionless	3820.04 psf	160.00	446.42 Kips	446.42 Kips
52.49 ft	Cohesionless	3886.28 psf	160.00	446.42 Kips	446.42 Kips
52.51 ft	Cohesionless	3887.64 psf	77.60	162.06 Kips	162.06 Kips
57.49 ft	Cohesionless	4224.28 psf	77.60	162.06 Kips	162.06 Kips
57.51 ft	Cohesive	N/A	N/A	N/A	21.65 Kips
62.49 ft	Cohesive	N/A	N/A	N/A	21.65 Kips
62.51 ft	Cohesionless	4513.59 psf	55.60	78.59 Kips	78.59 Kips
67.49 ft	Cohesionless	4825.33 psf	55.60	78.59 Kips	78.59 Kips

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.03 Kips	0.03 Kips
7.89 ft	6.11 Kips	26.65 Kips	32.75 Kips
7.91 ft	6.14 Kips	26.70 Kips	32.84 Kips
8.49 ft	7.06 Kips	27.68 Kips	34.73 Kips
8.51 ft	7.11 Kips	14.43 Kips	21.54 Kips
13.49 ft	26.68 Kips	14.43 Kips	41.11 Kips
13.51 ft	26.75 Kips	50.77 Kips	77.51 Kips
20.99 ft	48.45 Kips	69.21 Kips	117.66 Kips
21.01 ft	48.54 Kips	149.82 Kips	198.36 Kips
30.01 ft	105.95 Kips	201.66 Kips	307.61 Kips
32.49 ft	124.74 Kips	215.95 Kips	340.69 Kips
32.51 ft	124.92 Kips	417.24 Kips	542.16 Kips
37.49 ft	176.24 Kips	476.73 Kips	652.97 Kips
37.51 ft	176.43 Kips	199.94 Kips	376.37 Kips
42.49 ft	218.24 Kips	220.26 Kips	438.50 Kips
42.51 ft	218.45 Kips	411.99 Kips	630.44 Kips
51.51 ft	337.77 Kips	446.42 Kips	784.19 Kips
52.49 ft	352.01 Kips	446.42 Kips	798.43 Kips
52.51 ft	352.25 Kips	162.06 Kips	514.31 Kips
57.49 ft	400.92 Kips	162.06 Kips	562.98 Kips
57.51 ft	401.08 Kips	21.65 Kips	422.73 Kips
62.49 ft	429.74 Kips	21.65 Kips	451.39 Kips
62.51 ft	429.88 Kips	78.59 Kips	508.48 Kips
67.49 ft	474.19 Kips	78.59 Kips	552.78 Kips

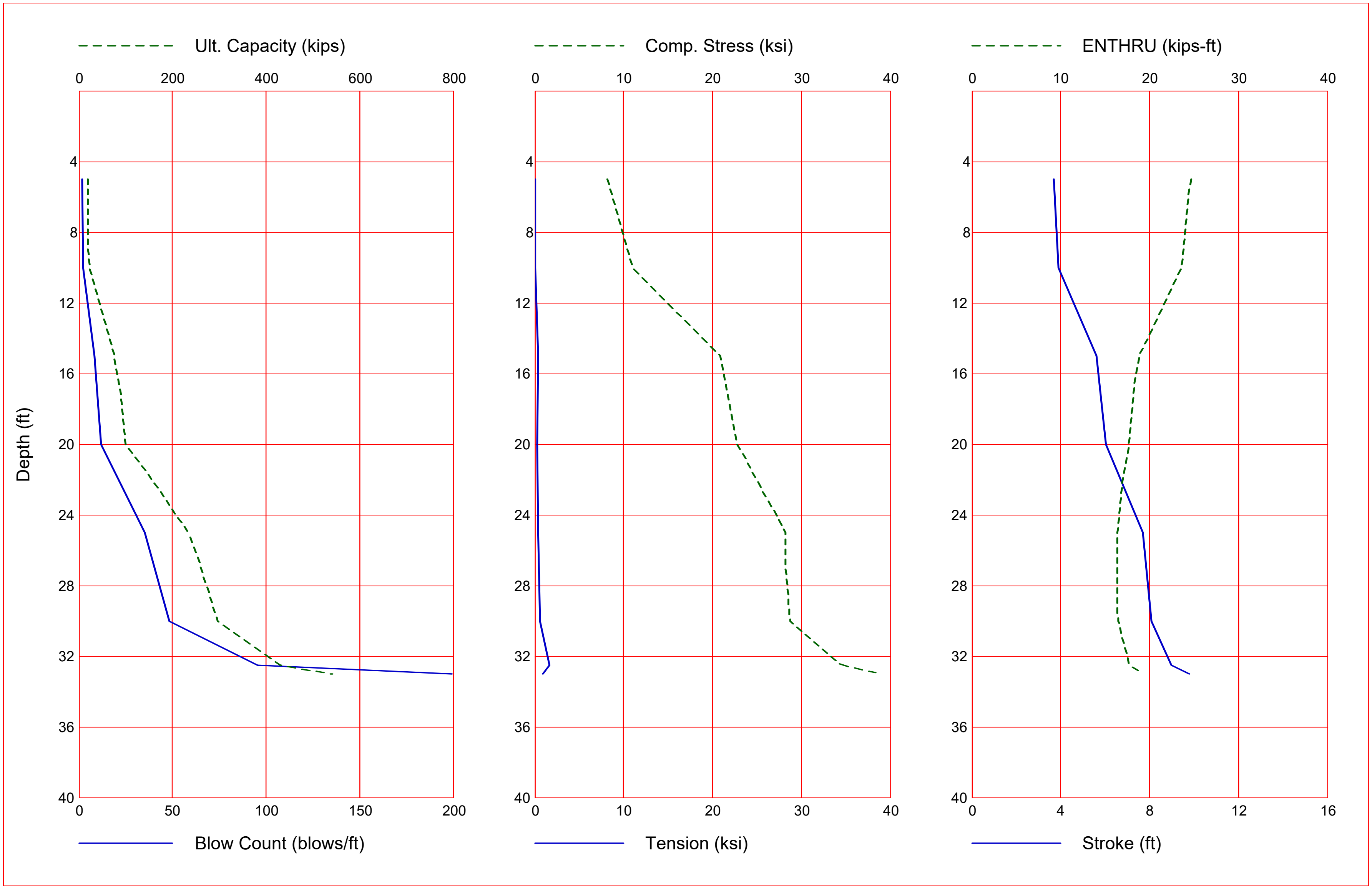
APPENDIX VI

GRLWEAP DRIVEABILITY ANALYSIS OUTPUTS

Resource International Inc
FRA-71-1518A - RA - B-108-9-15 - 14" CIP

Jul 06 2018
GRLWEAP Version 2010

Gain/Loss 3 at Shaft and Toe 0.670 / 1.000



Gain/Loss 3 at Shaft and Toe 0.670 / 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
5.0	18.9	2.0	16.9	1.6	8.178	0.000	3.68	24.7
10.0	24.2	9.8	14.4	2.1	11.039	0.000	3.91	23.6
15.0	76.6	22.1	54.4	8.6	20.880	-0.330	5.60	18.8
20.0	101.1	34.3	66.8	12.0	22.739	-0.313	6.04	17.7
25.0	233.3	60.5	172.8	35.4	28.165	-0.408	7.72	16.4
30.0	296.1	94.5	201.6	48.6	28.797	-0.552	8.07	16.5
32.5	430.1	113.5	316.6	95.6	34.421	-1.603	9.00	17.8
33.0	541.4	118.3	423.1	199.0	38.904	-0.940	9.78	19.3

Total Continuous Driving Time 16.00 minutes; Total Number of Blows 673

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-13-045 FRA-70-13.54 PROJECT 4A\ANALYSIS\FRA-71-1518A AND RETAINING WALL
4W11\DRIVEABILITY\14 IN CIP\B-108-9-15 - RA - 14 IN CIP.GWW
 Hammer File: C:\ProgramData\PDI\GRLWEAP\2010\Resource\HAMMER2003.GW
 Hammer File Version:  2003 (2/22/2013)

                       Input File Contents
     FRA-71-1518A - RA - B-108-9-15 - 14" CIP
 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    14.000            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
    33.000     18.43   29000.0   492.000     3.670         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      0.03    0.10    0.12    0.10    0.15    1.21     0.00     0.0
  7.89    0.42     26.65    0.10    0.12    0.10    0.15    1.21     0.00     0.0
  7.91    0.42     26.70    0.10    0.12    0.10    0.15    1.21     0.00     0.0
  8.49    0.44     27.68    0.10    0.12    0.10    0.15    1.21     0.00     0.0
  8.51    1.06     14.43    0.10    0.12    0.20    0.15    1.49     0.00     0.0
 13.49    1.07     14.43    0.10    0.12    0.20    0.15    1.49     0.00     0.0
 13.51    0.67     50.77    0.10    0.12    0.10    0.15    1.21     0.00     0.0
 20.99    0.91     69.21    0.10    0.12    0.10    0.15    1.21     0.00     0.0
 21.01    1.48    149.82    0.10    0.12    0.10    0.15    1.00     0.00     0.0
 30.01    2.00    201.66    0.10    0.12    0.05    0.15    1.00     0.00     0.0
 32.49    2.14    215.95    0.10    0.12    0.05    0.15    1.00     0.00     0.0
 32.51    2.62    417.24    0.10    0.12    0.05    0.15    1.00     0.00     0.0
 33.00    2.66    423.09    0.10    0.12    0.05    0.15    1.00     0.00     0.0
Gain/Loss factors: shaft and toe
 0.60400   0.63700   0.67000   0.70300   0.73600
 1.00000   1.00000   1.00000   1.00000   1.00000
   Dpth         L      Wait      Strk      Pmx%      Eff.      Stff       CoR
   5.00      0.00      0.00     0.000     0.000     0.000     0.000     0.000
  10.00      0.00      0.00     0.000     0.000     0.000     0.000     0.000
  15.00      0.00      0.00     0.000     0.000     0.000     0.000     0.000
  20.00      0.00      0.00     0.000     0.000     0.000     0.000     0.000
  25.00      0.00      0.00     0.000     0.000     0.000     0.000     0.000
  30.00      0.00      0.00     0.000     0.000     0.000     0.000     0.000
  32.50      0.00      0.00     0.000     0.000     0.000     0.000     0.000
  33.00      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-71-1518A - RA - B-108-9-15 - 14" CIP

```
       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     13496.7
```

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-71-1518A - RA - B-108-9-15 - 14" CIP 07/06/2018
Resource International Inc GRLWEAP Version 2010

Depth	(ft)	5.0			
Shaft Gain/Loss Factor		0.604	Toe Gain/Loss Factor		1.000

PILE PROFILE:

Toe Area	(in2)	144.000	Pile Type	Unknown
Pile Size	(inch)	14.000		

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	18.43	29000.	492.0	3.7	0	16524.	32.3
33.0	18.43	29000.	492.0	3.7	0	16524.	32.3

Wave Travel Time 2L/c (ms) 3.994

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	0.208	13497	0.010	0.000	0.85	0.0	0.000	0.100	3.30	3.7	18.4
2	0.208	13497	0.000	0.000	1.00	0.0	0.000	0.100	6.60	3.7	18.4
9	0.208	13497	0.000	0.000	1.00	0.2	0.100	0.100	29.70	3.7	18.4
10	0.208	13497	0.000	0.000	1.00	1.7	0.100	0.100	33.00	3.7	18.4
Toe						16.9	0.150	0.117			

2.078 kips total unreduced pile weight (g= 32.17 ft/s2)
2.078 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)	0.647
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	
5.00	10.81	1.00	0.800

FRA-71-1518A - RA - B-108-9-15 - 14" CIP 07/06/2018
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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	1.6	3.67	3.65	0.00	1	0	8.12	1	2	24.7	61.8
18.9	1.6	3.68	3.66	0.00	1	0	8.13	1	2	24.7	61.8
18.9	1.6	3.68	3.66	0.00	1	0	8.18	1	2	24.7	61.8
19.0	1.6	3.68	3.66	0.00	1	0	8.17	1	2	24.7	61.8
19.0	1.6	3.69	3.66	0.00	1	0	8.26	1	2	24.7	61.7

1 0 10.81000 11.86000

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Depth	(ft)	10.0			
Shaft Gain/Loss Factor		0.604	Toe Gain/Loss Factor		1.000

PILE PROFILE:

Toe Area	(in2)	144.000	Pile Type	Unknown

B-108-9-15 - RA - 14 IN CIP

Pile Size (inch) 14.000

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	18.43	29000.	492.0	3.7	0	16524.	32.3
33.0	18.43	29000.	492.0	3.7	0	16524.	32.3

Wave Travel Time 2L/c (ms) 3.994

Pile and Soil Model — Total Capacity Rut (kips) 23.6

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	0.208	13497	0.010	0.000	0.85	0.0	0.000	0.100	3.30	3.7	18.4
2	0.208	13497	0.000	0.000	1.00	0.0	0.000	0.100	6.60	3.7	18.4
7	0.208	13497	0.000	0.000	1.00	0.0	0.100	0.100	23.10	3.7	18.4
8	0.208	13497	0.000	0.000	1.00	0.9	0.100	0.100	26.40	3.7	18.4
9	0.208	13497	0.000	0.000	1.00	2.6	0.100	0.100	29.70	3.7	18.4
10	0.208	13497	0.000	0.000	1.00	5.7	0.169	0.100	33.00	3.7	18.4
Toe						14.4	0.150	0.117			

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

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
10.00	10.81	1.00	0.800

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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
23.6	2.1	3.88	3.90	0.00	1	0	10.70	1	2	23.7	60.0
23.9	2.1	3.90	3.92	0.00	1	0	10.87	1	2	23.7	59.9
24.2	2.1	3.91	3.93	0.00	1	0	11.04	1	2	23.6	59.8
24.5	2.2	3.93	3.95	0.00	1	0	11.16	1	2	23.5	59.7
24.8	2.2	3.94	3.96	0.00	1	0	11.30	1	2	23.5	59.5

1 0 10.81000 11.86000

Depth (ft) 15.0
Shaft Gain/Loss Factor 0.604 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 14.000

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	18.43	29000.	492.0	3.7	0	16524.	32.3
33.0	18.43	29000.	492.0	3.7	0	16524.	32.3

Wave Travel Time 2L/c (ms) 3.994

Pile and Soil Model — Total Capacity Rut (kips) 74.9

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	0.208	13497	0.010	0.000	0.85	0.0	0.000	0.100	3.30	3.7	18.4
2	0.208	13497	0.000	0.000	1.00	0.0	0.000	0.100	6.60	3.7	18.4
6	0.208	13497	0.000	0.000	1.00	0.3	0.100	0.100	19.80	3.7	18.4
7	0.208	13497	0.000	0.000	1.00	1.8	0.100	0.100	23.10	3.7	18.4
8	0.208	13497	0.000	0.000	1.00	3.5	0.100	0.100	26.40	3.7	18.4
9	0.208	13497	0.000	0.000	1.00	7.7	0.199	0.100	29.70	3.7	18.4
10	0.208	13497	0.000	0.000	1.00	7.3	0.165	0.100	33.00	3.7	18.4
Toe						54.4	0.150	0.117			

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

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	

15.00	10.81	1.00	0.800

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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
74.9	8.3	5.56	5.55	-0.29	7	46	20.71	7	3	18.9	50.0
75.8	8.5	5.53	5.58	-0.32	7	45	20.61	7	3	18.6	50.0
76.6	8.6	5.60	5.59	-0.33	7	45	20.88	7	3	18.8	49.8
77.4	8.7	5.62	5.61	-0.34	7	45	20.95	7	3	18.8	49.7
78.2	8.9	5.59	5.64	-0.36	7	45	20.89	7	3	18.5	49.7

1 0 10.81000 11.86000

FRA-71-1518A - RA - B-108-9-15 - 14" CIP 07/06/2018
Resource International Inc GRLWEAP Version 2010

Depth (ft) 20.0
Shaft Gain/Loss Factor 0.604 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 14.000

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	18.43	29000.	492.0	3.7	0	16524.	32.3
33.0	18.43	29000.	492.0	3.7	0	16524.	32.3

Wave Travel Time 2L/c (ms) 3.994

Pile and Soil Model Total Capacity Rut (kips) 98.9

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	0.208	13497	0.010	0.000	0.85	0.0	0.000	0.100	3.30	3.7	18.4
2	0.208	13497	0.000	0.000	1.00	0.0	0.000	0.100	6.60	3.7	18.4
4	0.208	13497	0.000	0.000	1.00	0.0	0.100	0.100	13.20	3.7	18.4
5	0.208	13497	0.000	0.000	1.00	0.9	0.100	0.100	16.50	3.7	18.4
6	0.208	13497	0.000	0.000	1.00	2.7	0.100	0.100	19.80	3.7	18.4
7	0.208	13497	0.000	0.000	1.00	5.8	0.171	0.100	23.10	3.7	18.4
8	0.208	13497	0.000	0.000	1.00	7.8	0.200	0.100	26.40	3.7	18.4
9	0.208	13497	0.000	0.000	1.00	7.0	0.104	0.100	29.70	3.7	18.4
10	0.208	13497	0.000	0.000	1.00	8.0	0.100	0.100	33.00	3.7	18.4
Toe						66.8	0.150	0.117			

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

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
20.00	10.81	1.00	0.800

FRA-71-1518A - RA - B-108-9-15 - 14" CIP 07/06/2018
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
98.9	11.6	6.00	6.02	-0.31	6	37	22.55	7	3	17.8	48.1
100.0	11.8	6.02	6.04	-0.31	6	37	22.64	7	3	17.7	48.0
101.1	12.0	6.04	6.07	-0.31	6	37	22.74	7	3	17.7	47.9
102.2	12.1	6.06	6.08	-0.31	6	37	22.86	7	3	17.7	47.8
103.3	12.3	6.09	6.10	-0.31	6	37	22.97	7	3	17.7	47.7

1 0 10.81000 11.86000

FRA-71-1518A - RA - B-108-9-15 - 14" CIP 07/06/2018
Resource International Inc GRLWEAP Version 2010

Depth (ft) 25.0
Shaft Gain/Loss Factor 0.604 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 14.000

B-108-9-15 - RA - 14 IN CIP

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	18.43	29000.	492.0	3.7	0	16524.	32.3
33.0	18.43	29000.	492.0	3.7	0	16524.	32.3

Wave Travel Time 2L/c (ms) 3.994

Pile and Soil Model — Total Capacity Rut (kips) 231.0

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	0.208	13497	0.010	0.000	0.85	0.0	0.000	0.100	3.30	3.7	18.4
2	0.208	13497	0.000	0.000	1.00	0.0	0.000	0.100	6.60	3.7	18.4
3	0.208	13497	0.000	0.000	1.00	0.3	0.100	0.100	9.90	3.7	18.4
4	0.208	13497	0.000	0.000	1.00	1.8	0.100	0.100	13.20	3.7	18.4
5	0.208	13497	0.000	0.000	1.00	3.5	0.100	0.100	16.50	3.7	18.4
6	0.208	13497	0.000	0.000	1.00	7.8	0.200	0.100	19.80	3.7	18.4
7	0.208	13497	0.000	0.000	1.00	7.3	0.162	0.100	23.10	3.7	18.4
8	0.208	13497	0.000	0.000	1.00	7.5	0.100	0.100	26.40	3.7	18.4
9	0.208	13497	0.000	0.000	1.00	10.5	0.099	0.100	29.70	3.7	18.4
10	0.208	13497	0.000	0.000	1.00	19.6	0.087	0.100	33.00	3.7	18.4
Toe						172.8	0.150	0.117			

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

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
25.00	10.81	1.00	0.800

FRA-71-1518A - RA - B-108-9-15 - 14" CIP 07/06/2018
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
231.0	34.8	7.70	7.70	-0.41	5	41	28.04	6	3	16.5	42.6
232.2	35.1	7.70	7.71	-0.41	5	40	28.08	6	3	16.4	42.6
233.3	35.4	7.72	7.72	-0.41	5	40	28.17	6	3	16.4	42.5
234.4	35.5	7.73	7.72	-0.42	5	40	28.22	6	3	16.5	42.5
235.6	35.9	7.73	7.73	-0.41	5	40	28.27	6	3	16.4	42.5

1 0 10.81000 11.86000

Depth (ft) 30.0
Shaft Gain/Loss Factor 0.604 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 14.000

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	18.43	29000.	492.0	3.7	0	16524.	32.3
33.0	18.43	29000.	492.0	3.7	0	16524.	32.3

Wave Travel Time 2L/c (ms) 3.994

Pile and Soil Model — Total Capacity Rut (kips) 293.9

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	0.208	13497	0.010	0.000	0.85	0.0	0.100	0.100	3.30	3.7	18.4
2	0.208	13497	0.000	0.000	1.00	1.0	0.100	0.100	6.60	3.7	18.4
3	0.208	13497	0.000	0.000	1.00	2.7	0.100	0.100	9.90	3.7	18.4
4	0.208	13497	0.000	0.000	1.00	5.9	0.173	0.100	13.20	3.7	18.4
5	0.208	13497	0.000	0.000	1.00	7.8	0.200	0.100	16.50	3.7	18.4
6	0.208	13497	0.000	0.000	1.00	7.0	0.100	0.100	19.80	3.7	18.4
7	0.208	13497	0.000	0.000	1.00	8.0	0.100	0.100	23.10	3.7	18.4
8	0.208	13497	0.000	0.000	1.00	16.0	0.095	0.100	26.40	3.7	18.4
9	0.208	13497	0.000	0.000	1.00	20.8	0.078	0.100	29.70	3.7	18.4
10	0.208	13497	0.000	0.000	1.00	23.0	0.059	0.100	33.00	3.7	18.4
Toe						201.6	0.150	0.117			

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

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


    Depth    Stroke  Pressure    Efficy
       ft        ft     Ratio
    30.00     10.81      1.00     0.800
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FRA-71-1518A - RA - B-108-9-15 - 14" CIP                             07/06/2018
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
  293.9    47.9   8.05   8.09   -0.55   4  35    28.70   4   2   16.5    41.6
  295.0    48.4   8.06   8.10   -0.55   4  35    28.74   4   2   16.5    41.6
  296.1    48.6   8.07   8.10   -0.55   4  34    28.80   4   2   16.5    41.6
  297.3    49.2   8.07   8.12   -0.55   4  34    28.84   4   2   16.5    41.6
  298.4    49.4   8.08   8.12   -0.55   4  34    28.90   4   2   16.5    41.5
        1          0   10.81000       11.86000
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FRA-71-1518A - RA - B-108-9-15 - 14" CIP                             07/06/2018
Resource International Inc                                 GRLWEAP Version 2010

 Depth                (ft)      32.5
 Shaft Gain/Loss Factor         0.604  Toe Gain/Loss Factor          1.000

 PILE PROFILE:
 Toe Area            (in2)   144.000  Pile Type                   Unknown
 Pile Size          (inch)    14.000

  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    18.43   29000.    492.0      3.7        0    16524.     32.3
     33.0    18.43   29000.    492.0      3.7        0    16524.     32.3

 Wave Travel Time 2L/c (ms)     3.994

       Pile and Soil Model           Total Capacity Rut  (kips)     427.8
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  0.208    13497 0.010 0.000 0.85     0.6   0.100 0.100   3.30   3.7    18.4
  2  0.208    13497 0.000 0.000 1.00     2.3   0.100 0.100   6.60   3.7    18.4
  3  0.208    13497 0.000 0.000 1.00     4.8   0.150 0.100   9.90   3.7    18.4
  4  0.208    13497 0.000 0.000 1.00     7.8   0.200 0.100  13.20   3.7    18.4
  5  0.208    13497 0.000 0.000 1.00     7.1   0.132 0.100  16.50   3.7    18.4
  6  0.208    13497 0.000 0.000 1.00     7.8   0.100 0.100  19.80   3.7    18.4
  7  0.208    13497 0.000 0.000 1.00    13.4   0.097 0.100  23.10   3.7    18.4
  8  0.208    13497 0.000 0.000 1.00    20.2   0.082 0.100  26.40   3.7    18.4
  9  0.208    13497 0.000 0.000 1.00    22.5   0.064 0.100  29.70   3.7    18.4
 10  0.208    13497 0.000 0.000 1.00    24.8   0.051 0.100  33.00   3.7    18.4
Toe                                    316.6   0.150 0.117

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


    Depth    Stroke  Pressure    Efficy
       ft        ft     Ratio
    32.50     10.81      1.00     0.800
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FRA-71-1518A - RA - B-108-9-15 - 14" CIP                             07/06/2018
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
  427.8    93.9   8.99   8.93   -1.74   7  31    34.68  10   4   17.8    39.5
  428.9    94.2   8.99   8.92   -1.69   7  31    34.56  10   4   17.9    39.6
  430.1    95.6   9.00   8.93   -1.60   7  31    34.42  10   4   17.8    39.5
  431.2    97.6   8.99   8.94   -1.50   7  31    34.31  10   4   17.7    39.5
  432.3    97.9   9.00   8.94   -1.46   6  31    34.18  10   4   17.8    39.5
        1          0   10.81000       11.86000
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FRA-71-1518A - RA - B-108-9-15 - 14" CIP                             07/06/2018
Resource International Inc                                 GRLWEAP Version 2010

 Depth                (ft)      33.0
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Shaft Gain/Loss Factor 0.604 Toe Gain/Loss Factor 1.000

PILE PROFILE:

Toe Area	(in2)	144.000	Pile Type	Unknown
Pile Size	(inch)	14.000		

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	18.43	29000.	492.0	3.7	0	16524.	32.3
33.0	18.43	29000.	492.0	3.7	0	16524.	32.3

Wave Travel Time 2L/c (ms) 3.994

Pile and Soil Model Total Capacity Rut (kips) 539.2

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	0.208	13497	0.010	0.000	0.85	0.8	0.100	0.100	3.30	3.7	18.4
2	0.208	13497	0.000	0.000	1.00	2.6	0.100	0.100	6.60	3.7	18.4
3	0.208	13497	0.000	0.000	1.00	5.5	0.166	0.100	9.90	3.7	18.4
4	0.208	13497	0.000	0.000	1.00	7.8	0.200	0.100	13.20	3.7	18.4
5	0.208	13497	0.000	0.000	1.00	7.0	0.113	0.100	16.50	3.7	18.4
6	0.208	13497	0.000	0.000	1.00	7.9	0.100	0.100	19.80	3.7	18.4
7	0.208	13497	0.000	0.000	1.00	15.0	0.096	0.100	23.10	3.7	18.4
8	0.208	13497	0.000	0.000	1.00	20.6	0.079	0.100	26.40	3.7	18.4
9	0.208	13497	0.000	0.000	1.00	22.8	0.061	0.100	29.70	3.7	18.4
10	0.208	13497	0.000	0.000	1.00	26.0	0.050	0.100	33.00	3.7	18.4
Toe						423.1	0.150	0.117			

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

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
33.00	10.81	1.00	0.800

FRA-71-1518A - RA - B-108-9-15 - 14" CIP 07/06/2018
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
539.2	189.5	9.79	9.76	-1.09	4	29	39.23	10	4	19.3	37.9
540.3	195.2	9.79	9.77	-0.99	4	29	39.05	10	4	19.3	37.9
541.4	199.0	9.78	9.77	-0.94	4	29	38.90	10	4	19.3	37.9
542.6	202.8	9.78	9.77	-0.88	4	29	38.77	10	4	19.3	37.9
543.7	206.7	9.78	9.77	-0.85	4	29	38.61	10	4	19.3	37.9

FRA-71-1518A - RA - B-108-9-15 - 14" CIP 07/06/2018
Resource International Inc GRLWEAP Version 2010

SUMMARY OVER DEPTHS

G/L at Shaft and Toe: 0.604 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
5.0	18.8	1.9	16.9	1.6	8.120	0.000	3.67	24.7
10.0	23.6	9.2	14.4	2.1	10.699	0.000	3.88	23.7
15.0	74.9	20.5	54.4	8.3	20.706	-0.295	5.56	18.9
20.0	98.9	32.2	66.8	11.6	22.552	-0.313	6.00	17.8
25.0	231.0	58.2	172.8	34.8	28.036	-0.413	7.70	16.5
30.0	293.9	92.3	201.6	47.9	28.699	-0.551	8.05	16.5
32.5	427.8	111.2	316.6	93.9	34.678	-1.740	8.99	17.8
33.0	539.2	116.1	423.1	189.5	39.225	-1.094	9.79	19.3

Total Driving Time 16 minutes; Total No. of Blows 660

G/L at Shaft and Toe: 0.637 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
5.0	18.9	2.0	16.9	1.6	8.133	0.000	3.68	24.7
10.0	23.9	9.5	14.4	2.1	10.870	0.000	3.90	23.7
15.0	75.8	21.3	54.4	8.5	20.609	-0.315	5.53	18.6
20.0	100.0	33.2	66.8	11.8	22.636	-0.314	6.02	17.7
25.0	232.2	59.4	172.8	35.1	28.080	-0.412	7.70	16.4
30.0	295.0	93.4	201.6	48.4	28.739	-0.547	8.06	16.5

32.5	428.9	112.4	316.6	94.2	34.560	-1.691	8.99	17.9
33.0	540.3	117.2	423.1	195.2	39.051	-0.989	9.79	19.3

Total Driving Time 16 minutes; Total No. of Blows 666

FRA-71-1518A - RA - B-108-9-15 - 14" CIP 07/06/2018
Resource International Inc GRLWEAP Version 2010

SUMMARY OVER DEPTHS

G/L at Shaft and Toe: 0.670 1.000

Depth ft	Rut kips	Frictn kips	End Bg kips	Bl Ct bl/ft	Com Str ksi	Ten Str ksi	Stroke ft	ENTHRU kip-ft
5.0	18.9	2.0	16.9	1.6	8.178	0.000	3.68	24.7
10.0	24.2	9.8	14.4	2.1	11.039	0.000	3.91	23.6
15.0	76.6	22.1	54.4	8.6	20.880	-0.330	5.60	18.8
20.0	101.1	34.3	66.8	12.0	22.739	-0.313	6.04	17.7
25.0	233.3	60.5	172.8	35.4	28.165	-0.408	7.72	16.4
30.0	296.1	94.5	201.6	48.6	28.797	-0.552	8.07	16.5
32.5	430.1	113.5	316.6	95.6	34.421	-1.603	9.00	17.8
33.0	541.4	118.3	423.1	199.0	38.904	-0.940	9.78	19.3

Total Driving Time 16 minutes; Total No. of Blows 673

G/L at Shaft and Toe: 0.703 1.000

Depth ft	Rut kips	Frictn kips	End Bg kips	Bl Ct bl/ft	Com Str ksi	Ten Str ksi	Stroke ft	ENTHRU kip-ft
5.0	19.0	2.1	16.9	1.6	8.170	0.000	3.68	24.7
10.0	24.5	10.1	14.4	2.2	11.161	0.000	3.93	23.5
15.0	77.4	23.0	54.4	8.7	20.954	-0.344	5.62	18.8
20.0	102.2	35.4	66.8	12.1	22.862	-0.312	6.06	17.7
25.0	234.4	61.6	172.8	35.5	28.217	-0.417	7.73	16.5
30.0	297.3	95.7	201.6	49.2	28.843	-0.546	8.07	16.5
32.5	431.2	114.6	316.6	97.6	34.308	-1.497	8.99	17.7
33.0	542.6	119.5	423.1	202.8	38.773	-0.878	9.78	19.3

Total Driving Time 16 minutes; Total No. of Blows 682

FRA-71-1518A - RA - B-108-9-15 - 14" CIP 07/06/2018
Resource International Inc GRLWEAP Version 2010

SUMMARY OVER DEPTHS

G/L at Shaft and Toe: 0.736 1.000

Depth ft	Rut kips	Frictn kips	End Bg kips	Bl Ct bl/ft	Com Str ksi	Ten Str ksi	Stroke ft	ENTHRU kip-ft
5.0	19.0	2.1	16.9	1.6	8.262	0.000	3.69	24.7
10.0	24.8	10.4	14.4	2.2	11.304	0.000	3.94	23.5
15.0	78.2	23.8	54.4	8.9	20.892	-0.356	5.59	18.5
20.0	103.3	36.5	66.8	12.3	22.968	-0.306	6.09	17.7
25.0	235.6	62.8	172.8	35.9	28.270	-0.414	7.73	16.4
30.0	298.4	96.8	201.6	49.4	28.898	-0.550	8.08	16.5
32.5	432.3	115.8	316.6	97.9	34.179	-1.457	9.00	17.8
33.0	543.7	120.6	423.1	206.7	38.611	-0.848	9.78	19.3

Total Driving Time 16 minutes; Total No. of Blows 688

FRA-71-1518A - RA - B-108-9-15 - 14" CIP 07/06/2018
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
5.00	33.00	0.00	10.81	1.00	0.80	1.00	1.00
10.00	33.00	0.00	10.81	1.00	0.80	1.00	1.00
15.00	33.00	0.00	10.81	1.00	0.80	1.00	1.00
20.00	33.00	0.00	10.81	1.00	0.80	1.00	1.00
25.00	33.00	0.00	10.81	1.00	0.80	1.00	1.00
30.00	33.00	0.00	10.81	1.00	0.80	1.00	1.00
32.50	33.00	0.00	10.81	1.00	0.80	1.00	1.00
33.00	33.00	0.00	10.81	1.00	0.80	1.00	1.00

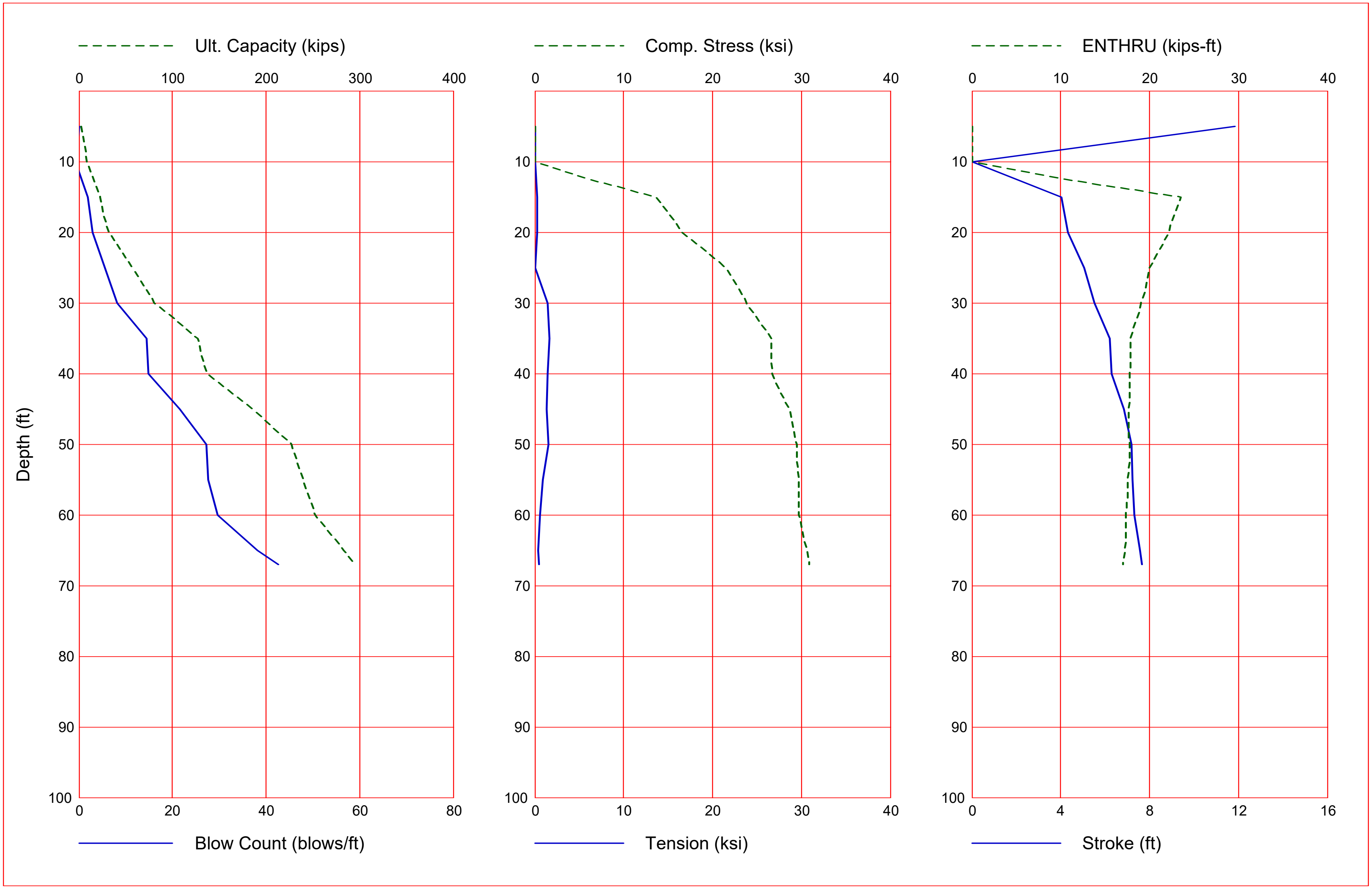
Soil Layer Resistance Values

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	0.03	0.100	0.117	0.100	0.150	0.515	0.000	0.000
7.89	0.42	26.65	0.100	0.117	0.100	0.150	0.515	0.000	0.000
7.91	0.42	26.70	0.100	0.117	0.100	0.150	0.515	0.000	0.000
8.49	0.44	27.68	0.100	0.117	0.100	0.150	0.515	0.000	0.000
8.51	1.06	14.43	0.100	0.117	0.200	0.150	1.000	0.000	0.000
13.49	1.07	14.43	0.100	0.117	0.200	0.150	1.000	0.000	0.000
13.51	0.67	50.77	0.100	0.117	0.100	0.150	0.515	0.000	0.000
20.99	0.91	69.21	0.100	0.117	0.100	0.150	0.515	0.000	0.000
21.01	1.48	149.82	0.100	0.117	0.100	0.150	0.000	0.000	0.000
30.01	2.00	201.66	0.100	0.117	0.050	0.150	0.000	0.000	0.000
32.49	2.14	215.95	0.100	0.117	0.050	0.150	0.000	0.000	0.000
32.51	2.62	417.24	0.100	0.117	0.050	0.150	0.000	0.000	0.000
33.00	2.66	423.09	0.100	0.117	0.050	0.150	0.000	0.000	0.000

Resource International Inc
FRA-71-1518A - RA - B-108-9-15 - HP10x42

Jul 06 2018
GRLWEAP Version 2010

Gain/Loss 3 at Shaft and Toe 0.670 / 1.000



Gain/Loss 3 at Shaft and Toe 0.670 / 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
5.0	2.8	1.5	1.4	0.0	0.000	0.000	11.86	0.0
10.0	8.9	7.8	1.2	-1.0	0.000	0.000	0.00	0.0
15.0	22.8	18.4	4.4	2.0	13.713	-0.266	4.05	23.5
20.0	32.1	26.8	5.3	2.9	16.600	-0.216	4.35	22.1
25.0	57.1	43.2	13.9	5.5	21.471	0.000	5.05	20.0
30.0	80.5	64.3	16.2	8.2	23.815	-1.403	5.52	19.0
35.0	127.0	91.0	36.0	14.4	26.570	-1.647	6.22	17.9
40.0	136.8	119.9	16.9	14.9	26.686	-1.467	6.28	17.8
45.0	186.1	152.1	34.0	21.5	28.670	-1.321	6.85	17.6
50.0	227.6	192.1	35.5	27.2	29.474	-1.594	7.19	17.8
55.0	239.6	226.5	13.1	27.6	29.682	-0.916	7.22	17.5
60.0	252.6	250.9	1.7	29.6	29.722	-0.606	7.33	17.3
65.0	283.1	276.7	6.3	38.1	30.642	-0.426	7.59	17.2
67.0	295.6	289.2	6.3	42.6	30.908	-0.519	7.65	17.0

Total Continuous Driving Time 21.00 minutes; Total Number of Blows 945

GRLWEAP - Version 2010
WAVE EQUATION ANALYSIS OF PILE FOUNDATIONS

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-13-045 FRA-70-13.54 PROJECT 4A\ANALYSIS\FRA-71-1518A AND RETAINING WALL
4W11\DRIVEABILITY\HP 10X42\B-108-9-15 - RA - HP10X42.GWW
 Hammer File: C:\ProgramData\PDI\GRLWEAP\2010\Resource\HAMMER2003.GW
 Hammer File Version:  2003 (2/22/2013)

                        Input File Contents
     FRA-71-1518A - RA - B-108-9-15 - 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
    67.000     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	0.10	0.15	1.21	0.00	0.0
7.89	0.34	2.15	0.10	0.10	0.10	0.15	1.21	0.00	0.0
7.91	0.34	2.15	0.10	0.10	0.10	0.15	1.21	0.00	0.0
8.49	0.35	2.23	0.10	0.10	0.10	0.15	1.21	0.00	0.0
8.51	1.06	1.16	0.10	0.10	0.20	0.15	1.49	0.00	0.0
13.49	1.11	1.16	0.10	0.10	0.20	0.15	1.49	0.00	0.0
13.51	0.51	4.09	0.10	0.10	0.10	0.15	1.21	0.00	0.0
20.99	0.70	5.50	0.10	0.10	0.10	0.15	1.21	0.00	0.0
21.01	1.02	12.07	0.10	0.10	0.10	0.15	1.00	0.00	0.0
30.01	1.38	16.24	0.10	0.10	0.10	0.15	1.00	0.00	0.0
32.49	1.47	17.39	0.10	0.10	0.10	0.15	1.00	0.00	0.0
32.51	1.75	33.61	0.10	0.10	0.05	0.15	1.00	0.00	0.0
37.49	2.00	38.40	0.10	0.10	0.05	0.15	1.00	0.00	0.0
37.51	1.53	16.11	0.10	0.10	0.05	0.15	1.00	0.00	0.0
42.49	1.70	17.74	0.10	0.10	0.05	0.15	1.00	0.00	0.0
42.51	2.19	33.19	0.10	0.10	0.05	0.15	1.00	0.00	0.0
51.51	2.61	35.96	0.10	0.10	0.05	0.15	1.00	0.00	0.0
52.49	2.65	35.96	0.10	0.10	0.05	0.15	1.00	0.00	0.0
52.51	1.86	13.05	0.10	0.10	0.05	0.15	1.21	0.00	0.0
57.49	2.02	13.05	0.10	0.10	0.05	0.15	1.21	0.00	0.0
57.51	1.57	1.74	0.10	0.10	0.15	0.15	1.21	0.00	0.0
62.49	1.57	1.74	0.10	0.10	0.15	0.15	1.21	0.00	0.0
62.51	1.80	6.33	0.10	0.10	0.05	0.15	1.00	0.00	0.0
67.00	1.92	6.33	0.10	0.10	0.05	0.15	1.00	0.00	0.0

Gain/Loss factors: shaft and toe

0.60400	0.63700	0.67000	0.70300	0.73600
1.00000	1.00000	1.00000	1.00000	1.00000

Dpth	L	Wait	Strk	Pmx%	Eff.	Stff	CoR
5.00	0.00	0.00	0.000	0.000	0.000	0.000	0.000
10.00	0.00	0.00	0.000	0.000	0.000	0.000	0.000
15.00	0.00	0.00	0.000	0.000	0.000	0.000	0.000
20.00	0.00	0.00	0.000	0.000	0.000	0.000	0.000
25.00	0.00	0.00	0.000	0.000	0.000	0.000	0.000
30.00	0.00	0.00	0.000	0.000	0.000	0.000	0.000
35.00	0.00	0.00	0.000	0.000	0.000	0.000	0.000
40.00	0.00	0.00	0.000	0.000	0.000	0.000	0.000
45.00	0.00	0.00	0.000	0.000	0.000	0.000	0.000
50.00	0.00	0.00	0.000	0.000	0.000	0.000	0.000
55.00	0.00	0.00	0.000	0.000	0.000	0.000	0.000
60.00	0.00	0.00	0.000	0.000	0.000	0.000	0.000
65.00	0.00	0.00	0.000	0.000	0.000	0.000	0.000
67.00	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-71-1518A - RA - B-108-9-15 - HP10x42

Hammer Model: D 19-42 Made by: DELMAG

No.	Weight kips	Stiffn k/inch	CoR	C-Slk ft	Dampg 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		8945.3			

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-71-1518A - RA - B-108-9-15 - HP10x42 07/06/2018
Resource International Inc GRLWEAP Version 2010

Depth	(ft)	5.0			
Shaft Gain/Loss Factor		0.604	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
67.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 8.109

Pile and Soil Model — Total Capacity Rut (kips) 2.8

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	8945	0.010	0.000	0.85	0.0	0.000	0.100	3.35	3.3	12.4
2	0.142	8945	0.000	0.000	1.00	0.0	0.000	0.100	6.70	3.3	12.4
19	0.142	8945	0.000	0.000	1.00	0.2	0.100	0.100	63.65	3.3	12.4
20	0.142	8945	0.000	0.000	1.00	1.2	0.100	0.100	67.00	3.3	12.4
Toe						1.4	0.150	0.100			

2.839 kips total unreduced pile weight (g= 32.17 ft/s2)
2.839 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	
5.00	10.81	1.00	0.800

INITIAL STATIC ANALYSIS: Total Wt, Sum(R) 5.5 2.8
Hammer+Pile Weight > Rult: Pile Runs

INITIAL STATIC ANALYSIS: Total Wt, Sum(R) 5.5 2.8
Hammer+Pile Weight > Rult: Pile Runs

INITIAL STATIC ANALYSIS: Total Wt, Sum(R) 5.5 2.8
Hammer+Pile Weight > Rult: Pile Runs

INITIAL STATIC ANALYSIS: Total Wt, Sum(R) 5.5 2.8
Hammer+Pile Weight > Rult: Pile Runs

INITIAL STATIC ANALYSIS: Total Wt, Sum(R) 5.5 2.9
Hammer+Pile Weight > Rult: Pile Runs

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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
2.8	0.0	10.81	0.00	0.00	1	0	0.00	1	0	0.0	78.4
2.8	0.0	11.86	0.00	0.00	1	0	0.00	1	0	0.0	74.4
2.8	0.0	11.86	0.00	0.00	1	0	0.00	1	0	0.0	74.4
2.8	0.0	11.86	0.00	0.00	1	0	0.00	1	0	0.0	74.4
2.9	0.0	11.86	0.00	0.00	1	0	0.00	1	0	0.0	74.4

1 0 10.81000 11.86000

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Depth (ft) 10.0
Shaft Gain/Loss Factor 0.604 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
67.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 8.109

Pile and Soil Model Total Capacity Rut (kips) 8.4

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	8945	0.010	0.000	0.85	0.0	0.000	0.100	3.35	3.3	12.4
2	0.142	8945	0.000	0.000	1.00	0.0	0.000	0.100	6.70	3.3	12.4
18	0.142	8945	0.000	0.000	1.00	0.6	0.100	0.100	60.30	3.3	12.4
19	0.142	8945	0.000	0.000	1.00	1.9	0.100	0.100	63.65	3.3	12.4
20	0.142	8945	0.000	0.000	1.00	4.8	0.173	0.100	67.00	3.3	12.4
Toe						1.2	0.150	0.100			

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

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
10.00	10.81	1.00	0.800

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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
8.4	Hammer did not run										
8.7	Hammer did not run										
8.9	Hammer did not run										
9.2	Hammer did not run										
9.4	Hammer did not run										

1 0 10.81000 11.86000

FRA-71-1518A - RA - B-108-9-15 - HP10x42 07/06/2018
Resource International Inc GRLWEAP Version 2010

Depth (ft)	15.0		
Shaft Gain/Loss Factor	0.604	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
67.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 8.109

Pile and Soil Model — Total Capacity Rut (kips) 21.3

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	8945	0.010	0.000	0.85	0.0	0.000	0.100	3.35	3.3	12.4
2	0.142	8945	0.000	0.000	1.00	0.0	0.000	0.100	6.70	3.3	12.4
16	0.142	8945	0.000	0.000	1.00	0.1	0.100	0.100	53.60	3.3	12.4
17	0.142	8945	0.000	0.000	1.00	1.2	0.100	0.100	56.95	3.3	12.4
18	0.142	8945	0.000	0.000	1.00	2.5	0.100	0.100	60.30	3.3	12.4
19	0.142	8945	0.000	0.000	1.00	6.9	0.198	0.100	63.65	3.3	12.4
20	0.142	8945	0.000	0.000	1.00	6.2	0.172	0.100	67.00	3.3	12.4
Toe						4.4	0.150	0.100			

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

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
15.00	10.81	1.00	0.800

FRA-71-1518A - RA - B-108-9-15 - HP10x42 07/06/2018
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
21.3	1.9	4.00	3.96	-0.29	5	12	13.08	1	2	23.9	59.4
22.0	2.0	4.03	4.00	-0.29	5	12	13.41	1	2	23.7	59.2
22.8	2.0	4.05	4.03	-0.27	5	12	13.71	1	2	23.5	58.9
23.5	2.1	4.08	4.07	-0.23	5	12	13.98	1	2	23.3	58.7
24.2	2.2	4.08	4.11	-0.18	5	12	14.12	1	2	23.1	58.5

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FRA-71-1518A - RA - B-108-9-15 - HP10x42 07/06/2018
Resource International Inc GRLWEAP Version 2010

Depth (ft)	20.0		
Shaft Gain/Loss Factor	0.604	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
67.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 8.109

Pile and Soil Model — Total Capacity Rut (kips) 30.3

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	8945	0.010	0.000	0.85	0.0	0.000	0.100	3.35	3.3	12.4
2	0.142	8945	0.000	0.000	1.00	0.0	0.000	0.100	6.70	3.3	12.4
15	0.142	8945	0.000	0.000	1.00	0.6	0.100	0.100	50.25	3.3	12.4
16	0.142	8945	0.000	0.000	1.00	1.8	0.100	0.100	53.60	3.3	12.4
17	0.142	8945	0.000	0.000	1.00	4.7	0.172	0.100	56.95	3.3	12.4

18	0.142	8945	0.000	0.000	1.00	7.3	0.200	0.100	60.30	3.3	12.4
19	0.142	8945	0.000	0.000	1.00	5.0	0.111	0.100	63.65	3.3	12.4
20	0.142	8945	0.000	0.000	1.00	5.6	0.100	0.100	67.00	3.3	12.4
Toe						5.3	0.150	0.100			

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

Depth ft	Stroke ft	Pressure Ratio	Efficy
20.00	10.81	1.00	0.800

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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
30.3	2.7	4.28	4.30	-0.27	4	12	16.07	1	2	22.3	57.1
31.2	2.8	4.31	4.34	-0.25	4	12	16.35	1	2	22.2	56.9
32.1	2.9	4.35	4.38	-0.22	4	12	16.60	1	2	22.1	56.7
33.0	3.0	4.37	4.41	-0.18	4	12	16.78	2	2	21.9	56.5
33.9	3.1	4.40	4.45	-0.16	4	12	16.98	4	3	21.8	56.3

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FRA-71-1518A - RA - B-108-9-15 - HP10x42 07/06/2018
Resource International Inc GRLWEAP Version 2010

Depth (ft) 25.0
Shaft Gain/Loss Factor 0.604 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
67.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 8.109

Pile and Soil Model Total Capacity Rut (kips) 55.3

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	8945	0.010	0.000	0.85	0.0	0.000	0.100	3.35	3.3	12.4
2	0.142	8945	0.000	0.000	1.00	0.0	0.000	0.100	6.70	3.3	12.4
13	0.142	8945	0.000	0.000	1.00	0.1	0.100	0.100	43.55	3.3	12.4
14	0.142	8945	0.000	0.000	1.00	1.2	0.100	0.100	46.90	3.3	12.4
15	0.142	8945	0.000	0.000	1.00	2.5	0.100	0.100	50.25	3.3	12.4
16	0.142	8945	0.000	0.000	1.00	6.9	0.197	0.100	53.60	3.3	12.4
17	0.142	8945	0.000	0.000	1.00	6.2	0.173	0.100	56.95	3.3	12.4
18	0.142	8945	0.000	0.000	1.00	5.2	0.100	0.100	60.30	3.3	12.4
19	0.142	8945	0.000	0.000	1.00	7.0	0.100	0.100	63.65	3.3	12.4
20	0.142	8945	0.000	0.000	1.00	12.3	0.100	0.100	67.00	3.3	12.4
Toe						13.9	0.150	0.100			

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

Depth ft	Stroke ft	Pressure Ratio	Efficy
25.00	10.81	1.00	0.800

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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
55.3	5.3	5.01	4.97	0.00	1	0	21.17	15	5	20.2	52.8
56.2	5.4	5.03	5.00	0.00	1	0	21.33	15	5	20.1	52.7
57.1	5.5	5.05	5.02	0.00	1	0	21.47	15	5	20.0	52.5
58.1	5.7	5.08	5.05	0.00	1	0	21.62	16	5	20.0	52.4
59.0	5.8	5.11	5.07	0.00	1	0	21.80	16	5	19.9	52.2

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FRA-71-1518A - RA - B-108-9-15 - HP10x42 07/06/2018
Resource International Inc GRLWEAP Version 2010

Depth (ft) 30.0
Shaft Gain/Loss Factor 0.604 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
67.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 8.109

Pile and Soil Model Total Capacity Rut (kips) 78.6

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	8945	0.010	0.000	0.85	0.0	0.000	0.100	3.35	3.3	12.4
2	0.142	8945	0.000	0.000	1.00	0.0	0.000	0.100	6.70	3.3	12.4
12	0.142	8945	0.000	0.000	1.00	0.6	0.100	0.100	40.20	3.3	12.4
13	0.142	8945	0.000	0.000	1.00	1.8	0.100	0.100	43.55	3.3	12.4
14	0.142	8945	0.000	0.000	1.00	4.6	0.170	0.100	46.90	3.3	12.4
15	0.142	8945	0.000	0.000	1.00	7.3	0.200	0.100	50.25	3.3	12.4
16	0.142	8945	0.000	0.000	1.00	5.1	0.114	0.100	53.60	3.3	12.4
17	0.142	8945	0.000	0.000	1.00	5.6	0.100	0.100	56.95	3.3	12.4
18	0.142	8945	0.000	0.000	1.00	10.0	0.100	0.100	60.30	3.3	12.4
19	0.142	8945	0.000	0.000	1.00	13.0	0.100	0.100	63.65	3.3	12.4
20	0.142	8945	0.000	0.000	1.00	14.5	0.100	0.100	67.00	3.3	12.4
Toe						16.2	0.150	0.100			

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

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
30.00	10.81	1.00	0.800

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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
78.6	8.0	5.49	5.47	-1.42	12	48	23.56	14	5	19.0	50.2
79.6	8.1	5.50	5.48	-1.42	12	48	23.68	14	5	19.0	50.2
80.5	8.2	5.52	5.50	-1.40	12	48	23.82	14	5	19.0	50.1
81.4	8.4	5.54	5.53	-1.39	13	48	23.93	14	5	18.9	50.0
82.4	8.6	5.50	5.56	-1.37	13	47	23.84	14	5	18.7	50.0

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Depth (ft) 35.0
Shaft Gain/Loss Factor 0.604 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
67.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 8.109

Pile and Soil Model Total Capacity Rut (kips) 125.1

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

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1	0.142	8945	0.010	0.000	0.85	0.0	0.000	0.100	3.35	3.3	12.4
2	0.142	8945	0.000	0.000	1.00	0.0	0.000	0.100	6.70	3.3	12.4
10	0.142	8945	0.000	0.000	1.00	0.1	0.100	0.100	33.50	3.3	12.4
11	0.142	8945	0.000	0.000	1.00	1.2	0.100	0.100	36.85	3.3	12.4
12	0.142	8945	0.000	0.000	1.00	2.4	0.100	0.100	40.20	3.3	12.4
13	0.142	8945	0.000	0.000	1.00	6.8	0.197	0.100	43.55	3.3	12.4
14	0.142	8945	0.000	0.000	1.00	6.2	0.174	0.100	46.90	3.3	12.4
15	0.142	8945	0.000	0.000	1.00	5.2	0.100	0.100	50.25	3.3	12.4
16	0.142	8945	0.000	0.000	1.00	6.9	0.100	0.100	53.60	3.3	12.4
17	0.142	8945	0.000	0.000	1.00	12.3	0.100	0.100	56.95	3.3	12.4
18	0.142	8945	0.000	0.000	1.00	13.7	0.100	0.100	60.30	3.3	12.4
19	0.142	8945	0.000	0.000	1.00	15.2	0.100	0.100	63.65	3.3	12.4
20	0.142	8945	0.000	0.000	1.00	19.0	0.061	0.100	67.00	3.3	12.4
Toe						36.0	0.150	0.100			

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

Depth ft	Stroke ft	Pressure Ratio	Efficy
35.00	10.81	1.00	0.800

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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
125.1	14.1	6.18	6.21	-1.60	11	36	26.35	13	4	17.9	47.3
126.0	14.2	6.20	6.22	-1.62	11	36	26.49	13	4	17.9	47.2
127.0	14.4	6.22	6.23	-1.65	11	36	26.57	13	4	17.9	47.2
127.9	14.5	6.23	6.25	-1.67	11	36	26.70	13	4	17.9	47.1
128.8	14.7	6.25	6.26	-1.68	11	36	26.79	13	4	17.8	47.1

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Resource International Inc GRLWEAP Version 2010

Depth (ft) 40.0
Shaft Gain/Loss Factor 0.604 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
67.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 8.109

Pile and Soil Model Total Capacity Rut (kips) 135.0

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	8945	0.010	0.000	0.85	0.0	0.000	0.100	3.35	3.3	12.4
2	0.142	8945	0.000	0.000	1.00	0.0	0.000	0.100	6.70	3.3	12.4
9	0.142	8945	0.000	0.000	1.00	0.6	0.100	0.100	30.15	3.3	12.4
10	0.142	8945	0.000	0.000	1.00	1.8	0.100	0.100	33.50	3.3	12.4
11	0.142	8945	0.000	0.000	1.00	4.5	0.169	0.100	36.85	3.3	12.4
12	0.142	8945	0.000	0.000	1.00	7.3	0.200	0.100	40.20	3.3	12.4
13	0.142	8945	0.000	0.000	1.00	5.1	0.116	0.100	43.55	3.3	12.4
14	0.142	8945	0.000	0.000	1.00	5.6	0.100	0.100	46.90	3.3	12.4
15	0.142	8945	0.000	0.000	1.00	9.9	0.100	0.100	50.25	3.3	12.4
16	0.142	8945	0.000	0.000	1.00	13.0	0.100	0.100	53.60	3.3	12.4
17	0.142	8945	0.000	0.000	1.00	14.4	0.100	0.100	56.95	3.3	12.4
18	0.142	8945	0.000	0.000	1.00	16.6	0.086	0.100	60.30	3.3	12.4
19	0.142	8945	0.000	0.000	1.00	20.7	0.050	0.100	63.65	3.3	12.4
20	0.142	8945	0.000	0.000	1.00	18.5	0.050	0.100	67.00	3.3	12.4
Toe						16.9	0.150	0.100			

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

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
40.00	10.81	1.00	0.800

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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
135.0	14.5	6.24	6.26	-1.54	11	35	26.47	11	4	17.8	47.1
135.9	14.8	6.25	6.28	-1.51	11	35	26.56	11	4	17.7	47.0
136.8	14.9	6.28	6.29	-1.47	11	35	26.69	11	4	17.8	47.0
137.8	15.1	6.30	6.31	-1.45	10	36	26.79	11	4	17.8	46.9
138.7	15.3	6.31	6.33	-1.44	10	36	26.87	11	4	17.8	46.8

1 0 10.81000 11.86000

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Depth (ft) 45.0
Shaft Gain/Loss Factor 0.604 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
67.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 8.109

Pile and Soil Model Total Capacity Rut (kips) 184.2

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	8945	0.010	0.000	0.85	0.0	0.000	0.100	3.35	3.3	12.4
2	0.142	8945	0.000	0.000	1.00	0.0	0.000	0.100	6.70	3.3	12.4
7	0.142	8945	0.000	0.000	1.00	0.1	0.100	0.100	23.45	3.3	12.4
8	0.142	8945	0.000	0.000	1.00	1.2	0.100	0.100	26.80	3.3	12.4
9	0.142	8945	0.000	0.000	1.00	2.4	0.100	0.100	30.15	3.3	12.4
10	0.142	8945	0.000	0.000	1.00	6.7	0.196	0.100	33.50	3.3	12.4
11	0.142	8945	0.000	0.000	1.00	6.3	0.175	0.100	36.85	3.3	12.4
12	0.142	8945	0.000	0.000	1.00	5.2	0.100	0.100	40.20	3.3	12.4
13	0.142	8945	0.000	0.000	1.00	6.8	0.100	0.100	43.55	3.3	12.4
14	0.142	8945	0.000	0.000	1.00	12.3	0.100	0.100	46.90	3.3	12.4
15	0.142	8945	0.000	0.000	1.00	13.7	0.100	0.100	50.25	3.3	12.4
16	0.142	8945	0.000	0.000	1.00	15.2	0.100	0.100	53.60	3.3	12.4
17	0.142	8945	0.000	0.000	1.00	18.9	0.061	0.100	56.95	3.3	12.4
18	0.142	8945	0.000	0.000	1.00	20.4	0.050	0.100	60.30	3.3	12.4
19	0.142	8945	0.000	0.000	1.00	17.9	0.050	0.100	63.65	3.3	12.4
20	0.142	8945	0.000	0.000	1.00	23.3	0.050	0.100	67.00	3.3	12.4
Toe						34.0	0.150	0.100			

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

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
45.00	10.81	1.00	0.800

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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
184.2	21.1	6.83	6.79	-1.36	9	29	28.48	10	4	17.6	45.1
185.2	21.2	6.84	6.79	-1.35	9	29	28.59	10	4	17.7	45.1
186.1	21.5	6.85	6.81	-1.32	9	29	28.67	10	4	17.6	45.0
187.0	21.5	6.87	6.82	-1.31	9	29	28.76	10	4	17.7	45.0
188.0	21.7	6.87	6.83	-1.29	9	29	28.82	10	4	17.6	45.0

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Depth	(ft)	50.0		
Shaft Gain/Loss Factor		0.604	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
67.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 8.109

Pile and Soil Model — Total Capacity Rut (kips) 225.7

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	8945	0.010	0.000	0.85	0.0	0.000	0.100	3.35	3.3	12.4
2	0.142	8945	0.000	0.000	1.00	0.0	0.000	0.100	6.70	3.3	12.4
6	0.142	8945	0.000	0.000	1.00	0.5	0.100	0.100	20.10	3.3	12.4
7	0.142	8945	0.000	0.000	1.00	1.8	0.100	0.100	23.45	3.3	12.4
8	0.142	8945	0.000	0.000	1.00	4.5	0.168	0.100	26.80	3.3	12.4
9	0.142	8945	0.000	0.000	1.00	7.3	0.200	0.100	30.15	3.3	12.4
10	0.142	8945	0.000	0.000	1.00	5.1	0.119	0.100	33.50	3.3	12.4
11	0.142	8945	0.000	0.000	1.00	5.6	0.100	0.100	36.85	3.3	12.4
12	0.142	8945	0.000	0.000	1.00	9.8	0.100	0.100	40.20	3.3	12.4
13	0.142	8945	0.000	0.000	1.00	13.0	0.100	0.100	43.55	3.3	12.4
14	0.142	8945	0.000	0.000	1.00	14.4	0.100	0.100	46.90	3.3	12.4
15	0.142	8945	0.000	0.000	1.00	16.6	0.087	0.100	50.25	3.3	12.4
16	0.142	8945	0.000	0.000	1.00	20.7	0.050	0.100	53.60	3.3	12.4
17	0.142	8945	0.000	0.000	1.00	18.6	0.050	0.100	56.95	3.3	12.4
18	0.142	8945	0.000	0.000	1.00	19.8	0.050	0.100	60.30	3.3	12.4
19	0.142	8945	0.000	0.000	1.00	25.5	0.050	0.100	63.65	3.3	12.4
20	0.142	8945	0.000	0.000	1.00	27.2	0.050	0.100	67.00	3.3	12.4
Toe						35.5	0.150	0.100			

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

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
50.00	10.81	1.00	0.800

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Rut	Bl Ct	Stroke (ft) down	up	Ten Str	i	t	Comp Str	i	t	ENTHRU	Bl Rt
kips	b/ft			ksi			ksi			kip-ft	b/min
225.7	26.8	7.17	7.13	-1.60	8	50	29.33	8	3	17.8	44.1
226.7	26.9	7.18	7.14	-1.61	8	50	29.40	8	3	17.8	44.0
227.6	27.2	7.19	7.15	-1.59	8	50	29.47	8	3	17.8	44.0
228.5	27.4	7.19	7.16	-1.57	8	50	29.51	8	3	17.8	44.0
229.5	27.7	7.21	7.17	-1.54	8	50	29.60	8	3	17.8	43.9

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Depth	(ft)	55.0		
Shaft Gain/Loss Factor		0.604	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
67.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 8.109

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Pile and Soil Model — Total Capacity Rut (kips) 237.2

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	8945	0.010	0.000	0.85	0.0	0.000	0.100	3.35	3.3	12.4
2	0.142	8945	0.000	0.000	1.00	0.0	0.000	0.100	6.70	3.3	12.4
4	0.142	8945	0.000	0.000	1.00	0.1	0.100	0.100	13.40	3.3	12.4
5	0.142	8945	0.000	0.000	1.00	1.2	0.100	0.100	16.75	3.3	12.4
6	0.142	8945	0.000	0.000	1.00	2.4	0.100	0.100	20.10	3.3	12.4
7	0.142	8945	0.000	0.000	1.00	6.7	0.196	0.100	23.45	3.3	12.4
8	0.142	8945	0.000	0.000	1.00	6.3	0.176	0.100	26.80	3.3	12.4
9	0.142	8945	0.000	0.000	1.00	5.2	0.100	0.100	30.15	3.3	12.4
10	0.142	8945	0.000	0.000	1.00	6.7	0.100	0.100	33.50	3.3	12.4
11	0.142	8945	0.000	0.000	1.00	12.2	0.100	0.100	36.85	3.3	12.4
12	0.142	8945	0.000	0.000	1.00	13.7	0.100	0.100	40.20	3.3	12.4
13	0.142	8945	0.000	0.000	1.00	15.1	0.100	0.100	43.55	3.3	12.4
14	0.142	8945	0.000	0.000	1.00	18.9	0.062	0.100	46.90	3.3	12.4
15	0.142	8945	0.000	0.000	1.00	20.4	0.050	0.100	50.25	3.3	12.4
16	0.142	8945	0.000	0.000	1.00	17.8	0.050	0.100	53.60	3.3	12.4
17	0.142	8945	0.000	0.000	1.00	23.2	0.050	0.100	56.95	3.3	12.4
18	0.142	8945	0.000	0.000	1.00	26.3	0.050	0.100	60.30	3.3	12.4
19	0.142	8945	0.000	0.000	1.00	28.0	0.050	0.100	63.65	3.3	12.4
20	0.142	8945	0.000	0.000	1.00	19.9	0.050	0.100	67.00	3.3	12.4
Toe						13.1	0.150	0.100			

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

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
55.00	10.81	1.00	0.800

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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
237.2	26.9	7.20	7.16	-1.03	6	50	29.51	7	3	17.5	44.0
238.4	27.3	7.20	7.17	-0.97	6	50	29.58	7	3	17.5	43.9
239.6	27.6	7.22	7.19	-0.92	6	50	29.68	7	3	17.5	43.9
240.8	27.8	7.24	7.20	-0.85	6	50	29.81	7	3	17.5	43.9
242.0	28.1	7.25	7.21	-0.81	5	46	29.90	7	3	17.5	43.8

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Depth (ft) 60.0
Shaft Gain/Loss Factor 0.604 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
67.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 8.109

Pile and Soil Model — Total Capacity Rut (kips) 249.2

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	8945	0.010	0.000	0.85	0.0	0.000	0.100	3.35	3.3	12.4
2	0.142	8945	0.000	0.000	1.00	0.0	0.000	0.100	6.70	3.3	12.4
3	0.142	8945	0.000	0.000	1.00	0.5	0.100	0.100	10.05	3.3	12.4
4	0.142	8945	0.000	0.000	1.00	1.8	0.100	0.100	13.40	3.3	12.4
5	0.142	8945	0.000	0.000	1.00	4.4	0.167	0.100	16.75	3.3	12.4
6	0.142	8945	0.000	0.000	1.00	7.3	0.200	0.100	20.10	3.3	12.4
7	0.142	8945	0.000	0.000	1.00	5.2	0.121	0.100	23.45	3.3	12.4
8	0.142	8945	0.000	0.000	1.00	5.5	0.100	0.100	26.80	3.3	12.4
9	0.142	8945	0.000	0.000	1.00	9.7	0.100	0.100	30.15	3.3	12.4
10	0.142	8945	0.000	0.000	1.00	12.9	0.100	0.100	33.50	3.3	12.4
11	0.142	8945	0.000	0.000	1.00	14.4	0.100	0.100	36.85	3.3	12.4

12	0.142	8945	0.000	0.000	1.00	16.5	0.088	0.100	40.20	3.3	12.4
13	0.142	8945	0.000	0.000	1.00	20.6	0.050	0.100	43.55	3.3	12.4
14	0.142	8945	0.000	0.000	1.00	18.7	0.050	0.100	46.90	3.3	12.4
15	0.142	8945	0.000	0.000	1.00	19.7	0.050	0.100	50.25	3.3	12.4
16	0.142	8945	0.000	0.000	1.00	25.5	0.050	0.100	53.60	3.3	12.4
17	0.142	8945	0.000	0.000	1.00	27.2	0.050	0.100	56.95	3.3	12.4
18	0.142	8945	0.000	0.000	1.00	25.8	0.050	0.100	60.30	3.3	12.4
19	0.142	8945	0.000	0.000	1.00	17.1	0.050	0.100	63.65	3.3	12.4
20	0.142	8945	0.000	0.000	1.00	14.8	0.120	0.100	67.00	3.3	12.4
Toe						1.7	0.150	0.100			

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

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
60.00	10.81	1.00	0.800

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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
249.2	28.8	7.29	7.26	-0.58	5	45	29.53	5	3	17.2	43.7
250.9	29.1	7.31	7.27	-0.57	5	45	29.63	5	3	17.3	43.7
252.6	29.6	7.33	7.29	-0.61	4	44	29.72	5	3	17.3	43.6
254.3	30.0	7.34	7.31	-0.67	4	44	29.81	5	3	17.3	43.6
256.0	30.7	7.37	7.33	-0.71	4	44	29.91	5	3	17.2	43.5

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Depth (ft) 65.0
Shaft Gain/Loss Factor 0.604 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
67.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 8.109

Pile and Soil Model Total Capacity Rut (kips) 279.2

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	8945	0.010	0.000	0.85	0.1	0.100	0.100	3.35	3.3	12.4
2	0.142	8945	0.000	0.000	1.00	1.1	0.100	0.100	6.70	3.3	12.4
3	0.142	8945	0.000	0.000	1.00	2.4	0.100	0.100	10.05	3.3	12.4
4	0.142	8945	0.000	0.000	1.00	6.6	0.195	0.100	13.40	3.3	12.4
5	0.142	8945	0.000	0.000	1.00	6.3	0.178	0.100	16.75	3.3	12.4
6	0.142	8945	0.000	0.000	1.00	5.2	0.100	0.100	20.10	3.3	12.4
7	0.142	8945	0.000	0.000	1.00	6.6	0.100	0.100	23.45	3.3	12.4
8	0.142	8945	0.000	0.000	1.00	12.2	0.100	0.100	26.80	3.3	12.4
9	0.142	8945	0.000	0.000	1.00	13.7	0.100	0.100	30.15	3.3	12.4
10	0.142	8945	0.000	0.000	1.00	15.1	0.100	0.100	33.50	3.3	12.4
11	0.142	8945	0.000	0.000	1.00	18.8	0.063	0.100	36.85	3.3	12.4
12	0.142	8945	0.000	0.000	1.00	20.5	0.050	0.100	40.20	3.3	12.4
13	0.142	8945	0.000	0.000	1.00	17.8	0.050	0.100	43.55	3.3	12.4
14	0.142	8945	0.000	0.000	1.00	23.1	0.050	0.100	46.90	3.3	12.4
15	0.142	8945	0.000	0.000	1.00	26.3	0.050	0.100	50.25	3.3	12.4
16	0.142	8945	0.000	0.000	1.00	28.0	0.050	0.100	53.60	3.3	12.4
17	0.142	8945	0.000	0.000	1.00	20.0	0.050	0.100	56.95	3.3	12.4
18	0.142	8945	0.000	0.000	1.00	16.6	0.070	0.100	60.30	3.3	12.4
19	0.142	8945	0.000	0.000	1.00	13.8	0.150	0.100	63.65	3.3	12.4
20	0.142	8945	0.000	0.000	1.00	18.6	0.073	0.100	67.00	3.3	12.4
Toe						6.3	0.150	0.100			

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

Depth ft	Stroke ft	Pressure Ratio	Efficy
65.00	10.81	1.00	0.800

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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
279.2	36.9	7.56	7.53	-0.39	3	41	30.43	4	3	17.2	42.9
281.2	37.5	7.56	7.54	-0.41	4	41	30.51	4	3	17.2	42.9
283.1	38.1	7.59	7.55	-0.43	4	41	30.64	4	3	17.2	42.9
285.0	39.2	7.60	7.58	-0.41	4	41	30.73	4	3	17.1	42.8
286.9	39.8	7.62	7.59	-0.41	4	41	30.85	4	3	17.2	42.8

1 0 10.81000 11.86000

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Depth (ft) 67.0
Shaft Gain/Loss Factor 0.604 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
67.0	12.40	29000.	492.0	3.3	0	16524.	21.8

Wave Travel Time 2L/c (ms) 8.109

Pile and Soil Model Total Capacity Rut (kips) 291.7

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	8945	0.010	0.000	0.85	0.6	0.100	0.100	3.35	3.3	12.4
2	0.142	8945	0.000	0.000	1.00	1.9	0.100	0.100	6.70	3.3	12.4
3	0.142	8945	0.000	0.000	1.00	4.8	0.174	0.100	10.05	3.3	12.4
4	0.142	8945	0.000	0.000	1.00	7.3	0.200	0.100	13.40	3.3	12.4
5	0.142	8945	0.000	0.000	1.00	5.0	0.106	0.100	16.75	3.3	12.4
6	0.142	8945	0.000	0.000	1.00	5.6	0.100	0.100	20.10	3.3	12.4
7	0.142	8945	0.000	0.000	1.00	10.3	0.100	0.100	23.45	3.3	12.4
8	0.142	8945	0.000	0.000	1.00	13.1	0.100	0.100	26.80	3.3	12.4
9	0.142	8945	0.000	0.000	1.00	14.5	0.100	0.100	30.15	3.3	12.4
10	0.142	8945	0.000	0.000	1.00	16.9	0.083	0.100	33.50	3.3	12.4
11	0.142	8945	0.000	0.000	1.00	20.8	0.050	0.100	36.85	3.3	12.4
12	0.142	8945	0.000	0.000	1.00	18.3	0.050	0.100	40.20	3.3	12.4
13	0.142	8945	0.000	0.000	1.00	20.3	0.050	0.100	43.55	3.3	12.4
14	0.142	8945	0.000	0.000	1.00	25.6	0.050	0.100	46.90	3.3	12.4
15	0.142	8945	0.000	0.000	1.00	27.3	0.050	0.100	50.25	3.3	12.4
16	0.142	8945	0.000	0.000	1.00	24.7	0.050	0.100	53.60	3.3	12.4
17	0.142	8945	0.000	0.000	1.00	17.2	0.050	0.100	56.95	3.3	12.4
18	0.142	8945	0.000	0.000	1.00	14.5	0.130	0.100	60.30	3.3	12.4
19	0.142	8945	0.000	0.000	1.00	16.0	0.112	0.100	63.65	3.3	12.4
20	0.142	8945	0.000	0.000	1.00	20.7	0.050	0.100	67.00	3.3	12.4
Toe						6.3	0.150	0.100			

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

Depth ft	Stroke ft	Pressure Ratio	Efficy
67.00	10.81	1.00	0.800

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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
291.7	40.7	7.62	7.61	-0.42	3	40	30.69	3	3	17.0	42.8
293.6	41.8	7.64	7.63	-0.48	3	40	30.82	3	2	17.0	42.7

295.6	42.6	7.65	7.64	-0.52	3	39	30.91	3	3	17.0	42.7
297.5	43.2	7.67	7.65	-0.54	3	39	31.06	3	3	17.1	42.6
299.4	44.3	7.70	7.67	-0.53	3	39	31.17	3	3	17.1	42.6

SUMMARY OVER DEPTHS

G/L at Shaft and Toe: 0.604 1.000

Depth ft	Rut kips	Frictn kips	End Bg kips	Bl Ct bl/ft	Com Str ksi	Ten Str ksi	Stroke ft	ENTHRU kip-ft
5.0	2.8	1.4	1.4	0.0	0.000	0.000	10.81	0.0
10.0	8.4	7.2	1.2	Hammer did not run				
15.0	21.3	17.0	4.4	1.9	13.077	-0.295	4.00	23.9
20.0	30.3	25.0	5.3	2.7	16.075	-0.270	4.28	22.3
25.0	55.3	41.3	13.9	5.3	21.173	0.000	5.01	20.2
30.0	78.6	62.4	16.2	8.0	23.557	-1.421	5.49	19.0
35.0	125.1	89.1	36.0	14.1	26.353	-1.601	6.18	17.9
40.0	135.0	118.0	16.9	14.5	26.472	-1.539	6.24	17.8
45.0	184.2	150.3	34.0	21.1	28.479	-1.360	6.83	17.6
50.0	225.7	190.2	35.5	26.8	29.325	-1.602	7.17	17.8
55.0	237.2	224.1	13.1	26.9	29.510	-1.032	7.20	17.5
60.0	249.2	247.5	1.7	28.8	29.532	-0.584	7.29	17.2
65.0	279.2	272.9	6.3	36.9	30.428	-0.388	7.56	17.2
67.0	291.7	285.4	6.3	40.7	30.691	-0.421	7.62	17.0

Total Driving Time 20 minutes; Total No. of Blows 920

G/L at Shaft and Toe: 0.637 1.000

Depth ft	Rut kips	Frictn kips	End Bg kips	Bl Ct bl/ft	Com Str ksi	Ten Str ksi	Stroke ft	ENTHRU kip-ft
5.0	2.8	1.4	1.4	0.0	0.000	0.000	11.86	0.0
10.0	8.7	7.5	1.2	Hammer did not run				
15.0	22.0	17.7	4.4	2.0	13.411	-0.287	4.03	23.7
20.0	31.2	25.9	5.3	2.8	16.355	-0.247	4.31	22.2
25.0	56.2	42.3	13.9	5.4	21.330	0.000	5.03	20.1
30.0	79.6	63.3	16.2	8.1	23.682	-1.415	5.50	19.0
35.0	126.0	90.0	36.0	14.2	26.493	-1.623	6.20	17.9
40.0	135.9	119.0	16.9	14.8	26.556	-1.513	6.25	17.7
45.0	185.2	151.2	34.0	21.2	28.588	-1.347	6.84	17.7
50.0	226.7	191.2	35.5	26.9	29.395	-1.609	7.18	17.8
55.0	238.4	225.3	13.1	27.3	29.579	-0.972	7.20	17.5
60.0	250.9	249.2	1.7	29.1	29.628	-0.566	7.31	17.3
65.0	281.2	274.8	6.3	37.5	30.507	-0.412	7.56	17.2
67.0	293.6	287.3	6.3	41.8	30.823	-0.476	7.64	17.0

Total Driving Time 21 minutes; Total No. of Blows 933

SUMMARY OVER DEPTHS

G/L at Shaft and Toe: 0.670 1.000

Depth ft	Rut kips	Frictn kips	End Bg kips	Bl Ct bl/ft	Com Str ksi	Ten Str ksi	Stroke ft	ENTHRU kip-ft
5.0	2.8	1.5	1.4	0.0	0.000	0.000	11.86	0.0
10.0	8.9	7.8	1.2	Hammer did not run				
15.0	22.8	18.4	4.4	2.0	13.713	-0.266	4.05	23.5
20.0	32.1	26.8	5.3	2.9	16.600	-0.216	4.35	22.1
25.0	57.1	43.2	13.9	5.5	21.471	0.000	5.05	20.0
30.0	80.5	64.3	16.2	8.2	23.815	-1.403	5.52	19.0
35.0	127.0	91.0	36.0	14.4	26.570	-1.647	6.22	17.9
40.0	136.8	119.9	16.9	14.9	26.686	-1.467	6.28	17.8
45.0	186.1	152.1	34.0	21.5	28.670	-1.321	6.85	17.6
50.0	227.6	192.1	35.5	27.2	29.474	-1.594	7.19	17.8
55.0	239.6	226.5	13.1	27.6	29.682	-0.916	7.22	17.5
60.0	252.6	250.9	1.7	29.6	29.722	-0.606	7.33	17.3
65.0	283.1	276.7	6.3	38.1	30.642	-0.426	7.59	17.2
67.0	295.6	289.2	6.3	42.6	30.908	-0.519	7.65	17.0

Total Driving Time 21 minutes; Total No. of Blows 945

G/L at Shaft and Toe: 0.703 1.000

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Depth	Rut	Frictn	End Bg	Bl Ct	Com Str	Ten Str	Stroke	ENTHRU
ft	kips	kips	kips	bl/ft	ksi	ksi	ft	kip-ft
5.0	2.8	1.5	1.4	0.0	0.000	0.000	11.86	0.0
10.0	9.2	8.0	1.2	Hammer did not run				
15.0	23.5	19.1	4.4	2.1	13.983	-0.231	4.08	23.3
20.0	33.0	27.7	5.3	3.0	16.784	-0.180	4.37	21.9
25.0	58.1	44.1	13.9	5.7	21.618	0.000	5.08	20.0
30.0	81.4	65.2	16.2	8.4	23.930	-1.392	5.54	18.9
35.0	127.9	91.9	36.0	14.5	26.697	-1.667	6.23	17.9
40.0	137.8	120.8	16.9	15.1	26.790	-1.451	6.30	17.8
45.0	187.0	153.1	34.0	21.5	28.764	-1.310	6.87	17.7
50.0	228.5	193.0	35.5	27.4	29.508	-1.572	7.19	17.8
55.0	240.8	227.7	13.1	27.8	29.811	-0.852	7.24	17.5
60.0	254.3	252.6	1.7	30.0	29.805	-0.671	7.34	17.3
65.0	285.0	278.7	6.3	39.2	30.726	-0.407	7.60	17.1
67.0	297.5	291.1	6.3	43.2	31.058	-0.536	7.67	17.1

Total Driving Time 21 minutes; Total No. of Blows 957

SUMMARY OVER DEPTHS

G/L at Shaft and Toe: 0.736 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
5.0	2.9	1.5	1.4	0.0	0.000	0.000	11.86	0.0
10.0	9.4	8.3	1.2	Hammer did not run				
15.0	24.2	19.8	4.4	2.2	14.119	-0.181	4.08	23.1
20.0	33.9	28.6	5.3	3.1	16.981	-0.162	4.40	21.8
25.0	59.0	45.1	13.9	5.8	21.797	0.000	5.11	19.9
30.0	82.4	66.1	16.2	8.6	23.836	-1.374	5.50	18.7
35.0	128.8	92.8	36.0	14.7	26.785	-1.680	6.25	17.8
40.0	138.7	121.8	16.9	15.3	26.869	-1.442	6.31	17.8
45.0	188.0	154.0	34.0	21.7	28.820	-1.287	6.87	17.6
50.0	229.5	194.0	35.5	27.7	29.604	-1.544	7.21	17.8
55.0	242.0	228.9	13.1	28.1	29.895	-0.812	7.25	17.5
60.0	256.0	254.3	1.7	30.7	29.912	-0.715	7.37	17.2
65.0	286.9	280.6	6.3	39.8	30.853	-0.412	7.62	17.2
67.0	299.4	293.1	6.3	44.3	31.168	-0.531	7.70	17.1

Total Driving Time 22 minutes; Total No. of Blows 972

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Table of Depths Analyzed with Driving System Modifiers

Depth	Temp. Length	Wait Time	Equivalent Stroke	Pressure Ratio	Efficy.	Stiffn. Factor	Cushion CoR
ft	ft	hr	ft				
5.00	67.00	0.00	10.81	1.00	0.80	1.00	1.00
10.00	67.00	0.00	10.81	1.00	0.80	1.00	1.00
15.00	67.00	0.00	10.81	1.00	0.80	1.00	1.00
20.00	67.00	0.00	10.81	1.00	0.80	1.00	1.00
25.00	67.00	0.00	10.81	1.00	0.80	1.00	1.00
30.00	67.00	0.00	10.81	1.00	0.80	1.00	1.00
35.00	67.00	0.00	10.81	1.00	0.80	1.00	1.00
40.00	67.00	0.00	10.81	1.00	0.80	1.00	1.00
45.00	67.00	0.00	10.81	1.00	0.80	1.00	1.00
50.00	67.00	0.00	10.81	1.00	0.80	1.00	1.00
55.00	67.00	0.00	10.81	1.00	0.80	1.00	1.00
60.00	67.00	0.00	10.81	1.00	0.80	1.00	1.00
65.00	67.00	0.00	10.81	1.00	0.80	1.00	1.00
67.00	67.00	0.00	10.81	1.00	0.80	1.00	1.00

Soil Layer Resistance Values

Depth	Shaft Res.	End Bearing	Shaft Quake	Toe Quake	Shaft Damping	Toe Damping	Soil Setup	Limit Distance	Setup Time
ft	k/ft2	kips	inch	inch	s/ft	s/ft	Normlzd	ft	hrs
0.01	0.00	0.00	0.100	0.100	0.100	0.150	0.515	0.000	0.000
7.89	0.34	2.15	0.100	0.100	0.100	0.150	0.515	0.000	0.000

7.91	0.34	2.15	0.100	0.100	0.100	0.150	0.515	0.000	0.000
8.49	0.35	2.23	0.100	0.100	0.100	0.150	0.515	0.000	0.000
8.51	1.06	1.16	0.100	0.100	0.200	0.150	1.000	0.000	0.000
13.49	1.11	1.16	0.100	0.100	0.200	0.150	1.000	0.000	0.000
13.51	0.51	4.09	0.100	0.100	0.100	0.150	0.515	0.000	0.000
20.99	0.70	5.50	0.100	0.100	0.100	0.150	0.515	0.000	0.000
21.01	1.02	12.07	0.100	0.100	0.100	0.150	0.000	0.000	0.000
30.01	1.38	16.24	0.100	0.100	0.100	0.150	0.000	0.000	0.000
32.49	1.47	17.39	0.100	0.100	0.100	0.150	0.000	0.000	0.000
32.51	1.75	33.61	0.100	0.100	0.050	0.150	0.000	0.000	0.000
37.49	2.00	38.40	0.100	0.100	0.050	0.150	0.000	0.000	0.000
37.51	1.53	16.11	0.100	0.100	0.050	0.150	0.000	0.000	0.000
42.49	1.70	17.74	0.100	0.100	0.050	0.150	0.000	0.000	0.000
42.51	2.19	33.19	0.100	0.100	0.050	0.150	0.000	0.000	0.000
51.51	2.61	35.96	0.100	0.100	0.050	0.150	0.000	0.000	0.000
52.49	2.65	35.96	0.100	0.100	0.050	0.150	0.000	0.000	0.000
52.51	1.86	13.05	0.100	0.100	0.050	0.150	0.515	0.000	0.000
57.49	2.02	13.05	0.100	0.100	0.050	0.150	0.515	0.000	0.000
57.51	1.57	1.74	0.100	0.100	0.150	0.150	0.515	0.000	0.000
62.49	1.57	1.74	0.100	0.100	0.150	0.150	0.515	0.000	0.000
62.51	1.80	6.33	0.100	0.100	0.050	0.150	0.000	0.000	0.000
67.00	1.92	6.33	0.100	0.100	0.050	0.150	0.000	0.000	0.000

APPENDIX VII

DRILLED SHAFT CALCULATIONS

W-13-045 FRA-70-12.68 - FRA-71-1518A
Drilled Shaft Calculations

End Bearing Resistance in Bedrock: Limestone

Intact Rock (Minimum Rock Socket Length ≥ 1.5B):

$q_p = 2.5q_u$ Equation 10.8.3.5.4c-1

q_u =	1,121	ksf
q_p =	2,804	ksf

Jointed Rock (or Shafts with Rock Socket Length < 1.5B):

$$q_p = A + q_u\left[m_b\left(\frac{A}{q_u}\right) + s\right]^a$$ Equation 10.8.3.5.4c-2:

$$A = \sigma'_{vb} + q_u\left[m_b\frac{\sigma'_{vb}}{q_u} + s\right]^a$$ Equation 10.8.3.5.4c-3

q_u =	1,121	ksf	
GSI =	70	Per Figure 10.4.6.4-1	
D =	0.0	Per Section 10.4.6.4 for undisturbed foundation excavation	
m_i =	9	Per Table 10.4.6.4-1	
s =	0.036	Per Equation 10.4.6.4-2	
a =	0.50	Per Equation 10.4.6.4-3	
m_b =	3.08	Per Equation 10.4.6.4-4	
σ'_{vb} =	3.92	ksf	Considering overburden depth of 68 feet and bouyant unit weight of overburden of 57.6 pcf
A =	245	ksf	Per Equation 10.8.3.5.4c-3
q_p =	1,188	ksf	

APPENDIX VIII

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-015-7-13	721.8 to 718.8	A-7-6	C	3	37	37	125 psf	125 psf	Su = 4,625 psf	1,540 pci	0.0045	-
	718.8 to 716.3	A-1-a	G	4	32	46	130 psf	130 psf	φ = 42°	355 pci	-	-
	716.3 to 708.8	A-6a	C	3	11	11	115 psf	115 psf	Su = 1,375 psf	435 pci	0.0075	-
	708.8 to 703.8	A-2-6	G	4	14	14	125 psf	125 psf	φ = 35°	135 pci	-	-
	703.8 to 689.8	A-1-b	G	4	53	45	135 psf	135 psf	φ = 41°	315 pci	-	-
	689.8 to 679.8	A-1-b	G	4	100	74	135 psf	72.6 psf	φ = 42°	195 pci	-	-
	679.8 to 674.8	A-1-b	G	4	52	37	135 psf	72.6 psf	φ = 40°	155 pci	-	-
	674.8 to 669.8	A-6b	C	2	61	61	130 psf	67.6 psf	Su = 7,625 psf	2,540 pci	0.0035	-
	669.8 to 651.3	A-1-b	G	4	100	63	135 psf	72.6 psf	φ = 42°	195 pci	-	-
	651.3 to 641.3	Dolomite	R	9	-	-	165 psf	102.6 psf	Qu = 10,000 psi	0.00005	1,000,000 psi	85
B-108-9-15	722.4 to 714.4	A-2-4	G	4	14	21	125 psf	125 psf	φ = 37°	190 pci	-	-
	714.4 to 709.4	A-6b	C	1	6	6	115 psf	115 psf	Su = 750 psf	100 pci	0.0100	-
	709.4 to 704.4	A-4a	G	4	21	21	125 psf	125 psf	φ = 34°	115 pci	-	-
	704.4 to 701.9	A-6b	C	3	16	16	120 psf	120 psf	Su = 2,000 psf	665 pci	0.0063	-
	701.9 to 690.4	A-1-b	G	4	31	26	130 psf	130 psf	φ = 38°	215 pci	-	-
	690.4 to 685.4	A-1-a	G	4	47	35	135 psf	72.6 psf	φ = 41°	175 pci	-	-
	685.4 to 680.4	A-1-a	G	4	19	14	125 psf	62.6 psf	φ = 37°	110 pci	-	-
	680.4 to 670.4	A-1-a	G	4	40	28	130 psf	67.6 psf	φ = 40°	155 pci	-	-
	670.4 to 665.4	A-2-6	G	4	31	20	130 psf	67.6 psf	φ = 36°	95 pci	-	-
	665.4 to 660.4	A-4a	C	2	18	18	120 psf	57.6 psf	Su = 2,250 psf	750 pci	0.0060	-
	660.4 to 655.4	A-3a	G	4	22	14	125 psf	62.6 psf	φ = 34°	70 pci	-	-
	655.4 to 642.3	Limestone	R	9	-	-	165 psf	102.6 psf	Qu = 10,000 psi	0.00005	1,000,000 psi	69

APPENDIX IX

MSE WALL CALCULATIONS

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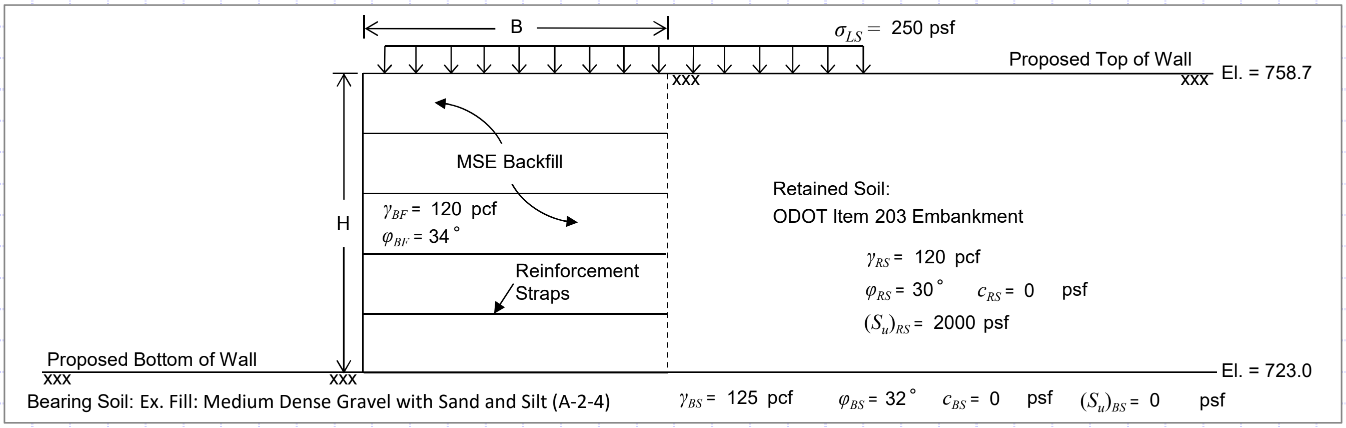
FRA-71-1518A - MSE Wall - Rear Abutment - B-108-3-13 and B-108-9-15 - 35.7 ft. Wall Height



MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	35.7	ft
MSE Wall Width (Reinforcement Length), (B) =	28.6	ft
MSE Wall Length, (L) =	32	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion [1], (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	32	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(S_u)_{BS}$] =	0	psf
Embedment Depth, (D_f) =	3.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	7.9	ft

LRFD Load Factors

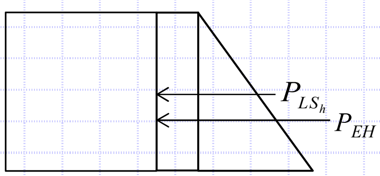
	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

(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:

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



$$P_{EH} = \tfrac{1}{2}\gamma_{RS} H^2 K_a \gamma_{EH} = \text{½(120 pcf)(35.7 ft)²(0.297)(1.5)} = 34.07 \text{ kip/ft}$$

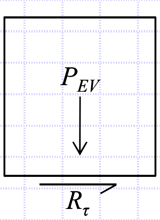
$$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS} = \text{(250 psf)(35.7 ft)(0.297)(1.75)} = 4.64 \text{ kip/ft}$$

$$P_H = \text{34.07 kip/ft + 4.64 kip/ft} = 38.71 \text{ kip/ft}$$

Check Sliding Resistance - Drained Condition

Nominal Sliding Resistance:

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



$$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV} = \text{(120 pcf)(35.7 ft)(28.6 ft)(1.00)} = 122.52 \text{ kip/ft}$$

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

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

$$R_\tau = \text{(122.52 kip/ft)(0.62)} = 75.96 \text{ kip/ft}$$

Verify Sliding Force Less Than Factored Sliding Resistance - Drained Condition

$$P_H \le R_\tau \cdot \phi_\tau \longrightarrow \text{38.71 kip/ft} \le \text{(75.96 kip/ft)(1.0) = 75.96 kip/ft} \longrightarrow \text{38.71 kip/ft} \le \text{75.96 kip/ft}$$

OK

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

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	35.7	ft
MSE Wall Width (Reinforcement Length), (B) =	28.6	ft
MSE Wall Length, (L) =	32	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	32	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	0	psf
Embedment Depth, (D_f) =	3.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	7.9	ft

LRFD Load Factors

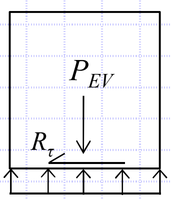
	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

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

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

Check Sliding Resistance - Undrained Condition

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



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

$q_s = \sigma_v / 2$ = (4.28 ksf) / 2 = 2.14 ksf

$\sigma_v = P_{EV} / B$ = (122.52 kip/ft) / (28.6 ft) = 4.28 ksf

R_τ = (N/A ksf ≤ 2.14 ksf)(28.6 ft) = N/A kip/ft

Verify Sliding Force Less Than Factored Sliding Resistance - Undrained Condition

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

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

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	35.7	ft
MSE Wall Width (Reinforcement Length), (B) =	28.6	ft
MSE Wall Length, (L) =	32	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, $[(S_u)_{RS}]$ =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

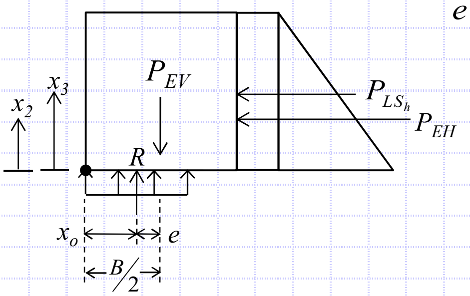
Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	32	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	0	psf
Embedment Depth, (D_f) =	3.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	7.9	ft

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



$$e = B/2 - x_o$$

$$x_o = \frac{M_{EV} - M_H}{P_{EV}}$$ = (1752.04 kip·ft/ft - 488.26 kip·ft/ft) / (122.52 kip/ft) = 10.31 ft

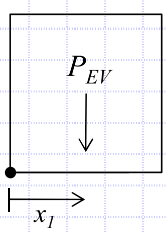
M_{EV}	=	1752.04	kip·ft/ft
M_H	=	488.26	kip·ft/ft
P_{EV}	=	122.52	kip/ft

Defined below

$e =$ (28.6 ft)/2 - 10.31 ft = 3.99 ft

Resisting Moment, M_{EV}:

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



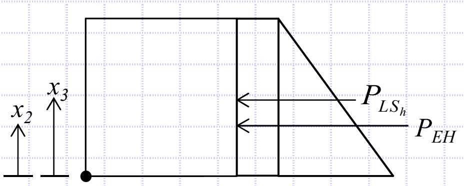
$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV}$ = (120 pcf)(35.7 ft)(28.6 ft)(1.00) = 122.52 kip/ft

$x_1 = B/2$ = (28.6 ft) / 2 = 14.30 ft

$M_{EV} =$ (122.52 kip/ft)(14.30 ft) = 1752.04 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)(35.7 ft)²(0.297)(1.5) = 34.07 kip/ft

$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS}$ = (250 psf)(35.7 ft)(0.297)(1.75) = 4.64 kip/ft

$x_2 = H/3$ = (35.7 ft) / 3 = 11.90 ft

$x_3 = H/2$ = (35.7 ft) / 2 = 17.85 ft

$M_H =$ (34.07 kip/ft)(11.9 ft) + (4.64 kip/ft)(17.85 ft) = 488.26 kip·ft/ft

Check Eccentricity

$e < e_{max} \longrightarrow$ 3.99 ft < 9.53 ft **OK**

Limiting Eccentricity: $e_{max} = B/3 \longrightarrow e_{max} =$ (28.6 ft) / 3 = 9.53 ft

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MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	35.7	ft
MSE Wall Width (Reinforcement Length), (B) =	28.6	ft
MSE Wall Length, (L) =	32	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

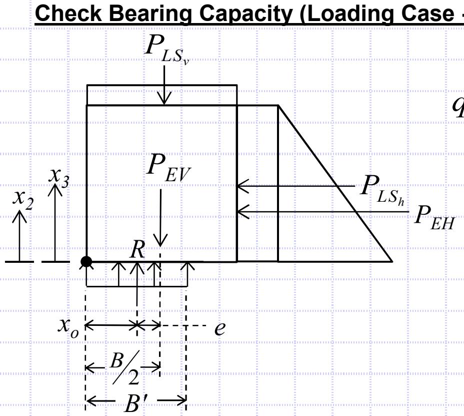
Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	1500	psf
Embedment Depth, (D_f) =	3.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	7.9	ft

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



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

$B' = B - 2e$ = 28.6 ft - 2(2.74 ft) = 23.12 ft

$e = B/2 - x_o$ = (28.6 ft) / 2 - 11.56 ft = 2.74 ft

$x_o = \dfrac{M_V - M_H}{P_V}$ = (2544.22 kip·ft/ft - 488.2 kip·ft/ft) / 177.92 kip/ft = 11.56 ft

$q_{eq} =$ (177.92 kip/ft) / (23.12 ft) = 7.70 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)(35.7 ft)(28.6 ft)(1.35)](14.3 ft) + [(250 psf)(28.6 ft)(1.75)](14.3 ft) = 2544.22 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)(35.7 ft)²(0.297)(1.5)](11.9 ft) + [(250 psf)(35.7 ft)(0.297)(1.75)](17.85 ft) = 488.20 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)(35.7 ft)(28.6 ft)(1.35) + (250 psf)(28.6 ft)(1.75) = 177.92 kip/ft

Check Bearing Resistance - Drained Condition

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

$N_{cm} = N_c s_c i_c$ = 30.82

N_c = 22.25
s_c = 1+(23.12 ft/32 ft)(11.85/22.25)
= 1.385
i_c = 1.000 (Assumed)

$N_{qm} = N_q s_q d_q i_q$ = 16.66

N_q = 11.85
s_q = 1.352
d_q = 1+2tan(26°)[1-sin(26°)]²tan⁻¹(3.0 ft/23.12 ft)
1.040
i_q = 1.000 (Assumed)
C_{wq} = 7.9 ft > 3.0 ft = 1.000

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 8.92

N_γ = 12.54
s_γ = 0.711
i_γ = 1.000 (Assumed)
$C_{w\gamma}$ = 7.9 ft < 1.5(23.12 ft) + 3.0 ft = 0.614

$q_n =$ (0 psf)(30.816) + (120 pcf)(3.0 ft)(16.662)(1.000) + ½(120 pcf)(23.1 ft)(8.916)(0.614) = 13.59 ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

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

$q_{eq} \le q_n \cdot \phi_b$ ⟶ 7.70 ksf ≤ (13.59 ksf)(0.65) = 8.83 ksf ⟶ 7.70 ksf ≤ 8.83 ksf **OK**

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MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	35.7	ft
MSE Wall Width (Reinforcement Length), (B) =	28.6	ft
MSE Wall Length, (L) =	32	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	1500	psf
Embedment Depth, (D_f) =	3.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	7.9	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: $$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$$

$N_{cm} = N_c s_c i_c$ = 5.890

- N_c = 5.140
- s_c = 1+(23.12 ft/[(5)(32 ft)]) = 1.145
- i_c = 1.000 (Assumed)

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

- N_q = 1.000
- s_q = 1.000
- d_q = 1+2tan(0°)[1-sin(0°)]²tan⁻¹(3.0 ft/23.12 ft) = 1.000
- i_q = 1.000 (Assumed)
- C_{wq} = 7.9 ft > 3.0 ft = 1.000

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

- N_γ = 0.000
- s_γ = 1.000
- i_γ = 1.000 (Assumed)
- $C_{w\gamma}$ = 7.9 ft < 1.5(23.12 ft) + 3.0 ft = 0.614

q_n = (1500 psf)(5.890) + (120 pcf)(3.0 ft)(1.000)(1.000) + ½(120 pcf)(23.1 ft)(0.000)(0.614) = 9.20 ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

$q_{eq} \le q_n \cdot \phi_b$ ⟶ 7.70 ksf ≤ (9.20 ksf)(0.65) = 5.98 ksf ⟶ 7.70 ksf ≤ 5.98 ksf **ERROR!!**

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

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MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	35.7	ft
MSE Wall Width (Reinforcement Length), (B) =	28.6	ft
MSE Wall Length, (L) =	32	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{BS}) =	0	psf
Retained Soil Undrained Shear Strength, [(S_u)$_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.297	
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°

Bearing Soil Properties:

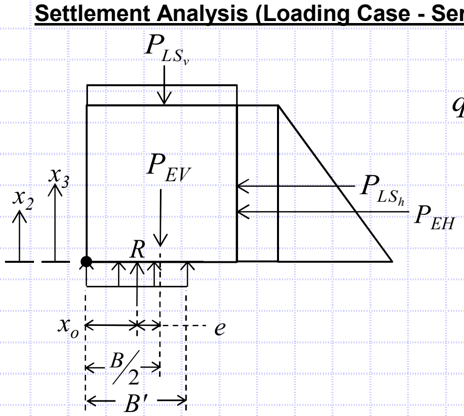
Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	32	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [(s_u)$_{BS}$] =	0	psf
Embedment Depth, (D_f) =	3.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	7.9	ft

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



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

$B' = B - 2e$ = 28.6 ft - 2(2.45 ft) = 23.70 ft

$e = B/2 - x_o$ = (28.6 ft) / 2 - 11.85 ft = 2.45 ft

$x_o = \dfrac{M_V - M_H}{P_V}$ = (1854.32 kip·ft/ft - 317.58 kip·ft/ft) / 129.67 kip/ft = 11.85 ft

$q_{eq} =$ (129.67 kip/ft) / (23.7 ft) = 5.47 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)(35.7 ft)(28.6 ft)(1.00)](14.3 ft) + [(250 psf)(28.6 ft)(1.00)](14.3 ft) = 1854.32 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)(35.7 ft)²(0.297)(1.00)](11.9 ft) + [(250 psf)(35.7 ft)(0.297)(1.00)](17.85 ft) = 317.58 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)(35.7 ft)(28.6 ft)(1.00) + (250 psf)(28.6 ft)(1.00) = 129.67 kip/ft

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

Total Settlement at Center of Reinforced Soil Mass:	S_t =	**5.323**	in
Total Settlement at Wall Facing:	S_t =	**3.156**	in

Time Rate of Consolidation and Downdrag Depths and Loads:

Hold Period	Degree of Consolidation	Settlement at Completion of Hold Period	Depth of Downdrag
15 days	100 %	3.156 in	0.0 ft

W-13-045 - FRA-70-12.86 - FRA-71-1518A / Retaining Wall 4W11

MSE Wall Settlement - Rear Abutment

Calculated By: BRT Date: 7/4/2018

Checked By: JPS Date: 7/5/2018

Boring B-108-3-13 and B-108-9-15

H=	35.7	ft	Total wall height
B'=	23.7	ft	Effective footing width due to eccentricity
D_w =	7.9	ft	Depth below bottom of footing
q_e =	5,470	psf	Equivalent bearing pressure at bottom of wall

																				Total Settlement at Center of Reinforced Soil Mass					Total Settlement at Facing of Wall				
Layer	Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
1	A-2-4	G	0.0	4.0	4.0	2.0	125	500	250	250	3,250					14	24	83	0.08	0.998	5,459	5,709	0.066	0.789	0.500	2,734	2,984	0.052	0.625
	A-2-4	G	4.0	8.5	4.5	6.3	125	1,063	781	781	3,781					14	18	71	0.26	0.954	5,217	5,998	0.056	0.673	0.496	2,715	3,497	0.041	0.495
2	A-6b	C	8.5	11.0	2.5	9.8	115	1,350	1,206	1,091	4,091	37	0.243	0.012	0.561				0.41	0.874	4,781	5,872	0.072	0.867	0.488	2,668	3,759	0.010	0.125
	A-6b	C	11.0	13.5	2.5	12.3	115	1,638	1,494	1,222	4,222	37	0.243	0.024	0.561				0.52	0.808	4,417	5,640	0.070	0.838	0.478	2,615	3,837	0.019	0.232
3	A-1-b	G	13.5	16.0	2.5	14.8	125	1,950	1,794	1,366	4,366					19	21	77	0.62	0.742	4,057	5,423	0.019	0.232	0.466	2,547	3,913	0.015	0.177
4	A-4a	G	16.0	18.5	2.5	17.3	125	2,263	2,106	1,523	4,523					23	25	47	0.73	0.680	3,721	5,244	0.028	0.339	0.451	2,468	3,991	0.022	0.264
5	A-6b	C	18.5	21.0	2.5	19.8	120	2,563	2,413	1,673	4,673	37	0.243	0.024	0.561				0.83	0.625	3,418	5,091	0.032	0.382	0.435	2,382	4,055	0.015	0.180
6	A-1-b	G	21.0	26.5	5.5	23.8	130	3,278	2,920	1,931	4,931					31	31	103	1.00	0.549	3,003	4,934	0.022	0.261	0.409	2,236	4,167	0.018	0.214
	A-1-b	G	26.5	32.5	6.0	29.5	130	4,058	3,668	2,320	5,320					31	30	98	1.24	0.463	2,535	4,854	0.020	0.237	0.371	2,029	4,348	0.017	0.201
7	A-1-a	G	32.5	42.5	10.0	37.5	130	5,358	4,708	2,860	5,860					37	33	107	1.58	0.378	2,066	4,927	0.022	0.266	0.323	1,768	4,628	0.020	0.235
	A-1-a	G	42.5	52.5	10.0	47.5	130	6,658	6,008	3,536	6,536					37	30	99	2.00	0.305	1,669	5,206	0.017	0.204	0.274	1,501	5,038	0.016	0.186
8	A-2-6	G	52.5	57.5	5.0	55.0	130	7,308	6,983	4,043	7,043					31	24	83	2.32	0.266	1,456	5,499	0.008	0.097	0.245	1,341	5,385	0.008	0.090
9	A-4a	C	57.5	62.5	5.0	60.0	120	7,908	7,608	4,356	7,356	18	0.072	0.007	0.413				2.53	0.245	1,341	5,697	0.003	0.036	0.229	1,250	5,606	0.003	0.034
10	A-3a	G	62.5	67.5	5.0	65.0	125	8,533	8,220	4,657	7,657					22	16	60	2.74	0.227	1,242	5,899	0.009	0.103	0.214	1,169	5,826	0.008	0.097
																					Total Settlement:			5.323 in		Total Settlement:			3.156 in

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

W-13-045 - FRA-70-12.86 - FRA-71-1518A / Retaining Wall 4W11
MSE Wall Settlement - Rear Abutment

Calculated By: BRT Date: 07/04/2018
Checked By: JPS Date: 07/05/2018

Boring B-108-3-13 and B-108-9-15

H=	35.7	ft	Total wall height
B'=	23.7	ft	Effective footing width due to eccentricity
D_w =	7.9	ft	Depth below bottom of footing
q_e =	5,470	psf	Equivalent bearing pressure at bottom of wall

	A-6b	A-4a			
c_v =	400	1000		ft^2/yr	Coefficient of consolidation
t =	15	15		days	Time following completion of construction
H_{dr} =	2.5	2.5		ft	Length of longest drainage path considered
T_v =	2.630	6.575			Time factor
U =	100	100		%	Degree of consolidation

$(S_c)_t$ = 3.156 in Settlement complete at 100% of primary consolidation

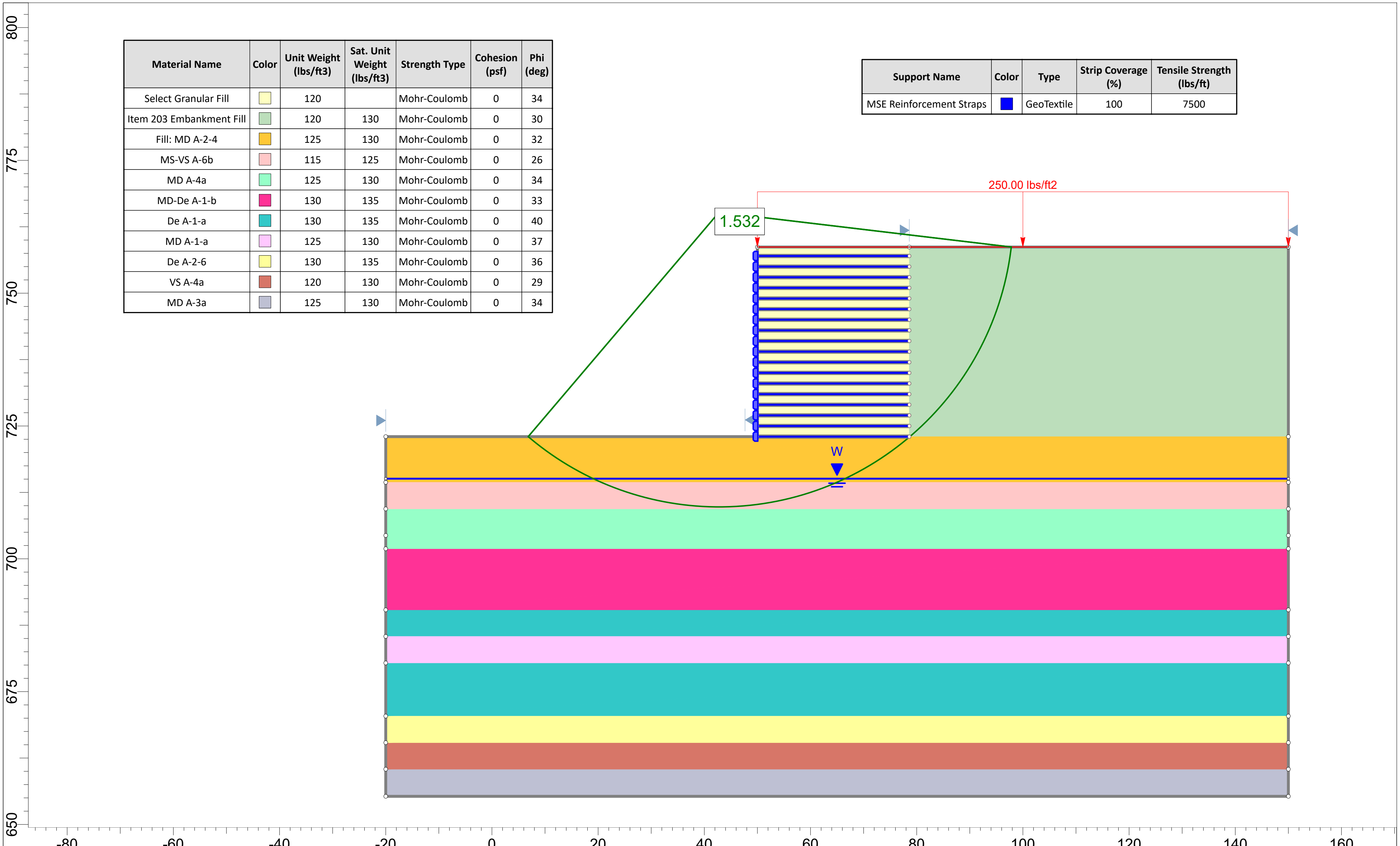
Layer	Soil Type	Soil Type	Layer Depth (ft)		Layer Thickness (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	Total Settlement at Facing of Wall: S_c (9,10) (ft)	Total Settlement at Facing of Wall: S_c (in)	Total Settlement at Facing of Wall: Layer Settlement (in)	Settlement Complete at 100% of Primary Consolidation: $(S_c)_t$ (11) (in)	Settlement Complete at 100% of Primary Consolidation: Layer Settlement (in)
1	A-2-4	G	0.0	4.0	4.0	2.0	125	500	250	250	4,250					14	24	83	0.08	0.500	2,734	2,984	0.052	0.625	1.120	0.625	1.120
	A-2-4	G	4.0	8.5	4.5	6.3	125	1,063	781	781	4,781					14	18	71	0.26	0.496	2,715	3,497	0.041	0.495		0.495	
2	A-6b	C	8.5	11.0	2.5	9.8	115	1,350	1,206	1,091	5,091	37	0.243	0.012	0.561				0.41	0.488	2,668	3,759	0.010	0.125	0.357	0.125	0.357
	A-6b	C	11.0	13.5	2.5	12.3	115	1,638	1,494	1,222	5,222	37	0.243	0.024	0.561				0.52	0.478	2,615	3,837	0.019	0.232		0.232	
3	A-1-b	G	13.5	16.0	2.5	14.8	125	1,950	1,794	1,366	5,366					19	21	77	0.62	0.466	2,547	3,913	0.015	0.177	0.177	0.177	0.177
4	A-4a	G	16.0	18.5	2.5	17.3	125	2,263	2,106	1,523	5,523					23	25	47	0.73	0.451	2,468	3,991	0.022	0.264	0.264	0.264	0.264
5	A-6b	C	18.5	21.0	2.5	19.8	120	2,563	2,413	1,673	5,673	37	0.243	0.024	0.561				0.83	0.435	2,382	4,055	0.015	0.180	0.180	0.180	0.180
6	A-1-b	G	21.0	26.5	5.5	23.8	130	3,278	2,920	1,931	5,931					31	31	103	1.00	0.409	2,236	4,167	0.018	0.214	0.416	0.214	0.416
	A-1-b	G	26.5	32.5	6.0	29.5	130	4,058	3,668	2,320	6,320					31	30	98	1.24	0.371	2,029	4,348	0.017	0.201		0.201	
7	A-1-a	G	32.5	42.5	10.0	37.5	130	5,358	4,708	2,860	6,860					37	33	107	1.58	0.323	1,768	4,628	0.020	0.235	0.422	0.235	0.422
	A-1-a	G	42.5	52.5	10.0	47.5	130	6,658	6,008	3,536	7,536					37	30	99	2.00	0.274	1,501	5,038	0.016	0.186		0.186	
8	A-2-6	G	52.5	57.5	5.0	55.0	130	7,308	6,983	4,043	8,043					31	24	83	2.32	0.245	1,341	5,385	0.008	0.090	0.090	0.090	0.090
9	A-4a	C	57.5	62.5	5.0	60.0	120	7,908	7,608	4,356	8,356	18	0.072	0.007	0.413				2.53	0.229	1,250	5,606	0.003	0.034	0.034	0.034	0.034
10	A-3a	G	62.5	67.5	5.0	65.0	125	8,533	8,220	4,657	8,657					22	16	60	2.74	0.214	1,169	5,826	0.008	0.097	0.097	0.097	0.097

Settlement Remaining After Hold Period: 0.000 in

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

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Select Granular Fill		120		Mohr-Coulomb	0	34
Item 203 Embankment Fill		120	130	Mohr-Coulomb	0	30
Fill: MD A-2-4		125	130	Mohr-Coulomb	0	32
MS-VS A-6b		115	125	Mohr-Coulomb	0	26
MD A-4a		125	130	Mohr-Coulomb	0	34
MD-De A-1-b		130	135	Mohr-Coulomb	0	33
De A-1-a		130	135	Mohr-Coulomb	0	40
MD A-1-a		125	130	Mohr-Coulomb	0	37
De A-2-6		130	135	Mohr-Coulomb	0	36
VS A-4a		120	130	Mohr-Coulomb	0	29
MD A-3a		125	130	Mohr-Coulomb	0	34

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

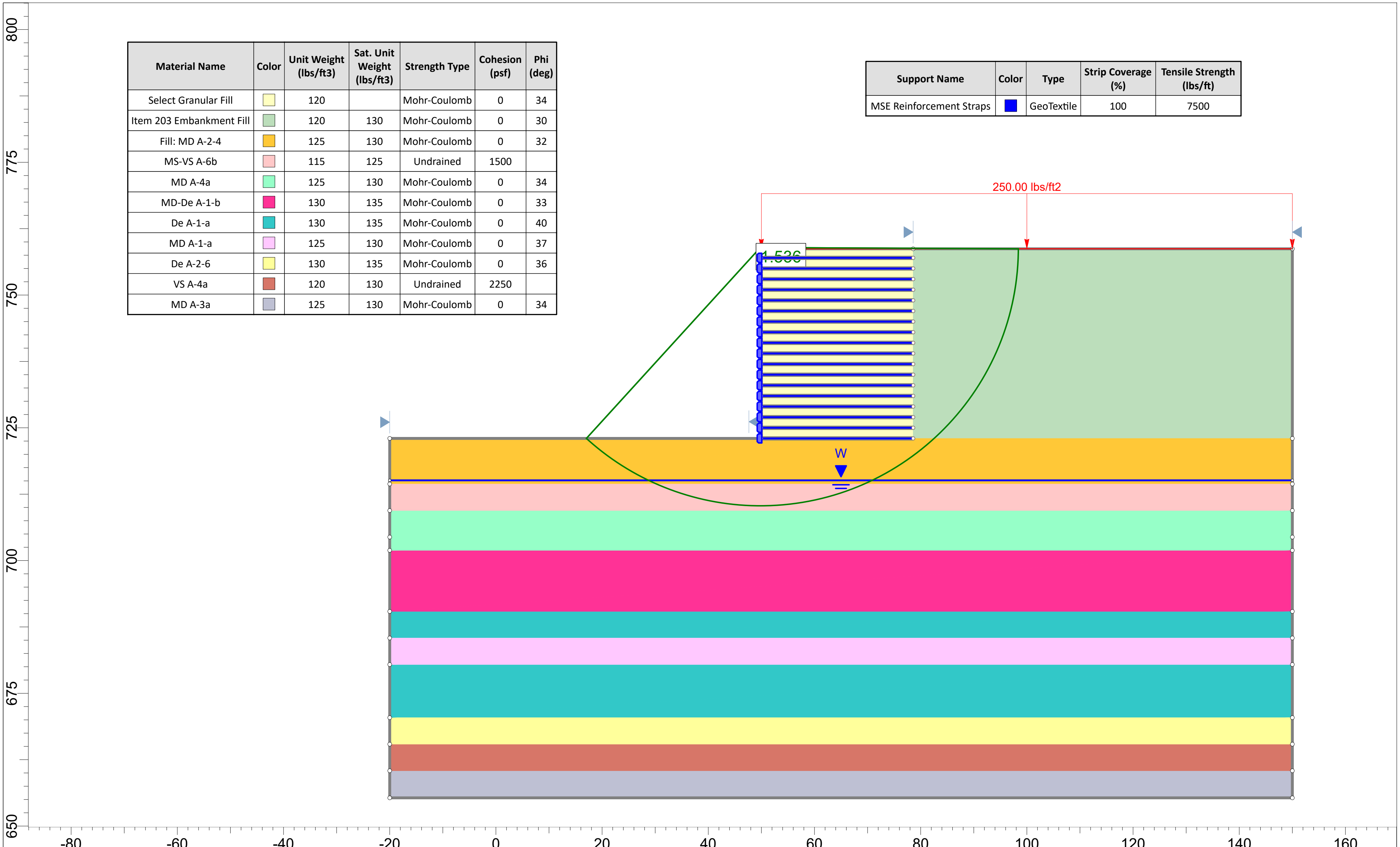


Project	Retaining Wall 4W11 - Sta. 0+00 to 1+78 - MSE Wall Global Stability		
Analysis Description	B-108-9-13 - 35.7 ft Wall Height - Drained - Circular - Spencer		
Drawn By	BRT	Scale	1:300
Company	Resource International, Inc.		
Date	7/4/2018	File Name	Retaining Wall 4W11 - Abutment Section - MSE Wall Global Stability.slim

SLIDEINTERPRET 7.020

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Select Granular Fill		120		Mohr-Coulomb	0	34
Item 203 Embankment Fill		120	130	Mohr-Coulomb	0	30
Fill: MD A-2-4		125	130	Mohr-Coulomb	0	32
MS-VS A-6b		115	125	Undrained	1500	
MD A-4a		125	130	Mohr-Coulomb	0	34
MD-De A-1-b		130	135	Mohr-Coulomb	0	33
De A-1-a		130	135	Mohr-Coulomb	0	40
MD A-1-a		125	130	Mohr-Coulomb	0	37
De A-2-6		130	135	Mohr-Coulomb	0	36
VS A-4a		120	130	Undrained	2250	
MD A-3a		125	130	Mohr-Coulomb	0	34

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



Project
Retaining Wall 4W11 - Sta. 0+00 to 1+78 - MSE Wall Global Stability

Analysis Description
B-108-9-13 - 35.7 ft Wall Height - Undrained - Circular - Spencer

Drawn By BRT | *Scale* 1:300 | *Company* Resource International, Inc.

Date 7/4/2018 | *File Name* Retaining Wall 4W11 - Abutment Section - MSE Wall Global Stability.slim

SLIDEINTERPRET 7.020

RESOURCE INTERNATIONAL, INC.
6350 PRESIDENTIAL GATEWAY
COLUMBUS, OHIO 43231
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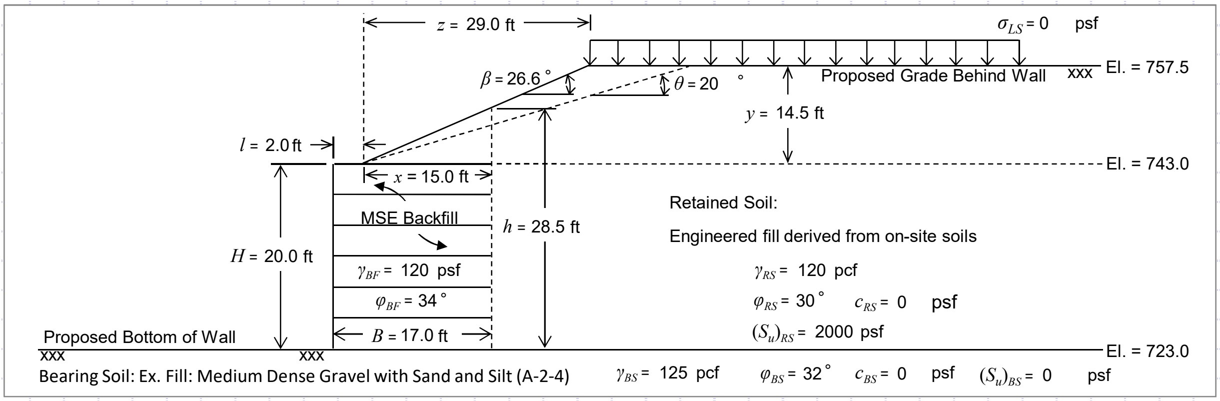
JOB	FRA-70-12.68	NO.	W-13-045
SHEET NO.	1	OF	6
CALCULATED BY	BRT	DATE	7/4/2018
CHECKED BY	JPS	DATE	7/5/2018

Retaining Wall 4W11 - Sta. 1+78 to 3+38

Retaining Wall 4W11 - Sta. 1+78 to 2+30 - 2:1 Backslope - 20.0 ft. Wall Height - B-108-2-13, B-108-3-13 and B-108-9-15



MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	20.0	ft
MSE Wall Width (Reinforcement Length), (B) =	17.0	ft
Distance from Wall Face to Toe of Backslope, (l) =	2.0	ft
MSE Wall Length, (L) =	160.0	ft
MSE Wall Effective Height, (h) =	28.5	ft
Retained Soil Backslope, (β) =	26.6	°
Effective Retained Soil Backslope, (θ) =	20	°
Distance from Toe to Top of Backslope, (z) =	29.0	ft
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{RS}) =	0	psf
Retained Soil Undrained Shear Strength, $[(S_u)_{RS}]$ =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.414	
Live Surcharge Load, (σ_{LS}) =	0	psf

MSE Backfill and Bearing Soil Properties:

MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°
Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	32	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, $[(S_u)_{BS}]$ =	0	psf
Embedment Depth, (D_f) =	3.0	ft
Depth to GW (Below Bot. of Wall), (D_W) =	7.9	ft

LRFD Load Factors

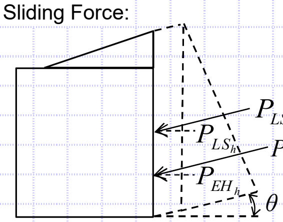
	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

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

1. Drained cohesion for retained soil not accounted for in external stability analyses. This parameter is utilized in global stability analysis.

Check Sliding (Loading Case - Strength Ia) - AASHTO LRFD BDM Sections 11.6.3.6 and 11.10.5.3

Sliding Force:



$$P_H = (P_{EH} + P_{LS})\cos\theta$$

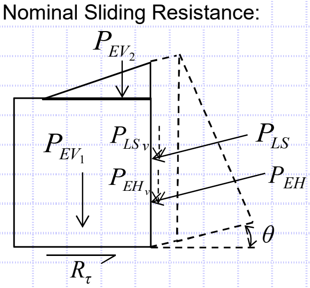
$P_{EH} = \frac{1}{2}\gamma_{RS} h^2 K_a \gamma_{EH}$ = ½(120 pcf)(28.5 ft)²(0.414)(1.50) = 30.29 kip/ft

$P_{LS} = \sigma_{LS} h K_a \gamma_{LS}$ = (0 psf)(28.5 ft)(0.414)(1.75) = 0.00 kip/ft

P_H = (30.29 kip/ft + 0.00 kip/ft)cos(20°) = 28.46 kip/ft

Check Sliding Resistance - Drained Condition

Nominal Sliding Resistance: $R_\tau = (P_{EV_1} + P_{EV_2} + P_{EH}\sin\theta)\tan\delta$ (Neglect P_{LS_v} for conservatism)



$P_{EV_1} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV}$ = (120 pcf)(20.0 ft)(17.0 ft)(1.00) = 40.8 kip/ft

$P_{EV_2} = \frac{1}{2}\gamma_{RS}(h-H)(B-l)\gamma_{EV}$ = ½(120 pcf)(28.5 ft - 20.0 ft)(17.0 ft - 2.0 ft)(1.00) = 7.66 kip/ft

$P_{EH} = \frac{1}{2}\gamma_{RS} h^2 K_a \gamma_{EH}$ = ½(120 pcf)(28.5 ft)²(0.414)(1.50) = 30.29 kip/ft

$\tan\delta = (\tan\varphi_{BS} \le \tan\varphi_{BF}) \longrightarrow$ tan(32) ≤ tan(34) $\longrightarrow$ 0.62 ≤ 0.67 = 0.62

R_τ = [40.80 kip/ft + 7.66 kip/ft + (30.29 kip/ft)sin(20°)](0.62) = 36.47 kip/ft

Verify Sliding Force Less Than Factored Sliding Resistance - Drained Condition

$P_H \le R_\tau \cdot \phi_\tau \longrightarrow$ 28.46 kip/ft ≤ (36.47 kip/ft)(1.0) = 36.47 kip/ft $\longrightarrow$ 28.46 kip/ft ≤ 36.47 kip/ft **OK**

Use φ_τ = **1.0** *(Per AASHTO LRFD BDM Table 11.5.6-1)*

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MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	20.0	ft
MSE Wall Width (Reinforcement Length), (B) =	17.0	ft
Distance from Wall Face to Toe of Backslope, (l) =	2.0	ft
MSE Wall Length, (L) =	160.0	ft
MSE Wall Effective Height, (h) =	28.5	ft
Retained Soil Backslope, (β) =	26.6	°
Effective Retained Soil Backslope, (θ) =	20	°
Distance from Toe to Top of Backslope, (z) =	29.0	ft
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{RS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.414	
Live Surcharge Load, (σ_{LS}) =	0	psf

Bearing Soil Properties:

MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°
Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	32	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	0	psf
Embedment Depth, (D_f) =	3.0	ft
Depth to GW (Below Bot. of Wall), (D_W) =	7.9	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

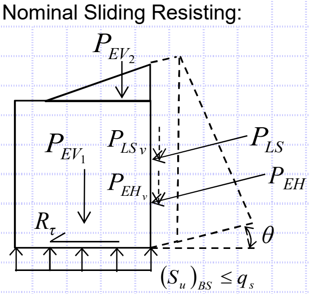
(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

1. Drained cohesion for retained soil not accounted for in external stability analyses. This parameter is utilized in global stability analysis.

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

Check Sliding Resistance - Undrained Condition

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



(Neglect $P_{LS\,v}$ for conservatism)

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

$$q_s = \sigma_v / 2$$

$$\sigma_v = P_V / B$$

$$P_V = P_{EV_1} + P_{EV_2} + P_{EH} \sin\theta$$

$P_{EV_1} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV}$ = (120 pcf)(20.0 ft)(17.0 ft)(1.00) = 40.8 kip/ft

$$P_{EV_2} = \tfrac{1}{2}\gamma_{RS}(h-H)(B-l)\gamma_{EV}$$

P_{EV_2} = ½(120 pcf)(28.5 ft - 20.0 ft)(17.0 ft - 2.0 ft)(1.00) = 7.66 kip/ft

$P_{EH} = \tfrac{1}{2}\gamma_{RS} h^2 K_a \gamma_{EH}$ = ½(120 pcf)(28.5 ft)²(0.414)(1.50) = 30.29 kip/ft

P_V = 40.8 kip/ft + 7.66 kip/ft + (30.29 kip/ft)sin(20°) = 58.82 kip/ft

σ_v = (58.82 kip/ft) / (17.0 ft) = 3.46 ksf

q_s = (3.46 ksf) / 2 = 1.73 ksf

R_τ = (N/A ksf ≤ 1.73 ksf)(17.0 ft) = N/A kip/ft

Verify Sliding Force Less Than Factored Sliding Resistance - Undrained Condition

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

Use φ_τ = **1.0** *(Per AASHTO LRFD BDM Table 11.5.6-1)*

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MSE Wall Length, (L) =	160.0	ft
MSE Wall Effective Height, (h) =	28.5	ft
Retained Soil Backslope, (β) =	26.6	°
Effective Retained Soil Backslope, (θ) =	20	°
Distance from Toe to Top of Backslope, (z) =	29.0	ft
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{RS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.414	
Live Surcharge Load, (σ_{LS}) =	0	psf

Bearing Soil Properties:

MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°
Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	32	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	0	psf
Embedment Depth, (D_f) =	3.0	ft
Depth to GW (Below Bot. of Wall), (D_W) =	7.9	ft

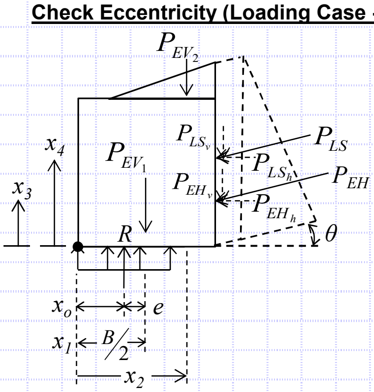
LRFD Load Factors

	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)

1. Drained cohesion for retained soil not accounted for in external stability analyses. This parameter is utilized in global stability analysis.

Check Eccentricity (Loading Case - Strength Ia) - AASHTO LRFD BDM Section 11.6.3.3



$$e = B/2 - x_o$$

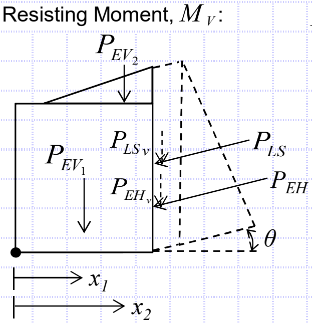
$$x_o = \frac{M_V - M_H}{P_V}$$ = (614.84 kip·ft/ft - 270.4 kip·ft/ft) / (58.82 kip/ft) = 5.86 ft

M_V = 614.84 kip·ft/ft
M_H = 270.40 kip·ft/ft
Defined below

$P_V = P_{EV_1} + P_{EV_2} + P_{EH}\sin\theta$ = 40.8 kip/ft + 7.66 kip/ft + (30.29 kip/ft)sin(20°) = 58.82 kip/ft

$e =$ (17.0 ft/ 2) - 5.86 ft = 2.64 ft

Resisting Moment, M_V:



$$M_V = P_{EV_1}(x_1) + P_{EV_2}(x_2) + P_{EH}\sin\theta(B)$$ (Neglect P_{LS_v} for conservatism)

$P_{EV_1} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV}$ = (120 pcf)(20.0 ft)(17.0 ft)(1.00) = 40.80 kip/ft

$P_{EV_2} = \tfrac{1}{2}\gamma_{RS}(h-H)(B-l)\gamma_{EV}$ = ½(120 pcf)(28.5 ft - 20.0 ft)(17.0 ft - 2.0 ft)(1.00) = 7.66 kip/ft

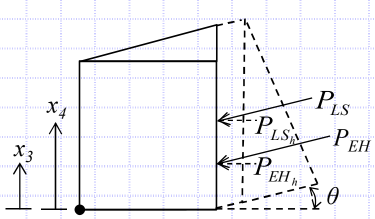
$P_{EH} = \tfrac{1}{2}\gamma_{RS}h^2K_a\gamma_{EH}$ = ½(120 pcf)(28.5 ft)²(0.414)(1.50) = 30.29 kip/ft

$x_1 = B/2$ = (17.0 ft) / 2 = 8.50 ft

$x_2 = l + \tfrac{2}{3}(B-l)$ = 2.0 ft + ⅔(17.0 ft - 2.0 ft) = 12.00 ft

M_V = (40.8 kip/ft)(8.50 ft) + (7.66 kip/ft)(12 ft) + (30.29 kip/ft)sin(20°)(17 ft) = 614.84 kip·ft/ft

Overturning Moment, M_H:



$$M_H = P_{EH}\cos\theta(x_3) + P_{LS}\cos\theta(x_4)$$

$P_{EH} = \tfrac{1}{2}\gamma_{RS}h^2K_a\gamma_{EH}$ = ½(120 pcf)(28.5 ft)²(0.414)(1.50) = 30.29 kip/ft

$P_{LS} = \sigma_{LS}hK_a\gamma_{LS}$ = (0 psf)(28.5 ft)(0.414)(1.75) = 0.00 kip/ft

$x_3 = h/3$ = (28.5 ft) / 3 = 9.50 ft

$x_4 = h/2$ = (28.5 ft) / 2 = 14.26 ft

M_H = (30.29 kip/ft)cos(20°)(9.50 ft) + (0 kip/ft)cos(20°)(14.26 ft) = 270.40 kip·ft/ft

Check Eccentricity

Limiting Eccentricity: $e_{\max} = B/3 \longrightarrow e_{\max} =$ (17.0 ft) / 3 = 5.67 ft

$e < e_{\max} \longrightarrow$ 2.64 ft < 5.67 ft **OK**

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Distance from Wall Face to Toe of Backslope, (l) =	2.0	ft
MSE Wall Length, (L) =	160.0	ft
MSE Wall Effective Height, (h) =	28.5	ft
Retained Soil Backslope, (β) =	26.6	°
Effective Retained Soil Backslope, (θ) =	20	°
Distance from Toe to Top of Backslope, (z) =	29.0	ft
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{RS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.414	
Live Surcharge Load, (σ_{LS}) =	0	psf

Bearing Soil Properties:

MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°
Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	1500	psf
Embedment Depth, (D_f) =	3.0	ft
Depth to GW (Below Bot. of Wall), (D_W) =	7.9	ft

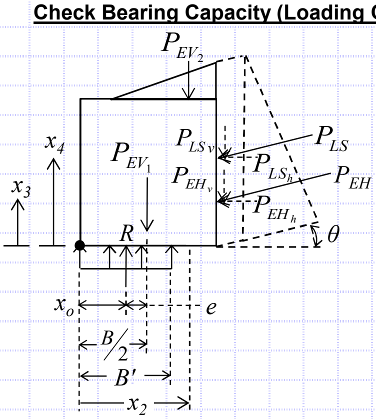
LRFD Load Factors

	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)

1. Drained cohesion for retained soil not accounted for in external stability analyses. This parameter is utilized in global stability analysis.

Check Bearing Capacity (Loading Case - Strength Ib) - AASHTO LRFD BDM Section 11.6.3.2



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

$$B' = B - 2e \quad = \quad 17.0\text{ ft} - 2(1.93\text{ ft}) \quad = \quad 13.14 \quad \text{ft}$$

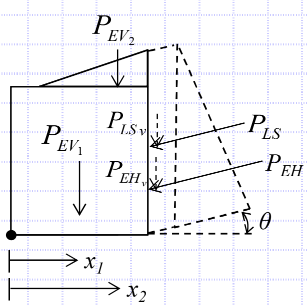
$$e = B/2 - x_o \quad = \quad (17.0\text{ ft} / 2) - 6.57\text{ ft} \quad = \quad 1.93 \quad \text{ft}$$

$$x_o = \frac{M_V - M_H}{P_V} \quad = \quad (768.38\text{ kip·ft/ft} - 270.40\text{ kip·ft/ft}) / 75.78\text{ kip/ft} \quad = \quad 6.57 \quad \text{ft}$$

$$q_{eq} = \quad (75.78\text{ kip/ft}) / (13.14\text{ ft}) \quad = \quad 5.77 \quad \text{ksf}$$

Resisting Moment, M_V:

$$M_V = P_{EV_1}(x_1) + P_{EV_2}(x_2) + P_{EH}\sin\theta(B)$$



$$P_{EV_1} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV} \quad = \quad (120\text{ pcf})(20.0\text{ ft})(17.0\text{ ft})(1.35) \quad = \quad 55.08 \quad \text{kip/ft}$$

$$P_{EV_2} = \tfrac{1}{2}\gamma_{RS}(h - H)(B - l)\gamma_{EV} = \tfrac{1}{2}(120\text{ pcf})(28.5\text{ ft} - 20.0\text{ ft})(17.0\text{ ft} - 2.0\text{ ft})(1.35) = \quad 10.34 \quad \text{kip/ft}$$

$$P_{EH} = \tfrac{1}{2}\gamma_{RS}h^2K_a\gamma_{EH} = \quad \tfrac{1}{2}(120\text{ pcf})(28.5\text{ ft})^2(0.414)(1.50) \quad = \quad 30.29 \quad \text{kip/ft}$$

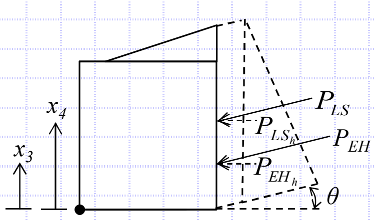
$$x_1 = B/2 \quad = \quad (17.0\text{ ft}) / 2 \quad = \quad 8.50 \quad \text{ft}$$

$$x_2 = l + \tfrac{2}{3}(B - l) \quad = \quad 2.0\text{ ft} + \tfrac{2}{3}(17.0\text{ ft} - 2.0\text{ ft}) \quad = \quad 12.00 \quad \text{ft}$$

$$M_V = \quad (55.08\text{ kip/ft})(8.50\text{ ft}) + (10.34\text{ kip/ft})(12.0\text{ ft}) + (30.29\text{ kip/ft})\sin(20°)(17\text{ ft}) \quad = \quad 768.38 \quad \text{kip·ft/ft}$$

Overturning Moment, M_H:

$$M_H = P_{EH}\cos\theta(x_3) + P_{LS}\cos\theta(x_4)$$



$$P_{EH} = \tfrac{1}{2}\gamma_{RS}h^2K_a\gamma_{EH} = \quad \tfrac{1}{2}(120\text{ pcf})(28.5\text{ ft})^2(0.414)(1.50) \quad = \quad 30.29 \quad \text{kip/ft}$$

$$P_{LS} = \sigma_{LS}hK_a\gamma_{LS} \quad = \quad (0\text{ psf})(28.5\text{ ft})(0.414)(1.75) \quad = \quad 0.00 \quad \text{kip/ft}$$

$$x_3 = h/3 \quad = \quad (28.5\text{ ft}) / 3 \quad = \quad 9.50 \quad \text{ft}$$

$$x_4 = h/2 \quad = \quad (28.5\text{ ft}) / 2 \quad = \quad 14.26 \quad \text{ft}$$

$$M_H = \quad (30.29\text{ kip/ft})\cos(20°)(9.50\text{ ft}) + (0\text{ kip/ft})\cos(20°)(14.26\text{ ft}) \quad = \quad 270.40 \quad \text{kip·ft/ft}$$

Vertical Forces, P_V:

$$P_V = P_{EV_1} + P_{EV_2} + P_{EH}\sin\theta$$

$$P_V = \quad 55.08\text{ kip/ft} + 10.34\text{ kip/ft} + (30.29\text{ kip/ft})\sin(20°) \quad = \quad 75.78 \quad \text{kip/ft}$$

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Effective Retained Soil Backslope, (θ) =	20	°
Distance from Toe to Top of Backslope, (z) =	29.0	ft
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{RS}) =	0	psf
Retained Soil Undrained Shear Strength, [(S_u)$_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.414	
Live Surcharge Load, (σ_{LS}) =	0	psf

Bearing Soil Properties:

MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°
Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [(s_u)$_{BS}$] =	1500	psf
Embedment Depth, (D_f) =	3.0	ft
Depth to GW (Below Bot. of Wall), (D_W) =	7.9	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)

1. Drained cohesion for retained soil not accounted for in external stability analyses. This parameter is utilized in global stability analysis.

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

Check Bearing Resistance - Drained Condition

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

$N_{cm} = N_c s_c i_c$ = 22.25

N_c = 22.25
s_c = 1+(13.14 ft/160 ft)(11.85/22.25)
= 1.000
i_c = 1.000 (Assumed)

$N_{qm} = N_q s_q d_q i_q$ = 14.3

N_q = 11.85
s_q = 1+(13.14 ft/160 ft)tan(26°) = 1.100
d_q = 1+2tan(26°)[1-sin(26°)]²tan⁻¹(3.0 ft/13.14 ft)
= 1.100
i_q = 1.000 (Assumed)
C_{wq} = 7.9 ft > 3.0 ft = 1.000

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 12.5

N_γ = 12.54
s_γ = 1-0.4(13.14 ft/160 ft) = 1.000
i_γ = 1.000 (Assumed)
$C_{w\gamma}$ = 7.9 ft < 1.5(13.14 ft) + 3.0 ft = 0.500

$q_n =$ (0 psf)(22.25) + (120 pcf)(3.0 ft)(14.3)(1.0) + ½(120 pcf)(13.1 ft)(12.5)(0.5) = 10.11 ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

$q_{eq} \le q_n \cdot \phi_b$ ⟶ 5.77 ksf ≤ (10.11 ksf)(0.65) = 6.57 ksf ⟶ 5.77 ksf ≤ 6.57 ksf **OK**

Use φ_b = **0.65** *(Per AASHTO LRFD BDM Table 11.5.6-1)*

Check Bearing Resistance - Undrained Condition

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

$N_{cm} = N_c s_c i_c$ = 5.140

N_c = 5.140
s_c = 1+(13.14 ft/[(5)(160 ft)] = 1.000
i_c = 1.000 (Assumed)

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

N_q = 1.000
s_q = 1.000
d_q = 1+2tan(0°)[1-sin(0°)]²tan⁻¹(3.0 ft/13.14 ft)
= 1.000
i_q = 1.000 (Assumed)
C_{wq} = 7.9 ft > 3.0 ft = 1.000

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

N_γ = 0.000
s_γ = 1.000
i_γ = 1.000 (Assumed)
$C_{w\gamma}$ = 7.9 ft < 1.5(13.14 ft) + 3.0 ft = 0.500

$q_n =$ (1500 psf)(5.14) + (120 pcf)(3.0 ft)(1.0)(1.0) + ½(120 pcf)(13.1 ft)(0.0)(0.5) = 8.07 ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

$q_{eq} \le q_n \cdot \phi_b$ ⟶ 5.77 ksf ≤ (8.07 ksf)(0.65) = 5.25 ksf ⟶ 5.77 ksf ≤ 5.25 ksf **ERROR!!**

Use φ_b = **0.65** *(Per AASHTO LRFD BDM Table 11.5.6-1)*

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Effective Retained Soil Backslope, (θ) =	20	°
Distance from Toe to Top of Backslope, (z) =	29.0	ft
Retained Soil Unit Weight, (γ_{RS}) =	120	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Drained Cohesion, (c_{RS}) =	0	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	2000	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.414	
Live Surcharge Load, (σ_{LS}) =	0	psf

1. Drained cohesion for retained soil not accounted for in external stability analyses. This parameter is utilized in global stability analysis.

Bearing Soil Properties:

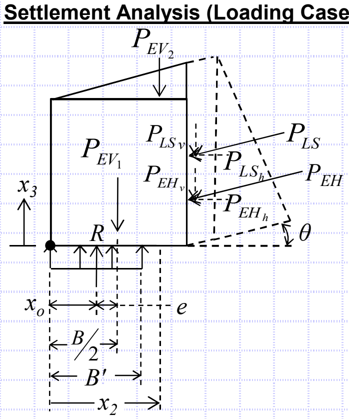
MSE Backfill Unit Weight, (γ_{BF}) =	120	pcf
MSE Backfill Friction Angle, (φ_{BF}) =	34	°
Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	32	°
Bearing Soil Drained Cohesion, (c_{BS}) =	0	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	0	psf
Embedment Depth, (D_f) =	3.0	ft
Depth to GW (Below Bot. of Wall), (D_W) =	7.9	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



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

$B' = B - 2e$ = 17.0 ft - 2(1.71 ft) = 13.58 ft

$e = B/2 - x_o$ = (17.0 ft / 2) - 6.79 ft = 1.71 ft

$x_o = \dfrac{M_V - M_H}{P_V}$ = (556.16 kip·ft/ft - 180.36 kip·ft/ft) / 55.37 kip/ft = 6.79 ft

q_{eq} = (55.37 kip/ft) / (13.58 ft) = 4.08 ksf

$$M_V = P_{EV_1}(x_1) + P_{EV_2}(x_2) + P_{EH}\sin\theta(B) = (\gamma_{BF} H B \gamma_{EV})(\tfrac{1}{2}B) + (\tfrac{1}{2}\gamma_{RS}(h-H)(B-l)\gamma_{EV})(l + \tfrac{2}{3}(B-l)) + (\tfrac{1}{2}\gamma_{RS} h^2 K_a \gamma_{EH}\sin\theta)(B)$$

M_V = [(120 pcf)(20.0 ft)(17.0 ft)(1.00)][½(17.0 ft)] + [½(120 pcf)(28.5 ft - 20.0 ft)(17.0 ft - 2.0 ft)(1.00)][2.0 ft + ⅔(17.0 ft - 2.0 ft)] + [½(120 pcf)(28.5 ft)²(0.414)(1.00)sin(20°)](17.0 ft) = 556.16 kip·ft/ft

$$M_H = P_{EH}\cos\theta(x_3) + P_{LS}\cos\theta(x_4) = (\tfrac{1}{2}\gamma_{RS} h^2 K_a \gamma_{EH}\cos\theta)(h/3) + (\sigma_{LS} h K_a \gamma_{LS}\cos\theta)(h/2)$$

M_H = ½[(120 pcf)(28.5 ft)²(0.414)(1.00)cos(20°)](28.5 ft /3) + [(0 psf)(28.5 ft)(0.414)(1.00)cos(20°)](28.5 ft /2) = 180.36 kip·ft/ft

$$P_V = P_{EV_1} + P_{EV_2} + P_{EH}\sin\theta = (\gamma_{BF} H B \gamma_{EV}) + (\tfrac{1}{2}\gamma_{RS}(h-H)(B-l)\gamma_{EV}) + (\tfrac{1}{2}\gamma_{RS} h^2 K_a \gamma_{EH}\sin\theta)$$

P_V = (120 pcf)(20.0 ft)(17.0 ft)(1.00) + ½(120 pcf)(28.5 ft - 20.0 ft)(17.0 ft - 2.0 ft)(1.00) + ½(120 pcf)(28.5 ft)²(0.414)(1.00)sin(20°) = 55.37 kip/ft

Settlement (See Attached Spreadsheet Calculations):

Total Settlement at Center of Reinforced Soil Mass: S_t = **2.769** in

Total Settlement at Wall Facing: S_t = **2.287** in

Time Rate of Consolidation Settlement at Wall Facing (See Attached Spreadsheet Calculations):

$(S_c)_{100}$ = **2.287** in at **15** days following completion of construction

W-13-045 - FRA-70-12.86 - Retaining Wall 4W11
MSE Wall Settlement - Sta. 1+78 to Sta. 3+38

Calculated By: BRT Date: 7/4/2018
Checked By: JPS Date: 7/5/2018

Boring B-108-2-13, B-108-3-13 and B-108-9-15

H=	20.0	ft	Total wall height
B'=	13.6	ft	Effective footing width due to eccentricity
D_w =	7.9	ft	Depth below bottom of footing
q_e =	4,080	psf	Equivalent bearing pressure at bottom 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	Total Settlement at Center of Reinforced Soil Mass: I [7]	$\Delta\sigma_v$ [8] (psf)	σ_{vf}' Midpoint (psf)	S_c [9,10] (ft)	S_c (in)	Total Settlement at Facing of Wall: I [7]	$\Delta\sigma_v$ [8] (psf)	σ_{vf}' Midpoint (psf)	S_c [9,10] (ft)	S_c (in)
1	A-2-4	G	0.0	4.0	4.0	2.0	125	500	250	250	3,250					14	24	83	0.15	0.990	4,040	4,290	0.060	0.717	0.499	2,037	2,287	0.046	0.558
	A-2-4	G	4.0	8.5	4.5	6.3	125	1,063	781	781	3,781					14	18	71	0.46	0.844	3,444	4,225	0.046	0.557	0.484	1,973	2,755	0.035	0.416
2	A-6b	C	8.5	11.0	2.5	9.8	115	1,350	1,206	1,091	4,091	37	0.243	0.012	0.561				0.72	0.686	2,801	3,891	0.011	0.129	0.453	1,847	2,938	0.008	0.100
	A-6b	C	11.0	13.5	2.5	12.3	115	1,638	1,494	1,222	4,222	37	0.243	0.024	0.561				0.90	0.593	2,418	3,641	0.018	0.221	0.425	1,734	2,956	0.015	0.179
3	A-1-b	G	13.5	16.0	2.5	14.8	125	1,950	1,794	1,366	4,366					19	21	77	1.08	0.517	2,110	3,476	0.013	0.157	0.396	1,615	2,981	0.011	0.131
4	A-4a	G	16.0	18.5	2.5	17.3	125	2,263	2,106	1,523	4,523					23	25	47	1.27	0.456	1,862	3,384	0.018	0.219	0.367	1,498	3,021	0.016	0.188
5	A-6b	C	18.5	21.0	2.5	19.8	120	2,563	2,413	1,673	4,673	37	0.243	0.024	0.561				1.45	0.407	1,661	3,334	0.012	0.140	0.341	1,390	3,063	0.010	0.123
6	A-1-b	G	21.0	26.5	5.5	23.8	130	3,278	2,920	1,931	4,931					31	31	103	1.75	0.346	1,412	3,343	0.013	0.153	0.303	1,235	3,166	0.011	0.138
	A-1-b	G	26.5	32.5	6.0	29.5	130	4,058	3,668	2,320	5,320					31	30	98	2.17	0.284	1,157	3,477	0.011	0.130	0.259	1,055	3,374	0.010	0.120
7	A-1-a	G	32.5	42.5	10.0	37.5	130	5,358	4,708	2,860	5,860					37	33	107	2.76	0.226	922	3,782	0.011	0.137	0.213	868	3,729	0.011	0.130
	A-1-a	G	42.5	52.5	10.0	47.5	130	6,658	6,008	3,536	6,536					37	30	99	3.49	0.180	734	4,270	0.008	0.099	0.173	706	4,242	0.008	0.096
8	A-2-6	G	52.5	57.5	5.0	55.0	130	7,308	6,983	4,043	7,043					31	24	83	4.04	0.156	636	4,679	0.004	0.046	0.151	617	4,661	0.004	0.045
9	A-4a	C	57.5	62.5	5.0	60.0	120	7,908	7,608	4,356	7,356	18	0.072	0.007	0.413				4.41	0.143	584	4,940	0.001	0.017	0.140	569	4,926	0.001	0.016
10	A-3a	G	62.5	67.5	5.0	65.0	125	8,533	8,220	4,657	7,657					22	16	60	4.78	0.132	540	5,196	0.004	0.048	0.129	528	5,185	0.004	0.047
																					Total Settlement:			2.769 in		Total Settlement:			2.287 in

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

W-13-045 - FRA-70-12.86 - Retaining Wall 4W11

MSE Wall Settlement - Sta. 1+78 to Sta. 3+38

Calculated By: BRT Date: 07/04/2018

Checked By: JPS Date: 07/05/2018

Boring B-108-2-13, B-108-3-13 and B-108-9-15

H=	20.0	ft	Total wall height
B'=	13.6	ft	Effective footing width due to eccentricity
D_w =	7.9	ft	Depth below bottom of footing
q_e =	4,080	psf	Equivalent bearing pressure at bottom of wall

	A-6b	A-4a			
c_v =	400	1000		ft^2/yr	Coefficient of consolidation
t =	15	15		days	Time following completion of construction
H_{dr} =	2.5	2.5		ft	Length of longest drainage path considered
T_v =	2.630	6.575			Time factor
U =	100	100		%	Degree of consolidation

$(S_c)_t$ = 2.287 in Settlement complete at 100% of primary consolidation

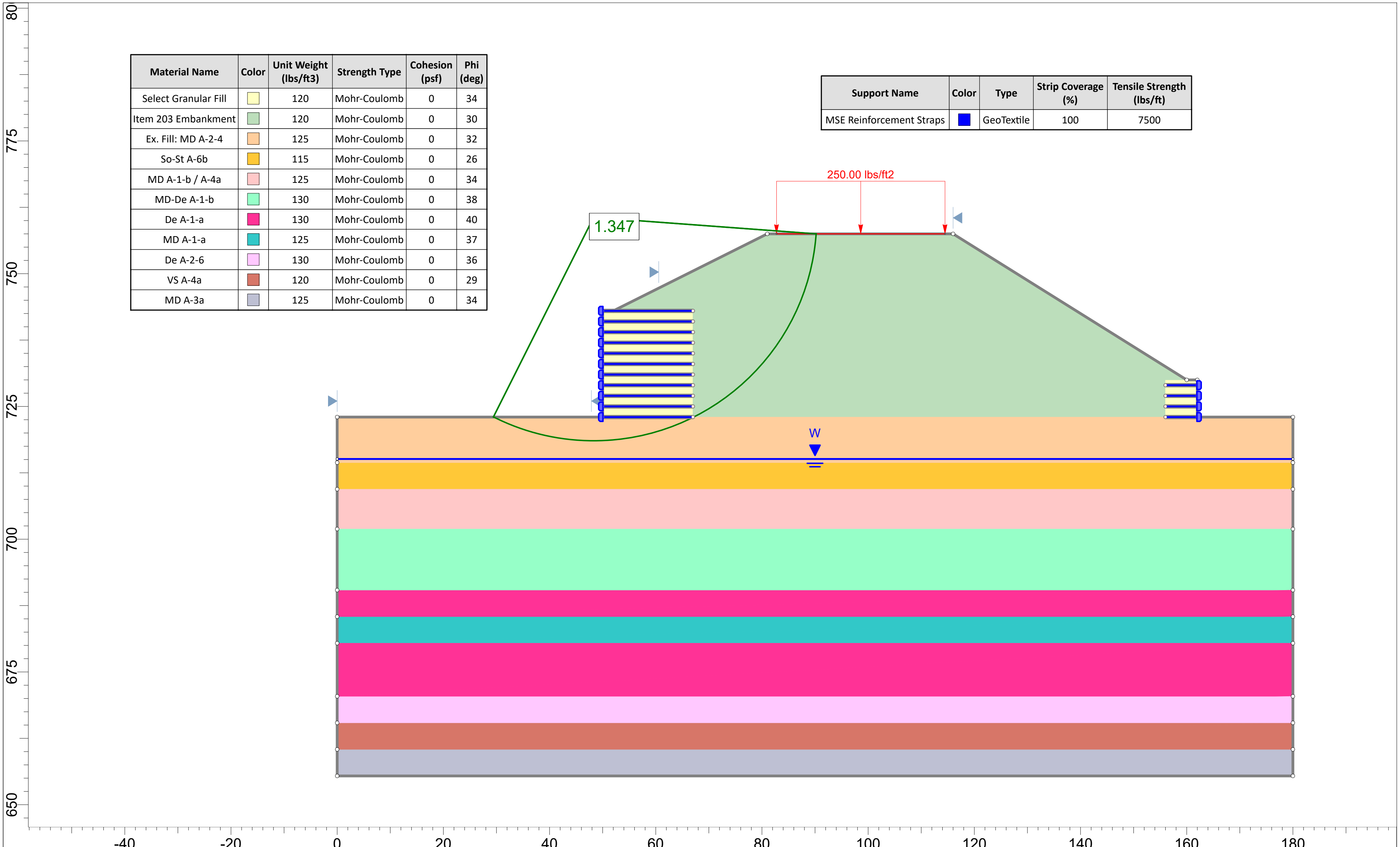
Layer	Soil Type	Soil Type	Layer Depth (ft)		Layer Thickness (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	Total Settlement at Facing of Wall: S_c (9,10) (ft)	S_c (in)	Layer Settlement (in)	Settlement Complete at 100% of Primary Consolidation: $(S_c)_t$ (11) (in)	Layer Settlement (in)
1	A-2-4	G	0.0	4.0	4.0	2.0	125	500	250	250	4,250					14	24	83	0.15	0.499	2,037	2,287	0.046	0.558	0.974	0.558	0.974
	A-2-4	G	4.0	8.5	4.5	6.3	125	1,063	781	781	4,781					14	18	71	0.46	0.484	1,973	2,755	0.035	0.416		0.416	
2	A-6b	C	8.5	11.0	2.5	9.8	115	1,350	1,206	1,091	5,091	37	0.243	0.012	0.561				0.72	0.453	1,847	2,938	0.008	0.100	0.280	0.100	0.280
	A-6b	C	11.0	13.5	2.5	12.3	115	1,638	1,494	1,222	5,222	37	0.243	0.024	0.561				0.90	0.425	1,734	2,956	0.015	0.179		0.179	
3	A-1-b	G	13.5	16.0	2.5	14.8	125	1,950	1,794	1,366	5,366					19	21	77	1.08	0.396	1,615	2,981	0.011	0.131	0.131	0.131	0.131
4	A-4a	G	16.0	18.5	2.5	17.3	125	2,263	2,106	1,523	5,523					23	25	47	1.27	0.367	1,498	3,021	0.016	0.188	0.188	0.188	0.188
5	A-6b	C	18.5	21.0	2.5	19.8	120	2,563	2,413	1,673	5,673	37	0.243	0.024	0.561				1.45	0.341	1,390	3,063	0.010	0.123	0.123	0.123	0.123
6	A-1-b	G	21.0	26.5	5.5	23.8	130	3,278	2,920	1,931	5,931					31	31	103	1.75	0.303	1,235	3,166	0.011	0.138	0.258	0.138	0.258
	A-1-b	G	26.5	32.5	6.0	29.5	130	4,058	3,668	2,320	6,320					31	30	98	2.17	0.259	1,055	3,374	0.010	0.120		0.120	
7	A-1-a	G	32.5	42.5	10.0	37.5	130	5,358	4,708	2,860	6,860					37	33	107	2.76	0.213	868	3,729	0.011	0.130	0.225	0.130	0.225
	A-1-a	G	42.5	52.5	10.0	47.5	130	6,658	6,008	3,536	7,536					37	30	99	3.49	0.173	706	4,242	0.008	0.096		0.096	
8	A-2-6	G	52.5	57.5	5.0	55.0	130	7,308	6,983	4,043	8,043					31	24	83	4.04	0.151	617	4,661	0.004	0.045	0.045	0.045	0.045
9	A-4a	C	57.5	62.5	5.0	60.0	120	7,908	7,608	4,356	8,356	18	0.072	0.007	0.413				4.41	0.140	569	4,926	0.001	0.016	0.016	0.016	0.016
10	A-3a	G	62.5	67.5	5.0	65.0	125	8,533	8,220	4,657	8,657					22	16	60	4.78	0.129	528	5,185	0.004	0.047	0.047	0.047	0.047

Settlement Remaining After Hold Period: 0.000 in

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

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
Ex. Fill: MD A-2-4		125	Mohr-Coulomb	0	32
So-St A-6b		115	Mohr-Coulomb	0	26
MD A-1-b / A-4a		125	Mohr-Coulomb	0	34
MD-De A-1-b		130	Mohr-Coulomb	0	38
De A-1-a		130	Mohr-Coulomb	0	40
MD A-1-a		125	Mohr-Coulomb	0	37
De A-2-6		130	Mohr-Coulomb	0	36
VS A-4a		120	Mohr-Coulomb	0	29
MD A-3a		125	Mohr-Coulomb	0	34

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

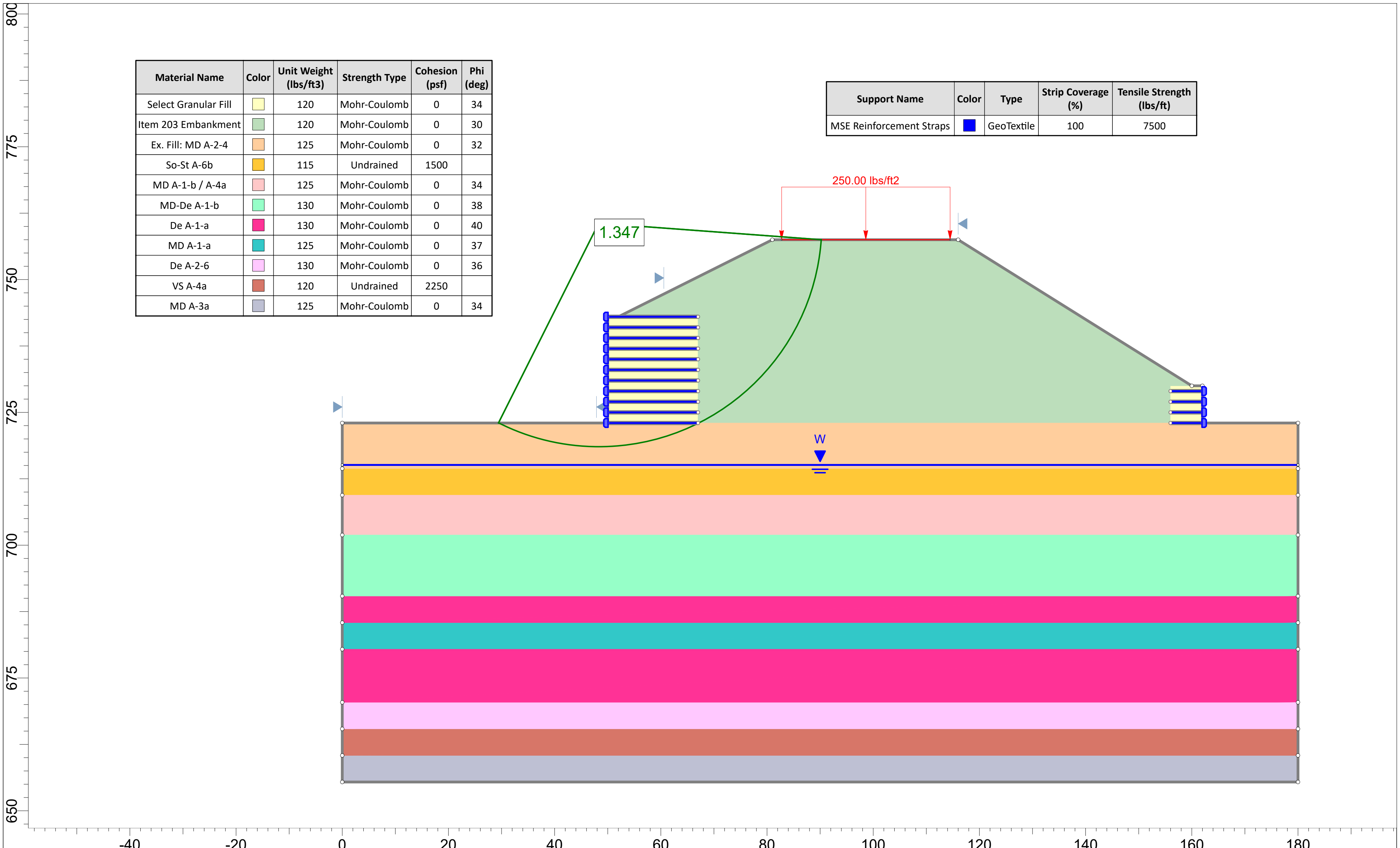


Project	Retaining Wall 4W11 - Sta. 1+78 to 3+38 - MSE Wall Global Stability
Analysis Description	B-108-9-13 - 20.0 ft Wall Height - Drained - Circular - Spencer
Drawn By	BRT
Scale	1:300
Company	Resource International, Inc.
Date	7/4/2018
File Name	Retaining Wall 4W11 - Sta. 1+78 to 3+38 - Global Stability.slim

SLIDEINTERPRET 7.020

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
Ex. Fill: MD A-2-4		125	Mohr-Coulomb	0	32
So-St A-6b		115	Undrained	1500	
MD A-1-b / A-4a		125	Mohr-Coulomb	0	34
MD-De A-1-b		130	Mohr-Coulomb	0	38
De A-1-a		130	Mohr-Coulomb	0	40
MD A-1-a		125	Mohr-Coulomb	0	37
De A-2-6		130	Mohr-Coulomb	0	36
VS A-4a		120	Undrained	2250	
MD A-3a		125	Mohr-Coulomb	0	34

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



SLIDEINTERPRET 7.020

Project	Retaining Wall 4W11 - Sta. 1+78 to 3+38 - MSE Wall Global Stability
Analysis Description	B-108-9-13 - 20.0 ft Wall Height - Undrained - Circular - Spencer
Drawn By	BRT
Scale	1:300
Company	Resource International, Inc.
Date	7/4/2018
File Name	Retaining Wall 4W11 - Sta. 1+78 to 3+38 - Global Stability.slim