Resource International, Inc.

FRA-70-14.05 PROJECT 4B
RETAINING WALL 4W16 AND 4W22
PID NO. 96053
FRANKLIN COUNTY, OHIO

STRUCTURE FOUNDATION EXPLORATION REPORT

Prepared For:
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Rii Project No. W-15-126

July 2022

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July 8, 2022

Mr. Christopher W. Luzier, P.E.
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Columbus, OH 43215

Re: Structure Foundation Exploration Report
FRA-70-14.05 Project 4B
Retaining Wall 4W16 and 4W22
PID No. 96053
Rii Project No. W-15-126

Mr. Luzier:

Resource International, Inc. (Rii) is pleased to submit this structure foundation exploration report for the above referenced project. Engineering logs have been prepared and are attached to this report along with the results of laboratory testing. This report includes recommendations for the design and construction of the proposed Retaining Wall 4W16 and 4W22 as part of the FRA-70-14.05 Project 4B 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

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EXECUTIVE SUMMARY

Resource International, Inc. (Rii) has completed a structure foundation exploration for the design and construction of the proposed Retaining Wall 4W16 located along the north side of I-70 westbound between the S. High Street bridge (FRA-70-1405C) and the S. Third Street bridge (FRA-33-1747) over I-70/I-71. The wall height along the alignment of the wall ranges from 31.0 to 39.7 feet, as measured from the bottom of footing (bottom of wall facing panels) to the top of the wall, and the overall length of the wall is approximately 498 feet.

Retaining Wall 4W22 is located within the proposed median between I-70 eastbound and westbound and will connect to the east cap of the S. High Street bridge (FRA-70-1405C) over I-70/I-71. The wall measures approximately 138 lineal feet, with a proposed wall height of 8.8 feet. The retaining wall is proposed to be constructed as a cast-in-place (CIP) wall.

Drilled Shaft Recommendations

While the design of a tangent shaft retaining wall is controlled by lateral check of the shaft elements, the drilled shafts may be designed using the axial design parameters provided in Table 6 in Section 5.1 of the full report. Based on the subsurface conditions encountered, the embedded sections of the shafts will bear in very loose to very dense gravel, gravel and sand, gravel with sand and silt, fine sand, coarse and fine sand and sandy silt (ODOT A-1-a, A-1-b, A-2-4, A-3, A-3a, A-4a) with seams of very stiff to hard sandy silt and silty clay (ODOT A-4a, A-6b). Given that the drilled shafts will be constructed tangent to each other, group efficiency of the foundation for axial resistance will also need to be considered, as outlined in Section 5.1.1 of the full report. Lateral analysis of the shafts should be performed to determine the required embedment depth and cross section of the shafts as outlined in Section 5.1.2 of the full report.

Shallow Foundation Recommendations

It is understood that the shallow spread foundations will be utilized for the retaining wall 4W22. The bearing soils for wall 4W22 is anticipated to consist of hard silty clay and sandy silt (ODOT A-6b, A-4a) overlying dense to very dense gravel (ODOT A-1-a). Shallow spread foundations bearing on these competent natural soils may be proportioned for a nominal bearing resistance as presented in Table 9 in Section 5.2 of the full report.

Based on the maximum service limit bearing pressures provided in the design documents, total settlements ranging from total settlements of up to 0.59 inches are anticipated along the alignment of retaining wall 4W22. Additionally, the maximum factored bearing pressure of 3.31 ksf will not exceed the factored bearing resistance at the strength limit of 15.16 ksf.

Since the bearing soils consist of cohesive material, it is recommended to consider the sliding resisting for both drained and undrained conditions. For drained conditions, the design can consider friction angle of 30 degrees and a coefficient of sliding friction “f” of 0.58 times the total vertical force on the base should be taken as the sliding resistance. 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 wall. An undrained shear strength of 5.25 ksf may be used to model the bearing soil to determine the bearing resistance

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-70-14.05 Project 4B in Columbus, Ohio. The projects represent the central portion of FRA-70-8.93 (PID 77369) I-70/71 south innerbelt improvements project. The FRA-70-14.05 Project 4B phase will consist of all work associated with the construction of the I-70/I-71 corridor from just east of S. High Street to just west of Grant Avenue, as well as a minimal amount of work Fulton Street and at the intersections of S. Third Street and S. Fourth Street with Livingston Avenue. This project includes the replacement of the FRA-33-1747 (S. Third Street) and FRA-23-1075 (S. Fourth Street) bridge structures over I-70/71, as well as the construction of three (3) new retaining walls along the north side and two (2) new retaining walls along the south side of I-70/71 to accommodate the new configuration.

This report is a presentation of the structure foundation exploration performed for the design and construction of the proposed Retaining Wall 4W16 and 4W22. Retaining Wall 4W16 is located along the north side of I-70 westbound between the S. High Street bridge (FRA-70-1405C) and the S. Third Street bridge (FRA-33-1747) over I-70/I-71. Based on design information provided by GPD GROUP, it is understood that the proposed structure will consist of a tangent drilled shaft retaining wall type, which will connect to the forward abutment of the FRA-70-1405C bridge structure at Sta. 193+67.29 (BL I-70 WB) and extend east to connect to the forward abutment of the FRA-33-1747 bridge structure at Sta. 198+66.28 (BL I-70 WB). The wall height along the alignment of the wall ranges from 31.0 to 39.7 feet, as measured from the bottom of footing (bottom of wall facing panels) to the top of the wall, and the overall length of the wall is approximately 498 feet.

Retaining Wall 4W22 is located within the proposed median between I-70 eastbound and westbound and will connect to the east cap of the S. High Street bridge (FRA-70-1405C) over I-70/I-71. Based on the proposed plan information provided by GPD GROUP, Retaining Wall 4W22 begins at Sta. 193+20.21 and ends at Sta. 194+59.92 (BL I-70 EB) and will provide grade separation between the eastbound and westbound lanes of I-70 in the final configuration. Retaining wall 4W22 measures approximately 138 lineal feet, with a proposed wall height of 8.8 feet. The retaining wall is proposed to be constructed as a cast-in-place (CIP) wall.

2.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1 Site Geology

Several episodes of ice advanced throughout Ohio during the Pleistocene Epoch. Both the Illinoian and Wisconsinan glaciers advanced over two-thirds of the state, leaving behind glacial features such as moraines, kame deposits, lacustrine deposits and outwash terraces. The glacial and non-glacial regions comprise five physiographic sections grouped by age, depositional process and geomorphic occurrence (physical features or landforms). The project area lies within the Columbus Lowland District of the Till Plains Section. The project area is characterized by flat to gently rolling ground moraine deposits of the Late Wisconsinan age with large alluvium and outwash deposits bordering the Scioto River, its tributaries and floodplain areas. Ground moraines are deposited during the retreat of a glacier, which results in an undifferentiated mixture of clay, silt, sand and gravel. Alluvium and alluvial terrace deposits range from silty clay to cobble sized deposits, usually deposited in present and former floodplain areas. Outwash deposits consist of undifferentiated sand and gravel deposited by meltwater in front of glacial ice.

Based on bedrock geology and topography maps obtained from Ohio Department of Natural Resources (ODNR), the bedrock beneath the project site consists of three formations. The project alignment extends east from the top of the eastern slope of a bedrock valley that generally follows the Scioto River valley, with the youngest formation at the top of the slope and the oldest formation within the bedrock valley. The youngest formation consists of the Upper Devonian-aged Ohio Shale Formation, which consists of three members, from youngest to oldest: the Cleveland, Chagrin, and Huron Members. These members consist of primarily shale with siltstone and very fine-grained sandstone, varying in color from brownish black to greenish gray. The bedding ranges from laminated to thinly bedded and the overall formation ranges between 250 to over 500 feet thick. The Middle Devonian-aged Delaware Limestone formation, which can be present along the slopes of the bedrock valley, consists of bluish-gray, dolomitic limestone, with thin to medium bedding, and contains nodules and layers of chert. The formation ranges between 0 to 45 feet thick and is not present south of Franklin County. The oldest unit, which present within the bedrock valley, is the Middle to Lower Devonian-aged Columbus Limestone Formation, which is further subdivided into four members, two of which are predominant in the central portion of the state, known as the Delhi and Bellepoint Members. The Delhi Member consists of light gray, finely to coarsely crystalline, irregularly bedded, fossiliferous limestone. The Bellepoint Member consists of variable brown, finely crystalline, massively bedded, limy dolomite. Both of these members contain chert nodules, and the entire formation ranges between 0 to 105 feet thick.

The bedrock surface in the vicinity of the site forms a broad valley which roughly follows the present-day Scioto River valley. The site lies on a slight plateaued area and slope along the east side of the valley where the underlying bedrock surface lies at an approximate elevation of 625 to 630 feet mean sea level and slopes down toward the west to an approximate elevation of 600 feet msl in the bedrock valley. According to bedrock topography mapping, the depth to the bedrock surface below the site ranges between approximately 105 to 135 feet below existing grade. Shale bedrock was encountered in several of the borings performed along the corridor at elevations ranging from 630 to 650 feet msl, increasing in elevation from west to east across the project alignment.

2.2 Existing Conditions

The proposed Retaining Walls 4W16 and 4W22 are located along the north side of I-70/I-71 and within the median of I-70/71 between S. High Street and S. Third Street, approximately 0.8 miles east of the Scioto River. The existing I-70/I-71 in the vicinity of the structure is a six-lane, bi-directional, composite asphalt and concrete paved roadway that is generally east-west aligned through downtown Columbus, Ohio. The existing I-70 profile is lowered from the surrounding terrain, as the existing corridor was cut approximately 20 to 30 below the existing grade of S. High Street and S. Third Street as well as the surrounding downtown area. The proposed wall alignment is situated along the existing entrance ramp from S. Third Street to I-70 westbound. Grass covered graded slopes extend up and down from the existing ramp to W. Fulton Street and the I-70 roadway. This traffic volume along the project alignment is very high, and the alignment traverses primarily commercial and government properties. The surrounding terrain across the site is relatively flat-lying.

3.0 EXPLORATION

Between November 30 and December 2, 2015, two (2) structure borings, designated as B-030-2-15 and B-032-1-15, were advanced to a completion depth of 54.3 and 65.0 feet below the existing ground surface, respectively, along the proposed wall alignment. In addition to the boring performed by Rii as part of the current exploration, one (1) boring, designated as B-030-0-08, was performed by DLZ along the proposed wall alignment as part of the FRA-70-8.93 preliminary exploration (PID 77369), and their findings were published in a report dated September 24, 2009. The boring was performed between July 20 and 23, 2008, and was advanced to a completion depth of 111.0 feet below the existing ground surface. The current and preliminary exploration boring locations are shown on the boring plan provided in Appendix I of this report and summarized in Table 1 below.

Table 1. Test Boring Summary

Boring Number	Reference Alignment	Station	Offset	Latitude	Longitude	Ground Elevation (feet msl)	Boring Depth (feet)
B-030-0-08	BL I-70 WB	194+14.54	24.2' Lt.	39.953275	-82.998195	736.7	111.0
B-030-2-15	BL I-70 WB	195+91.21	39.9' Lt.	39.953401	-82.997585	740.0	54.3
B-032-1-15	BL I-70 WB	197+75.35	50.0' Lt.	39.953517	-82.996945	748.9	65.0

The location for the current exploration borings performed by Rii were determined and located in the field by Rii representatives. Rii utilized a handheld GPS unit to obtain geographic latitude and longitude coordinates of the boring locations. The ground surface elevation at the boring locations were interpolated using topographic mapping information provided by GPD GROUP.

The borings performed by Rii for the current exploration and DLZ for the preliminary exploration were drilled using a truck mounted rotary drilling machine, utilizing a 3.25-inch inside diameter hollow-stem auger to advance the holes.

Standard penetration test (SPT) and split spoon sampling were performed in boring B-030-0-08 continuously below the pavement section to a depth of 12.0 feet, at 2.5-foot intervals to a depth of 30.0 feet and at 5.0-foot intervals thereafter to the top of bedrock. Borings B-030-2-15 and B-032-2-15 were sampled at 2.5-foot intervals to a depth of 20.0 feet and at 5.0-foot intervals thereafter to the boring termination depths. 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. A calibrated automatic drop hammer was utilized by Rii and DLZ to generate consistent energy transfer to the sampler. Driving resistance is recorded on the boring logs in terms of blows per 6.0-inch interval of the driving distance. The second and third intervals are added to obtain the number of blows per foot (N). Standard penetration blow counts aid in determining soil properties applicable in foundation system design. Measured blow count (N) values are corrected to an equivalent (60%) energy ratio, N_{60}, by the following equation. Both values are represented on boring logs in Appendix III.

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

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

The hammer for the CME 55 truck mounted drill rig operated by Rii was calibrated on October 20, 2014, and has a drill rod energy ratio of 92.0 percent. The hammer for the CME 75 drill rig operated by DLZ was calibrated on February 11, 2009, and has a drill rod energy ratio 62.1 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.

Where boring B-030-0-08 was extended into the underlying bedrock by DLZ, an NMX or NQ-sized double-tube diamond bit core barrel (utilizing wire line equipment) was used to core the bedrock. The Rock Quality Designation (RQD) for each rock core run was provided on the boring log.

The borings performed by Rii for the current exploration were backfilled with a mixture of bentonite chips and soil cuttings, and the pavement surface was patched with an equivalent thickness of cold patch asphalt. Abandonment notes for the boring performed by DLZ were not provided on the boring log.

During drilling for the borings performed by Rii, field logs were prepared by Rii personnel showing the encountered subsurface conditions. Soil samples obtained from the drilling operation were preserved and sealed in glass jars and delivered to the soil laboratory. In the laboratory, the soil samples obtained by Rii 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	32
Plastic and Liquid Limits	AASHTO T89, T90	13
Gradation – Sieve/Hydrometer	AASHTO T88	13

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

4.0 FINDINGS

Interpreted engineering logs have been prepared based on the field logs, visual examination of samples and laboratory test results. Classification follows the 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 performed as part of the preliminary engineering phase and current exploration and what is represented on the boring logs.

4.1 Surface Materials

Borings B-030-0-08, B-030-2-15 and B-032-1-15 were performed within the existing pavement along the ramp from S. Third Street to I-70 westbound and encountered 6.0 to 11.0 inches of asphalt overlying 6.0 to 14.0 inches of aggregate base at the ground surface.

4.2 Subsurface Soils

Beneath the pavement in boring B-030-2-15, material identified as existing fill was encountered extending to a depth of 13.5 feet below exiting grade, which corresponds to an elevation of 726.5 feet msl. The fill material was described as medium dense to very dense, dark brown and gray gravel and gravel with sand and silt (ODOT A-1-a, A-2-4). The gravel was observed to consist primarily of concrete fragments.

Underlying the surficial materials and existing fill, natural granular soils were encountered with intermittent seams and layers of cohesive material. The granular soils were described as very loose to very dense, gray, brown and brownish gray gravel, gravel and sand, gravel with sand and silt, fine sand, coarse and fine sand and sandy silt (ODOT A-1-a, A-1-b, A-2-4, A-3, A-3a, A-4a). The seams and layers of cohesive material were described as very stiff to hard, gray, brown and brownish gray sandy silt, silt, silt and clay and silty clay (ODOT A-4a, A-4b, A-6a, A-6b).

The relative density of granular soils is primarily derived from SPT blow counts (N_{60}). Based on the SPT blow counts obtained, the granular soil encountered ranged from very loose (N_{60} < 5 blows per foot [bpf]) to very dense (N_{60} > 50 bpf). Blow counts recorded from the SPT sampling within the granular soils ranged from 3 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 very stiff (2.0 < HP ≤ 4.0 tsf) to hard (HP > 4.0 tsf). The unconfined compressive strength of the cohesive soil samples tested, obtained from the hand penetrometer, ranged from 4.0 to over 4.5 tsf (limit of instrument).

Natural moisture contents of the cohesive soil samples ranged from 9 to 12 percent. The natural moisture content of the cohesive soil samples tested for plasticity index ranged from 6 percent below to at their corresponding plastic limits. The seams of cohesive soil exhibited natural moisture contents considered to be moderately below to at optimum moisture levels. Natural moisture contents of the granular soil samples ranged from 1 to 21 percent, which were visually described as moist to wet.

4.3 Bedrock

Bedrock was encountered in boring B-030-0-08 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-030-0-08	736.7	103.5	633.2	105.0	631.7

Top of bedrock was encountered in boring B-030-0-08 at a depth of 103.5 feet below existing grade, which corresponds to an elevation of 633.2 feet msl. The bedrock consisted of gray to dark gray severely to highly weathered shale. The cored shale bedrock is described as dark gray, highly weathered, weak, laminated, slightly calcareous, pyritic, fissile and highly fractured.

The percent recovery and RQD values from the bedrock core runs in boring B-030-0-08 are summarized in Table 4.

Table 4. Rock Core Summary

Boring	Core No.	Depth (feet)	Recovery (%)	RQD (%)
B-030-0-08	R-1	105.0 to 110.0	95	37
	R-2	110.0 to 111.0	100	42

It should be noted that bedrock can experience mechanical breaks during the drilling and coring processes. It is anticipated that DLZ attempted to account for fresh, manmade breaks during tabulation of the RQD analysis, per ODOT SGE specifications. The quality of the shale bedrock, according to the RQD values, was poor (25 < RQD ≤ 50%).

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-030-0-08	736.7	16.0	720.7	13.6	723.1
B-030-2-15	740.0	17.5	722.5	N/A [1]	-
B-032-1-15	748.9	28.0	720.9	N/A [1]	-

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

Groundwater was encountered initially during the drilling process in borings B-030-0-08, B-030-2-15 and B-032-1-15 at a depths ranging from 16.0 to 28.0 feet below the existing ground surface, which corresponds to elevations ranging from 720.7 to 722.5 feet msl. At the completion of drilling, groundwater was encountered in boring B-030-0-08 at a depth of 13.6 feet below grade, which corresponds to an elevation of 723.1 feet msl. The groundwater level at the completion of drilling in borings B-030-2-15 and B-032-1-15 could not be measured due to the addition of water or mud to counteract heaving sands.

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 current and historic subsurface explorations have been used to determine the foundation support capabilities and the settlement potential for the soil encountered at the site. These parameters have been used to provide guidelines for the design of the 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 retaining walls were provided by GPD GROUP. Based on the information provided, it is understood that Retaining Wall 4W16 will be a tangent drilled shaft wall type. The roadway profile grade along the proposed I-70 westbound will be cut approximately 3.5 to 19.5 feet from the existing ground surface grade to the proposed roadway profile grade, as measured from the top of the wall to the toe of the barrier at the facing of the wall. Fill heights of up to approximately 25.0 feet along the western end of the wall and 6.0 feet at the eastern end of the wall will be required to achieve the proposed grade behind wall.

Retaining Wall 4W22 is located within the proposed median between I-70 eastbound and westbound and will connect to the east cap of the S. High Street bridge (FRA-70-1405C) over I-70/I-71. Based on the proposed plan information provided by GPD GROUP, the retaining wall is proposed to be constructed as cast-in-place (CIP) cantilevered wall type with a proposed height of 8.8 feet. Based on design calculations provided by GPD GROUP, the footing for Retaining Wall 4W22 has been designed to produce a maximum service limit bearing pressure of 2.29 ksf and a maximum factored bearing pressure of 3.31 ksf at the strength limit state. The stability analysis for bearing, eccentricity (overturning), sliding and final CIP wall dimensions and design considerations were performed by GPD GROUP and the calculations are presented in Appendix VIII.

5.1 Drilled Shaft Recommendations (Retaining Wall 4W16)

While the design of a tangent shaft retaining wall is controlled by lateral check of the shaft elements, the drilled shafts may be designed using the axial design parameters provided in Table 6. In the analysis, the top of shaft elevations for the embedded sections of the shafts were considered at the bottom of wall elevation. Based on the subsurface conditions encountered, the embedded sections of the shafts will bear in very loose to very dense gravel, gravel and sand, gravel with sand and silt, fine sand, coarse and fine sand and sandy silt (ODOT A-1-a, A-1-b, A-2-4, A-3, A-3a, A-4a) with seams of very stiff to hard sandy silt and silty clay (ODOT A-4a, A-6b). The drilled shafts should be proportioned for a nominal bearing resistance as presented in Table 6.

Table 6. Retaining Wall 4W16 Drilled Shaft Axial Design Parameters

Boring	Elevation [1] (feet msl)	Shaft Length (feet)	Soil Type	Nominal Unit Resistance (ksf)		Resistance Factor	
				End	Side	End	Side
B-030-0-08	731.0-728.7	0.0-2.3	A-6b	29	2.19	0.40	0.45
	728.7-725.7	2.3-5.3	A-4a	38	2.37	0.40	0.45
	725.7-721.2	5.3-9.8	A-1-a	60	2.48	0.50	0.55
	721.2-713.2	9.8-17.8	A-1-a	58	1.85	0.50	0.55
	713.2-674.7	17.8-56.3	A-3a	60	2.52	0.50	0.55
	674.7-669.7	56.3-61.3	A-3	60	2.59	0.50	0.55
	669.7-633.2	61.3-97.8	A-1-b	60	5.79	0.50	0.55
B-030-2-15	731.0-729.5	0.0-1.5	A-2-4	28	0.34	0.50	0.55
	729.5-724.5	1.5-6.5	A-1-a	20	0.49	0.50	0.55
	724.5-718.0	6.5-13.0	A-1-b	3	0.21	0.50	0.55
	718.0-713.0	13.0-18.0	A-4a	49	2.47	0.40	0.45
	713.0-708.0	18.0-23.0	A-3a	60	1.70	0.50	0.55
	708.0-698.0	23.0-33.0	A-4a	60	3.71	0.50	0.55
	698.0-688.0	33.0-43.0	A-1-b	60	4.71	0.50	0.55
	688.0-686.0	43.0-45.0	A-2-4	60	4.82	0.50	0.55
B-032-1-15	727.0-721.9	0.0-5.1	A-1-b	60	1.42	0.50	0.55
	721.9-716.9	5.1-10.1	A-1-a	39	1.04	0.50	0.55
	716.9-711.9	10.1-15.1	A-1-a	60	3.43	0.50	0.55
	711.9-706.9	15.1-20.1	A-4a	72	3.60	0.40	0.45
	706.9-701.9	20.1-25.1	A-1-b	10	0.65	0.50	0.55
	701.9-683.9	25.1-43.1	A-1-b	60	3.57	0.50	0.55

1. *Top of shaft elevation based on structure information provided by GPD GROUP.*

Drilled shaft lengths should measure a minimum of three (3) times the shaft diameter. Per Section 10.8.3.5.3 of the 2017 AASHTO LRFD Bridge Design Specifications (BDS), where drilled shafts are extended to end bear in a strong soil layer overlying a weaker soil layer, the end bearing resistance shall be reduced if the tip elevation is within 1.5 times the diameter of the drilled shaft above the top of the weaker soil layer. A weighted average that varies linearly from the full end bearing resistance in the overlying strong soil layer at a distance of 1.5 times the diameter of the drilled shaft above the top of the weak soil layer to the end bearing resistance of the weak soil layer at the top of the weak soil layer should be used to determine the end bearing resistance utilized in the design. Therefore, the end bearing resistance utilized in the design will need to be adjusted accordingly if the tip elevation of the drilled shafts will be within 1.5 times the diameter of the drilled shaft above the underlying weaker soil layer.

It is anticipated that 100 percent of the side friction resistance will be mobilized at a displacement of 1.0 percent of the diameter of the shaft, which is approximately 0.6 inches for a 5.0-foot diameter shaft. At this displacement, approximately 30 percent of the end bearing resistance will be mobilized. Therefore, the nominal end bearing resistance noted in Table 6 should be reduced to 30 percent of the values provided for the respective tip elevation in the determination of the design shaft resistance. Drilled shaft calculations are provided in Appendix IV.

5.1.1 Group Efficiency

The axial resistance of a group of shafts may be less than the sum of the individual shaft resistance within a group of shafts. Per Section 10.8.3.6.3 of the 2017 AASHTO LRFD BDS, for soil profiles that consist of primarily granular soils, the individual nominal resistance of each drilled shaft shall be reduced by applying an adjustment factor, η, as defined in Table 10.8.3.6.1-1 of the 2017 AASHTO LRFD BDS. The following criteria are recommended for the group resistance of any shaft groups:

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

Please note that the adjustment factor should be applied to the total individual nominal shaft resistance (including both end bearing side resistance along the shaft length).

Given that the drilled shafts will be constructed tangent to each other, the shaft group capacity should also be checked using the block failure mechanism. Since the soil profile consists primarily of dense granular soils, the analysis should be performed considering the entire drilled shaft group as an equivalent strip footing with a length equal to the length of the tangent shaft wall and equivalent width equal to the total end area of the drilled shafts divided by the length of the drilled shaft wall. A resistance factor of φ_b = 0.45 should be utilized in calculating the factored bearing resistance for this failure mode at the strength limit state.

The total group resistance shall be the lesser of the sum of the individual drilled shafts multiplied by the applicable group efficiency factor, η, or the factored resistance of the group in block failure mode.

5.1.2 Lateral Design

If lateral load or moments are expected to be applied on the foundation elements, they should be analyzed to verify the shaft 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 V. In order to evaluate the lateral capacity, it is recommended that a derivation of COM624, such as LPILE, be utilized to determine the proper embedment depth and cross section required to resist the lateral load for a given end condition and deflection. Table 7 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 V.

Table 7. 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)

For the case of closely spaced drilled shafts, a pile group reduction factor will need to be applied to the p-y curves that are internally generated by the lateral analysis software. Reese, Isenhower, and Wang published an equation for the pile group p-reduction factor, otherwise known as p-multiplier (β_a), for a single row of piles placed side by side in the publication “Analysis and Design of Shallow and Deep Foundations” (2006), as follows:

$$\beta_a = 0.64(S/D)^{0.34}$$

In which:

$$1 \leq S/D < 3.75 \text{ and } 0.5 \leq \beta_a \leq 1.0$$

Where:
S = center to center spacing of the drilled shafts
D = diameter of drilled shafts

It is understood that GPD GROUP has performed an analysis of the lateral loading on the drilled shaft elements, which were utilized to determine the shaft tip elevation provided in the design plans.

5.1.3 Drilled Shaft Axial Resistance

The nominal and factored drilled shaft axial resistance has been calculated for Retaining Wall 4W16, which is summarized in Table 8 below. The tip elevation ranges from 679.9 to 684.8 feet msl based on the plan information provided, so the higher elevation of 684.8 feet msl was utilized in the axial resistance calculations. For the traditional drilled shaft analysis, only end bearing resistance was accounted for in the determination of the nominal and factored axial resistance. A group reduction factor of 0.9 was utilized based on the center to center spacing of the shafts. Based on the tip elevation provided, the drilled shafts will end bear within a layer of very dense coarse and fine sand (ODOT A-3a), which has a calculated nominal end bearing resistance of 60 ksf.

The bearing resistance for the block failure mode was also checked since the drilled shafts will be constructed tangent to each other. Based on the shaft tip elevation provided, the shafts will be bearing in very dense coarse and fine sand (ODOT A-3a). Using the friction angle for the very dense coarse and fine sand (ODOT A-3a), the resulting nominal unit bearing resistance is 278.4 ksf and the factored unit bearing resistance is 139.2 ksf, considering a resistance factor of 0.5.

Table 8. Retaining Wall 4W16 Drilled Shaft Recommendations

Drilled Shaft Analysis Methodology	Shaft Diameter (feet)	Shaft Elevation (feet msl)		Shaft Length (feet)	C-C Shaft Spacing (feet)	Nominal Resistance [1] (kips)			Factored Resistance (kips)		
		Top [2]	Tip			End	Side	Total	End [3]	Side	Total
Traditional	5.0	731.0	684.8	46.2	5.0	1,060	N/A	1,060	530	N/A	530
Traditional	6.0	731.0	684.8	46.2	6.0	1,527	N/A	1,527	763	N/A	763
Block	5.0 / 6.0	731.0	684.8	46.2	5.0 / 6.0	5,467	N/A	5,467	2,734	N/A	2,734

1. *A group reduction factor of 0.9 was utilized based on the center-to-center spacing of the shafts for the traditional analysis methodology.*
2. *Top of shaft elevation corresponds to the bottom of wall elevation.*
3. *A resistance factor of 0.5 was utilized for both the traditional drilled shaft analysis methodology and the block failure mode.*

The controlling resistance between the traditional drilled shaft analysis methodology and block failure mode is 530 and 763 kips/shaft for 60 and 72-inch diameter shafts, respectively. The maximum factored load per shaft is 169 and 211 kips/shaft for 60 and 72-inch diameter shafts, respectively, based on the structural loading information provided by GPD GROUP. Calculations for the drilled shaft axial resistance are provided in Appendix IV.

5.1.4 Drilled Shaft Considerations

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

- A qualified inspector should record the material types being removed from the hole as excavation proceeds.
- When the bearing material has been encountered and identified and/or the design tip elevation has been reached, the shaft walls and base should be observed for anomalies, unexpected soft soil conditions, obstructions or caving.
- Due to the presence of granular soils with relatively high groundwater, it is recommend mud or slurry be utilized in the shaft excavation to counterbalance the hydrostatic head at the bottom of the excavation and minimize the potential for “heave” of the soils up and into the shaft excavation.
- Concrete placed freefall should not be allowed to hit the sidewalls of the excavation or the rebar cage and should not pass through any water.
- Structural stability of the rebar cage should be maintained during the concrete pour to prevent buckling.

- The volume of concrete should be checked to ensure voids did not result during extraction of the casing (if utilized).

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

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

- Pulling casing with insufficient concrete inside should be restricted.

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

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

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

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

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

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

It is anticipated that conventional drilled shaft equipment (with a standard soil bit) will be able to penetrate the surficial soils to the required tip elevation. However, based on conditions encountered in other borings performed within the corridor, cobbles and boulders were encountered throughout the very dense sand and gravel deposits. Therefore, difficult drilling conditions or boulders should be anticipated to be encountered during installation of the drilled shafts. If boulders are encountered during installation of the drilled shafts, specialized drilling/coring equipment may be required to advance the drilled shaft excavation beyond the obstruction.

5.2 Shallow Foundation Recommendations (Retaining Wall 4W22)

The foundation for the proposed retaining wall will bear at a minimum depth of 5.0 feet below the existing grade of I-70, at an elevation 729.6 feet msl. At this elevation, the bearing soil for Retaining Wall 4W22 is anticipated to consist of hard silty clay and sandy silt (ODOT A-6b, A-4a) overlying dense to very dense gravel (ODOT A-1-a). Shallow foundations bearing on these competent natural soils may be proportioned for a nominal bearing resistance as presented in Table 9. It is understood that the external stability calculations (including check for sliding, overturning and bearing) for Retaining Wall 4W22 are being performed by the wall designer, GPD GROUP. Therefore, Rii has provided a graphical plot and tabulated the nominal and factored bearing resistance, as well as the anticipated settlement resulting from the service limit bearing pressure, as a function of the base width for use in final design of the wall system.

Table 9. Shallow Foundation Analysis – Retaining Wall 4W22

Boring Number	Effective Footing Width (feet)	Service Limit Bearing Pressure (ksf) [1]			Nominal Bearing Resistance (ksf)	Factored Bearing Resistance [2] (ksf)
		0.5-inch	1.0-inch	1.5-inch		
B-030-0-08	3	3.21	7.46	13.17	27.40	15.07
	4	2.71	6.65	11.48	27.44	15.09
	5	2.40	6.11	10.35	27.48	15.11
	6	2.20	5.70	9.52	27.51	15.13
	7	2.05	5.39	8.88	27.55	15.15
	8	1.93	5.13	8.37	27.59	15.17
	9	1.84	4.92	7.95	27.63	15.20
	10	1.77	4.74	7.60	27.67	15.22
	11	1.71	4.58	7.30	27.70	15.24
	12	1.66	4.35	7.04	27.74	15.26
	13	1.61	4.16	6.81	27.78	15.28

1. *Service limit bearing pressure was calculated at total settlement values of 0.5, 1.0 and 1.5 inches.*
2. *Resistance factor of φ_b = 0.55 was utilized in calculating the factored nominal bearing resistance at the strength limit state.*

The service limit bearing pressure that results in a maximum total settlement of 0.5, 1.0 and 1.5 inches was calculated and presented in Table 9 for retaining wall 4W22. A geotechnical resistance factor of φ_b = 0.55 has been considered in calculating the factored bearing resistance at the strength limit state. Based on the bearing pressures provided in Table 9, and applying the geotechnical resistance factor provided to the nominal bearing resistance at the strength limit state, the service limit state should control the minimum footing dimensions for all effective footing widths analyzed. A graphical representation of the service limit bearing pressures and factored bearing resistance at the strength limit state is presented in Appendix VI. Calculations for settlement and nominal and factored bearing resistance for the shallow spread foundations are provided in Appendix VII.

Based on the maximum service limit bearing pressures provided in the design documents and noted in Section 5.0, total settlements of up to 0.59 inches are anticipated along the alignment of retaining wall 4W22. Additionally, the maximum factored bearing pressure of 3.31 ksf will not exceed the factored bearing resistance at the strength limit of 15.16 ksf.

5.2.1 Sliding Resistance

The resistance of the footings to sliding will be dependent on the friction between the concrete footing and bearing soils. Since the bearing soils consist of cohesive material, it is recommended to consider the sliding resisting for both drained and undrained conditions. For drained conditions, the design can consider friction angle of 30 degrees and a coefficient of sliding friction “f” of 0.58 times the total vertical force on the base should be taken as the sliding resistance. 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 wall. An undrained shear strength of 5.25 ksf may be used to model the bearing soil to determine the bearing resistance.

A geotechnical resistance factor of φ_τ = 1.0 should be considered when calculating the factored shear resistance between the soil and foundation for sliding.

5.2.2 Overall (Global) Stability

A slope stability analysis was performed to check the global stability of the wall along the alignment. As per AASHTO LRFD BDS, safety against global stability failure shall be evaluated at the service limit state. Soil parameters utilized in external stability analysis are presented in Table 10. For the global stability condition, it was considered that the failure plane will not cross through any portion of the resisting soil mass above the concrete or through the concrete footing itself.

Table 10. Shear Strength Parameters Utilized in Stability Analyses

Material Type	Unit Weight, γ (pcf)	Effective Friction Angle, φ' (°)	Effective Cohesion, c' (psf)	Undrained Shear Strength, S_u (psf)
Item 203 Embankment Fill	120	28	0	2,000
Hard Cohesive Soils (ODOT A-6b, A-4a)	130	25 to 26	0	4,500 to 5,250
Loose to Very Dense Granular Soils (ODOT A-1-a, A-1-b)	120 to 135	36 to 42	0	N/A

Per Section 11.6.2.3 of the AASHTO LRFD BDS, overall (global) stability for CIP wall not supporting structural foundations on spread footings is satisfied if the product of the factor of safety from the slope stability output multiplied by the resistance factor φ=0.75 is greater than 1.0. Therefore, global stability is satisfied when a minimum factor of safety of 1.33 is obtained. Based on the footing dimensions provided in the proposed design documents, the resulting factor of safety under drained conditions (long-term stability) and undrained (short-term stability) was greater than 1.33. Calculations for overall (global) stability of the CIP Cantilevered Wall 4W22 is provided in Appendix IX.

5.3 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 11 and Table 12.

Table 11. 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	120	1,500	0°	N/A	N/A	N/A
Very Stiff to Hard Cohesive Soil	130	3,000	0°	N/A	N/A	N/A
Loose Granular Soil	125	0	28°	0.32	0.53	5.07
Medium Dense Granular Soil	130	0	32°	0.27	0.47	6.82
Dense to Very Dense Granular Soil	135	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 12. 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	120	0	26°	0.35	0.56	4.53
Very Stiff to Hard Cohesive Soil	130	0	28°	0.32	0.53	5.07
Loose Granular Soil	125	0	28°	0.32	0.53	5.07
Medium Dense Granular Soil	130	0	32°	0.27	0.47	6.82
Dense to Very Dense Granular Soil	135	0	36°	0.23	0.41	9.09
Compacted Cohesive Engineered Fill	120	0	30°	0.30	0.50	5.58
Compacted Granular Engineered Fill	120	0	32°	0.27	0.47	6.82

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

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

5.4 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).

5.4.1 Excavation Considerations

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

Table 13. 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

5.4.2 Groundwater Considerations

Based on the groundwater observations made during drilling, groundwater is anticipated during construction of the drilled shafts. Where groundwater is encountered, proper groundwater control should be employed and maintained to prevent disturbance to excavation bottoms consisting of cohesive soil, and to prevent the possible development of a quick or "boiling" condition where soft silts and/or fine sands are encountered. It is preferable that the groundwater level, if encountered, be maintained at least 36 inches below the deepest excavation.

In the case of drilled shafts, the utilization of casing will be required below the water table to maintain an open hole and prevent the sidewalls from collapse. In addition, concrete placed below the water table should be placed by tremie method using a rigid tremie pipe. Given the granular nature of the soils, groundwater may not be able to be controlled by pumping from temporary sumps, and more significant dewatering efforts, such as deep well or well points system will likely be required. Note that determining and maintaining actual groundwater levels during construction of drilled shafts 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.

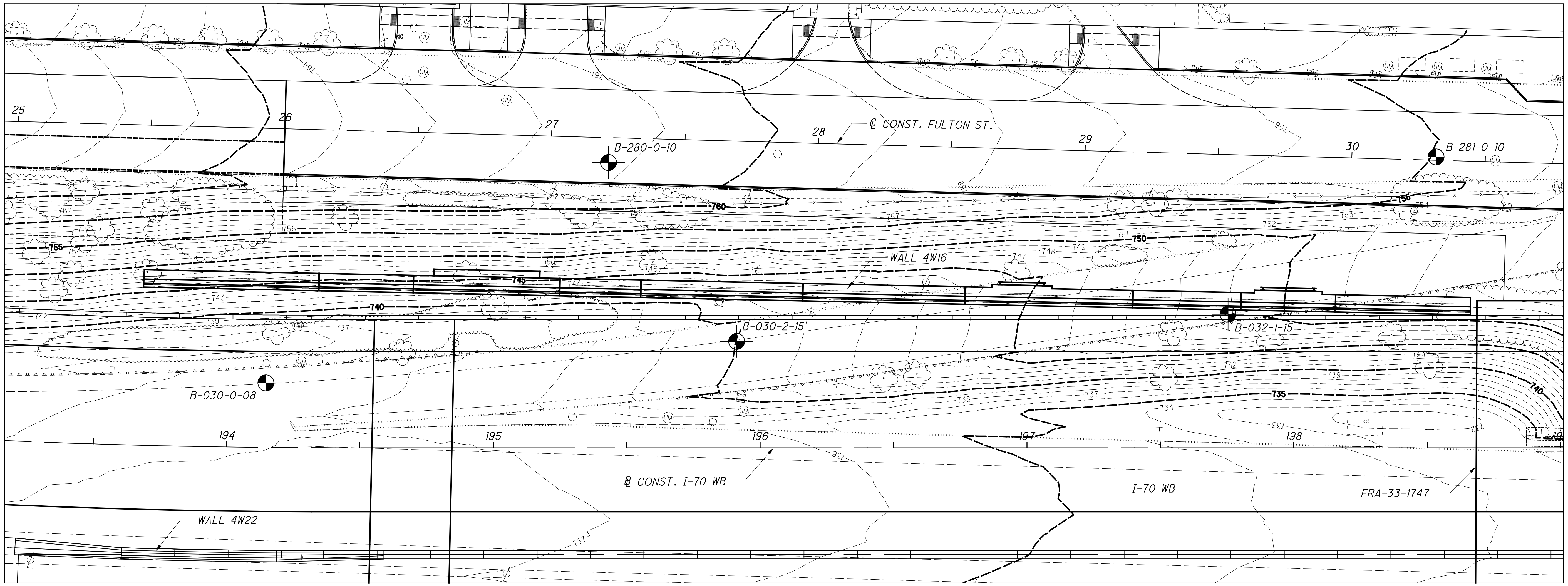
The conclusions and recommendations herein have been based upon the available soil and bedrock information and the design details furnished by a representative of the owner of the proposed project. Any revision in the plans for the proposed construction from those anticipated in this report should be 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 - RETAINING WALL 4W16 AND 4W22

FRA-70-14.05

FRANKLIN COUNTY, OHIO

RII PROJECT NO. W-15-126	DRAWN JAS
SCALE: 1"=20' (0 10 20)	REVIEWED BRT
	DATE 07-09-22

RESOURCE INTERNATIONAL

APPENDIX II

DESCRIPTION OF SOIL TERMS

DESCRIPTION OF SOIL TERMS

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

CLASSIFICATION OF SOILS

Ohio Department of Transportation

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

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

MATERIAL CLASSIFIED BY VISUAL INSPECTION

- Sod and Topsoil
- Pavement or Base
- Uncontrolled Fill (Describe)
- Bouldery Zone
- 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-030-0-08, B-030-2-15 and B-032-1-15

DLZ Ohio, Inc. * 6121 Huntley Road, Columbus, Ohio 43229 * (614) 888-0040

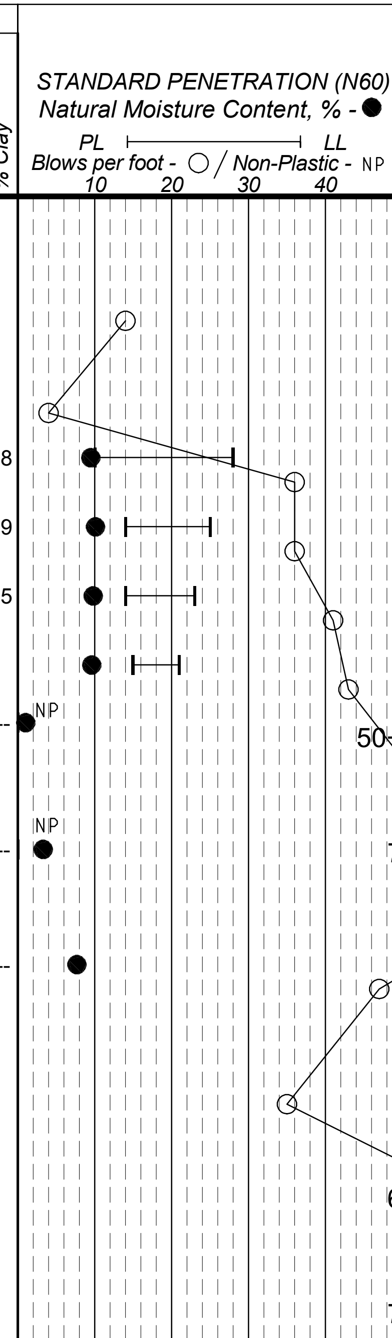
Client: ms consultants | **Project:** FRA-70-8.93 | **Job No.** 0221-1004.01

LOG OF: Boring B-030-0-08 | **Location:** Sta. 194+14.54, 24.17' LT., BL I-70 WB | **Date Drilled:** 7/20/2008 to 7/23/2008

WATER OBSERVATIONS:
Water seepage at: 16.0'
Water level at completion: 13.6' (includes drilling water)

FIELD NOTES:
Advanced boring using 3.25" diameter hollowstem augers.

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
	736.7												
1.4	735.3						Asphalt Concrete - 11" Aggregate Base - 6"						
		4 6 8	11	1			Loose to medium dense brown GRAVEL WITH SAND (A-1-b), some fine to coarse sand, little silt; damp.						
5.0	731.7	3 3 1	8	2									
6.5	730.2	4 14 21	18	3		4.5+	Hard gray SILTY CLAY (A-6b), some fine to coarse sand, trace gravel; contains few rock fragments; damp to moist.	9	9	---	14	40	28
8.0	728.7	10 16 19	18	4		4.5+	Hard gray SILT AND CLAY (A-6a), some fine to coarse sand, trace gravel; damp.	6	10	---	15	40	29
		13 19 21	18	5		4.5+	Hard gray SANDY SILT (A-4a), some fine to coarse sand, trace gravel; damp.	5	11	---	18	41	25
11.0	725.7	11 15 27	18	6		4.5+							
		16 50/5	8	7			Dense to very dense gray GRAVEL (A-1-a), little to some fine to coarse sand, trace to little silty clay; damp.	71	12	---	5	--12--	
		11 33 38	13	8				59	21	---	8	--12--	
		18 23 23	13	9			@ 16.0', becomes wet.	61	20	---	8	--11--	
		12 13 21	12	10									
23.5	713.2	13 28 36	12	11									
25	711.7	17 34 40	14	12			Very dense gray COARSE AND FINE SAND (A-3a), some silty clay; wet.						



DLZ Ohio, Inc. * 6121 Huntley Road, Columbus, Ohio 43229 * (614) 888-0040

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

LOG OF: Boring B-030-0-08 | Location: Sta. 194+14.54, 24.17' LT., BL I-70 WB | Date Drilled: 7/20/2008 to 7/23/2008

WATER OBSERVATIONS:
Water seepage at: 16.0'
Water level at completion: 13.6' (includes drilling water)

FIELD NOTES:
Advanced boring using 3.25" diameter hollowstem augers.

Depth (ft)	Elev. (ft) 711.7	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N60)
							Very dense gray COARSE AND FINE SAND (A-3a), some silty clay; wet.							
		23 45 50/4	15	13			@ 26.0', 0.4 feet sand heave.							50+
30		9 21 31	16	14										53
35		14 30 45	16	15				0	8	---	69	--23--		77
40		50/4	4	16			@ 38.5'-38.8', contains sandy silt seams.							50+
							@ 38.5'-43.5', difficult drilling.							
45		11 32 50/5	17	17			@ 43.5', 1.9 feet sand heave.							50+
47.0	689.7						Very dense gray GRAVEL WITH SAND (A-1-b), trace silt; wet.							
50	686.7	12 50/5	10	18				15	54	---	23	--8--		50+

STANDARD PENETRATION (N60)
Natural Moisture Content, % - ●
PL ⊢—⊣ LL
Blows per foot - ○ / Non-Plastic - NP
10 20 30 40

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

LOG OF: Boring B-030-0-08 | Location: Sta. 194+14.54, 24.17' LT., BL I-70 WB | Date Drilled: 7/20/2008 to 7/23/2008

WATER OBSERVATIONS:
Water seepage at: 16.0'
Water level at completion: 13.6' (includes drilling water)

FIELD NOTES:
Advanced boring using 3.25" diameter hollowstem augers.

STANDARD PENETRATION (N60)
Natural Moisture Content, % - ●
PL |—| LL
Blows per foot - ○ / Non-Plastic - NP
10 20 30 40

Depth (ft)	Elev. (ft) 686.7	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	N60
							Very dense gray COARSE AND FINE SAND (A-3a), little gravel, little to some silty clay; wet.							
55		41 50/6	10	19			@ 53.5', 0.7 feet sand heave.							50+
60		24 39 45	13	20										86
62.0	674.7						Very dense gray FINE SAND (A-3), trace silt, trace gravel; wet.							
65		16 31 41	15	21										73
67.0	669.7						Very dense gray GRAVEL WITH SAND (A-1-b), trace to little silty clay; wet.							
70		33 50/5	9	22				49	29	---	13	--9--		50+
75	661.7	43 50/3	9	23										50+

Client: ms consultants	Project: FRA-70-8.93	Job No. 0221-1004.01
LOG OF: Boring B-030-0-08	Location: Sta. 194+14.54, 24.17' LT., BL I-70 WB	Date Drilled: 7/20/2008 to 7/23/2008

WATER OBSERVATIONS:
Water seepage at: 16.0'
Water level at completion: 13.6' (includes drilling water)

FIELD NOTES:
Advanced boring using 3.25" diameter hollowstem augers.

Depth (ft)	Elev. (ft) 661.7	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N60) / Natural Moisture Content, %
							Very dense gray GRAVEL WITH SAND (A-1-b), trace to little silty clay; wet.							
80		35 50/5	10	24			@ 78.5', 1.0 foot sand heave.							50+
85		21 47 47	13	25			@ 83.5', 4.6 feet sand heave.	15	39	---	33	--13--		96
90		15 23 46	9	26										70
95		37 50/3	9	27										50+
100	636.7	38 50/5	7	28										50+

Legend: PL ⊢—⊣ LL; Blows per foot - ○ / Non-Plastic - NP; scale 10 20 30 40

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

LOG OF: Boring B-030-0-08 | Location: Sta. 194+14.54, 24.17' LT., BL I-70 WB | Date Drilled: 7/20/2008 to 7/23/2008

WATER OBSERVATIONS:
Water seepage at: 16.0'
Water level at completion: 13.6' (includes drilling water)

FIELD NOTES:
Advanced boring using 3.25" diameter hollowstem augers.

Depth (ft)	Elev. (ft) 636.7	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	Graphic Log	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N60) / Natural Moisture Content, % - ● / PL ⊢—⊣ LL / Blows per foot - ○ / Non-Plastic - NP (10 20 30 40)
103.5	633.2						Very dense gray GRAVEL WITH SAND (A-1-b), trace to little silty clay; wet. @ 100.0'-103.5', 25.0 to 27.3 feet sand heave; washed out twice.								
105.0	631.7	50/3	3	29			Severely weathered gray SHALE.								50+
		Core 60"	Rec 57"	RQD 37%	R1		Shale, dark gray, highly weathered, weak, laminated, slightly calcareous, contains abundant pyritic inclusions, fissile, laminated, fractured to highly fractured, RQD 38%, Loss 4%. @ 107.7'-107.8', encounterd large pyritic inclusion. @ 108.1'-109.1', moderately weathered, moderately strong.								
111.0	625.7	Core 12"	Rec 12"	RQD 42%	R2		@ 111.0', core steel damaged. Abandoned boring at 111.0'.								

Bottom of Boring - 111.0'

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-14.05 PROJECT 4B	DRILLING FIRM / OPERATOR: RII / S.B.	DRILL RIG: CME 55 (SN 386345)	STATION / OFFSET: 195+91.21 / 39.9" LT
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: RII / C.D.	HAMMER: CME AUTOMATIC	ALIGNMENT: BL CONST. I-70 WB
PID: 96053 BR ID: NA	DRILLING METHOD: 3.25" - HSA	CALIBRATION DATE: 10/20/14	ELEVATION: 740.0 (MSL) EOB: 54.3 ft.
START: 12/1/15 END: 12/2/15	SAMPLING METHOD: SPT	ENERGY RATIO (%): 92	COORD: 39.953401, -82.997585

EXPLORATION ID B-030-2-15

MATERIAL DESCRIPTION AND NOTES	ELEV. 740.0	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
0.5' - ASPHALT (6.0")	739.5															
1.0' - AGGREGATE BASE (12.0")	738.5															
FILL: MEDIUM DENSE TO VERY DENSE, DARK BROWN **GRAVEL WITH SAND AND SILT**, LITTLE CLAY, MOIST.		4 7 9	24	33	SS-1	-	-	-	-	-	-	-	-	-	8	A-2-4 (V)
-BRICK FRAGMENTS PRESENT IN SS-1 THROUGH SS-3		4 8 10	27	61	SS-2	-	34	22	10	19	15	21	15	6	10	A-2-4 (0)
		6 20 14	51	67	SS-3	-	-	-	-	-	-	-	-	-	8	A-2-4 (V)
	729.5	7 8 5	20	50	SS-4	-	-	-	-	-	-	-	-	-	-	A-2-4 (V)
FILL: MEDIUM DENSE, GRAY **GRAVEL (CONCRETE FRAGMENTS)**, DAMP.	726.5	33 7 8	23	44	SS-5	-	-	-	-	-	-	-	-	-	6	A-1-a (V)
VERY LOOSE TO MEDIUM DENSE, BROWN **GRAVEL**, "AND" COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST TO WET.		8 6 5	17	0	SS-6	-	-	-	-	-	-	-	-	-	-	
		5	-	100	3S-6A	-	52	32	5	9	2	NP	NP	NP	9	A-1-a (0)
-MUD ADDED @ 17.5'		3 1 1	3	0	SS-7	-	-	-	-	-	-	-	-	-	-	
	721.5	1	-	100	3S-7A	-	-	-	-	-	-	-	-	-	13	A-1-a (V)
VERY LOOSE, BROWN **GRAVEL AND SAND**, LITTLE SILT, TRACE CLAY, WET.	718.0	2 1 1	3	61	SS-8	-	44	37	6	10	3	NP	NP	NP	21	A-1-b (0)
VERY STIFF, BROWNISH GRAY **SANDY SILT**, SOME CLAY, TRACE FINE GRAVEL, MOIST.	713.0	10 14 15	44	100	SS-9	4.00	-	-	-	-	-	-	-	-	12	A-4a (V)
VERY DENSE, GRAY **COARSE AND FINE SAND**, SOME SILT, TRACE CLAY, TRACE FINE GRAVEL, MOIST.		6 14 33	71	100	SS-10	-	7	26	36	23	8	NP	NP	NP	14	A-3a (0)

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

W

BACK FILL

PID: 96053	BR ID: NA	PROJECT: FRA-70-14.05 PROJECT 4B	STATION / OFFSET: 195+91.21 / 39.9' LT	START: 12/1/15	END: 12/2/15	PG 2 OF 2	B-030-2-15

MATERIAL DESCRIPTION AND NOTES	ELEV. 710.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
VERY DENSE, GRAY **COARSE AND FINE SAND**, SOME SILT, TRACE CLAY, TRACE FINE GRAVEL, MOIST. *(same as above)*	708.0	31–32																
VERY DENSE, GRAY **SANDY SILT**, SOME FINE GRAVEL, TRACE CLAY, MOIST.		34–35	8 31 48	119	89	SS-11	-	-	-	-	-	-	-	-	-	9	A-4a (V)	
		39–40	4 23 33	84	100	SS-12	-	20	16	21	36	7	NP	NP	NP	10	A-4a (2)	
	698.0	42																
VERY DENSE, GRAY **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, MOIST.		44	9 50/5"	-	100	SS-13	-	37	55	4	4	0	NP	NP	NP	11	A-1-b (0)	
		49	30 50/2"	-	88	SS-14	-	-	-	-	-	-	-	-	-	13	A-1-b (V)	
	688.0	52																
VERY DENSE, GRAY **GRAVEL WITH SAND AND SILT**, TRACE CLAY, MOIST.	685.7	54 EOB	27 50/4"	-	80	SS-15	-	-	-	-	-	-	-	-	-	8	A-2-4 (V)	

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 17.5'

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

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-14.05 PROJECT 4B	DRILLING FIRM / OPERATOR: RII / S.B.	DRILL RIG: CME 55 (SN 386345)	STATION / OFFSET: 197+75.35 / 50.0" LT
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: RII / C.D.	HAMMER: CME AUTOMATIC	ALIGNMENT: BL CONST. I-70 WB
PID: 96053 BR ID: NA	DRILLING METHOD: 3.25" - HSA	CALIBRATION DATE: 10/20/14	ELEVATION: 748.9 (MSL) EOB: 65.0 ft.
START: 11/30/15 END: 12/1/15	SAMPLING METHOD: SPT	ENERGY RATIO (%): 92	COORD: 39.953517, -82.996945

EXPLORATION ID **B-032-1-15**

PAGE 1 OF 3

MATERIAL DESCRIPTION AND NOTES	ELEV. 748.9
0.7' - ASPHALT (8.0")	748.2
0.3' - AGGREGATE BASE (4.0")	747.9
HARD, BROWN **SILT**, SOME FINE GRAVEL, LITTLE COARSE TO FINE SAND, LITTLE CLAY, DAMP.	743.4
DENSE TO VERY DENSE, GRAY TO BROWNISH GRAY **GRAVEL WITH SAND AND SILT**, SOME CLAY, DAMP.	735.9
VERY DENSE, BROWNISH GRAY **GRAVEL**, SOME COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, DAMP.	733.4
HARD, GRAY **SANDY SILT**, LITTLE FINE GRAVEL, LITTLE CLAY, DAMP.	731.9
VERY DENSE, BROWN **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, MOIST.	721.9
DENSE TO VERY DENSE, BROWNISH GRAY TO GRAY **GRAVEL**, "AND" COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST. -INTRODUCED WATER @ 28.5'	

DEPTHS: 1 – 29 (W at 28.5')

SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
3-7-10	26	39	SS-1	4.5+	-	-	-	-	-	-	-	-	12	A-4b (V)
9-15-14	44	100	SS-2	4.5+	21	9	6	50	14	20	15	5	9	A-4b (6)
11-20-25	68	100	SS-3	-	-	-	-	-	-	-	-	-	10	A-2-4 (V)
13-19-24	65	100	SS-4	-	27	23	20	9	21	25	15	10	9	A-2-4 (0)
23-50/3"	-	89	SS-5	-	-	-	-	-	-	-	-	-	6	A-2-4 (V)
45-50/2"	-	100	SS-6	-	-	-	-	-	-	-	-	-	4	A-1-a (V)
10-16-45	92	100	SS-7 A	4.5+	16	17	17	39	11	21	15	6	10	A-4a (3)
			SS-7B	-	-	-	-	-	-	-	-	-	5	A-1-b (V)
16-25-32	86	100	SS-8	-	42	51	3	3	1	NP	NP	NP	4	A-1-b (0)
13-20-22	63	100	SS-9	-	-	-	-	-	-	-	-	-	5	A-1-b (V)
7-9-13	33	100	SS-10	-	55	37	4	3	1	NP	NP	NP	9	A-1-a (0)

PID: 96053	BR ID: NA	PROJECT: FRA-70-14.05 PROJECT 4B	STATION / OFFSET: 197+75.35 / 50.0' LT	START: 11/30/15	END: 12/1/15	PG 2 OF 3	B-032-1-15

MATERIAL DESCRIPTION AND NOTES	ELEV. 718.9
DENSE TO VERY DENSE, BROWNISH GRAY TO GRAY **GRAVEL**, "AND" COARSE TO FINE SAND, TRACE SILT, TRACE CLAY, MOIST. *(same as above)*	711.9
HARD, GRAY **SANDY SILT**, SOME GRAVEL, TRACE CLAY, MOIST.	706.9
LOOSE, BROWNISH GRAY **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, MOIST.	701.9
VERY DENSE, GRAY AND DARK GRAY **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, MOIST.	

DEPTHS: 31, 32, 33, 34, 35, 36, 37, 38, 39, 40, 41, 42, 43, 44, 45, 46, 47, 48, 49, 50, 51, 52, 53, 54, 55, 56, 57, 58, 59, 60, 61

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)
4 17 50/5"	-	100	SS-11	-	-	-	-	-	-	-	-	-	8	A-1-a (V)
42 5/5"	-	100	SS-12	4.5+	-	-	-	-	-	-	-	-	10	A-4a (V)
1 2 4	9	100	SS-13	-	46	39	7	7	1	NP	NP	NP	13	A-1-b (0)
47 50/1"	-	100	SS-14	-	-	-	-	-	-	-	-	-	12	A-1-b (V)
4 14 40	81	100	SS-15	-	-	-	-	-	-	-	-	-	13	A-1-b (V)
5 15 32	71	100	SS-16	-	24	63	6	6	1	NP	NP	NP	10	A-1-b (0)

BACK FILL

PID: 96053	BR ID: NA	PROJECT: FRA-70-14.05 PROJECT 4B	STATION / OFFSET: 197+75.35 / 50.0' LT	START: 11/30/15	END: 12/1/15	PG 3 OF 3	B-032-1-15

MATERIAL DESCRIPTION AND NOTES	ELEV. 686.8	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
VERY DENSE, GRAY AND DARK GRAY **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, MOIST. *(same as above)*		63																
	683.9	64 EOB 65	8 12 25	56	100	SS-17	-	-	-	-	-	-	-	-	-	16	A-1-b (V)	

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 28.0'

ABANDONMENT METHODS, MATERIALS, QUANTITIES: COMPACTED WITH THE AUGER 100 LBS BENTONITE CHIPS AND SOIL CUTTINGS

APPENDIX IV

DRILLED SHAFT CALCULATIONS

Boring	Proposed Top of Shaft Elevation (ft msl)	D_w (ft)	Shaft Diameter, D (ft)	Soil Class.	Material Type [1]	Stratum Depth, z (ft)	Stratum Thickness (ft)	Bottom Elevation (ft msl)	N_{60} [2]	γ (pcf)	σ_v' (Midpoint) (psf)	σ_v (Bottom) (psf)	S_u [3] (psf)	N_c [4]	α [5]	$(N_1)_{60}$ [6]	φ'_f [7]	σ_p' [8] (psf)	β [9]	Boring	Elevation (ft msl)	Shaft Length (ft)	Nominal Unit Tip Resistance, q_p [10,11] (ksf)	Nominal Unit Side Resistance, q_s [12,13] (ksf)	φ_{qp} [14]	φ_{qs} [15]
B-030-0-08	731.0	0.0	5.0	A-6b	C	2.3	2.3	728.7	36	130	78	299	4,500	6.6	0.49					B-030-0-08	731.0-728.7	0.0-2.3	29	2.19	0.40	0.45
				A-a	C	5.3	3.0	725.7	42	130	257	689	5,250	7.3	0.45						728.7-725.7	2.3-5.3	38	2.37	0.40	0.45
				A-1-a	G	9.8	4.5	721.2	96	135	522	1,297				101	43	30,528	4.76		725.7-721.2	5.3-9.8	60	2.48	0.50	0.55
				A-1-a	G	17.8	8.0	713.2	49	135	975	2,377				47	42	15,582	1.90		721.2-713.2	9.8-17.8	58	1.85	0.50	0.55
				A-3a	G	56.3	38.5	674.7	99	140	2,760	7,767				75	40	15,697	0.92		713.2-674.7	17.8-56.3	60	2.52	0.50	0.55
				A-3	G	61.3	5.0	669.7	73	140	4,447	8,467				47	38	13,075	0.58		674.7-669.7	56.3-61.3	60	2.59	0.50	0.55
				A-1-b	G	97.8	36.5	633.2	109	140	6,058	13,577				61	42	34,662	0.96		669.7-633.2	61.3-97.8	60	5.79	0.50	0.55
B-030-2-15	731.0	0.0	5.0	A-2-4	G	1.5	1.5	729.5	24	125	47	188				33	39	7,632	7.39	B-030-2-15	731.0-729.5	0.0-1.5	28	0.34	0.50	0.55
				A-1-a	G	6.5	5.0	724.5	17	125	250	813				18	38	5,406	1.99		729.5-724.5	1.5-6.5	20	0.49	0.50	0.55
				A-1-b	G	13.0	6.5	718.0	3	115	578	1,560				3	30	954	0.37		724.5-718.0	6.5-13.0	3	0.21	0.50	0.55
				A-4a	C	18.0	5.0	713.0	44	135	930	2,235	5,500	9.0	0.45						718.0-713.0	13.0-18.0	49	2.47	0.40	0.45
				A-3a	G	23.0	5.0	708.0	71	140	1,306	2,935				61	40	12,858	1.30		713.0-708.0	18.0-23.0	60	1.70	0.50	0.55
				A-4a	G	33.0	10.0	698.0	101	140	1,888	4,335				81	38	39,984	1.97		708.0-698.0	23.0-33.0	60	3.71	0.50	0.55
				A-1-b	G	43.0	10.0	688.0	120	140	2,664	5,735				89	42	38,160	1.77		698.0-688.0	33.0-43.0	60	4.71	0.50	0.55
				A-2-4	G	45.0	2.0	686.0	120	140	3,129	6,015				85	41	38,160	1.54		688.0-686.0	43.0-45.0	60	4.82	0.50	0.55
B-032-1-15	727.0	0.0	5.0	A-1-b	G	5.1	5.1	721.9	75	135	185	689				65	42	23,850	7.69	B-032-1-15	727.0-721.9	0.0-5.1	60	1.42	0.50	0.55
				A-1-a	G	10.1	5.0	716.9	33	130	539	1,339				26	39	10,494	1.94		721.9-716.9	5.1-10.1	39	1.04	0.50	0.55
				A-1-a	G	15.1	5.0	711.9	120	140	902	2,039				89	43	38,160	3.81		716.9-711.9	10.1-15.1	60	3.43	0.50	0.55
				A-4a	C	20.1	5.0	706.9	120	135	1,278	2,714	8,000	9.0	0.45						711.9-706.9	15.1-20.1	72	3.60	0.40	0.45
				A-1-b	G	25.1	5.0	701.9	9	130	1,628	3,364				6	33	2,862	0.40		706.9-701.9	20.1-25.1	10	0.65	0.50	0.55
				A-1-b	G	43.1	18.0	683.9	82	140	2,496	5,884				52	42	26,076	1.43		701.9-683.9	25.1-43.1	60	3.57	0.50	0.55

1. C = cohesive soil stratum; G = granular soil stratum
2. N_{60} = average energy corrected N-values over stratum thickness
3. $S_u = 125(N_{60}) \le 8{,}000$ psf (cohesive soil layers)
4. $N_C = 6[1+0.2(Z/D)] \le 9$; Ref. Section 10.8.3.5.1c, AASHTO LRFD BDS (cohesive soil layers)
5. $\alpha = 0.55$ for $S_u/P_a \le 1.5$; $\alpha = 0.55-0.1(S_u/P_a-1.5)$ for $1.5 \le S_u/P_a \le 2.5$, where P_a = 2.12 ksf = 2,120 psf; Ref. Section 10.8.3.5.1b AASHTO LRFD BDS (cohesive soil layers)
6. $(N_1)_{60} = C_nN_{60}$, where $C_N = [0.77\log(40/\sigma_v')] \le 2.0$ ksf, where σ_v' = vetical effective stress at midpoint of soil layer with respect to the entire soil profile for the respective boring; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS (granular soil layers)
7. φ'_f estimated per Table 10.4.6.2.4-1; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS (granular soil layers)
8. $\sigma_p' = n(N_{60})^m(P_a)$, where n = 0.15 and m = 1.0 for A-1-a/1-b and A-2-4/2-6, n = 0.47 and m = 0.6 for A-3/3a, n = 0.47 and m = 0.8 for A-4a/4b soils, and P_a = 2.12 ksf = 2,120 psf; Ref. Section 10.8.3.5.2b, AASHTO LRFD BDS (granular soil layers)
9. $\beta = \tan\varphi'_f(1-\sin\varphi'_f)(\sigma_p'/\sigma_v')$^$(\sin\varphi'_f)$, where σ_v' = vetical effective stress at midpoint of soil layer; Ref. Section 10.8.3.5.2b, AASHTO LRFD BDS (granular soil layers)
10. $q_p = N_CS_u \le 80.0$ ksf; Ref. Section 10.8.3.5.1c, AASHTO LRFD BDS (cohesive soil layers)
11. $q_p = 1.2N_{60} \le 60$ ksf; Ref. Section 10.8.3.5.2c, AASHTO LRFD BDS (granular soil layers)
12. $q_s = \alpha S_u$; Ref. Section 10.8.3.5.1b, AASHTO LRFD BDS (cohesive soil layers)
13. $q_s = \beta\sigma_v'$, where σ_v' = vetical effective stress at midpoint of soil layer; Ref. Section 10.8.3.5.2b, AASHTO LRFD BDS (granular soil layers)
14. φ_{qp} = 0.50 for granular soils layers and 0.40 for cohesive soil layers; Ref. Table 10.5.5.2.4-1, AASHTO LRFD BDS
15. φ_{qs} = 0.55 for granular soils layers and 0.45 for cohesive soil layers; Ref. Table 10.5.5.2.4-1, AASHTO LRFD BDS

Shaft Length (ft)	Shaft Tip Elevation (ft msl)	Nominal Tip Resistance, R_p (kips)	Nominal Side Resistance, R_s (kips)	Total Nominal Resistance, R_n (kips)	Factored Tip Resistance, $\varphi_{qp}R_p$ (kips)	Factored Side Resistance, $\varphi_{qs}R_s$ (kips)	Total Factored Resistance, R_R (kips)
46.2	684.8	1,060		1,060	530		530

Group Efficiency Factor, η = 0.9

Boring	Proposed Top of Shaft Elevation (ft msl)	D_w (ft)	Shaft Diameter, D (ft)	Soil Class.	Material Type [1]	Stratum Depth, z (ft)	Stratum Thickness (ft)	Bottom Elevation (ft msl)	N_{60} [2]	γ (pcf)	σ_v' (Midpoint) (psf)	σ_v (Bottom) (psf)	S_u [3] (psf)	N_c [4]	α [5]	$(N_1)_{60}$ [6]	φ'_f [7]	σ_p' [8] (psf)	β [9]	Boring	Elevation (ft msl)	Shaft Length (ft)	Nominal Unit Tip Resistance, q_p [10,11] (ksf)	Nominal Unit Side Resistance, q_s [12,13] (ksf)	φ_{qp} [14]	φ_{qs} [15]
B-030-0-08	731.0	0.0	6.0	A-6b	C	2.3	2.3	728.7	36	130	78	299	4,500	6.5	0.49					B-030-0-08	731.0-728.7	0.0-2.3	29	2.19	0.40	0.45
				A-a	C	5.3	3.0	725.7	42	130	257	689	5,250	7.1	0.45						728.7-725.7	2.3-5.3	37	2.37	0.40	0.45
				A-1-a	G	9.8	4.5	721.2	96	135	522	1,297				101	43	30,528	4.76		725.7-721.2	5.3-9.8	60	2.48	0.50	0.55
				A-1-a	G	17.8	8.0	713.2	49	135	975	2,377				47	42	15,582	1.90		721.2-713.2	9.8-17.8	58	1.85	0.50	0.55
				A-3a	G	56.3	38.5	674.7	99	140	2,760	7,767				75	40	15,697	0.92		713.2-674.7	17.8-56.3	60	2.52	0.50	0.55
				A-3	G	61.3	5.0	669.7	73	140	4,447	8,467				47	38	13,075	0.58		674.7-669.7	56.3-61.3	60	2.59	0.50	0.55
				A-1-b	G	97.8	36.5	633.2	109	140	6,058	13,577				61	42	34,662	0.96		669.7-633.2	61.3-97.8	60	5.79	0.50	0.55
B-030-2-15	731.0	0.0	6.0	A-2-4	G	1.5	1.5	729.5	24	125	47	188				33	39	7,632	7.39	B-030-2-15	731.0-729.5	0.0-1.5	28	0.34	0.50	0.55
				A-1-a	G	6.5	5.0	724.5	17	125	250	813				18	38	5,406	1.99		729.5-724.5	1.5-6.5	20	0.49	0.50	0.55
				A-1-b	G	13.0	6.5	718.0	3	115	578	1,560				3	30	954	0.37		724.5-718.0	6.5-13.0	3	0.21	0.50	0.55
				A-4a	C	18.0	5.0	713.0	44	135	930	2,235	5,500	9.0	0.45						718.0-713.0	13.0-18.0	49	2.47	0.40	0.45
				A-3a	G	23.0	5.0	708.0	71	140	1,306	2,935				61	40	12,858	1.30		713.0-708.0	18.0-23.0	60	1.70	0.50	0.55
				A-4a	G	33.0	10.0	698.0	101	140	1,888	4,335				81	38	39,984	1.97		708.0-698.0	23.0-33.0	60	3.71	0.50	0.55
				A-1-b	G	43.0	10.0	688.0	120	140	2,664	5,735				89	42	38,160	1.77		698.0-688.0	33.0-43.0	60	4.71	0.50	0.55
				A-2-4	G	45.0	2.0	686.0	120	140	3,129	6,015				85	41	38,160	1.54		688.0-686.0	43.0-45.0	60	4.82	0.50	0.55
B-032-1-15	727.0	0.0	6.0	A-1-b	G	5.1	5.1	721.9	75	135	185	689				65	42	23,850	7.69	B-032-1-15	727.0-721.9	0.0-5.1	60	1.42	0.50	0.55
				A-1-a	G	10.1	5.0	716.9	33	130	539	1,339				26	39	10,494	1.94		721.9-716.9	5.1-10.1	39	1.04	0.50	0.55
				A-1-a	G	15.1	5.0	711.9	120	140	902	2,039				89	43	38,160	3.81		716.9-711.9	10.1-15.1	60	3.43	0.50	0.55
				A-4a	C	20.1	5.0	706.9	120	135	1,278	2,714	8,000	9.0	0.45						711.9-706.9	15.1-20.1	72	3.60	0.40	0.45
				A-1-b	G	25.1	5.0	701.9	9	130	1,628	3,364				6	33	2,862	0.40		706.9-701.9	20.1-25.1	10	0.65	0.50	0.55
				A-1-b	G	43.1	18.0	683.9	82	140	2,496	5,884				52	42	26,076	1.43		701.9-683.9	25.1-43.1	60	3.57	0.50	0.55

Shaft Length (ft)	Shaft Tip Elevation (ft msl)	Nominal Tip Resistance, R_p (kips)	Nominal Side Resistance, R_s (kips)	Total Nominal Resistance, R_n (kips)	Factored Tip Resistance, $\varphi_{qp}R_p$ (kips)	Factored Side Resistance, $\varphi_{qs}R_s$ (kips)	Total Factored Resistance, R_R (kips)
46.2	684.8	1,527		1,527	763		763

1. C = cohesive soil stratum; G = granular soil stratum
2. N_{60} = average energy corrected N-values over stratum thickness
3. $S_u = 125(N_{60}) \leq 8{,}000$ psf (cohesive soil layers)
4. $N_C = 6[1+0.2(Z/D)] \leq 9$; Ref. Section 10.8.3.5.1c, AASHTO LRFD BDS (cohesive soil layers)
5. $\alpha = 0.55$ for $S_u/P_a \leq 1.5$; $\alpha = 0.55-0.1(S_u/P_a-1.5)$ for $1.5 \leq S_u/P_a \leq 2.5$, where P_a = 2.12 ksf = 2,120 psf; Ref. Section 10.8.3.5.1b AASHTO LRFD BDS (cohesive soil layers)
6. $(N_1)_{60} = C_nN_{60}$, where $C_N = [0.77\log(40/\sigma_v')] \leq 2.0$ ksf, where σ_v' = vetical effective stress at midpoint of soil layer with respect to the entire soil profile for the respective boring; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS (granular soil layers)
7. φ'_f estimated per Table 10.4.6.2.4-1; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS (granular soil layers)
8. $\sigma_p' = n(N_{60})^m(P_a)$, where n = 0.15 and m = 1.0 for A-1-a/1-b and A-2-4/2-6, n = 0.47 and m = 0.6 for A-3/3a, n = 0.47 and m = 0.8 for A-4a/4b soils, and P_a = 2.12 ksf = 2,120 psf; Ref. Section 10.8.3.5.2b, AASHTO LRFD BDS (granular soil layers)
9. $\beta = \tan\varphi'_f(1-\sin\varphi'_f)(\sigma_p'/\sigma_v')\text{^}(\sin\varphi'_f)$, where σ_v' = vetical effective stress at midpoint of soil layer; Ref. Section 10.8.3.5.2b, AASHTO LRFD BDS (granular soil layers)
10. $q_p = N_CS_u \leq 80.0$ ksf; Ref. Section 10.8.3.5.1c, AASHTO LRFD BDS (cohesive soil layers)
11. $q_p = 1.2N_{60} \leq 60$ ksf; Ref. Section 10.8.3.5.2c, AASHTO LRFD BDS (granular soil layers)
12. $q_s = \alpha S_u$; Ref. Section 10.8.3.5.1b, AASHTO LRFD BDS (cohesive soil layers)
13. $q_s = \beta\sigma_v'$, where σ_v' = vetical effective stress at midpoint of soil layer; Ref. Section 10.8.3.5.2b, AASHTO LRFD BDS (granular soil layers)
14. φ_{qp} = 0.50 for granular soils layers and 0.40 for cohesive soil layers; Ref. Table 10.5.5.2.4-1, AASHTO LRFD BDS
15. φ_{qs} = 0.55 for granular soils layers and 0.45 for cohesive soil layers; Ref. Table 10.5.5.2.4-1, AASHTO LRFD BDS

Group Efficiency Factor, η = 0.9

W-15-126 - FRA-70-14.05 Project 4B		Calculated By: BRT	Date: 7/8/2022
Tangent Shafts - Block Failure Mode - Retaining Wall 4W16		Checked By: JPS	Date: 7/8/2022

Boring B-030-0-08

$D =$	5.0	ft	Diameter of individual drilled shafts
$B' =$	3.4	ft	Equivalent footing width based on overall end bearing area of drilled shafts
$L =$	497.5	ft	
$c =$	0	psf	
$\gamma =$	135	pcf	
$D_f =$	46.2	ft	
$\varphi =$	40	deg	
$D_w =$	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma} = 278.44 \text{ ksf}$$

$$N_{cm} = N_c s_c i_c = 75.75 \qquad N_{qm} = N_q s_q d_q i_q = 85.27 \qquad N_{\gamma m} = N_\gamma s_\gamma i_\gamma = 109.11$$

$N_c =$	75.31	$s_c =$	1+(3.4 ft/497.5 ft)(64.2/75.31) =	1.006	$i_c =$	1.000	$d_q =$	1+2tan(40°)[1-sin(40°)]²tan⁻¹(46.2 ft/3.4 ft) =	1.321
$N_q =$	64.20	$s_q =$	1+(3.4 ft/497.5 ft)tan(40°) =	1.006	$i_q =$	1.000	$C_{wq} =$	0.0 ft < 46.2 ft =	0.500
$N_\gamma =$	109.41	$s_\gamma =$	1-0.4(3.4 ft/497.5 ft) =	0.997	$i_\gamma =$	1.000	$C_{w\gamma} =$	0.0 ft < 1.5(3.4 ft) + 46.2 ft =	0.500

$$q_R = q_n \cdot \phi_b = 139.22 \text{ ksf}$$

$\varphi_b =$ 0.5

$$\boldsymbol{R_R = q_R \cdot A_p} = 2{,}734 \text{ kips}$$

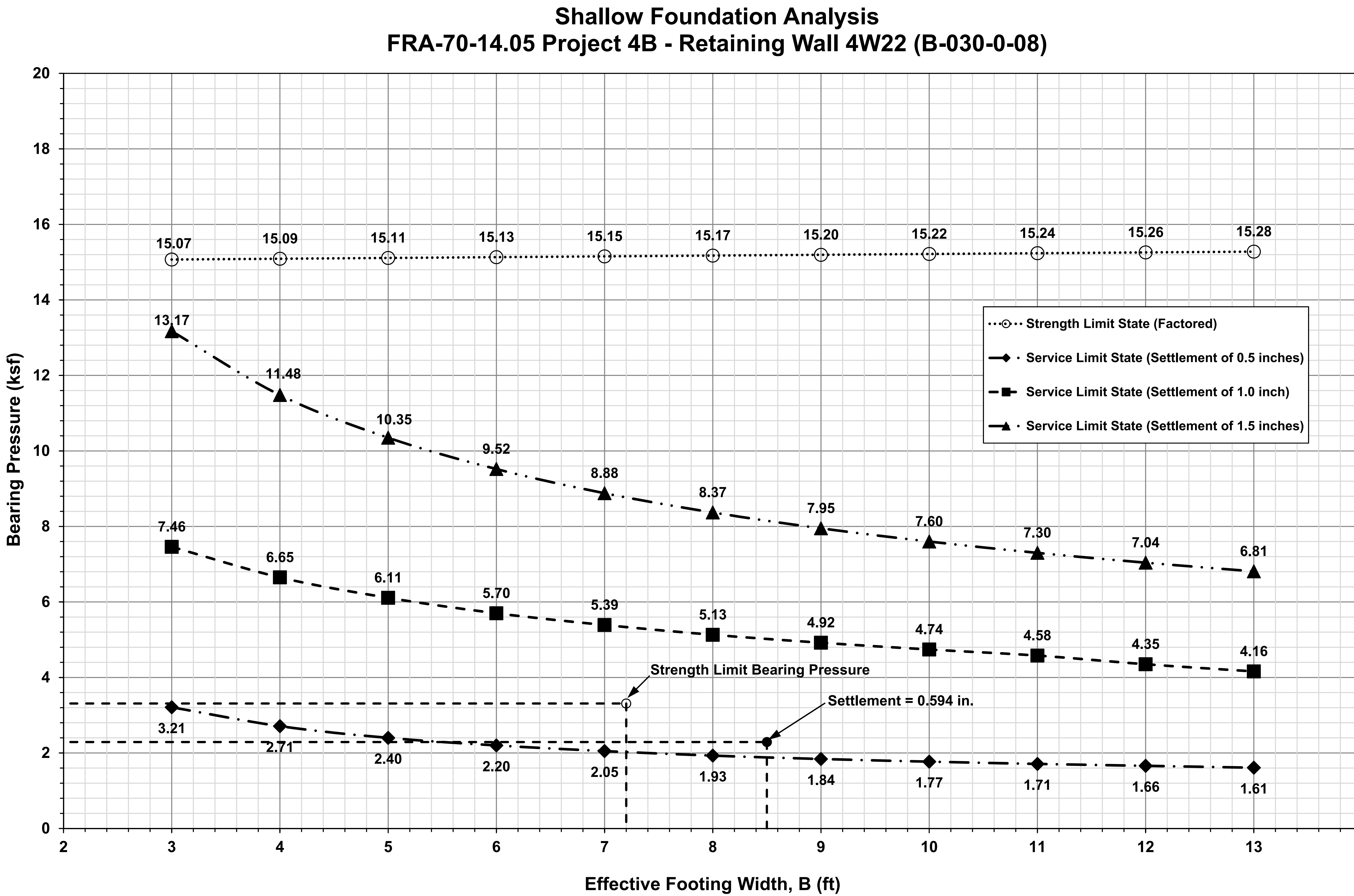
APPENDIX V

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-030-0-08	736.7 to 731.7	A-1-b	G	4	9	15	120	120	φ = 36°	160 pci	-	-
	731.7 to 728.7	A-6b	C	3	36	36	130	130	Su = 4,500 psf	1,500 pci	0.0045	-
	728.7 to 725.7	A-4a	C	3	42	42	130	130	Su = 5,250 psf	1,750 pci	0.0043	-
	725.7 to 721.2	A-1-a	G	4	96	101	140	140	φ = 43°	395 pci	-	-
	721.2 to 713.2	A-1-a	G	4	49	47	135	73	φ = 42°	195 pci	-	-
	713.2 to 674.7	A-3a	G	4	99	75	140	78	φ = 40°	155 pci	-	-
	674.7 to 669.7	A-3	G	4	73	47	140	78	φ = 38°	125 pci	-	-
	669.7 to 633.2	A-1-b	G	4	109	61	140	78	φ = 42°	195 pci	-	-
	633.2 to 625.7	Shale	R	9	-	-	150	88	Qu = 750 psi	0.00025	68,000 psi	38
B-030-2-15	740.0 to 729.5	A-2-4	G	4	24	33	125	125	φ = 39°	250 pci	-	-
	729.5 to 724.5	A-1-a	G	4	17	18	130	130	φ = 38°	215 pci	-	-
	724.5 to 718.0	A-1-b	G	4	3	3	120	58	φ = 30°	30 pci	-	-
	718.0 to 713.0	A-4a	C	2	44	44	140	78	Su = 5,500 psf	1,835 pci	0.0042	-
	713.0 to 708.0	A-3a	G	4	71	61	140	78	φ = 40°	155 pci	-	-
	708.0 to 698.0	A-4a	G	4	101	81	140	78	φ = 38°	125 pci	-	-
	698.0 to 688.0	A-1-b	G	4	120	89	140	78	φ = 42°	195 pci	-	-
	688.0 to 686.0	A-2-4	G	4	120	85	140	78	φ = 41°	175 pci	-	-
B-032-1-15	748.9 to 743.4	A-4b	C	3	35	35	130	130	Su = 4,375 psf	1,460 pci	0.0045	-
	743.4 to 738.4	A-2-4	G	4	66	80	135	135	φ = 41°	315 pci	-	-
	738.4 to 733.4	A-2-4	G	4	120	126	140	140	φ = 41°	315 pci	-	-
	733.4 to 731.9	A-4a	C	3	50	50	135	135	Su = 6,250 psf	2,085 pci	0.0039	-
	731.9 to 721.9	A-1-b	G	4	75	65	140	140	φ = 42°	355 pci	-	-
	721.9 to 716.9	A-1-a	G	4	33	26	140	78	φ = 39°	140 pci	-	-
	716.9 to 711.9	A-1-a	G	4	120	89	140	78	φ = 43°	215 pci	-	-
	711.9 to 706.9	A-4a	C	2	120	120	140	78	Su = 8,000 psf	2,665 pci	0.0033	-
	706.9 to 701.9	A-1-b	G	4	9	6	130	68	φ = 33°	60 pci	-	-
	701.9 to 683.9	A-1-b	G	4	82	52	140	78	φ = 42°	195 pci	-	-

APPENDIX VI

BEARING RESISTANCE CHART


Shallow Foundation Analysis
FRA-70-14.05 Project 4B - Retaining Wall 4W22 (B-030-0-08)
Bearing Pressure (ksf)
Effective Footing Width, B (ft)
Strength Limit State (Factored)
Service Limit State (Settlement of 0.5 inches)
Service Limit State (Settlement of 1.0 inch)
Service Limit State (Settlement of 1.5 inches)
Strength Limit Bearing Pressure
Settlement = 0.594 in.
15.07
15.09
15.11
15.13
15.15
15.17
15.20
15.22
15.24
15.26
15.28
13.17
11.48
10.35
9.52
8.88
8.37
7.95
7.60
7.30
7.04
6.81
7.46
6.65
6.11
5.70
5.39
5.13
4.92
4.74
4.58
4.35
4.16
3.21
2.71
2.40
2.20
2.05
1.93
1.84
1.77
1.71
1.66
1.61

APPENDIX VII

SHALLOW FOUNDATION CALCULATIONS

W-15-126 - FRA-70-14.05 Project 4B - Retaining Wall 4W22
Shallow Foundation Analysis - Settlement

Calculated By: BRT Date: 7/8/2022
Checked By: JPS Date: 7/8/2022

Boring B-030-0-08

B =	8.5	ft	Effective Footing width
D_w =	0.0	ft	Depth below bottom of footing
q =	2,290	psf	Service limit bearing pressure at bottom of wall
q_{net} =	1,690	psf	Net bearing pressure at bottom of wall (considers initial overburden stress of 600 psf from 5-foot cut to bottom of footing elevation)

Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	N_{60}	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_m (1) (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	$(N1)_{60}$ (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)
A-4a	C	0	1.2	1.2	0.6	42	130	156	78	41	4,000	4,041	22	0.108	0.011	0.444			0.07	0.999	1,688	1,729	0.015	0.176
A-1-a	G	1.2	3.2	2.0	2.2	96	135	426	291	154							179	300	0.26	0.956	1,615	1,769	0.007	0.085
A-1-a	G	3.2	5.7	2.5	4.5	96	135	764	595	317							155	300	0.52	0.803	1,358	1,675	0.006	0.072
A-1-a	G	5.7	9.7	4.0	7.7	49	130	1,284	1,024	543							70	270	0.91	0.590	998	1,541	0.007	0.080
A-1-a	G	9.7	13.7	4.0	11.7	49	135	1,824	1,554	823							64	233	1.38	0.426	720	1,544	0.005	0.056
A-3a	G	13.7	19.7	6.0	16.7	99	140	2,664	2,244	1,201							116	300	1.96	0.311	525	1,727	0.003	0.038
A-3a	G	19.7	25.7	6.0	22.7	99	140	3,504	3,084	1,667							105	300	2.67	0.233	394	2,061	0.002	0.022
A-3a	G	25.7	31.7	6.0	28.7	99	140	4,344	3,924	2,133							97	300	3.38	0.186	314	2,447	0.001	0.014
A-3a	G	31.7	37.7	6.0	34.7	99	140	5,184	4,764	2,598							91	300	4.08	0.154	261	2,859	0.001	0.010
A-3a	G	37.7	44.7	7.0	41.2	99	140	6,164	5,674	3,103							85	275	4.85	0.130	220	3,323	0.001	0.009
A-3a	G	44.7	52.2	7.5	48.5	99	140	7,214	6,689	3,665							79	250	5.70	0.111	188	3,853	0.001	0.008
A-3	G	52.2	57.2	5.0	54.7	73	140	7,914	7,564	4,150							55	128	6.44	0.099	167	4,317	0.001	0.008
A-1-b	G	57.2	64.2	7.0	60.7	109	140	8,894	8,404	4,616							79	300	7.14	0.089	150	4,766	0.000	0.004
A-1-b	G	64.2	71.2	7.0	67.7	109	140	9,874	9,384	5,159							75	295	7.96	0.080	135	5,294	0.000	0.003
A-1-b	G	71.2	78.2	7.0	74.7	109	140	10,854	10,364	5,702							71	274	8.79	0.072	122	5,824	0.000	0.003
A-1-b	G	78.2	85.2	7.0	81.7	109	140	11,834	11,344	6,245							68	255	9.61	0.066	112	6,357	0.000	0.003
A-1-b	G	85.2	93.7	8.5	89.5	109	140	13,024	12,429	6,847							64	237	10.52	0.060	102	6,949	0.000	0.003
																					Total Settlement:			0.594 in

1. $\sigma_p' = \sigma_{vo}'+\sigma_m$; Estimate σ_m of 4,000 psf for moderately overconsolidated soil deposit; Ref. Table 11.2, Coduto 2003
2. C_c = 0.009(LL-10); Ref. Table 6-9, FHWA GEC 5
3. $C_r = 0.10(C_c)$; Ref. Chapter 8.11, Holtz and Kovacs 1981
4. $e_o = (C_c/0.54)+0.35$; Ref. Table 6-11, FHWA GEC 5
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; $I = [\beta+\sin(\beta)\cos(\beta+2\delta)]/\pi$, where $\beta = \tan^{-1}[(x+B/2)/Z_f]-\delta$, $\delta = \tan^{-1}[(x-B/2)/Z_f]$ and x = horizontal distance from center of footing; Ref. Figure 6.13 and Equation 6.24, Das 2005
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)

Boring B-030-0-08

B =	7.2	ft	
L =	138	ft	
c =	5,250	psf	
γ =	120	pcf	
D_f =	5.0	ft	
φ =	0	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma} = 27.56 \text{ ksf}$$

$$N_{cm} = N_c s_c i_c = 5.19$$

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

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

N_c =	5.14	s_c =	1+(7.2 ft/138 ft)(1/5.14) =	1.010	i_c =	1.000	d_q =	1+2tan(0°)[1-sin(0°)]²tan⁻¹(5 ft/7.2 ft) =	1.000
N_q =	1.00	s_q =	1+(7.2 ft/138 ft)tan(0°) =	1.000	i_q =	1.000	C_{wq} =	0.0 ft < 5.0 ft =	0.500
N_γ =	0.00	s_γ =	1-0.4(7.2 ft/138 ft) =	0.979	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(7.2 ft) + 5 ft =	0.500

$$q_R = q_n \cdot \phi_b = 15.16 \text{ ksf}$$

φ_b = 0.55 (Per Table 11.5.7-1, AASHTO LRFD BDS) Semigravity Wall

APPENDIX VIII

EXTERNAL STABILITY ANALYSIS CALCULATIONS BY GPD GROUP

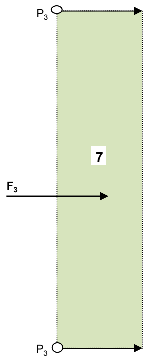
GPD GROUP	Client:	ODOT/District 6	Job No.:	2015370	
	Project:	FRA-70 Project 4B	Page No.:	1	Of 3
	Subject:	Wall 4W22 Design	Designed:	DJC	Date: 1/18/2022
			Checked:	MOJ	Date: 5/2/2022

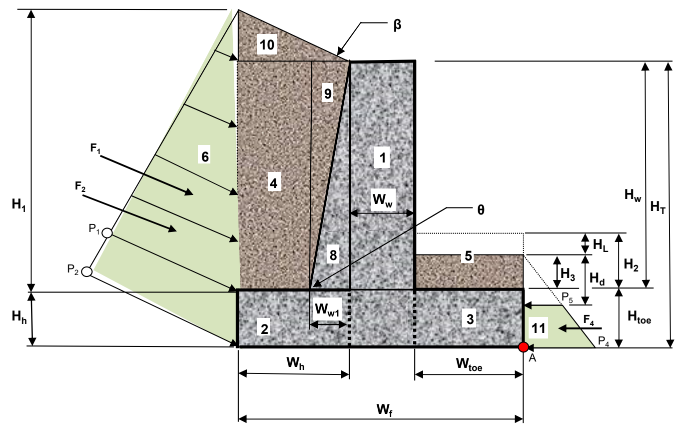
Spread Footing Retaining Wall Design

Based on AASHTO LRFD Bridge Design Specifications (9th edition) and the 2020 ODOT BDM.

Wall Data:

Concrete Unit Weight, γ_c =	0.150	kcf
Toe Height, H_{toe} =	2.25	ft
Heel Height, H_h =	2.00	ft
Wall Height, H_w =	11.35	ft
Total Height, $H_T = H_w + H_{toe}$ =	13.60	ft
Soil Height over Heel, $H_1 = H_T - H_h + (W_h * S_e)$ =	11.60	ft
Max. Soil Height over Toe, H_2 =	4.00	ft
Future Loss of Soil over Toe, H_L =	0.00	ft
Min. Soil Height over Toe, $H_3 = \max(0, H_2 - H_L)$ =	4.00	ft
Depth of Disturbance, H_d =	2.67	ft
Wall Width, W_w =	4.26	ft
Toe Width, W_{toe} =	2.41	ft
Heel Width, W_h =	2.83	ft
Additional Wall, W_{w1} =	0.00	ft
Theta, θ =	90.00	deg.
Footing Width, W_f =	9.50	ft





Soil Data:

Is Retained Soil Sloped?	No		
Slope of Embankment, S_e =	0.00		
Beta, β =	0.00	deg.	
Include Surcharge over Heel?	Yes		
Include Surcharge over Toe?	Yes		
Is traffic less than $(H_h+H_1)/2$ from back of ftg.?	Yes		
Dist. from back of ftg. to edge of traffic =	-2.83	ft	*LRFD 3.11.6.4*
Minimum Soil Unit Weight for LLS, $\gamma_{soil\ LLS}$ =	0.125	kcf	*BDM 307.1.1*
Surcharge Height behind Wall, H_s =	2.96	ft	*LRFD Table 3.11.6.4-1*
Surcharge Height in front of Wall, H_{sf} =	4.63	ft	
$P_{soil\ LLS} = \gamma_{soil\ LLS} * (k_a \text{ or } k_0)$ =	37.16	pcf	
Active or At Rest Pressure?	Active		
Retained Soil Unit Weight, γ_{soil} =	0.120	kcf	*BDM Table 307-1*
Footing Resting On?	Granular		
Internal Friction Angle of Soil, δ =	35.00	deg.	*@ Base of the Footer*
Internal Friction Angle of Fill, ϕ'_{fill} =	30.00	deg.	
Friction Angle between Fill & Wall, δ =	20.00	deg.	*LRFD 3.11.5.3*
Active Lateral Earth Press. Coefficient, ka =	0.30		*LRFD 3.11.5.3-1 (Coulomb)*
$P_{soil} = \gamma_{soil} * (k_a \text{ or } k_0)$ =	35.68	pcf	
Bearing on soil or rock?	Soil		*LRFD 10.6.1.4*
Factor Bearing Resistance (Strength) =	22.150	ksf	
Bearing Capacity (Service) =	5.335	ksf	*To Check Settlement*
Consider Passive Force on Toe?	No		
Passive Lat. Earth Pressure Coeff., k_p =	3.00		$k_p = \tan^2(45° + \phi'_f/2)$

Horizontal Sliding Resistance:
LRFD 10.6.3.4

For cohesionless soils:

V_{min} =	16.09	kips
Resistance, $R_\tau = V_{min} * \tan(\delta)$ =	11.27	kips

For cohesive soils:

The lesser of: C_u =	N.A.	ksf
$0.5*\sigma'_v$ =	N.A.	ksf
Unit Shear Resistance: Use =	N.A.	ksf
Resistance, $R\tau$ =	N.A.	kips

Manual Override:

Override Friction Factor =		
Resistance, $R\tau$ =	N.A.	kips

Typical values for friction factor:
LRFD Table C3.11.5.3-1

rock = 0.70
course grained soil w/out silt = 0.55
course grained soil w/silt = 0.45

Additional friction factors for other common substrates
shale = 0.55
silt = 0.35

Soil Pressure Calculations:

$P_1 = P_{soil} * H_1/1000$ =	0.41	ksf
$P_2 = P_{soil} * (H_1+H_h)/1000$ =	0.49	ksf
$P_3 = H_s * P_{soil\ LLS}/1000$ =	0.11	ksf
$P_4 = \gamma_{soil} * k_p * (H_{toe}+H_2-H_L)$ =	2.25	ksf
$P_5 = \gamma_{soil} * k_p * H_d$ =	0.96	ksf

Soil Sliding Force Calculations:

$F_1 = P_1 * H_1 * 0.5$ =	2.40	kips
$F_2 = P_2 * (H_1+H_h) * 0.5$ =	3.30	kips
$F_3 = P_3 * H_1$ =	1.50	kips
F_4 (Trapezoid 11) =	0.00	kips

Additional Dead Load =	0.92	kips	*Assumes 1.25 max. & 0.90 min. load factors.*
Moment Arm for Additional Dead Load =	5.59	ft	*from Point A*

Force and Moment Arm Calculations:

Area 1 = γ_c x W_w x H_T =	0.150 kcf x	4.26 ft. x	13.60 ft. x	1.00 ft. =		**8.69 kips**
Arm 1 = W_{toe} + W_w / 2 =	2.41 ft. +	4.26 ft.	/ 2.00 =			**4.71 ft.**
Area 2 = γ_c x W_h x H_h =	0.150 kcf x	2.83 ft. x	2.00 ft. x	1.00 ft. =		**0.85 kips**
Arm 2 = W_{toe} + W_w + W_h / 2 =	2.41 ft. +	4.26 ft. +	2.83 ft.	/ 2.00 =		**8.08 ft.**
Area 3 = γ_c x W_{toe} x H_{toe} =	0.150 kcf x	2.41 ft. x	2.25 ft. x	1.00 ft. =		**0.81 kips**
Arm 3 = W_{toe} / 2 =	2.41 ft.	/ 2.00 =				**1.20 ft.**
Area 4 = γ_s x (W_h - W_{w1}) x H_W =	0.120 kcf x	(2.83 ft. -	0.00 ft.) x	11.35 ft. x	1.00 ft. =	**3.86 kips**
Arm 4 = W_{toe} + W_w + W_{w1} + (W_h - W_{w1}) / 2 =	2.41 ft. +	4.26 ft. +	0.00 ft. +	(2.83 ft. -	0.00 ft.) / 2 =	**8.08 ft.**
Area 5 **(Max.)** = γ_s x W_{toe} x H_2 =	0.120 kcf x	2.41 ft. x	4.00 ft. x	1.00 ft. =		**1.16 kips**
Area 5 **(Min.)** = γ_s x W_{toe} x H_3 =	0.120 kcf x	2.41 ft. x	4.00 ft. x	1.00 ft. =		**1.16 kips**
Arm 5 = W_{toe} / 2 =	2.41 ft.	/ 2.00 =				**1.20 ft.**
Area 6 **(Horiz. Comp.)** = F_2 x cos(δ) =	3.30 kips x	cos (	20.00 deg.) =			**3.10 kips**
Arm 6 = (H_1 + H_h) / 3 =	(11.60 ft. +	2.00 ft.)	/ 3.00 =			**4.53 ft.**
Area 6 **(Vertical Comp.)** = F_2 x sin(δ) =	3.30 kips x	sin (	20.00 deg.) =			**1.13 kips**
Arm 6 = Wf =	9.50 ft.					**9.50 ft.**
Area 7 = F_3 =	1.50 kips					**1.50 kips**
Arm 7 = (H_1 + H_h) / 2 =	(11.60 ft. +	2.00 ft.)	/ 2.00 =			**6.80 ft.**
Area 8 = 0.5 x γ_c x W_{w1} x H_W =	0.5 x 0.150 kcf x	0.00 ft. x	11.35 ft. x	1.00 ft. =		**0.00 kips**
Arm 8 = W_{toe} + W_w + W_{w1} / 3 =	2.41 ft. +	4.26 ft. +	0.00 ft.	/ 3.00 =		**6.67 ft.**

Revised for wall geometry.

Force and Moment Arm Calculations (Continued):

Area 9 = 0.5 x γ_s x W_{w1} x H_W =	0.5 x 0.120 kcf x	0.00 ft. x	11.35 ft. x	1.00 ft. =		0.00 kips
Arm 9 = W_{toe} + W_w + W_{w1} x 2/3 =	2.41 ft. +	4.26 ft. +	0.00 ft.	x 2.00	/ 3.00 =	6.67 ft.
Area 10 = 0.5 x γ_s x (S_e x W_h) x W_h =	0.5 x 0.120 kcf x	(0.00 x	2.83 ft.) x	2.83 ft. x	1.00 ft. =	0.00 kips
Arm 10 = W_F - W_h / 3 =	9.50 ft. -	2.83 ft.	/ 3.00 =			8.56 ft.
Area 11 = F_4 =	0.00 kips					0.00 kips
Surcharge on Heel = $\gamma_{soil\ LLS}$ x W_h x H_s =	0.125 kcf x	2.83 ft. x	2.96 ft. x	1.00 ft. =		1.05 kips
Arm for Heel Surcharge = W_F - W_h / 2 =	9.50 ft. -	2.83 ft.	/ 2.00 =			8.08 ft.
Surcharge on Toe = $\gamma_{soil\ LLS}$ x W_{toe} x H_{sf} =	0.125 kcf x	2.41 ft. x	4.63 ft. x	1.00 ft. =		1.39 kips
Arm for Toe Surcharge = W_{toe} / 2 =	2.41 ft.	/ 2.00 =				1.20 ft.

Check Bearing Pressure:
per BDM 307.1.5 and LRFD 11.6.3.2.

Factored Bearing Resistance = **22.15 ksf**

Maximum Strength Load Pressures:
Bearing pressure at Toe = **3.31 ksf** **OK**
Bearing pressure at Heel = **3.31 ksf** **OK**

Check Eccentricity:
per BDM 307.1.4 and LRFD 11.6.3.3.

Maximum Allowable e = B/3 = **3.17 ft**
Controlling Eccentricity = **1.35 ft** **OK**

Check Sliding:
per BDM 307.1.3 and LRFD 11.6.3.6.

Resistance factor, ϕ_τ (Sliding) = *1.00* *LRFD Table 11.5.7-1*
Resistance factor, ϕ_{ep} (Passive pressure) = *0.50* *LRFD Table 10.5.5.2.2-1*

Sliding Resistance:
Unfactored Horizontal Sliding Resistance = 11.27 kips
Factored Horizontal Sliding Resistance = 11.27 kips

Passive Resistance on Footing Toe:
Unfactored Passive Resistance = 0.00 kips
Factored Passive Resistance = 0.00 kips

Passive Resistance on Footing Key or Sheet Piling (Below bottom of Footing):
Vertical Projection Below Footing = *0.00* ft

Pressure at Bottom of Footing (P_4) = 2.25 ksf
Pressure at Bottom of Disturbance (P_5) = 0.96 ksf
Pressure at Bottom of Key or Sheeting = 2.25 ksf

Unfactored Passive Resistance = 0.00 kips
Factored Passive Resistance = 0.00 kips

Total Factored Resisting Force = **11.27 kips**
Driving Force = **7.27 kips** **OK**

Check Settlement:

Service Bearing Capacity = **5.33 ksf**
Service Bearing Pressure at Toe = **2.28 ksf** **OK**
Service Bearing Pressure at Heel = **2.28 ksf** **OK**

Summary of Load Effects:

	MAX. BEARING PRESSURE	MIN. BEARING PRESSURE	ECCENTRICITY MAX. LF	ECCENTRICITY MIN. LF	SLIDING FORCES MAX. LF	VERTICAL FORCES MIN. LF
STRENGTH I	3.31	3.31	1.13	1.35	7.27	16.09
SERVICE I	2.28	2.28	0.81	N/A	4.60	16.57

Load Modification Factors:
LRFD 1.3.3, LRFD 1.3.4, LRFD 1.3.5, & BDM 1001

Ductility η_D = *1.00* *(use 1.00 for all limit states)*
Redundancy η_R = *1.00* *(use 1.00 for redundant structures and 1.05 for non-redundant structures)*
Operational importance η_I = *1.00* *(use 1.00 for all limit states)*

STRENGTH I Load Combination

Sliding Forces & Overturning Moments

*1.50*EH+1.75*LS(H). Ignores resisting moments from passive force on toe/key/sheeting, which is conservative.*

ΣM about point "A"

Area/Force	Unfactored Load	Load Factor	Force (k)	Moment Arm (ft)	Moment (k-ft) Max. Load Factor	
6 (Horizontal comp.)	3.10	1.50	4.65	4.53	21.08	Horiz. Forces
7	1.50	1.75	2.62	6.80	17.80	
		Σ Sliding Forces, F_s =	7.27 kips	Σ Overturning Moments =	38.89 k*ft.	

Vertical Forces & Resisting Moments

*1.5*DC+1.35*EV+1.75*LS $_V$ (Max.)   0.9*DC+1.0*EV (Min.)*

Note: Area 1 load factors revised for wall geometry.

ΣM about point "A"

Area/Force	Force (k) Unfactored Load	Max. Load Factor	Force (k) Max. Load Factor	Min. Load Factor	*This column is for stability* Force (k) Min. Load Factor	Moment Arm (ft)	Moment (k-ft) Max. Load Factor	*This column is for stability* Moment (k-ft) Min. Load Factor	
1	8.69	**1.13**	9.80	**0.81**	7.06	4.71	46.19	33.26	Dead Loads From Concrete
2	0.85	1.25	1.06	0.90	0.77	8.08	8.59	6.18	
3	0.81	1.25	1.02	0.90	0.73	1.20	1.22	0.88	
8	0.00	1.25	0.00	0.90	0.00	6.67	0.00	0.00	
4	3.86	1.35	5.21	1.00	3.86	8.08	42.11	31.19	Dead Loads From Soil *(Do not include 5 (Min.) and 5 (Max.) simultaneously)*
5 (Max.)	1.16	1.35	1.56	1.00	1.16	1.20	1.88	1.39	
5 (Min.)	1.16	1.35	1.56	1.00	1.16	1.20	1.88	1.39	
6 (Vertical comp.)	1.13	1.50	1.69	1.50	1.69	9.50	16.08	16.08	
9	0.00	1.35	0.00	1.00	0.00	6.67	0.00	0.00	
10	0.00	1.35	0.00	1.00	0.00	8.56	0.00	0.00	
Surcharge on Heel	1.05	1.75	1.83	0.00	0.00	8.08	14.83	0.00	External Loads
Surcharge on Toe	1.39	1.75	2.43	0.00	0.00	1.20	2.93	0.00	
DC	0.92	1.25	1.16	0.90	0.83	5.59	6.46	4.65	
		Σ Vert. Forces =	23.93 kips	Σ Vert. Forces =	16.09 kips	Σ Resist. Moments =	125.46 k*ft.	93.64 k*ft.	

Note: Calculations for each controlling load case are not necessarily shown below, but have been included in the design checks.

Max. Load Factor Calculations (Worst case bearing pressure shown.)	
Overturning Moment = Σ Overturning Moments =	38.89 k-ft.
Resisting Moment = Σ Max. Resisting Moments =	125.46 k-ft.
Net Moment = Resisting Moment - Overturning Moment =	86.57 k-ft.
Total Vertical Force (TVF) = Σ Vert. Forces =	23.93 kips
Dist. from Point A (Ā) = Net. Moment / TVF =	3.62 ft.
Eccentricity "e" = (0.5*W_f) - Ā =	1.13 ft.
Maximum Bearing Pressure = TVF/(Wf-2*e) =	3.31 ksf
Minimum Bearing Pressure = TVF/(Wf-2*e) =	3.31 ksf

Min. Load Factor Calculations (Worst case eccentricity shown.)	
Overturning Moment = Σ Overturning Moments =	38.89 k-ft.
Resisting Moment = Σ Min. Resisting Moments =	93.64 k-ft.
Net Moment = Resisting Moment - Overturning Moment =	54.75 k-ft.
Total Vertical Force (TVF) = Σ Vert. Forces =	16.09 kips
Dist. from Point A (Ā) = Net. Moment / TVF =	3.40 ft.
Eccentricity "e" = (0.5*W_f) - Ā =	1.35 ft.

SERVICE I Load Combination

Sliding Forces & Overturning Moments

*1.0*EH+1.0*LS $_H$. Ignores resisting moments from passive force on toe/key/sheeting, which is conservative.*

ΣM about point "A"

Area/Force	Unfactored Load	Load Factor	Force (k)	Moment Arm (ft)	Moment (k-ft) Max. Load Factor	
6 (Horizontal comp.)	3.10	1.00	3.10	4.53	14.06	Horiz. Forces
7	1.50	1.00	1.50	6.80	10.17	
		Σ Sliding Forces, F_s =	4.60 kips	Σ Overturning Moments =	24.23 k*ft.	

Vertical Forces & Resisting Moments

*1.0*DC+1.0*EV+1.0*LS $_V$*

Note: Area 1 load factor revised for wall geometry.

ΣM about point "A"

Area/Force	Force (k) Unfactored Load	Load Factor	Force (k)	Moment Arm (ft)	Moment (k-ft)	
1	8.69	**0.90**	7.84	4.71	36.95	Dead Loads From Concrete
2	0.85	1.00	0.85	8.08	6.87	
3	0.81	1.00	0.81	1.20	0.98	
8	0.00	1.00	0.00	6.67	0.00	
4	3.86	1.00	3.86	8.08	31.19	Dead Loads From Soil (Do not include 5 (Min.) and 5 (Max.) simultaneously)
5 (Max.)	1.16	1.00	1.16	1.20	1.39	
5 (Min.)	1.16	1.00	1.16	1.20	1.39	
6 (Vertical comp.)	1.13	1.00	1.13	9.50	10.72	
9	0.00	1.00	0.00	6.67	0.00	
10	0.00	1.00	0.00	8.56	0.00	
Surcharge on Heel	1.05	1.00	1.05	8.08	8.47	External Loads
Surcharge on Toe	1.39	1.00	1.39	1.20	1.67	
DC	0.92	1.00	0.92	5.59	5.17	
		Σ Vert. Forces =	19.01 kips	Σ Resisting Moments =	103.42 k*ft.	

Note: Calculations for each controlling load case are not necessarily shown below, but have been included in the design checks.

Calculations for worst case bearing pressure shown.	
Overturning Moment = Σ Overturning Moments =	24.23 k-ft.
Resisting Moment = Σ Max. Resisting Moments =	103.42 k-ft.
Net Moment = Resisting Moment - Overturning Moment =	79.19 k-ft.
Total Vertical Force (TVF) = Σ Vert. Forces =	19.01 kips
Dist. from Point A (Ā) = Net. Moment / TVF =	4.17 ft.
Eccentricity "e" = (0.5*W_f) - Ā =	0.58 ft.
Maximum Bearing Pressure = TVF/(Wf-2*e) =	2.28 ksf
Minimum Bearing Pressure = TVF/(Wf-2*e) =	2.28 ksf

- Where the wall is supported by a soil foundation:

the vertical stress shall be calculated assuming a uniformly distributed pressure over an effective base area as shown in Figure 11.6.3.2-1.

The vertical stress shall be calculated as follows:

$$\sigma_v = \frac{\Sigma V}{B - 2e} \quad (11.6.3.2\text{-}1)$$

- Where the wall is supported by a rock foundation:

the vertical stress shall be calculated assuming a linearly distributed pressure over an effective base area as shown in Figure 11.6.3.2-2. If the resultant is within the middle one-third of the base:

$$\sigma_{vmax} = \frac{\Sigma V}{B}\left(1 + 6\frac{e}{B}\right) \quad (11.6.3.2\text{-}2)$$

$$\sigma_{vmin} = \frac{\Sigma V}{B}\left(1 - 6\frac{e}{B}\right) \quad (11.6.3.2\text{-}3)$$

where the variables are as defined in Figure 11.6.3.2-2. If the resultant is outside the middle one-third of the base:

$$\sigma_{vmax} = \frac{2\Sigma V}{3[(B/2) - e)]} \quad (11.6.3.2\text{-}4)$$

$$\sigma_{vmin} = 0 \quad (11.6.3.2\text{-}5)$$

where the variables are as defined in Figure 11.6.3.2-2.

Client:	ODOT/District 6	Job No.:	2015370		
Project:	FRA-70 Project 4B	Page No.:	1	Of	3
Subject:	Wall 4W22 Transition Design	Designed:	DJC	Date:	1/18/2022
		Checked:	MOJ	Date:	5/2/2022

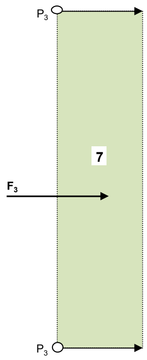
GPD GROUP

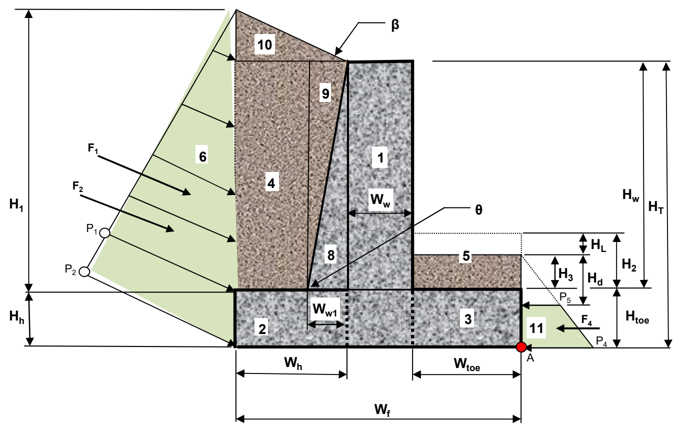
Spread Footing Retaining Wall Design

Based on AASHTO LRFD Bridge Design Specifications (9th edition) and the 2020 ODOT BDM.

Wall Data:

Concrete Unit Weight, γ_c =	0.150	kcf
Toe Height, H_{toe} =	2.25	ft
Heel Height, H_h =	2.00	ft
Wall Height, H_w =	11.35	ft
Total Height, $H_T = H_w+H_{toe}$ =	13.60	ft
Soil Height over Heel, $H_1 = H_T-H_h+(W_h*S_e)$ =	11.60	ft
Max. Soil Height over Toe, H_2 =	4.00	ft
Future Loss of Soil over Toe, H_L =	0.00	ft
Min. Soil Height over Toe, $H_3 = \max(0,H_2-H_L)$ =	4.00	ft
Depth of Disturbance, H_d =	2.67	ft
Wall Width, W_w =	6.24	ft
Toe Width, W_{toe} =	2.33	ft
Heel Width, W_h =	0.93	ft
Additional Wall, W_{w1} =	0.00	ft
Theta, θ =	90.00	deg.
Footing Width, W_f =	9.50	ft





Soil Data:

Is Retained Soil Sloped?	No		
Slope of Embankment, S_e =	0.00		
Beta, β =	0.00	deg.	
Include Surcharge over Heel?	Yes		
Include Surcharge over Toe?	Yes		
Is traffic less than $(H_h+H_1)/2$ from back of ftg.?	Yes		
Dist. from back of ftg. to edge of traffic =	-0.93	ft	*LRFD 3.11.6.4*
Minimum Soil Unit Weight for LLS, $\gamma_{soil\ LLS}$ =	0.125	kcf	*BDM 307.1.1*
Surcharge Height behind Wall, H_s =	2.96	ft	*LRFD Table 3.11.6.4-1*
Surcharge Height in front of Wall, H_{sf} =	4.63	ft	
$P_{soil\ LLS} = \gamma_{soil\ LLS} * (k_a \text{ or } k_0)$ =	37.16	pcf	
Active or At Rest Pressure?	Active		
Retained Soil Unit Weight, γ_{soil} =	0.120	kcf	*BDM Table 307-1*
Footing Resting On?	Granular		
Internal Friction Angle of Soil, δ =	35.00	deg.	*@ Base of the Footer*
Internal Friction Angle of Fill, ϕ'_{fill} =	30.00	deg.	
Friction Angle between Fill & Wall, δ =	20.00	deg.	*LRFD 3.11.5.3*
Active Lateral Earth Press. Coefficient, ka =	0.30		*LRFD 3.11.5.3-1 (Coulomb)*
$P_{soil} = \gamma_{soil} * (k_a \text{ or } k_0)$ =	35.68	pcf	
Bearing on soil or rock?	Soil		*LRFD 10.6.1.4*
Factor Bearing Resistance (Strength) =	22.590	ksf	
Bearing Capacity (Service) =	5.232	ksf	*To Check Settlement*
Consider Passive Force on Toe?	No		
Passive Lat. Earth Pressure Coeff., k_p =	3.00		$k_p = \tan^2(45°+\phi'_f/2)$

Horizontal Sliding Resistance:
LRFD 10.6.3.4

For cohesionless soils:

V_{min} =	17.25	kips
Resistance, $R_\tau = V_{min} * \tan(\delta)$ =	12.08	kips

For cohesive soils:

The lesser of: C_u =	N.A.	ksf
$0.5*\sigma'_v$ =	N.A.	ksf
Unit Shear Resistance: Use =	N.A.	ksf
Resistance, Rτ =	N.A.	kips

Manual Override:

Override Friction Factor =		
Resistance, Rτ =	N.A.	kips

Typical values for friction factor:
LRFD Table C3.11.5.3-1

rock = 0.70
course grained soil w/out silt = 0.55
course grained soil w/silt = 0.45

Additional friction factors for other common substrates
shale = 0.55
silt = 0.35

Soil Pressure Calculations:

$P_1 = P_{soil}*H_1/1000$ =	0.41	ksf
$P_2 = P_{soil}*(H_1+H_h)/1000$ =	0.49	ksf
$P_3 = H_s*P_{soil\ LLS}/1000$ =	0.11	ksf
$P_4 = \gamma_{soil}*k_p*(H_{toe}+H_2-H_L)$ =	2.25	ksf
$P_5 = \gamma_{soil}*k_p*H_d$ =	0.96	ksf

Soil Sliding Force Calculations:

$F_1 = P_1*H_1*0.5$ =	2.40	kips
$F_2 = P_2*(H_1+H_h)*0.5$ =	3.30	kips
$F_3 = P_3*H_1$ =	1.50	kips
F_4 (Trapezoid 11) =	0.00	kips

Additional Dead Load =	1.69	kips	*Assumes 1.25 max. & 0.90 min. load factors.*
Moment Arm for Additional Dead Load =	6.95	ft	*from Point A*

Force and Moment Arm Calculations:

Area 1 = γ_c x W_w x H_T =	0.150 kcf x	6.24 ft. x	13.60 ft. x	1.00 ft. =			12.73 kips	
Arm 1 = W_{toe} + W_w / 2 =	2.33 ft. +	6.24 ft.	/ 2.00 =				5.64 ft.	**Revised for wall geometry.**
Area 2 = γ_c x W_h x H_h =	0.150 kcf x	0.93 ft. x	2.00 ft. x	1.00 ft. =			0.28 kips	
Arm 2 = W_{toe} + W_w + W_h / 2 =	2.33 ft. +	6.24 ft. +	0.93 ft.	/ 2.00 =			9.04 ft.	
Area 3 = γ_c x W_{toe} x H_{toe} =	0.150 kcf x	2.33 ft. x	2.25 ft. x	1.00 ft. =			0.79 kips	
Arm 3 = W_{toe} / 2 =	2.33 ft.	/ 2.00 =					1.17 ft.	
Area 4 = γ_s x (W_h -W_{w1}) x H_W =	0.120 kcf x	(0.93 ft. -	0.00 ft.) x	11.35 ft. x	1.00 ft. =		1.26 kips	
Arm 4 = W_{toe} + W_w + W_{w1} + (W_h -W_{w1}) / 2 =	2.33 ft. +	6.24 ft. +	0.00 ft. +	(0.93 ft. -	0.00 ft.) / 2 =		9.04 ft.	
Area 5 **(Max.)** = γ_s x W_{toe} x H_2 =	0.120 kcf x	2.33 ft. x	4.00 ft. x	1.00 ft. =			1.12 kips	
Area 5 **(Min.)** = γ_s x W_{toe} x H_3 =	0.120 kcf x	2.33 ft. x	4.00 ft. x	1.00 ft. =			1.12 kips	
Arm 5 = W_{toe} / 2 =	2.33 ft.	/ 2.00 =					1.17 ft.	
Area 6 **(Horiz. Comp.)** = F_2 x cos(δ) =	3.30 kips x	cos (	20.00 deg.) =				3.10 kips	
Arm 6 = (H_1 + H_h) / 3 =	(11.60 ft. +	2.00 ft.)	/ 3.00 =				4.53 ft.	
Area 6 **(Vertical Comp.)** = F_2 x sin(δ) =	3.30 kips x	sin (	20.00 deg.) =				1.13 kips	
Arm 6 = Wf =	9.50 ft.						9.50 ft.	
Area 7 = F_3 =	1.50 kips						1.50 kips	
Arm 7 = (H_1 + H_h) / 2 =	(11.60 ft. +	2.00 ft.)	/ 2.00 =				6.80 ft.	
Area 8 = 0.5 x γ_c x W_{w1} x H_W =	0.5 x 0.150 kcf x	0.00 ft. x	11.35 ft. x	1.00 ft. =			0.00 kips	
Arm 8 = W_{toe} + W_w + W_{w1} / 3 =	2.33 ft. +	6.24 ft. +	0.00 ft.	/ 3.00 =			8.57 ft.	

Force and Moment Arm Calculations (Continued):

Area 9 = 0.5 x γ_s x W_{w1} x H_W =	0.5 x 0.120 kcf x	0.00 ft. x	11.35 ft. x	1.00 ft. =		**0.00 kips**
Arm 9 = W_{toe} + W_w + W_{w1} x 2/3 =	2.33 ft. +	6.24 ft. +	0.00 ft.	x 2.00	/ 3.00 =	**8.57 ft.**
Area 10 = 0.5 x γ_s x (S_e x W_h) x W_h =	0.5 x 0.120 kcf x	(0.00 x	0.93 ft.) x	0.93 ft. x	1.00 ft. =	**0.00 kips**
Arm 10 = W_F - W_h / 3 =	9.50 ft. -	0.93 ft.	/ 3.00 =			**9.19 ft.**
Area 11 = F_4 =	0.00 kips					**0.00 kips**
Surcharge on Heel = $\gamma_{soil\ LLS}$ x W_h x H_s =	0.125 kcf x	0.93 ft. x	2.96 ft. x	1.00 ft. =		**0.34 kips**
Arm for Heel Surcharge = W_F - W_h / 2 =	9.50 ft. -	0.93 ft.	/ 2.00 =			**9.04 ft.**
Surcharge on Toe = $\gamma_{soil\ LLS}$ x W_{toe} x H_{sf} =	0.125 kcf x	2.33 ft. x	4.63 ft. x	1.00 ft. =		**1.35 kips**
Arm for Toe Surcharge = W_{toe} / 2 =	2.33 ft.	/ 2.00 =				**1.17 ft.**

Check Bearing Pressure:
per BDM 307.1.5 and LRFD 11.6.3.2.

Factored Bearing Resistance = **22.59 ksf**

Maximum Strength Load Pressures:
Bearing pressure at Toe = **3.27 ksf** **OK**
Bearing pressure at Heel = **3.27 ksf** **OK**

Check Eccentricity:
per BDM 307.1.4 and LRFD 11.6.3.3.

Maximum Allowable e = B/3 = **3.17 ft**
Controlling Eccentricity = **1.05 ft** **OK**

Check Sliding:
per BDM 307.1.3 and LRFD 11.6.3.6.

Resistance factor, ϕ_τ (Sliding) = *1.00* *LRFD Table 11.5.7-1*
Resistance factor, ϕ_{ep} (Passive pressure) = *0.50* *LRFD Table 10.5.5.2.2-1*

Sliding Resistance:
Unfactored Horizontal Sliding Resistance = 12.08 kips
Factored Horizontal Sliding Resistance = 12.08 kips

Passive Resistance on Footing Toe:
Unfactored Passive Resistance = 0.00 kips
Factored Passive Resistance = 0.00 kips

Passive Resistance on Footing Key or Sheet Piling (Below bottom of Footing):
Vertical Projection Below Footing = *0.00* ft

Pressure at Bottom of Footing (P_4) = 2.25 ksf
Pressure at Bottom of Disturbance (P_5) = 0.96 ksf
Pressure at Bottom of Key or Sheeting = 2.25 ksf

Unfactored Passive Resistance = 0.00 kips
Factored Passive Resistance = 0.00 kips

Total Factored Resisting Force = **12.08 kips**
Driving Force = **7.27 kips** **OK**

Check Settlement:

Service Bearing Capacity = **5.23 ksf**
Service Bearing Pressure at Toe = **2.29 ksf** **OK**
Service Bearing Pressure at Heel = **2.29 ksf** **OK**

Summary of Load Effects:

	MAX. BEARING PRESSURE	MIN. BEARING PRESSURE	ECCENTRICITY MAX. LF	ECCENTRICITY MIN. LF	SLIDING FORCES MAX. LF	VERTICAL FORCES MIN. LF
STRENGTH I	3.27	3.27	0.84	1.05	7.27	17.25
SERVICE I	2.29	2.29	0.50	N/A	4.60	18.15

Load Modification Factors:
LRFD 1.3.3, LRFD 1.3.4, LRFD 1.3.5, & BDM 1001

Ductility η_D = *1.00* *(use 1.00 for all limit states)*
Redundancy η_R = *1.00* *(use 1.00 for redundant structures and 1.05 for non-redundant structures)*
Operational importance η_I = *1.00* *(use 1.00 for all limit states)*

GPD GROUP	Client: ODOT/District 6		Job No.: 2015370	
	Project: FRA-70 Project 4B		Page No.: 1	Of 3
	Subject: Wall 4W22 Transition Design		Designed: DJC	Date: 1/18/2022
			Checked: MOJ	Date: 5/2/2022

STRENGTH I Load Combination

Sliding Forces & Overturning Moments

*1.50*EH+1.75*LS(H). Ignores resisting moments from passive force on toe/key/sheeting, which is conservative.*

ΣM about point "A"

Area/Force	Unfactored Load	Load Factor	Force (k)	Moment Arm (ft)	Moment (k-ft) Max. Load Factor	
6 (Horizontal comp.)	3.10	1.50	4.65	4.53	21.08	Horiz. Forces
7	1.50	1.75	2.62	6.80	17.80	
		Σ Sliding Forces, F_s =	7.27 kips	Σ Overturning Moments =	38.89 k*ft.	

Vertical Forces & Resisting Moments

*1.5*DC+1.35*EV+1.75*LS$_V$ (Max.)* *0.9*DC+1.0*EV (Min.)*

Note: Area 1 load factors revised for wall geometry.

ΣM about point "A"

Area/Force	Force (k) Unfactored Load	Max. Load Factor	Force (k) Max. Load Factor	Min. Load Factor	*This column is for stability* Force (k) Min. Load Factor	Moment Arm (ft)	Moment (k-ft) Max. Load Factor	*This column is for stability* Moment (k-ft) Min. Load Factor	
1	12.73	**1.17**	14.85	**0.84**	10.69	5.64	83.75	60.30	
2	0.28	1.25	0.35	0.90	0.25	9.04	3.14	2.26	Dead Loads
3	0.79	1.25	0.98	0.90	0.71	1.17	1.15	0.83	From Concrete
8	0.00	1.25	0.00	0.90	0.00	8.57	0.00	0.00	
4	1.26	1.35	1.70	1.00	1.26	9.04	15.40	11.41	Dead Loads
5 (Max.)	1.12	1.35	1.51	1.00	1.12	1.17	1.76	1.31	From Soil *(Do*
5 (Min.)	1.12	1.35	1.51	1.00	1.12	1.17	1.76	1.31	*not include 5*
6 (Vertical comp.)	1.13	1.50	1.69	1.50	1.69	9.50	16.08	16.08	*(Min.) and 5*
9	0.00	1.35	0.00	1.00	0.00	8.57	0.00	0.00	*(Max.)*
10	0.00	1.35	0.00	1.00	0.00	9.19	0.00	0.00	*simultaneously)*
Surcharge on Heel	0.34	1.75	0.60	0.00	0.00	9.04	5.42	0.00	
Surcharge on Toe	1.35	1.75	2.36	0.00	0.00	1.17	2.75	0.00	External Loads
DC	1.69	1.25	2.11	0.90	1.52	6.95	14.67	10.56	
		Σ Vert. Forces =	25.56 kips	Σ Vert. Forces =	17.25 kips	Σ Resist. Moments =	138.71 k*ft.	102.75 k*ft.	

Note: Calculations for each controlling load case are not necessarily shown below, but have been included in the design checks.

Max. Load Factor Calculations (Worst case bearing pressure shown.)	
Overturning Moment = Σ Overturning Moments =	38.89 k-ft.
Resisting Moment = Σ Max. Resisting Moments =	138.71 k-ft.
Net Moment = Resisting Moment - Overturning Moment =	99.82 k-ft.
Total Vertical Force (TVF) = Σ Vert. Forces =	25.56 kips
Dist. from Point A (Ā) = Net. Moment / TVF =	3.91 ft.
Eccentricity "e" = (0.5*W_f) - Ā =	0.84 ft.
Maximum Bearing Pressure = TVF/(Wf-2*e) =	3.27 ksf
Minimum Bearing Pressure = TVF/(Wf-2*e) =	3.27 ksf

Min. Load Factor Calculations (Worst case eccentricity shown.)	
Overturning Moment = Σ Overturning Moments =	38.89 k-ft.
Resisting Moment = Σ Min. Resisting Moments =	102.75 k-ft.
Net Moment = Resisting Moment - Overturning Moment =	63.86 k-ft.
Total Vertical Force (TVF) = Σ Vert. Forces =	17.25 kips
Dist. from Point A (Ā) = Net. Moment / TVF =	3.70 ft.
Eccentricity "e" = (0.5*W_f) - Ā =	1.05 ft.

SERVICE I Load Combination

Sliding Forces & Overturning Moments

*1.0*EH+1.0*LS$_H$. Ignores resisting moments from passive force on toe/key/sheeting, which is conservative.*

ΣM about point "A"

Area/Force	Unfactored Load	Load Factor	Force (k)	Moment Arm (ft)	Moment (k-ft) Max. Load Factor	
6 (Horizontal comp.)	3.10	1.00	3.10	4.53	14.06	Horiz. Forces
7	1.50	1.00	1.50	6.80	10.17	
		Σ Sliding Forces, F_s =	4.60 kips	Σ Overturning Moments =	24.23 k*ft.	

Vertical Forces & Resisting Moments

*1.0*DC+1.0*EV+1.0*LS$_V$*

Note: Area 1 load factor revised for wall geometry.

ΣM about point "A"

Area/Force	Force (k) Unfactored Load	Load Factor	Force (k)	Moment Arm (ft)	Moment (k-ft)	
1	12.73	**0.93**	11.88	5.64	67.00	
2	0.28	1.00	0.28	9.04	2.51	Dead Loads From
3	0.79	1.00	0.79	1.17	0.92	Concrete
8	0.00	1.00	0.00	8.57	0.00	
4	1.26	1.00	1.26	9.04	11.41	Dead Loads
5 (Max.)	1.12	1.00	1.12	1.17	1.31	From Soil (Do not
5 (Min.)	1.12	1.00	1.12	1.17	1.31	include 5 (Min.) and
6 (Vertical comp.)	1.13	1.00	1.13	9.50	10.72	5 (Max.)
9	0.00	1.00	0.00	8.57	0.00	simultaneously)
10	0.00	1.00	0.00	9.19	0.00	
Surcharge on Heel	0.34	1.00	0.34	9.04	3.10	
Surcharge on Toe	1.35	1.00	1.35	1.17	1.57	External Loads
DC	1.69	1.00	1.69	6.95	11.73	
		Σ Vert. Forces =	19.49 kips	Σ Resisting Moments =	107.18 k*ft.	

Note: Calculations for each controlling load case are not necessarily shown below, but have been included in the design checks.

Calculations for worst case bearing pressure shown.	
Overturning Moment = Σ Overturning Moments =	24.23 k-ft.
Resisting Moment = Σ Max. Resisting Moments =	107.18 k-ft.
Net Moment = Resisting Moment - Overturning Moment =	82.95 k-ft.
Total Vertical Force (TVF) = Σ Vert. Forces =	19.49 kips
Dist. from Point A (Ā) = Net. Moment / TVF =	4.25 ft.
Eccentricity "e" = (0.5*W_f) - Ā =	0.50 ft.
Maximum Bearing Pressure = TVF/(Wf-2*e) =	2.29 ksf
Minimum Bearing Pressure = TVF/(Wf-2*e) =	2.29 ksf

- Where the wall is supported by a soil foundation:

the vertical stress shall be calculated assuming a uniformly distributed pressure over an effective base area as shown in Figure 11.6.3.2-1.

The vertical stress shall be calculated as follows:

$$\sigma_v = \frac{\sum V}{B - 2e} \quad (11.6.3.2\text{-}1)$$

- Where the wall is supported by a rock foundation:

the vertical stress shall be calculated assuming a linearly distributed pressure over an effective base area as shown in Figure 11.6.3.2-2. If the resultant is within the middle one-third of the base:

$$\sigma_{vmax} = \frac{\sum V}{B}\left(1 + 6\frac{e}{B}\right) \quad (11.6.3.2\text{-}2)$$

$$\sigma_{vmin} = \frac{\sum V}{B}\left(1 - 6\frac{e}{B}\right) \quad (11.6.3.2\text{-}3)$$

where the variables are as defined in Figure 11.6.3.2-2. If the resultant is outside the middle one-third of the base:

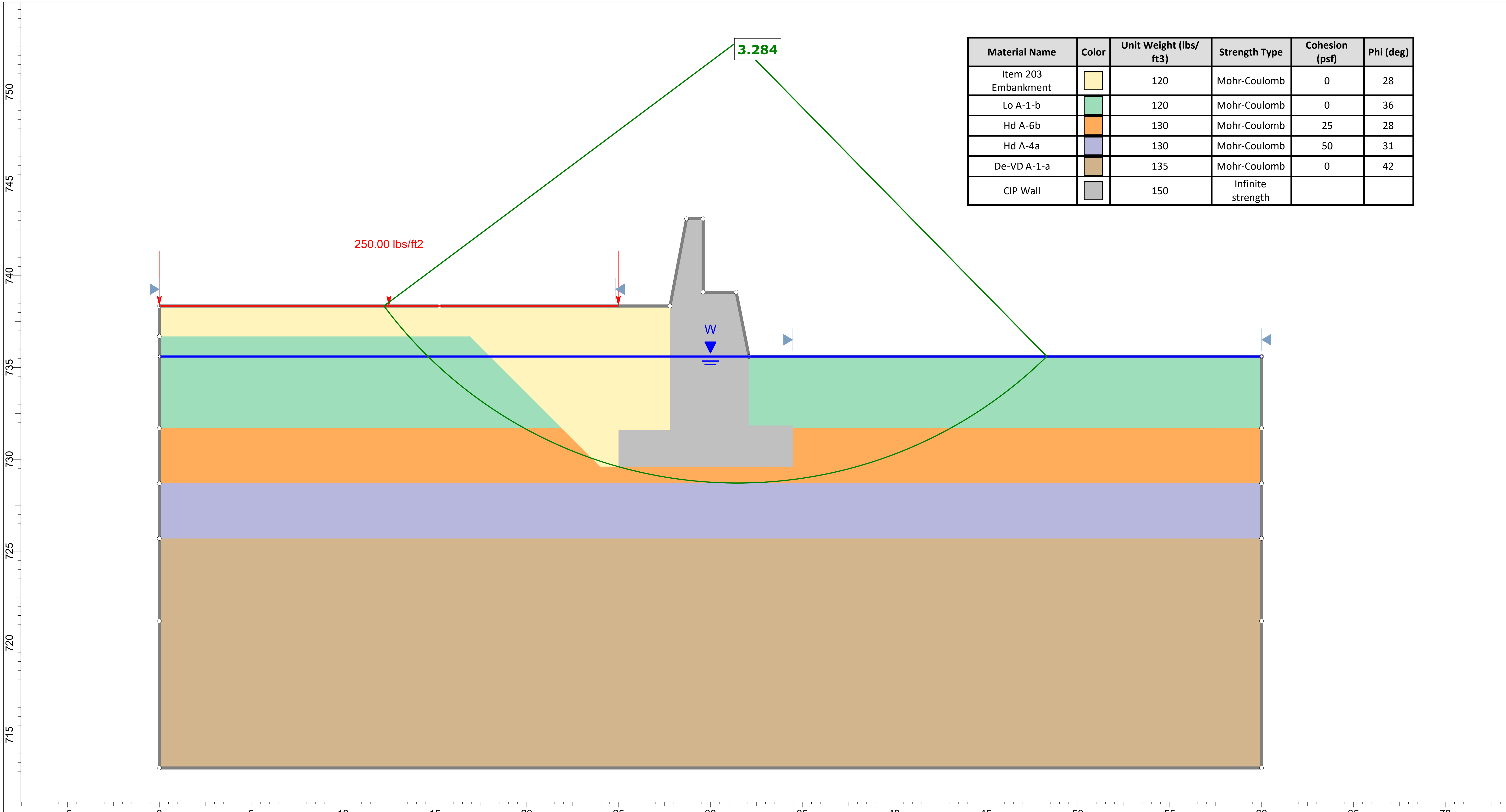
$$\sigma_{vmax} = \frac{2\sum V}{3[(B/2) - e)]} \quad (11.6.3.2\text{-}4)$$

$$\sigma_{vmin} = 0 \quad (11.6.3.2\text{-}5)$$

where the variables are as defined in Figure 11.6.3.2-2.

APPENDIX IX

GLOBAL STABILITY ANALYSIS OUTPUT



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Item 203 Embankment		120	Mohr-Coulomb	0	28
Lo A-1-b		120	Mohr-Coulomb	0	36
Hd A-6b		130	Mohr-Coulomb	25	28
Hd A-4a		130	Mohr-Coulomb	50	31
De-VD A-1-a		135	Mohr-Coulomb	0	42
CIP Wall		150	Infinite strength		

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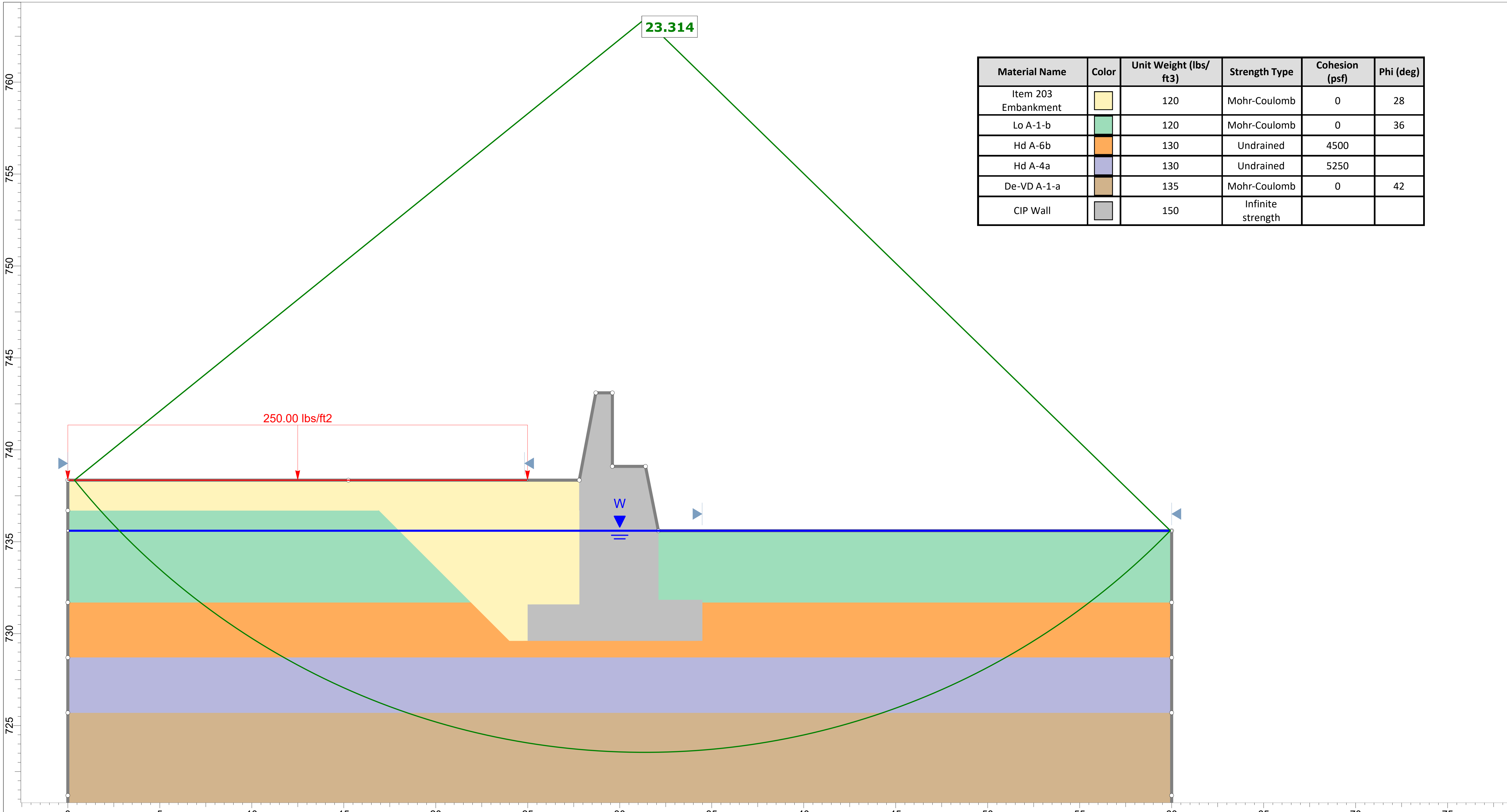
SLIDEINTERPRET 9.019

Project	FRA-70-14.05 Project 4B - PID No. 96053 - Retaining Wall 4W22
Analysis Description	Global Stability Analysis - B-030-0-08 - Drained - Spencer's
Drawn By	BRT
Scale	1:60
Company	Resource International, Inc.
Date	7/9/2022, 4:04:08 PM
File Name	Retaining Wall 4W22 - Global Stability.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Item 203 Embankment		120	Mohr-Coulomb	0	28
Lo A-1-b		120	Mohr-Coulomb	0	36
Hd A-6b		130	Undrained	4500	
Hd A-4a		130	Undrained	5250	
De-VD A-1-a		135	Mohr-Coulomb	0	42
CIP Wall		150	Infinite strength		

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SLIDEINTERPRET 9.019

Project: FRA-70-14.05 Project 4B - PID No. 96053 - Retaining Wall 4W22

Analysis Description: Global Stability Analysis - B-030-0-08 - Undrained - Spencer's

Drawn By: BRT

Scale: 1:60

Company: Resource International, Inc.

Date: 7/9/2022, 4:04:08 PM

File Name: Retaining Wall 4W22 - Global Stability.slmd