Resource International, Inc.

FRA-70-13.11 PROJECT 4A
RETAINING WALL 4W3
PID NO. 77372
FRANKLIN COUNTY, OHIO

STRUCTURE FOUNDATION EXPLORATION REPORT (REV. 2)

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

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RESOURCE INTERNATIONAL, INC.
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Planning
Engineering
Construction Management
Technology

April 7, 2015 (Revised July 8, 2022)

Mr. Christopher W. Luzier, P.E.
Project Manager
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Re: Structure Foundation Exploration Report (Rev. 2)
FRA-70-13.11 Project 4A
Retaining Wall 4W3
PID No. 77372
Rii Project No. W-13-045

Mr. Luzier:

Resource International, Inc. (Rii) is pleased to submit this revised structure foundation exploration report for the above referenced project. Engineering logs have been prepared and are attached to this report, along with the results of laboratory testing. This report includes recommendations for the design and construction of proposed Retaining Wall 4W3 as part of the FRA-70-13.11 Project 4A 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.
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Enclosure: Structure Foundation Exploration Report (Rev. 3)

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EXECUTIVE SUMMARY

Resource International, Inc. (Rii) has completed a structure foundation exploration for retaining wall 4W3 as part of the FRA-70-13.11 Project 4. Based on the proposed plan information provided by GPD GROUP, retaining Wall 4W3 begins at Sta. 187+89.09, 40.00' Lt. (BL I-70 EB) where it abuts the pier substructure of proposed FRA-70-1395C bridge over I-70, and continues to the east to Sta. 191+02.36, 40.00' Lt. (BL I-70 EB) where it abuts the pier substructure of proposed FRA-70-1405C over I-70. The proposed wall measures approximately 309 feet in length, with a proposed stem height above the footing varying from 11.9 feet at the west end to 9.7 feet at the east end of the wall. The wall is proposed to be constructed as cast-in-place (CIP) wall.

Exploration and Findings

On August 7, 2013, one (1) structural boring, designated as B-027-1-13, was drilled as part of the current investigation to completion depth of 49.3 feet below the existing ground surface. In addition, between July 8 and 24, 2008, three (3) borings, designated as B-027-0-08, B-028-0-08, and B-029-0-08, were performed tom completion depths ranging from 10.0 to 136.5 feet below grade by DLZ as part of the FRA-70-8.93 preliminary exploration. Additionally, boring B-001-C-59, performed as part of the 1959 historic exploration, was extended to a depth of 71.0 feet below the existing grade at the time of the exploration in the vicinity of the wall.

Boring B-027-1-13 was performed within the existing roadway of W. Fulton Street on the south side of the proposed retaining wall location, and encountered 2.0 inches of asphalt overlying 8.0 inches of concrete at the existing ground surface. Borings B-027-0-08 and B-029-0-08 were drilled through the existing pavement of the ramp from I-70 eastbound to Fourth Street and Livingston Avenue and encountered 7.0 inches of asphalt overlying 11.0 and 8.0 inches of concrete followed by 11.0 inches of aggregate base, respectively, at the ground surface. Boring B-028-0-08 was performed in the shoulder of I-70 westbound and encountered 12.0 inches of asphalt overlying 6.0 inches of aggregate base at the ground surface.

Beneath the surface materials in borings B-027-0-08 through B-029-0-08, material identified as existing fill or possible fill was encountered extending to depths ranging from 3.0 to 10.0 feet below the ground surface. The fill material consisted of brown and gray gravel and gravel and sand (ODOT A-1-a, A-1-b) and contained brick fragments throughout.

Underlying the surficial materials and existing fill, natural granular soils were encountered in borings B-027-0-08, B-027-1-13 and B-029-0-08, with intermittent seams of cohesive material. The granular soils were generally described as gray gravel, gravel with sand, gravel with sand and silt, fine sand, coarse and fine sand, sandy silt and silt (ODOT A-1-a, A-1-b, A-2-4, A-3, A-3a, A-4a, A-4b). The cohesive materials were described as gray sandy silt, silt, silt and clay and silty clay (ODOT A-4a, A-4b, A-6a, A-6b).

Top of bedrock was encountered in boring B-029-0-08 at a depth of 113.5 feet below existing grade, which corresponds to an elevation of 628.8 feet msl. The upper portion of the bedrock consisted of gray, severely weathered shale overlying dark gray, moderately to highly weathered shale overlying competent limestone bedrock, which was encountered at an elevation of 621.7 feet msl.

The natural soils encountered in boring B-001-C-59 consisted of medium stiff silt and clay (ODOT A-6a) to a depth of 10.0 feet overlying dense to very dense granular soils described as brown and gray gravel, gravel with sand and gravel with sand and silt (ODOT A-1-a, A-1-b, A-2-4). Glacial boulders (igneous and limestone) were noted throughout the granular soil deposits encountered below upper cohesive soil, beginning at elevation 752.4 feet msl and extending to the boring termination depth.

Analyses and Recommendations

Design details of the proposed retaining wall were provided by GPD GROUP. Retaining wall 4W3 begins at the pier substructure of the proposed FRA-70-1395C bridge over I-70 at the west end of the wall alignment and will extend east along the median between I-70 eastbound and westbound, where it abuts the pier substructure of the of the proposed FRA-70-1405C bridge over I-70. Based on plan information provided by GPD GROUP, the footings for retaining wall 4W3 have been designed to produce a maximum service limit bearing pressure of 2.30 ksf and a maximum factored bearing pressure of 3.20 ksf at the strength limit state. The wall is proposed to be constructed as a cast-in-place (CIP) wall.

Based on plan information provided by GPD Group, the foundation for the proposed retaining wall will bear at a minimum depth of 4.0 feet below the existing grade of I-70, at elevations ranging from 721.0 to 725.3 feet msl. At these elevations, the bearing soils are anticipated to consist of dense to very dense gravel and gravel with sand and silt (ODOT A-1-a, A-2-4). Shallow foundations bearing on these competent natural soils may be proportioned for a factored bearing resistance as presented in the following table.

Retaining Wall 4W3 Shallow Foundation Analysis

Effective Footing Width (feet)	Service Limit Bearing Pressure (ksf) [1]			Bearing Resistance (ksf)	
	0.5-inch	1.0-inch	1.5-inch	Nominal	Factored [2]
3.0	3.24	9.80	16.92	27.30	15.01
4.0	2.50	7.48	12.94	27.31	15.02
5.0	2.05	6.10	10.59	27.33	15.03
6.0	1.76	5.19	9.06	27.35	15.04
7.0	1.55	4.55	7.99	27.36	15.05
8.0	1.40	4.07	7.21	27.38	15.06
9.0	1.28	3.70	6.62	27.40	15.07
10.0	1.19	3.40	6.15	27.41	15.08
11.0	1.11	3.16	5.78	27.43	15.09
12.0	1.05	2.96	5.48	27.45	15.10
13.0	1.00	2.80	5.24	27.47	15.11

1. *The service limit bearing pressure was calculated at total settlement values of 0.5, 1.0 and 2.0 inches.*
2. *A resistance factor of φ_b = 0.55 was utilized in calculating the factored nominal bearing resistance at the strength limit state.*

Based on the maximum service limit bearing pressures provided in the design documents, total settlements up to 0.67 inches are anticipated along the alignment of retaining wall 4W3. Additionally, the maximum factored bearing pressure of 3.20 ksf will not exceed the factored bearing resistance at the strength limit of 15.05 ksf.

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

1.0 INTRODUCTION

The overall purpose of this project is to provide detailed subsurface information and recommendations for the design and construction of the FRA-70-12.68/13.11/14.05C (Project 4R/4H/4A) projects in Columbus, Ohio. The projects represent the central portion of FRA-70-8.93 (PID 77369) I-70/71 south innerbelt improvements project. The FRA-70-13.11 (Project 4A) phase will consist of all work associated with the construction of I-70 eastbound, starting just west of the FRA-70-1321R bridge structure over the Scioto River and extending east to just east of High Street. The proposed I-70 eastbound will consist of two-lane mainline travel lanes with shoulders from the beginning of the project limits to just east of the FRA-70-1358R bridge structure, where it will merge with Ramp C5. From this point to just west of High Street, I-70 eastbound will be reconstructed as part of the FRA-70-12.68 Project 4R under a separate contract. Some additional work along I-70 eastbound will be required to the east and west of High Street to complete the final configuration of the highway in this area. This project includes the replacement of two (2) bridge structures along I-70 eastbound as well as the construction of one (1) new retaining wall between Front Street and High Street.

This report is a presentation of the structure foundation exploration performed for proposed retaining wall 4W3 as part of the FRA-70-13.11 Project 4A, as shown on the vicinity map and boring plan presented in Appendix I. Based on the proposed plan information provided by GPD GROUP, retaining Wall 4W3 begins at Sta. 187+89.09, 40.00’ Lt. (BL I-70 EB) where it abuts the pier substructure of proposed FRA-70-1395C bridge over I-70, and continues to the east to Sta. 191+02.36, 40.00’ Lt. (BL I-70 EB) where it abuts the pier substructure of proposed FRA-70-1405C over I-70. The proposed wall measures approximately 309 feet in length, with a proposed wall height varying from 11.9 feet at the west end to 9.7 feet at the east end of the wall. The wall is proposed to be constructed as cast-in-place (CIP) wall.

2.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1 Site Geology

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

According to the bedrock geology and topography maps obtained from the Ohio Department of Natural Resources (ODNR), the underlying bedrock consists predominantly of the Middle to Lower Devonian-aged Columbus Limestone. This formation is further subdivided into two members in the central portion of the state, known as the Delhi and Bellepoint Members. The Delhi Member consists of light gray, finely to coarsely crystalline, irregularly bedded, fossiliferous limestone. The Bellepoint Member consists of variable brown, finely crystalline, massively bedded limy dolomite. Both of these members contain chert nodules. Just east of High Street, the underlying bedrock consists of the Upper Devonian Ohio Shale Formation overlying the Middle Devonian-aged Delaware Limestone Formation. The Ohio Shale formation consists of brownish black to greenish gray, thinly bedded, fissile, carbonaceous shale. The Delaware Limestone consists of bluish gray, thin to medium bedded dolomitic limestone with nodules and layers of chert. Regionally, the bedrock surface forms a broad valley aligned roughly north-to-south beneath the Scioto River. According to bedrock topography mapping, the elevation of the bedrock surface ranges from approximately 600 feet mean sea level (msl) in the valley to approximately 625 feet msl near the project limits. Within the borings performed for this current investigation, shale bedrock was encountered at a depth of 113.5 feet below the ground surface which corresponds to El. 628.8 feet msl.

2.2 Existing Conditions

The proposed retaining wall be situated along the existing median of I-70 between the S. Front Street and S. High Street over I-70/71 bridge structures, approximately 0.7 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 Third Street entrance ramp converges with I-70 westbound at the S. High Street structure crossing, creating a fourth lane beneath the structure, and the existing Fourth Street exit ramp from I-70 eastbound is aligned just south of I-70. The existing I-70 profile is lowered from the surrounding terrain, as the existing corridor was cut approximately 25 to 35 feet below the existing grade of S. High Street and the surrounding downtown area. The 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

On August 7, 2013, one (1) structural boring, designated as B-027-1-13, was drilled as part of the current investigation to completion depth of 49.3 feet below the existing ground surface at the location shown on the boring plan provided in Appendix I of this report and summarized in Table 1 below. In addition, between July 8 and 24, 2008, three (3) borings, designated as B-027-0-08, B-028-0-08, and B-029-0-08, were performed by DLZ as part of the FRA-70-8.93 preliminary exploration and their findings were published in a report dated September 24, 2009. Borings B-027-0-08 and B-028-0-08 were advanced to a completion depth of 14.0 and 10.0 feet below the existing ground surface, respectively, within the existing lanes of I-70 for evaluation of the subgrade for the proposed roadway improvements. Boring B-029-0-08 was advanced to completion depth of 136.5 feet below the existing ground surface within the existing ramp from I-70 eastbound to Fourth Street and Livingston Avenue for evaluation of the proposed retaining walls for the trench widening.

Table 1. Test Boring Summary

Boring Number	Station	Offset	Baseline	Latitude	Longitude	Ground Elevation (feet msl)	Boring Depth (feet)
B-027-0-08	187+70.27	12.1' RT	I-70 EB	39.952864	-83.000428	735.9	14.0
B-027-1-13	189+32.64	78.7’ RT	I-70 EB	39.952672	-82.999847	755.5	49.3
B-028-0-08	191+29.78	14.9' LT	I-70 WB	39.953161	-82.999193	731.7	10.0
B-029-0-08	191+53.21	46.3' RT	I-70 EB	39.952780	-82.999049	742.3	136.5

The location of boring B-027-1-13 was determined and located in the field by Rii representatives. Rii utilized a handheld GPS unit to obtain northing and easting coordinates of the boring location. The ground surface elevation at the boring location was interpolated using topographic mapping information provided by GPD GROUP.

Boring B-027-1-13 was performed with an all-terrain-vehicle (ATV) mounted rotary drilling machine utilizing a 3.25-inch ID, continuous HSA to advance the hole. The borings performed by DLZ 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 testing (SPT) and split-spoon sampling were performed in boring B-027-1-13 was sampled at 2.5-foot increments of depth to 20 feet and at 5.0-foot increments thereafter to the boring termination depth. In borings B-027-0-08 and B-029-0-08, SPT and split spoon sampling were performed continuously to a depth of 9.0 feet in each boring. Below this depth in boring B-027-0-08 and form the ground surface in boring B-028-0-08, SPT and split spoon sampling was performed at 2.5-foot intervals to the boring termination depths. In boring B-029-0-08, below the continuous sampling interval, SPT and split spoon sampling was performed at 2.5-foot increments to a depth of 35 feet and at 5.0-foot increments thereafter to the top of bedrock.

The SPT, per the American Society for Testing and Materials (ASTM) designation D1586, is conducted using a 140-pound hammer falling 30.0 inches to drive a 2.0-inch outside diameter split spoon sampler 18.0 inches. Rii and DLZ utilized a calibrated automatic drop hammer to generate consistent energy transfer to the sampler. Driving resistance is recorded on the boring logs in terms of blow per 6.0-inch interval of the driving distance. The second and third intervals are added to obtain the number of blows per foot (N). Standard penetration blow counts aid in determining soil properties applicable in foundation system design. Measured blow count (N) values are corrected to an equivalent (60%) energy ratio, N_{60}, by the following equation. Both values are represented on project 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 ATV-mounted drill rig operated by Rii was calibrated on April 26, 2013, and has a drill rod energy ratio of 82.6 percent. The hammer for the CME drill rig operated by DLZ has a drill rod energy ratio of 61.2 percent. No calibration date was provided on the boring logs. No calibration factor was applied to the blow counts presented on the historic boring logs, as these were performed using a manual hammer.

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.

During drilling for the borings, a field log was prepared for boring B-027-1-13 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 were visually classified and select samples were tested, as noted in Table 2.

Table 2. Laboratory Test Schedule

Laboratory Test	Test Designation	Number of Tests Performed
Natural Moisture Content	ASTM D 2216	14
Plastic and Liquid Limits	AASHTO T89, T90	4
Gradation – Sieve/Hydrometer	AASHTO T88	4

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.

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

Where borings were extended into the competent bedrock (after encountering auger refusal), an NQ double-tube diamond bit core barrel (utilizing wire line equipment) was used to core the bedrock. Coring produced 1.85 inch diameter cores from which the type of rock and its geological characteristics were determined.

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

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

In addition to the borings performed for the current exploration, historic borings performed in 1959 by the Department of Highways as part of the FRA-40-12.82 project were obtained from the construction documents on record. One (1) boring, designated as B-001-C-59, was present in the vicinity of the wall. The boring were extended to a depth of 71.0 feet below the existing grade at the time of the exploration. Please note that the elevation provided on the historic boring log is referenced to the North American Datum (NAD) 27. The current design survey is referenced to NAD 83. The NAD 27 datum is 0.6 feet lower than the NAD 83 datum. **Therefore, all elevations noted in this report with respect to the historic boring are adjusted to the current NAD 83 datum.** The historic boring location is shown on the boring plan provided in Appendix I, and the historic boring log is provided in Appendix IV.

4.0 FINDINGS

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

4.1 Surface Materials

Boring B-027-1-13 was performed within the existing roadway of W. Fulton Street on the south side of the proposed retaining wall location, and encountered 2.0 inches of asphalt overlying 8.0 inches of concrete at the existing ground surface. Borings B-027-0-08 and B-029-0-08 were drilled through the existing pavement of the ramp from I-70 eastbound to Fourth Street and Livingston Avenue and encountered 7.0 inches of asphalt overlying 11.0 and 8.0 inches of concrete followed by 11.0 inches of aggregate base, respectively, at the ground surface. Boring B-028-0-08 was performed in the shoulder of I-70 westbound and encountered 12.0 inches of asphalt overlying 6.0 inches of aggregate base at the ground surface. Surface materials were not noted on the historic boring logs.

4.2 Subsurface Soils

Beneath the surface materials in borings B-027-0-08 through B-029-0-08, material identified as existing fill or possible fill was encountered extending to depths ranging from 3.0 to 10.0 feet below the ground surface. The fill material consisted of brown and gray gravel and gravel and sand (ODOT A-1-a, A-1-b) and contained brick fragments throughout.

Underlying the surficial materials and existing fill, natural granular soils were encountered in borings B-027-0-08, B-027-1-13 and B-029-0-08, with intermittent seams of cohesive material. The granular soils were generally described as gray gravel, gravel with sand, gravel with sand and silt, fine sand, coarse and fine sand, sandy silt and silt (ODOT A-1-a, A-1-b, A-2-4, A-3, A-3a, A-4a, A-4b). The cohesive materials were described as 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 loose ($5 \leq N_{60} < 10$ blows per foot [bpf]) to very dense ($N_{60} > 50$ bpf). Overall blow counts recorded from the SPT sampling ranged from 7 bpf to split spoon sampler refusal. The shear strength and consistency of the cohesive soils are primarily derived from the hand penetrometer values (HP). The cohesive soil encountered ranged from stiff ($1.0 < HP \leq 2.0$ tsf) to hard ($HP > 4.0$ tsf). The unconfined compressive strength of the cohesive soil samples tested, obtained from the hand penetrometer, ranged from 1.5 to over 4.5 tsf (limit of instrument).

Natural moisture contents of the soil samples tested ranged from 5 to 17 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. In general, the soil exhibited natural moisture contents considered to be significantly below to near optimum moisture levels.

4.3 Bedrock

Bedrock was encountered in boring B-029-0-08 as presented in Table 3.

Table 3. Top of Bedrock Elevations

Boring Number	Ground Surface Elevation (feet msl)	Top of Bedrock (Sampler Refusal)		Top of Bedrock Core (Auger Refusal)	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-029-0-08	742.3	113.5	628.8	116.5	625.8

Top of bedrock was encountered in boring B-029-0-08 at a depth of 113.5 feet below existing grade, which corresponds to an elevation of 628.8 feet msl. The upper portion of the bedrock consisted of gray, severely weathered shale overlying dark gray, moderately to highly weathered shale overlying competent limestone bedrock, which was encountered at an elevation of 621.7 feet msl. The cored shale is described as dark gray, moderately to highly weathered, weak, thinly laminated, calcareous, pyritic, fissile, friable, jointed and fractured with tight, slightly rough apertures. The limestone is described as brownish gray to light gray, slightly to moderately weathered, moderately strong to strong, very thin to thin bedded, fossiliferous, stylolitic, cherty, pyritic and slightly to moderately fractured with tight to open, slightly rough apertures.

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

Table 4. Rock Core Summary

Boring	Core No.	Elevation (feet msl)	Recovery (%)	RQD (%)	Unconfined Compressive Strength
B-029-0-08	R-1	625.8 to 622.3	90	0	N/A
	R-2	622.3 to 617.3	100	88	q_u @ 121.2' = 14,137 psi
	R-3	617.3 to 612.3	100	85	N/A
	R-4	612.3 to 607.3	90	76	N/A
	R-5	607.3 to 605.8	100	100	N/A

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 very poor (RQD ≤ 25%), and the quality of the limestone bedrock was good (75 < RQD ≤ 90%) to excellent (RQD > 90%).

4.4 Groundwater

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

Table 5. Groundwater

Boring Number	Ground Elevation (feet msl)	Initial Groundwater		Upon Completion	
		Depth (feet)	Elevation (feet msl)	Depth (feet)	Elevation (feet msl)
B-027-0-08	735.9	5.0	730.9	Dry	-
B-027-1-13	755.5	37.0	718.5	N/A [1]	-
B-028-0-08	731.7	Dry	-	Dry	-
B-029-0-08	742.3	21.0	721.3	20.5 [2]	721.8

1. *The groundwater level at completion could was not obtained.*
2. *The groundwater level at completion was measured after the rock coring process, which included the addition of water for coring.*

Groundwater was not encountered during or at the completion of drilling in boring B-028-0-08. Groundwater was encountered initially during the drilling process in borings B-027-0-08, B-027-1-13 and B-029-0-08 at a depth of 5.0, 37.0 and 21.0 feet below the existing ground surface, respectively, which corresponds to an elevation of 730.9, 718.5 and 721.3 feet msl. Boring B-027-0-08 was observed to be dry at the completion of drilling. No groundwater reading was obtained in boring B-027-1-13 at the completion of drilling. The groundwater level at the completion of drilling in boring B-029-0-08 was 20.5 feet below existing grade following the rock coring process. Additionally, DLZ noted that they frequently added water to the borehole to clean out the augers after encountering sand heave of varying amounts at various depths.

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.

4.5 Historic Boring

The natural soils encountered in boring B-001-C-59 consisted of medium stiff silt and clay (ODOT A-6a) to a depth of 10.0 feet overlying dense to very dense granular soils described as brown and gray gravel, gravel with sand and gravel with sand and silt (ODOT A-1-a, A-1-b, A-2-4). Glacial boulders (igneous and limestone) were noted throughout the granular soil deposits encountered below upper cohesive soil, beginning at elevation 752.4 feet msl and extending to the boring termination depth. Bedrock was not encountered in the historic boring prior to the termination depth. Groundwater levels were not noted on the historic boring log. In general, the subsurface conditions encountered in the historic boring matched relatively closely with the subsurface conditions encountered in the current and preliminary engineering exploration borings.

5.0 ANALYSES AND RECOMMENDATIONS

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

Design details of the proposed retaining wall were provided by GPD GROUP. Retaining wall 4W3 begins at the pier substructure of the proposed FRA-70-1395C bridge over I-70 at the west end of the wall alignment and will extend east along the median between I-70 eastbound and westbound, where it abuts the pier substructure of the of the proposed FRA-70-1405C bridge over I-70. Based on plan information provided by GPD GROUP, the footings for retaining wall 4W3 have been designed to produce a maximum service limit bearing pressure of 2.30 ksf and a maximum factored bearing pressure of 3.20 ksf at the strength limit state. The wall is proposed to be constructed as a cast-in-place (CIP) wall.

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 Shallow Foundation Recommendations

Based on plan information provided by GPD Group, the foundation for the proposed retaining wall will bear at a minimum depth of 4.0 feet below the existing grade of I-70, at elevations ranging from 721.0 to 725.3 feet msl. At these elevations, the bearing soils are anticipated to consist of dense to very dense gravel and gravel with sand and silt (ODOT A-1-a, A-2-4). Shallow foundations bearing on these competent natural soils may be proportioned for a factored bearing resistance as presented in Table 6. Based on correspondence with GPD GROUP, it is understood that the external stability calculations 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 6. Retaining Wall 4W3 Shallow Foundation Analysis

Effective Footing Width (feet)	Service Limit Bearing Pressure (ksf) [1]			Bearing Resistance (ksf)	
	0.5-inch	1.0-inch	1.5-inch	Nominal	Factored [2]
3.0	3.24	9.80	16.92	27.30	15.01
4.0	2.50	7.48	12.94	27.31	15.02
5.0	2.05	6.10	10.59	27.33	15.03
6.0	1.76	5.19	9.06	27.35	15.04
7.0	1.55	4.55	7.99	27.36	15.05
8.0	1.40	4.07	7.21	27.38	15.06
9.0	1.28	3.70	6.62	27.40	15.07
10.0	1.19	3.40	6.15	27.41	15.08
11.0	1.11	3.16	5.78	27.43	15.09
12.0	1.05	2.96	5.48	27.45	15.10
13.0	1.00	2.80	5.24	27.47	15.11

1. *The service limit bearing pressure was calculated at total settlement values of 0.5, 1.0 and 1.5 inches.*
2. *A 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 6. 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 6, 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 for the total settlement values considered in the analysis. A graphical representation of the service limit bearing pressures and factored bearing resistance at the strength limit state is presented in Appendix V.

Based on the maximum service limit bearing pressures provided in the design documents and noted in Section 5.0, total settlements up to 0.67 inches are anticipated along the alignment of retaining wall 4W3. Additionally, the maximum factored bearing pressure of 3.20 ksf will not exceed the factored bearing resistance at the strength limit of 15.05 ksf. Calculations for settlement and nominal and factored bearing resistance for the shallow spread foundations are provided in Appendix VI.

5.1.1 Sliding Resistance

The resistance of the footings to sliding will be dependent on the friction between the concrete footing and bearing surface. For concrete footings that rest on cohesionless soil, a coefficient "f" of 0.70 times the total vertical force on the base should be taken as the sliding 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.

5.1.2 Overall (Global) Stability

A slope stability analysis was performed to check the global stability of the walls along the alignment. As per the AASHTO LRFD BDS, safety against global stability failure shall be evaluated at the service limit state. Soil parameters utilized in external stability analyses are presented in Table 7. For the global stability condition, it was considered that the failure plane will not cross through any portion of the supported soil mass above the concrete or through the concrete footing itself.

Table 7. 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)
Hard Sandy Silt ODOT A-4a	125	31	0	4,000
Dense to Very Dense Granular Soils	130 to 135	33 to 35	0	N/A

Per Section 11.6.2.3 of the AASHTO LRFD BDS, overall (global) stability for CIP walls 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 1.3 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) along the alignment was greater than 1.3. Graphical outputs for overall (global) stability of the wall are provided in Appendix VII.

5.2 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 8 and Table 9.

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

Soil Type	γ (pcf) [1]	c (psf)	φ	k_a	k_o	k_p
Soft to Stiff Cohesive Soil	115	1,500	0°	N/A	N/A	N/A
Very Stiff to Hard Cohesive Soil	125	3,000	0°	N/A	N/A	N/A
Loose Granular Soil	120	0	28°	0.32	0.53	5.07
Medium Dense Granular Soil	125	0	32°	0.27	0.47	6.82
Dense to Very Dense Granular Soil	130	0	36°	0.23	0.41	9.09
Compacted Cohesive Engineered Fill	120	2,000	0°	N/A	N/A	N/A
Compacted Granular Engineered Fill	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 9. Estimated Drained (Long-term) Soil Parameters for Design

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

5.2.1 Excavation Considerations

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

Table 10. Excavation Back Slopes

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

5.3 Groundwater Considerations

Based on the groundwater observations made during drilling, groundwater may be encountered during excavation of the foundation along the wall alignment. Where/if groundwater is encountered, proper groundwater control should be employed and maintained to prevent disturbance to excavation bottoms consisting of cohesive soil, and to prevent the possible development of a quick or "boiling" condition where soft silts and/or fine sands are encountered. It is preferable that the groundwater level, if encountered, be maintained at least 36 inches below the deepest excavation. Any seepage or groundwater encountered at this site should be able to be controlled by pumping from temporary sumps. Additional measures may be required depending on seasonal fluctuations of the groundwater level. Note that determining and maintaining actual groundwater levels during construction is the responsibility of the contractor.

6.0 LIMITATIONS OF STUDY

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

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

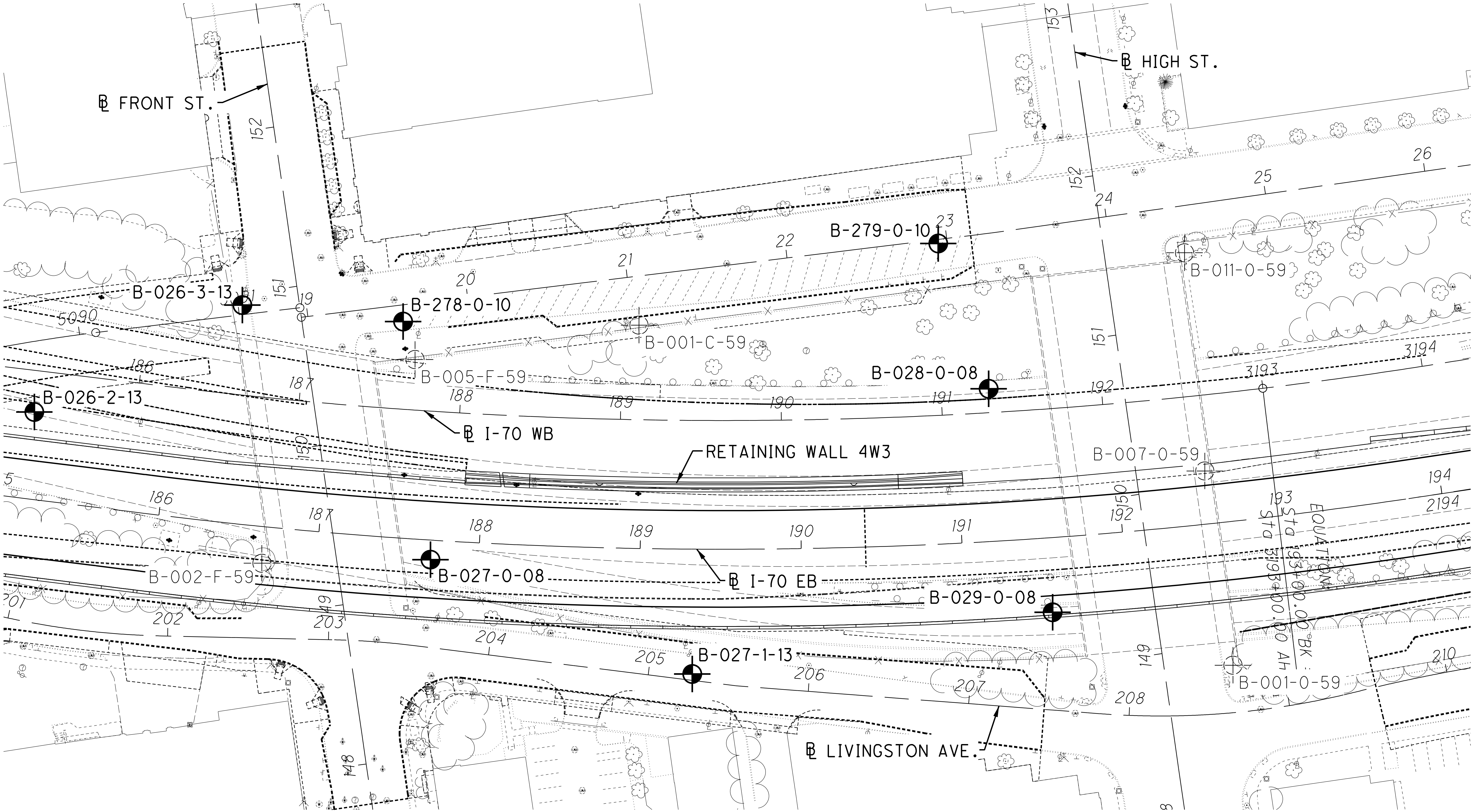
The conclusions and recommendations herein have been based upon the available soil and bedrock information and the design details furnished by a representative of the owner of the proposed project. Any revision in the plans for the proposed construction from those anticipated in this report should be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

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

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

APPENDIX I

VICINITY MAP AND BORING PLAN



BORING PLAN

FRA-70-13.11 PROJECT 4A - RETAINING WALL 4W3

FRANKLIN COUNTY, OHIO

RII PROJECT NO.
W-13-045

SCALE: 1"=60'
0 30 60

N

DRAWN
RRM

REVIEWED
BRT

DATE
6/26/2020

RESOURCE INTERNATIONAL, INC.

APPENDIX II

DESCRIPTION OF SOIL TERMS

DESCRIPTION OF SOIL TERMS

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

CLASSIFICATION OF SOILS

Ohio Department of Transportation

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

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

MATERIAL CLASSIFIED BY VISUAL INSPECTION

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

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

DESCRIPTION OF ROCK TERMS

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

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

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

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

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

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

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

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

Degree of Fracturing

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

Aperture Width

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

Surface Roughness

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

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

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

APPENDIX III

PROJECT BORING LOGS:

B-027-0-08, B-027-1-13, B-028-0-08
and B-029-0-08

BORING LOGS

Definitions of Abbreviations

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

Classification Test Data

Gradation (as defined on Description of Soil Terms):

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

Atterberg Limits:

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

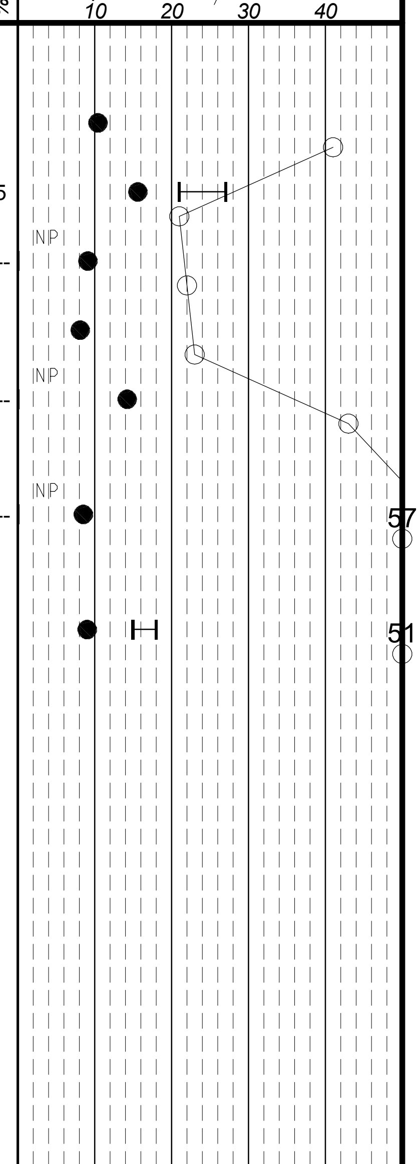
Client: ms consultants	Project: FRA-70-8.93	Job No. 0221-1004.01
LOG OF: Boring B-027-0-08	Location: Sta. 187+70.27, 12.1' RT., BL I-70 EB	Date Drilled: 7/8/2008

WATER OBSERVATIONS:
Water seepage at: 5.0'
Water level at completion: None

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

Depth (ft)	Elev. (ft) 735.9	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
							Asphalt Concrete Pavement - 7" Portland Cement Concrete - 11" Aggregate Base - 11"						
2.4	733.5	16 23 17	18	1									
3.0	732.9						FILL: Brick fragments, little fine to coarse sand; damp.						
		3 8 12	18	2			Medium dense to dense brown GRAVEL WITH SAND (A-1-b), little to some silt; damp.	19	34	---	25	17	5
		13 12 9	18	3				39	32	---	10	--19--	
		8 6 16	14	4									
		9 14 28	15	5			@ 7.5'-9.0', contains rust stains.	28	34	---	15	--23--	
10.0	725.9												
		13 26 29	13	6			Dense to very dense brown SANDY SILT (A-4a), trace to little gravel; moist.	17	20	---	25	--38--	
14.0	721.9	9 27 22	12	7									
							Bottom of Boring - 14.0'						

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



RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-70-12.68 - PHASE 4A TYPE: STRUCTURE PID: 77372 BR ID: N/A START: 8/7/13 END: 8/7/13	DRILLING FIRM / OPERATOR: RII / S.M. SAMPLING FIRM / LOGGER: RII / K.S. DRILLING METHOD: 3.25" HSA SAMPLING METHOD: SPT	DRILL RIG: CME-750 (SN 98048) HAMMER: CME AUTOMATIC CALIBRATION DATE: 4/26/13 ENERGY RATIO (%): 82.6	STATION / OFFSET: 189+32.64 / 78.7' RT ALIGNMENT: BL I-70 EB ELEVATION: 755.5 (MSL) EOB: 49.3 ft. LAT / LONG: 39.952671524, -82.999846757	EXPLORATION ID **B-027-1-13** PAGE 1 OF 2

MATERIAL DESCRIPTION AND NOTES	ELEV. 755.5	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
0.2' - ASPHALT (2.0")	755.3																	
0.7' - CONCRETE (8.0")	754.7																	
VERY DENSE, BROWN **SANDY SILT**, SOME FINE GRAVEL, TRACE CLAY, DAMP.	752.5	1–2	12 36 18	74	33	SS-1	-	-	-	-	-	-	-	-	-	5	A-4a (V)	
MEDIUM STIFF TO STIFF, BROWN **SILT AND CLAY**, SOME COARSE TO FINE SAND, LITTLE FINE GRAVEL, DAMP TO MOIST.		4–5	4 2 3	7	72	SS-2	1.75	18	19	15	19	29	33	18	15	13	A-6a (4)	
-COBBLES PRESENT @ 8.0'	747.5	6–7	4 4 6	14	44	SS-3	1.50	-	-	-	-	-	-	-	-	17	A-6a (V)	
MEDIUM DENSE TO VERY DENSE, BROWN **GRAVEL AND SAND**, TRACE SILT, TRACE CLAY, DAMP TO MOIST. -LIMESTONE FRAGMENTS PRESENT IN SS-4		9–10	15 11 18	40	44	SS-4	-	-	-	-	-	-	-	-	-	6	A-1-b (V)	
		11–12	8 7 5	17	69	SS-5	-	-	-	-	-	-	-	-	-	14	A-1-b (V)	
		14–15	8 9 10	26	72	SS-6	-	52	23	9	8	8	21	18	3	7	A-1-b (0)	
		16–17	8 32 28	83	44	SS-7	-	-	-	-	-	-	-	-	-	6	A-1-b (V)	
		19–20	14 13 10	32	72	SS-8	-	-	-	-	-	-	-	-	-	7	A-1-b (V)	
	728.5	24–25	5 5 8	18	67	SS-9	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
VERY DENSE, GRAY **GRAVEL WITH SAND AND SILT**, LITTLE CLAY, DAMP.		29	6 23 32	76	100	SS-10	-	24	25	16	16	19	20	13	7	8	A-2-4 (0)	

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

PID: 77372	BR ID: N/A	PROJECT: FRA-70-12.68 - PHASE 4A	STATION / OFFSET: 189+32.64 / 78.7 RT	START: 8/7/13	END: 8/7/13	PG 2 OF 2	B-027-1-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 725.5	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
VERY DENSE, GRAY **GRAVEL WITH SAND AND SILT**, LITTLE CLAY, DAMP. *(same as above)*	723.5	31 – 32																
DENSE TO VERY DENSE, GRAY **GRAVEL AND SAND**, TRACE CLAY, TRACE SILT, MOIST.		33																
		34 – 35	7 18 15	45	100	SS-11	-	-	-	-	-	-	-	-	-	13	A-1-b (V)	
		36 – 37 (W) – 38																
		39 – 40	7 18 18	50	100	SS-12	-	15	35	37	6	7	NP	NP	NP	16	A-1-b (0)	
-COBBLES PRESENT @ 41.0'		41 – 43																
		44 – 45	8 49 50/5"	-	88	SS-13	-	-	-	-	-	-	-	-	-	8	A-1-b (V)	
		46 – 48																
	706.2	49 – EOB	47 50/3"	-	100	SS-14	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	

NOTES: GROUNDWATER INITIALLY ENCOUNTERED @ 37.0'

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

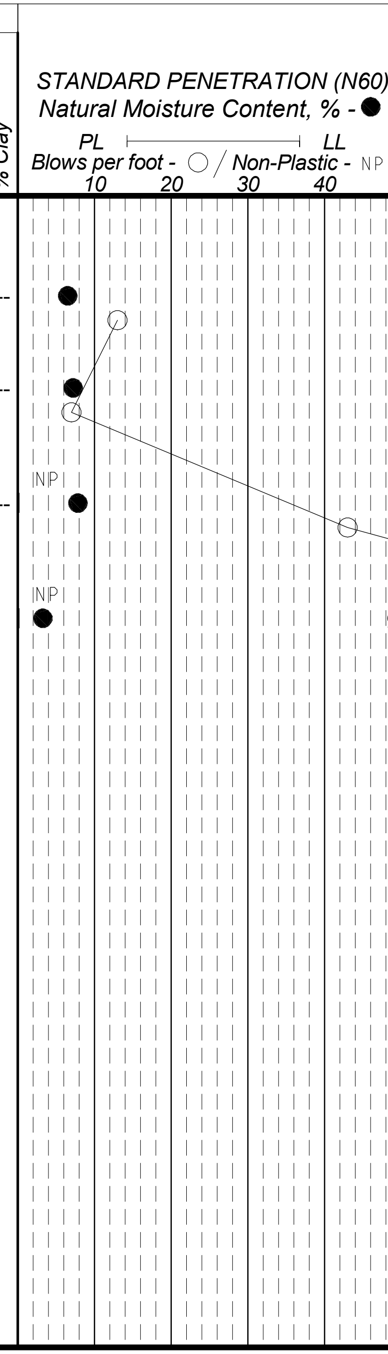
Client: ms consultants | Project: FRA-70-8.93 | Job No. 0221-1004.01

LOG OF: Boring B-028-0-08 | Location: Sta. 191+29.78, 14.9' LT., BL I-70 WB | Date Drilled: 7/24/2008

WATER OBSERVATIONS:
Water seepage at: None
Water level at completion: None

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

Depth (ft)	Elev. (ft) 731.7	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
1.5	730.2						Asphalt Concrete - 12" Aggregate Base - 6"						
		8 6 7	8	1			FILL: Loose to medium dense brown GRAVEL (A-1-a), little to some fine to coarse sand, little silty clay; moist.	65	16	---	6	--13--	
6.0	725.7	4 5 2	7	2				70	14	---	5	--11--	
8.5	723.2	10 20 22	16	3			FILL: Dense brown GRAVEL WITH SAND (A-1-b), little silt; contains brick fragments and silty clay seams; damp.	34	35	---	14	--17--	
10.0	721.7	47 49 46	12	4			FILL: Very dense gray GRAVEL (A-1-a), trace fine to coarse sand; contains few silty clay seams; damp.	84	5	---	3	--8--	
							Bottom of Boring - 10.0'						



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

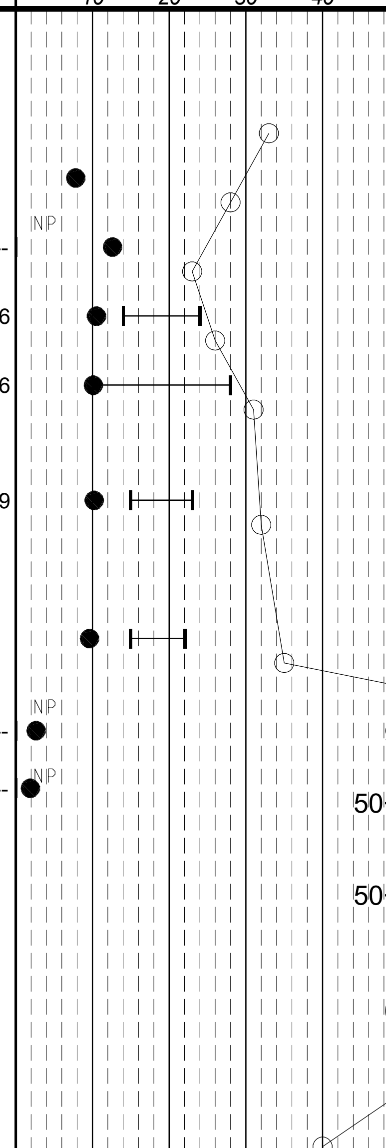
LOG OF: Boring B-029-0-08 | *Location:* Sta. 191+53.21, 46.3' RT., BL I-70 EB | *Date Drilled:* 7/9/2008 to 7/14/2008

WATER OBSERVATIONS:
Water seepage at: 21.0'-25.0',30.0'-32.0',40.0'-110.0'
Water level at completion: 29.4' (beginning of shift, 7/10/08) 20.5' (includes drilling water)

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

Depth (ft)	Elev. (ft) 742.3	Blows per 6"	Recovery	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
							Asphalt Concrete Pavement - 7" Portland Cement Concrete - 9" Aggregate Base - 11"						
2.3	740.0	5 15 17	7	1			POSSIBLE FILL: Medium dense to dense brown GRAVEL WITH SAND (A-1-b), little to some silt; damp.						
		13 15 12	10	2									
		11 13 10	6	3				55	17	---	9	--19--	
6.0	736.3	4 10 15	14	4		4.5	Hard gray SANDY SILT (A-4a), some fine to coarse sand, little gravel; damp.	12	11	---	16	35	26
7.5	734.8	12 15 15	12	5		4.5	Hard gray SILTY CLAY (A-6b), some fine to coarse sand, trace gravel; damp to moist.	9	9	---	15	41	26
10.0	732.3	10 14 17	18	6		4.5	Hard gray SANDY SILT (A-4a), some fine to coarse sand, trace gravel; damp. @ 11.5'-21.0', difficult drilling.	8	8	---	15	40	29
		8 15 19	18	7		4.5							
15.0	727.3	15 48 45	12	8			Very dense gray GRAVEL (A-1-a), some fine to coarse sand, trace to little silt; damp.	55	24	---	10	--11--	
		29 50/3	8	9				61	16	---	12	--11--	
		50/5	1	10			@ 18.5'-18.9', rock fragments; possible cobble blocking shoe.						
21.0	721.3	21 28 33	6	11			Very dense gray GRAVEL WITH SAND AND SILT (A-2-4), some silt; wet.						
23.5	718.8	11 18 21	16	12			Dense gray SILT (A-4b), little fine sand; contains interbedded sand seams; wet.						
25	717.3												

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



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-029-0-08 | *Location:* Sta. 191+53.21, 46.3' RT., BL I-70 EB | *Date Drilled:* 7/9/2008 to 7/14/2008

WATER OBSERVATIONS:
Water seepage at: 21.0'-25.0',30.0'-32.0',40.0'-110.0'
Water level at completion: 29.4' (beginning of shift, 7/10/08)
20.5' (includes drilling water)

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

Depth (ft)	Elev. (ft) 717.3	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)
		10 19 25	15	13		4.5+	Hard gray SANDY SILT (A-4a), "and" fine to coarse sand, trace gravel; contains interbedded sand seams; damp.	9	8	---	33	34	16	
28.5	713.8	13 23 31	13	14			Dense to very dense gray COARSE AND FINE SAND (A-3a), some silt; moist.							55
31.8	710.5	3 6 29	14	15A 15B		4.5+	Hard gray SANDY SILT (A-4a), some gravel, little to some fine to coarse sand; damp. @ 31.0'-43.5', difficult drilling.							
		13 26 31	9	16		--	@ 33.5', 5 inches sand heave.							58
		14 23 33	14	17		4.5+		28	15	---	19	25	13	57
42.0	700.3						Dense gray COARSE AND FINE SAND (A-3a), little silt; contains silty clay seams; wet.							
44.2	698.1	10 20 29	18	18A 18B			Dense gray SANDY SILT (A-4a), some fine to coarse sand, trace gravel; moist.							
47.0	695.3						Very dense gray GRAVEL WITH SAND (A-1-b), trace silt; wet.							
50	692.3	19 37 50/3	15	19			@ 48.5', 6 inches sand heave.	21	48	---	27	--4--		NP 50+

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-029-0-08 | *Location:* Sta. 191+53.21, 46.3' RT., BL I-70 EB | *Date Drilled:* 7/9/2008 to 7/14/2008

WATER OBSERVATIONS:
Water seepage at: 21.0'-25.0',30.0'-32.0',40.0'-110.0'
Water level at completion: 29.4' (beginning of shift, 7/10/08)
20.5' (includes drilling water)

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

Depth (ft)	Elev. (ft) 692.3	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 GRAVEL WITH SAND (A-1-b), trace silt; wet.							
55		10, 16, 50/5	13	20			@ 53.5', 1.7 feet sand heave; washed out.							50+
57.0	685.3						Very dense gray COARSE AND FINE SAND (A-3a), little to some silt, trace to little gravel; wet.							
60		31, 50/5	11	21			@ 58.5'-68.5', three to six inches sand heave.							50+
65		21, 50/5	11	22				18	13	---	52	--17--		50+; NP; moisture ●
70		13, 29, 41	14	23										71
73.8	668.5						Very stiff gray SILT (A-4b), little fine sand; moist.							
75	667.3	25, 38, 50/6	18	24A, 24B		3.75								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-029-0-08	Location: Sta. 191+53.21, 46.3' RT., BL I-70 EB	Date Drilled: 7/9/2008 to 7/14/2008

WATER OBSERVATIONS:
Water seepage at: 21.0'-25.0',30.0'-32.0',40.0'-110.0'
Water level at completion: 29.4' (beginning of shift, 7/10/08)
20.5' (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	STANDARD PENETRATION (N60)
	667.3						Stiff gray SILT AND CLAY (A-6a), some to "and" fine to coarse sand, trace to little gravel; damp to moist.							
		50/4	4	25		1.5		25	20	---	18	26	11	50+
84.0	658.3	31 31 50/3	15	26A 26B		4.5+	Very dense gray GRAVEL WITH SAND (A-1-b), trace silt; wet.							50+
		15 50/4	10	27										50+
		13 26 30	18	28				31	48	---	18	--3--		NP 57
		41 50/2	8	29			@ 98.5', 1.2 feet sand heave.							50+
100	642.3													

Depth scale: 80, 85, 90, 95, 100

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-029-0-08 | *Location:* Sta. 191+53.21, 46.3' RT., BL I-70 EB | *Date Drilled:* 7/9/2008 to 7/14/2008

WATER OBSERVATIONS:
Water seepage at: 21.0'-25.0',30.0'-32.0',40.0'-110.0'
Water level at completion: 29.4' (beginning of shift, 7/10/08) 20.5' (includes drilling water)

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

Depth (ft)	Elev. (ft) 642.3	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, % - ● / PL – LL / Blows per foot - ○ / Non-Plastic - NP
							Very dense gray FINE SAND (A-3), little coarse sand, trace gravel; wet.							
		2 12 49	18	30			@ 103.5', 2.3 feet sand heave.	3	13	---	76	--8--		NP, 62
105														
		41 50/0	6	31										50+
110														
							@ 113.5', 22 feet sand heave; washed out.							
113.5	628.8	50/2	2	32			Severely weathered gray SHALE.							50+
115														
116.5	625.8	Core 42"	Rec 38"	RQD 0%	R-1		Shale, dark gray, moderately to highly weathered, weak, thinly laminated, calcareous, pyritic, fissile, friable, jointed, fractured, tight, slightly rough; RQD 0%, Loss 10%.							
120														
120.6	621.7	Core 60"	Rec 60"	RQD 88%	R-2		Limestone, brownish-gray, moderately weathered, moderately strong to strong, very thinly bedded, pyritic, cherty, moderately fractured, tight, slightly rough, RQD 86%, Loss 0%. [Delaware Limestone] @ 121.2' - 121.5', qu = 14,137 psi							
125	617.3													

Client: ms consultants	Project: FRA-70-8.93	Job No. 0221-1004.01
LOG OF: Boring B-029-0-08	Location: Sta. 191+53.21, 46.3' RT., BL I-70 EB	Date Drilled: 7/9/2008 to 7/14/2008

WATER OBSERVATIONS:
Water seepage at: 21.0'-25.0',30.0'-32.0',40.0'-110.0'
Water level at completion: 29.4' (beginning of shift, 7/10/08) 20.5' (includes drilling water)
FIELD NOTES:
Advanced boring using 3.25" diameter hollowstem augers.

Depth (ft)	Elev. (ft) 617.3	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
		Core 60"	Rec 60"	RQD 85%	R-3		Limestone, brownish-gray, moderately weathered, moderately strong to strong, very thinly bedded, pyritic, cherty, moderately fractured, tight, slightly rough, RQD 86%, Loss 0%. [Delaware Limestone]								
130.2	612.1	Core 60"	Rec 54"	RQD 76%	R-4		Limestone, light gray, slightly weathered, strong, thinly bedded, slightly fossiliferous, stylolitic, slightly fractured, tight to narrow, slightly rough, RQD 82%, Loss 8%. [Columbus Limestone]								
		Core 18"	Rec 18"	RQD 100%	R-5										
136.5	605.8						Bottom of Boring - 136.5'								

Depth scale: 130, 135, 140, 145, 150

APPENDIX IV

HISTORIC BORING LOG:

B-001-C-59

B-001-C-59

STATE OF OHIO
DEPARTMENT OF HIGHWAYS
TESTING LABORATORY

SHEET 3

LOG OF BORING

CO., RT. NO., SEC. FRA-40-12.82 BRIDGE NO.
RETAINING WALL C RETAINING WALL C

LOCATION: T.H. 1B STA. 62+75 OFFSET 102'LT FED. NO.

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
763.0	0			
	2			
	4			
758.0	6	3/4	29107	Brown Sandy Gravelly Clay
	8			
753.0	10			
		26/32	29108	Brown Silty Sandy Gravel
	12			
	14			
748.0	16	-----		Brown Silty Sandy Gravel
	18			
743.0	20			
	22	96/90	29109	Brown Silty Sandy Gravel
	24			
738.0	26	33/47	29110	Gray Gravel
735.5	28			
		24/16		
733.0	30			
	32	37/36	29111	Brown Silty Sandy Gravel
	34			
728.0	36	27/18	29112	Brown Sandy Gravel

Glacial Boulders (igneous and limestone)

BM
8-24-59

B-001-C-59

LOG OF BORING (CONTINUED)

SHEET 4

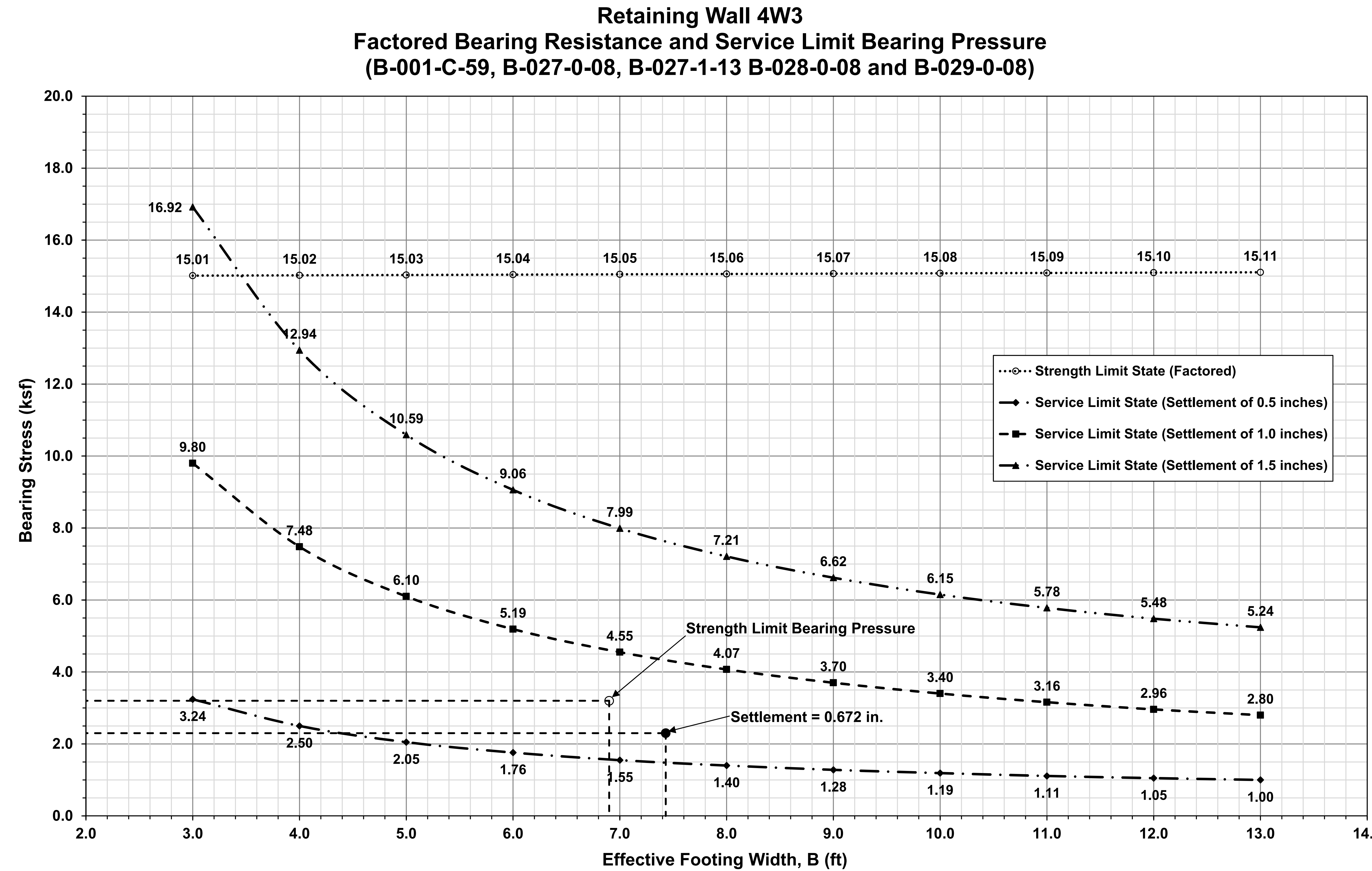
BRIDGE NO. RETAINING WALL C T.H. 1B

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
725.5	38	34/34	29113	Brown Sandy Gravel
723.0	40	21/22	29114	Gray Silty Sand
720.5	42, 44	40/76	29115	Gray Silty Sand
718.0	46	28/42	29116	Gray Silty Gravelly Sand
716.5	48	72/*	29117	Gray Silty Sand
713.0	50	59/36	29118	Gray Silty Sandy Gravel
711.5	52, 54	24/39	29119	Gray Sandy Gravel
708.0	56	27/29	29120	Gray Sand
706.5	58	100*	29121	Brown Silty Sandy Gravel
703.0	60, 62	43/53	-----	Gray Silty Sandy Gravel
	64			
698.0	66	33/*	29122	Gray Silty Gravelly Sand
	68			
693.0	70			
692.0		------	29123	Gray Sandy Gravelly Silt
	72			BOTTOM OF BORING
	74			*Refusal
	76			
	78			
	80			
	82			

Glacial Boulders (igneous and limestone)

APPENDIX V

BEARING RESISTANCE CHARTS


Retaining Wall 4W3
Factored Bearing Resistance and Service Limit Bearing Pressure
(B-001-C-59, B-027-0-08, B-027-1-13 B-028-0-08 and B-029-0-08)
Bearing Stress (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 inches)
Service Limit State (Settlement of 1.5 inches)
Strength Limit Bearing Pressure
Settlement = 0.672 in.
15.01
15.02
15.03
15.04
15.05
15.06
15.07
15.08
15.09
15.10
15.11
16.92
12.94
10.59
9.06
7.99
7.21
6.62
6.15
5.78
5.48
5.24
9.80
7.48
6.10
5.19
4.55
4.07
3.70
3.40
3.16
2.96
2.80
3.24
2.50
2.05
1.76
1.55
1.40
1.28
1.19
1.11
1.05
1.00

APPENDIX VI

SHALLOW FOUNDATION CALCULATIONS

W-13-045 - FRA-70-13.11 Project 4A - Retaining Wall 4W3

Shallow Foundation Analysis - Settlement

Calculated By: BRT Date: 7/8/2022

Checked By: JPS Date: 7/8/2022

Borings B-001-C-59, B-027-0-08, B-027-1-13, B-028-08 and B-029-0-08

B =	7.4	ft	Footing width
D_w =	0.0	ft	Depth below bottom of footing
q =	2,300	psf	Gross bearing pressure at bottom of wall

Soil Class.	Soil Type	Layer Depth (ft)		Layer Thickness H (ft)	Depth to Midpoint (ft)	γ (pcf)	σ_{vo} Bottom (psf)	σ_{vo} Midpoint (psf)	σ_{vo}' Midpoint (psf)	σ_p' (1) (psf)	LL	C_c (2)	C_r (3)	e_o (4)	N_{60}	$(N1)_{60}$ (5)	C' (6)	Z_f/B	I (7)	$\Delta\sigma_v$ (8) (psf)	σ_{vf}' Midpoint (psf)	S_c (9,10) (ft)	S_c (in)
A-1-a	G	0.0	2.0	2.0	1.0	135	270	135	73	4,073					90	180	1313	0.13	0.992	2,282	2,355	0.002	0.028
A-2-a	G	2.0	4.5	2.5	3.3	135	608	439	236	4,236					90	154	241	0.44	0.858	1,973	2,209	0.010	0.121
A-4a	C	4.5	9.5	5.0	7.0	130	1,258	933	496	4,496	19	0.081	0.008	0.420				0.94	0.574	1,321	1,816	0.016	0.193
A-3a	G	9.5	12.5	3.0	11.0	135	1,663	1,460	774	4,774					55	73	222	1.48	0.400	921	1,694	0.005	0.055
A-4a	C	12.5	17.5	5.0	15.0	130	2,313	1,988	1,052	5,052	22	0.108	0.011	0.444				2.02	0.303	697	1,749	0.008	0.099
A-4a	C	17.5	23.0	5.5	20.3	130	3,028	2,670	1,406	5,406	22	0.108	0.011	0.444				2.73	0.228	526	1,932	0.006	0.068
A-4a	G	23.0	28.0	5.0	25.5	135	3,703	3,365	1,774	5,774					100	104	165	3.43	0.183	421	2,195	0.003	0.034
A-1-b	G	28.0	33.0	5.0	30.5	135	4,378	4,040	2,137	6,137					100	98	453	4.10	0.154	353	2,490	0.001	0.009
A-1-b	G	33.0	38.0	5.0	35.5	135	5,053	4,715	2,500	6,500					100	93	414	4.78	0.132	304	2,804	0.001	0.007
A-3a	G	38.0	46.0	8.0	42.0	135	6,133	5,593	2,972	6,972					100	87	286	5.65	0.112	258	3,229	0.001	0.012
A-3a	G	46.0	54.5	8.5	50.3	135	7,280	6,706	3,571	7,571					100	81	257	6.76	0.094	216	3,786	0.001	0.010
A-6a	C	54.5	59.5	5.0	57.0	130	7,930	7,605	4,048	8,048	22	0.108	0.011	0.444				7.67	0.083	190	4,239	0.001	0.009
A-6a	C	59.5	65.0	5.5	62.3	130	8,645	8,288	4,403	8,403	22	0.108	0.011	0.444				8.38	0.076	174	4,577	0.001	0.008
A-1-b	G	65.0	73.0	8.0	69.0	135	9,725	9,185	4,879	8,879					100	70	270	9.29	0.068	157	5,037	0.000	0.005
A-1-b	G	73.0	81.0	8.0	77.0	135	10,805	10,265	5,460	9,460					100	67	249	10.36	0.061	141	5,601	0.000	0.004
A-3	G	81.0	87.5	6.5	84.3	135	11,683	11,244	5,987	9,987					100	64	148	11.34	0.056	129	6,116	0.000	0.005
A-3	G	87.5	94.5	7.0	91.0	135	12,628	12,155	6,477	10,477					100	61	141	12.25	0.052	119	6,596	0.000	0.005
																				Total Settlement:			0.672 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 26, FHWA GEC 5
3. $C_r = 0.10(C_c)$; Ref. Section 5.4.2.5 of FHWA GEC 5
4. $e_o = (C_c/1.15)+0.35$; Ref. Table 8-2, Holtz and Kovacs 1981
5. $(N1)_{60} = C_n N_{60}$, where $C_N = [0.77\log(40/\sigma_{vo}')] \leq 2.0$ ksf; Ref. Section 10.4.6.2.4, AASHTO LRFD BDS
6. Bearing capacity index; Ref. Figure 10.6.2.4.2-1, AASHTO LRFD BDS
7. Influence factor for strip loaded footing
8. $\Delta\sigma_v = q_e(I)$
9. $S_c = [C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_{vo}')$ for $\sigma_p' \leq \sigma_{vo}' < \sigma_{vf}'$; $[C_r/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')$ for $\sigma_{vo}' < \sigma_{vf}' \leq \sigma_p'$; $[Cr/(1+e_o)](H)\log(\sigma_p'/\sigma_{vo}')+[C_c/(1+e_o)](H)\log(\sigma_{vf}'/\sigma_p')$ for $\sigma_{vo}' < \sigma_p' < \sigma_{vf}'$; Ref. Section 10.6.2.4.3, AASHTO LRFD BDS (Cohesiv soil layers)
10. $S_c = H(1/C')\log(\sigma_{vf}'/\sigma_{vo}')$; Ref. Section 10.6.2.4.2, AASHTO LRFD BDS (Granular soil layers)

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

Borings B-001-A-59, B-003-A-59, B-021-2-13, B-021-4-14 and B-023-0-08

B =	6.9	ft	
L =	309	ft	
c =	5,250	psf	
γ =	130	pcf	
D_f =	4.0	ft	
φ =	0	deg	
D_w =	0.0	ft	Below ground surface

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \tfrac{1}{2}\gamma B N_{\gamma m} C_{w\gamma}$$ = 27.36 ksf

$$N_{cm} = N_c s_c i_c$$ = 5.16

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

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

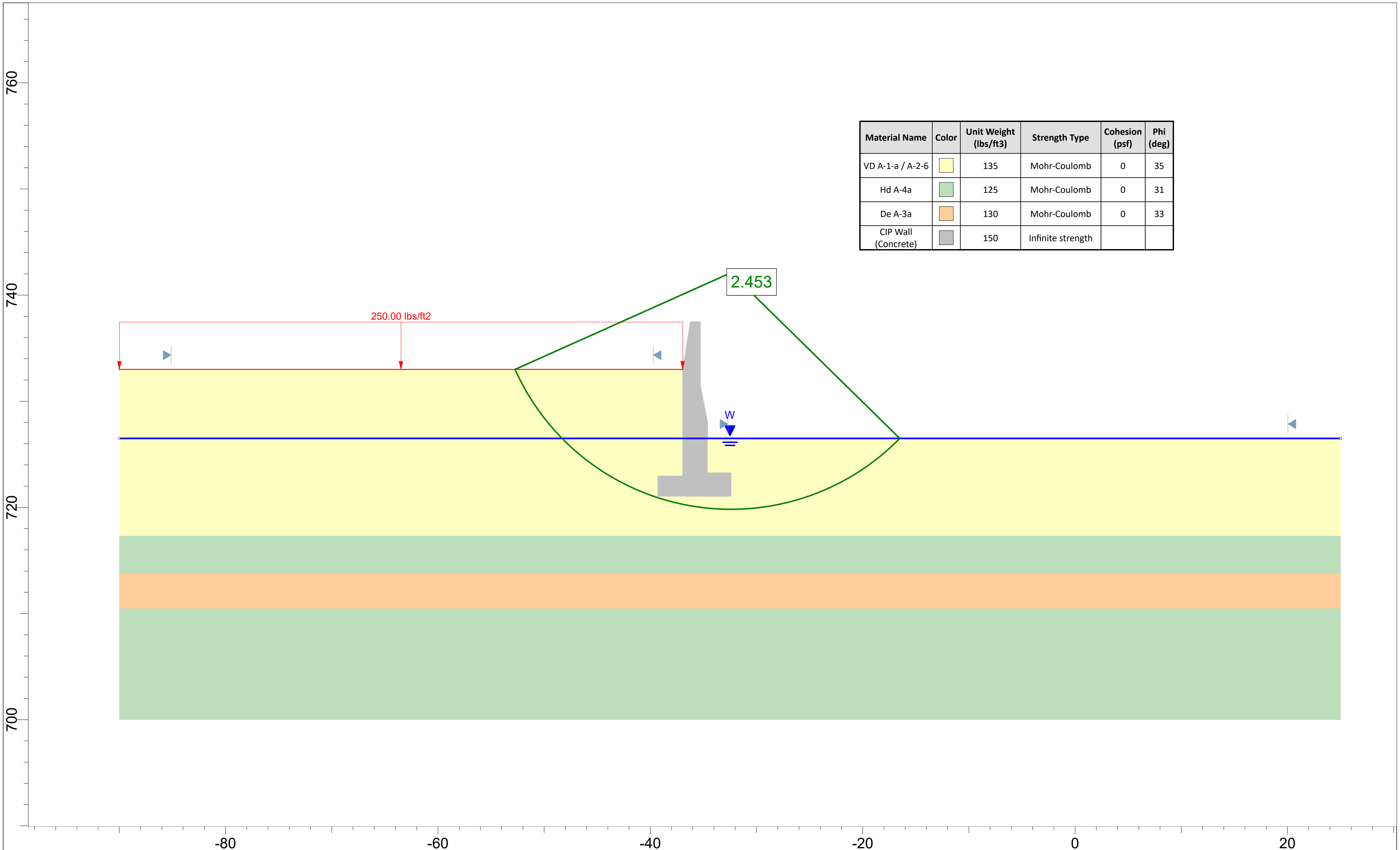
N_c =	5.14	s_c =	1+(6.9 ft/309 ft)(1/5.14) =	1.004	i_c =	1.000	d_q =	1+2tan(0°)[1-sin(0°)]^{2}tan^{-1}(4 ft/6.9 ft) =	1.000
N_q =	1.00	s_q =	1+(6.9 ft/309 ft)tan(0°) =	1.000	i_q =	1.000	C_{wq} =	0.0 ft < 4.0 ft =	0.500
N_γ =	0.00	s_γ =	1-0.4(6.9 ft/309 ft) =	0.991	i_γ =	1.000	$C_{w\gamma}$ =	0.0 ft < 1.5(6.9 ft) + 4 ft =	0.500

$$q_R = q_n \cdot \phi_b$$ = 15.05 ksf

φ_b = 0.55

APPENDIX VII

GLOBAL STABILITY ANALYSIS OUTPUTS

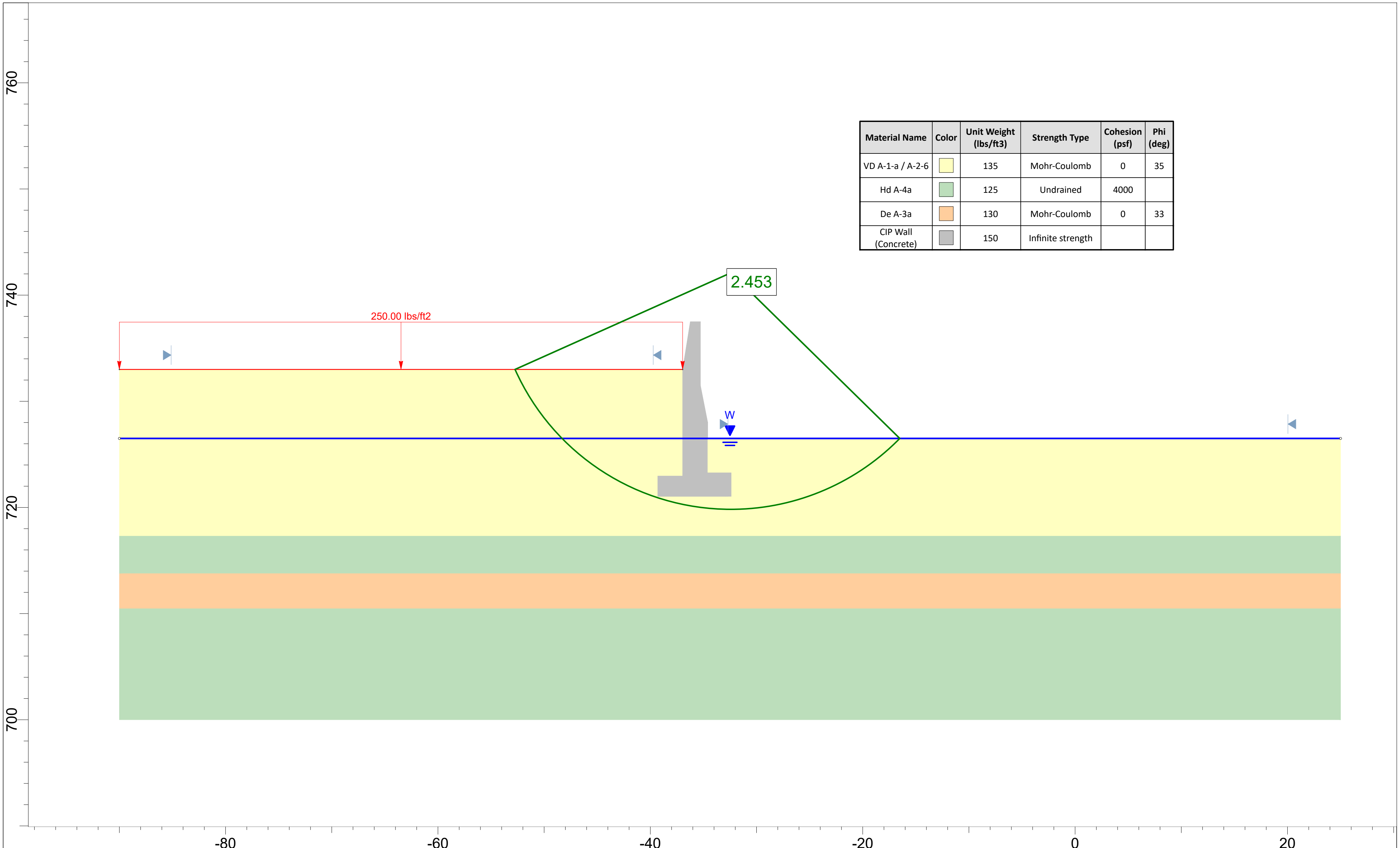


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
VD A-1-a / A-2-6		135	Mohr-Coulomb	0	35
Hd A-4a		125	Mohr-Coulomb	0	31
De A-3a		130	Mohr-Coulomb	0	33
CIP Wall (Concrete)		150	Infinite strength		

Resource International, Inc.
Planning | Engineering | Construction Management | Technology

Project	FRA-70-13.11 Project 4A - Retaining Wall 4W3
Analysis Description	CIP Wall Type - Drained - Circular - Spencer's
Drawn By	BRT
Scale	1:150
Company	Resource International, Inc.
Date	6/27/2020
File Name	Retaining Wall 4W3.slim

SLIDEINTERPRET 8.020



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
VD A-1-a / A-2-6		135	Mohr-Coulomb	0	35
Hd A-4a		125	Undrained	4000	
De A-3a		130	Mohr-Coulomb	0	33
CIP Wall (Concrete)		150	Infinite strength		

Resource International, Inc.
Planning | Engineering | Construction Management | Technology

Project	FRA-70-13.11 Project 4A - Retaining Wall 4W3
Analysis Description	CIP Wall Type - Undrained - Circular - Spencer's
Drawn By	BRT
Scale	1:150
Company	Resource International, Inc.
Date	6/27/2020
File Name	Retaining Wall 4W3.slim

SLIDEINTERPRET 8.020

APPENDIX VIII

EXTERNAL STABILITY ANALYSIS CALCULATIONS BY GPD GROUP

Client:	ODOT/District 6	Job No.:	2012048		
Project:	FRA-70 Project 4A	Page No.:	1	Of	3
Subject:	Wall 4W3 Design	Designed:	DJC	Date:	12/30/2019
		Checked:	MOJ	Date:	5/2/2022

GPD GROUP

Spread Footing Retaining Wall Design

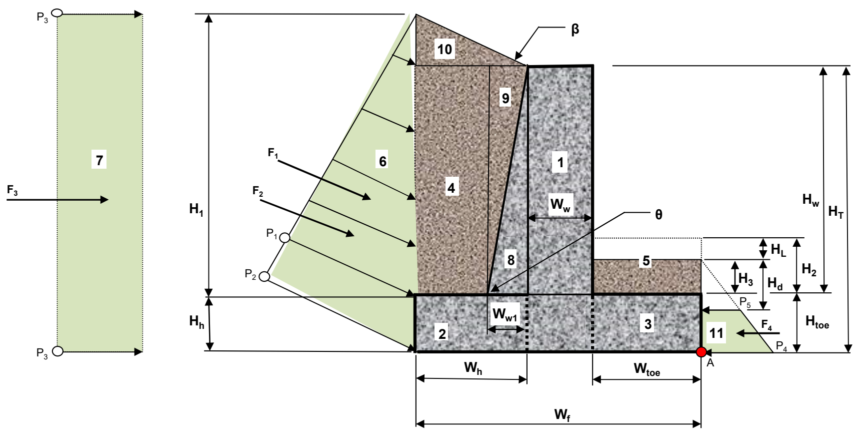
Based on AASHTO LRFD Bridge Design Specifications (9th edition) and the 2020 ODOT BDM.

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 =	*9.60*	ft	
Total Height, $H_T = H_w+H_{toe}$ =	11.85	ft	
Soil Height over Heel, $H_1 = H_T-H_h+(W_h*S_e)$ =	9.85	ft	
Max. Soil Height over Toe, H_2 =	*5.20*	ft	
Future Loss of Soil over Toe, H_L =	*0.00*	ft	
Min. Soil Height over Toe, $H_3 = \max(0,H_2-H_L)$ =	5.20	ft	
Depth of Disturbance, H_d =	*2.67*	ft	
Wall Width, W_w =	*4.26*	ft	
Toe Width, W_{toe} =	*1.91*	ft	
Heel Width, W_h =	*2.58*	ft	
Additional Wall, W_{w1} =	*0.00*	ft	
Theta, θ =	90.00	deg.	
Footing Width, W_f =	8.75	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 =	*-2.58*	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 =	3.22	ft	*LRFD Table 3.11.6.4-1*
Surcharge Height in front of Wall, H_{sf} =	4.27	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) =	*21.602*	ksf	
Bearing Capacity (Service) =	*6.077*	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} =	14.08	kips
Resistance, $R_\tau = V_{min} * \tan(\delta)$ =	9.86	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:

$P_1 = P_{soil}*H_1/1000$ =	0.35	ksf
$P_2 = P_{soil}*(H_1+H_h)/1000$ =	0.42	ksf
$P_3 = H_s*P_{soil\ LLS}/1000$ =	0.12	ksf
$P_4 = \gamma_{soil}*k_p*(H_{toe}+H_2-H_L)$ =	2.68	ksf
$P_5 = \gamma_{soil}*k_p*H_d$ =	0.96	ksf

Soil Sliding Force Calculations:

$F_1 = P_1*H_1*0.5$ =	1.73	kips
$F_2 = P_2*(H_1+H_h)*0.5$ =	2.50	kips
$F_3 = P_3*H_1$ =	1.42	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 =	*5.09*	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	11.85 ft. x	1.00 ft. =		**7.57 kips**
Arm 1 = W_{toe} + W_w / 2 =	1.91 ft. +	4.26 ft.	/ 2.00 =			**4.12 ft.** Revised for wall geometry.
Area 2 = γ_c x W_h x H_h =	0.150 kcf x	2.58 ft. x	2.00 ft. x	1.00 ft. =		**0.78 kips**
Arm 2 = W_{toe} + W_w + W_h / 2 =	1.91 ft. +	4.26 ft. +	2.58 ft.	/ 2.00 =		**7.46 ft.**
Area 3 = γ_c x W_{toe} x H_{toe} =	0.150 kcf x	1.91 ft. x	2.25 ft. x	1.00 ft. =		**0.64 kips**
Arm 3 = W_{toe} / 2 =	1.91 ft.	/ 2.00 =				**0.95 ft.**
Area 4 = γ_s x (W_h -W_{w1}) x H_W =	0.120 kcf x	(2.58 ft. -	0.00 ft.) x	9.60 ft. x	1.00 ft. =	**2.98 kips**
Arm 4 = W_{toe} + W_w + W_{w1} + (W_h -W_{w1}) / 2 =	1.91 ft. +	4.26 ft. +	0.00 ft. +	(2.58 ft. -	0.00 ft.) / 2 =	**7.46 ft.**
Area 5 **(Max.)** = γ_s x W_{toe} x H_2 =	0.120 kcf x	1.91 ft. x	5.20 ft. x	1.00 ft. =		**1.19 kips**
Area 5 **(Min.)** = γ_s x W_{toe} x H_3 =	0.120 kcf x	1.91 ft. x	5.20 ft. x	1.00 ft. =		**1.19 kips**
Arm 5 = W_{toe} / 2 =	1.91 ft.	/ 2.00 =				**0.95 ft.**
Area 6 **(Horiz. Comp.)** = F_2 x cos(δ) =	2.50 kips x	cos (	20.00 deg.) =			**2.35 kips**
Arm 6 = (H_1 + H_h) / 3 =	(9.85 ft. +	2.00 ft.)	/ 3.00 =			**3.95 ft.**
Area 6 **(Vertical Comp.)** = F_2 x sin(δ) =	2.50 kips x	sin (	20.00 deg.) =			**0.86 kips**
Arm 6 = Wf =	8.75 ft.					**8.75 ft.**
Area 7 = F_3 =	1.42 kips					**1.42 kips**
Arm 7 = (H_1 + H_h) / 2 =	(9.85 ft. +	2.00 ft.)	/ 2.00 =			**5.93 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	9.60 ft. x	1.00 ft. =		**0.00 kips**
Arm 8 = W_{toe} + W_w + W_{w1} / 3 =	1.91 ft. +	4.26 ft. +	0.00 ft.	/ 3.00 =		**6.17 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	9.60 ft. x	1.00 ft. =			**0.00 kips**
Arm 9 = W_{toe} + W_w + W_{w1} x 2/3 =	1.91 ft. +	4.26 ft. +	0.00 ft.	x 2.00	/ 3.00	=	**6.17 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	2.58 ft.) x	2.58 ft. x	1.00 ft.	=	**0.00 kips**
Arm 10 = W_F - W_h / 3 =	8.75 ft. -	2.58 ft.	/ 3.00 =				**7.89 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	2.58 ft. x	3.22 ft. x	1.00 ft. =			**1.04 kips**
Arm for Heel Surcharge = W_F - W_h / 2 =	8.75 ft. -	2.58 ft.	/ 2.00 =				**7.46 ft.**
Surcharge on Toe = $\gamma_{soil\ LLS}$ x W_{toe} x H_{sf} =	0.125 kcf x	1.91 ft. x	4.27 ft. x	1.00 ft. =			**1.02 kips**
Arm for Toe Surcharge = W_{toe} / 2 =	1.91 ft.	/ 2.00 =					**0.95 ft.**

Check Bearing Pressure:
per BDM 307.1.5 and LRFD 11.6.3.2.

Factored Bearing Resistance = **21.60 ksf**

Maximum Strength Load Pressures:
Bearing pressure at Toe = **3.19 ksf** **OK**
Bearing pressure at Heel = **3.19 ksf** **OK**

Check Eccentricity:
per BDM 307.1.4 and LRFD 11.6.3.3.

Maximum Allowable e = B/3 = **2.92 ft**
Controlling Eccentricity = **1.34 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 = 9.86 kips
Factored Horizontal Sliding Resistance = 9.86 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) = 2.68 ksf
Pressure at Bottom of Disturbance (P_5) = 0.96 ksf
Pressure at Bottom of Key or Sheeting = 2.68 ksf

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

Total Factored Resisting Force = **9.86 kips**
Driving Force = **6.01 kips** **OK**

Check Settlement:

Service Bearing Capacity = **6.08 ksf**
Service Bearing Pressure at Toe = **2.22 ksf** **OK**
Service Bearing Pressure at Heel = **2.22 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	3.19	3.19	1.14	1.34	6.01	14.08
SERVICE I	2.22	2.22	0.85	N/A	3.77	14.61

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)*

Client:	ODOT/District 6	Job No.:	2012048
Project:	FRA-70 Project 4A	Page No.:	1 Of 3
Subject:	Wall 4W3 Design	Designed:	DJC Date: 12/30/2019
		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.)	2.35	1.50	3.53	3.95	13.95	Horiz. Forces
7	1.42	1.75	2.48	5.93	14.72	
		Σ Sliding Forces, F_s =	6.01 kips	Σ Overturning Moments =	28.66 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	7.57	1.20	9.05	0.86	6.52	4.12	37.25	26.82	
2	0.78	1.25	0.97	0.90	0.70	7.46	7.23	5.20	Dead Loads From Concrete
3	0.64	1.25	0.80	0.90	0.58	0.95	0.77	0.55	
8	0.00	1.25	0.00	0.90	0.00	6.17	0.00	0.00	
4	2.98	1.35	4.02	1.00	2.98	7.46	29.96	22.20	Dead Loads From Soil *(Do not include 5 (Min.) and 5 (Max.) simultaneously)*
5 (Max.)	1.19	1.35	1.61	1.00	1.19	0.95	1.53	1.13	
5 (Min.)	1.19	1.35	1.61	1.00	1.19	0.95	1.53	1.13	
6 (Vertical comp.)	0.86	1.50	1.29	1.50	1.29	8.75	11.24	11.24	
9	0.00	1.35	0.00	1.00	0.00	6.17	0.00	0.00	
10	0.00	1.35	0.00	1.00	0.00	7.89	0.00	0.00	
Surcharge on Heel	1.04	1.75	1.82	0.00	0.00	7.46	13.58	0.00	External Loads
Surcharge on Toe	1.02	1.75	1.78	0.00	0.00	0.95	1.70	0.00	
DC	0.92	1.25	1.16	0.90	0.83	5.09	5.88	4.23	
		Σ Vert. Forces =	20.67 kips	Σ Vert. Forces =	14.08 kips	Σ Resist. Moments =	95.56 k*ft.	71.39 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 =	28.66 k-ft.
Resisting Moment = Σ Max. Resisting Moments =	95.56 k-ft.
Net Moment = Resisting Moment - Overturning Moment =	66.90 k-ft.
Total Vertical Force (TVF) = Σ Vert. Forces =	20.67 kips
Dist. from Point A (Ā) = Net. Moment / TVF =	3.24 ft.
Eccentricity "e" = $(0.5{*}W_f)$ - Ā =	1.14 ft.
Maximum Bearing Pressure = TVF/(Wf-2*e) =	3.19 ksf
Minimum Bearing Pressure = TVF/(Wf-2*e) =	3.19 ksf

Min. Load Factor Calculations (Worst case eccentricity shown.)	
Overturning Moment = Σ Overturning Moments =	28.66 k-ft.
Resisting Moment = Σ Min. Resisting Moments =	71.39 k-ft.
Net Moment = Resisting Moment - Overturning Moment =	42.72 k-ft.
Total Vertical Force (TVF) = Σ Vert. Forces =	14.08 kips
Dist. from Point A (Ā) = Net. Moment / TVF =	3.04 ft.
Eccentricity "e" = $(0.5{*}W_f)$ - Ā =	1.34 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.)	2.35	1.00	2.35	3.95	9.30	Horiz. Forces
7	1.42	1.00	1.42	5.93	8.41	
		Σ Sliding Forces, F_s =	3.77 kips	Σ Overturning Moments =	17.71 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	7.57	0.96	7.24	4.12	29.80	
2	0.78	1.00	0.78	7.46	5.78	Dead Loads From Concrete
3	0.64	1.00	0.64	0.95	0.61	
8	0.00	1.00	0.00	6.17	0.00	
4	2.98	1.00	2.98	7.46	22.20	Dead Loads From Soil (Do not include 5 (Min.) and 5 (Max.) simultaneously)
5 (Max.)	1.19	1.00	1.19	0.95	1.13	
5 (Min.)	1.19	1.00	1.19	0.95	1.13	
6 (Vertical comp.)	0.86	1.00	0.86	8.75	7.50	
9	0.00	1.00	0.00	6.17	0.00	
10	0.00	1.00	0.00	7.89	0.00	
Surcharge on Heel	1.04	1.00	1.04	7.46	7.76	External Loads
Surcharge on Toe	1.02	1.00	1.02	0.95	0.97	
DC	0.92	1.00	0.92	5.09	4.70	
		Σ Vert. Forces =	15.62 kips	Σ Resisting Moments =	72.70 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 =	17.71 k-ft.
Resisting Moment = Σ Max. Resisting Moments =	72.70 k-ft.
Net Moment = Resisting Moment - Overturning Moment =	54.99 k-ft.
Total Vertical Force (TVF) = Σ Vert. Forces =	15.62 kips
Dist. from Point A (Ā) = Net. Moment / TVF =	3.52 ft.
Eccentricity "e" = $(0.5{*}W_f)$ - Ā =	0.85 ft.
Maximum Bearing Pressure = TVF/(Wf-2*e) =	2.22 ksf
Minimum Bearing Pressure = TVF/(Wf-2*e) =	2.22 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:

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

- 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:

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

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

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:

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

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

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

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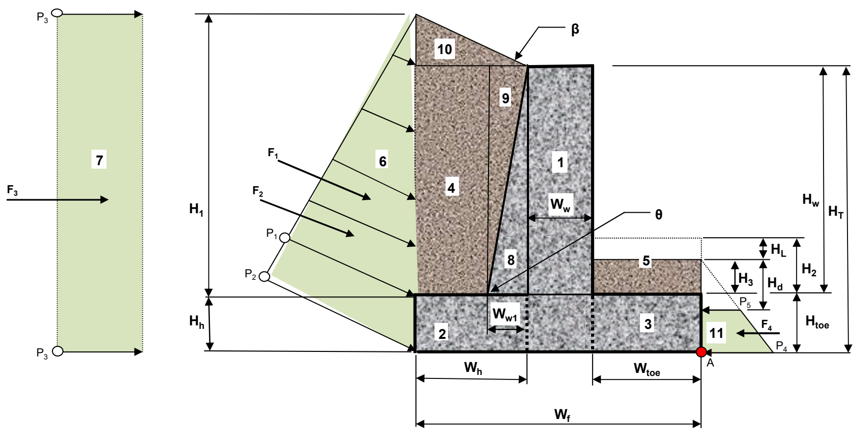
			Checked:	MOJ	Date: 5/2/2022

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 =	*9.64*	ft
Total Height, $H_T = H_w + H_{toe}$ =	11.89	ft
Soil Height over Heel, $H_1 = H_T - H_h + (W_h * S_e)$ =	9.89	ft
Max. Soil Height over Toe, H_2 =	*5.10*	ft
Future Loss of Soil over Toe, H_L =	*0.00*	ft
Min. Soil Height over Toe, $H_3 = \max(0, H_2 - H_L)$ =	5.10	ft
Depth of Disturbance, H_d =	*2.67*	ft
Wall Width, W_w =	*6.24*	ft
Toe Width, W_{toe} =	*1.01*	ft
Heel Width, W_h =	*1.51*	ft
Additional Wall, W_{w1} =	*0.00*	ft
Theta, θ =	90.00	deg.
Footing Width, W_f =	8.75	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 =	*-1.51*	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 =	3.22	ft	*LRFD Table 3.11.6.4-1*
Surcharge Height in front of Wall, H_{sf} =	4.30	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) =	*21.880*	ksf	
Bearing Capacity (Service) =	*5.871*	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} =	15.58	kips
Resistance, $R_\tau = V_{min} * \tan(\delta)$ =	10.91	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:

$P_1 = P_{soil} * H_1 / 1000$ =	0.35	ksf
$P_2 = P_{soil} * (H_1 + H_h) / 1000$ =	0.42	ksf
$P_3 = H_s * P_{soil\ LLS} / 1000$ =	0.12	ksf
$P_4 = \gamma_{soil} * k_p * (H_{toe} + H_2 - H_L)$ =	2.65	ksf
$P_5 = \gamma_{soil} * k_p * H_d$ =	0.96	ksf

Soil Sliding Force Calculations:

$F_1 = P_1 * H_1 * 0.5$ =	1.74	kips
$F_2 = P_2 * (H_1 + H_h) * 0.5$ =	2.52	kips
$F_3 = P_3 * H_1$ =	1.42	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 =	*5.62*	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	11.89 ft. x	1.00 ft. =		**11.13 kips**	
Arm 1 = W_{toe} + W_w / 2 =	1.01 ft. +	6.24 ft.	/ 2.00 =			**4.22 ft.**	**Revised for wall geometry.**
Area 2 = γ_c x W_h x H_h =	0.150 kcf x	1.51 ft. x	2.00 ft. x	1.00 ft. =		**0.45 kips**	
Arm 2 = W_{toe} + W_w + W_h / 2 =	1.01 ft. +	6.24 ft. +	1.51 ft.	/ 2.00 =		**8.00 ft.**	
Area 3 = γ_c x W_{toe} x H_{toe} =	0.150 kcf x	1.01 ft. x	2.25 ft. x	1.00 ft. =		**0.34 kips**	
Arm 3 = W_{toe} / 2 =	1.01 ft.	/ 2.00 =				**0.50 ft.**	
Area 4 = γ_s x (W_h - W_{w1}) x H_W =	0.120 kcf x	(1.51 ft. -	0.00 ft.) x	9.64 ft. x	1.00 ft. =	**1.75 kips**	
Arm 4 = W_{toe} + W_w + W_{w1} + (W_h - W_{w1}) / 2 =	1.01 ft. +	6.24 ft. +	0.00 ft. +	(1.51 ft. -	0.00 ft.) / 2 =	**8.00 ft.**	
Area 5 **(Max.)** = γ_s x W_{toe} x H_2 =	0.120 kcf x	1.01 ft. x	5.10 ft. x	1.00 ft. =		**0.62 kips**	
Area 5 **(Min.)** = γ_s x W_{toe} x H_3 =	0.120 kcf x	1.01 ft. x	5.10 ft. x	1.00 ft. =		**0.62 kips**	
Arm 5 = W_{toe} / 2 =	1.01 ft.	/ 2.00 =				**0.50 ft.**	
Area 6 **(Horiz. Comp.)** = F_2 x cos(δ) =	2.52 kips x	cos (	20.00 deg.) =			**2.37 kips**	
Arm 6 = (H_1 + H_h) / 3 =	(9.89 ft. +	2.00 ft.)	/ 3.00 =			**3.96 ft.**	
Area 6 **(Vertical Comp.)** = F_2 x sin(δ) =	2.52 kips x	sin (	20.00 deg.) =			**0.86 kips**	
Arm 6 = Wf =	8.75 ft.					**8.75 ft.**	
Area 7 = F_3 =	1.42 kips					**1.42 kips**	
Arm 7 = (H_1 + H_h) / 2 =	(9.89 ft. +	2.00 ft.)	/ 2.00 =			**5.95 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	9.64 ft. x	1.00 ft. =		**0.00 kips**	
Arm 8 = W_{toe} + W_w + W_{w1} / 3 =	1.01 ft. +	6.24 ft. +	0.00 ft.	/ 3.00 =		**7.25 ft.**	

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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	9.64 ft. x	1.00 ft. =		**0.00 kips**
Arm 9 = W_{toe} + W_w + W_{w1} x 2/3 =	1.01 ft. +	6.24 ft. +	0.00 ft.	x 2.00	/ 3.00 =	**7.25 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	1.51 ft.) x	1.51 ft. x	1.00 ft. =	**0.00 kips**
Arm 10 = W_F - W_h / 3 =	8.75 ft. -	1.51 ft.	/ 3.00 =			**8.25 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	1.51 ft. x	3.22 ft. x	1.00 ft. =		**0.61 kips**
Arm for Heel Surcharge = W_F - W_h / 2 =	8.75 ft. -	1.51 ft.	/ 2.00 =			**8.00 ft.**
Surcharge on Toe = $\gamma_{soil\ LLS}$ x W_{toe} x H_{sf} =	0.125 kcf x	1.01 ft. x	4.30 ft. x	1.00 ft. =		**0.54 kips**
Arm for Toe Surcharge = W_{toe} / 2 =	1.01 ft.	/ 2.00 =				**0.50 ft.**

Check Bearing Pressure:

per BDM 307.1.5 and LRFD 11.6.3.2.

Factored Bearing Resistance = **21.88 ksf**

Maximum Strength Load Pressures:

Bearing pressure at Toe = **3.20 ksf** **OK**

Bearing pressure at Heel = **3.20 ksf** **OK**

Check Eccentricity:

per BDM 307.1.4 and LRFD 11.6.3.3.

Maximum Allowable e = B/3 = **2.92 ft**

Controlling Eccentricity = **1.20 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 = 10.91 kips

Factored Horizontal Sliding Resistance = 10.91 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) = 2.65 ksf

Pressure at Bottom of Disturbance (P_5) = 0.96 ksf

Pressure at Bottom of Key or Sheeting = 2.65 ksf

Unfactored Passive Resistance = 0.00 kips

Factored Passive Resistance = 0.00 kips

Total Factored Resisting Force = **10.91 kips**

Driving Force = **6.04 kips** **OK**

Check Settlement:

Service Bearing Capacity = **5.87 ksf**

Service Bearing Pressure at Toe = **2.30 ksf** **OK**

Service Bearing Pressure at Heel = **2.30 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	3.20	3.20	0.94	1.20	6.04	15.58
SERVICE I	2.30	2.30	0.68	N/A	3.79	16.48

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)*

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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.)	2.37	1.50	3.55	3.96	14.09	Horiz. Forces
7	1.42	1.75	2.49	5.95	14.79	
		Σ Sliding Forces, F_s =	6.04 kips	Σ Overturning Moments =	28.88 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	Force (k) Min. Load Factor (*This column is for stability*)	Moment Arm (ft)	Moment (k-ft) Max. Load Factor	Moment (k-ft) Min. Load Factor (*This column is for stability*)	
1	11.13	**1.21**	13.46	**0.87**	9.69	4.22	56.76	40.87	
2	0.45	1.25	0.57	0.90	0.41	8.00	4.53	3.26	Dead Loads
3	0.34	1.25	0.42	0.90	0.31	0.50	0.21	0.15	From Concrete
8	0.00	1.25	0.00	0.90	0.00	7.25	0.00	0.00	
4	1.75	1.35	2.36	1.00	1.75	8.00	18.85	13.97	Dead Loads
5 (Max.)	0.62	1.35	0.83	1.00	0.62	0.50	0.42	0.31	From Soil *(Do*
5 (Min.)	0.62	1.35	0.83	1.00	0.62	0.50	0.42	0.31	*not include 5*
6 (Vertical comp.)	0.86	1.50	1.29	1.50	1.29	8.75	11.33	11.33	*(Min.) and 5*
9	0.00	1.35	0.00	1.00	0.00	7.25	0.00	0.00	*(Max.)*
10	0.00	1.35	0.00	1.00	0.00	8.25	0.00	0.00	*simultaneously)*
Surcharge on Heel	0.61	1.75	1.06	0.00	0.00	8.00	8.50	0.00	
Surcharge on Toe	0.54	1.75	0.94	0.00	0.00	0.50	0.48	0.00	External Loads
DC	1.69	1.25	2.11	0.90	1.52	5.62	11.87	8.54	
		Σ Vert. Forces =	21.99 kips	Σ Vert. Forces =	15.58 kips	Σ Resist. Moments =	104.44 k*ft.	78.42 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.)		*Min. Load Factor Calculations (Worst case eccentricity shown.)*	
Overturning Moment = Σ Overturning Moments =	28.88 k-ft.	Overturning Moment = Σ Overturning Moments =	28.88 k-ft.
Resisting Moment = Σ Max. Resisting Moments =	104.44 k-ft.	Resisting Moment = Σ Min. Resisting Moments =	78.42 k-ft.
Net Moment = Resisting Moment - Overturning Moment =	75.56 k-ft.	Net Moment = Resisting Moment - Overturning Moment =	49.55 k-ft.
Total Vertical Force (TVF) = Σ Vert. Forces =	21.99 kips	Total Vertical Force (TVF) = Σ Vert. Forces =	15.58 kips
Dist. from Point A (Ā) = Net. Moment / TVF =	3.44 ft.	Dist. from Point A (Ā) = Net. Moment / TVF =	3.18 ft.
Eccentricity "e" = (0.5*W_f) - Ā =	0.94 ft.	Eccentricity "e" = (0.5*W_f) - Ā =	1.20 ft.
Maximum Bearing Pressure = TVF/(Wf-2*e) =	3.20 ksf		
Minimum Bearing Pressure = TVF/(Wf-2*e) =	3.20 ksf		

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.)	2.37	1.00	2.37	3.96	9.39	Horiz. Forces
7	1.42	1.00	1.42	5.95	8.45	
		Σ Sliding Forces, F_s =	3.79 kips	Σ Overturning Moments =	17.84 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	11.13	**0.97**	10.77	4.22	45.41	
2	0.45	1.00	0.45	8.00	3.62	Dead Loads From
3	0.34	1.00	0.34	0.50	0.17	Concrete
8	0.00	1.00	0.00	7.25	0.00	
4	1.75	1.00	1.75	8.00	13.97	
5 (Max.)	0.62	1.00	0.62	0.50	0.31	Dead Loads
5 (Min.)	0.62	1.00	0.62	0.50	0.31	From Soil (Do not
6 (Vertical comp.)	0.86	1.00	0.86	8.75	7.55	include 5 (Min.) and
9	0.00	1.00	0.00	7.25	0.00	5 (Max.)
10	0.00	1.00	0.00	8.25	0.00	simultaneously)
Surcharge on Heel	0.61	1.00	0.61	8.00	4.85	
Surcharge on Toe	0.54	1.00	0.54	0.50	0.27	External Loads
DC	1.69	1.00	1.69	5.62	9.49	
		Σ Vert. Forces =	17.02 kips	Σ Resisting Moments =	80.79 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 =	17.84 k-ft.
Resisting Moment = Σ Max. Resisting Moments =	80.79 k-ft.
Net Moment = Resisting Moment - Overturning Moment =	62.95 k-ft.
Total Vertical Force (TVF) = Σ Vert. Forces =	17.02 kips
Dist. from Point A (Ā) = Net. Moment / TVF =	3.70 ft.
Eccentricity "e" = (0.5*W_f) - Ā =	0.68 ft.
Maximum Bearing Pressure = TVF/(Wf-2*e) =	2.30 ksf
Minimum Bearing Pressure = TVF/(Wf-2*e) =	2.30 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.