Resource International, Inc.

FRA-70-13.10 PHASE 6A
FRA-70-1373L
I-70 WB OVER SHORT STREET
PID NO. 89464
FRANKLIN COUNTY, OHIO

STRUCTURE FOUNDATION EXPLORATION REPORT (REV. 1)

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

June 2021

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May 13, 2015 (Revised March 10, 2021)

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

Re: Structure Foundation Exploration Report (Rev. 1)
FRA-70-13.10 Phase 6A
FRA-70-1373L – I-70 WB over Short Street
PID No. 89464
Rii Project No. W-13-072

Mr. Mosure:

Resource International, Inc. (Rii) is pleased to submit this revised structure foundation exploration report for the above referenced project. Engineering logs have been prepared and are attached to this report along with the results of laboratory testing. This report includes recommendations for the design and construction of the proposed FRA-70-1373L bridge structure carrying I-70 westbound over Short Street as part of the FRA-70-13.10 Phase 6A project (PID 89464) 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 Services

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

Enclosure: Structure Foundation Exploration Report (Rev. 1)

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APPENDICIES

EXECUTIVE SUMMARY

Resource International, Inc. (Rii) has completed a structure foundation exploration for the design and construction of the proposed FRA-70-1373L bridge structure carrying I-70 westbound over Short Street, as shown on the vicinity map and boring plan presented in Appendix I. The existing structure is a three-span bridge with a total length of approximately 118 feet. It is understood that the existing structure consists of a reinforced concrete deck on continuous steel beams, and will be completely removed and replaced with a single-span composite prestressed concrete AASHTO Type IV superstructure with a reinforced concrete deck and semi-integral abutments behind mechanically stabilized earth (MSE) walls. The proposed structure will have a total length of approximately 93 feet and width of approximately 64 feet, and the proposed structure alignment will be shift approximately 15 to 20 feet south of the existing bridge alignment. In addition, the roadway profile will be elevated approximately 25 feet above the existing I-70 westbound profile grade. Please note that the analysis and recommendations for retaining wall E9 at the rear abutment and retaining wall E7, between Sta. 705+61 and 706+28 (BL Wall E7), at the forward abutment are presented under this report cover.

Exploration and Findings

Between February 18, 2014, and January 29, 2015, two (2) structural borings, designated as B-020-5-13 and B-020-7-13, were drilled to a completion depth of 90.0 and 80.4 feet below the existing ground surface, respectively, at the locations shown on the boring plan provided in Appendix I of the full report. On February 23, 2014, auger refusal was encountered in boring B-020-5-13 at a depth of 75.5 feet below the ground surface, and a 1.1-foot rock core run recovered 9.0-inches of granite from a boulder. The boring could not be advanced beyond this depth using the hollow-stem augers, and due to time restrictions for the traffic control, the boring was terminated at this depth. On January 22, 2015, boring B-020-5-13 was extended to bedrock and cored to the depth noted above in accordance with ODOT SGE requirements and per the comment provided for the Stage 1 preliminary report.

Boring B-020-5-13 was drilled in the median shoulder of I-70 westbound and encountered 6.0 inches of concrete overlying 6.0 inches of aggregate base at the ground surface. Boring B-020-7-13 was drilled through the existing sidewalk along the east side of Short Street, below the existing structure and between the curb and pier columns, and encountered 8.0 inches of concrete at the ground surface.

Boring B-020-5-13 encountered existing embankment fill consisting of brown silt and clay (ODOT A-6a) extending to a depth of 25.5 feet below the existing ground surface. The fill contained wood and brick fragments. Boring B-020-7-13 encountered material identified as possible fill consisting of brown and dark brown gravel with sand and silt, sandy silt, silty clay and clay (ODOT A-2-4, A-4a, A-6a, A-6b, A-7-6) extending to a depth of 20.5 feet below the existing ground surface.

Beneath the fill materials, natural soils were encountered consisting of both granular and cohesive material. The granular soils were generally described as brown, gray and black gravel, gravel with sand and gravel with sand and silt (ODOT A-1-a, A-1-b, A-2-4). The cohesive soils were generally described as brown, gray and brownish gray sandy silt, silt and clay and silty clay (ODOT A-4a, A-6a, A-6b). Granite boulders were encountered in borings B-020-5-13 and B-020-7-13 at an elevation of 657 feet msl. Auger refusal was encountered at this elevation in boring B-020-7-13, and rock coring was performed below this elevation. Based on the lack of recovery and observation of the soil washout in the circulation fluid, it is anticipated that this material is a hard cohesive soil. Significant recovery of mudstone bedrock occurred in core runs RC-4 and RC-5.

Top of bedrock was encountered in borings B-020-5-13 and B-020-7-13 at an elevation of 656.8 and 648.1 feet msl, respectively. The upper 3.4 feet of shale bedrock in boring B-020-5-13 was able to be augered and sampled. The cored bedrock consists of mudstone overlying shale bedrock in boring B-020-5-13 at an elevation of 645.4 feet msl.

Analyses and Recommendations

Driven Pile Recommendations

Given the proposed loading at each substructure unit, friction bearing piles will not be a feasible foundation option as the required ultimate bearing value per pile exceeds the values provided in Section 305.3.4 of the 2020 ODOT BDM based on the maximum factored load per pile. Therefore, given the depth of bedrock encountered in the borings performed, it is recommended that steel H-piles (ODOT Item 507.06) driven to refusal on bedrock be employed for foundation support. Per Section 305.3.1.2 of the 2020 ODOT BDM, 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 the recommended pile lengths and the corresponding factored structural axial resistance ($R_{R\ max}$) of the steel H-piles:

FRA-70-1373L Driven Pile Recommendations

Substructure Reference	Ground Elevation [1] (feet msl)	Pile Size	Pile Elevation (feet msl)		Pile Length [4] (feet)	$R_{R\ max}$ [5] (kips/pile)	Sleeve Length [6] (feet)	φ [7]
			Top [2]	Tip [3]				
Rear Abutment (B-020-5-13)	733.4	HP 12x53 [8]	746.3	656.8	95	380	13.8 / 33.8	N/A
Forward Abutment (B-020-7-13)	713.5	HP 12x53 [8]	746.0	648.1	105	380	35.6	N/A

1. *Ground elevation listed is the ground elevation at the respective boring locations.*
2. *The top of pile elevation corresponds to the pile cutoff elevation, which is considered to be 1.0-foot above the proposed bottom of footing elevation per Section 305.3.5.1 of the 2020 ODOT BDM.*
3. *The pile tip elevation is based on the top of bedrock elevation in the nearest boring per Section 305.3.5.2 of the 2020 ODOT BDM.*
4. *Per Section 305.3.5.2 of the 2020 ODOT BDM, the estimated pile length was determined as the pile cutoff elevation (top) minus the pile tip elevation, rounded up to the nearest 5.0 feet.*
5. *The factored structural axial resistance for H-piles is based on the structural limit state of the steel H-pile section per Section 305.3.3 of the 2020 ODOT BDM.*
6. *Sleeve length represents the required length of pile that should be sleeved within the MSE wall backfill. Multiple values represent the minimum and maximum sleeve length, respectively, where the wall steps up along the existing embankment slopes*
7. *For H-piles driven to refusal on bedrock, no geotechnical resistance factor should be applied to the factored structural axial resistance values presented, as the values presented account for the structural resistance factor, $\varphi_c = 0.50$, for H-piles subject to damage due to severe driving conditions.*
8. *A steel pile point is recommended to protect the tips of the steel H-piles during pile installation.*

The anticipated total settlement at the facing of the MSE wall at the rear and cellular concrete wall at the forward abutment is 3.35 and 2.93 inches, respectively. Results of the settlement analysis indicate that approximately 90 percent of the primary consolidation of the cohesive layers at the rear and forward abutment will be complete within 90 and 50 days following the placement if the surcharge load, respectively. Therefore, if the above noted waiting period is specified following completion of construction of the MSE wall at the rear abutment, downdrag forces along the piles will be eliminated.

MSE Wall Recommendations

Based upon the proposed plan information, the wall height at the rear abutment (Wall E9) ranges from 27.2 to 48.2 feet, as measured from the top of the leveling pad to the proposed profile grade of the roadway at the face of the wall. The footprint of the MSE wall at rear abutment will be within the limits of the existing embankment supporting I-70 eastbound and westbound, and the wall will be constructed full height from the proposed grade of Short Street to the proposed profile grade of I-70 westbound. Based on existing profile information provided, the existing embankment height is approximately 22 feet from the existing profile grade of I-70 to the top of the level pad elevation. Since the wall is located within an existing floodplain, the analysis was performed using a design groundwater level at the ground surface.

The anticipated bearing materials at the rear abutment (Wall E9) are anticipated to consist of hard silty clay (ODOT A-6b) overlying medium dense to very dense gravel with sand and gravel with sand and silt (ODOT A-1-b, A-2-4). MSE wall foundations bearing on the natural soils may be proportioned for a factored bearing resistance as indicated in the following table. A geotechnical resistance factor of φ_b=0.65 was considered in calculating the factored bearing resistance at the strength limit state.

FRA-70-1373L (Retaining Wall E9) MSE Wall Design Parameters

Substructure Unit (Boring	Wall Height Analyzed (feet)	Backslope Behind Wall	Minimum Required Reinforcement Length [1] (feet)	Bearing Resistance at Strength Limit (ksf)		Strength Limit Equivalent Bearing Pressure [3] (ksf)
				Nominal	Factored [2]	
Rear Abutment / Retaining Wall E9 (B-020-5-13)	48.2	Level	43.4 (0.90H)	18.88	12.27	9.68

1. *The minimum reinforcement length is based on the maximum wall height analyzed. The value in parentheses represent the required reinforcement length expressed as a percentage of the wall height, H.*
2. *A geotechnical resistance factor of φ_b=0.65 was considered in calculating the factored bearing resistance at the strength limit state.*
3. *The strength limit equivalent bearing pressure is the uniformly distributed pressure asserted by the wall over an effective base width based on the eccentricity of the wall system at the strength limit state.*

Total settlements of up to 5.29 inches at the center of the reinforced soil mass and 3.35 inches at the facing of the wall are anticipated at the rear abutment. Based on the results of the analysis, 90 percent of the total settlement is anticipated to occur over a period of approximately 90 days.

Based on the results of the external and global stability analysis performed for the MSE wall at the rear abutment (Retaining Wall E9), the recommended controlling strap length is 0.90 times the maximum height of the MSE wall (measured from the top of the leveling pad to the proposed profile grade of the roadway). Global stability under drained conditions was the controlling factor in the determination of the recommended strap length of 90 percent of the wall height at the rear abutment.

Lightweight (Cellular Concrete) Wall Recommendations

Based on the conditions encountered in boring B-020-7-13, existing fill consisting of loose to medium dense gravel with sand and silt (ODOT A-2-4) overlying medium stiff to stiff silty clay and clay (ODOT A-6b, A-7-6) was encountered at the proposed bearing elevation at the forward abutment, which extends to a depth of 17.0 feet below the bottom of wall elevation.

Given the presence of existing fill material to significant depths, as well as the significant amount of existing utilities present along the east side of Short Street, it is understood that lightweight fill material consisting of cellular concrete is being considered to be utilized as the backfill along the length of Retaining Wall E7 where it crosses in front of the forward abutment (between Sta. 705+61 and 706+28, BL Wall E7). The use of the lightweight cellular concrete will eliminate the need for undercut or ground improvement to stabilize the underlying existing fill material and control settlement to tolerable limits. Based on information provided by the Rii design group, two types of lightweight cellular concrete will be utilized in lieu of typical embankment fill and select granular fill, which is typically used for MSE wall applications. The wall facing will be connected to geosynthetic straps that are embedded into the cellular concrete and supported on a leveling pad, similar to traditional MSE walls. Since the wall is located within an existing floodplain, the analysis was performed using a design groundwater level at the ground surface.

It is recommended that the reinforcement extend the minimum length of 70 percent of the wall height into the cellular concrete backfill, similar to traditional MSE walls.

Based on the plan information provided, it is understood that the cellular concrete fill will be placed the full height of the embankment within the limits of I-70 westbound to the west side of the Franklin main, which is approximately 150 feet east of Short Street. Provided that all backslopes cut into the existing I-70 embankment are graded no steeper than 2H:1V, external and global stability calculations will not be required for this section of Retaining Wall E7. However, if bearing resistance must be checked, then a factored bearing resistance of 2.86 ksf should be utilized for design at the strength limit state.

A total settlement of 4.05 inches at the center of the wall mass and 2.93 inches at the facing of the wall is anticipated along Retaining Wall E7 where is crosses in front of the forward abutment. Based on the results of the analysis, 90 percent of the total settlement is anticipated to occur within 50 days following construction of the wall.

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

1.0 INTRODUCTION

The overall purpose of this project is to provide detailed subsurface information and recommendations for the design and construction of the FRA-70/71-13.10/14.36 (Projects 6A/6R) project in Columbus, Ohio. The projects represent the central portion of the FRA-70-8.93 (PID 77369) I-70/71 south innerbelt improvements project, which includes all improvements along I-70 westbound from the I-71/SR-315 interchange to Front Street and along I-71 southbound from I-70 to Greenlawn Avenue. The FRA-70-13.10 (Project 6A) phase will consist of all work associated with the construction of I-70 westbound from SR 315 to Front Street, including Ramps D3 and D7. This project includes the construction of one (1) new bridge structure for Ramp D3 over the Scioto River (FRA-70-1323C) and the reconstruction of three (3) bridges, including I-70 westbound over the Scioto River (FRA-70-1322L), I-70 westbound over CSX and Norfolk Southern (NS) Railroad (FRA-70-1358L) and I-70 westbound over Short Street (FRA-70-1373L), as well as the construction of five (5) new retaining walls (Walls E2, E3, E4, E7 and E9) to accommodate the new configuration.

This report is a presentation of the structure foundation exploration performed for the design and construction of the proposed FRA-70-1373L bridge structure carrying I-70 westbound over Short Street, as shown on the vicinity map and boring plan presented in Appendix I. The existing structure is a three-span bridge with a total length of approximately 118 feet. It is understood that the existing structure consists of a reinforced concrete deck on continuous steel beams, and will be completely removed and replaced with a single-span composite prestressed concrete AASHTO Type IV superstructure with a reinforced concrete deck and semi-integral abutments behind mechanically stabilized earth (MSE) walls. The proposed structure will have a total length of approximately 93 feet and width of approximately 64 feet, and the proposed structure alignment will be shifted approximately 15 to 20 feet south of the existing bridge alignment. In addition, the roadway profile will be elevated approximately 25 feet above the existing I-70 westbound profile grade. Please note that the analysis and recommendations for Retaining Wall E9 at the rear abutment and Retaining Wall E7, between Sta. 705+61 and 706+28 (BL Wall E7), at the forward abutment are presented under this report cover. Design recommendations for the remaining alignment of Retaining Wall E7 is under a separate cover.

2.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1 Site Geology

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

According to the bedrock geology and topography maps obtained from the Ohio Department of Natural Resources (ODNR), the underlying bedrock west of the Scioto River consists predominantly of the Middle to Lower Devonian-aged Columbus Limestone. This formation is further subdivided into two members in the central portion of the state, known as the Delhi and Bellepoint Members. The Delhi Member consists of light gray, finely to coarsely crystalline, irregularly bedded, fossiliferous limestone. The Bellepoint Member consists of variable brown, finely crystalline, massively bedded limey dolomite. Both of these members contain chert nodules. East of the Scioto River, the underlying bedrock consists of the Upper Devonian Ohio Shale Formation overlying the Middle Devonian-aged Delaware Limestone Formation. The Ohio Shale formation consists of brownish black to greenish gray, thinly bedded, fissile, carbonaceous shale. The Delaware Limestone consists of bluish gray, thin to medium bedded dolomitic limestone with nodules and layers of chert. Regionally, the bedrock surface forms a broad valley aligned roughly north-to-south beneath the Scioto River. According to bedrock topography mapping, the elevation of the bedrock surface ranges from approximately 600 feet mean sea level (msl) in the valley to approximately 625 feet msl near the project limits. Within the borings performed for the current investigation, shale bedrock was encountered at elevations of 648.1 and 656.8 feet msl.

2.2 Existing Conditions

The proposed FRA-70-1373L structure is located at the existing I-70 over Short Street overpass, approximately 200 feet south of the intersection of Short Street and Mound Street and approximately 1,100 feet west of the Front Street overpass. I-70 westbound in the vicinity of the structure is a three-lane, asphalt paved roadway that is aligned east-to-west, and Short Street in the vicinity of the structure is a two-lane, asphalt paved roadway that is aligned north-to-south. The existing Mound Street Ramp to I-70 westbound merges with the existing I-70 westbound lanes and creates a fourth lane on the north side of the roadway, just east of the bridge structure. The existing I-70 roadway profile grade is elevated approximately 18 feet above the Short Street profile grade. The terrain along both roadways and the surrounding area is relatively flat-lying, and the I-70 roadway is elevated on existing embankments from the surrounding terrain. Based on utility plans provided by ms consultants, there are many buried utilities within the Short Street roadway and beneath the surrounding sidewalks, including the Olentangy Scioto Interceptor Sewer (OSIS), which runs north to south within the roadway of Short Street.

3.0 EXPLORATION

Between February 18, 2014, and January 29, 2015, two (2) structural borings, designated as B-020-5-13 and B-020-7-13, were drilled at the locations shown on the boring plan provided in Appendix I of this report and summarized in Table 1. The borings were advanced to a completion depth of 90.0 and 80.4 feet below the existing ground surface, respectively. On February 23, 2014, auger refusal was encountered in boring B-020-5-13 at a depth of 75.5 feet below the ground surface, and a 1.1-foot rock core run recovered 9.0-inches of granite from a boulder. The boring could not be advanced beyond this depth using the hollow-stem augers, and due to time restrictions for the traffic control, the boring was terminated at this depth. On January 22, 2015, boring B-020-5-13 was extended to bedrock and cored to the depth noted above in accordance with ODOT SGE requirements and per the comment provided for the Stage 1 preliminary report. Boring B-020-7-13 was located within the eastern sidewalk underneath of the existing bridge structure due to the inability to perform the boring from the roadway grade above. Due to the limited overhead clearance, the drilling for this boring was performed by Stock Drilling using a low-head clearance rig.

Table 1. Test Boring Summary

Boring Number	Station [1]	Offset [1]	Latitude	Longitude	Ground Elevation (feet msl)	Boring Depth (feet)
B-020-5-13	175+57.74	11.8' Rt.	39.953452196	-83.004773258	733.4	90.0
B-020-7-13	176+68.64	1.8' Rt.	39.953451540	-83.004376859	713.5	80.4

1. Station and offset referenced to the proposed baseline of I-70 westbound.

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

The borings were drilled using a truck or all-terrain vehicle (ATV) mounted rotary drilling machine, utilizing a 4.25-inch inside diameter, hollow-stem auger to advance the holes. In general, standard penetration test (SPT) and split spoon sampling was performed in boring B-020-5-13 at 5.0-foot increments of depth to a depth of 20.0 feet, at 2.5-foot intervals for the next 30.0 feet and then at 5.0-foot intervals to the top of bedrock. Boring B-020-7-20 was sampled at 2.5-foot increments of depth to 30.0 feet and at 5.0-foot increments thereafter to the top of bedrock.

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

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

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

The hammers for the CME 750 drill rig operated by Rii was calibrated on April 26, 2013, and has a drill rod energy ratio of 82.6 percent. The hammer for the CME 55-LC drill rig operated by Stock Drilling was calibrated on March 28, 2013, and has a drill rod energy ratio of 73.2 percent.

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

The depth to bedrock was determined by visual inspection of the very weak to weak shale and mudstone samples and based on the blow counts obtained from the SPT testing. Where borings were extended into the bedrock (after encountering auger refusal), an NQ or HQ-sized double-tube diamond bit core barrel (utilizing wire line equipment) was used to core the bedrock. Coring produced 1.8 or 2.5 inch diameter cores, from which the type of rock and geological characteristics were determined.

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

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

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

Table 2. Laboratory Test Schedule

Laboratory Test	Test Designation	Number of Tests Performed
Natural Moisture Content	ASTM D 2216	39
Plastic and Liquid Limits	AASHTO T89, T90	14
Gradation – Sieve/Hydrometer	AASHTO T88	14
One-Dimensional Consolidation	ASTM D2435	1
Consolidated Undrained (CU) Triaxial Test	ASTM D4767	1
Point Load Strength Index of Rock Specimens	ASTM D5731	1

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

4.0 FINDINGS

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

4.1 Surface Materials

Boring B-020-5-13 was drilled in the median shoulder of I-70 westbound and encountered 6.0 inches of concrete overlying 6.0 inches of aggregate base at the ground surface. Boring B-020-7-13 was drilled through the existing sidewalk along the east side of Short Street, below the existing structure and between the curb and pier columns, and encountered 8.0 inches of concrete at the ground surface.

4.2 Subsurface Soils

Boring B-020-5-13 encountered existing embankment fill consisting of brown silt and clay (ODOT A-6a) extending to a depth of 25.5 feet below the existing ground surface. The fill contained wood and brick fragments. Boring B-020-7-13 encountered material identified as possible fill consisting of brown and dark brown gravel with sand and silt, sandy silt, silty clay and clay (ODOT A-2-4, A-4a, A-6a, A-6b, A-7-6) extending to a depth of 20.5 feet below the existing ground surface.

Beneath the fill materials, natural soils were encountered consisting of both granular and cohesive material. The granular soils were generally described as brown, gray and black gravel, gravel with sand and gravel with sand and silt (ODOT A-1-a, A-1-b, A-2-4). The cohesive soils were generally described as brown, gray and brownish gray sandy silt, silt and clay and silty clay (ODOT A-4a, A-6a, A-6b). Granite boulders were encountered in borings B-020-5-13 and B-020-7-13 at an elevation of 657 feet msl. Auger refusal was encountered at this elevation in boring B-020-7-13, and rock coring was performed below this elevation. Based on the lack of recovery and observation of the soil washout in the circulation fluid, it is anticipated that this material is a hard cohesive soil. Significant recovery of mudstone bedrock occurred in core runs RC-4 and RC-5.

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 6 bpf to split spoon sampler refusal. Split spoon sampler refusal is defined as exceeding 50 blows from the hammer with less than 6.0 inches of penetration by the split spoon sampler. The shear strength and consistency of the cohesive soils are primarily derived from the hand penetrometer values (HP). The cohesive soil encountered ranged from medium stiff ($0.5 \leq HP \leq 1.0$ tsf) to hard ($HP > 4.0$ tsf). The unconfined compressive strength of the cohesive soil samples tested, obtained from the hand penetrometer, ranged from 0.75 to over 4.5 tsf (limit of instrument).

Natural moisture contents of the soil samples tested ranged from 6 to 26 percent. The natural moisture content of the cohesive soil samples tested for plasticity index ranged from 2 percent below to 7 percent above their corresponding plastic limits. In general, the soil exhibited natural moisture contents considered to be slightly 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		Top of Bedrock Core	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-020-5-13	733.4	76.6	656.8	80.0	653.4
B-020-7-13	713.5	65.4	648.1	65.4	648.1

Top of bedrock was encountered in borings B-020-5-13 and B-020-7-13 at an elevation of 656.8 and 648.1 feet msl, respectively. The upper 3.4 feet of shale bedrock in boring B-020-5-13 was able to be augered and sampled. The cored bedrock consists of mudstone overlying shale bedrock in boring B-020-5-13 at an elevation of 645.4 feet msl. The mudstone is described as gray, slightly to highly weathered, very weak to weak, thinly laminated to thick bedded, arenaceous, calcareous, friable, fissile and slightly to moderately fractured with tight to open, slightly to very rough apertures. The shale is described as gray and black, unweathered to slightly weathered, very weak to slightly strong, laminated to thick bedded, calcareous, arenaceous, friable, fissile and moderately to highly fractured with tight to open, rough to very rough apertures.

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

Table 4. Rock Core Summary

Boring	Core No.	Depth (feet)	Recovery (%)	RQD (%)	Unconfined Compressive Strength
B-020-5-13	RC-2	80.0 to 85.0	80	63	N/A
	RC-3	85.0 to 90.0	100	72	N/A
B-020-7-13	RC-4	65.4 to 75.4	97	89	q_u @ 69.4' = 224 psi [1]
	RC-5	75.4 to 80.4	100	45	N/A

1. *Represents the mean unconfined compressive strength based on correlations with the mean point load strength index.*

It should be noted that bedrock experiences mechanical breaks during the drilling and coring processes. Rii attempted to account for fresh, manmade breaks during tabulation of the RQD analysis. The zones where boulders were encountered that required rock coring techniques to advance through these zones are not included in the RQD tabulation above. The quality of the cored bedrock, according to the RQD values, ranged from poor (25 < RQD ≤ 50%) to good (75 < RQD ≤ 90%).

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-020-5-13	733.4	43.5	689.9	N/A [2]	N/A
B-020-7-13	713.5	N/A [1]	N/A	N/A [2]	N/A

1. *Groundwater was not encountered in boring B-020-7-13 prior to introducing water to the borehole.*
2. *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 boring B-020-5-13 at a depth of 43.5 feet below the existing ground surface. Groundwater was not encountered in boring B-020-7-13 prior to introducing water to the borehole. The groundwater levels at the completion of drilling could not be measured due to the addition of mud to counteract heaving sands as well as water as a circulating fluid during the rock coring process. Please note that short-term water level readings, especially in cohesive soils, are not necessarily an accurate indication of the actual groundwater level. In addition, groundwater levels or the presence of groundwater are considered to be dependent on seasonal fluctuations in precipitation.

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

5.0 ANALYSES AND RECOMMENDATIONS

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

Design details of the proposed structure were provided by Jacobs and ms consultants. Based on the information provided, it is understood that the existing three-span bridge will be completely removed and replaced with a single-span composite prestressed concrete AASHTO Type IV superstructure with a reinforced concrete deck and semi-integral abutments behind mechanically stabilized earth (MSE) walls. The proposed structure will have a total length of approximately 93 feet and width of approximately 64 feet, and the proposed structure alignment will be shift approximately 15 to 20 feet south of the existing bridge alignment. In addition, the roadway profile will be elevated approximately 25 feet above the existing I-70 westbound profile grade.

Retaining Wall E9 will be located along the rear abutment of the proposed structure to provide the required grade separation to support the configuration. The maximum wall height at the rear abutment is 48.2 feet, and the total wall length along the abutment is approximately 65 lineal feet. The wall will connect to the existing Retaining Wall 4W8 (which is part of the FRA-70-13.11 Project 4A, PID 77372) at the south side of the bridge structure, and the wall alignment will turn west on the north side of the structure and step up along the existing embankment for approximately 83.5 feet to a height of 27.2 feet at the end of the wall. It is understood that a mechanically stabilized earth (MSE) wall type is being considered as the preferred wall type for Retaining Wall E9.

A portion of Retaining Wall E7, between Sta. 705+61 and 706+28 (BL Wall E7), will be located along the forward abutment of the proposed structure to provide the required grade separation to support the configuration. The maximum wall height at the forward abutment is 48.2 feet, and the total wall length along the abutment is approximately 67 lineal feet. The wall will connect to the existing Retaining Wall 4W5 (which is part of the FRA-70-13.11 Project 4A, PID 77372) on the south side of the structure and to Retaining Wall E7 on the north side of the structure, which will be constructed as part of the FRA-71-14.36 Project 6R (PID 105588). The wall turns to the east on the north side of the FRA-71-1503L bridge structure and follows the alignment of Ramp D6. It is understood that a mechanically stabilized earth (MSE) wall type is being considered as the preferred wall type for Retaining Wall E7. However, given the presence of existing fill material to significant depths, as well as the significant amount of existing utilities within the footprint of this wall, it is understood that lightweight fill material consisting of cellular concrete will be utilized along the length of the wall that crosses in front of the forward abutment. Design recommendations for the remaining alignment of Retaining Wall E7 is presented under separate covers.

Proposed structural data was obtained from design details provided by Jacobs and ms consultants 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 / Retaining Wall E9 (B-020-5-13)	Profile Grade	758.7	330 kips/pile
	Bottom of Footing	745.3	
	Bottom of Wall (Top Leveling Pad)	710.5 / 731.5 [2]	
Forward Abutment / Retaining Wall E7 (Sta. 705+61 to 706+28) (B-020-7-13)	Profile Grade	758.2	330 kips/pile
	Bottom of Footing	745.0	
	Bottom of Wall (Top Leveling Pad)	710.0	

1. *Proposed foundation elevations and structural loading based on structure information provided by Jacobs and ms consultants.*
2. *Multiple values represent the minimum and maximum bottom of wall elevation at the rear abutment where the wall steps up along the existing embankment slopes*

5.1 Driven Pile Recommendations

Given the proposed loading at each substructure unit, friction bearing piles will not be a feasible foundation option as the required ultimate bearing value per pile exceeds the values provided in Section 305.3.4 of the 2020 ODOT Bridge Design Manual (BDM) based on the maximum factored load per pile. Therefore, given the depth of bedrock encountered in the borings performed, it is recommended that steel H-piles (ODOT Item 507.06) driven to refusal on bedrock be employed for foundation support. Per Section 305.3.1.2 of the 2020 ODOT BDM, 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 the recommended pile lengths and the corresponding factored structural axial resistance ($R_{R\ max}$) of the steel H-piles:

Table 7. FRA-70-1373L Driven Pile Recommendations

Substructure Reference	Ground Elevation [1] (feet msl)	Pile Size	Pile Elevation (feet msl)		Pile Length [4] (feet)	$R_{R\ max}$ [5] (kips/pile)	Sleeve Length [6] (feet)	φ [7]
			Top [2]	Tip [3]				
Rear Abutment (B-020-5-13)	733.4	HP 12x53 [8]	746.3	656.8	95	380	13.8 / 33.8	N/A
Forward Abutment (B-020-7-13)	713.5	HP 12x53 [8]	746.0	648.1	105	380	35.6	N/A

1. *Ground elevation listed is the ground elevation at the respective boring locations.*
2. *The top of pile elevation corresponds to the pile cutoff elevation, which is considered to be 1.0-foot above the proposed bottom of footing elevation per Section 305.3.5.1 of the 2020 ODOT BDM.*
3. *The pile tip elevation is based on the top of bedrock elevation in the nearest boring per Section 305.3.5.2 of the 2020 ODOT BDM.*
4. *Per Section 305.3.5.2 of the 2020 ODOT BDM, the estimated pile length was determined as the pile cutoff elevation (top) minus the pile tip elevation, rounded up to the nearest 5.0 feet.*
5. *The factored structural axial resistance for H-piles is based on the structural limit state of the steel H-pile section per Section 305.3.3 of the 2020 ODOT BDM.*
6. *Sleeve length represents the required length of pile that should be sleeved within the MSE wall backfill. Multiple values represent the minimum and maximum sleeve length, respectively, where the wall steps up along the existing embankment slopes*
7. *For H-piles driven to refusal on bedrock, no geotechnical resistance factor should be applied to the factored structural axial resistance values presented, as the values presented account for the structural resistance factor, φ_c = 0.50, for H-piles subject to damage due to severe driving conditions.*
8. *A steel pile point is recommended to protect the tips of the steel H-piles during pile installation.*

Per Section 305.3.3 of the 2020 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 (φ_c = 0.50 per Section 6.5.4.2 of the 2020 AASHTO LRFD BDS) and a pile fully braced along its length. **These bearing values should not be used for piles that are subjected to bending moments or are not supported by soil for their entire length.** Static or dynamic load testing is not required for H-piles driven to refusal on bedrock. It is anticipated that the piles will be able to be driven a short distance into the surficial bedrock before satisfying the driving conditions that meet the refusal criterion. Settlement is estimated to be less than 1.0 inch for H-piles driven to refusal on bedrock.

5.1.1 Downdrag Considerations

The anticipated total settlement at the facing of the MSE wall at the rear and cellular concrete wall at the forward abutment is 3.35 and 2.93 inches, respectively. Given the anticipated amount of settlement at the MSE wall facing, downdrag loads may be induced on the pile elements if installed to the final tip elevation prior to construction of the wall. To reduce the amount of downdrag induced on the piles, it is recommended that the piles be pre-driven into the soil only as far as necessary to remain vertical and that the MSE wall should be constructed around the piles and then allowed to sit for a specified holding period such that a percentage of the consolidation can occur prior to driving the piles the to the design tip elevation and reduce the amount of downdrag on the piles. In order to consolidate the underlying soil to the required settlement, consideration should be given to the placement of a surcharge load in order to preload the site under the full weight of the MSE wall height (from the bottom of wall elevation to the profile grade) and left in place until approximately 90 percent of consolidation of the subsurface soils has occurred to prevent downdrag loads from developing along the pile elements. Results of the settlement analysis indicate that approximately 90 percent of the primary consolidation of the cohesive layers at the rear and forward abutment will be complete within 90 and 50 days following the placement if the surcharge load, respectively. 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 2020 AASHTO LRFD BDS using the GRLWEAP software program, and the results are provided in Appendix V. In the driveability analysis, a Delmag 19-42 hammer with a rated energy of approximately 43,000 ft-lbs was used in conjunction with the H-pile sections. Based on the results of this analysis, driving stresses induced on the H-piles **would not exceed** 90 percent of the yield stress of the steel (f_y = 50 ksi, 0.9f_y = 45 ksi) if driven through the overburden soils to the bedrock elevation provided in Table 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 305.3.5.6 of the 2020 ODOT BDM, although the surficial bedrock consists of weak rock with an unconfined compressive strength less than 500 psi, steel pile points **shall be used** when the piles are extended through overburden containing layers of very dense granular soils as well as boulders.

5.1.3 Lateral Design

If lateral loads or moments are expected to be applied on the foundation elements, they should be analyzed to verify the 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 VI. 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 required to resist the lateral load for a given end condition and deflection. Table 8 lists the eleven 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 VI.

Table 8. 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)

5.2 MSE Wall Recommendations

It is proposed to construct MSE walls at the rear abutment (Retaining Wall E9) and forward abutment (Retaining Wall E7 between Sta. 705+61 and 706+28, BL Wall E7) of the proposed bridge structure. As previously discussed, given the presence of existing fill material to significant depths, as well as the significant amount and critical nature of existing utilities within the footprint of Retaining Wall E7, it is understood that lightweight fill material consisting of cellular concrete will be utilized along the length of the wall that crosses in front of the forward abutment. While it is understood that the wall facing will be connected to geosynthetic straps that will be embedded in the cellular concrete backfill, the analysis approach for this type of system differs from that of a traditional MSE wall. Therefore, the recommendations for this system are presented in Section **Error! Reference source not found.**.

MSE walls are constructed on earthen foundations at a minimum depth of 3.0 feet below grade, as defined by the top of the leveling pad to the ground surface located 4.0 feet from the face of the wall. Per Section 840.04.A of ODOT Supplemental Specification 840 (SS 840), the height of the MSE wall at the bridge abutment is defined as the elevation difference between the profile grade at the face of the wall and the top of the leveling pad, and where the wall does not cross in front of the abutment, the height of the wall is defined as the elevation difference between the top of coping and the top of the leveling pad. However, at the abutment, 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 201.4.1.C.7 of the 2020 ODOT BDM, a minimum of one row of soil reinforcement straps should be attached to the backside of the abutment footing to resist horizontal forces from the bridge structure and lateral pressures along the back wall of the abutment footing, and prevent any load transfer from these forces to the coping and facing panels. The width of the MSE wall foundation (B) is defined by the length of the reinforced soil mass. Per the 307.4.A of the 2020 ODOT BDM and Section 840.04.A.2 of ODOT SS 840, the minimum length of the reinforced soil mass is equal to 70 percent of the height of the MSE wall or 8.0 feet whichever is greater. A non-structural bearing leveling pad consisting of a minimum of 6.0-inches of unreinforced concrete should be placed at the base of the wall facing for constructability purposes. Please note that the leveling pad is not a structural foundation.

Based upon the proposed plan information, the wall height at the rear abutment (Wall E9) ranges from 27.2 to 48.2 feet, as measured from the top of the leveling pad to the proposed profile grade of the roadway at the face of the wall. The footprint of the MSE wall at rear abutment will be within the limits of the existing embankment supporting I-70 eastbound and westbound, and the wall will be constructed full height from the proposed grade of Short Street to the proposed profile grade of I-70 westbound. Based on existing profile information provided by Jacobs and ms consultants, the existing embankment height is approximately 22 feet from the existing profile grade of I-70 to the top of the level pad elevation. For the analysis, the foundation width was set at 70 percent of the wall height and the foundation width was increased, if required, until external and global stability requirements were satisfied.

Per Section 840.06.D of ODOT SS 840, the foundation subgrade should be inspected to verify that the subsurface conditions are the same as those anticipated in this report. Based on the conditions encountered in boring B-020-5-13, the anticipated soils at the proposed bearing elevation at the rear abutment will consist of hard silty clay (ODOT A-6b) overlying medium dense to very dense gravel with sand and gravel with sand and silt (ODOT A-1-b, A-2-4). This soil in its current condition is considered adequate for structural support.

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

Since the wall is located within an existing floodplain, the analysis was performed using a design groundwater level at the ground surface.

5.2.1 Strength Parameters Utilized in External and Global Stability Analyses

The shear strength parameters utilized in the external and global stability analyses for the MSE wall at the rear abutment are provided in Table 9.

Table 9. 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 and new embankment)	120	30	0	2,000
Existing Embankment Fill: Very Stiff to Hard Silt and Clay and Silty Clay (ODOT A-6a)	120	28	0	3,000 to 4,125
Medium Dense to Very Dense Gravel with Sand and Gravel with Sand and Silt (ODOT A-1-b, A-2-4)	125 to 135	35 to 41	0	N/A
Hard Sandy Silt (ODOT A-4a)	130	33	100	e,000
Hard Silt and Clay (ODOT A-6a)	130	29	100	8,000
Hard Silty Clay (ODOT A-6b)	125	27	50	8,000

1. *Per Figure 7-45, Section 7.6.9 of FHWA GEC 5 for cohesive soils and Table 10.4.6.2.4-1 of the 2020 AASHTO LRFS BDS for granular soils.*
2. *Estimated based on overconsolidated nature of soil.*
3. *S_u = 125(N_{60}), Terzaghi and Peck (1967).*

Shear strength parameters for the reinforced soil backfill and retained embankment are provided in Table 307-1 of the 2020 ODOT BDM and Section 840.04.A.3 of ODOT SS 840. Per these specifications, the select granular backfill in the reinforced zone and the retained embankment must meet the shear strength requirements provided in Table 9. 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, the 2020 AASHTO LRFD BDS and based on past experience in the vicinity of the site with projects performed in similar subsurface profiles.

5.2.2 Bearing Stability

The anticipated bearing materials at the rear abutment (Wall E9) are anticipated to consist of hard silty clay (ODOT A-6b) overlying medium dense to very dense gravel with sand and gravel with sand and silt (ODOT A-1-b, A-2-4). MSE wall foundations bearing on the natural soils may be proportioned for a factored bearing resistance as indicated in Table 10. 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 based on the maximum height of the wall at the rear abutment.

Table 10. FRA-70-1373L (Retaining Wall E9) MSE Wall Design Parameters

Substructure Unit (Boring	Wall Height Analyzed (feet)	Backslope Behind Wall	Minimum Required Reinforcement Length [1] (feet)	Bearing Resistance at Strength Limit (ksf)		Strength Limit Equivalent Bearing Pressure [3] (ksf)
				Nominal	Factored [2]	
Rear Abutment / Retaining Wall E9 (B-020-5-13)	48.2	Level	43.4 (0.90H)	18.88	12.27	9.68

1. *The minimum reinforcement length is based on the maximum wall height analyzed. The value in parentheses represent the required reinforcement length expressed as a percentage of the wall height, H.*
2. *A geotechnical resistance factor of φ_b=0.65 was considered in calculating the factored bearing resistance at the strength limit state.*
3. *The strength limit equivalent bearing pressure is the uniformly distributed pressure asserted by the wall over an effective base width based on the eccentricity of the wall system at the strength limit state.*

Rii performed a verification of the bearing pressure exerted on the subgrade soil for the maximum specified wall height indicated in Table 10. 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 or undrained conditions.

5.2.3 Settlement Evaluation

The compressibility parameters utilized in the settlement analyses of the proposed MSE wall at the rear abutment are provided in Table 11.

Table 11. Compressibility Parameters Utilized in Settlement Analysis

Material Type	γ **(pcf)**	LL **(%)**	C_c (1)	C_r (2)	e_o (3)	C_v (4) **(ft²/yr)**	N_{60}	C' (5)
Existing Embankment Fill: Very Stiff to Hard Silt and Clay (ODOT A-6a)	125	32	0.198	0.020	0.522	600	N/A	N/A
Medium Dense to Very Dense Gravel with Sand and Gravel with Sand and Silt (ODOT A-1-b, A-2-4)	125 to 135	N/A	N/A	N/A	N/A	N/A	21 to 65	50 to 317
Hard Sandy Silt (ODOT A-4a)	130	25	0.135	0.014	0.467	800	N/A	N/A
Hard Silt and Clay (ODOT A-6a)	130	25	0.135	0.014	0.467	600	N/A	N/A
Hard Silty Clay (ODOT A-6b)	125 to 130	33 to 35	0.207 to 0.225	0.021 to 0.023	0.530 to 0.546	300	N/A	N/A

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

Results of the settlement analysis are tabulated in Table 12. Total settlements of up to 5.29 inches at the center of the reinforced soil mass and 3.35 inches at the facing of the wall are anticipated at the rear abutment. As noted in Section 5.2, the footprint of the MSE wall will be within the limits of the existing embankment supporting I-70 eastbound and westbound, and the MSE wall will be constructed full height from the proposed grade of Short Street to the proposed profile grade of I-70 westbound. Therefore, the net bearing pressure considering the equivalent bearing pressure at the service state minus the pressure from the existing embankment fill that will be removed was utilized in the settlement analysis. Based on the results of the analysis, 90 percent of the total settlement is anticipated to occur over a period of approximately 90 days. Please note that the consolidation settlement and time rate of consolidation are based on estimates using correlated compressibility parameters provided in Table 11 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 12. MSE Wall Settlement Values

Substructure Unit (Boring)	Service Limit Equivalent Bearing Pressure [1] (ksf)	Total Settlement Values (inches)		Time for 90% Consolidation (Days)
		Center of Wall Mass	Facing of Wall	
Rear Abutment / Retaining Wall E9 (B-020-5-13)	6.95	5.29	3.35	90

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 307.1.6 of the 2020 ODOT BDM, the maximum allowable differential settlement in the longitudinal direction (regardless of the size of panels) is one (1) percent (1/100). Based on the total anticipated settlement at the facing of the walls, maximum differential settlements in the longitudinal directions are anticipated to be less than 1/1000, which is within the tolerable limit of 1/100. If the total or differential settlement values predicted for the proposed wall 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 wall. If preloading the site is not a desired option, then consideration could be given to ground improvement through the use of stone columns. Settlement calculations are provided in Appendix VII.

5.2.4 Eccentricity (Overturning Stability)

The resistance of the MSE wall to overturning will be dependent on the location of the resultant force at the bottom of the wall due to the overturning and resisting moments acting on the wall. For MSE walls, overturning stability is determined by calculating the eccentricity of the resultant force from the midpoint of the base of the wall and comparing this value to a limiting eccentricity value. Per Section 11.10.5.5 of the 2020 AASHTO LRFD BDS, for foundations bearing on soil, the location of the resultant of the reaction forces shall be within the middle two-thirds ($^2/_3$) of the base width. Therefore, the limiting eccentricity is one-third ($^1/_3$) of the base width of the wall. Rii performed a verification of the eccentricity of the resultant force for the maximum specified wall height indicated in Table 10. Based on the minimum length of reinforced soil mass presented in Table 10 and utilizing the soil parameters listed in Section 5.2.1 for the retained embankment material, the calculated eccentricity of the resultant force **will not exceed** the limiting eccentricity at the strength limit state at the rear abutment.

5.2.5 Sliding Stability

The resistance of the MSE wall to sliding was evaluated per Section 11.10.5.3 of the 2020 AASHTO LRFD BDS. Given that the bearing soils at the rear abutment consist of cohesive material, the sliding resistance was evaluated under both drained and undrained conditions. For drained conditions, the sliding resistance is determined by multiplying a coefficient of sliding friction “f” times the total vertical force at the base of the wall. The coefficient of sliding friction is determined based on the limiting friction angle between the foundation soil and the reinforced soil backfill. Based on the soil parameters listed in Section 5.2.1 for the foundation and reinforced soil mass, a coefficient of sliding friction at the rear abutment of 0.51 was utilized for design. For undrained conditions, the sliding resistance is taken as the limiting value between the undrained shear strength of the bearing soil and half of the vertical stress applied by the wall multiplied by the width of the MSE wall. Based on the soil parameters listed in Section 5.2.1, the undrained shear strength of the bearing material at the rear abutment is 3.75 ksf.

A geotechnical resistance factor of φ_{τ}=1.0 was considered in calculating the factored shear resistance between the reinforced soil mass and foundation soil for sliding. Based on the minimum length of reinforced soil mass presented in Table 10 and utilizing the soil parameters listed in Section 5.2.1 for the retained embankment material, the resultant horizontal forces on the back of the MSE wall **will not exceed** the factored shear resistance at the strength limit state under drained or undrained conditions at the rear abutment.

5.2.6 Overall (Global) Stability

A slope stability analysis was performed to check the global stability of the MSE wall at the rear abutment. As per Section 11.6.2.3 of the 2020 AASHTO LRFD BDS, safety against soil failure shall be evaluated at the service limit state by assuming the reinforced soil mass to be a rigid body. Soil parameters utilized in external stability analyses are presented in Section 5.2.1. For the global stability condition, it was considered that the failure plane will not cross through the reinforced soil mass. The computer software program Slide, manufactured by Rocscience Inc., was utilized to perform the analysis.

Per Section 307.1.2 of the 2020 ODOT BDM and Section 11.6.2.3 of the 2020 AASHTO LRFD BDS, overall (global) stability for MSE walls that are integrated with or supporting structural foundations or elements is satisfied if the product of the factor of safety from the slope stability output multiplied by the resistance factor φ=0.65 is greater than 1.0. Therefore, global stability is satisfied when a minimum factor of safety of 1.5 is obtained. For MSE walls designed with the minimum strap lengths listed in Table 10, the resulting factors of safety under drained conditions (long-term stability) and undrained conditions (short-term stability) using the Spencer’s analysis method was greater than 1.5 at the rear abutment.

5.2.7 Final MSE Wall Considerations

Based on the results of the external and global stability analysis performed for the MSE wall at the rear abutment (Retaining Wall E9), the recommended controlling strap length is 0.90 times the maximum height of the MSE wall (measured from the top of the leveling pad to the proposed profile grade of the roadway). Global stability under drained conditions was the controlling factor in the determination of the recommended strap length of 90 percent of the wall height at the rear abutment.

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

5.3 Lightweight (Cellular Concrete) Wall Recommendations

The footprint of the wall at the forward abutment (Wall E7) will be located along the existing spill through slope, with the wall facing near the existing forward pier. Based on the conditions encountered in boring B-020-7-13, existing fill consisting of loose to medium dense gravel with sand and silt (ODOT A-2-4) overlying medium stiff to stiff silty clay and clay (ODOT A-6b, A-7-6) was encountered at the proposed bearing elevation at the forward abutment, which extends to a depth of 17.0 feet below the bottom of wall elevation. As noted in Sections 5.0 and 5.2, given the presence of existing fill material to significant depths, as well as the significant amount of existing utilities present along the east side of Short Street, it is understood that lightweight fill material consisting of cellular concrete is being considered to be utilized as the backfill along the length of Retaining Wall E7 where it crosses in front of the forward abutment (between Sta. 705+61 and 706+28, BL Wall E7). The use of the lightweight cellular concrete will eliminate the need for undercut or ground improvement to stabilize the underlying existing fill material and control settlement to tolerable limits. Based on information provided by the Rii design group, two types of lightweight cellular concrete will be utilized in lieu of typical embankment fill and select granular fill, which is typically used for MSE wall applications. The wall facing will be connected to geosynthetic straps that are embedded into the cellular concrete and supported on a leveling pad, similar to traditional MSE walls.

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

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

Where,

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

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

Since the wall is located within an existing floodplain, the analysis was performed using a design groundwater level at the ground surface.

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

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

The active earth pressure coefficient, and consequently the active pressure on the back of the cellular concrete mass, will greatly reduce as the slope of the backfill soil flattens. Once the slope of the backfill flattens more than the internal friction angle of the backfill soil, the active earth pressure coefficient will go to zero. Therefore, if the backslope of any backfill is reduced to the internal friction angle of the backfill material, analysis of external stability is not required, with the exception of bearing and overall (global) stability. Based on the plan information provided, it is understood that the cellular concrete fill will be placed the full height of the embankment within the limits of I-70 westbound to the west side of the Franklin main, which is approximately 150 feet east of Short Street. Provided that all backslopes cut into the existing I-70 embankment are graded no steeper than 2H:1V, external and global stability calculations will not be required for this section of Retaining Wall E7. However, if bearing resistance must be checked, then a factored bearing resistance of 2.86 ksf should be utilized for design at the strength limit state.

The compressibility parameters utilized in the settlement analysis of the proposed cellular concrete backfilled MSE wall along Retaining Wall E7 at the forward abutment are provided in Table 11.

Table 13. 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: Loose Gravel with Sand and Silt (ODOT A-2-4)	120	N/A	N/A	N/A	N/A	N/A	8	66
Existing Fill: Medium Stiff to Stiff Clay (ODOT A-7-6)	115	43	0.297	0.045	0.608	150	N/A	N/A
Existing Fill: Medium Stiff to Stiff Silty Clay (ODOT A-6b)	115	38	0.252	0.038	0.569	300	N/A	N/A
Dense to Very Dense Granular Soils (ODOT A-1-a, A-2-4)	130 to 135	N/A	N/A	N/A	N/A	N/A	33 to 89	115 to 309
Hard Silt and Clay (ODOT A-6a)	125 to 130	27	0.153	0.015	0.483	600	N/A	N/A

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

Results of the settlement analysis are tabulated in Table 12. A total settlement of 4.05 inches at the center of the wall mass and 2.93 inches at the facing of the wall is anticipated along Retaining Wall E7 where is crosses in front of the forward abutment. Based on the results of the analysis, 90 percent of the total settlement is anticipated to occur within 50 days following construction of the wall. Please note that the consolidation settlement and time rate of consolidation are based on estimates using correlated compressibility parameters provided in Table 11 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 14. Retaining Wall E7 Settlement Results

Structure Reference / Substructure Unit (Boring)	Wall / Embankment Height (feet)	Pressure at Bottom of Wall / Embankment 1 (ksf)	Total Settlement Values (inches)		Time for 90% Consolidation (Days)
			Center of Wall Mass	Facing of Wall	
Forward Abutment / Retaining Wall E7 (Sta. 705+61 and 706+28) (B-020-7-13)	48.2	1.76	4.05	2.93	50

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

Per Section 307.1.6 of the 2020 ODOT BDM, the maximum allowable differential settlement in the longitudinal direction (regardless of the size of panels) is one (1) percent (1/100). Based on the total anticipated settlement at the facing of the wall, maximum differential settlement in the longitudinal direction is anticipated to be less than 1/1,000, which is within the tolerable limit of 1/100. If localized bearing pressures exerted on the leveling pad from the wall facing panels will be higher than the pressure exerted by the wall mass, then there is a potential for differential settlement to occur given the variability in the fill material.

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

5.4 Lateral Earth Pressure

For the soil types encountered in the borings, the “in-situ” unit weight (γ), cohesion (c), effective angle of friction (φ’), and lateral earth pressure coefficients for at-rest conditions (k_o), active conditions (k_a), and passive conditions (k_p) have been estimated and are provided in Table 15 and Table 16.

Table 15. Estimated Undrained (Short-term) Soil Parameters for Design

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

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

Table 16. Estimated Drained (Long-term) Soil Parameters for Design

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

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

These parameters are considered appropriate for the design of all subsurface structures and any excavation support systems. Subsurface structures (where the top of the structure is restrained from movement) should be designed based on at-rest conditions (k_o). For proposed temporary retaining structures (where the top of the structure is allowed to move), earth pressure distributions should be based on active (k_a) and passive (k_p) conditions. Active earth pressure is developed as the structure moves away from the backfill or retained soil, while passive pressure is developed as the structure moves towards the backfill. A relatively small amount of lateral movement is needed to reach the active condition (≥ 0.1 percent of the height), whereas the movements required to engage the passive condition are approximately ten times greater than those required to develop active earth pressure. The values in this table have been estimated from correlation charts based on minimum standards specified for compacted engineered fill materials.

These recommendations do not take into consideration the effect of any surcharge loading or a sloped ground surface (a flat surface is assumed). Earth pressures on excavation support systems will be dependent on the type of sheeting and method of bracing or anchorage. Surcharge loads, such as that imposed by traffic loading, will create additional lateral loading on the subsurface structures and excavation support systems. The resulting lateral earth pressure should be evaluated based on active (k_a) and at-rest (k_o) conditions and the anticipated magnitude of the loading.

Where necessary, temporary retaining structures such as sheet pile system should be designed using the undrained soil parameters provided in Table 15, and the design should follow all applicable guidelines for the type of retaining structure utilized. Permanent retaining and subsurface structures should be designed using the drained soil parameters provided in Table 16. Regardless of whether the retaining structure is temporary or permanent, the effective unit weight ($\gamma' = \gamma$ - 62.4 pcf) plus the hydrostatic water pressure ($\gamma_w * h_w$, where h_w is the height of water behind the wall above the base of the wall) should be utilized below the design groundwater level. The lateral earth pressure coefficients should only be applied to the horizontal pressure resulting from the effective overburden pressure, and should not be applied to the hydrostatic water pressure.

5.5 Construction Considerations

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

5.5.1 Excavation Considerations

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

Table 17. Excavation Back Slopes

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

5.5.2 Groundwater Considerations

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

6.0 LIMITATIONS OF STUDY

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

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

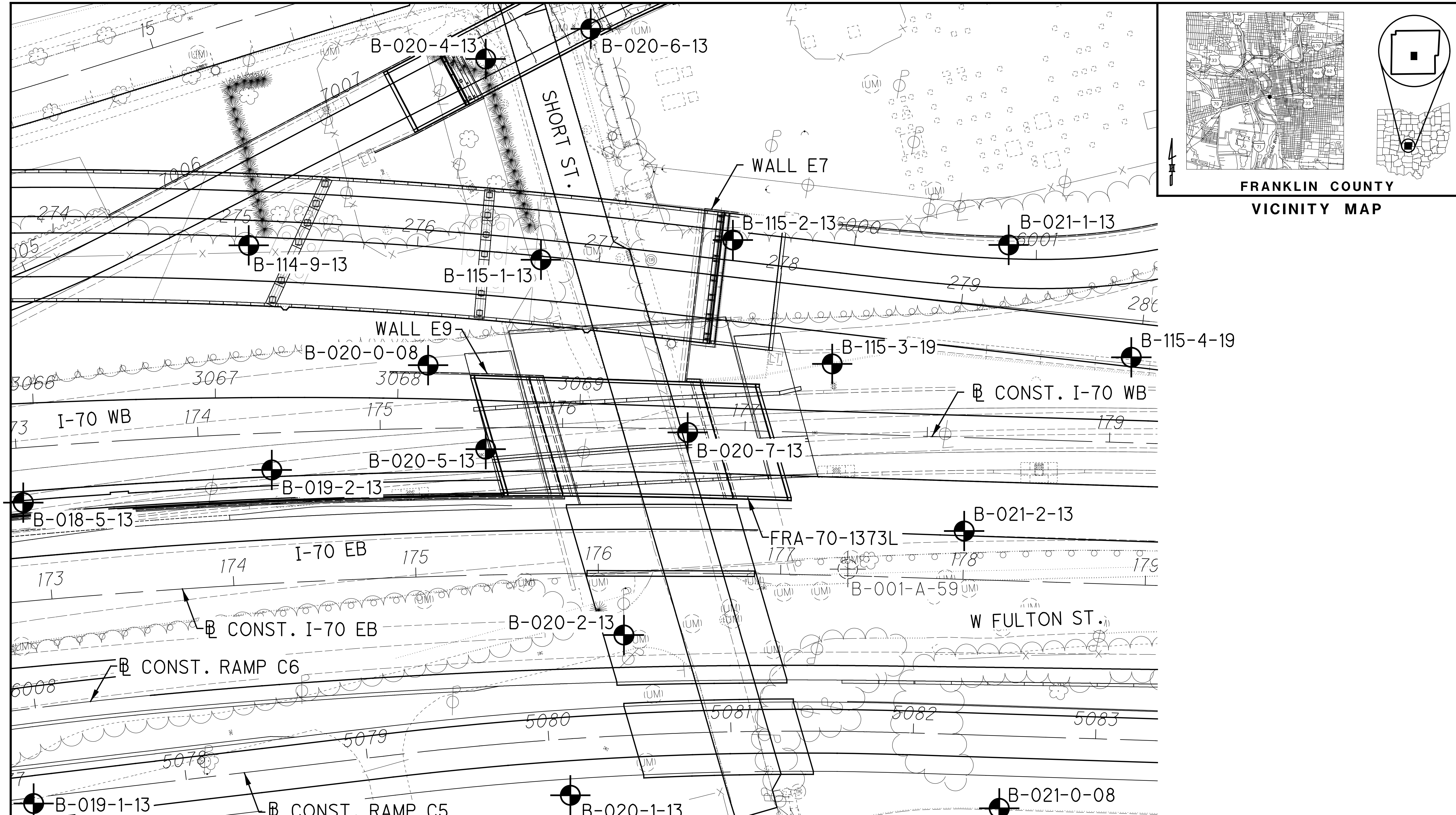
The conclusions and recommendations herein have been based upon the available soil and bedrock information and the design details furnished by a representative of the owner of the proposed project. Any revision in the plans for the proposed construction from those anticipated in this report should be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

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

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

APPENDIX I

VICINITY MAP AND BORING PLAN



BORING PLAN

FRA-70-1373L

FRANKLIN COUNTY, OHIO

PROJECT NO.	Rii W-13-072
SCALE: 1"=50'	0 25 50
DRAWN	RRM
REVIEWED	BRT
DATE	3/8/2021

RESOURCE INTERNATIONAL, INC.

APPENDIX II

DESCRIPTION OF SOIL TERMS

DESCRIPTION OF SOIL TERMS

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

DESCRIPTION OF ROCK TERMS

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

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

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

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

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

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

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

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

Degree of Fracturing

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

Aperture Width

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

Surface Roughness

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

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

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

CLASSIFICATION OF SOILS

Ohio Department of Transportation

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

SYMBOL	DESCRIPTION	Classifcation		$LL_O/LL \times 100$*	% Pass #40	% Pass #200	Liquid Limit (LL)	Plastic Index (PI)	Group Index Max.	REMARKS
		AASHTO	OHIO							
	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.

APPENDIX III

PROJECT BORING LOGS:

B-020-5-13 and B-020-7-13

BORING LOGS

Definitions of Abbreviations

AS	=	Auger sample
GI	=	Group index as determined from the Ohio Department of Transportation classification system
HP	=	Unconfined compressive strength as determined by a hand penetrometer (tons per square foot)
LL_o	=	Oven-dried liquid limit as determined by ASTM D4318. Per ASTM D2487, if LL_o/LL is less than 75 percent, soil is classified as “organic”.
LOI	=	Percent organic content (by weight) as determined by ASTM D2974 (loss on ignition test)
PID	=	Photo-ionization detector reading (parts per million)
QR	=	Unconfined compressive strength of intact rock core sample as determined by ASTM D2938 (pounds per square inch)
QU	=	Unconfined compressive strength of soil sample as determined by ASTM D2166 (pounds per square foot)
RC	=	Rock core sample
REC	=	Ratio of total length of recovered soil or rock to the total sample length, expressed as a percentage
RQD	=	Rock quality designation – estimate of the degree of jointing or fracture in a rock mass, expressed as a percentage: $\frac{\sum \text{segments equal to or longer than 4.0 inches}}{\text{core run length}} \times 100$
S	=	Sulfate content (parts per million)
SPT	=	Standard penetration test blow counts, per ASTM D1586. Driving resistance recorded in terms of blows per 6-inch interval while letting a 140-pound hammer free fall 30 inches to drive a 2-inch outer diameter (O.D.) split spoon sampler a total of 18 inches. The second and third intervals are added to obtain the number of blows per foot (N_m).
N_{60}	=	Measured blow counts corrected to an equivalent (60 percent) energy ratio (ER) by the following equation: $N_{60} = N_m$*(ER/60)
SS	=	Split spoon sample
2S	=	For instances of no recovery from standard SS interval, a 2.5 inch O.D. split spoon is driven the full length of the standard SS interval plus an additional 6.0 inches to obtain a representative sample. Only the final 6.0 inches of sample is retained. Blow counts from 2S sampling are not correlated with N_{60} values.
3S	=	Same as 2S, but using a 3.0 inch O.D. split spoon sampler.
TR	=	Top of rock
W	=	Initial water level measured during drilling
▼	=	Water level measured at completion of drilling

Classification Test Data

Gradation (as defined on Description of Soil Terms):

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

Atterberg Limits:

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

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

EXPLORATION ID B-020-5-13

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

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

MATERIAL DESCRIPTION AND NOTES	ELEV. 703.4	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
HARD, BROWN **SILTY CLAY**, LITTLE COARSE TO FINE SAND, LITTLE FINE GRAVEL, MOIST. *(same as above)*		31 – 32	9 / 14 / 15	40	72	SS-9	4.50	-	-	-	-	-	-	-	-	19	A-6b (V)	
	700.4	33																
MEDIUM DENSE TO VERY DENSE, BROWN TO REDDISH BROWN **GRAVEL WITH SAND AND SILT**, LITTLE CLAY, MOIST.		34 – 35	5 / 6 / 6	17	56	SS-10	-	-	-	-	-	-	-	-	-	14	A-2-4 (V)	
		36 – 37	10 / 8 / 10	25	100	SS-11	-	33	21	12	25	9	24	19	5	13	A-2-4 (0)	
		38 – 40	15 / 20 / 30	69	83	SS-12	-	-	-	-	-	-	-	-	-	13	A-2-4 (V)	
	692.9																	
MEDIUM DENSE TO VERY DENSE, BROWN TO GRAY **GRAVEL WITH SAND**, TRACE SILT, TRACE CLAY, DAMP TO MOIST.		41 – 42	19 / 19 / 17	50	78	SS-13	-	-	-	-	-	-	-	-	-	6	A-1-b (V)	
-INTRODUCED MUD @ 43.0'		W 43																
		44 – 45	18 / 19 / 24	59	56	SS-14	-	-	-	-	-	-	-	-	-	8	A-1-b (V)	
		46 – 47	11 / 7 / 10	23	100	SS-15	-	23	46	18	9	4	NP	NP	NP	14	A-1-b (0)	
-ROCK FRAGMENTS PRESENT THROUGHOUT		48																
		49 – 50	15 / 23 / 27	69	56	SS-16	-	-	-	-	-	-	-	-	-	8	A-1-b (V)	
		51 – 53																
-STRONG PETROLEUM ODOR PRESENT IN SS-17		54	50/5"	-	100	SS-17	-	-	-	-	-	-	-	-	-	13	A-1-b (V)	
	676.4	55 – 57																
HARD, GRAY **SANDY SILT**, SOME CLAY, TRACE FINE GRAVEL, MOIST.		58 – 60	15 / 30 / 50/5"	-	59	SS-18	4.50	5	9	18	45	23	25	15	10	17	A-4a (7)	
	671.4	61																

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

MATERIAL DESCRIPTION AND NOTES	ELEV. 671.3	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
VERY DENSE, BROWN **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, DAMP. *(same as above)*	669.2	63 – 64																
HARD, BROWN **SILT AND CLAY**, "AND" COARSE TO FINE SAND, LITTLE FINE GRAVEL, DAMP.		64 – 65	8 30 45	103	61	SS-19	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	
							4.50	-	-	-	-	-	-	-	-	9	A-6a (V)	
-ROCK FRAGMENTS PRESENT IN SS-20		69 – 70	25 30 50	110	78	SS-20	4.50	20	13	23	24	20	25	12	13	10	A-6a (3)	
	661.4																	
HARD, BROWN **SILTY CLAY**, LITTLE FINE GRAVEL, TRACE COARSE TO FINE SAND, DAMP.		74 – 75	24 17 30	65	89	SS-21	4.50	-	-	-	-	-	-	-	-	17	A-6b (V)	
AUGER REFUSAL @ 75.5'	657.9																	
GRANITE BOULDER	656.8	76 – 76.6 (TR)			62	RC-1	-	-	-	-	-	-	-	-	-	-		
-BORING TERMINATED @ 76.6' ON 2-23-14. RESUMED DRILLING ON 1-22-15 AND CONTINUED SAMPLING @ 78.5'.																		
SHALE : GRAY, HIGHLY WEATHERED, VERY WEAK.		79 – 80	49 29 37	91	17	SS-22	-	-	-	-	-	-	-	-	-	13	Rock (V)	
	653.4																	
MUDSTONE : GRAY, SLIGHTLY WEATHERED, VERY WEAK TO WEAK, VERY THIN TO THICK BEDDED, ARENACEOUS, CALCAREOUS, FRIABLE, FISSILE, PYRITIC, SLIGHTLY TO HIGHLY FRACTURED, TIGHT TO OPEN APERTURES, ROUGH TO VERY ROUGH; RQD 73%, REC 88%. -0.3' GRANITE BOULDER @ 80.0' -0.3' LIMESTONE SEAM @ 82.6'		80 – 85	63		80	RC-2											CORE	
	645.4	85 – 90	72		100	RC-3											CORE	
SHALE : BLACK AND GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, VERY WEAK TO SLIGHTLY STRONG, LAMINATED TO THICK BEDDED, ARENACEOUS, CALCAREOUS, FRIABLE, FISSILE, MODERATELY TO HIGHLY FRACTURED, TIGHT TO OPEN APERTURES, ROUGH TO VERY ROUGH; RQD 46%, REC 100%.	643.4	90 (EOB)																

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 43.5'

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

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

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

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-13.10 - PHASE 6A	DRILLING FIRM / OPERATOR: STOCK / J/M	DRILL RIG: CME 55-LC (SN 360485)	STATION / OFFSET: 176+68.64 / 1.8' RT
TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / K.R.	HAMMER: AUTOMATIC	ALIGNMENT: BL I-70 WB
PID: 89464 BR ID: FRA-70-1373L	DRILLING METHOD: 4.25" HSA / NQ	CALIBRATION DATE: 3/28/13	ELEVATION: 713.5 (MSL) EOB: 80.4 ft.
START: 1/19/15 END: 1/22/15	SAMPLING METHOD: SPT / RC	ENERGY RATIO (%): 73.2	LAT / LONG: 39.953452, -83.004377

EXPLORATION ID **B-020-7-13**

MATERIAL DESCRIPTION AND NOTES	ELEV. 713.5
0.7' - CONCRETE (8.0")	712.8
POSSIBBLE FILL: LOOSE TO MEDIUM DENSE, BROWN **GRAVEL WITH SAND AND SILT**, TRACE CLAY, MOIST.	706.5
POSSIBBLE FILL: STIFF, DARK BROWN **CLAY**, "AND" SILT, TRACE COARSE TO FINE SAND, MOIST. -SWITCHED TO ROTARY DRILLING TECHNIQUES WITH WATER AND CASING ADVANCER @ 10.0' -CONSOLIDATION TEST PERFORMED @ 11.8' -CU TRIAXIAL COMPRESSION TEST PERFORMED @ 12.0'	700.5
POSSIBBLE FILL: MEDIUM STIFF TO STIFF, BROWN **SILTY CLAY**, TRACE FINE SAND, TRACE FINE GRAVEL, MOIST.	696.4
POSSIBBLE FILL: HARD, REDDISH BROWN **SANDY SILT**, LITTLE FINE GRAVEL, LITTLE CLAY, MOIST.	695.5
POSSIBBLE FILL: MEDIUM STIFF, BROWN **CLAY**, "AND" SILT, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST.	693.0
VERY DENSE, BLACK **GRAVEL**, LITTLE COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST. -COBBLES PRESENT @ 24.0'	686.5
DENSE, BROWN AND BLACK **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, MOIST. -HEAVING SANDS ENCOUNTERED @ 28.5'	

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
5 4 4	10	83	SS-1	-	-	-	-	-	-	-	-	-	12	A-2-4 (V)	
WOH 2 3	6	67	SS-2	-	25	31	17	19	8	NP	NP	NP	13	A-2-4 (0)	
2 4 6	12	11	SS-3	-	-	-	-	-	-	-	-	-	23	A-7-6 (V)	
		71	ST-4	1.50	0	2	7	45	46	43	19	24	23	A-7-6 (14)	
2 3 4	9	81	S-5	1.25	1	0	8	49	42	38	19	19	26	A-6b (12)	
		63	ST-6	0.75	-	-	-	-	-	-	-	-	-	A-6b (V)	
				4.50	19	14	12	40	15	26	21	5	21	A-4a (4)	
1 2 3	6	33	SS-7	-	-	-	-	-	-	-	-	-	24	A-7-6 (V)	
		0	ST-8	-	-	-	-	-	-	-	-	-	-		
7 19 20	48	61	SS-9	-	78	11	5	4	2	NP	NP	NP	11	A-1-a (0)	
5 13 14	33	100	SS-10	-	-	-	-	-	-	-	-	-	15	A-1-b (V)	

DEPTHS: 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29

PID: 89464	BR ID: FRA-70-1373L	PROJECT: FRA-70-13.10 - PHASE 6A	STATION / OFFSET: 176+68.64 / 1.8 RT	START: 1/19/15	END: 1/22/15	PG 2 OF 3	B-020-7-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 683.5	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, BROWN AND BLACK **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, MOIST. *(same as above)*	681.5	31 – 32																
HARD, BROWNISH GRAY **SILT AND CLAY**, SOME COARSE TO FINE SAND, LITTLE FINE GRAVEL, MOIST.	679.5	32 – 34	11 20 33	65	72	SS-11	4.5+	11	11	19	33	26	27	15	12	13	A-6a (6)	
VERY DENSE, BLACK **GRAVEL WITH SAND**, TRACE SILT, TRACE CLAY, MOIST.		34 – 35					-	-	-	-	-	-	-	-	-	9	A-1-b (V)	
		39 – 40	6 20 38	71	94	SS-12	-	54	14	22	7	3	NP	NP	NP	14	A-1-b (0)	
-COBBLES PRESENT @ 40.0'	671.5	40 – 42																
HARD, GRAY **SILT AND CLAY**, SOME COARSE TO FINE SAND, LITTLE FINE GRAVEL, DAMP.		44 – 45	4 6 21	33	44	SS-13	4.50	-	-	-	-	-	-	-	-	15	A-6a (V)	
	666.5	45 – 47																
VERY DENSE, GRAY AND BLACK **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST.		49 – 50	9 32 41	89	100	SS-14	-	70	13	9	5	3	NP	NP	NP	10	A-1-a (0)	
	659.5	50 – 54	5 20 39	72	78	SS-15	-	-	-	-	-	-	-	-	-	12	A-1-a (V)	
HARD, GRAY **SILT AND CLAY**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST.		54 – 55					4.5+	-	-	-	-	-	-	-	-	13	A6a (V)	
AUGER REFUSAL @ 55.0'		55 – 56.5			25	RC-1	-	-	-	-	-	-	-	-	-	-	A-6a (V)	
-0.8' GRANITE BOULDER @ 56.5'		56.5 – 61.5			33	RC-2	-	-	-	-	-	-	-	-	-	-	A-6a (V)	
-0.8' MUDSTONE SEAM @ 60.7'																		

PID: 89464	BR ID: FRA-70-1373L	PROJECT: FRA-70-13.10 - PHASE 6A	STATION / OFFSET: 176+68.64 / 1.8 RT	START: 1/19/15	END: 1/22/15	PG 3 OF 3	B-020-7-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 651.4	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
HARD, GRAY **SILT AND CLAY**, LITTLE COARSE TO FINE SAND, TRACE FINE GRAVEL, MOIST. *(same as above)* -0.1' THICK PIECE OF GRANITE RECOVERED IN RC-3. REMAINING SOIL WASHED OUT DURING CORING.	648.1	63 – 65			2	RC-3	-	-	-	-	-	-	-	-	-	-	A-6a (V)	
MUDSTONE : GRAY, HIGHLY WEATHERED, VERY WEAK, THINLY LAMINATED TO THIN BEDDED, ARENACEOUS, CALCAREOUS, FRIABLE, FISSILE, SLIGHTLY TO HIGHLY FRACTURED, THIGHT APERTURES, SLIGHTLY ROUGH; RQD 74%, REC 98%. -POINT LOAD STRENGTH @ 69.4' to 74.8' -MEAN QU = 110 PSI		TR – 66 – 75	89		97	RC-4											CORE	
	633.1	76 – 80 EOB	45		100	RC-5											CORE	

NOTES: GROUNDWATER NOT ENCOUNTERED PRIOR TO INTORDUCTION OF WATER AS A DRILLING FLUID

ABANDONMENT METHODS, MATERIALS, QUANTITIES: COMPACTED WITH THE AUGER 150 LBS BENTONITE CHIPS AND SOIL CUTTINGS

B-020-7-13 – RC-1, RC-2, RC-3 and RC-4 – Depth from 55.0 to 73.5 feet

B-020-7-13 – RC-4 (cont.) and RC-5 – Depth from 73.5 to 80.4 feet

APPENDIX IV

LABORATORY TEST RESULTS

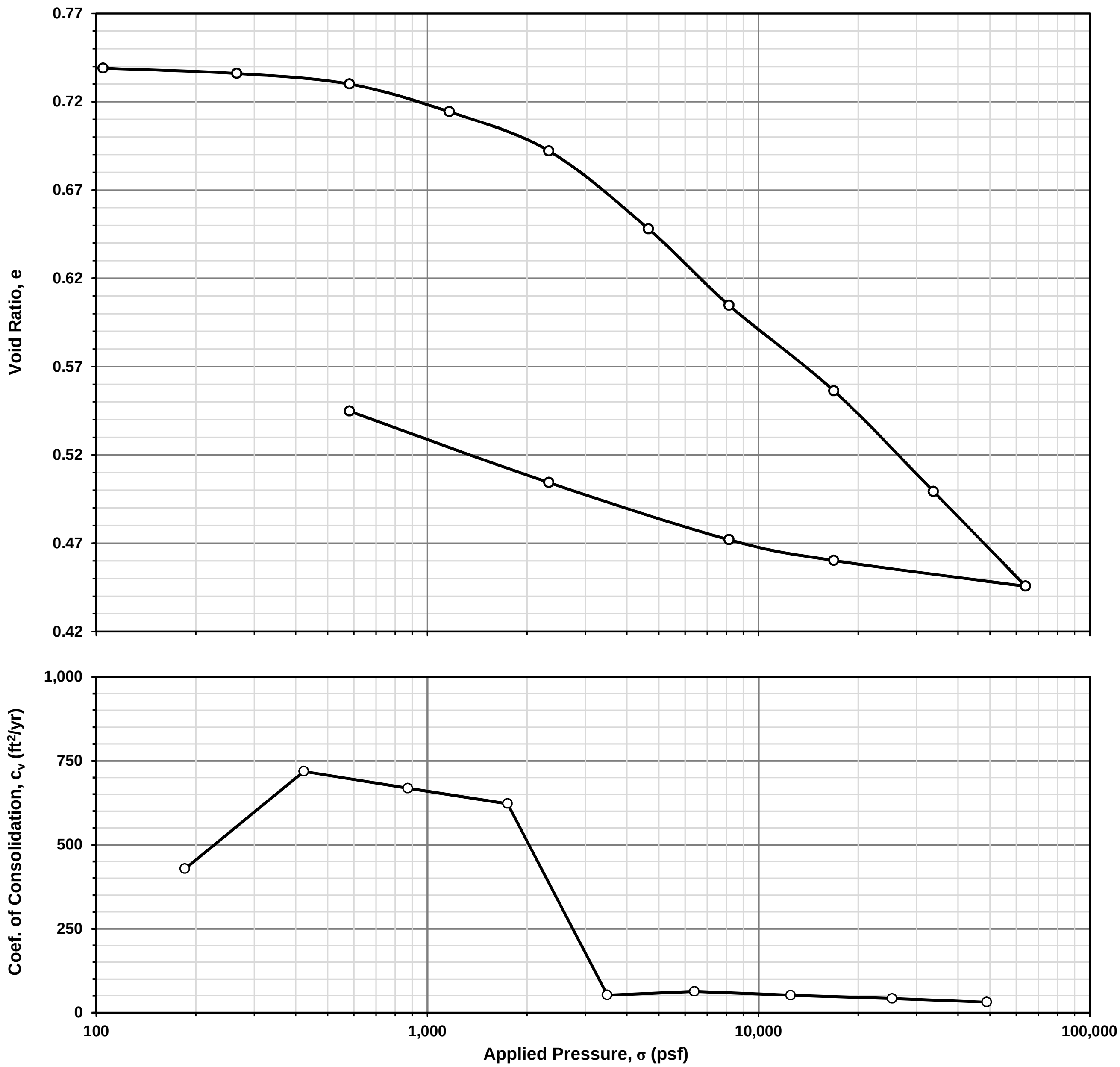
One-Dimensional Consolidation Test Report (ASTM D2435)

Project Number:	W-13-072	Boring Number:	B-020-7-13
Project Name:	FRA-70-13.10	Station / Offset:	176+68.64, 1.8' Rt.
Project Location:	Columbus, Ohio	Sample No. / Depth:	ST-4 / 11.8 ft
Client:	ms consultants, inc.	Date of Testing:	01/27/2015 to 02/12/2015

Soil Description: Dark brown CLAY, "and" silt, trace coarse to fine sand
Soil Classification: ODOT A-7-6

Physical Characteristics	L.L.	P.L.	P.I.	Gravel%	C. Sand%	F. Sand%	Silt%	Clay%
	43	19	24	0	2	7	45	46

Natural		γ_d (pcf)	γ_{sat} (pcf)	σ_{vo}' (psf)	S_G	e_o	σ_p' (psf)	c_c	c_r
S_o	w_o								
99.6%	23.3%	95.5	122.0	1,357	2.67	0.745	3,449	0.210	0.049



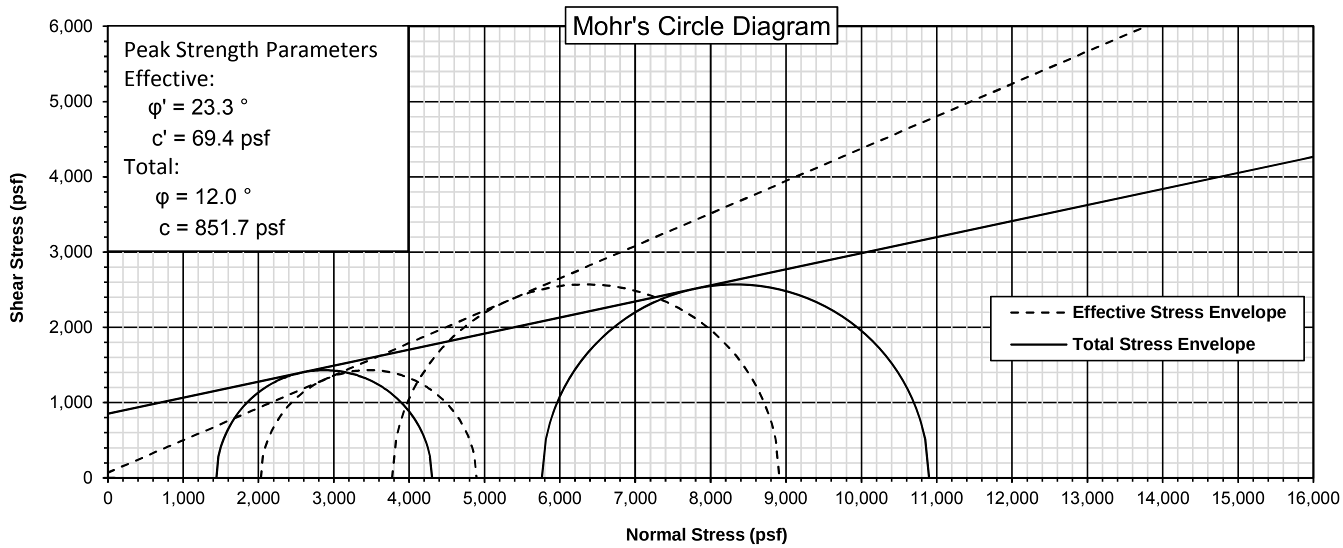
Consolidated, Undrained Triaxial Compression Test Report (ASTM D4767)

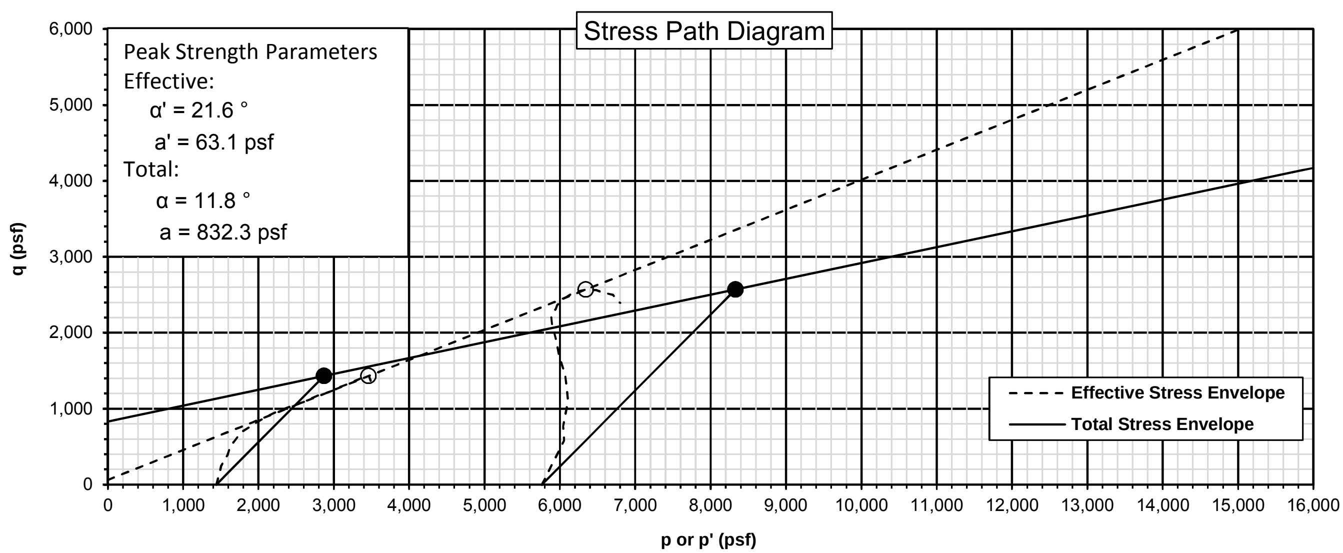
Project Number:	W-13-072	Boring Number:	B-020-7-13
Project Name:	FRA-70-13.10	Station / Offset:	176+68.64, 1.8' Rt.
Project Location:	Franklin County, Ohio	Sample No. / Depth:	ST-4 / 12.0 ft to 13.0 ft
Client:	ms consultants	Date of Testing:	01/28/2015 to 02/10/2015

Soil Description: Dark brown CLAY, "and" silt, trace coarse to fine sand
Soil Classification: ODOT A-7-6

Physical Characteristics	L.L.	P.L.	P.I.	Gravel%	C. Sand%	F. Sand%	Silt%	Clay%
	43	19	24	0	2	7	45	46

Stage	Boring No.	Sample No.	Depth (ft)	$(\sigma_3)_f$ (psf)	$(\sigma_1)_f$ (psf)	$(\sigma_3')_f$ (psf)	$(\sigma_1')_f$ (psf)	p'_f (psf)	q_f (psf)
1	B-020-7-13	ST-4	12.0-12.5	1,440.0	4,302.7	2,030.4	4,893.1	3,461.8	1,431.4
2	B-020-7-13	ST-4	12.5-13.0	5,760.0	10,900.3	3,772.8	8,913.1	6,343.0	2,570.2
3									





Notes:

Consolidated, Undrained Triaxial Compression Test (ASTM D4767)

Project Number:	W-13-072	Boring Number:	B-020-7-13
Project Name:	FRA-70-13.10	Station / Offset:	176+68.64, 1.8' Rt.
Project Location:	Franklin County, Ohio	Sample No. / Depth:	ST-4 / 12.0-12.5 ft
Client:	ms consultants	Date of Testing:	2/10/2015

Data for Specimen No. 1

Soil Description: Dark brown CLAY, "and" silt, trace coarse to fine sand

Soil Classification: ODOT A-7-6

Physical Characteristics	L.L.	P.L.	P.I.	Gravel%	C. Sand%	F. Sand%	Silt%	Clay%
	43	19	24	0	2	7	45	46

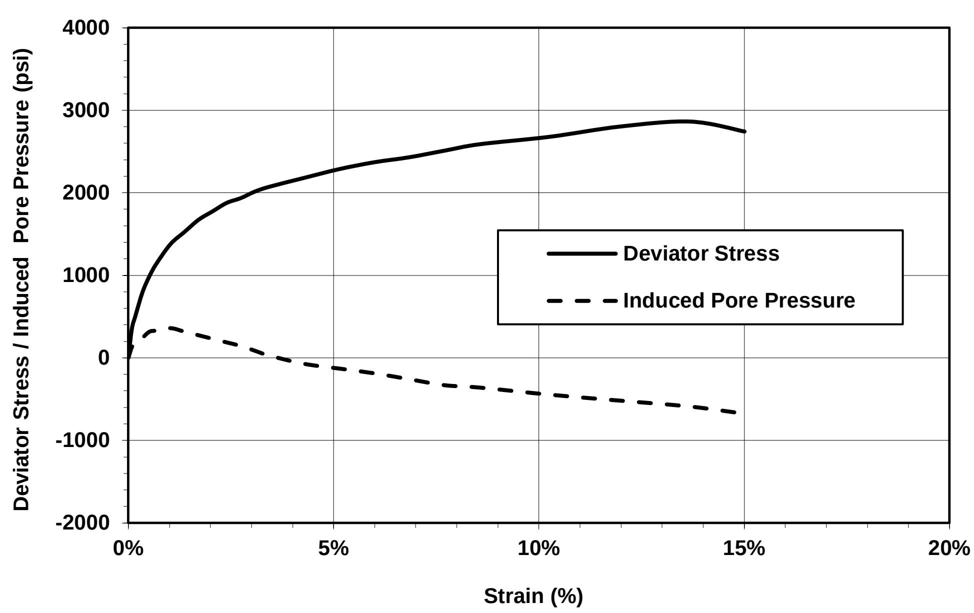
Diameter, D_0	2.872	in	Volume of Solids , V_s :	23.242	in^3
Area, A_0	6.478	in^2	Initial Volume of Voids, V_v :	15.246	in^3
Height, L_0	5.941	in	Initial Void Ratio, e_o :	0.656	
Volume, V_0	38.487	in^3	Initial Degree of Saturation, S_0 :	94.7	%

Water Content BEFORE Test			Water Content AFTER Test (Total Specimen)		
Tin No.:	X-16	g	Tin No.:	FUNKY	g
Wet Soil + Tin :	113.18	g	Wet Soil + Tin :	1316.50	g
Dry Soil + Tin :	97.47	g	Dry Soil + Tin :	1073.70	g
Tin Weight :	29.97	g	Tin Weight :	56.80	g
Dry Mass :	67.5	g	Dry Mass :	1016.90	g
Weight of water :	15.71	g	Weight of water :	242.80	g
Moisture :	23.27	%	Moisture :	23.88	%
			Wet Density :	124.69	pcf
			Dry Density :	100.65	pcf

Consolidation Cell Pressure:	140.0	psi	Deviator Stress @ Failure, Ds:	2,863	psf
Consolidation Back Pressure:	130.0	psi	Axial Strain @ Failure:	13.7	%
Effective Confining Stress, σ_3:	10.0	psi	Major Principal Stress @ Failure, σ_1:	4,303	psf
	1,440	psf	Induced Pore Pressure @ Failure:	-590	psf
Strain Rate:	0.0030	in/min	Effective Minor Principal Stress,σ'_3:	2,030	psf
			Effective Major Principal Stress, σ'_1:	4,893	psf

Failure Sketch

CU Compressive Strength and Induced Pore Pressure



Notes:

Consolidated, Undrained Triaxial Compression Test (ASTM D4767)

Project Number:	W-13-072	Boring Number:	B-020-7-13
Project Name:	FRA-70-13.10	Station / Offset:	176+68.64, 1.8' Rt.
Project Location:	Franklin County, Ohio	Sample No. / Depth:	ST-4 / 12.5-13.0 ft
Client:	ms consultants	Date of Testing:	10/11/2014

Data for Specimen No. 2

Soil Description: Dark brown CLAY, "and" silt, trace coarse to fine sand
Soil Classification: ODOT A-7-6

Physical Characteristics	L.L.	P.L.	P.I.	Gravel%	C. Sand%	F. Sand%	Silt%	Clay%
	43	19	24	0	2	7	45	46

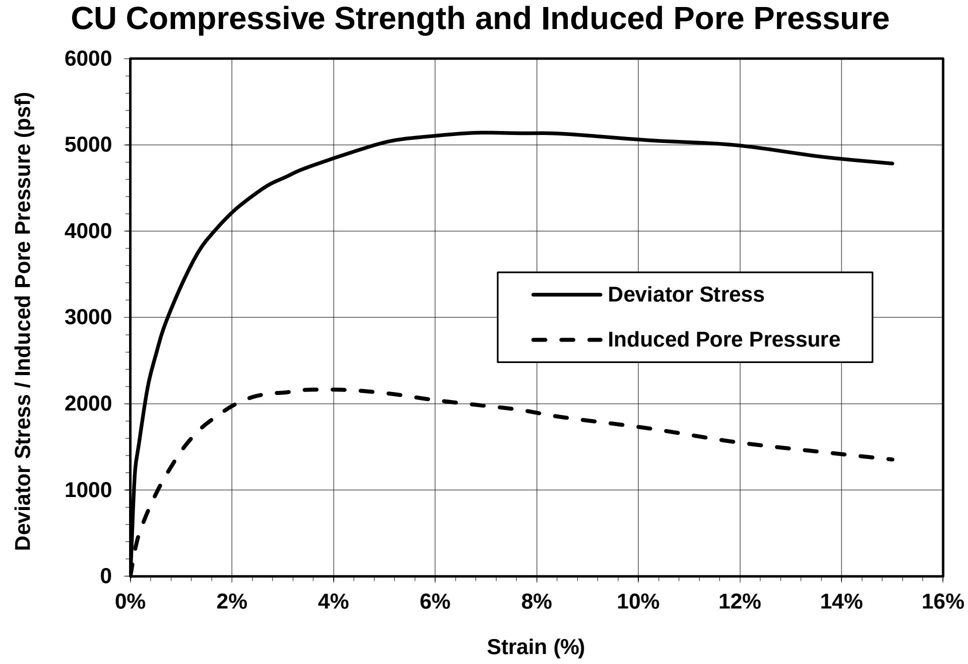
Diameter, D_0	2.875	in	Volume of Solids , V_s :	23.795	in^3
Area, A_0	6.492	in^2	Initial Volume of Voids, V_v :	14.946	in^3
Height, L_0	5.968	in	Initial Void Ratio, e_o :	0.628	
Volume, V_0	38.741	in^3	Initial Degree of Saturation, S_0 :	98.93	%

Water Content BEFORE Test			Water Content AFTER Test (Total Specimen)		
Tin No.:	X-16	g	Tin No.:	FUNKY	g
Wet Soil + Tin :	113.18	g	Wet Soil + Tin :	1325.30	g
Dry Soil + Tin :	97.47	g	Dry Soil + Tin :	1097.20	g
Tin Weight :	29.97	g	Tin Weight :	56.10	g
Dry Mass :	67.5	g	Dry Mass :	1041.10	g
Weight of water :	15.71	g	Weight of water :	228.10	g
Moisture :	23.27	%	Moisture :	21.91	%
			Wet Density :	124.80	pcf
			Dry Density :	102.37	pcf

Consolidation Cell Pressure:	143.0	psi	Deviator Stress @ Failure, Ds:	5,140	psf
Consolidation Back Pressure:	103.0	psi	Axial Strain @ Failure:	6.8	%
Effective Confining Stress, σ_3:	40.0	psi	Major Principal Stress @ Failure, σ_1:	10,900	psf
	5,760	psf	Induced Pore Pressure @ Failure:	1,987	psf
Strain Rate:	0.0030	in/min	Effective Minor Principal Stress,σ'_3:	3,773	psf
			Effective Major Principal Stress, σ'_1:	8,913	psf

Failure Sketch

CU Compressive Strength and Induced Pore Pressure



Notes:

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
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Point Load Strength Index of Rock Specimens

(ASTM D 5731-08)

Project: FRA-70-13.10
Project No.: W-13-072
Date of Testing: 2/2/2015
Test Performed by: E.M.

Rock Description: Gray Mudstone

Boring No.: B-020-7-13
Station / Offset: 176+68.64, 1.8' Rt.
Sample No. / Depth: RC-4 / 69.4' to 74.8'

Test Apparatus: Forney-LA 0080
Serial Number: A125/AZ/0014
Date of Calibration: 8/9/2014

Sample No.	Test Type	Depth (ft)	Width (mm)	Diameter (mm)	Load (N)	D_e^2 (mm^2)	D_e (mm)	F	Is (MPa)	$Is_{(50)}$ (MPa)	σ_c (MPa)
1	a ⊥	69.4	37.0	46.5	70	2,192	46.8	0.97	0.03	0.03	0.38
2	a ⊥	70.6	37.1	46.0	185	2,174	46.6	0.97	0.09	0.08	1.02
3	a ⊥	70.9	35.8	45.5	195	2,078	45.6	0.96	0.09	0.09	1.13
4	a ⊥	73.8	36.7	45.9	105	2,143	46.3	0.97	0.05	0.05	0.59
5	a ⊥	74.8	34.2	45.6	110	1,983	44.5	0.95	0.06	0.05	0.67
6											
7											
8											
9											
10											

STATISTICS	
Mean $Is_{(50)}$ ⊥	**0.06 MPa (9 psi)**
Mean $Is_{(50)}$ ∥	
$Ia_{(50)}$	

Specific Specimen Shape:

d = diametrical
a = axial
b = block
i = irregular lump
⊥ = perpendicular to bedding plane
∥ = parallel to bedding plane

Estimated Unaxial Compression, σ_c = K*Is

K = 12

*Per Section 206.1.3 of 2011 ODOT Rock Slope Design Guide

Mean σ_c = 0.76 MPa (110 psi)

Remarks:

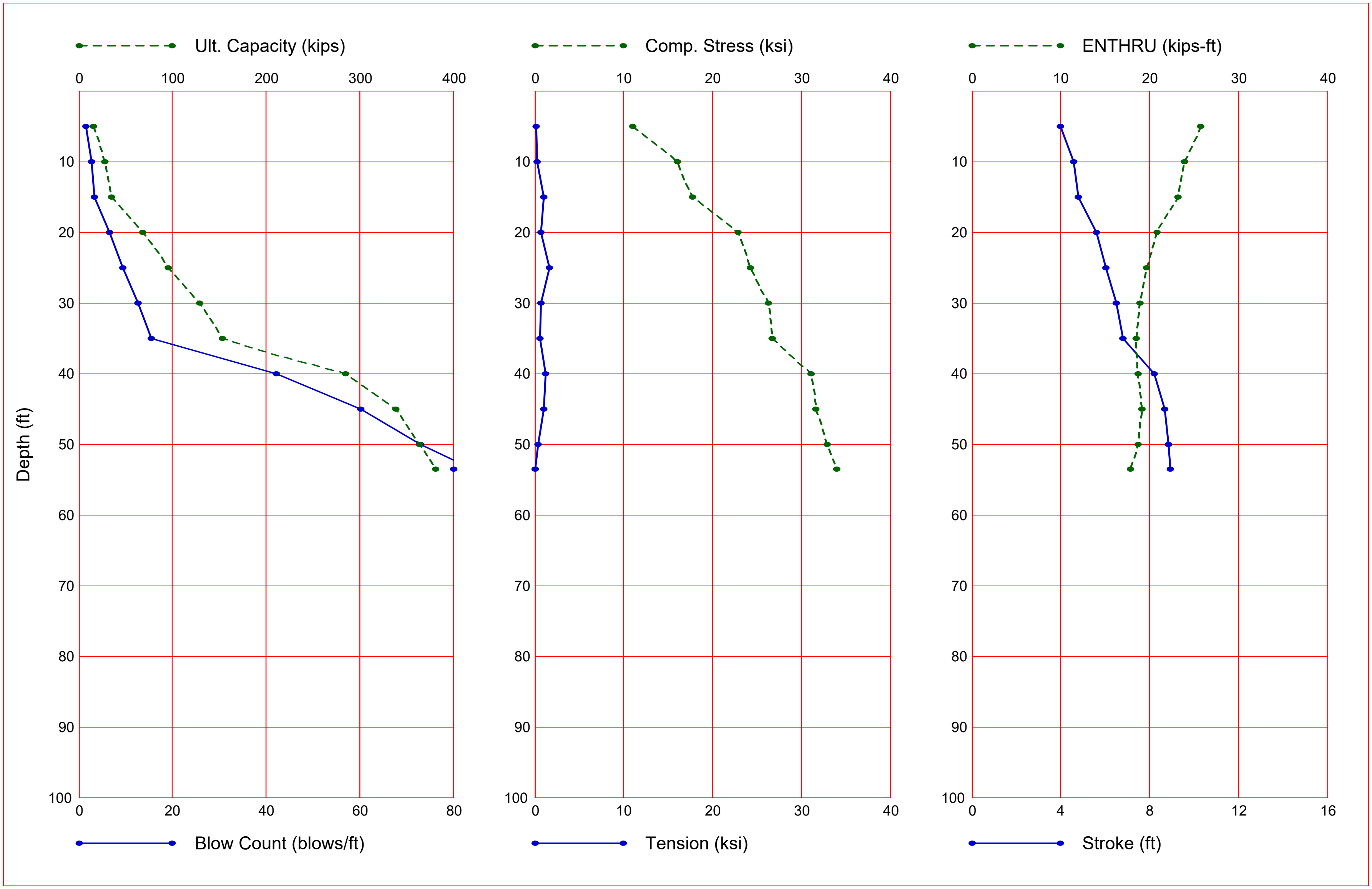
APPENDIX V

GRLWEAP DRIVEABILITY ANALYSIS OUTPUTS

Resource International Inc
FRA-70-1373L - RA - B-020-5-13 - HP12x53

Mar 01 2021
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	16.1	12.5	3.6	1.6	10.996	-0.200	3.98	25.8
10.0	28.0	24.3	3.6	2.8	16.096	-0.231	4.58	23.9
15.0	34.8	30.4	4.4	3.3	17.773	-1.049	4.78	23.2
20.0	68.3	46.7	21.6	6.5	22.844	-0.704	5.59	20.8
25.0	96.1	68.6	27.6	9.4	24.286	-1.622	6.03	19.7
30.0	129.3	95.7	33.6	12.7	26.257	-0.678	6.50	18.9
35.0	153.5	145.7	7.8	15.4	26.764	-0.622	6.81	18.5
40.0	285.3	277.5	7.8	42.3	31.070	-1.226	8.21	18.7
45.0	338.3	330.5	7.8	60.3	31.613	-1.060	8.68	19.1
50.0	363.6	355.9	7.8	73.1	32.861	-0.382	8.85	18.7
53.5	381.3	373.6	7.8	84.2	34.000	-0.020	8.94	17.9

Total Continuous Driving Time 30.00 minutes; Total Number of Blows 1226 (starting at penetration 5.0 ft)

GRLWEAP - Version 2010
WAVE EQUATION ANALYSIS OF PILE FOUNDATIONS

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

ABOUT THE WAVE EQUATION ANALYSIS RESULTS

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

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

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

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

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

```
 Input  File: J:\GEOTECH\PROJECTS\2013\W-13-072 FRA-70-13.10 PROJECT 6A\ANALYSIS\FRA-70-1373L\DRIVEABILITY\B-020-5-13\B-020-5-13.GWW
 Hammer File: C:\ProgramData\PDI\GRLWEAP\2010\Resource\HAMMER2010.GW
 Hammer File Version:  2003 (12/4/2018)

                       Input File Contents
     FRA-70-1373L - RA - B-020-5-13 - HP12x53
 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    12.000            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
    53.500     15.50   29000.0   492.000     3.970         0     0.850     0.010
  FFatigue        F0  0-Bottom
         0     0.000     0.000
 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    0.0020    0.0020     1.250      0.00      0.00
     P atm        P1        P2        P3        P4        P5
     14.70   1600.00   1440.00   1295.00   1165.00      0.00
    Stroke    Effic.  Pressure  R-Weight   T-Delay Exp-Coeff   Eps-Str  Total-AW
   10.8100    0.8000 1600.0000    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
Research Toe Plug: Res-int, Q-int, D-int, Res-plug, Q-plug, D-plug
0.000 0.000 0.000 0.000 0.000 0.000
Research Toe Plug: RD plug toe: m, d
0.000 0.000
Research Toe Plug: New Toe Plug Model is NOT applied
Res. Distribution

Dpth	Rskn	Rtoe	Qs	Qt	Js	Jt	SU F	LimL	TSf0
0.01	0.98	4.00	0.10	0.10	0.20	0.15	1.49	6.00	168.000
2.59	0.98	4.00	0.10	0.10	0.20	0.15	1.49	6.00	168.000
2.61	0.89	3.63	0.10	0.10	0.20	0.15	1.49	6.00	168.000
10.09	0.89	3.63	0.10	0.10	0.20	0.15	1.49	6.00	168.000
10.11	0.29	2.96	0.10	0.10	0.05	0.15	1.21	6.00	24.000
15.09	0.43	4.43	0.10	0.10	0.05	0.15	1.21	6.00	24.000
15.11	0.70	15.67	0.10	0.10	0.05	0.15	1.00	6.00	1.000
24.11	1.19	26.50	0.10	0.10	0.05	0.15	1.00	6.00	1.000
33.11	1.67	37.32	0.10	0.10	0.05	0.15	1.00	6.00	1.000
34.09	1.72	38.50	0.10	0.10	0.05	0.15	1.00	6.00	1.000
34.11	8.00	7.75	0.10	0.10	0.15	0.15	1.21	6.00	24.000
41.29	8.00	7.75	0.10	0.10	0.15	0.15	1.21	6.00	24.000
41.31	1.90	7.75	0.10	0.10	0.20	0.15	1.49	6.00	168.000
49.09	1.90	7.75	0.10	0.10	0.20	0.15	1.49	6.00	168.000
49.11	1.90	7.75	0.10	0.10	0.20	0.15	1.49	6.00	168.000
53.50	1.90	7.75	0.10	0.10	0.20	0.15	1.49	6.00	168.000

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	0.0	0.000	0.000	0.000
10.00	0.00	0.00	0.000	0.0	0.000	0.000	0.000
15.00	0.00	0.00	0.000	0.0	0.000	0.000	0.000
20.00	0.00	0.00	0.000	0.0	0.000	0.000	0.000
25.00	0.00	0.00	0.000	0.0	0.000	0.000	0.000
30.00	0.00	0.00	0.000	0.0	0.000	0.000	0.000
35.00	0.00	0.00	0.000	0.0	0.000	0.000	0.000
40.00	0.00	0.00	0.000	0.0	0.000	0.000	0.000
45.00	0.00	0.00	0.000	0.0	0.000	0.000	0.000
50.00	0.00	0.00	0.000	0.0	0.000	0.000	0.000
53.50	0.00	0.00	0.000	0.0	0.000	0.000	0.000
0.00	0.00	0.00	0.000	0.0	0.000	0.000	0.000

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

FRA-70-1373L - RA - B-020-5-13 - HP12x53

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	140046.6	1.000	0.0000	
3	0.800	140046.6	1.000	0.0000	
4	0.800	140046.6	1.000	0.0000	
5	0.800	140046.6	1.000	0.0000	
Imp Block	0.753	70735.6	0.900	0.0100	
Helmet	1.900	60155.0	0.800	0.0100	5.8
Combined Pile Top		11202.5			

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)	1600.00	Actual Pressure	(psi)	1600.00
Compression Exponent		1.350	Expansion Exponent		1.250
Ram Diameter	(inch)	12.60			
Combustion Delay	(s)	0.00200	Ignition Duration	(s)	0.00200

The Hammer Data Includes Estimated (NON-MEASURED) Quantities

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

FRA-70-1373L - RA - B-020-5-13 - HP12x53 03/01/2021
Resource International Inc GRLWEAP Version 2010

Depth	(ft)	5.0	Standard Soil Setup	
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)	12.000		

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	15.50	29000.	492.0	4.0	0	16524.	27.2
53.5	15.50	29000.	492.0	4.0	0	16524.	27.2

Wave Travel Time 2L/c (ms) 6.475

Pile and Soil Model Total Capacity Rut (kips) 14.9

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	0.177	11202	0.010	0.000	0.85	0.0	0.000	0.100	3.34	4.0	15.5
2	0.177	11202	0.000	0.000	1.00	0.0	0.000	0.100	6.69	4.0	15.5
15	0.177	11202	0.000	0.000	1.00	3.9	0.200	0.100	50.16	4.0	15.5
16	0.177	11202	0.000	0.000	1.00	7.4	0.200	0.100	53.50	4.0	15.5
Toe						3.6	0.150	0.100			

2.833 kips total unreduced pile weight (g= 32.17 ft/s2)
2.833 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)	0.544
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-70-1373L - RA - B-020-5-13 - HP12x53 03/01/2021
Resource International Inc GRLWEAP Version 2010

Rut	Bl Ct	Stroke (ft) down	Stroke (ft) up	Ten Str	i	t	Comp Str	i	t	ENTHRU	Bl Rt
kips	b/ft			ksi			ksi			kip-ft	b/min
14.9	1.5	3.88	3.89	-0.11	10	12	9.90	1	2	25.5	60.1
15.5	1.5	3.95	3.92	-0.21	9	11	10.57	1	2	25.7	59.7
16.1	1.6	3.98	3.95	-0.20	9	11	11.00	1	2	25.8	59.5
16.7	1.6	4.02	3.99	-0.18	9	11	11.37	1	2	25.8	59.2
17.3	1.7	4.05	4.03	-0.20	5	11	11.73	1	2	25.8	59.0

FRA-70-1373L - RA - B-020-5-13 - HP12x53 03/01/2021
Resource International Inc GRLWEAP Version 2010

Depth	(ft)	10.0	Standard Soil Setup	
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)	12.000		

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	15.50	29000.	492.0	4.0	0	16524.	27.2
53.5	15.50	29000.	492.0	4.0	0	16524.	27.2

Wave Travel Time 2L/c (ms) 6.475

Pile and Soil Model					Total Capacity Rut (kips)		25.6			
No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Perim	Area
	kips	k/in	ft	ft		kips	s/ft	inch	ft	ft	in2
1	0.177	11202	0.010	0.000	0.85	0.0	0.000	0.100	3.34	4.0	15.5
2	0.177	11202	0.000	0.000	1.00	0.0	0.000	0.100	6.69	4.0	15.5
14	0.177	11202	0.000	0.000	1.00	7.6	0.200	0.100	46.81	4.0	15.5
15	0.177	11202	0.000	0.000	1.00	7.2	0.200	0.100	50.16	4.0	15.5
16	0.177	11202	0.000	0.000	1.00	7.2	0.200	0.100	53.50	4.0	15.5
Toe						3.6	0.150	0.100			

2.833 kips total unreduced pile weight (g= 32.17 ft/s2)
2.833 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

FRA-70-1373L - RA - B-020-5-13 - HP12x53 03/01/2021
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
25.6	2.5	4.42	4.47	-0.20	3	11	15.09	1	2	24.1	56.2
26.8	2.6	4.52	4.51	-0.26	3	11	15.71	1	2	24.1	55.7
28.0	2.8	4.58	4.56	-0.23	3	11	16.10	1	2	23.9	55.4
29.2	2.9	4.63	4.61	-0.22	3	11	16.48	1	2	23.7	55.1
30.4	3.0	4.69	4.67	-0.20	3	11	16.84	1	2	23.5	54.8

FRA-70-1373L - RA - B-020-5-13 - HP12x53 03/01/2021
Resource International Inc GRLWEAP Version 2010

Depth (ft) 15.0 Standard Soil Setup
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) 12.000

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	15.50	29000.	492.0	4.0	0	16524.	27.2
53.5	15.50	29000.	492.0	4.0	0	16524.	27.2

Wave Travel Time 2L/c (ms) 6.475

Pile and Soil Model					Total Capacity Rut (kips)		32.1			
No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Perim	Area
	kips	k/in	ft	ft		kips	s/ft	inch	ft	ft	in2
1	0.177	11202	0.010	0.000	0.85	0.0	0.000	0.100	3.34	4.0	15.5
2	0.177	11202	0.000	0.000	1.00	0.0	0.000	0.100	6.69	4.0	15.5
12	0.177	11202	0.000	0.000	1.00	3.8	0.200	0.100	40.12	4.0	15.5
13	0.177	11202	0.000	0.000	1.00	7.4	0.200	0.100	43.47	4.0	15.5
14	0.177	11202	0.000	0.000	1.00	7.2	0.200	0.100	46.81	4.0	15.5
15	0.177	11202	0.000	0.000	1.00	5.4	0.165	0.100	50.16	4.0	15.5
16	0.177	11202	0.000	0.000	1.00	4.0	0.050	0.100	53.50	4.0	15.5
Toe						4.4	0.150	0.100			

2.833 kips total unreduced pile weight (g= 32.17 ft/s2)
2.833 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
32.1	3.0	4.68	4.65	-1.01	15	13	16.98	1	2	23.6	54.8
33.4	3.1	4.73	4.71	-1.03	15	13	17.37	12	4	23.4	54.5
34.8	3.3	4.78	4.76	-1.05	15	13	17.77	12	4	23.2	54.2
36.1	3.4	4.83	4.81	-1.06	15	13	18.15	12	4	23.0	53.9
37.4	3.6	4.89	4.86	-1.10	15	13	18.50	12	4	22.9	53.7

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Depth (ft) 20.0 Standard Soil Setup
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) 12.000

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	15.50	29000.	492.0	4.0	0	16524.	27.2
53.5	15.50	29000.	492.0	4.0	0	16524.	27.2

Wave Travel Time 2L/c (ms) 6.475

Pile and Soil Model — Total Capacity Rut (kips) 65.6

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	0.177	11202	0.010	0.000	0.85	0.0	0.000	0.100	3.34	4.0	15.5
2	0.177	11202	0.000	0.000	1.00	0.0	0.000	0.100	6.69	4.0	15.5
11	0.177	11202	0.000	0.000	1.00	7.6	0.200	0.100	36.78	4.0	15.5
12	0.177	11202	0.000	0.000	1.00	7.2	0.200	0.100	40.12	4.0	15.5
14	0.177	11202	0.000	0.000	1.00	3.7	0.064	0.100	46.81	4.0	15.5
15	0.177	11202	0.000	0.000	1.00	6.9	0.050	0.100	50.16	4.0	15.5
16	0.177	11202	0.000	0.000	1.00	11.6	0.050	0.100	53.50	4.0	15.5
Toe						21.6	0.150	0.100			

2.833 kips total unreduced pile weight (g= 32.17 ft/s2)
2.833 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

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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
65.6	6.2	5.52	5.50	-0.47	11	50	22.42	11	4	21.0	50.3
66.9	6.4	5.56	5.53	-0.59	11	50	22.63	11	4	20.9	50.1
68.3	6.5	5.59	5.57	-0.70	11	50	22.84	11	4	20.8	50.0
69.6	6.7	5.63	5.60	-0.74	11	50	23.06	11	4	20.8	49.8
70.9	6.9	5.66	5.64	-0.75	11	50	23.23	11	4	20.6	49.6

FRA-70-1373L - RA - B-020-5-13 - HP12x53 03/01/2021
Resource International Inc GRLWEAP Version 2010

Depth (ft) 25.0 Standard Soil Setup
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) 12.000

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	15.50	29000.	492.0	4.0	0	16524.	27.2
53.5	15.50	29000.	492.0	4.0	0	16524.	27.2

Wave Travel Time 2L/c (ms) 6.475

Pile and Soil Model — Total Capacity Rut (kips) 93.5

No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Perim	Area
	kips	k/in	ft	ft		kips	s/ft	inch	ft	ft	in2
1	0.177	11202	0.010	0.000	0.85	0.0	0.000	0.100	3.34	4.0	15.5
2	0.177	11202	0.000	0.000	1.00	0.0	0.000	0.100	6.69	4.0	15.5
9	0.177	11202	0.000	0.000	1.00	3.7	0.200	0.100	30.09	4.0	15.5
10	0.177	11202	0.000	0.000	1.00	7.4	0.200	0.100	33.44	4.0	15.5
11	0.177	11202	0.000	0.000	1.00	7.2	0.200	0.100	36.78	4.0	15.5
12	0.177	11202	0.000	0.000	1.00	5.4	0.166	0.100	40.12	4.0	15.5
13	0.177	11202	0.000	0.000	1.00	4.0	0.050	0.100	43.47	4.0	15.5
14	0.177	11202	0.000	0.000	1.00	10.2	0.050	0.100	46.81	4.0	15.5
15	0.177	11202	0.000	0.000	1.00	12.8	0.050	0.100	50.16	4.0	15.5
16	0.177	11202	0.000	0.000	1.00	15.2	0.050	0.100	53.50	4.0	15.5
Toe						27.6	0.150	0.100			

2.833 kips total unreduced pile weight (g= 32.17 ft/s2)
2.833 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
```

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FRA-70-1373L - RA - B-020-5-13 - HP12x53                              03/01/2021
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
   93.5     8.9   5.95   6.00   -1.41   9  40    23.95   9   4   19.8    48.2
   94.8     9.2   5.99   6.04   -1.55   9  40    24.12   9   4   19.7    48.1
   96.1     9.4   6.03   6.07   -1.62   9  40    24.29   9   4   19.7    47.9
   97.5     9.6   6.06   6.11   -1.67   9  40    24.47   9   4   19.6    47.8
   98.8     9.8   6.10   6.14   -1.71   9  40    24.62   9   4   19.6    47.6
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FRA-70-1373L - RA - B-020-5-13 - HP12x53                              03/01/2021
Resource International Inc                                   GRLWEAP Version 2010

 Depth                (ft)      30.0  Standard Soil Setup
 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)    12.000

  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    15.50   29000.    492.0      4.0        0    16524.     27.2
     53.5    15.50   29000.    492.0      4.0        0    16524.     27.2

 Wave Travel Time 2L/c (ms)     6.475

        Pile and Soil Model            Total Capacity Rut  (kips)     126.6
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  0.177    11202 0.010 0.000 0.85     0.0   0.000 0.100   3.34   4.0    15.5
  2  0.177    11202 0.000 0.000 1.00     0.0   0.000 0.100   6.69   4.0    15.5
  8  0.177    11202 0.000 0.000 1.00     7.5   0.200 0.100  26.75   4.0    15.5
  9  0.177    11202 0.000 0.000 1.00     7.2   0.200 0.100  30.09   4.0    15.5
 11  0.177    11202 0.000 0.000 1.00     3.7   0.068 0.100  36.78   4.0    15.5
 12  0.177    11202 0.000 0.000 1.00     6.8   0.050 0.100  40.12   4.0    15.5
 13  0.177    11202 0.000 0.000 1.00    11.6   0.050 0.100  43.47   4.0    15.5
 14  0.177    11202 0.000 0.000 1.00    14.0   0.050 0.100  46.81   4.0    15.5
 15  0.177    11202 0.000 0.000 1.00    16.4   0.050 0.100  50.16   4.0    15.5
 16  0.177    11202 0.000 0.000 1.00    18.8   0.050 0.100  53.50   4.0    15.5
Toe                                     33.6   0.150 0.100

     2.833 kips total unreduced pile weight (g= 32.17 ft/s2)
     2.833 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-70-1373L - RA - B-020-5-13 - HP12x53                              03/01/2021
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
  126.6    12.3   6.43   6.46   -0.61  12  35    25.92   8   3   18.9    46.4
  128.0    12.5   6.46   6.49   -0.63   8  35    26.09   8   3   18.9    46.3
  129.3    12.7   6.50   6.52   -0.68   8  35    26.26   8   3   18.9    46.2
  130.6    13.0   6.52   6.55   -0.73   8  34    26.39   8   3   18.8    46.1
  132.0    13.1   6.56   6.57   -0.77   8  34    26.58   8   3   18.8    46.0
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FRA-70-1373L - RA - B-020-5-13 - HP12x53                              03/01/2021
Resource International Inc                                   GRLWEAP Version 2010

 Depth                (ft)      35.0  Standard Soil Setup
 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)    12.000

  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    15.50   29000.    492.0      4.0        0    16524.     27.2
     53.5    15.50   29000.    492.0      4.0        0    16524.     27.2
```

Wave Travel Time 2L/c (ms) 6.475

Pile and Soil Model — Total Capacity Rut (kips) 149.9

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	0.177	11202	0.010	0.000	0.85	0.0	0.000	0.100	3.34	4.0	15.5
2	0.177	11202	0.000	0.000	1.00	0.0	0.000	0.100	6.69	4.0	15.5
6	0.177	11202	0.000	0.000	1.00	3.7	0.200	0.100	20.06	4.0	15.5
7	0.177	11202	0.000	0.000	1.00	7.4	0.200	0.100	23.41	4.0	15.5
8	0.177	11202	0.000	0.000	1.00	7.2	0.200	0.100	26.75	4.0	15.5
9	0.177	11202	0.000	0.000	1.00	5.4	0.167	0.100	30.09	4.0	15.5
10	0.177	11202	0.000	0.000	1.00	4.0	0.050	0.100	33.44	4.0	15.5
11	0.177	11202	0.000	0.000	1.00	10.2	0.050	0.100	36.78	4.0	15.5
12	0.177	11202	0.000	0.000	1.00	12.8	0.050	0.100	40.12	4.0	15.5
13	0.177	11202	0.000	0.000	1.00	15.2	0.050	0.100	43.47	4.0	15.5
14	0.177	11202	0.000	0.000	1.00	17.6	0.050	0.100	46.81	4.0	15.5
15	0.177	11202	0.000	0.000	1.00	20.0	0.050	0.100	50.16	4.0	15.5
16	0.177	11202	0.000	0.000	1.00	38.9	0.114	0.100	53.50	4.0	15.5
Toe						7.8	0.150	0.100			

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

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
35.00	10.81	1.00	0.800

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Rut	Bl Ct	Stroke (ft) down	Stroke (ft) up	Ten Str	i	t	Comp Str	i	t	ENTHRU	Bl Rt
kips	b/ft			ksi			ksi			kip-ft	b/min
149.9	14.8	6.74	6.76	-0.60	7	34	26.47	6	3	18.6	45.4
151.7	15.1	6.78	6.80	-0.61	7	34	26.62	6	3	18.6	45.3
153.5	15.4	6.81	6.83	-0.62	7	34	26.76	6	3	18.5	45.1
155.3	15.7	6.85	6.86	-0.68	6	30	26.93	6	3	18.6	45.0
157.1	16.1	6.89	6.90	-0.71	6	30	27.08	6	3	18.5	44.9

FRA-70-1373L - RA - B-020-5-13 - HP12x53 03/01/2021
Resource International Inc GRLWEAP Version 2010

Depth (ft) 40.0 Standard Soil Setup
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) 12.000

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	15.50	29000.	492.0	4.0	0	16524.	27.2
53.5	15.50	29000.	492.0	4.0	0	16524.	27.2

Wave Travel Time 2L/c (ms) 6.475

Pile and Soil Model — Total Capacity Rut (kips) 276.3

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	0.177	11202	0.010	0.000	0.85	0.0	0.000	0.100	3.34	4.0	15.5
2	0.177	11202	0.000	0.000	1.00	0.0	0.000	0.100	6.69	4.0	15.5
5	0.177	11202	0.000	0.000	1.00	7.4	0.200	0.100	16.72	4.0	15.5
6	0.177	11202	0.000	0.000	1.00	7.2	0.200	0.100	20.06	4.0	15.5
8	0.177	11202	0.000	0.000	1.00	3.7	0.071	0.100	26.75	4.0	15.5
9	0.177	11202	0.000	0.000	1.00	6.8	0.050	0.100	30.09	4.0	15.5
10	0.177	11202	0.000	0.000	1.00	11.6	0.050	0.100	33.44	4.0	15.5
11	0.177	11202	0.000	0.000	1.00	14.0	0.050	0.100	36.78	4.0	15.5
12	0.177	11202	0.000	0.000	1.00	16.4	0.050	0.100	40.12	4.0	15.5
13	0.177	11202	0.000	0.000	1.00	18.7	0.050	0.100	43.47	4.0	15.5
14	0.177	11202	0.000	0.000	1.00	21.1	0.050	0.100	46.81	4.0	15.5
15	0.177	11202	0.000	0.000	1.00	70.0	0.144	0.100	50.16	4.0	15.5
16	0.177	11202	0.000	0.000	1.00	84.5	0.150	0.100	53.50	4.0	15.5
Toe						7.8	0.150	0.100			

2.833 kips total unreduced pile weight (g= 32.17 ft/s2)
2.833 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 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
276.3	39.2	8.20	8.19	-1.15	5	41	30.88	5	3	18.7	41.3
280.8	41.0	8.16	8.23	-1.19	5	41	30.87	5	3	18.6	41.3
285.3	42.3	8.21	8.27	-1.23	5	40	31.07	5	3	18.7	41.2
289.8	43.6	8.25	8.30	-1.25	5	40	31.26	5	3	18.8	41.1
294.3	45.3	8.30	8.35	-1.27	5	40	31.42	5	3	18.7	40.9

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Depth (ft) 45.0 Standard Soil Setup
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) 12.000

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	15.50	29000.	492.0	4.0	0	16524.	27.2
53.5	15.50	29000.	492.0	4.0	0	16524.	27.2

Wave Travel Time 2L/c (ms) 6.475

Pile and Soil Model Total Capacity Rut (kips) 326.0

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	0.177	11202	0.010	0.000	0.85	0.0	0.000	0.100	3.34	4.0	15.5
2	0.177	11202	0.000	0.000	1.00	0.0	0.000	0.100	6.69	4.0	15.5
3	0.177	11202	0.000	0.000	1.00	3.6	0.200	0.100	10.03	4.0	15.5
4	0.177	11202	0.000	0.000	1.00	7.4	0.200	0.100	13.38	4.0	15.5
5	0.177	11202	0.000	0.000	1.00	7.2	0.200	0.100	16.72	4.0	15.5
6	0.177	11202	0.000	0.000	1.00	5.5	0.168	0.100	20.06	4.0	15.5
7	0.177	11202	0.000	0.000	1.00	4.0	0.050	0.100	23.41	4.0	15.5
8	0.177	11202	0.000	0.000	1.00	10.1	0.050	0.100	26.75	4.0	15.5
9	0.177	11202	0.000	0.000	1.00	12.8	0.050	0.100	30.09	4.0	15.5
10	0.177	11202	0.000	0.000	1.00	15.2	0.050	0.100	33.44	4.0	15.5
11	0.177	11202	0.000	0.000	1.00	17.5	0.050	0.100	36.78	4.0	15.5
12	0.177	11202	0.000	0.000	1.00	19.9	0.050	0.100	40.12	4.0	15.5
13	0.177	11202	0.000	0.000	1.00	38.3	0.113	0.100	43.47	4.0	15.5
14	0.177	11202	0.000	0.000	1.00	84.5	0.150	0.100	46.81	4.0	15.5
15	0.177	11202	0.000	0.000	1.00	77.1	0.151	0.100	50.16	4.0	15.5
16	0.177	11202	0.000	0.000	1.00	15.3	0.200	0.100	53.50	4.0	15.5
Toe						7.8	0.150	0.100			

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

Depth ft	Stroke ft	Pressure Ratio	Efficy
45.00	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
326.0	55.1	8.57	8.60	-0.76	4	18	31.25	4	2	18.9	40.3
332.1	57.9	8.63	8.65	-0.88	4	18	31.43	4	2	18.9	40.2
338.3	60.3	8.68	8.68	-1.06	4	18	31.61	3	2	19.1	40.1
344.4	64.0	8.73	8.74	-1.11	4	18	31.79	3	2	19.1	40.0
350.6	67.5	8.79	8.78	-1.18	4	18	31.99	3	2	19.1	39.9

FRA-70-1373L - RA - B-020-5-13 - HP12x53 03/01/2021
Resource International Inc GRLWEAP Version 2010

Depth (ft) 50.0 Standard Soil Setup
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) 12.000

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

B-020-5-13

0.0	15.50	29000.	492.0	4.0	0	16524.	27.2
53.5	15.50	29000.	492.0	4.0	0	16524.	27.2

Wave Travel Time 2L/c (ms)   6.475

Pile and Soil Model   Total Capacity Rut (kips)   348.8

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	0.177	11202	0.010	0.000	0.85	0.0	0.000	0.100	3.34	4.0	15.5
2	0.177	11202	0.000	0.000	1.00	7.4	0.200	0.100	6.69	4.0	15.5
3	0.177	11202	0.000	0.000	1.00	7.2	0.200	0.100	10.03	4.0	15.5
5	0.177	11202	0.000	0.000	1.00	3.8	0.074	0.100	16.72	4.0	15.5
6	0.177	11202	0.000	0.000	1.00	6.7	0.050	0.100	20.06	4.0	15.5
7	0.177	11202	0.000	0.000	1.00	11.6	0.050	0.100	23.41	4.0	15.5
8	0.177	11202	0.000	0.000	1.00	13.9	0.050	0.100	26.75	4.0	15.5
9	0.177	11202	0.000	0.000	1.00	16.3	0.050	0.100	30.09	4.0	15.5
10	0.177	11202	0.000	0.000	1.00	18.7	0.050	0.100	33.44	4.0	15.5
11	0.177	11202	0.000	0.000	1.00	21.1	0.050	0.100	36.78	4.0	15.5
12	0.177	11202	0.000	0.000	1.00	69.4	0.143	0.100	40.12	4.0	15.5
13	0.177	11202	0.000	0.000	1.00	84.5	0.150	0.100	43.47	4.0	15.5
14	0.177	11202	0.000	0.000	1.00	42.8	0.163	0.100	46.81	4.0	15.5
15	0.177	11202	0.000	0.000	1.00	15.3	0.200	0.100	50.16	4.0	15.5
16	0.177	11202	0.000	0.000	1.00	15.3	0.200	0.100	53.50	4.0	15.5
Toe						7.8	0.150	0.100			

2.833 kips total unreduced pile weight (g= 32.17 ft/s2)
2.833 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

↑
FRA-70-1373L - RA - B-020-5-13 - HP12x53 03/01/2021
Resource International Inc GRLWEAP Version 2010

Rut	Bl Ct	Stroke (ft) down	Stroke (ft) up	Ten Str	i	t	Comp Str	i	t	ENTHRU	Bl Rt
kips	b/ft	down	up	ksi			ksi			kip-ft	b/min
348.8	64.0	8.74	8.73	-0.36	2	34	32.37	2	2	18.6	40.0
356.2	68.4	8.80	8.78	-0.37	2	34	32.62	2	2	18.6	39.9
363.6	73.1	8.85	8.83	-0.38	2	33	32.86	2	2	18.7	39.8
371.0	78.2	8.91	8.89	-0.40	2	33	33.10	2	2	18.7	39.7
378.4	84.0	8.97	8.94	-0.41	2	33	33.34	2	2	18.8	39.5

↑
FRA-70-1373L - RA - B-020-5-13 - HP12x53 03/01/2021
Resource International Inc GRLWEAP Version 2010

Depth (ft) 53.5 Standard Soil Setup
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) 12.000

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	15.50	29000.	492.0	4.0	0	16524.	27.2
53.5	15.50	29000.	492.0	4.0	0	16524.	27.2

Wave Travel Time 2L/c (ms)   6.475

Pile and Soil Model   Total Capacity Rut (kips)   364.8

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	0.177	11202	0.010	0.000	0.85	7.7	0.200	0.100	3.34	4.0	15.5
2	0.177	11202	0.000	0.000	1.00	7.2	0.200	0.100	6.69	4.0	15.5
4	0.177	11202	0.000	0.000	1.00	3.6	0.058	0.100	13.38	4.0	15.5
5	0.177	11202	0.000	0.000	1.00	7.0	0.050	0.100	16.72	4.0	15.5
6	0.177	11202	0.000	0.000	1.00	11.7	0.050	0.100	20.06	4.0	15.5
7	0.177	11202	0.000	0.000	1.00	14.1	0.050	0.100	23.41	4.0	15.5
8	0.177	11202	0.000	0.000	1.00	16.4	0.050	0.100	26.75	4.0	15.5
9	0.177	11202	0.000	0.000	1.00	18.8	0.050	0.100	30.09	4.0	15.5
10	0.177	11202	0.000	0.000	1.00	21.2	0.050	0.100	33.44	4.0	15.5
11	0.177	11202	0.000	0.000	1.00	72.3	0.145	0.100	36.78	4.0	15.5
12	0.177	11202	0.000	0.000	1.00	84.5	0.150	0.100	40.12	4.0	15.5
13	0.177	11202	0.000	0.000	1.00	39.6	0.165	0.100	43.47	4.0	15.5
14	0.177	11202	0.000	0.000	1.00	15.3	0.200	0.100	46.81	4.0	15.5
16	0.177	11202	0.000	0.000	1.00	15.3	0.200	0.100	53.50	4.0	15.5
Toe						7.8	0.150	0.100			

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

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

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
53.50	10.81	1.00	0.800

FRA-70-1373L - RA - B-020-5-13 - HP12x53 03/01/2021
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
364.8	72.3	8.83	8.81	-0.43	14	11	33.49	1	2	17.7	39.8
373.1	78.6	8.88	8.87	-0.21	14	11	33.73	1	2	17.7	39.7
381.3	84.2	8.94	8.91	-0.02	14	11	34.00	1	2	17.9	39.6
389.6	92.4	8.99	8.97	0.00	1	0	34.30	1	2	17.8	39.5
397.9	100.6	9.04	9.02	0.00	1	0	34.58	1	2	17.9	39.4

FRA-70-1373L - RA - B-020-5-13 - HP12x53 03/01/2021
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	14.9	11.2	3.6	1.5	9.902	-0.114	3.88	25.5
10.0	25.6	21.9	3.6	2.5	15.091	-0.200	4.42	24.1
15.0	32.1	27.7	4.4	3.0	16.980	-1.006	4.68	23.6
20.0	65.6	44.0	21.6	6.2	22.424	-0.466	5.52	21.0
25.0	93.5	65.9	27.6	8.9	23.950	-1.412	5.95	19.8
30.0	126.6	93.1	33.6	12.3	25.921	-0.613	6.43	18.9
35.0	149.9	142.1	7.8	14.8	26.468	-0.602	6.74	18.6
40.0	276.3	268.5	7.8	39.2	30.878	-1.153	8.20	18.7
45.0	326.0	318.3	7.8	55.1	31.255	-0.764	8.57	18.9
50.0	348.8	341.1	7.8	64.0	32.370	-0.358	8.74	18.6
53.5	364.8	357.1	7.8	72.3	33.487	-0.425	8.83	17.7

Total Driving Time 27 minutes; Total No. of Blows 1114
Starting at penetration 5.0 ft

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	15.5	11.9	3.6	1.5	10.569	-0.210	3.95	25.7
10.0	26.8	23.1	3.6	2.6	15.708	-0.257	4.52	24.1
15.0	33.4	29.0	4.4	3.1	17.375	-1.026	4.73	23.4
20.0	66.9	45.4	21.6	6.4	22.631	-0.588	5.56	20.9
25.0	94.8	67.2	27.6	9.2	24.124	-1.546	5.99	19.7
30.0	128.0	94.4	33.6	12.5	26.092	-0.632	6.46	18.9
35.0	151.7	143.9	7.8	15.1	26.621	-0.607	6.78	18.6
40.0	280.8	273.0	7.8	41.0	30.872	-1.194	8.16	18.6
45.0	332.1	324.4	7.8	57.9	31.433	-0.882	8.63	18.9
50.0	356.2	348.5	7.8	68.4	32.624	-0.370	8.80	18.6
53.5	373.1	365.3	7.8	78.6	33.728	-0.210	8.88	17.7

Total Driving Time 28 minutes; Total No. of Blows 1170
Starting at penetration 5.0 ft

FRA-70-1373L - RA - B-020-5-13 - HP12x53 03/01/2021
Resource International Inc GRLWEAP Version 2010

SUMMARY OVER DEPTHS

G/L at Shaft and Toe: 0.670 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	16.1	12.5	3.6	1.6	10.996	-0.200	3.98	25.8
10.0	28.0	24.3	3.6	2.8	16.096	-0.231	4.58	23.9
15.0	34.8	30.4	4.4	3.3	17.773	-1.049	4.78	23.2
20.0	68.3	46.7	21.6	6.5	22.844	-0.704	5.59	20.8
25.0	96.1	68.6	27.6	9.4	24.286	-1.622	6.03	19.7
30.0	129.3	95.7	33.6	12.7	26.257	-0.678	6.50	18.9
35.0	153.5	145.7	7.8	15.4	26.764	-0.622	6.81	18.5
40.0	285.3	277.5	7.8	42.3	31.070	-1.226	8.21	18.7
45.0	338.3	330.5	7.8	60.3	31.613	-1.060	8.68	19.1
50.0	363.6	355.9	7.8	73.1	32.861	-0.382	8.85	18.7
53.5	381.3	373.6	7.8	84.2	34.000	-0.020	8.94	17.9

Total Driving Time 30 minutes; Total No. of Blows 1226
Starting at penetration 5.0 ft

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	16.7	13.1	3.6	1.6	11.367	-0.182	4.02	25.8
10.0	29.2	25.5	3.6	2.9	16.485	-0.216	4.63	23.7
15.0	36.1	31.7	4.4	3.4	18.153	-1.063	4.83	23.0
20.0	69.6	48.0	21.6	6.7	23.059	-0.737	5.63	20.8
25.0	97.5	69.9	27.6	9.6	24.469	-1.670	6.06	19.6
30.0	130.6	97.1	33.6	13.0	26.390	-0.730	6.52	18.8
35.0	155.3	147.6	7.8	15.7	26.932	-0.677	6.85	18.6
40.0	289.8	282.1	7.8	43.6	31.260	-1.253	8.25	18.8
45.0	344.4	336.7	7.8	64.0	31.791	-1.111	8.73	19.1
50.0	371.0	363.2	7.8	78.2	33.099	-0.397	8.91	18.7
53.5	389.6	381.8	7.8	92.4	34.300	0.000	8.99	17.8

Total Driving Time 31 minutes; Total No. of Blows 1293
Starting at penetration 5.0 ft

FRA-70-1373L - RA - B-020-5-13 - HP12x53 03/01/2021
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	17.3	13.7	3.6	1.7	11.734	-0.199	4.05	25.8
10.0	30.4	26.7	3.6	3.0	16.837	-0.201	4.69	23.5
15.0	37.4	33.0	4.4	3.6	18.503	-1.103	4.89	22.9
20.0	70.9	49.4	21.6	6.9	23.226	-0.751	5.66	20.6
25.0	98.8	71.2	27.6	9.8	24.623	-1.714	6.10	19.6
30.0	132.0	98.4	33.6	13.1	26.575	-0.775	6.56	18.8
35.0	157.1	149.4	7.8	16.1	27.083	-0.706	6.89	18.5
40.0	294.3	286.6	7.8	45.3	31.423	-1.266	8.30	18.7
45.0	350.6	342.8	7.8	67.5	31.988	-1.182	8.79	19.1
50.0	378.4	370.6	7.8	84.0	33.344	-0.413	8.97	18.8
53.5	397.9	390.1	7.8	100.6	34.584	0.000	9.04	17.9

Total Driving Time 33 minutes; Total No. of Blows 1365
Starting at penetration 5.0 ft

FRA-70-1373L - RA - B-020-5-13 - HP12x53 03/01/2021
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	53.50	0.00	10.81	1.00	0.80	1.00	1.00
10.00	53.50	0.00	10.81	1.00	0.80	1.00	1.00
15.00	53.50	0.00	10.81	1.00	0.80	1.00	1.00
20.00	53.50	0.00	10.81	1.00	0.80	1.00	1.00
25.00	53.50	0.00	10.81	1.00	0.80	1.00	1.00
30.00	53.50	0.00	10.81	1.00	0.80	1.00	1.00
35.00	53.50	0.00	10.81	1.00	0.80	1.00	1.00
40.00	53.50	0.00	10.81	1.00	0.80	1.00	1.00
45.00	53.50	0.00	10.81	1.00	0.80	1.00	1.00
50.00	53.50	0.00	10.81	1.00	0.80	1.00	1.00
53.50	53.50	0.00	10.81	1.00	0.80	1.00	1.00

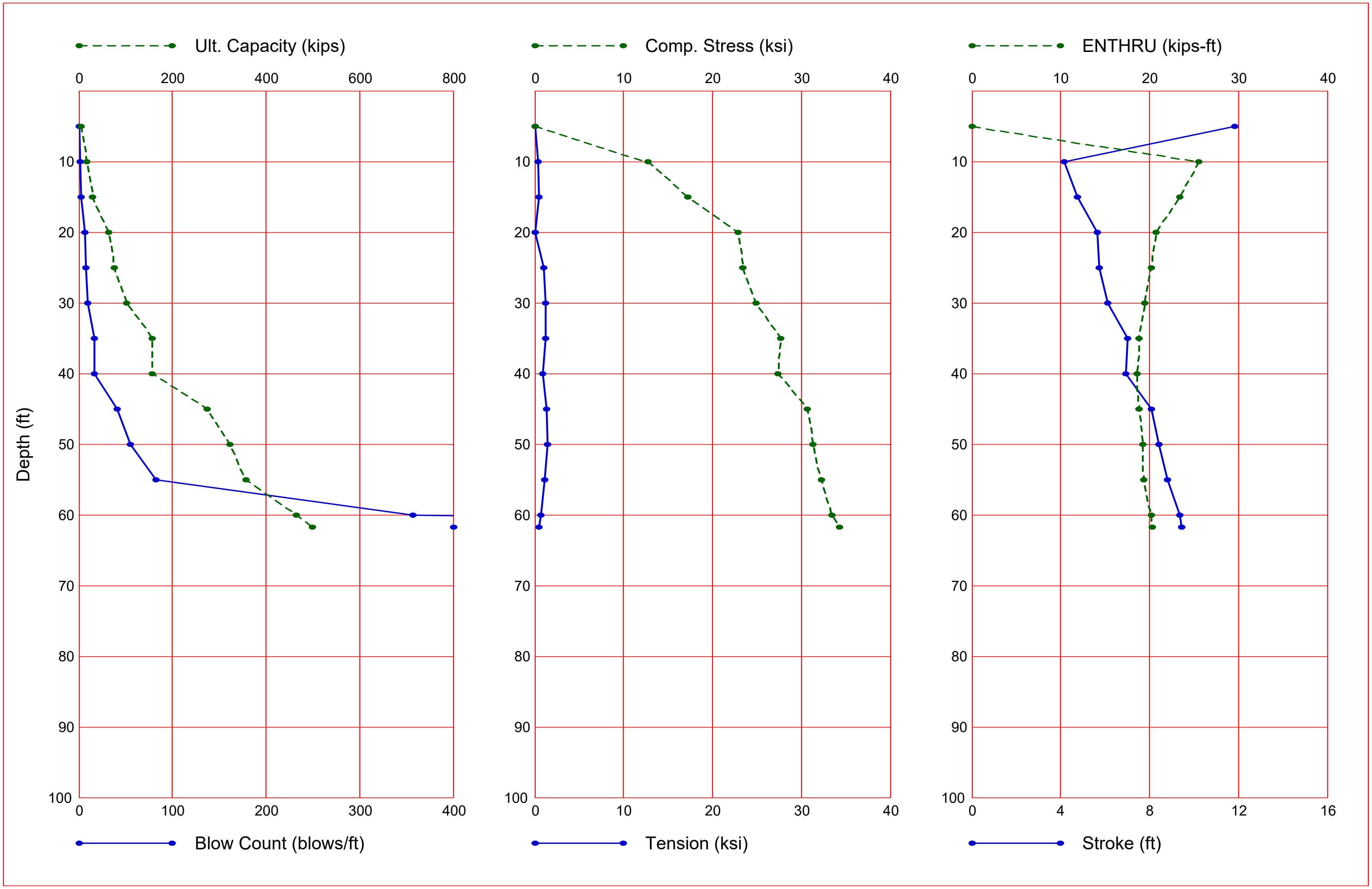
Soil Layer Resistance Values

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	0.98	4.00	0.100	0.100	0.200	0.150	1.000	6.000	168.000
2.59	0.98	4.00	0.100	0.100	0.200	0.150	1.000	6.000	168.000
2.61	0.89	3.63	0.100	0.100	0.200	0.150	1.000	6.000	168.000
10.09	0.89	3.63	0.100	0.100	0.200	0.150	1.000	6.000	168.000
10.11	0.29	2.96	0.100	0.100	0.050	0.150	0.515	6.000	24.000
15.09	0.43	4.43	0.100	0.100	0.050	0.150	0.515	6.000	24.000
15.11	0.70	15.67	0.100	0.100	0.050	0.150	0.000	6.000	1.000
24.11	1.19	26.50	0.100	0.100	0.050	0.150	0.000	6.000	1.000
33.11	1.67	37.32	0.100	0.100	0.050	0.150	0.000	6.000	1.000
34.09	1.72	38.50	0.100	0.100	0.050	0.150	0.000	6.000	1.000
34.11	8.00	7.75	0.100	0.100	0.150	0.150	0.515	6.000	24.000
41.29	8.00	7.75	0.100	0.100	0.150	0.150	0.515	6.000	24.000
41.31	1.90	7.75	0.100	0.100	0.200	0.150	1.000	6.000	168.000

Resource International Inc
FRA-70-1373L - FA - B-020-7-13 - HP12x53

Mar 01 2021
GRLWEAP Version 2010

Gain/Loss 3 at Shaft and Toe 0.500 / 1.000



Gain/Loss 3 at Shaft and Toe 0.500 / 1.000

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
5.0	5.6	4.2	1.5	0.0	0.000	0.000	11.86	0.0
10.0	17.7	16.7	1.1	1.7	12.731	-0.412	4.16	25.5
15.0	30.2	29.5	0.7	3.0	17.268	-0.482	4.76	23.4
20.0	64.4	41.5	23.0	6.6	22.915	0.000	5.65	20.7
25.0	76.3	60.3	15.9	7.4	23.375	-1.017	5.75	20.2
30.0	102.1	82.6	19.5	9.9	24.929	-1.211	6.14	19.5
35.0	156.5	112.1	44.4	17.2	27.716	-1.203	7.01	18.8
40.0	156.6	152.6	4.0	16.7	27.374	-0.933	6.92	18.6
45.0	275.3	203.7	71.6	41.4	30.687	-1.379	8.10	18.8
50.0	322.1	249.3	72.8	54.9	31.290	-1.419	8.42	19.2
55.0	357.6	349.9	7.8	82.3	32.297	-1.146	8.81	19.4
60.0	464.0	456.2	7.8	356.8	33.459	-0.697	9.35	20.2
61.7	499.5	491.7	7.8	1274.3	34.291	-0.507	9.44	20.3

Total Continuous Driving Time 88.00 minutes; Total Number of Blows 3484 (starting at penetration 5.0 ft)

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.

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 Input  File: J:\GEOTECH\PROJECTS\2013\W-13-072 FRA-70-13.10 PROJECT 6A\ANALYSIS\FRA-70-1373L\DRIVEABILITY\B-020-7-13\B-020-7.GWW
 Hammer File: C:\ProgramData\PDI\GRLWEAP\2010\Resource\HAMMER2010.GW
 Hammer File Version:  2003 (12/4/2018)

                       Input File Contents
     FRA-70-1373L - FA - B-020-7-13 - HP12x53
 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    12.000            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
    61.700     15.50   29000.0   492.000     3.970         0     0.850     0.010
  FFatigue        F0  0-Bottom
         0     0.000     0.000
 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    0.0020    0.0020     1.250      0.00      0.00
     P atm        P1        P2        P3        P4        P5
     14.70   1600.00   1440.00   1295.00   1165.00      0.00
    Stroke    Effic.  Pressure  R-Weight   T-Delay Exp-Coeff   Eps-Str  Total-AW
   10.8100    0.8000 1600.0000    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

Research Toe Plug: Res-int, Q-int, D-int, Res-plug, Q-plug, D-plug

0.000	0.000	0.000	0.000	0.000	0.000

Research Toe Plug: RD plug toe: m, d

0.000	0.000

Research Toe Plug: New Toe Plug Model is NOT applied

Res. Distribution

Dpth	Rskn	Rtoe	Qs	Qt	Js	Jt	SU F	LimL	TSf0
0.01	0.00	0.00	0.10	0.10	0.05	0.15	1.21	6.00	1.000
3.49	0.09	0.80	0.10	0.10	0.05	0.15	1.21	6.00	1.000
3.51	1.24	1.45	0.10	0.10	0.20	0.15	2.00	6.00	168.000
9.49	1.24	1.45	0.10	0.10	0.20	0.15	2.00	6.00	168.000
9.51	1.02	1.09	0.10	0.10	0.20	0.15	1.49	6.00	168.000
14.49	1.02	1.09	0.10	0.10	0.20	0.15	1.49	6.00	168.000
14.51	0.71	0.73	0.10	0.10	0.20	0.15	2.00	6.00	168.000
16.99	0.71	0.73	0.10	0.10	0.20	0.15	2.00	6.00	168.000
17.01	0.68	18.55	0.10	0.10	0.05	0.15	1.00	6.00	1.000
23.49	1.04	28.11	0.10	0.10	0.05	0.15	1.00	6.00	1.000
23.51	0.94	14.83	0.10	0.10	0.05	0.15	1.00	6.00	1.000
30.49	1.26	19.88	0.10	0.10	0.05	0.15	1.00	6.00	1.000
30.51	1.39	37.76	0.10	0.10	0.05	0.15	1.00	6.00	1.000
38.49	1.83	49.54	0.10	0.10	0.05	0.15	1.00	6.00	1.000
38.51	4.12	4.00	0.10	0.10	0.20	0.15	1.49	6.00	168.000
43.49	4.12	4.00	0.10	0.10	0.20	0.15	1.49	6.00	168.000
43.51	2.08	71.28	0.10	0.10	0.05	0.15	1.00	6.00	1.000
50.49	2.46	72.94	0.10	0.10	0.05	0.15	1.00	6.00	1.000
50.51	8.00	7.75	0.10	0.10	0.20	0.15	1.49	6.00	168.000
59.51	8.00	7.75	0.10	0.10	0.20	0.15	1.49	6.00	168.000
61.70	7.75	7.75	0.10	0.10	0.20	0.15	1.49	6.00	168.000

Gain/Loss factors: shaft and toe

0.40000	0.45000	0.50000	0.55000	0.60000
1.00000	1.00000	1.00000	1.00000	1.00000

Dpth	L	Wait	Strk	Pmx%	Eff.	Stff	CoR
5.00	0.00	0.00	0.000	0.0	0.000	0.000	0.000
10.00	0.00	0.00	0.000	0.0	0.000	0.000	0.000
15.00	0.00	0.00	0.000	0.0	0.000	0.000	0.000
20.00	0.00	0.00	0.000	0.0	0.000	0.000	0.000
25.00	0.00	0.00	0.000	0.0	0.000	0.000	0.000
30.00	0.00	0.00	0.000	0.0	0.000	0.000	0.000
35.00	0.00	0.00	0.000	0.0	0.000	0.000	0.000
40.00	0.00	0.00	0.000	0.0	0.000	0.000	0.000
45.00	0.00	0.00	0.000	0.0	0.000	0.000	0.000
50.00	0.00	0.00	0.000	0.0	0.000	0.000	0.000
55.00	0.00	0.00	0.000	0.0	0.000	0.000	0.000
60.00	0.00	0.00	0.000	0.0	0.000	0.000	0.000
61.70	0.00	0.00	0.000	0.0	0.000	0.000	0.000
0.00	0.00	0.00	0.000	0.0	0.000	0.000	0.000

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

FRA-70-1373L - FA - B-020-7-13 - HP12x53

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	140046.6	1.000	0.0000	
3	0.800	140046.6	1.000	0.0000	
4	0.800	140046.6	1.000	0.0000	
5	0.800	140046.6	1.000	0.0000	
Imp Block	0.753	70735.6	0.900	0.0100	
Helmet	1.900	60155.0	0.800	0.0100	5.8
Combined Pile Top		11535.0			

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)	1600.00	Actual Pressure	(psi)	1600.00
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

Depth	(ft)	5.0	Standard Soil Setup	
Shaft Gain/Loss Factor		0.400	Toe Gain/Loss Factor	1.000

PILE PROFILE:

Toe Area	(in2)	144.000	Pile Type	Unknown
Pile Size	(inch)	12.000		

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	15.50	29000.	492.0	4.0	0	16524.	27.2
61.7	15.50	29000.	492.0	4.0	0	16524.	27.2

Wave Travel Time 2L/c (ms) 7.468

Pile and Soil Model Total Capacity Rut (kips) 4.9

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	0.172	11535	0.010	0.000	0.85	0.0	0.000	0.100	3.25	4.0	15.5
2	0.172	11535	0.000	0.000	1.00	0.0	0.000	0.100	6.49	4.0	15.5
18	0.172	11535	0.000	0.000	1.00	0.1	0.050	0.100	58.45	4.0	15.5
19	0.172	11535	0.000	0.000	1.00	3.3	0.191	0.100	61.70	4.0	15.5
Toe						1.5	0.150	0.100			

3.268 kips total unreduced pile weight (g= 32.17 ft/s2)
3.268 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)	0.544
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) 5.9 4.9
Hammer+Pile Weight > Rult: Pile Runs

INITIAL STATIC ANALYSIS: Total Wt, Sum(R) 5.9 5.3
Hammer+Pile Weight > Rult: Pile Runs

INITIAL STATIC ANALYSIS: Total Wt, Sum(R) 5.9 5.6
Hammer+Pile Weight > Rult: Pile Runs

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
4.9	0.0	10.81	0.00	0.00	1	0	0.00	1	0	0.0	78.4
5.3	0.0	11.86	0.00	0.00	1	0	0.00	1	0	0.0	74.4
5.6	0.0	11.86	0.00	0.00	1	0	0.00	1	0	0.0	74.4

```
    6.0  Hammer did not run
    6.4  Hammer did not run
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FRA-70-1373L - FA - B-020-7-13 - HP12x53                             03/01/2021
Resource International Inc                                     GRLWEAP Version 2010

 Depth                 (ft)      10.0  Standard Soil Setup
 Shaft Gain/Loss Factor        0.400  Toe Gain/Loss Factor          1.000

 PILE PROFILE:
 Toe Area            (in2)   144.000  Pile Type                   Unknown
 Pile Size          (inch)    12.000

  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    15.50   29000.    492.0      4.0        0    16524.     27.2
    61.7    15.50   29000.    492.0      4.0        0    16524.     27.2

 Wave Travel Time 2L/c (ms)     7.468

       Pile and Soil Model            Total Capacity Rut  (kips)      14.6
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  0.172    11535 0.010 0.000 0.85     0.0   0.000 0.100   3.25   4.0    15.5
  2  0.172    11535 0.000 0.000 1.00     0.0   0.000 0.100   6.49   4.0    15.5
 16  0.172    11535 0.000 0.000 1.00     0.0   0.050 0.100  51.96   4.0    15.5
 17  0.172    11535 0.000 0.000 1.00     0.5   0.053 0.100  55.21   4.0    15.5
 18  0.172    11535 0.000 0.000 1.00     6.4   0.200 0.100  58.45   4.0    15.5
 19  0.172    11535 0.000 0.000 1.00     6.6   0.200 0.100  61.70   4.0    15.5
Toe                                      1.1   0.150 0.100

     3.268 kips total unreduced pile weight (g= 32.17 ft/s2)
     3.268 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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FRA-70-1373L - FA - B-020-7-13 - HP12x53                             03/01/2021
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
   14.6     1.5   4.00   3.98   -0.65   9  12    10.98   1   2   25.4    59.3
   16.2     1.6   4.05   4.08   -0.47   5  12    11.74   1   2   25.4    58.8
   17.7     1.7   4.16   4.16   -0.41   4  12    12.73   1   2   25.5    58.1
   19.3     1.8   4.26   4.23   -0.38   3  12    13.49   1   2   25.3    57.5
   20.9     2.0   4.30   4.32   -0.36   3  12    13.94   1   2   24.9    57.1
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FRA-70-1373L - FA - B-020-7-13 - HP12x53                             03/01/2021
Resource International Inc                                     GRLWEAP Version 2010

 Depth                 (ft)      15.0  Standard Soil Setup
 Shaft Gain/Loss Factor        0.400  Toe Gain/Loss Factor          1.000

 PILE PROFILE:
 Toe Area            (in2)   144.000  Pile Type                   Unknown
 Pile Size          (inch)    12.000

  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    15.50   29000.    492.0      4.0        0    16524.     27.2
    61.7    15.50   29000.    492.0      4.0        0    16524.     27.2

 Wave Travel Time 2L/c (ms)     7.468

       Pile and Soil Model            Total Capacity Rut  (kips)      25.8
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  0.172    11535 0.010 0.000 0.85     0.0   0.000 0.100   3.25   4.0    15.5
  2  0.172    11535 0.000 0.000 1.00     0.0   0.000 0.100   6.49   4.0    15.5
 15  0.172    11535 0.000 0.000 1.00     0.2   0.050 0.100  48.71   4.0    15.5
 16  0.172    11535 0.000 0.000 1.00     3.8   0.193 0.100  51.96   4.0    15.5
 17  0.172    11535 0.000 0.000 1.00     6.4   0.200 0.100  55.21   4.0    15.5
 18  0.172    11535 0.000 0.000 1.00     7.5   0.200 0.100  58.45   4.0    15.5
 19  0.172    11535 0.000 0.000 1.00     7.3   0.200 0.100  61.70   4.0    15.5
Toe                                      0.7   0.150 0.100

     3.268 kips total unreduced pile weight (g= 32.17 ft/s2)
     3.268 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-70-1373L - FA - B-020-7-13 - HP12x53 03/01/2021
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
25.8	2.5	4.57	4.55	-0.43	2	10	15.91	1	2	24.1	55.5
28.0	2.8	4.67	4.65	-0.47	2	10	16.61	1	2	23.7	54.9
30.2	3.0	4.76	4.74	-0.48	2	9	17.27	1	2	23.4	54.3
32.5	3.3	4.86	4.83	-0.46	2	9	17.79	4	3	23.1	53.8
34.7	3.5	4.94	4.92	-0.39	2	9	18.29	6	3	22.8	53.3

FRA-70-1373L - FA - B-020-7-13 - HP12x53 03/01/2021
Resource International Inc GRLWEAP Version 2010

Depth (ft) 20.0 Standard Soil Setup
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 12.000

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	15.50	29000.	492.0	4.0	0	16524.	27.2
61.7	15.50	29000.	492.0	4.0	0	16524.	27.2

Wave Travel Time 2L/c (ms) 7.468

Pile and Soil Model Total Capacity Rut (kips) 59.4

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	0.172	11535	0.010	0.000	0.85	0.0	0.000	0.100	3.25	4.0	15.5
2	0.172	11535	0.000	0.000	1.00	0.0	0.000	0.100	6.49	4.0	15.5
13	0.172	11535	0.000	0.000	1.00	0.0	0.050	0.100	42.22	4.0	15.5
14	0.172	11535	0.000	0.000	1.00	1.0	0.152	0.100	45.46	4.0	15.5
15	0.172	11535	0.000	0.000	1.00	6.4	0.200	0.100	48.71	4.0	15.5
16	0.172	11535	0.000	0.000	1.00	6.8	0.200	0.100	51.96	4.0	15.5
17	0.172	11535	0.000	0.000	1.00	7.9	0.200	0.100	55.21	4.0	15.5
18	0.172	11535	0.000	0.000	1.00	5.0	0.200	0.100	58.45	4.0	15.5
19	0.172	11535	0.000	0.000	1.00	9.4	0.061	0.100	61.70	4.0	15.5
Toe						23.0	0.150	0.100			

3.268 kips total unreduced pile weight (g= 32.17 ft/s2)
3.268 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-70-1373L - FA - B-020-7-13 - HP12x53 03/01/2021
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
59.4	6.0	5.52	5.50	0.00	1	0	22.18	15	5	21.0	50.3
61.9	6.3	5.58	5.57	0.00	1	0	22.55	15	5	20.9	50.0
64.4	6.6	5.65	5.64	0.00	1	0	22.91	15	5	20.7	49.7
66.9	7.0	5.65	5.71	-0.16	15	50	23.08	15	5	20.4	49.5
69.5	7.3	5.73	5.77	-0.35	15	50	23.47	15	5	20.3	49.2

FRA-70-1373L - FA - B-020-7-13 - HP12x53 03/01/2021
Resource International Inc GRLWEAP Version 2010

Depth (ft) 25.0 Standard Soil Setup
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 12.000

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	15.50	29000.	492.0	4.0	0	16524.	27.2
61.7	15.50	29000.	492.0	4.0	0	16524.	27.2

Wave Travel Time 2L/c (ms) 7.468

Pile and Soil Model — Total Capacity Rut (kips) 71.2

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	0.172	11535	0.010	0.000	0.85	0.0	0.000	0.100	3.25	4.0	15.5
2	0.172	11535	0.000	0.000	1.00	0.0	0.000	0.100	6.49	4.0	15.5
12	0.172	11535	0.000	0.000	1.00	0.2	0.050	0.100	38.97	4.0	15.5
13	0.172	11535	0.000	0.000	1.00	4.3	0.195	0.100	42.22	4.0	15.5
14	0.172	11535	0.000	0.000	1.00	6.4	0.200	0.100	45.46	4.0	15.5
15	0.172	11535	0.000	0.000	1.00	7.6	0.200	0.100	48.71	4.0	15.5
16	0.172	11535	0.000	0.000	1.00	6.9	0.200	0.100	51.96	4.0	15.5
17	0.172	11535	0.000	0.000	1.00	6.3	0.130	0.100	55.21	4.0	15.5
18	0.172	11535	0.000	0.000	1.00	11.0	0.050	0.100	58.45	4.0	15.5
19	0.172	11535	0.000	0.000	1.00	12.6	0.050	0.100	61.70	4.0	15.5
Toe						15.9	0.150	0.100			

3.268 kips total unreduced pile weight (g= 32.17 ft/s2)
3.268 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-70-1373L - FA - B-020-7-13 - HP12x53 03/01/2021
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
71.2	6.7	5.67	5.65	-0.37	9	46	22.87	13	4	20.6	49.6
73.7	7.0	5.73	5.72	-0.72	9	46	23.19	13	4	20.5	49.3
76.3	7.4	5.75	5.80	-1.02	9	46	23.37	13	4	20.2	49.1
78.8	7.8	5.82	5.87	-1.28	9	46	23.73	13	4	20.1	48.8
81.3	8.2	5.89	5.92	-1.45	9	46	24.09	13	4	20.0	48.5

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Depth (ft) 30.0 Standard Soil Setup
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 12.000

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	15.50	29000.	492.0	4.0	0	16524.	27.2
61.7	15.50	29000.	492.0	4.0	0	16524.	27.2

Wave Travel Time 2L/c (ms) 7.468

Pile and Soil Model — Total Capacity Rut (kips) 97.1

No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Perim	Area
	kips	k/in	ft	ft		kips	s/ft	inch	ft	ft	in2
1	0.172	11535	0.010	0.000	0.85	0.0	0.000	0.100	3.25	4.0	15.5
2	0.172	11535	0.000	0.000	1.00	0.0	0.000	0.100	6.49	4.0	15.5
10	0.172	11535	0.000	0.000	1.00	0.0	0.050	0.100	32.47	4.0	15.5
11	0.172	11535	0.000	0.000	1.00	1.5	0.172	0.100	35.72	4.0	15.5
12	0.172	11535	0.000	0.000	1.00	6.4	0.200	0.100	38.97	4.0	15.5
13	0.172	11535	0.000	0.000	1.00	6.9	0.200	0.100	42.22	4.0	15.5
14	0.172	11535	0.000	0.000	1.00	7.9	0.200	0.100	45.46	4.0	15.5
15	0.172	11535	0.000	0.000	1.00	4.7	0.200	0.100	48.71	4.0	15.5
16	0.172	11535	0.000	0.000	1.00	10.0	0.050	0.100	51.96	4.0	15.5
17	0.172	11535	0.000	0.000	1.00	12.2	0.050	0.100	55.21	4.0	15.5
18	0.172	11535	0.000	0.000	1.00	13.0	0.050	0.100	58.45	4.0	15.5
19	0.172	11535	0.000	0.000	1.00	15.0	0.050	0.100	61.70	4.0	15.5
Toe						19.5	0.150	0.100			

3.268 kips total unreduced pile weight (g= 32.17 ft/s2)
3.268 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 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
97.1	9.2	6.04	6.07	-1.32	8	45	24.37	12	4	19.7	47.9
99.6	9.5	6.09	6.12	-1.19	8	45	24.65	12	4	19.6	47.7
102.1	9.9	6.14	6.18	-1.21	12	40	24.93	12	4	19.5	47.5
104.6	10.2	6.19	6.22	-1.40	12	40	25.22	12	4	19.4	47.3
107.1	10.6	6.25	6.28	-1.55	12	40	25.48	12	4	19.3	47.1

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Depth (ft) 35.0 Standard Soil Setup
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 12.000

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	15.50	29000.	492.0	4.0	0	16524.	27.2
61.7	15.50	29000.	492.0	4.0	0	16524.	27.2

Wave Travel Time 2L/c (ms) 7.468

Pile and Soil Model Total Capacity Rut (kips) 151.5

No.	Weight kips	Stiffn k/in	C-Slk ft	T-Slk ft	CoR	Soil-S kips	Soil-D s/ft	Quake inch	LbTop ft	Perim ft	Area in2
1	0.172	11535	0.010	0.000	0.85	0.0	0.000	0.100	3.25	4.0	15.5
2	0.172	11535	0.000	0.000	1.00	0.0	0.000	0.100	6.49	4.0	15.5
9	0.172	11535	0.000	0.000	1.00	0.2	0.050	0.100	29.23	4.0	15.5
10	0.172	11535	0.000	0.000	1.00	4.7	0.196	0.100	32.47	4.0	15.5
11	0.172	11535	0.000	0.000	1.00	6.4	0.200	0.100	35.72	4.0	15.5
12	0.172	11535	0.000	0.000	1.00	7.7	0.200	0.100	38.97	4.0	15.5
13	0.172	11535	0.000	0.000	1.00	6.6	0.200	0.100	42.22	4.0	15.5
14	0.172	11535	0.000	0.000	1.00	6.8	0.118	0.100	45.46	4.0	15.5
15	0.172	11535	0.000	0.000	1.00	11.2	0.050	0.100	48.71	4.0	15.5
16	0.172	11535	0.000	0.000	1.00	12.7	0.050	0.100	51.96	4.0	15.5
17	0.172	11535	0.000	0.000	1.00	14.1	0.050	0.100	55.21	4.0	15.5
18	0.172	11535	0.000	0.000	1.00	16.7	0.050	0.100	58.45	4.0	15.5
19	0.172	11535	0.000	0.000	1.00	20.0	0.050	0.100	61.70	4.0	15.5
Toe						44.4	0.150	0.100			

3.268 kips total unreduced pile weight (g= 32.17 ft/s2)
3.268 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
151.5	16.3	6.84	6.85	-1.29	10	33	27.00	10	4	18.7	45.1
154.0	16.7	6.97	6.90	-1.25	10	33	27.49	10	4	18.8	44.8
156.5	17.2	7.01	6.95	-1.20	10	33	27.72	10	4	18.8	44.6
159.0	17.8	7.06	7.01	-1.20	10	33	28.00	10	4	18.8	44.5
161.5	18.3	7.09	7.05	-1.17	10	33	28.18	10	4	18.8	44.4

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Depth (ft) 40.0 Standard Soil Setup
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 12.000

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	15.50	29000.	492.0	4.0	0	16524.	27.2
61.7	15.50	29000.	492.0	4.0	0	16524.	27.2

Wave Travel Time 2L/c (ms) 7.468

Pile and Soil Model Total Capacity Rut (kips) 150.0

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	0.172	11535	0.010	0.000	0.85	0.0	0.000	0.100	3.25	4.0	15.5
2	0.172	11535	0.000	0.000	1.00	0.0	0.000	0.100	6.49	4.0	15.5
7	0.172	11535	0.000	0.000	1.00	0.0	0.050	0.100	22.73	4.0	15.5
8	0.172	11535	0.000	0.000	1.00	2.0	0.181	0.100	25.98	4.0	15.5
9	0.172	11535	0.000	0.000	1.00	6.4	0.200	0.100	29.23	4.0	15.5
10	0.172	11535	0.000	0.000	1.00	7.0	0.200	0.100	32.47	4.0	15.5
11	0.172	11535	0.000	0.000	1.00	7.9	0.200	0.100	35.72	4.0	15.5
12	0.172	11535	0.000	0.000	1.00	4.7	0.189	0.100	38.97	4.0	15.5
13	0.172	11535	0.000	0.000	1.00	10.1	0.050	0.100	42.22	4.0	15.5
14	0.172	11535	0.000	0.000	1.00	12.3	0.050	0.100	45.46	4.0	15.5
15	0.172	11535	0.000	0.000	1.00	13.2	0.050	0.100	48.71	4.0	15.5
16	0.172	11535	0.000	0.000	1.00	15.1	0.050	0.100	51.96	4.0	15.5
17	0.172	11535	0.000	0.000	1.00	18.8	0.050	0.100	55.21	4.0	15.5
18	0.172	11535	0.000	0.000	1.00	21.2	0.050	0.100	58.45	4.0	15.5
19	0.172	11535	0.000	0.000	1.00	27.2	0.150	0.100	61.70	4.0	15.5
Toe						4.0	0.150	0.100			

3.268 kips total unreduced pile weight (g= 32.17 ft/s2)
3.268 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
150.0	15.2	6.78	6.80	-0.77	6	34	26.75	9	3	18.6	45.2
153.3	16.0	6.85	6.87	-0.73	8	31	27.08	9	3	18.6	45.0
156.6	16.7	6.92	6.93	-0.93	8	30	27.37	9	3	18.6	44.8
160.0	17.5	6.99	6.99	-1.09	8	30	27.69	9	3	18.6	44.6
163.3	18.1	7.12	7.05	-1.17	8	30	28.16	9	3	18.7	44.3

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Depth (ft) 45.0 Standard Soil Setup
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 12.000

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	15.50	29000.	492.0	4.0	0	16524.	27.2
61.7	15.50	29000.	492.0	4.0	0	16524.	27.2

Wave Travel Time 2L/c (ms) 7.468

Pile and Soil Model Total Capacity Rut (kips) 264.9

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	0.172	11535	0.010	0.000	0.85	0.0	0.000	0.100	3.25	4.0	15.5
2	0.172	11535	0.000	0.000	1.00	0.0	0.000	0.100	6.49	4.0	15.5
6	0.172	11535	0.000	0.000	1.00	0.3	0.050	0.100	19.48	4.0	15.5
7	0.172	11535	0.000	0.000	1.00	5.2	0.197	0.100	22.73	4.0	15.5
8	0.172	11535	0.000	0.000	1.00	6.4	0.200	0.100	25.98	4.0	15.5
9	0.172	11535	0.000	0.000	1.00	7.8	0.200	0.100	29.23	4.0	15.5
10	0.172	11535	0.000	0.000	1.00	6.3	0.200	0.100	32.47	4.0	15.5
11	0.172	11535	0.000	0.000	1.00	7.3	0.105	0.100	35.72	4.0	15.5
12	0.172	11535	0.000	0.000	1.00	11.4	0.050	0.100	38.97	4.0	15.5
13	0.172	11535	0.000	0.000	1.00	12.8	0.050	0.100	42.22	4.0	15.5
14	0.172	11535	0.000	0.000	1.00	14.2	0.050	0.100	45.46	4.0	15.5
15	0.172	11535	0.000	0.000	1.00	17.0	0.050	0.100	48.71	4.0	15.5
16	0.172	11535	0.000	0.000	1.00	20.1	0.050	0.100	51.96	4.0	15.5
17	0.172	11535	0.000	0.000	1.00	22.5	0.050	0.100	55.21	4.0	15.5
18	0.172	11535	0.000	0.000	1.00	32.1	0.200	0.100	58.45	4.0	15.5
19	0.172	11535	0.000	0.000	1.00	29.9	0.154	0.100	61.70	4.0	15.5
Toe						71.6	0.150	0.100			

3.268 kips total unreduced pile weight (g= 32.17 ft/s2)
3.268 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
264.9	37.6	8.07	8.07	-1.35	7	45	30.37	7	3	18.9	41.6
270.1	39.5	8.04	8.12	-1.35	7	44	30.45	7	3	18.8	41.5
275.3	41.4	8.10	8.17	-1.38	7	44	30.69	7	3	18.8	41.4
280.5	43.1	8.16	8.22	-1.43	7	21	30.98	7	3	19.0	41.3
285.7	45.1	8.21	8.27	-1.49	7	21	31.22	7	3	19.0	41.1

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Depth (ft) 50.0 Standard Soil Setup
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 12.000

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	15.50	29000.	492.0	4.0	0	16524.	27.2
61.7	15.50	29000.	492.0	4.0	0	16524.	27.2

Wave Travel Time 2L/c (ms) 7.468

Pile and Soil Model Total Capacity Rut (kips) 311.7

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	0.172	11535	0.010	0.000	0.85	0.0	0.000	0.100	3.25	4.0	15.5
2	0.172	11535	0.000	0.000	1.00	0.0	0.000	0.100	6.49	4.0	15.5
4	0.172	11535	0.000	0.000	1.00	0.1	0.050	0.100	12.99	4.0	15.5
5	0.172	11535	0.000	0.000	1.00	2.5	0.186	0.100	16.24	4.0	15.5
6	0.172	11535	0.000	0.000	1.00	6.4	0.200	0.100	19.48	4.0	15.5
7	0.172	11535	0.000	0.000	1.00	7.1	0.200	0.100	22.73	4.0	15.5
8	0.172	11535	0.000	0.000	1.00	7.9	0.200	0.100	25.98	4.0	15.5
9	0.172	11535	0.000	0.000	1.00	4.8	0.177	0.100	29.23	4.0	15.5
10	0.172	11535	0.000	0.000	1.00	10.3	0.050	0.100	32.47	4.0	15.5
11	0.172	11535	0.000	0.000	1.00	12.4	0.050	0.100	35.72	4.0	15.5
12	0.172	11535	0.000	0.000	1.00	13.3	0.050	0.100	38.97	4.0	15.5
13	0.172	11535	0.000	0.000	1.00	15.3	0.050	0.100	42.22	4.0	15.5
14	0.172	11535	0.000	0.000	1.00	19.1	0.050	0.100	45.46	4.0	15.5
15	0.172	11535	0.000	0.000	1.00	21.4	0.050	0.100	48.71	4.0	15.5
16	0.172	11535	0.000	0.000	1.00	28.0	0.160	0.100	51.96	4.0	15.5
17	0.172	11535	0.000	0.000	1.00	32.1	0.200	0.100	55.21	4.0	15.5
18	0.172	11535	0.000	0.000	1.00	28.0	0.050	0.100	58.45	4.0	15.5
19	0.172	11535	0.000	0.000	1.00	30.2	0.050	0.100	61.70	4.0	15.5
Toe						72.8	0.150	0.100			

3.268 kips total unreduced pile weight (g= 32.17 ft/s2)
3.268 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)		Ten Str	i	t	Comp Str	i	t	ENTHRU	Bl Rt
kips	b/ft	down	up	ksi			ksi			kip-ft	b/min
311.7	49.9	8.31	8.37	-1.34	6	41	30.82	6	3	19.1	40.9
316.9	52.6	8.37	8.42	-1.38	6	41	31.05	6	3	19.1	40.8
322.1	54.9	8.42	8.45	-1.42	6	40	31.29	6	3	19.2	40.7
327.3	57.9	8.47	8.50	-1.45	5	40	31.51	6	3	19.2	40.6
332.5	61.0	8.52	8.54	-1.47	5	40	31.73	6	3	19.3	40.4

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Depth (ft) 55.0 Standard Soil Setup
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

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

Pile Size (inch) 12.000

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	15.50	29000.	492.0	4.0	0	16524.	27.2
61.7	15.50	29000.	492.0	4.0	0	16524.	27.2

Wave Travel Time 2L/c (ms) 7.468

Pile and Soil Model Total Capacity Rut (kips) 337.8

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	0.172	11535	0.010	0.000	0.85	0.0	0.000	0.100	3.25	4.0	15.5
2	0.172	11535	0.000	0.000	1.00	0.0	0.000	0.100	6.49	4.0	15.5
3	0.172	11535	0.000	0.000	1.00	0.4	0.050	0.100	9.74	4.0	15.5
4	0.172	11535	0.000	0.000	1.00	5.6	0.198	0.100	12.99	4.0	15.5
5	0.172	11535	0.000	0.000	1.00	6.4	0.200	0.100	16.24	4.0	15.5
6	0.172	11535	0.000	0.000	1.00	7.9	0.200	0.100	19.48	4.0	15.5
7	0.172	11535	0.000	0.000	1.00	5.9	0.200	0.100	22.73	4.0	15.5
8	0.172	11535	0.000	0.000	1.00	7.8	0.093	0.100	25.98	4.0	15.5
9	0.172	11535	0.000	0.000	1.00	11.6	0.050	0.100	29.23	4.0	15.5
10	0.172	11535	0.000	0.000	1.00	12.8	0.050	0.100	32.47	4.0	15.5
11	0.172	11535	0.000	0.000	1.00	14.4	0.050	0.100	35.72	4.0	15.5
12	0.172	11535	0.000	0.000	1.00	17.3	0.050	0.100	38.97	4.0	15.5
13	0.172	11535	0.000	0.000	1.00	20.3	0.050	0.100	42.22	4.0	15.5
14	0.172	11535	0.000	0.000	1.00	23.3	0.076	0.100	45.46	4.0	15.5
15	0.172	11535	0.000	0.000	1.00	32.1	0.200	0.100	48.71	4.0	15.5
16	0.172	11535	0.000	0.000	1.00	29.6	0.143	0.100	51.96	4.0	15.5
17	0.172	11535	0.000	0.000	1.00	29.2	0.050	0.100	55.21	4.0	15.5
18	0.172	11535	0.000	0.000	1.00	43.1	0.151	0.100	58.45	4.0	15.5
19	0.172	11535	0.000	0.000	1.00	62.3	0.200	0.100	61.70	4.0	15.5
Toe						7.8	0.150	0.100			

3.268 kips total unreduced pile weight (g= 32.17 ft/s2)
3.268 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
337.8	66.8	8.64	8.65	-1.07	4	38	31.61	4	2	19.2	40.2
347.7	73.9	8.73	8.71	-1.11	4	38	31.97	4	2	19.4	40.0
357.6	82.3	8.81	8.79	-1.15	4	37	32.30	4	2	19.4	39.9
367.6	90.9	8.88	8.84	-1.18	4	37	32.61	4	2	19.5	39.7
377.5	101.5	8.94	8.88	-1.20	4	37	32.88	4	2	19.6	39.6

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Depth (ft) 60.0 Standard Soil Setup
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 12.000

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	15.50	29000.	492.0	4.0	0	16524.	27.2
61.7	15.50	29000.	492.0	4.0	0	16524.	27.2

Wave Travel Time 2L/c (ms) 7.468

Pile and Soil Model Total Capacity Rut (kips) 433.6

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	0.172	11535	0.010	0.000	0.85	0.1	0.050	0.100	3.25	4.0	15.5
2	0.172	11535	0.000	0.000	1.00	2.9	0.189	0.100	6.49	4.0	15.5
3	0.172	11535	0.000	0.000	1.00	6.4	0.200	0.100	9.74	4.0	15.5
4	0.172	11535	0.000	0.000	1.00	7.2	0.200	0.100	12.99	4.0	15.5
5	0.172	11535	0.000	0.000	1.00	7.9	0.200	0.100	16.24	4.0	15.5
6	0.172	11535	0.000	0.000	1.00	5.0	0.164	0.100	19.48	4.0	15.5
7	0.172	11535	0.000	0.000	1.00	10.5	0.050	0.100	22.73	4.0	15.5
8	0.172	11535	0.000	0.000	1.00	12.5	0.050	0.100	25.98	4.0	15.5
9	0.172	11535	0.000	0.000	1.00	13.5	0.050	0.100	29.23	4.0	15.5

10	0.172	11535	0.000	0.000	1.00	15.6	0.050	0.100	32.47	4.0	15.5
11	0.172	11535	0.000	0.000	1.00	19.3	0.050	0.100	35.72	4.0	15.5
12	0.172	11535	0.000	0.000	1.00	21.6	0.050	0.100	38.97	4.0	15.5
13	0.172	11535	0.000	0.000	1.00	28.7	0.168	0.100	42.22	4.0	15.5
14	0.172	11535	0.000	0.000	1.00	31.7	0.193	0.100	45.46	4.0	15.5
15	0.172	11535	0.000	0.000	1.00	28.1	0.050	0.100	48.71	4.0	15.5
16	0.172	11535	0.000	0.000	1.00	30.4	0.050	0.100	51.96	4.0	15.5
17	0.172	11535	0.000	0.000	1.00	60.0	0.196	0.100	55.21	4.0	15.5
18	0.172	11535	0.000	0.000	1.00	62.3	0.200	0.100	58.45	4.0	15.5
19	0.172	11535	0.000	0.000	1.00	62.2	0.200	0.100	61.70	4.0	15.5
Toe						7.8	0.150	0.100			

3.268 kips total unreduced pile weight (g= 32.17 ft/s2)
3.268 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
433.6	202.5	9.26	9.19	-0.78	3	35	32.97	2	2	20.2	39.0
448.8	263.1	9.30	9.23	-0.75	3	35	33.21	2	2	20.2	38.9
464.0	356.8	9.35	9.27	-0.70	3	34	33.46	2	2	20.2	38.8
479.1	526.0	9.38	9.30	-0.66	3	34	33.68	2	2	20.3	38.7
494.3	967.7	9.41	9.32	-0.62	3	34	33.91	2	2	20.2	38.7

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Resource International Inc GRLWEAP Version 2010

Depth (ft) 61.7 Standard Soil Setup
Shaft Gain/Loss Factor 0.400 Toe Gain/Loss Factor 1.000

PILE PROFILE:
Toe Area (in2) 144.000 Pile Type Unknown
Pile Size (inch) 12.000

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	15.50	29000.	492.0	4.0	0	16524.	27.2
61.7	15.50	29000.	492.0	4.0	0	16524.	27.2

Wave Travel Time 2L/c (ms) 7.468

Pile and Soil Model Total Capacity Rut (kips) 465.6

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	0.172	11535	0.010	0.000	0.85	0.4	0.050	0.100	3.25	4.0	15.5
2	0.172	11535	0.000	0.000	1.00	6.0	0.199	0.100	6.49	4.0	15.5
3	0.172	11535	0.000	0.000	1.00	6.5	0.200	0.100	9.74	4.0	15.5
4	0.172	11535	0.000	0.000	1.00	7.9	0.200	0.100	12.99	4.0	15.5
5	0.172	11535	0.000	0.000	1.00	5.6	0.200	0.100	16.24	4.0	15.5
6	0.172	11535	0.000	0.000	1.00	8.3	0.084	0.100	19.48	4.0	15.5
7	0.172	11535	0.000	0.000	1.00	11.7	0.050	0.100	22.73	4.0	15.5
8	0.172	11535	0.000	0.000	1.00	12.9	0.050	0.100	25.98	4.0	15.5
9	0.172	11535	0.000	0.000	1.00	14.5	0.050	0.100	29.23	4.0	15.5
10	0.172	11535	0.000	0.000	1.00	17.5	0.050	0.100	32.47	4.0	15.5
11	0.172	11535	0.000	0.000	1.00	20.5	0.050	0.100	35.72	4.0	15.5
12	0.172	11535	0.000	0.000	1.00	24.0	0.092	0.100	38.97	4.0	15.5
13	0.172	11535	0.000	0.000	1.00	32.1	0.200	0.100	42.22	4.0	15.5
14	0.172	11535	0.000	0.000	1.00	29.3	0.134	0.100	45.46	4.0	15.5
15	0.172	11535	0.000	0.000	1.00	29.3	0.050	0.100	48.71	4.0	15.5
16	0.172	11535	0.000	0.000	1.00	45.1	0.159	0.100	51.96	4.0	15.5
17	0.172	11535	0.000	0.000	1.00	62.3	0.200	0.100	55.21	4.0	15.5
19	0.172	11535	0.000	0.000	1.00	61.6	0.200	0.100	61.70	4.0	15.5
Toe						7.8	0.150	0.100			

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

Depth	Stroke	Pressure	Efficy
ft	ft	Ratio	
61.70	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
465.6	371.7	9.39	9.31	-0.52	3	34	33.76	2	2	20.3	38.7
482.5	586.8	9.42	9.34	-0.50	2	34	34.04	2	2	20.3	38.6
499.5	1274.3	9.44	9.36	-0.51	2	33	34.29	2	2	20.3	38.6
516.4	9999.0	9.47	9.39	-0.51	2	33	34.56	2	2	20.3	38.6
533.3	9999.0	9.39	9.38	-0.50	2	33	34.58	2	2	20.0	38.6

SUMMARY OVER DEPTHS

G/L at Shaft and Toe: 0.400 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	4.9	3.4	1.5	0.0	0.000	0.000	10.81	0.0
10.0	14.6	13.5	1.1	1.5	10.982	-0.647	4.00	25.4
15.0	25.8	25.1	0.7	2.5	15.906	-0.427	4.57	24.1
20.0	59.4	36.5	23.0	6.0	22.176	0.000	5.52	21.0
25.0	71.2	55.3	15.9	6.7	22.867	-0.373	5.67	20.6
30.0	97.1	77.6	19.5	9.2	24.365	-1.324	6.04	19.7
35.0	151.5	107.1	44.4	16.3	26.995	-1.288	6.84	18.7
40.0	150.0	146.0	4.0	15.2	26.748	-0.769	6.78	18.6
45.0	264.9	193.2	71.6	37.6	30.373	-1.351	8.07	18.9
50.0	311.7	238.8	72.8	49.9	30.822	-1.338	8.31	19.1
55.0	337.8	330.0	7.8	66.8	31.615	-1.065	8.64	19.2
60.0	433.6	425.9	7.8	202.5	32.971	-0.779	9.26	20.2
61.7	465.6	457.9	7.8	371.7	33.763	-0.518	9.39	20.3

Total Driving Time 51 minutes; Total No. of Blows 2054
Starting at penetration 5.0 ft

G/L at Shaft and Toe: 0.450 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	5.3	3.8	1.5	0.0	0.000	0.000	11.86	0.0
10.0	16.2	15.1	1.1	1.6	11.736	-0.470	4.05	25.4
15.0	28.0	27.3	0.7	2.8	16.609	-0.474	4.67	23.7
20.0	61.9	39.0	23.0	6.3	22.552	0.000	5.58	20.9
25.0	73.7	57.8	15.9	7.0	23.192	-0.716	5.73	20.5
30.0	99.6	80.1	19.5	9.5	24.646	-1.194	6.09	19.6
35.0	154.0	109.6	44.4	16.7	27.493	-1.248	6.97	18.8
40.0	153.3	149.3	4.0	16.0	27.078	-0.727	6.85	18.6
45.0	270.1	198.4	71.6	39.5	30.454	-1.347	8.04	18.8
50.0	316.9	244.1	72.8	52.6	31.047	-1.375	8.37	19.1
55.0	347.7	340.0	7.8	73.9	31.973	-1.111	8.73	19.4
60.0	448.8	441.1	7.8	263.1	33.215	-0.745	9.30	20.2
61.7	482.5	474.8	7.8	586.8	34.040	-0.504	9.42	20.3

Total Driving Time 62 minutes; Total No. of Blows 2509
Starting at penetration 5.0 ft

SUMMARY OVER DEPTHS

G/L at Shaft and Toe: 0.500 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	5.6	4.2	1.5	0.0	0.000	0.000	11.86	0.0
10.0	17.7	16.7	1.1	1.7	12.731	-0.412	4.16	25.5
15.0	30.2	29.5	0.7	3.0	17.268	-0.482	4.76	23.4
20.0	64.4	41.5	23.0	6.6	22.915	0.000	5.65	20.7
25.0	76.3	60.3	15.9	7.4	23.375	-1.017	5.75	20.2
30.0	102.1	82.6	19.5	9.9	24.929	-1.211	6.14	19.5
35.0	156.5	112.1	44.4	17.2	27.716	-1.203	7.01	18.8
40.0	156.6	152.6	4.0	16.7	27.374	-0.933	6.92	18.6
45.0	275.3	203.7	71.6	41.4	30.687	-1.379	8.10	18.8
50.0	322.1	249.3	72.8	54.9	31.290	-1.419	8.42	19.2
55.0	357.6	349.9	7.8	82.3	32.297	-1.146	8.81	19.4
60.0	464.0	456.2	7.8	356.8	33.459	-0.697	9.35	20.2
61.7	499.5	491.7	7.8	1274.3	34.291	-0.507	9.44	20.3

Total Driving Time 88 minutes; Total No. of Blows 3484
Starting at penetration 5.0 ft

G/L at Shaft and Toe: 0.550 1.000

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

ft	kips	kips	kips	bl/ft	ksi	ksi	ft	kip-ft
5.0	6.0	4.6	1.5	Hammer did not run				
10.0	19.3	18.2	1.1	1.8	13.485	-0.381	4.26	25.3
15.0	32.5	31.7	0.7	3.3	17.794	-0.461	4.86	23.1
20.0	66.9	44.0	23.0	7.0	23.081	-0.157	5.65	20.4
25.0	78.8	62.9	15.9	7.8	23.731	-1.282	5.82	20.1
30.0	104.6	85.1	19.5	10.2	25.218	-1.396	6.19	19.4
35.0	159.0	114.6	44.4	17.8	27.996	-1.195	7.06	18.8
40.0	160.0	156.0	4.0	17.5	27.690	-1.092	6.99	18.6
45.0	280.5	208.9	71.6	43.1	30.981	-1.432	8.16	19.0
50.0	327.3	254.5	72.8	57.9	31.509	-1.454	8.47	19.2
55.0	367.6	359.8	7.8	90.9	32.612	-1.176	8.88	19.5
60.0	479.1	471.4	7.8	526.0	33.683	-0.659	9.38	20.3
61.7	516.4	508.6	7.8	9999.0	34.564	-0.505	9.47	20.3

Refusal occurred; no driving time output possible

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SUMMARY OVER DEPTHS

G/L at Shaft and Toe: 0.600 1.000

Depth	Rut	Frictn	End Bg	Bl Ct	Com Str	Ten Str	Stroke	ENTHRU
ft	kips	kips	kips	bl/ft	ksi	ksi	ft	kip-ft
5.0	6.4	5.0	1.5	Hammer did not run				
10.0	20.9	19.8	1.1	2.0	13.936	-0.361	4.30	24.9
15.0	34.7	34.0	0.7	3.5	18.290	-0.391	4.94	22.8
20.0	69.5	46.5	23.0	7.3	23.468	-0.346	5.73	20.3
25.0	81.3	65.4	15.9	8.2	24.093	-1.446	5.89	20.0
30.0	107.1	87.6	19.5	10.6	25.483	-1.547	6.25	19.3
35.0	161.5	117.1	44.4	18.3	28.178	-1.175	7.09	18.8
40.0	163.3	159.3	4.0	18.1	28.156	-1.175	7.12	18.7
45.0	285.7	214.1	71.6	45.1	31.220	-1.488	8.21	19.0
50.0	332.5	259.7	72.8	61.0	31.729	-1.474	8.52	19.3
55.0	377.5	369.8	7.8	101.5	32.885	-1.202	8.94	19.6
60.0	494.3	486.6	7.8	967.7	33.907	-0.624	9.41	20.2
61.7	533.3	525.5	7.8	9999.0	34.575	-0.499	9.39	20.0

Refusal occurred; no driving time output possible

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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	61.70	0.00	10.81	1.00	0.80	1.00	1.00
10.00	61.70	0.00	10.81	1.00	0.80	1.00	1.00
15.00	61.70	0.00	10.81	1.00	0.80	1.00	1.00
20.00	61.70	0.00	10.81	1.00	0.80	1.00	1.00
25.00	61.70	0.00	10.81	1.00	0.80	1.00	1.00
30.00	61.70	0.00	10.81	1.00	0.80	1.00	1.00
35.00	61.70	0.00	10.81	1.00	0.80	1.00	1.00
40.00	61.70	0.00	10.81	1.00	0.80	1.00	1.00
45.00	61.70	0.00	10.81	1.00	0.80	1.00	1.00
50.00	61.70	0.00	10.81	1.00	0.80	1.00	1.00
55.00	61.70	0.00	10.81	1.00	0.80	1.00	1.00
60.00	61.70	0.00	10.81	1.00	0.80	1.00	1.00
61.70	61.70	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	0.050	0.150	0.340	6.000	1.000
3.49	0.09	0.80	0.100	0.100	0.050	0.150	0.340	6.000	1.000
3.51	1.24	1.45	0.100	0.100	0.200	0.150	1.000	6.000	168.000
9.49	1.24	1.45	0.100	0.100	0.200	0.150	1.000	6.000	168.000
9.51	1.02	1.09	0.100	0.100	0.200	0.150	0.660	6.000	168.000
14.49	1.02	1.09	0.100	0.100	0.200	0.150	0.660	6.000	168.000
14.51	0.71	0.73	0.100	0.100	0.200	0.150	1.000	6.000	168.000
16.99	0.71	0.73	0.100	0.100	0.200	0.150	1.000	6.000	168.000
17.01	0.68	18.55	0.100	0.100	0.050	0.150	0.000	6.000	1.000
23.49	1.04	28.11	0.100	0.100	0.050	0.150	0.000	6.000	1.000
23.51	0.94	14.83	0.100	0.100	0.050	0.150	0.000	6.000	1.000
30.49	1.26	19.88	0.100	0.100	0.050	0.150	0.000	6.000	1.000

30.51	1.39	37.76	0.100	0.100	0.050	0.150	0.000	6.000	1.000
38.49	1.83	49.54	0.100	0.100	0.050	0.150	0.000	6.000	1.000
38.51	4.12	4.00	0.100	0.100	0.200	0.150	0.660	6.000	168.000
43.49	4.12	4.00	0.100	0.100	0.200	0.150	0.660	6.000	168.000
43.51	2.08	71.28	0.100	0.100	0.050	0.150	0.000	6.000	1.000
50.49	2.46	72.94	0.100	0.100	0.050	0.150	0.000	6.000	1.000
50.51	8.00	7.75	0.100	0.100	0.200	0.150	0.660	6.000	168.000
59.51	8.00	7.75	0.100	0.100	0.200	0.150	0.660	6.000	168.000
61.70	7.75	7.75	0.100	0.100	0.200	0.150	0.660	6.000	168.000

49.09	1.90	7.75	0.100	0.100	0.200	0.150	1.000	6.000	168.000
49.11	1.90	7.75	0.100	0.100	0.200	0.150	1.000	6.000	168.000
53.50	1.90	7.75	0.100	0.100	0.200	0.150	1.000	6.000	168.000

APENDIX VI

LATERAL DESIGN PARAMETERS

FRA-70-1373L I-70 WB over Short Street
Lateral Design Parameters

Boring No.	Elevation (feet msl)	Soil Class.	Soil Type	Strata	N_{60}	$N1_{60}$	γ (pcf)	γ' (pcf)	Strength Parameter	k (soil) k_{rm} (rock)	ε_{50} (soil) E_r (rock)	RQD (rock)
B-020-5-13	733.4 to 716.4	A-6a	C	3	24	24	120	120	Su = 3,000 psf	1,000 pci	0.0050	-
	716.4 to 707.9	A-6a	C	3	33	33	125	125	Su = 4,125 psf	1,375 pci	0.0046	-
	707.9 to 700.4	A-6b	C	3	30	30	125	125	Su = 3,750 psf	1,250 pci	0.0048	-
	700.4 to 695.4	A-2-4	G	4	21	16	125	125	φ = 35°	135 pci	-	-
	695.4 to 676.4	A-1-b	G	4	65	42	135	72.6	φ = 41°	175 pci	-	-
	676.4 to 669.2	A-4a	C	2	120	120	130	67.6	Su = 8,000 psf	2,665 pci	0.0033	-
	669.2 to 661.4	A-6a	C	2	106	106	130	67.6	Su = 8,000 psf	2,665 pci	0.0033	-
	661.4 to 656.8	A-6b	C	2	65	65	130	67.6	Su = 8,000 psf	2,665 pci	0.0033	-
	656.8 to 653.4	Shale	R	9	-	-	150	87.6	Qu = 200 psi	0.0005	20,000 psi	20
	653.4 to 645.4	Mudstone	R	9	-	-	150	87.6	Qu = 360 psi	0.0005	32,000 psi	73
	645.4 to 643.4	Shale	R	9	-	-	150	87.6	Qu = 1,125 psi	0.00015	100,000 psi	46
B-020-7-13	713.5 to 706.5	A-2-4	G	4	8	12	120	120	φ = 34°	115 pci	-	-
	706.5 to 700.5	A-7-6	C	3	12	12	115	115	Su = 1,500 psf	500 pci	0.0070	-
	700.5 to 695.5	A-6b	C	3	9	9	115	115	Su = 1,125 psf	300 pci	0.0085	-
	695.5 to 693.0	A-7-6	C	1	6	6	115	115	Su = 750 psf	100 pci	0.0100	-
	693.0 to 686.5	A-1-a	G	4	48	43	135	72.6	φ = 42°	195 pci	-	-
	686.5 to 679.5	A-1-b	G	4	33	28	130	67.6	φ = 39°	140 pci	-	-
	679.5 to 671.5	A-1-b	G	4	67	53	135	72.6	φ = 42°	195 pci	-	-
	671.5 to 666.5	A-6a	C	2	33	33	125	62.6	Su = 4,125 psf	1,375 pci	0.0046	-
	666.5 to 659.5	A-1-a	G	4	89	64	135	72.6	φ = 43°	215 pci	-	-
	659.5 to 648.1	A-6a	C	2	120	120	130	67.6	Su = 8,000 psf	2,665 pci	0.0033	-
	648.1 to 633.1	Mudstone	R	9	-	-	150	87.6	Qu = 200 psi	0.0005	20,000 psi	74

APPENDIX VII

MSE WALL CALCULATIONS

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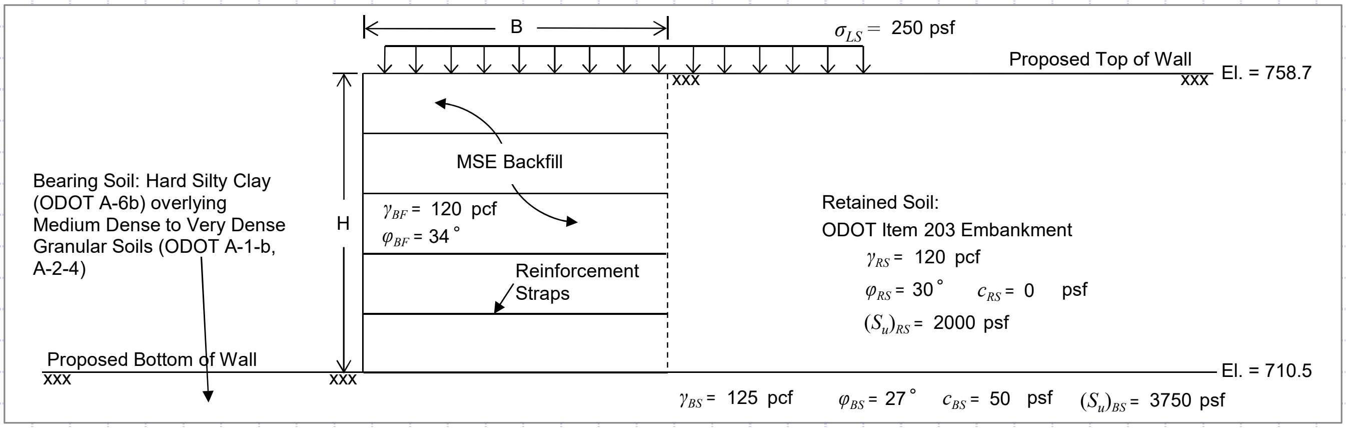
FRA-70-1373L - Retaining Wall E9 - MSE Wall - Rear Abutment - B-020-5-13 - 48.2 ft. Wall Height



MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	48.2	ft
MSE Wall Width (Reinforcement Length), (B) =	43.4	ft
MSE Wall Length, (L) =	65	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}) =	27	°
Bearing Soil Drained Cohesion, (c_{BS}) =	50	psf
Bearing Soil Undrained Shear Strength, $[(S_u)_{BS}]$ =	3750	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

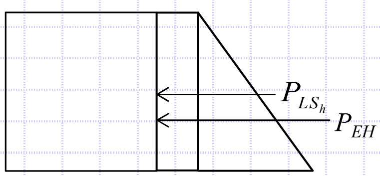
	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

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

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

Sliding Force:

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



$$P_{EH} = \tfrac{1}{2}\gamma_{RS} H^2 K_a \gamma_{EH} = \text{½(120 pcf)(48.2 ft)²(0.297)(1.5)} = 62.1 \text{ kip/ft}$$

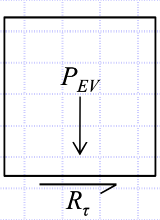
$$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS} = \text{(250 psf)(48.2 ft)(0.297)(1.75)} = 6.26 \text{ kip/ft}$$

$$P_H = \text{62.1 kip/ft + 6.26 kip/ft} = 68.36 \text{ kip/ft}$$

Check Sliding Resistance - Drained Condition

Nominal Sliding Resistance:

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



$$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV} = \text{(120 pcf)(48.2 ft)(43.4 ft)(1.00)} = 251.03 \text{ kip/ft}$$

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

$$\tan\delta = \tan(27) \le \tan(34) \longrightarrow 0.51 \le 0.67 \longrightarrow \tan\delta = 0.51$$

$$R_\tau = \text{(251.03 kip/ft)(0.51)} = 128.03 \text{ kip/ft}$$

Verify Sliding Force Less Than Factored Sliding Resistance - Drained Condition

$$P_H \le R_\tau \cdot \phi_\tau \longrightarrow \text{68.36 kip/ft} \le \text{(128.03 kip/ft)(1.0)} = \text{128.03 kip/ft} \longrightarrow \text{68.36 kip/ft} \le \text{128.03 kip/ft}$$

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) =	48.2	ft
MSE Wall Width (Reinforcement Length), (B) =	43.4	ft
MSE Wall Length, (L) =	65	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}) =	27	°
Bearing Soil Drained Cohesion, (c_{BS}) =	50	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	3750	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

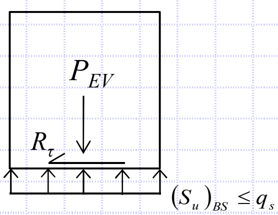
	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

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

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

Check Sliding Resistance - Undrained Condition

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



$(S_u)_{BS} =$ 3.75 ksf

$q_s = \sigma_v / 2$ = (5.78 ksf) / 2 = 2.89 ksf

$\sigma_v = P_{EV} / B$ = (251.03 kip/ft) / (43.4 ft) = 5.78 ksf

R_τ = (3.75 ksf ≤ 2.89 ksf)(43.4 ft) = 125.43 kip/ft

Verify Sliding Force Less Than Factored Sliding Resistance - Undrained Condition

$P_H \le R_\tau \cdot \phi_\tau$ ⟶ 68.36 kip/ft ≤ (125.43 kip/ft)(1.0) = 125.43 kip/ft ⟶ 68.36 kip/ft ≤ 125.43 kip/ft **OK**

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

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JOB	FRA-70-13.10	NO.	W-13-072
SHEET NO.	3	OF	6
CALCULATED BY	BRT	DATE	1/30/2021
CHECKED BY	JPS	DATE	3/5/2021

FRA-70-1373L - Rear Abutment - Wall E9

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	48.2	ft
MSE Wall Width (Reinforcement Length), (B) =	43.4	ft
MSE Wall Length, (L) =	65	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:

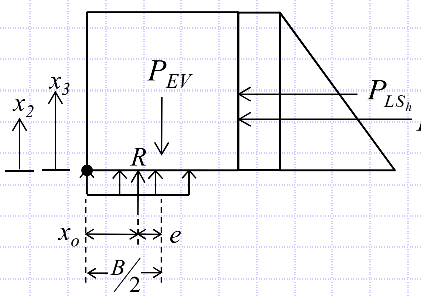
Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	27	°
Bearing Soil Drained Cohesion, (c_{BS}) =	50	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	3750	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

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

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

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



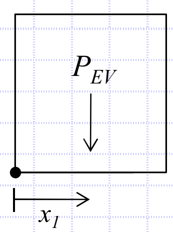
$$e = B/2 - x_o$$

$$x_o = \frac{M_{EV} - M_H}{P_{EV}}$$ = 5447.35 kip·ft/ft - 1148.81 kip·ft/ft) / (251.03 kip/ft = 17.12 ft

M_{EV} = 5447.35 kip·ft/ft
M_H = 1148.81 kip·ft/ft — Defined below
P_{EV} = 251.03 kip/ft

$e =$ (43.4 ft)/2 - 17.12 ft = 4.58 ft

Resisting Moment, M_{EV}:



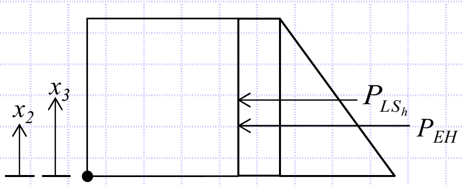
$$M_{EV} = P_{EV}(x_1)$$

$P_{EV} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV}$ = (120 pcf)(48.2 ft)(43.4 ft)(1.00) = 251.03 kip/ft

$x_1 = B/2$ = (43.4 ft) / 2 = 21.70 ft

$M_{EV} =$ (251.03 kip/ft)(21.70 ft) = 5447.35 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)(48.2 ft)²(0.297)(1.5) = 62.10 kip/ft

$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS}$ = (250 psf)(48.2 ft)(0.297)(1.75) = 6.26 kip/ft

$x_2 = H/3$ = (48.2 ft) / 3 = 16.07 ft

$x_3 = H/2$ = (48.2 ft) / 2 = 24.10 ft

$M_H =$ (62.1 kip/ft)(16.07 ft) + (6.26 kip/ft)(24.10 ft) = 1148.81 kip·ft/ft

Check Eccentricity

$e < e_{max} \longrightarrow$ 4.58 ft < 14.47 ft **OK**

Limiting Eccentricity: $e_{max} = B/3 \longrightarrow e_{max} =$ (43.4 ft) / 3 = 14.47 ft

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	48.2	ft
MSE Wall Width (Reinforcement Length), (B) =	43.4	ft
MSE Wall Length, (L) =	65	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:

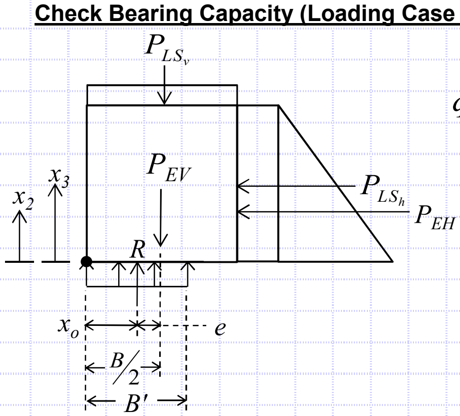
Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	27	°
Bearing Soil Drained Cohesion, (c_{BS}) =	50	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	3750	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

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

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

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



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

$B' = B - 2e$ = 43.4 ft - 2(3.21 ft) = 36.98 ft

$e = B/2 - x_o$ = (43.4 ft) / 2 - 18.49 ft = 3.21 ft

$x_o = \dfrac{M_V - M_H}{P_V}$ = (7765.82 kip·ft/ft - 1148.89 kip·ft/ft) / 357.87 kip/ft = 18.49 ft

$q_{eq} =$ (357.87 kip/ft) / (36.98 ft) = 9.68 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)(48.2 ft)(43.4 ft)(1.35)](21.7 ft) + [(250 psf)(43.4 ft)(1.75)](21.7 ft) = 7765.82 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)(48.2 ft)²(0.297)(1.5)](16.07 ft) + [(250 psf)(48.2 ft)(0.297)(1.75)](24.1 ft) = 1,148.89 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)(48.2 ft)(43.4 ft)(1.35) + (250 psf)(43.4 ft)(1.75) = 357.87 kip/ft

Check Bearing Resistance - Drained Condition

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

$N_{cm} = N_c s_c i_c$ = 31.46

N_c = 23.94
s_c = 1+(36.98 ft/65 ft)(13.2/23.94)
= 1.314
i_c = 1.000 (Assumed)

$N_{qm} = N_q s_q d_q i_q$ = 17.59

N_q = 13.20
s_q = 1.290
d_q = 1+2tan(27°)[1-sin(27°)]²tan⁻¹(4.0 ft/36.98 ft)
1.033
i_q = 1.000 (Assumed)
C_{wq} = 0.0 ft > 4.0 ft = 0.500

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 11.17

N_γ = 14.47
s_γ = 0.772
i_γ = 1.000 (Assumed)
$C_{w\gamma}$ = 0.0 ft < 1.5(36.98 ft) + 4.0 ft = 0.500

$q_n =$ (50 psf)(31.457) + (125 pcf)(4.0 ft)(17.590)(0.500) + ½(125 pcf)(37.0 ft)(11.171)(0.500) = 18.88 ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

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

$q_{eq} \le q_n \cdot \phi_b$ ⟶ 9.68 ksf ≤ (18.88 ksf)(0.65) = 12.27 ksf ⟶ 9.68 ksf ≤ 12.27 ksf **OK**

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	48.2	ft
MSE Wall Width (Reinforcement Length), (B) =	43.4	ft
MSE Wall Length, (L) =	65	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}) =	27	°
Bearing Soil Drained Cohesion, (c_{BS}) =	50	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	3750	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

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

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

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

Check Bearing Resistance - Undrained Condition

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

$N_{cm} = N_c s_c i_c$ = 5.730

N_c = 5.140

s_c = 1+(36.98 ft/[(5)(65 ft)]) = 1.114

i_c = 1.000 (Assumed)

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

N_q = 1.000

s_q = 1.000

d_q = 1+2tan(0°)[1-sin(0°)]²tan⁻¹(4.0 ft/36.98 ft)

1.000

i_q = 1.000 (Assumed)

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

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

N_γ = 0.000

s_γ = 1.000

i_γ = 1.000 (Assumed)

$C_{w\gamma}$ = 0.0 ft < 1.5(36.98 ft) + 4.0 ft = 0.500

q_n = (3750 psf)(5.730) + (125 pcf)(4.0 ft)(1.000)(0.500) + ½(125 pcf)(37.0 ft)(0.000)(0.500) = 21.74 ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

$q_{eq} \leq q_n \cdot \phi_b$ ⟶ 9.68 ksf ≤ (21.74 ksf)(0.65) = 14.13 ksf ⟶ 9.68 ksf ≤ 14.13 ksf **OK**

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

RESOURCE INTERNATIONAL, INC.
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JOB	FRA-70-13.10	NO.	W-13-072
SHEET NO.	6	OF	6
CALCULATED BY	BRT	DATE	1/30/2021
CHECKED BY	JPS	DATE	3/5/2021

FRA-70-1373L - Rear Abutment - Wall E9

MSE Wall Dimensions and Retained Soil Parameters

MSE Wall Height, (H) =	48.2	ft
MSE Wall Width (Reinforcement Length), (B) =	43.4	ft
MSE Wall Length, (L) =	65	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_{RS}) =	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:

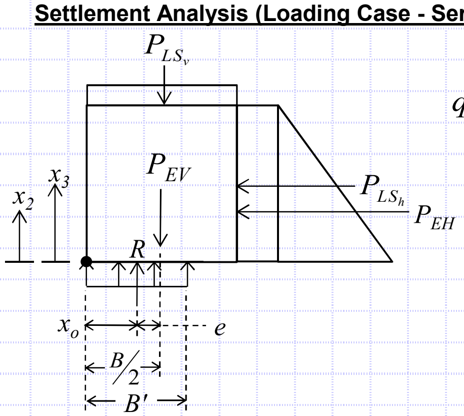
Bearing Soil Unit Weight, (γ_{BS}) =	125	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	27	°
Bearing Soil Drained Cohesion, (c_{BS}) =	50	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	3750	psf
Embedment Depth, (D_f) =	4.0	ft
Depth to Grounwater (Below Bot. of Wall), (D_W) =	0.0	ft

LRFD Load Factors

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

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

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



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

$B' = B - 2e$ = 43.4 ft - 2(2.87 ft) = 37.66 ft

$e = B/2 - x_o$ = (43.4 ft) / 2 - 18.83 ft = 2.87 ft

$x_o = \dfrac{M_V - M_H}{P_V}$ = (5682.70 kip·ft/ft - 751.55 kip·ft/ft) / 261.88 kip/ft = 18.83 ft

q_{eq} = (261.88 kip/ft) / (37.66 ft) = 6.95 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)(48.2 ft)(43.4 ft)(1.00)](21.7 ft) + [(250 psf)(43.4 ft)(1.00)](21.7 ft) = 5682.70 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)(48.2 ft)²(0.297)(1.00)](16.07 ft) + [(250 psf)(48.2 ft)(0.297)(1.00)](24.1 ft) = 751.55 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)(48.2 ft)(43.4 ft)(1.00) + (250 psf)(43.4 ft)(1.00) = 261.88 kip/ft

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

Total Settlement at Center of Reinforced Soil Mass:	S_t =	5.294	in
Total Settlement at Wall Facing:	S_t =	3.353	in

Time Rate of Consolidation and Downdrag Depths and Loads:

Hold Period	Degree of Consolidation	Settlement Remaining at Completion of Hold Period	Depth of Downdrag
72 days	88 %	0.398 in	0.0 ft
90 days	90 %	0.324 in	0.0 ft

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

MSE Wall Settlement - Rear Abutment - Retaining Wall E9

Calculated By: BRT Date: 1/30/2021

Checked By: JPS Date: 3/5/2021

Boring B-020-5-13

H=	48.2	ft	Total wall height
B'=	37.7	ft	Effective footing width due to eccentricity
D_w =	0.0	ft	Depth below bottom of footing
q_e =	6,950	psf	Equivalent bearing pressure at bottom of wall
q_{net} =	4,790	psf	Net bearing pressure considering 18 ft height for the existing embankment at the wall facing

																				Total Settlement at Center of Reinforced Soil Mass					Total Settlement at Facing of Wall				
Layer	Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
1	A-6a	C	0.0	2.6	2.6	1.3	125	325	163	81	4,081	32	0.198	0.020	0.522				0.03	1.000	4,789	4,871	0.083	1.002	0.500	2,395	2,476	0.050	0.602
2	A-6b	C	2.6	5.1	2.5	3.9	125	638	481	241	4,241	33	0.207	0.021	0.530				0.10	0.997	4,774	5,015	0.067	0.801	0.500	2,394	2,635	0.035	0.422
		C	5.1	7.6	2.5	6.4	125	950	794	398	4,398	33	0.207	0.021	0.530				0.17	0.986	4,722	5,119	0.058	0.692	0.499	2,390	2,788	0.029	0.343
		C	7.6	10.1	2.5	8.9	125	1,263	1,106	554	4,554	33	0.207	0.021	0.530				0.23	0.965	4,625	5,179	0.050	0.598	0.497	2,383	2,937	0.024	0.294
3	A-2-4	G	10.1	12.6	2.5	11.4	125	1,575	1,419	711	4,711					21	28	94	0.30	0.936	4,485	5,195	0.023	0.275	0.495	2,370	3,081	0.017	0.203
		G	12.6	15.1	2.5	13.9	125	1,888	1,731	867	4,867					21	27	50	0.37	0.900	4,313	5,180	0.039	0.465	0.491	2,352	3,219	0.028	0.341
4	A-1-b	G	15.1	19.1	4.0	17.1	135	2,428	2,158	1,090	5,090					65	78	317	0.45	0.848	4,061	5,151	0.009	0.102	0.484	2,319	3,410	0.006	0.075
		G	19.1	24.1	5.0	21.6	135	3,103	2,765	1,417	5,417					65	73	118	0.57	0.772	3,699	5,116	0.024	0.283	0.472	2,259	3,677	0.017	0.210
		G	24.1	29.1	5.0	26.6	135	3,778	3,440	1,780	5,780					65	68	111	0.71	0.693	3,319	5,099	0.021	0.247	0.454	2,176	3,957	0.016	0.188
		G	29.1	34.1	5.0	31.6	135	4,453	4,115	2,143	6,143					65	64	105	0.84	0.622	2,982	5,125	0.018	0.217	0.435	2,082	4,225	0.014	0.169
5	A-4a	C	34.1	37.1	3.0	35.6	130	4,843	4,648	2,426	6,426	25	0.135	0.014	0.467				0.94	0.573	2,746	5,172	0.009	0.109	0.418	2,002	4,428	0.007	0.087
		C	37.1	41.3	4.2	39.2	130	5,389	5,116	2,669	6,669	25	0.135	0.014	0.467				1.04	0.534	2,558	5,227	0.011	0.135	0.403	1,930	4,599	0.009	0.110
6	A-6a	C	41.3	49.1	7.8	45.2	130	6,403	5,896	3,075	7,075	25	0.135	0.014	0.467				1.20	0.478	2,288	5,363	0.017	0.208	0.378	1,810	4,885	0.014	0.173
7	A-6b	C	49.1	53.7	4.6	51.4	130	7,001	6,702	3,494	7,494	35	0.225	0.023	0.546				1.36	0.430	2,058	5,552	0.013	0.162	0.353	1,692	5,186	0.011	0.138
																						Total Settlement:		5.294 in			Total Settlement:		3.353 in

1. $\sigma_p' = \sigma_{vo}' + \sigma_m$; Estimate σ_m of 2,000 psf in existing fill material and 4,000 psf (moderately overconsolidated) for natural soil deposits; Ref. Table 11.2, Coduto 2003
2. C_c = 0.009(LL-10); Ref. Table 6-9, FHWA GEC 5
3. C_r = 0.15(Cc) for the existing fill and 0.10(Cc) for the natural soil deposits; Ref. Section 8.11, Holtz and Kovacs 1981
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_N N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \le 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \le \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \le \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (Cohesiv soil layers)
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)

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

MSE Wall Settlement - Rear Abutment - Retaining Wall E9

Calculated By: BRT Date: 01/30/2021

Checked By: JPS Date: 03/05/2021

Boring B-020-5-13

H=	48.2	ft	Total wall height
B'=	37.7	ft	Effective footing width due to eccentricity
D_w =	0.0	ft	Depth below bottom of footing
q_e =	6,950	psf	Equivalent bearing pressure at bottom of wall
q_e =	4,790	psf	Net bearing pressure considering 12.5 ft height for the existing embankment at the wall facing

	A-6a (Upper)	A-6b (Upper)	A-4a	A-6a (Lower)	A-6b (Lower)		
c_v =	600	300	800	600	300	ft^2/yr	Coefficient of consolidation
t =	90	90	90	90	90	days	Time following completion of construction
H_{dr} =	2.5	13.0	7.2	15.0	19.5	ft	Length of longest drainage path considered
T_v =	23.671	0.438	3.805	0.658	0.195		Time factor
U =	100	72	100	84	50	%	Degree of consolidation

$(S_c)_t$ = 2.891 in Settlement complete at 90% of primary consolidation

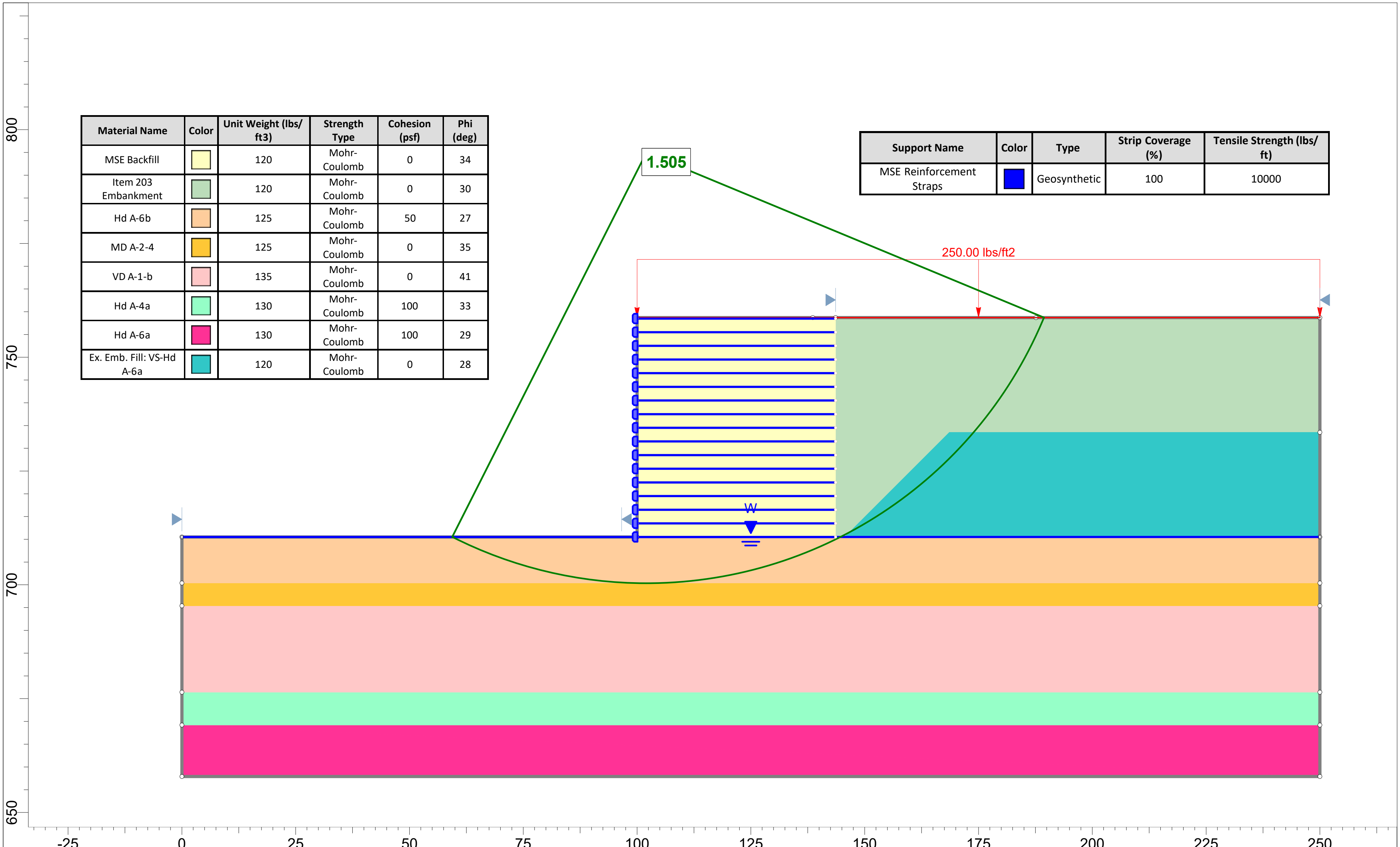
																							Total Settlement at Facing of Wall			Settlement Complete at 90% of Primary Consolidation	
Layer	Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)	Layer Settlement (in)	$(S_c)_t$ (11) (in)	Layer Settlement (in)
1	A-6a	C	0.0	2.6	2.6	1.3	125	325	163	81	4,081	32	0.198	0.020	0.522				0.03	0.500	2,395	2,476	0.050	0.602	0.602	0.602	0.602
2	A-6b	C	2.6	5.1	2.5	3.9	125	638	481	241	4,241	33	0.207	0.021	0.530				0.10	0.500	2,394	2,635	0.035	0.422	1.059	0.304	0.762
		C	5.1	7.6	2.5	6.4	125	950	794	398	4,398	33	0.207	0.021	0.530				0.17	0.499	2,390	2,788	0.029	0.343		0.247	
		C	7.6	10.1	2.5	8.9	125	1,263	1,106	554	4,554	33	0.207	0.021	0.530				0.23	0.497	2,383	2,937	0.024	0.294		0.212	
3	A-2-4	G	10.1	12.6	2.5	11.4	125	1,575	1,419	711	4,711					21	28	94	0.30	0.495	2,370	3,081	0.017	0.203	0.544	0.203	0.544
		G	12.6	15.1	2.5	13.9	125	1,888	1,731	867	4,867					21	27	50	0.37	0.491	2,352	3,219	0.028	0.341		0.341	
4	A-1-b	G	15.1	19.1	4.0	17.1	135	2,428	2,158	1,090	5,090					65	78	317	0.45	0.484	2,319	3,410	0.006	0.075	0.641	0.075	0.641
		G	19.1	24.1	5.0	21.6	135	3,103	2,765	1,417	5,417					65	73	118	0.57	0.472	2,259	3,677	0.017	0.210		0.210	
		G	24.1	29.1	5.0	26.6	135	3,778	3,440	1,780	5,780					65	68	111	0.71	0.454	2,176	3,957	0.016	0.188		0.188	
		G	29.1	34.1	5.0	31.6	135	4,453	4,115	2,143	6,143					65	64	105	0.84	0.435	2,082	4,225	0.014	0.169		0.169	
5	A-4a	C	34.1	37.1	3.0	35.6	130	4,843	4,648	2,426	6,426	25	0.135	0.014	0.467				0.94	0.418	2,002	4,428	0.007	0.087	0.196	0.087	0.196
		C	37.1	41.3	4.2	39.2	130	5,389	5,116	2,669	6,669	25	0.135	0.014	0.467				1.04	0.403	1,930	4,599	0.009	0.110		0.110	
6	A-6a	C	41.3	49.1	7.8	45.2	130	6,403	5,896	3,075	7,075	25	0.135	0.014	0.467				1.20	0.378	1,810	4,885	0.014	0.173	0.173	0.145	0.145
7	A-6b	C	49.1	53.7	4.6	51.4	130	7,001	6,702	3,494	7,494	35	0.225	0.023	0.546				1.36	0.353	1,692	5,186	0.011	0.138	0.138	0.069	0.069

Settlement Remaining After Hold Period: 0.324 in

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

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
MSE Backfill		120	Mohr-Coulomb	0	34
Item 203 Embankment		120	Mohr-Coulomb	0	30
Hd A-6b		125	Mohr-Coulomb	50	27
MD A-2-4		125	Mohr-Coulomb	0	35
VD A-1-b		135	Mohr-Coulomb	0	41
Hd A-4a		130	Mohr-Coulomb	100	33
Hd A-6a		130	Mohr-Coulomb	100	29
Ex. Emb. Fill: VS-Hd A-6a		120	Mohr-Coulomb	0	28

Support Name	Color	Type	Strip Coverage (%)	Tensile Strength (lbs/ft)
MSE Reinforcement Straps		Geosynthetic	100	10000



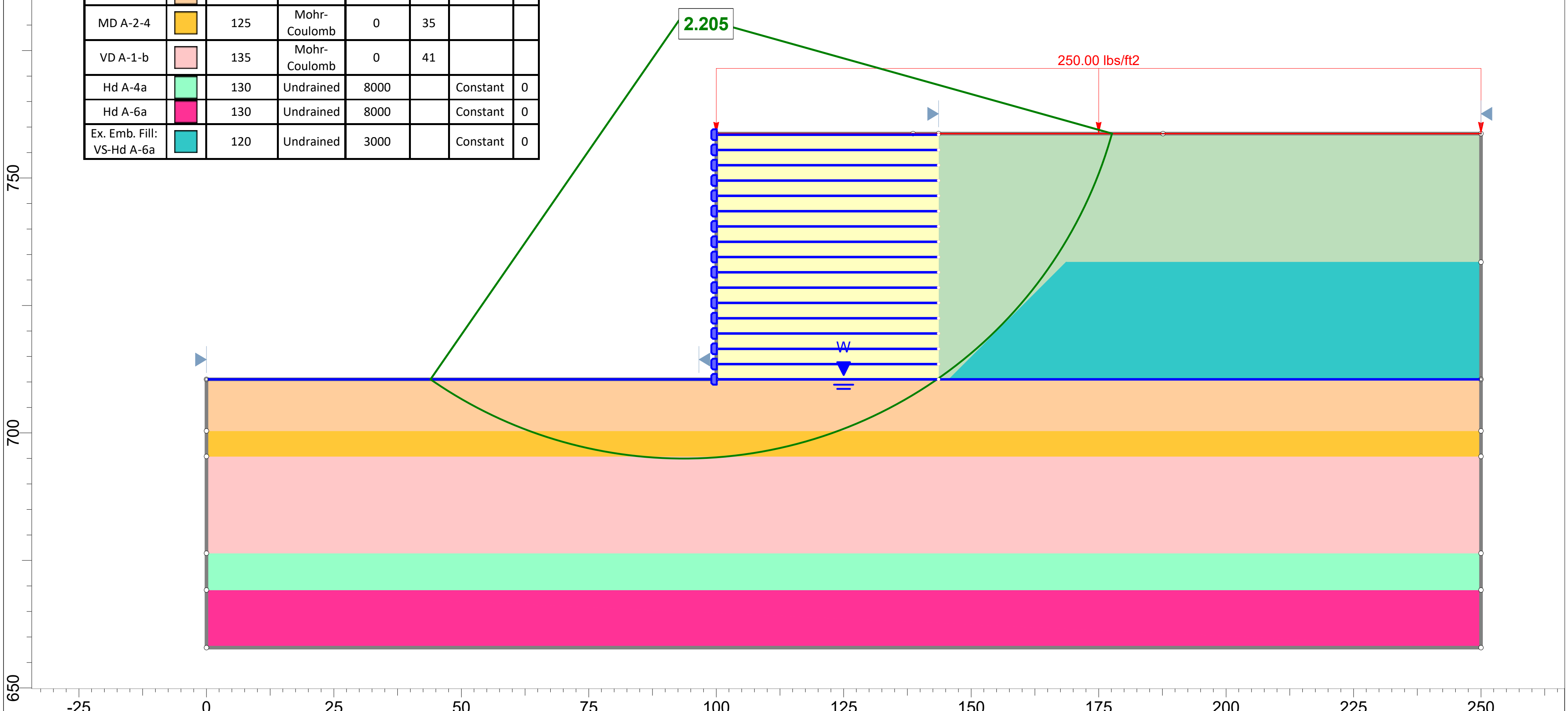
rocscience

SLIDEINTERPRET 9.012

Project	FRA-70-13.10 - FRA-70-1373L - Rear Abutment - Retaining Wall E9 - Global Stability
Analysis Description	48.2 ft Wall Height - Drained - Circular - Spencer's
Drawn By	BRT
Scale	1:350
Company	Resource International, Inc.
Date	1/30/2021
File Name	FRA-70-1373L - Global Stability - Rear Abutment.slim

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Hu
MSE Backfill		120	Mohr-Coulomb	0	34		
Item 203 Embankment		120	Mohr-Coulomb	0	30		
Hd A-6b		125	Undrained	3750		Constant	0
MD A-2-4		125	Mohr-Coulomb	0	35		
VD A-1-b		135	Mohr-Coulomb	0	41		
Hd A-4a		130	Undrained	8000		Constant	0
Hd A-6a		130	Undrained	8000		Constant	0
Ex. Emb. Fill: VS-Hd A-6a		120	Undrained	3000		Constant	0

Support Name	Color	Type	Strip Coverage (%)	Tensile Strength (lbs/ft)
MSE Reinforcement Straps		Geosynthetic	100	10000



rocscience	*Project* FRA-70-13.10 - FRA-70-1373L - Rear Abutment - Retaining Wall E9 - Global Stability		
	Analysis Description 48.2 ft Wall Height - Undrained - Circular - Spencer's		
	Drawn By BRT	*Scale* 1:350	*Company* Resource International, Inc.
SLIDEINTERPRET 9.012	*Date* 1/30/2021		*File Name* FRA-70-1373L - Global Stability - Rear Abutment.slim

APPENDIX VIII

CELLULAR CONCRETE WALL CALCULATIONS

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

MSE Wall with Cellular Concrete Backfill - Forward Abutment - Retaining Wall E7 (Sta. 705+61 and 706+28)

Boring	Boring Elevation	Profile Elevation (ft msl)	Bottom of Wall Elevation (ft msl)	Wall Height (ft)	Pressure at Bottom of Wall [1] (psf)	Total Settlement at Center of Wall (in)	Total Settlement at Wall Facing (in)
B-020-7-13	713.5	758.2	710.0	48.2	1,758	4.05	2.93

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

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

MSE Wall with Cellular Concrete Backfill - Forward Abutment - Retaining Wall E7 (Sta. 705+61 and 706+28)

Calculated By: BRT Date: 1/30/2021

Checked By: JPS Date: 2/5/2021

Boring B-020-7-13

H =	48.2	ft	Total wall height from profile grade to top of leveling pad
B =	33.7	ft	Wall width considered in analysis, equal to 70% of the wall height
D_w =	0.0	ft	Depth below bottom of wall
q =	1,758	psf	Bearing pressure at bottom of wall (see summary sheet)

																				Total Settlement at Center of Reinforced Soil Mass					Total Settlement at Facing of Wall				
Layer	Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' [(1)] (psf)	LL	C_c [(2)]	C_r [(3)]	e_o [(4)]	N_{60}	$(N1)_{60}$ [(5)]	C' [(6)]	Z_f/B	I [(7)]	$\Delta\sigma_v$ [(8)] (psf)	σ_{vf}' Midpoint (psf)	S_c [(9,10)] (ft)	S_c (in)	I [(7)]	$\Delta\sigma_v$ [(8)] (psf)	σ_{vf}' Midpoint (psf)	S_c [(9,10)] (ft)	S_c (in)
1	A-2-4	G	0.0	3.5	3.5	1.8	120	420	210	101	2,101					8	16	66	0.05	1.000	1,757	1,858	0.067	0.801	0.500	879	980	0.052	0.625
2	A-7-6	C	3.5	6.5	3.0	5.0	115	765	593	281	2,281	43	0.297	0.045	0.608				0.15	0.990	1,740	2,021	0.071	0.855	0.499	878	1,158	0.051	0.614
	A-7-6	C	6.5	9.5	3.0	8.0	115	1,110	938	438	2,438	43	0.297	0.045	0.608				0.24	0.964	1,696	2,134	0.057	0.685	0.497	874	1,313	0.040	0.475
3	A-6b	C	9.5	12.0	2.5	10.8	115	1,398	1,254	583	2,583	38	0.252	0.038	0.569				0.32	0.927	1,630	2,213	0.035	0.419	0.494	868	1,451	0.024	0.286
	A-6b	C	12.0	14.5	2.5	13.3	115	1,685	1,541	714	2,714	38	0.252	0.038	0.569				0.39	0.885	1,556	2,270	0.030	0.363	0.489	860	1,574	0.021	0.248
4	A-7-6	C	14.5	17.0	2.5	15.8	115	1,973	1,829	846	2,846	43	0.297	0.045	0.608				0.47	0.839	1,475	2,321	0.030	0.364	0.483	849	1,695	0.021	0.251
5	A-1-a	G	17.0	20.0	3.0	18.5	135	2,378	2,175	1,021	3,021					48	59	210	0.55	0.787	1,384	2,405	0.005	0.064	0.474	834	1,855	0.004	0.045
	A-1-a	G	20.0	23.5	3.5	21.8	135	2,850	2,614	1,257	5,257					48	56	194	0.65	0.728	1,280	2,536	0.006	0.066	0.463	813	2,070	0.004	0.047
6	A-1-b	G	23.5	30.5	7.0	27.0	130	3,760	3,305	1,620	5,620					33	35	115	0.80	0.641	1,127	2,747	0.014	0.167	0.440	774	2,394	0.010	0.124
7	A-1-b	G	30.5	38.5	8.0	34.5	135	4,840	4,300	2,147	6,147					67	66	243	1.02	0.540	950	3,097	0.005	0.063	0.405	713	2,860	0.004	0.049
8	A-6a	C	38.5	43.5	5.0	41.0	125	5,465	5,153	2,594	6,594	27	0.153	0.015	0.483				1.22	0.472	830	3,424	0.006	0.075	0.375	659	3,254	0.005	0.061
9	A-1-a	G	43.5	50.5	7.0	47.0	135	6,410	5,938	3,005	7,005					89	77	309	1.39	0.421	741	3,746	0.002	0.026	0.349	613	3,618	0.002	0.022
10	A-6a	C	50.5	56.0	5.5	53.3	130	7,125	6,768	3,445	7,445	27	0.153	0.015	0.483				1.58	0.378	665	4,110	0.004	0.052	0.323	569	4,013	0.004	0.045
	A-6a	C	56.0	61.9	5.9	59.0	130	7,892	7,509	3,830	7,830	27	0.153	0.015	0.483				1.75	0.345	607	4,437	0.004	0.047	0.302	532	4,362	0.003	0.041
																					Total Settlement:			4.047 in		Total Settlement:			2.933 in

1. $\sigma_p' = \sigma_{vo}'+\sigma_m$; Estimate σ_m of 2,000 psf in existing fill material and 4,000 psf (moderately overconsolidated) for natural soil deposits; Ref. Table 11.2, Coduto 2003
2. $C_c = 0.009(LL-10)$; Ref. Table 6-9, FHWA GEC 5
3. $C_r = 0.15(Cc)$ for the existing fill and $0.10(Cc)$ for the natural soil deposits; Ref. Section 8.11, Holtz and Kovacs 1981
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_nN_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \leq \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (Cohesiv soil layers)
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)

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

MSE Wall with Cellular Concrete Backfill - Forward Abutment - Retaining Wall E7 (Sta. 705+61 and 706+28)

Calculated By: BRT   Date: 1/30/2021

Checked By: JPS   Date: 2/5/2021

Boring B-020-7-13

H =	48.2	ft	Total wall/embankment height from profile grade to top of leveling pad
B =	33.7	ft	Wall/embankment width considered in analysis, equal to 70% of the wall height
D_w =	0.0	ft	Depth below bottom of wall/embankment
q =	1,758	psf	Bearing pressure at bottom of wall/embankment (see summary sheet)

	A-7-6 (Upper)	A-6b	A-7-6 (Lower)	A-6a (Upper)	A-6a (Lower)		
c_v =	150	300	150	600	600	ft^2/yr	Coefficient of consolidation
t =	50	50	50	50	50	days	Time following completion of construction
H_{dr} =	6	7	2.5	2.5	11.9	ft	Length of longest drainage path considered
T_v =	0.571	0.839	3.288	13.151	0.580		Time factor
U =	80	90	100	100	81	%	Degree of consolidation

$(S_c)_t$ = 2.645 in   Settlement complete at 90% of primary consolidation

Layer	Soil Type	Soil Type	Layer Depth (ft)		Layer Thickness (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	Total Settlement at Facing of Wall: S_c (9,10) (ft)	S_c (in)	Layer Settlement (in)	Settlement Complete at 90% of Primary Consolidation: $(S_c)_t$ (11) (in)	Layer Settlement (in)
1	A-2-4	G	0.0	3.5	3.5	1.8	120	420	210	101	2,101					8	16	66	0.05	0.500	879	980	0.052	0.625	0.625	0.625	0.625
2	A-7-6	C	3.5	6.5	3.0	5.0	115	765	593	281	2,281	43	0.297	0.045	0.608				0.15	0.499	878	1,158	0.051	0.614	1.089	0.491	0.871
	A-7-6	C	6.5	9.5	3.0	8.0	115	1,110	938	438	2,438	43	0.297	0.045	0.608				0.24	0.497	874	1,313	0.040	0.475		0.380	
3	A-6b	C	9.5	12.0	2.5	10.8	115	1,398	1,254	583	2,583	38	0.252	0.038	0.569				0.32	0.494	868	1,451	0.024	0.286	0.534	0.258	0.481
	A-6b	C	12.0	14.5	2.5	13.3	115	1,685	1,541	714	2,714	38	0.252	0.038	0.569				0.39	0.489	860	1,574	0.021	0.248		0.223	
4	A-7-6	C	14.5	17.0	2.5	15.8	115	1,973	1,829	846	4,846	43	0.297	0.045	0.608				0.47	0.483	849	1,695	0.021	0.251	0.251	0.251	0.251
5	A-1-a	G	17.0	20.0	3.0	18.5	135	2,378	2,175	1,021	5,021					48	59	210	0.55	0.474	834	1,855	0.004	0.045	0.092	0.045	0.092
	A-1-a	G	20.0	23.5	3.5	21.8	135	2,850	2,614	1,257	5,257					48	56	194	0.65	0.463	813	2,070	0.004	0.047		0.047	
6	A-1-b	G	23.5	30.5	7.0	27.0	130	3,760	3,305	1,620	5,620					33	35	115	0.80	0.440	774	2,394	0.010	0.124	0.124	0.124	0.124
7	A-1-b	G	30.5	38.5	8.0	34.5	135	4,840	4,300	2,147	6,147					67	66	243	1.02	0.405	713	2,860	0.004	0.049	0.049	0.049	0.049
8	A-6a	C	38.5	43.5	5.0	41.0	125	5,465	5,153	2,594	6,594	27	0.153	0.015	0.483				1.22	0.375	659	3,254	0.005	0.061	0.061	0.061	0.061
9	A-1-a	G	43.5	50.5	7.0	47.0	135	6,410	5,938	3,005	7,005					89	77	309	1.39	0.349	613	3,618	0.002	0.022	0.022	0.022	0.022
10	A-6a	C	50.5	56.0	5.5	53.3	130	7,125	6,768	3,445	7,445	27	0.153	0.015	0.483				1.58	0.323	569	4,013	0.004	0.045	0.086	0.037	0.070
	A-6a	C	56.0	61.9	5.9	59.0	130	7,892	7,509	3,830	7,830	27	0.153	0.015	0.483				1.75	0.302	532	4,362	0.003	0.041		0.033	

Settlement Remaining After Hold Period: 0.288 in

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

B =	33.7	ft	
L =	67	ft	
c =	1,125	psf	
γ =	115	pcf	
D_f =	0.0	ft	
φ =	0	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$$ = 6.35 ksf

$$N_{cm} = N_c s_c i_c$$ = 5.64

$$N_{qm} = N_q s_q d_q i_q$$ = 1.00

$$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$$ = 0.00

N_c =	5.14	s_c =	1+(33.7 ft/67 ft)(1/5.14) =	1.098	i_c =	1.000	d_q =	1+2tan(0°)[1-sin(0°)]²tan⁻¹(0 ft/33.7 ft) =	1.000
N_q =	1.00	s_q =	1+(33.7 ft/67 ft)tan(0°) =	1.000	i_q =	1.000	C_{wq} =	0.0 ft > 0.0 ft =	1.000
N_γ =	0.00	s_γ =	1-0.4(33.7 ft/67 ft) =	0.799	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(33.7 ft) + 0 ft =	0.500

$$q_R = q_n \cdot \phi_b$$ = 2.86 ksf

φ_b = 0.45