GEOTECHNICAL EXPLORATION REPORT

ODOT LPA: CUY-CR57-4.43 (WALLINGS ROAD), PID 106239
BROADVIEW, OHIO

SME Project Number: 078832.00
November 12, 2021

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November 12, 2021

Shelley M. Kiwala, PE
OSBORN ENGINEERING
1100 Superior Avenue - Suite 300
Cleveland, OH 44114-2530

E-mail to: skiwala@osborn-eng.com

Re: ODOT LPA: CUY-CR57-4.43
Wallings Road Bridge Widening
Structure Foundation Exploration - Final
Broadview Heights, Ohio
SME Project No. 078832.00

Dear Ms. Kiwala:

We have completed the subgrade exploration report for the planned widening of Wallings Road in Broadview Heights, Ohio. The attached revised report presents the results of our subsurface investigation and our recommendations for the Walling Road bridge foundation.

If you have any questions, or wish to discuss our recommendations, please call.

Very truly yours,

SME

Alan J. Esser, PE, D.GE
Chief Consultant

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EXECUTIVE SUMMARY

The existing Wallings Road Bridge over Interstate 77 is heavily traveled with most traffic either entering or leaving the Interstate. The interchange at Walling Road is a major access point for the predominantly residential communities of Broadview Heights and North Royalton to the west, and Brecksville to the east. The existing bridge will be widened to accommodate increased traffic flow. The replacement bridge will be similar in construction to the existing bridge with four spans supported on the end abutments and intermediate piers.

This section of Interstate 77 was constructed circa 1965 in a shale cut at Wallings Road such that the shallow foundations supporting the existing bridge all bear on shale. The shale appears to be Mississippian age Bedford Shale, which in the area of the project is a fractured, weak to very weak, argillaceous shale that weathers rapidly to clay. Some borings for this widening encountered a thin layer of residual clay at the surface of the rock. A 2003 widening project along the Interstate lowered the highway profile grades under the bridge by about 3 feet.

The geotechnical evaluation for this project focused on the shale as the bearing material for shallow foundations. A maximum bearing pressure of 25 ksf and 16 ksf are recommended for design of the bridge and abutment foundations, respectively. Lateral loads should be based on active lateral earth pressures. An equivalent fluid unit weight of 36 pcf, and lateral earth pressure coefficient of 0.3 for surcharges are recommended. Recommendations for excavation and backfill are based on ODOT Construction and Materials Specifications.

This summary should not be used without reading and understanding the assumptions in the entire report.

INTRODUCTION

This report presents the results of a subsurface exploration for the widening of the Wallings Road Bridge over Interstate 77 in Broadview Heights, Ohio. Subsurface conditions were evaluated by drilling five borings near the abutments and piers of the existing bridge. The borings were drilled in general conformance with the current ODOT Specifications for Geotechnical Explorations.

Samples from the borings were taken to our laboratory for classification and testing. The results of the field and laboratory tests were interpreted to provide foundation recommendations for the new bridge spans.

GEOLOGY AND OBSERVATIONS

GEOLOGY

The geology of the area surrounding the Wallings Road Bridge consists of ground moraine over shale bedrock — probably the Bedford Shale. Bedford Shale is a Mississippian age rock that falls between the Berea Sandstone above and the Devonian age Cleveland Shale below in the geologic column. The Bedford formation ranges from a hard siltstone in southern Ohio to soft, gray to blue-gray, mudrock in northern Ohio. In northern Ohio the formation contains two distinct siltstone members, the Sagamore and Euclid siltstone lentils. The upper part of the formation contains thin seams of harder silty shale. This was generally the case in the borings

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drilled for this project where the Bedford is a very week to weak, thinly bedded, fractured, argillaceous shale that weathers rapidly to sticky mud. Thin silty shale seams characteristic of the Bedford formation were encountered.

RECONNAISSANCE

The Wallings Road Bridge is a four-span structure with access ramps at each end of the bridge. Traffic data indicates that the majority of traffic exiting the Interstate at Walling Road travels in the westbound direction away from the bridge. The heaviest traffic entering the Interstate comes from the west, across the bridge, to enter via the northbound entrance ramp. The area surrounding the bridge is primarily residential. A church northeast of the bridge has increased traffic during times when members are coming to or leaving after services.

Alan Esser with SME performed a site reconnaissance on April 25, 2018. Pavement patches are evident in the approach slabs at both ends of the bridge, apparently from storm sewer repairs or improvements. A concentration of underground utilities behind the abutments required care in locating the abutment borings.

The bridge piers are protected by concrete barriers and guard rail. The center pier is in a narrow grass median between concrete barriers. A storm sewer within the grass strip parallels the barrier wall on the east side of the pier. Original plans for the Interstate indicate that the invert of this sewer is about elevation 1049 feet. This storm sewer crosses beneath an 8-inch sanitary sewer that is parallel to, and on the north side, of the bridge. Slopes under the bridge at the abutments are at 2H:1V and 3.3H:1V ratios at the west and east ends of the bridge, respectively. Slopes are flatter at the west end immediately north and south of the bridge where the slope ratios are closer to 6H:1V. There is a relatively wide, level strip between the edge of pavement and the toe of slope. Lighting along the interstate is in this level strip on both sides of the highway. Slopes under the bridge are covered with dump rock.

EXISTING DATA

Borings were completed in 1965 along the I77 corridor for the original construction and in 2004 for a subsequent widening project. The original construction borings included borings at Wallings Road for the rear abutment and the forward pier. Three borings from the 2004 widening project were drilled along the northbound lanes close to the bridge. Two of the borings encountered shale within about 2.5 feet of the ground surface. The 1965 borings encountered shale at depths of 10 and 6 feet at the abutment and pier respectively. Two of the 1965 borings, (B-001-0-65 and B-008-0-65) and two 2003 borings (P-014-0-03 and S-010-A-03) were used for this analysis and are included in the Soil Profile Sheets. The bridge deck was constructed near existing grade and the roadway was built in a cut that extended at least 10 to 15 feet into the shale. Structural drawings for the bridge show foundation bearing elevations for the abutments and for each of the three piers.

EXPLORATION

Exploration for this project consisted of five borings what were drilled to rock and then cored 10 feet into rock. The borings were continuous sampled to 10 feet or rock then at 2.5-foot intervals until bedrock was encountered. We augured into the rock sampling at up to 5-foot intervals until rock hard enough for coring was encountered. We used N-size wireline for the coring. Total boring depths ranged from 19 to 28.5 feet.

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Drilling equipment consisted of a CME-55 truck mounted rig equipped with a CME auto hammer having an energy transfer ratio of 81.3 percent based on our calibrations performed on April 5, 2017. We observed the boreholes for the presence of groundwater during drilling and prior to coring. Groundwater was only encountered at B-001-0-18. The boreholes were sealed with cement-bentonite grout (logs show bentonite grout) on completion.

Samples were placed in clean glass jars each marked with the project, boring, depth interval, and blow-count data. Cores were placed in plastic core boxes. Samples were taken to our laboratory at the end of each day's work. The cores were covered with damp towels and kept in a moist condition until they were classified and tested.

Samples were classified in accordance with the ODOT-modified AASHTO procedure. Laboratory testing included water contents on all SPT samples, mechanical sieve and hydrometer tests on five representative soil samples, and uniaxial compression or point load tests on representative shale samples. Because of the degree of fracturing, few rock specimens were long enough for uniaxial compression testing, so we supplemented those tests with point load tests.

We interpreted the point load tests in accordance with ASTM D5731 to determine the equivalent uniaxial compressive strengths. The compressive strengths are reported on the boring logs with the results for the uniaxial compression tests. The core was photographed and measured for RQD. In some instances, where the degree of fracturing varied over the length of the core, we divided the core into shorter lengths with similar characteristics and reported the RQD for each length.

FINDINGS

Our current borings encountered shale at depths consistent with the bearing elevation of the existing foundations and the 1965 and 2004 borings. The fractured argillaceous rock is only slightly weathered with hardness rated as very weak to weak. Based on the elevation of the rock, it appears that the existing foundations bear on the surface of the rock or were excavated up to about 2 feet into the rock.

All of the borings encountered fill. The fill is predominantly clay (A-7-6) except at the west abutment where the fill is predominantly silt and clay or silty clay (A-6a and A-6b). The fill extends to or slightly above the rock. Fill at B-002-0-18 and B-003-0-18 consisted of 1.5 feet of clay over coarse and fine sand. The sand fill extended to depths of 3 and 6 feet at B-002-0-18 and B-003-0-18, respectively. The fill at B-003-0-18 is probably backfill for the storm sewer in the median between the east barrier wall and the center pier, where the fill is underlain by a thin layer of residual clay weathered from the rock.

Groundwater was encountered at a depth of 10.5 feet in the silty clay fill at B-001-0-18. Groundwater was not encountered in the remaining borings at the time of field testing and prior to the start of coring.

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ANALYSIS AND RECOMMENDATIONS

FOUNDATIONS

We calculated bearing resistance based on the approach in the AASHTO LRFD Manual, Article 10.6.3.1.2a with strength parameters c' and ϕ' based on the RMR per Bieniawski (1989). Based on the RQD of the rock cores, we interpret the rock mass structure to be poor or not competent per the RMR system. This results in a recommended bearing resistance of 25 ksf. Eccentricity of the loads are also checked for the pier footings. Settlement is expected to be negligible as shown in the calculations.

Footings should bear at a depth similar to the existing bridge foundations except that any softened, decomposed rock encountered at bearing depth should be removed and any over-excavation required during construction should be filled with concrete. Key foundations at least 3 inches into weak, or very weak to weak shale.

If the new piers are close to the deeper excavations for the storm and sanitary sewers, the footings should be deepened such that the closest edge of footing is no closer than a line parallel to the deeper excavation determined by extending up from the nearest bottom edge of the sewer excavation at a 2V:1H ratio.

LATERAL EARTH PRESSURES

Lateral earth pressures for design of the abutments should assume active earth pressures. Use a lateral earth pressure coefficient of 0.3 (per AASHTO LRFD Manual, Article 3.11.5.3) and assume a unit weight for the backfill soil of 120 pcf (equivalent fluid density of 36 pcf). Surcharges result in a uniform lateral pressure on the wall equal to 0.3 times the vertical surcharge with its resultant acting at mid-height. To arrive at these values, we assumed the effect of compacted backfill and an effective friction angle for the backfill soil of 30 degrees.

The calculated recommended bearing resistance for abutment footings is 16 ksf per AASHTO LRFD Manual, Article 10.6.3.1.2a. Abutments are also checked for bearing resistance, eccentricity, sliding, and global stability. The calculations are included with the attachments.

EXCAVATIONS AND BACKFILL

Fill placed around foundations should be compacted in accordance with ODOT Construction and Materials Specifications (CMS) Item 503.08 except that clay fill placed around piers should be compacted in lifts to reduce water penetration and provide protection against weathering of the shale. Embankment fill should conform to the requirements of CMS Item 203.

Provide porous backfill behind abutments in accordance with ODOT CMS Item 518.05. Compact the porous stone in accordance with CMS Item 603.11D. Provide a geotechnical separating fabric conforming to CMS Item 712.09D (AOS ≤ 0.3 mm) between the porous backfill and sides of the excavation. Extend the fabric up to subgrade and laterally to the ends of the abutment.

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Excavations should conform to ODOT CMS Items 503.04 and 503.05. Where concrete is placed on shale and placement will be delayed for 24 hours or more, after obtaining the engineer's approval, place a mud mat to protect the shale. As an alternative leave the excavation at least 6 inches high and excavate to the final bearing surface when ready for inspection and concrete placement.

SEISMIC CONSIDERATIONS

Structural calculations that consider seismic effects should assume Site Class B with A_S=0.045g, S_{DS}=0.09g, and S_{D1}=0.035g in accordance with the AASHTO LRFD Bridge Design Specifications Sections 3.10.2.1, 3.10.3.1, 3.10.3.2, and 3.10.4.

GENERAL COMMENTS

This report was prepared in accordance with generally accepted geotechnical engineering practices at the time and location of the project. If the scope of the project changes from that described in this report, SME should review and validate the conclusions and recommendations of this report.

Our recommendations are based on a limited sampling of the soil and rock at this project site. Variations in soil and groundwater conditions may occur between or away from sampling locations. The nature and extent of the variations may not become evident until the time of construction. If conditions encountered during construction are not consistent with the anticipated subsurface conditions, SME should be contacted to evaluate the encountered subsurface conditions.

This report has been prepared solely for the use of our client and only for this project. Any use or reliance on this report by others without SME's express written consent will be at the sole risk of the user. SME is not responsible for the misinterpretation of our geotechnical data and recommendations by others.

Report prepared by:

Jalal Fatemi
Project Engineer

Report reviewed by:

STATE OF OHIO
ALAN JAMES ESSER
E-40092
REGISTERED
PROFESSIONAL ENGINEER

Alan J. Esser, PE, D.GE
Chief Consultant

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APPENDIX A

BORING LOCATION PLAN
BORING LOG TERMINOLOGY
BORING LOGS
TEST DATA AND ROCK CORE PHOTOS
EXISTING DATA
CALCULATIONS

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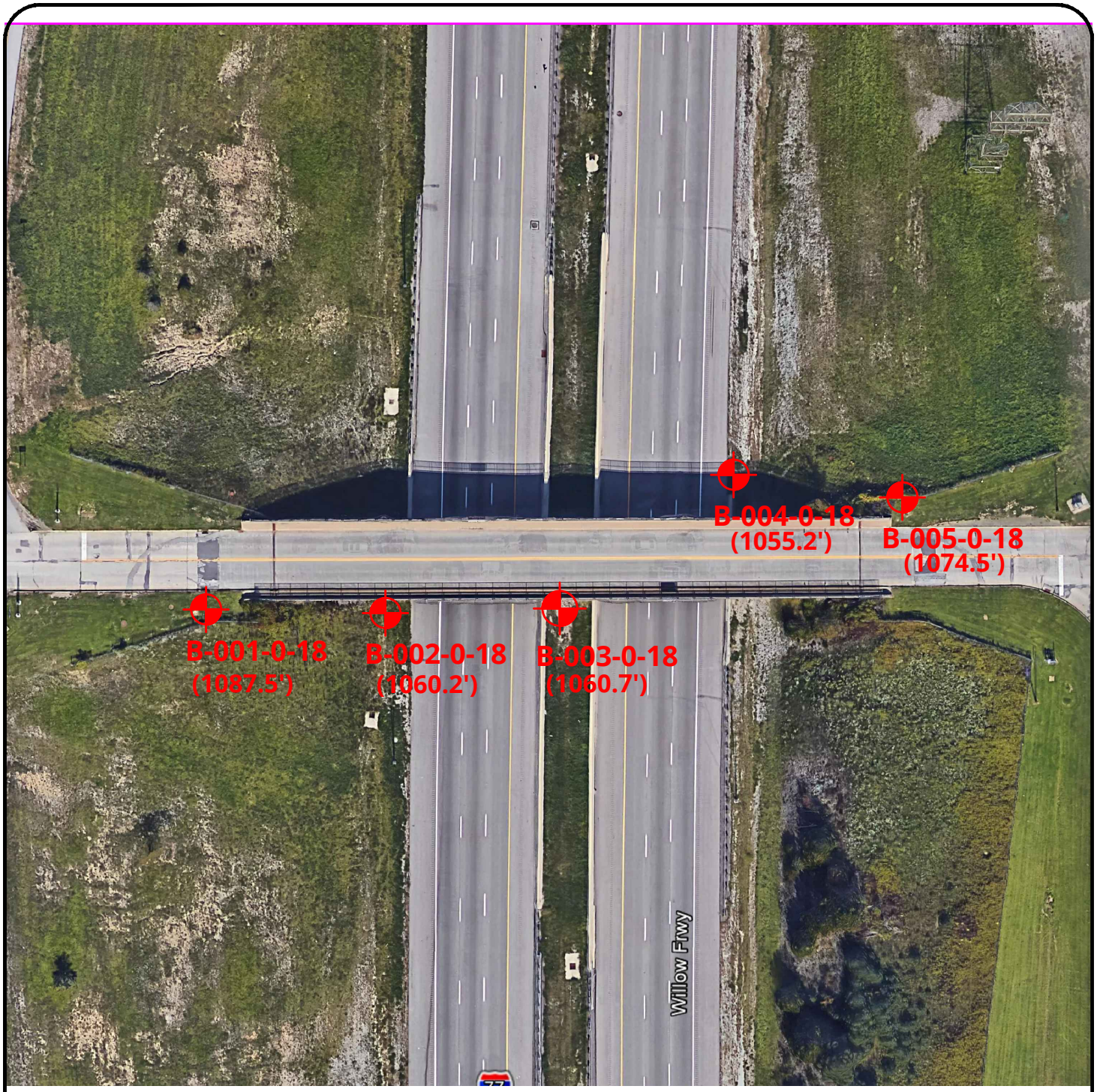

LEGEND

APPROXIMATE BORING LOCATION

NOTES:

1. DRAWING INFORMATION TAKEN FROM GOOGLE EARTH

NORTH

No.	Revision Date		
		Date	08/23/2018
	08/23/2018	Drawn By	JF
		Checked By	AJE
		Scale	NTS
		Project	078832.00

ODOT LPA: CUY-CR57-4.43
PID 106239
EAST WALLINGS ROAD
CLEVELAND, OH

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Figure No. 1

1) STRENGTH OF SOIL:

Non-Cohesive (granular) Soils - Compactness	
Description	**Blows Per Ft.**
Very Loose	≤ 4
Loose	5 – 10
Medium Dense	11 – 30
Dense	31 – 50
Very Dense	> 50

2) COLOR :

If a color is a uniform color throughout, the term is single, modified by an adjective such as light or dark. If the predominate color is shaded by a secondary color, the secondary color procedes the primary color. If two major and distinct colors are swirled throughout the soil, the colors are modified by the term "mottled"

3) PRIMARY COMPONENT

Use **DESCRIPTION** from ODOT Soil Classification Chart on Back

Cohesive (fine grained) Soils - Consistency

Description	Qu (TSF)	Blows Per Ft.	Hand Manipulation
Very Soft	<0.25	<2	Easily penetrates 2" by fist
Soft	0.25-0.5	2 - 4	Easily penetrates 2" by thumb
Medium Stiff	0.5-1.0	5 - 8	Penetrates by thumb with moderate effort
Stiff	1.0-2.0	9 - 15	Readily indents by thumb, but not penetrate
Very Stiff	2.0-4.0	16 - 30	Readily indents by thumbnail
Hard	>4.0	>30	Indent with difficulty by thumbnail

4) COMPONENT MODIFIERS:

Description	Percentage By Weight
Trace	0% - 10%
Little	10% - 20%
Some	20% - 35%
"And"	35% -50%

5) Soil Organic Content

Description	% by Weight
Slightly Organic	2% - 4%
Moderately Organic	4% - 10%
Highly Organic	> 10%

6) Relative Visual Moisture

Description	Criteria	
	Cohesive Soil	**Non-cohesive Soils**
Dry	Powdery; Cannot be rolled; Water content well below the plastic limit	No moisture present
Damp	Leaves very little moisture when pressed between fingers; Crumbles at or before rolled to $^1/_8$"; Water content below plastic limit	Internal moisture, but no to little surface moisture
Moist	Leaves small amounts of moisture when pressed between fingers; Rolled to $^1/_8$" or smaller before crumbling; Water content above plastic limit to -3% of the liquid limit	Free water on surface, moist (shiny) appearance
Wet	Very mushy; Rolled multiple times to $^1/_8$" or smaller before crumbles; Near or above the liquid limit	Voids filled with free water, can be poured from split spoon.

CLASSIFICATION OF SOILS

Ohio Department of Transportation

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

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

PROJECT: CUY-77-04.79	DRILLING FIRM / OPERATOR: SME / RH/RM	DRILL RIG: SME CME-55 TRUCK	STATION / OFFSET: 93+97, 24' RT.
TYPE: BRIDGE	SAMPLING FIRM / LOGGER: SME / JF	HAMMER: CME AUTOMATIC	ALIGNMENT: CL CONST. WALLINGS RD.
PID: 106239 SFN: 1805975	DRILLING METHOD: 3.75" HSA / NWL	CALIBRATION DATE: 4/5/17	ELEVATION: 1087.5 (MSL) EOB: 24.0 ft.
START: 7/25/18 END: 7/25/18	SAMPLING METHOD: SPT/NWL	ENERGY RATIO (%): 81.3	LAT / LONG: 41.344775, -81.650462

EXPLORATION ID B-001-0-18

PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
	1087.5																	
2" TOPSOIL (Drillers Description)	1087.4																	
VERY STIFF, BROWN WITH GRAY, **SILTY CLAY**, TRACE SAND, **FILL**, DAMP	1086.0	1	6 9 12	28	100	SS-1	-	-	-	-	-	-	-	-	-	10	A-6b (V)	
HARD, BROWN WITH GRAY, **SILT AND CLAY**, LITTLE SAND, TRACE GRAVEL, **FILL**, MOIST	1084.5	2 3	8 5 5	14	-	SS-2	4.50	21	5	5	35	34	33	20	13	14	A-6a (8)	
MEDIUM STIFF, BROWN WITH GRAY, **SILTY CLAY**, TRACE SAND, TRACE GRAVEL, TRACE CINDERS, **FILL**, MOIST	1083.0	4	3 4 3	9	33	SS-3	-	-	-	-	-	-	-	-	-	20	A-6b (V)	
VERY STIFF, BROWN WITH GRAY, **SILTY CLAY**, LITTLE SAND, TRACE GRAVEL, **FILL**, MOIST	1081.5	5 6	1 1 3	5	33	SS-4	2.50	16	5	5	36	38	35	19	16	17	A-6b (10)	
STIFF, BROWN, **SILTY CLAY**, SOME SAND, TRACE GRAVEL, **FILL**, MOIST	1080.0	7	1 1 1	3	33	SS-5	1.25	-	-	-	-	-	-	-	-	18	A-6b (V)	
SOFT TO MEDIUM STIFF, BROWN, **SILT AND CLAY**, SOME SAND, TRACE GRAVEL, **FILL**, MOIST	1078.0	8 9	1 1 1	3	67	SS-6	0.50	22	5	5	37	31	31	19	12	19	A-6a (7)	
STIFF TO VERY STIFF, BROWN AND GRAY, **SILTY CLAY**, SOME SAND, TRACE GRAVEL, **FILL**, WET	W 1077.0	10 11	1 1 2	4	100	SS-7	2.00	-	-	-	-	-	-	-	-	18	A-6b (V)	
VERY STIFF, BROWN, **SILTY CLAY**, SOME SAND, TRACE GRAVEL, **FILL**, DAMP	1075.5	12																
SHALE, GRAY WITH BROWN, UNWEATHERED TO SLIGHTLY WEATHERED, VERY WEAK TO WEAK; RQD 8%, REC 100%.	1074.5	TR 13 14	10 25 42	91	100	SS-8A	-	-	-	-	-	-	-	-	-	16	A-6b (V)	
						SS-8B	-	-	-	-	-	-	-	-	-	-	Rock (V)	
@ 18 feet ; Qu = 700 psi		15 16 17 18 19	8		100	NWL-9											CORE	
@ 21 feet ; Qu = 1170 psi		20 21 22 23																
	1063.5	EOB 24																

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 11/10/21 12:15 - \\SME-INC\PZ\ARCHIVES\PROJ\078000\078832.00\PROJECT DATA\GINT\078832.00-ODOT LOGS.GPJ

PROJECT: CUY-77-04.79	DRILLING FIRM / OPERATOR: SME / RH/RM	DRILL RIG: SME CME-55 TRUCK	STATION / OFFSET: 94+81, 27' RT.
TYPE: BRIDGE	SAMPLING FIRM / LOGGER: SME / JF	HAMMER: CME AUTOMATIC	ALIGNMENT: CL CONST. WALLINGS RD.
PID: 106239 SFN: 1805975	DRILLING METHOD: 3.25" HSA / NWL	CALIBRATION DATE: 4/5/17	ELEVATION: 1060.2 (MSL) EOB: 19.0 ft.
START: 7/26/18 END: 7/26/18	SAMPLING METHOD: SPT/NWL	ENERGY RATIO (%): 81.3	LAT / LONG: 41.344767, -81.650156

EXPLORATION ID B-002-0-18

PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
	1060.2																	
HARD, GRAY WITH BROWN, **CLAY**, SOME SHALE, SOME SAND, **FILL**, DAMP	1058.7	1	7 11 12	31	100	SS-1	-	-	-	-	-	-	-	-	-	8	A-7-6 (V)	
MEDIUM DENSE, BROWN, **FINE SAND**, **FILL**, MOIST (F-S)	1057.2	2	7 4 11	20	100	SS-2	-	-	-	-	-	-	-	-	-	7	A-3 (V)	
SHALE, GRAY, UNWEATHERED, VERY WEAK.		TR 3	20 50/6"	-	83	SS-3	-	-	-	-	-	-	-	-	-	5	Rock (V)	
		4																
		5	50/6"	-	100	SS-4	-	-	-	-	-	-	-	-	-	5	Rock (V)	
		6																
		7																
		8																
	1051.2	9																
SHALE, GRAY, UNWEATHERED, VERY WEAK TO WEAK; RQD 23%, REC 100%.		10																
		11																
@ 11.5 feet ; Qu = 1750, 1880 psi		12																
		13																
		14	23		100	NWL-5											CORE	
		15																
		16																
@ 16 feet ; Qu = 560 psi		17																
		18																
	1041.2	EOB 19																

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 11/10/21 12:15 - \\SME-INC\PZ\ARCHIVES\PROJ\078000\078832.00\PROJECT DATA\GINT\078832.00-ODOT LOGS.GPJ

PROJECT: CUY-77-04.79	DRILLING FIRM / OPERATOR: SME / RH/RM	DRILL RIG: SME CME-55 TRUCK	STATION / OFFSET: 95+64, 26' RT.
TYPE: BRIDGE	SAMPLING FIRM / LOGGER: SME / JF	HAMMER: CME AUTOMATIC	ALIGNMENT: CL CONST. WALLINGS RD.
PID: 106239 SFN: 1805975	DRILLING METHOD: 3.25" HSA / NWL	CALIBRATION DATE: 4/5/17	ELEVATION: 1060.7 (MSL) EOB: 20.0 ft.
START: 7/26/18 END: 7/26/18	SAMPLING METHOD: SPT/NWL	ENERGY RATIO (%): 81.3	LAT / LONG: 41.344770, -81.666523

EXPLORATION ID B-003-0-18

PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
STIFF, GRAY, **CLAY**, SOME SHALE, **FILL**, DAMP	1060.7 / 1059.2	1	3 3 6	12	33	SS-1	-	-	-	-	-	-	-	-	-	11	A-7-6 (V)	
MEDIUM DENSE, BROWN, **COARSE AND FINE SAND**, LITTLE GRAVEL, **FILL**, MOIST	1054.7	2 3 4 5	8 9 12	28	100	SS-2	-	-	-	-	-	-	-	-	-	4	A-3a (V)	
			7 4 6	14	33	SS-3	-	-	-	-	-	-	-	-	-	5	A-3a (V)	
			4 5 6	15	100	SS-4	-	-	-	-	-	-	-	-	-	10	A-3a (V)	
HARD, GRAY, **CLAY**, SOME SHALE (RESIDUAL), DAMP	1054.2	6 TR	23 50/3"	-	100	SS-5A	-	-	-	-	-	-	-	-	-	27	A-7-6 (V)	
SHALE, GRAY, UNWEATHERED, VERY WEAK.	1050.7	7 8 9 10				SS-5B	-	-	-	-	-	-	-	-	-	-	Rock (V)	
SHALE, GRAY, UNWEATHERED, VERY WEAK; REC 100%. @ 12 feet ; Qu = 640 psi 10'-15' : RQD = 4% 15'-20' : RQD = 58% @ 15 feet ; Qu = 720 psi	1040.7	11 12 13 14 15 16 17 18 19 20 EOB	31		100	NWL-6										5	CORE	

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 11/10/21 12:15 - \\SME-INC\PZ\ARCHIVES\PROJ\078000\078832.00\PROJECT DATA\GINT\078832.00-ODOT LOGS.GPJ

PROJECT: CUY-77-04.79	DRILLING FIRM / OPERATOR: SME / RH/RM	DRILL RIG: SME CME-55 TRUCK	STATION / OFFSET: 96+47, 36' LT.
TYPE: BRIDGE	SAMPLING FIRM / LOGGER: SME / JF	HAMMER: CME AUTOMATIC	ALIGNMENT: CL CONST. WALLINGS RD.
PID: 106239 SFN: 1805975	DRILLING METHOD: 3.25" HSA / NWL	CALIBRATION DATE: 4/5/17	ELEVATION: 1055.2 (MSL) EOB: 19.0 ft.
START: 7/27/18 END: 7/27/18	SAMPLING METHOD: SPT/NWL	ENERGY RATIO (%): 81.3	LAT / LONG: 41.344910, -81.665933

EXPLORATION ID B-004-0-18

PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
STIFF, GRAY, **CLAY**, SOME SHALE, LITTLE SAND, TRACE GRAVEL, **FILL**, MOIST	1055.2 / 1053.7	TR, 1	1 1 8	12	56	SS-1	-	-	-	-	-	-	-	-	-	12	A-7-6 (V)	
SHALE, GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, VERY WEAK.	1052.2	2	18 27 38	88	100	SS-2	-	-	-	-	-	-	-	-	-	6	Rock (V)	
SHALE, GRAY, UNWEATHERED, VERY WEAK.	1050.7	3	50/6"	-	100	SS-3	-	-	-	-	-	-	-	-	-	5	Rock (V)	
SHALE, GRAY, UNWEATHERED, WEAK.	1046.2	4, 5, 6, 7, 8	50/4"	-	100	SS-4	-	-	-	-	-	-	-	-	-	6	Rock (V)	
SHALE, GRAY, UNWEATHERED, WEAK; REC 100%. 9'-14' : RQD = 30% 14'-19' : RQD = 59%	1036.2	9, 10, 11, 12, 13, 14, 15, 16, 17, 18, EOB 19	44		100	NWL-5											CORE	

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 11/10/21 12:15 - \\SME-INC\PZ\ARCHIVES\PROJ\078000\078832.00\PROJECT DATA\GINT\078832.00-ODOT LOGS.GPJ

PROJECT: CUY-77-04.79	DRILLING FIRM / OPERATOR: SME / RH/RM	DRILL RIG: SME CME-55 TRUCK	STATION / OFFSET: 97+26, 24' LT.
TYPE: BRIDGE	SAMPLING FIRM / LOGGER: SME / JF	HAMMER: CME AUTOMATIC	ALIGNMENT: CL CONST. WALLINGS RD.
PID: 106239 SFN: 1805975	DRILLING METHOD: 3.75" HSA / NWL	CALIBRATION DATE: 4/5/17	ELEVATION: 1074.5 (MSL) EOB: 28.5 ft.
START: 7/25/18 END: 7/25/18	SAMPLING METHOD: SPT/NWL	ENERGY RATIO (%): 81.3	LAT / LONG: 41.344942, -81.666221

EXPLORATION ID B-005-0-18

PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
	1074.5																	
HARD, BROWN AND GRAY, **CLAY**, TRACE SILT, TRACE SAND, AND TRACE GRAVEL, **FILL**, DAMP	1071.5	1	4 5 5	14	100	SS-1	4.50	-	-	-	-	-	-	-	-	13	A-7-6 (V)	
		2	4 4 6	14	100	SS-2	4.00	-	-	-	-	-	-	-	-	16	A-7-6 (V)	
VERY STIFF TO HARD, BROWN WITH GRAY, **SILT AND CLAY**, TRACE SAND, AND TRACE GRAVEL, **FILL**, DAMP	1070.0	3	4 4 5	12	100	SS-3	4.50	14	4	3	38	41	34	20	14	15	A-6a (10)	
HARD, BROWN, **CLAY**, SOME SAND, AND TRACE SILT, **FILL**, DAMP	1068.5	4	4 2 5	9	100	SS-4	2.00	-	-	-	-	-	-	-	-	15	A-7-6 (V)	
STIFF TO VERY STIFF, BROWN AND GRAY, **CLAY**, LITTLE GRAVEL, AND TRACE SAND, **POSSIBLE FILL**, MOIST	1067.0	5	2 2 4	8	100	SS-5	4.00	10	3	2	37	48	45	21	24	16	A-7-6 (15)	
VERY STIFF, BROWN AND GRAY, **CLAY**, TRACE SILT, TRACE SAND, AND TRACE GRAVEL, **POSSIBLE FILL**, MOIST	1065.5	6	2 4 5	12	100	SS-6	3.00	-	-	-	-	-	-	-	-	25	A-7-6 (V)	
VERY STIFF, BROWN AND GRAY, **SILTY CLAY**, TRACE SAND, **POSSIBLE FILL**, DAMP	1062.5	7	2 4 5	12	100	SS-7	3.50	-	-	-	-	-	-	-	-	17	A-6b (V)	
		8																
		9																
		10																
		11																
HARD, BROWN AND GRAY, **CLAY**, SOME SHALE (RESIDUAL), DAMP	1060.5	12	6 14 18	43	100	SS-8A	4.50	-	-	-	-	-	-	-	-	14	A-6b (V)	
		13				SS-8B	-	-	-	-	-	-	-	-	-	-	A-7-6 (V)	
		14																
SHALE, GRAY, UNWEATHERED, VERY WEAK.	1057.5	15	18 21 30	69	100	SS-9	-	-	-	-	-	-	-	-	-	8	Rock (V)	
		16																
		17	56/6"	-	100	SS-10	-	-	-	-	-	-	-	-	-	10	Rock (V)	
SHALE, GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, WEAK; RQD 0%, REC 100%.		18																
		19																
@ 20 feet ; Qu =1420 psi		20																
		21																
		22																
		23	0		100	NWL-11											CORE	
@ 23.5 feet ; Qu = 980 psi		24																
		25																
		26																
		27																
	1046.0	28																

TR

EOB

NOTES: NONE

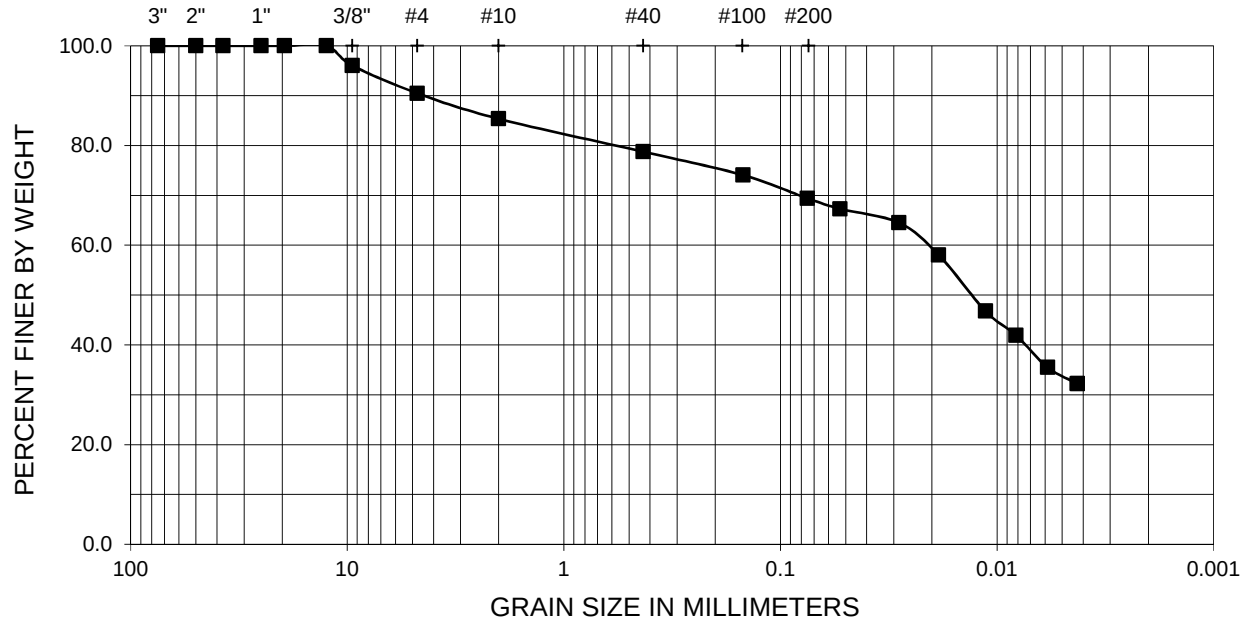
ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 11/10/21 12:15 - \\SME-INC\PZ\ARCHIVES\PROJ\078000\078832.00\PROJECT DATA\GINT\078832.00-ODOT LOGS.GPJ

9375 CHILLICOTHE ROAD, KIRTLAND, OH 44094
PHONE: 440-256-6500 FAX: 440-256-6507

PARTICLE SIZE ANALYSIS WITH HYDROMETER AASHTO T88

PROJECT INFORMATION		SAMPLE INFORMATION		
Project:	ODOT LPA: CUY-CR57-4.43	***ASTM Description***	Sandy LEAN CLAY	CL
Location:	Broadview Heights, Ohio			
Project #:	076220.00	***OHIO Modified AASHTO***	Silt and clay	A-6a (8)
Test Date:	August 17, 2018			
Sample #:	SB	***Sample Location***	B-1; 1.5' - 3'	



SIEVE ANALYSIS

Sieve	Sieve size, mm	Percent Passing
3"	75	100.0
2"	50	100.0
1-1/2"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	96.1
#4	4.75	90.5
#10	2	85.4
#40	0.43	78.8
#100	0.15	74.1
#200	0.074	69.4
#270	0.053	67.3

DISPERSION

Device	ASTM D422, Type A
Agent	Sodium Hexametaphosphate

HYDROMETER ANALYSIS

Particle Size	Percent Passing
0.074 mm	69.4
0.053 mm	67.3
0.005 mm	33.6
0.0042 mm	32.3

ATTERBERG LIMITS

LIQUID LIMIT	33
PLASTIC LIMIT	20
PLASTICITY INDEX	13

ODOT SIZE DIVISIONS

Division	Percent
Gravel, GR	14.6
Coarse Sand, CS	6.6
Fine Sand, FS	9.4
Silt,SI	35.8
Clay, CL	33.6

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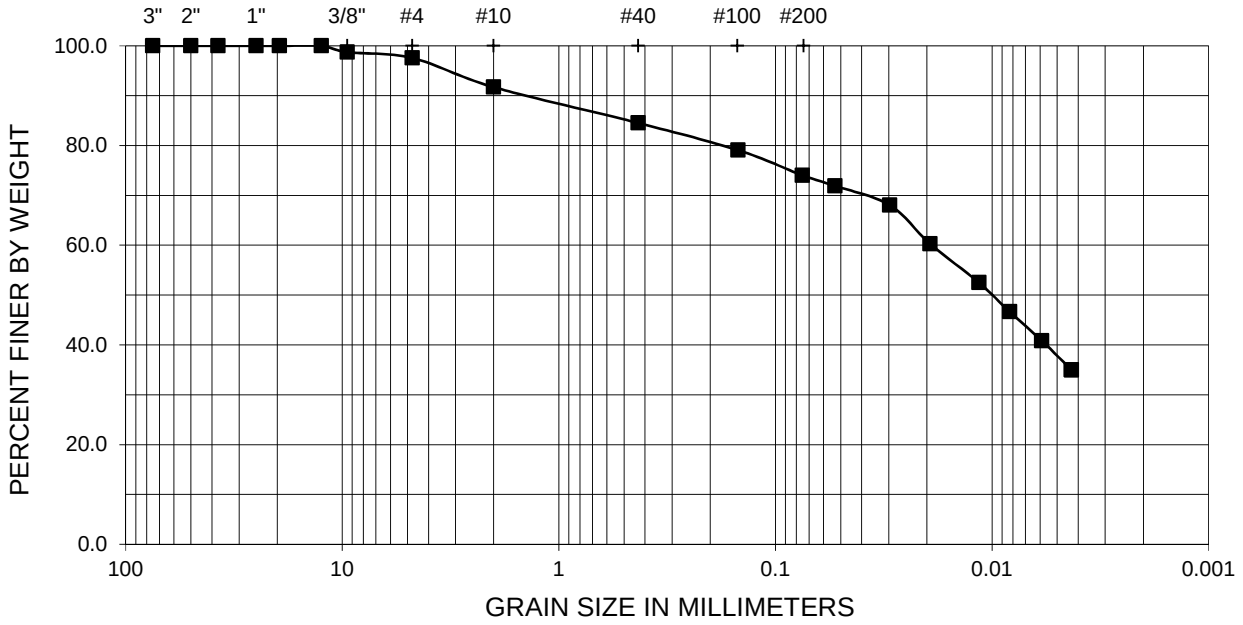
PARTICLE SIZE ANALYSIS WITH HYDROMETER AASHTO T88

PROJECT INFORMATION		SAMPLE INFORMATION		
Project:	ODOT LPA: CUY-CR57-4.43	***ASTM Description***	LEAN CLAY with sand	CL
Location:	Broadview Heights, Ohio			
Project #:	076220.00	***OHIO Modified AASHTO***	SILTY CLAY	A-6b (10)
Test Date:	August 17, 2018			
Sample #:	SB	***Sample Location***	B-1; 4.5' - 6'	



SIEVE ANALYSIS

Sieve	Sieve size, mm	Percent Passing
3"	75	100.0
2"	50	100.0
1-1/2"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	98.7
#4	4.75	97.6
#10	2	91.7
#40	0.43	84.5
#100	0.15	79.1
#200	0.074	74.0
#270	0.053	71.9

DISPERSION

Device	ASTM D422, Type A
Agent	Sodium Hexametaphosphate

HYDROMETER ANALYSIS

Particle Size	Percent Passing
0.074 mm	74.0
0.053 mm	71.9
0.005 mm	37.9
0.0043 mm	35.0

ATTERBERG LIMITS

LIQUID LIMIT	35
PLASTIC LIMIT	19
PLASTICITY INDEX	16

ODOT SIZE DIVISIONS

Division	Percent
Gravel, GR	8.3
Coarse Sand, CS	7.2
Fine Sand, FS	10.5
Silt,SI	36.1
Clay, CL	37.9

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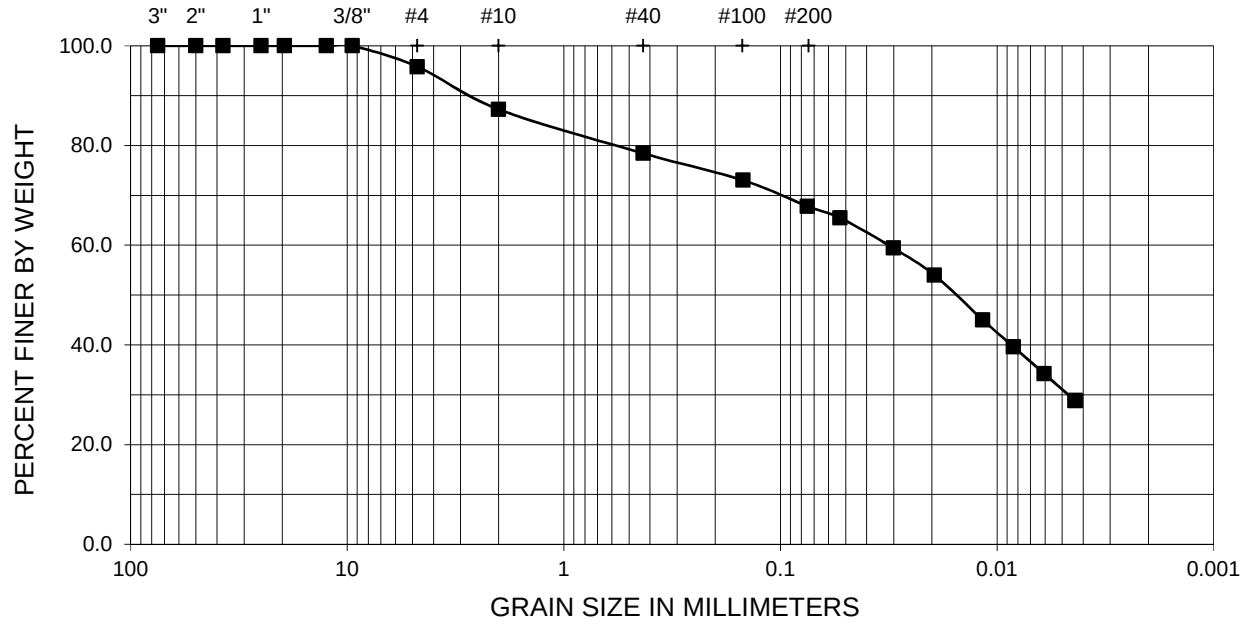
PARTICLE SIZE ANALYSIS WITH HYDROMETER AASHTO T88

PROJECT INFORMATION		SAMPLE INFORMATION		
Project:	ODOT LPA: CUY-CR57-4.43	***ASTM Description***	Brown SANDY LEAN CLAY	CL
Location:	Broadview Heights, Ohio			
Project #:	076220.00	***OHIO Modified AASHTO***	Brown SILT and CLAY	A-6a (7)
Test Date:	August 17, 2018			
Sample #:	SB	***Sample Location***	B-1; 7.5' - 9'	



SIEVE ANALYSIS

Sieve	Sieve size, mm	Percent Passing
3"	75	100.0
2"	50	100.0
1-1/2"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	95.8
#10	2	87.3
#40	0.43	78.4
#100	0.15	73.0
#200	0.074	67.8
#270	0.053	65.5

DISPERSION

Device	ASTM D422, Type A
Agent	Sodium Hexametaphosphate

HYDROMETER ANALYSIS

Particle Size	Percent Passing
0.074 mm	67.8
0.053 mm	65.5
0.005 mm	31.0
0.0044 mm	28.8

ATTERBERG LIMITS

LIQUID LIMIT	31
PLASTIC LIMIT	19
PLASTICITY INDEX	12

ODOT SIZE DIVISIONS

Division	Percent
Gravel, GR	12.7
Coarse Sand, CS	8.8
Fine Sand, FS	10.6
Silt,SI	36.8
Clay, CL	31.0

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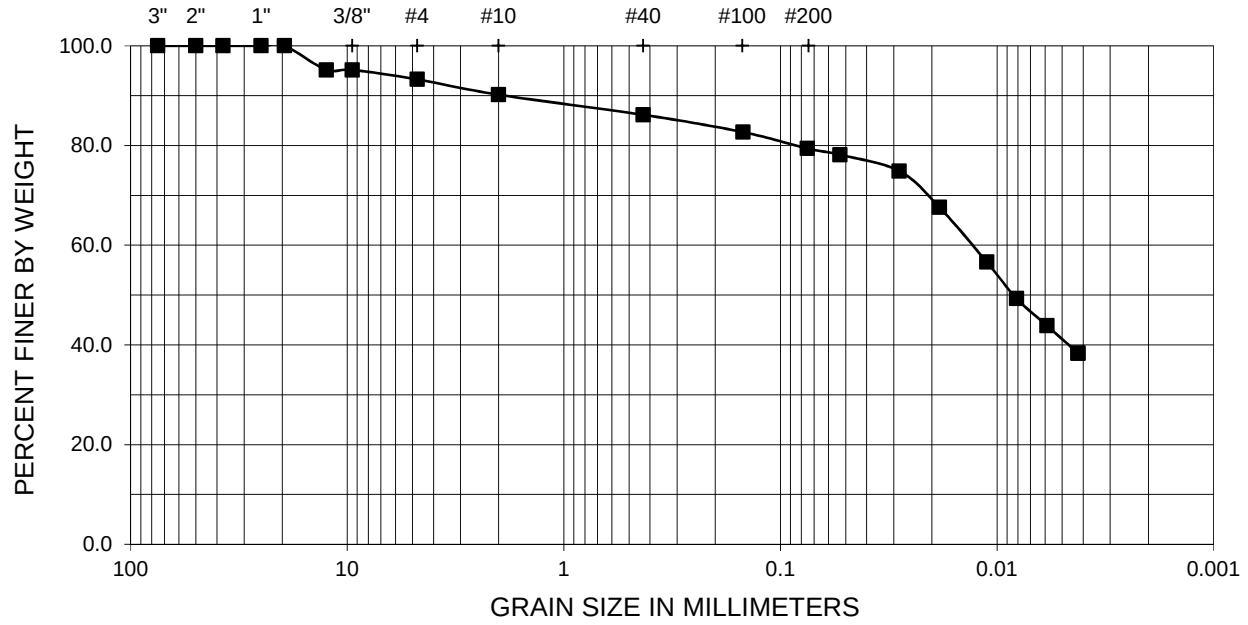
PARTICLE SIZE ANALYSIS WITH HYDROMETER AASHTO T88

PROJECT INFORMATION		SAMPLE INFORMATION		
Project:	ODOT LPA: CUY-CR57-4.43	***ASTM Description***	Brown LEAN CLAY with Sand	CL
Location:	Broadview Heights, Ohio			
Project #:	076220.00	***OHIO Modified AASHTO***	Brown SILT and CLAY	A-6a (10)
Test Date:	August 17, 2018			
Sample #:	SB	***Sample Location***	B-5; 3' - 4.5'	



SIEVE ANALYSIS

Sieve	Sieve size, mm	Percent Passing
3"	75	100.0
2"	50	100.0
1-1/2"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	95.2
#4	4.75	93.3
#10	2	90.2
#40	0.43	86.2
#100	0.15	82.7
#200	0.074	79.4
#270	0.053	78.1

DISPERSION

Device	ASTM D422, Type A
Agent	Sodium Hexametaphosphate

HYDROMETER ANALYSIS

Particle Size	Percent Passing
0.074 mm	79.4
0.053 mm	78.1
0.005 mm	41.2
0.0042 mm	38.4

ATTERBERG LIMITS

LIQUID LIMIT	34
PLASTIC LIMIT	20
PLASTICITY INDEX	14

ODOT SIZE DIVISIONS

Division	Percent
Gravel, GR	9.8
Coarse Sand, CS	4.0
Fine Sand, FS	6.8
Silt,SI	38.2
Clay, CL	41.2

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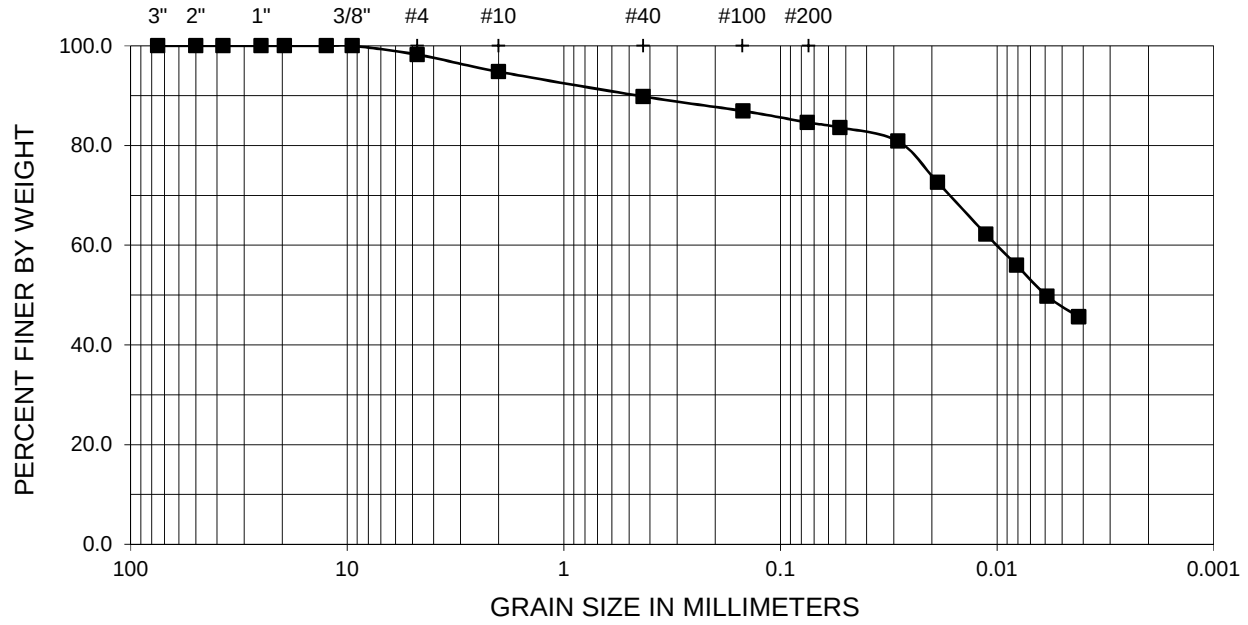
PARTICLE SIZE ANALYSIS WITH HYDROMETER AASHTO T88

PROJECT INFORMATION		SAMPLE INFORMATION		
Project:	ODOT LPA: CUY-CR57-4.43	***ASTM Description***	lean clay with sand	CL
Location:	Broadview Heights, Ohio			
Project #:	076220.00	***OHIO Modified AASHTO***	CLAY	A-7-6 (15)
Test Date:	August 17, 2018			
Sample #:	SB	***Sample Location***	B-5; 6' - 7.5'	



SIEVE ANALYSIS

Sieve	Sieve size, mm	Percent Passing
3"	75	100.0
2"	50	100.0
1-1/2"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	98.3
#10	2	94.8
#40	0.43	89.8
#100	0.15	86.9
#200	0.074	84.7
#270	0.053	83.6

DISPERSION

Device	ASTM D422, Type A
Agent	Sodium Hexametaphosphate

HYDROMETER ANALYSIS

Particle Size	Percent Passing
0.074 mm	84.7
0.053 mm	83.6
0.005 mm	47.7
0.0042 mm	45.6

ATTERBERG LIMITS

LIQUID LIMIT	45
PLASTIC LIMIT	21
PLASTICITY INDEX	24

ODOT SIZE DIVISIONS

Division	Percent
Gravel, GR	5.2
Coarse Sand, CS	5.0
Fine Sand, FS	5.2
Silt,SI	37.0
Clay, CL	47.7

9375 CHILLICOTHE ROAD, KIRTLAND, OH 44094
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Compressive Strength of Intact Rock Core Specimens
ASTM D7012

PROJECT	ODOT LPA: CUY-CR57-4.43
LOCATION	Broadview Heights, Ohio
DATE	August 22, 2018
PROJECT #	078832.00
CLIENT	ODOT

SAMPLE	**1**	**2**	**3**	**4**
SAMPLE LOCATION	B-1; 21'	B-3; 12'	B-3; 15'	
DATE TESTED	August 22, 2018	August 22, 2018	August 22, 2018	
ORIGINAL LENGTH, in	---	---	---	
CAPPED LENGTH, in	4.14	3.98	4.46	
DIAMETER, in	1.98	1.99	1.98	
AREA, sq. in.	3.09	3.09	3.09	
LOAD AT FAILURE, lbs.	3,610	1,985	2,220	
GROSS UNIT STRESS, psi	1,168	641	718	
LENGTH/DIAMETER RATIO	2.1	2.0	2.2	
UNIT STRESS CORRECTED, psi	**1,170**	**640**	**720**	
MOISTURE CONDITION WHEN TESTED	MOIST	MOIST	MOIST	

REMARKS: Samples tested do not meet the requirements for sample preparation per ASTM D4543

9375 CHILLICOTHE ROAD, KIRTLAND, OH 44094
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POINT LOAD STRENGTH INDEX OF ROCK FOR NX CORES, ASTM D5731

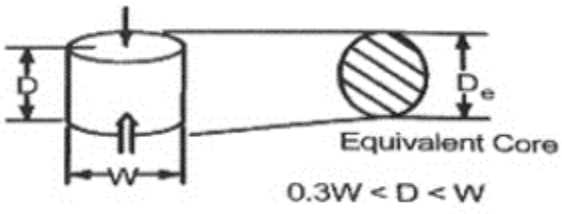
Project CUY-CR57-4.43 (Wallings Road)
Location Broadview Heights, Ohio
Project # 076220.00
Test Date 8/21/2018
Date obtained: 7/25/2018
Obtained by: RH

Sample Description SHALE
Storage Environment: Moist room
Test Apparatus: Tecnotest Point Load Tester
Last calibration: Apr-2018
Tested by: SM

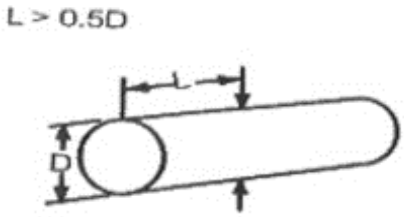
Test No.	Sample Location	Sample Depth	Test Type	Width, W (mm)	Diameter, D (mm)	Load, P (N)	D_e^2, (mm^2)	D_e, (mm)	I_s, (MPa)	F	$I_{s(50)}$, (Mpa)	S_c (MPa)	S_c (psi)
1	B-1	18'	A	50.27	34.65	480	2217	47.1	0.22	0.97	0.21	4.83	700
2	B-2	11.5'	A	50.47	46.25	1495	2972	54.5	0.50	1.04	0.525	12.08	1750
3	B-2	11.5'	A	50.34	39.12	1415	2507	50.1	0.56	1.00	0.565	12.99	1880
4	B-2	16'	A	50.29	34.34	380	2199	46.9	0.17	0.97	0.167	3.85	560
5	B-5	20'	A	50.39	23.95	740	1537	39.2	0.48	0.89	0.426	9.81	1420
6	B-5	23.5'	A	50.09	44.02	805	2807	53.0	0.29	1.03	0.295	6.79	980
7													
8													
9													
10													

REMARKS: K = 23, F = (De/50)^0.5, $De^2 = 4WD/\pi$

Test Types:

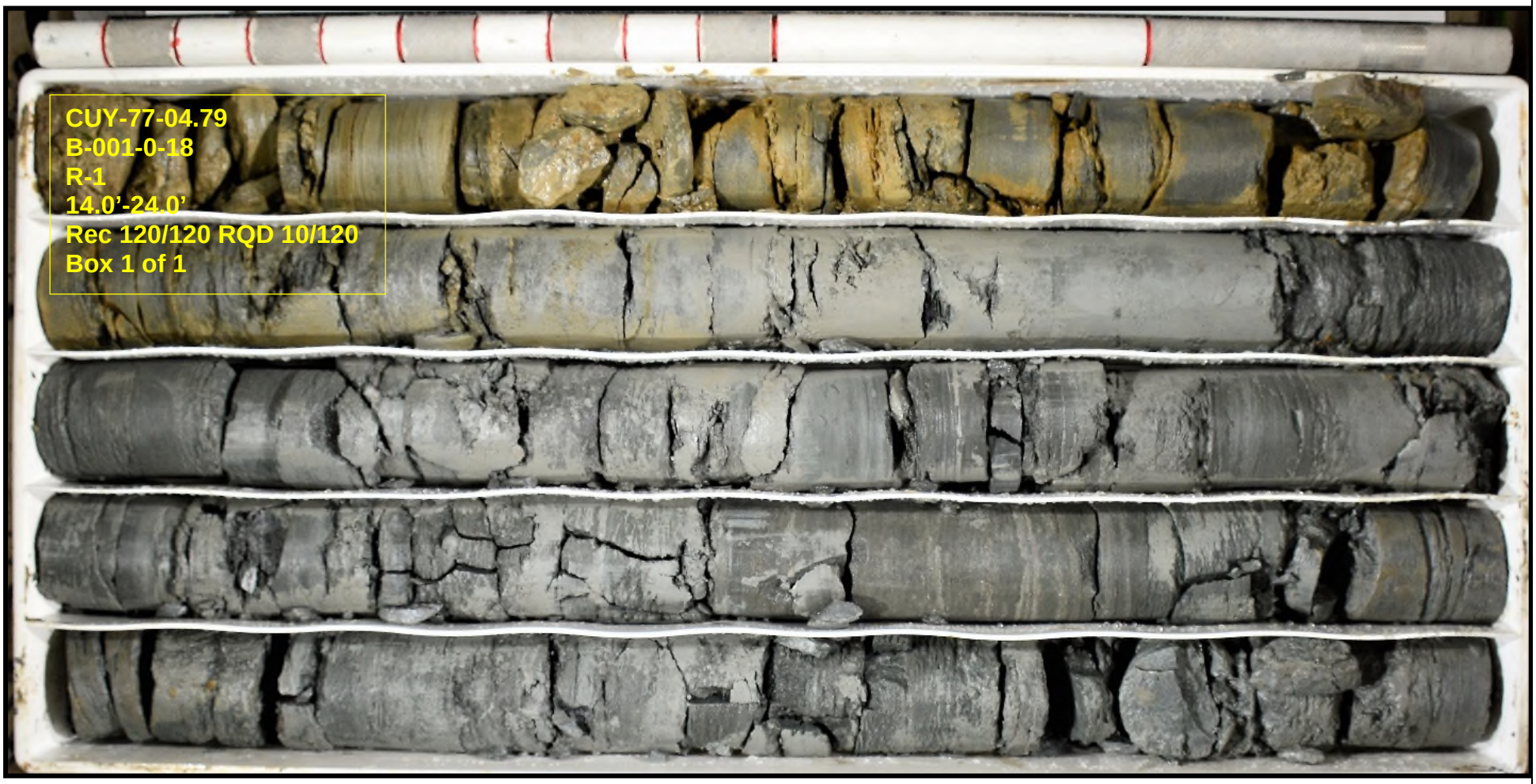


A - Axial



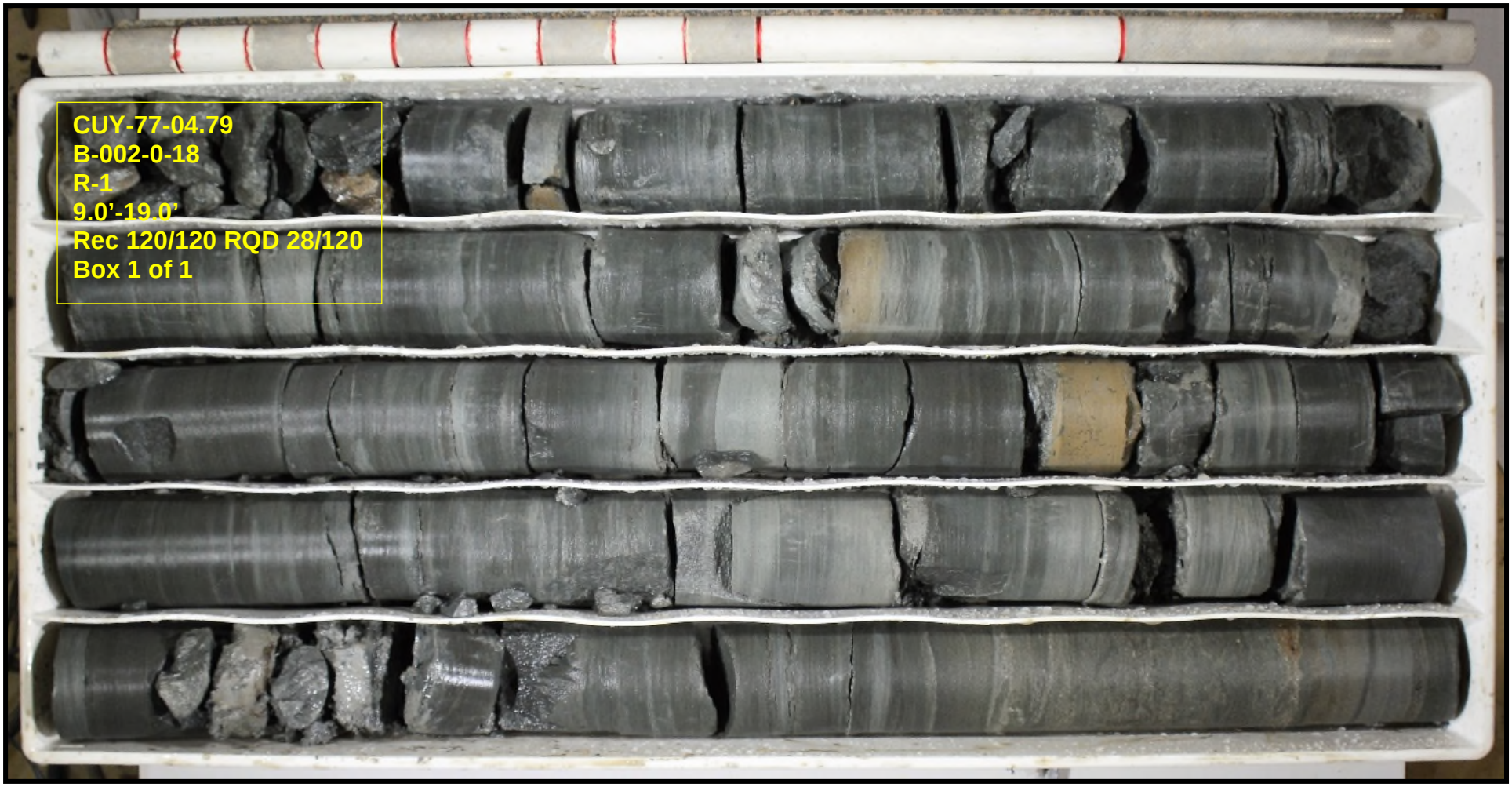
D - Diametral

B-001-0-18



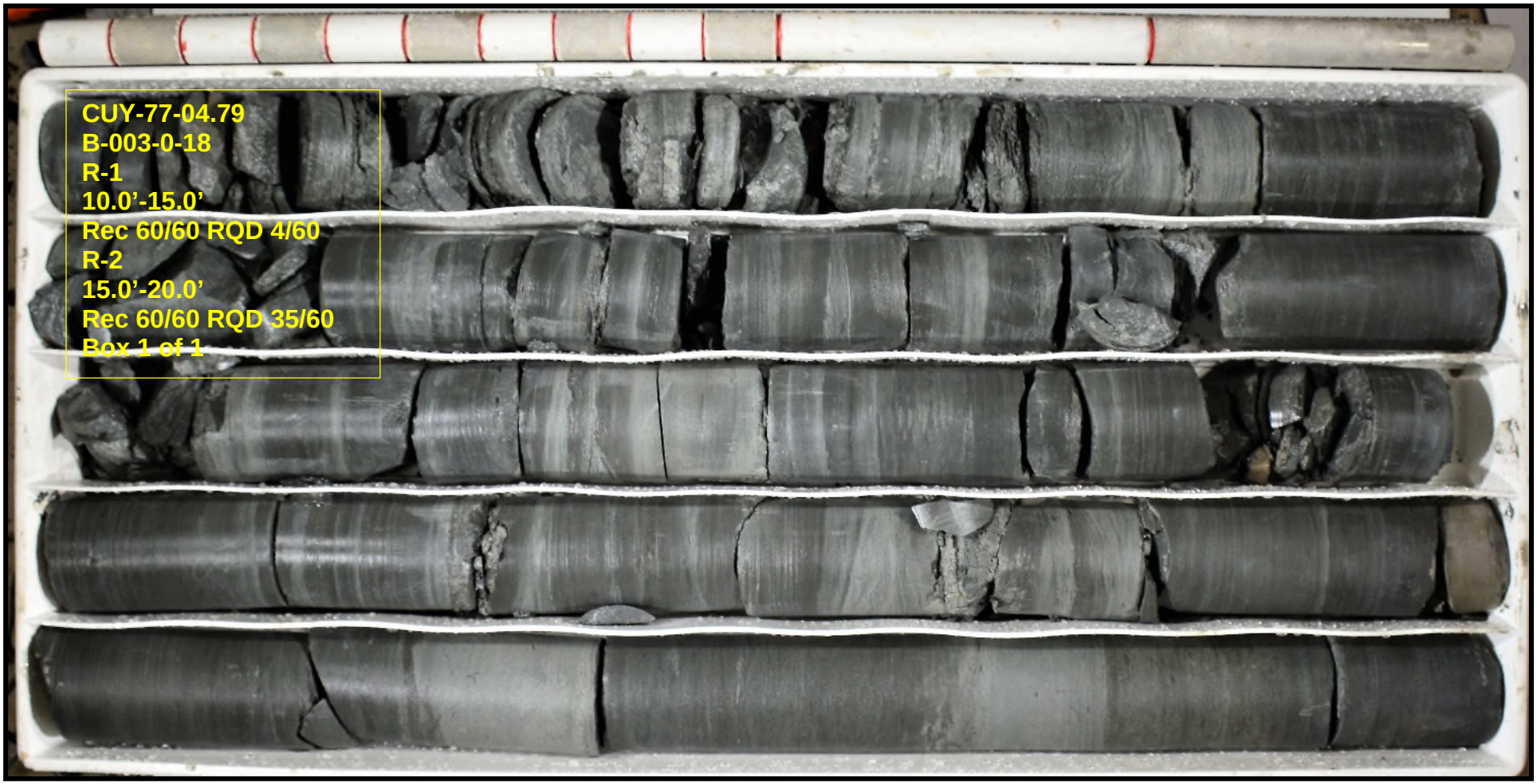
Run#:	Depth		Recovery		RQD	
NX-1	14.0’	24.0’	120/120	100%	10/120	8%
CUY-77-04.79 PID 106239						

B-002-0-18



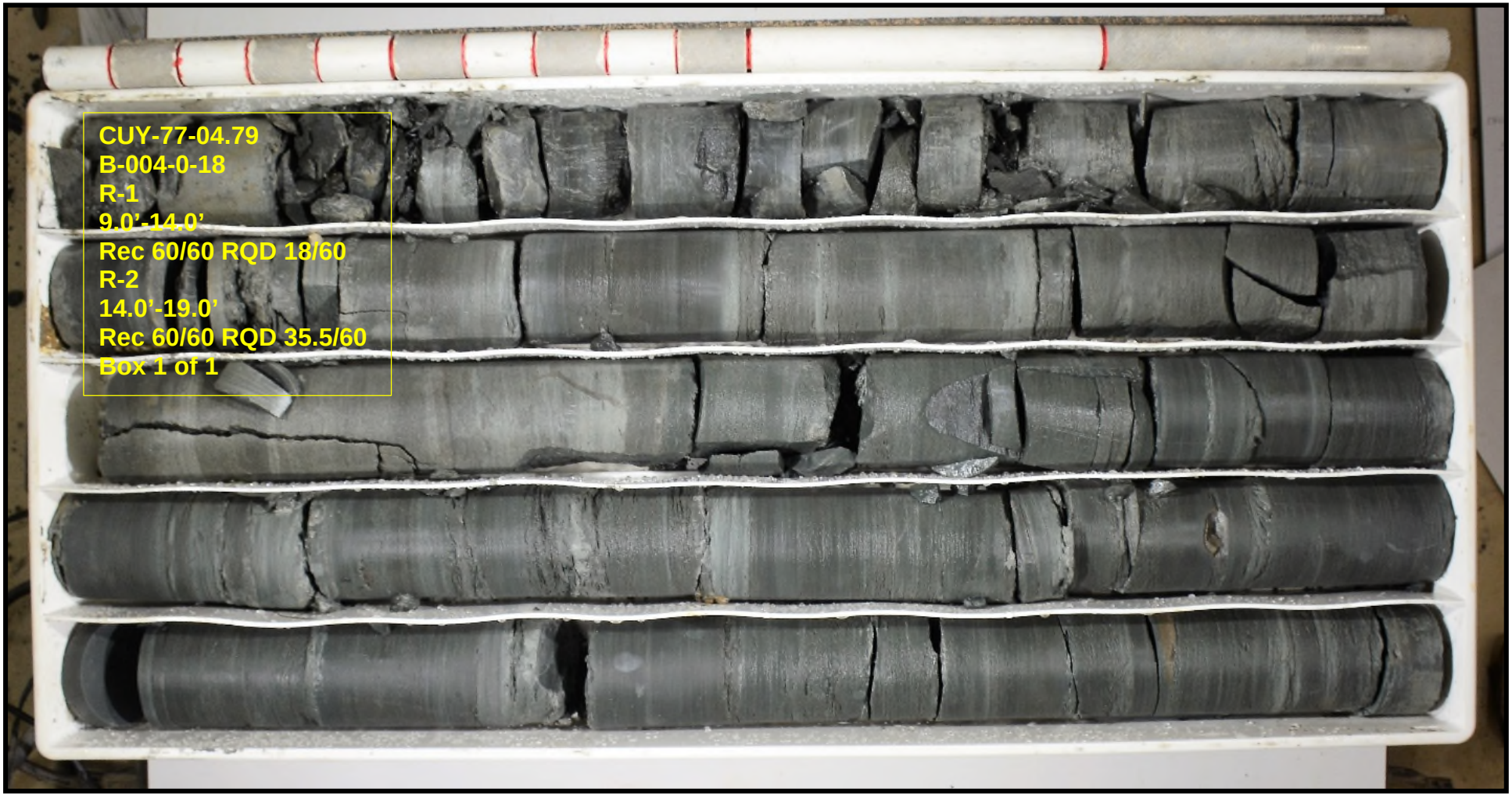
Run#:	Depth		Recovery		RQD	
NX-1	9.0’	19.0’	120/120	100%	28/120	23%
CUY-77-04.79 PID 106239						

B-003-0-18



Run#:	Depth		Recovery		RQD	
NX-1	10.0’	15.0’	60/60	100%	4/60	6%
NX-2	15.0’	20.0’	60/60	100%	35/60	58%
CUY-77-04.79 PID 106239						

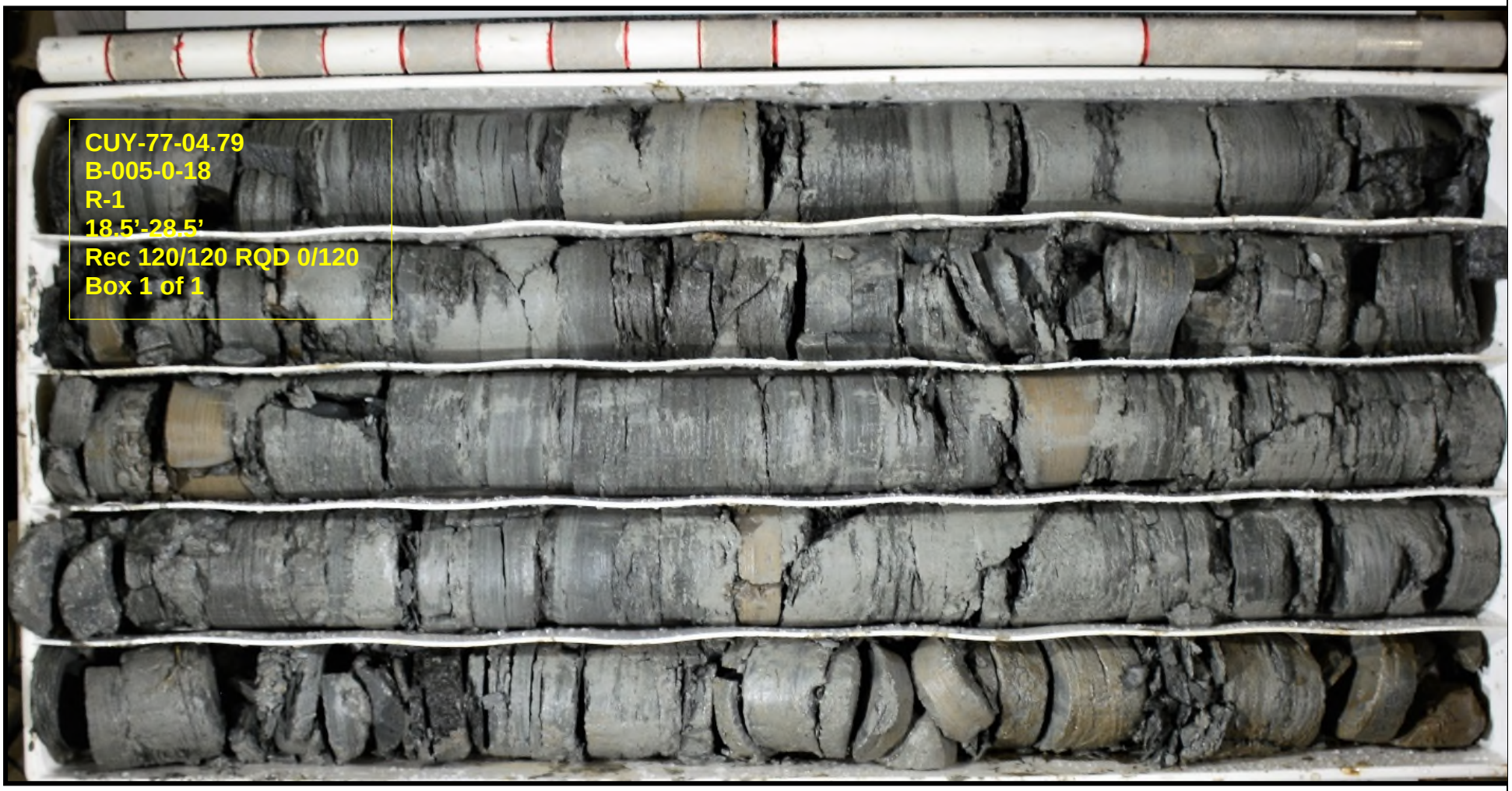
B-004-0-18



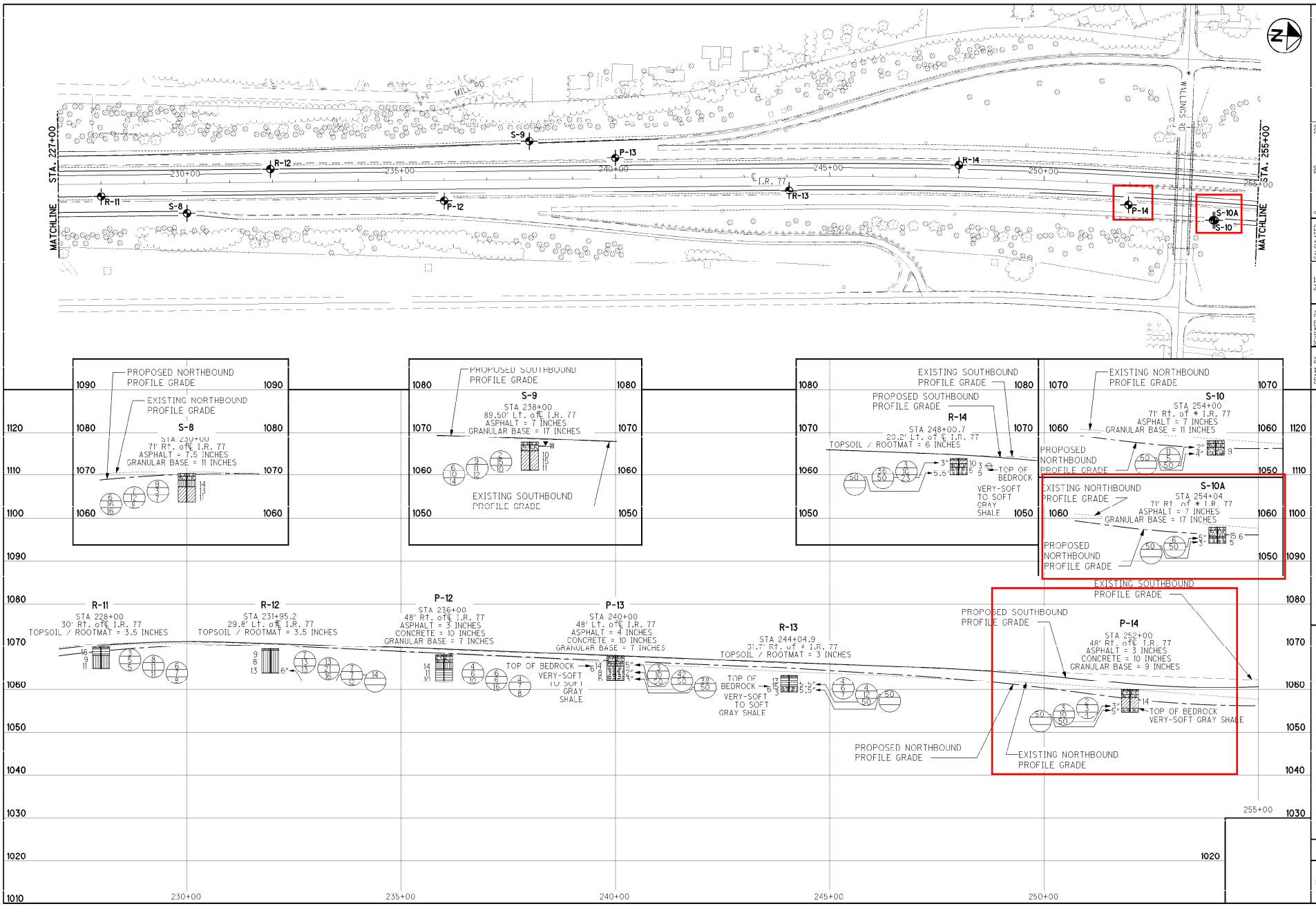
Run#:	Depth		Recovery		RQD	
NX-1	9.0'	14.0'	60/60	100%	18/60	30%
Nx-2	14.0'	19.0'	60/60	100%	35.5/60	59%

CUY-77-04.79 PID 106239

B-005-0-18



Run#:	Depth		Recovery		RQD	
NX-1	18.5’	28.5’	120/120	100%	0/120	0%
CUY-77-04.79 PID 106239						



BBCM

HORIZONTAL SCALE 1" = 10'

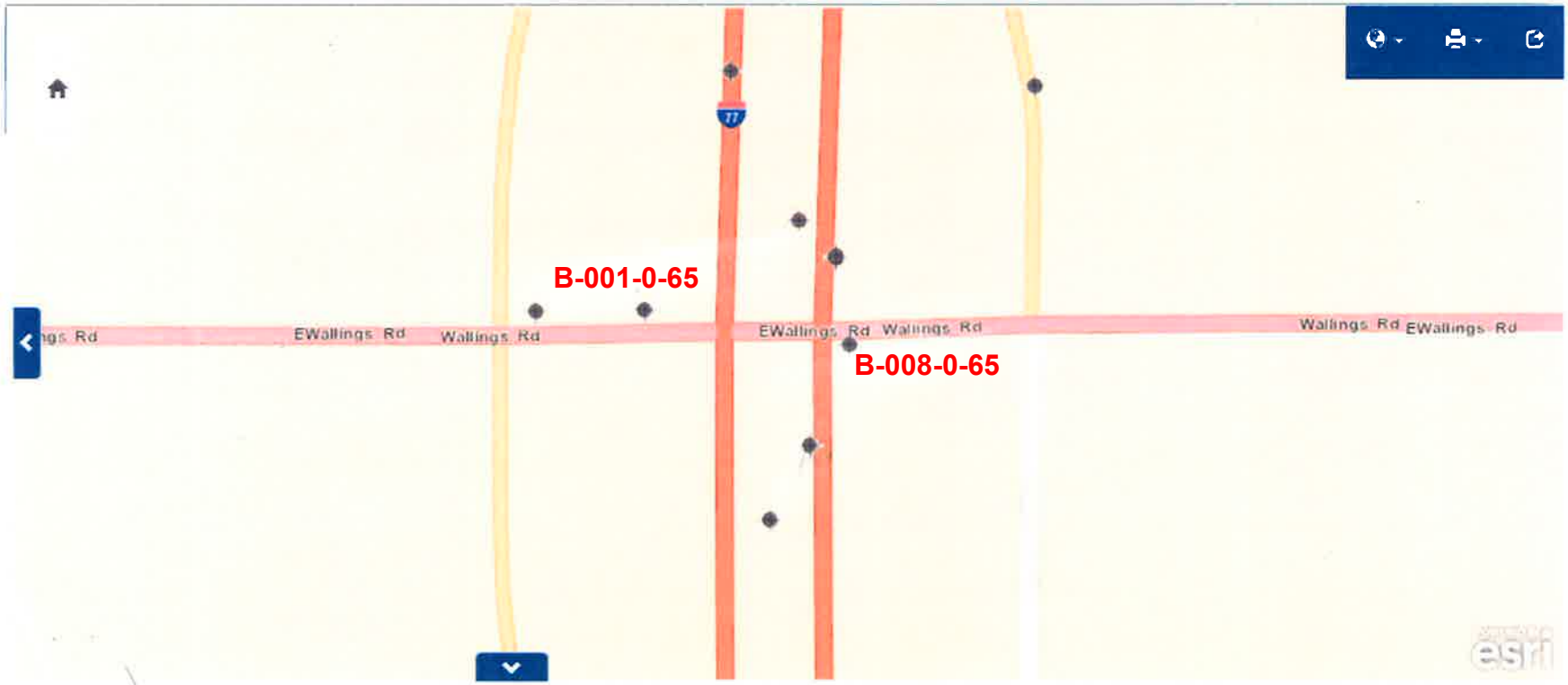
CALCULATED	DATE	REVIEWED BY	DRAWN BY
CHECKED RLW	11/22/04	BLS	JLP

SOIL PROFILE - I.R. 77'
STA. 227+00 TO 255+00

CUYAHOGA COUNTY
CUY-77-1.89

7 / 15

BBCM ENGINEERING, INC. DRAWING NO. BBCM FILENAME


B-001-0-65
B-008-0-65
ngs Rd
EWallings Rd
Wallings Rd
EWallings Rd
Wallings Rd
Wallings Rd
EWallings Rd
77
esri

B-001-0-65

LOG OF BORING

Date Started 6-24-65 | Sampler Type SS | Dia. 1 3/8" | Water Elev. _____

Date Completed 6-25-65 | Casing: Length _____ | Dia. _____

Boring No. B-1 | Station & Offset 94+24, 20' Lt. (REAR ABUTMENT) | Surface Elev. 1083.1' 1082.4 ft (NAVD 88)

Elev.	Depth	Std. Pen. (N)	Rec. ft.	Loss ft.	Description	Sample No.	Physical Characteristics % Agg.	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	SHTL Class.
1083.1	0														
	2														
	4														
1078.1	6	11/13			Brown Silty Sand	1	V	I	S	U	A	L	29	10	22
	8														
1073.1	10				TOP OF ROCK										
	12														
	14		2.2	1.8											
	16				Shale, gray, argillaceous, sub-fissile, medium-firm, jointed and broken, with many thin sandstone seams. Core loss:22%										
	18		5.0	0.0											
	20														
	22		3.5	1.5											
1058.1	24				BOTTOM OF BORING										

B-008-0-65

C-24-22-048

LOG OF BORING

Date Started 6-28-65 | Sampler Type SS | Dia. 1 3/8" | Water Elev. _____

Date Completed 6-29-65 | Casing: Length _____ | Dia. _____

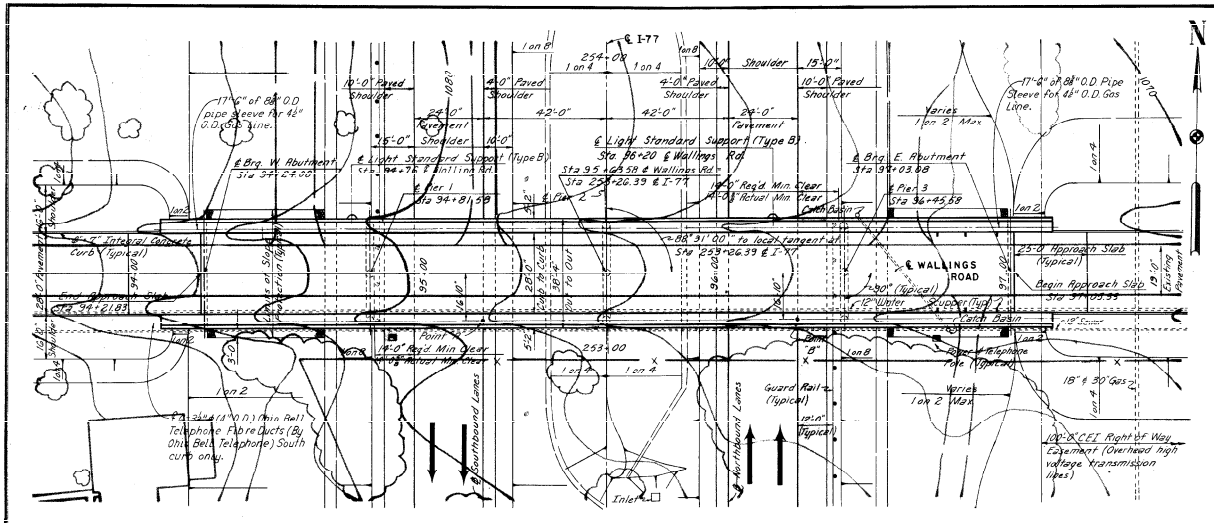
Boring No. B-8 | Station & Offset 96+44, 16' RT. (FORWARD PIER) | Surface Elev. 1073.5' 1072.8 ft (NAVD 88)

Elev.	Depth	Std. Pen. (N)	Rec. ft.	Loss ft.	Description	Sample No.	Physical Characteristics % Agg.	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	SHTL Class.
1073.5	0														
	2														
	4														
1068.5					TOP OF ROCK										
1067.5	6	6/17			Brownish-Gray Sandy Gravelly Clay	1	23	11	5	35	26	35	14	15	
	8		2.8	1.2	Shale, brown, slightly arenaceous, soft and crumbly, fissile, with sandstone seams, badly broken and moderately weathered. Core loss: 30%										
1063.5	10														
	12														
	14		3.2	1.8	Shale, gray, arenaceous, fissile, medium-firm, jointed and badly broken, with few clay seams and many sandstone seams and stringers. *Core loss 25%										
1058.5	16														
	18		3.0	2.0											
1053.5	20														
	22														
	24		3.2	1.8	*High core loss partially due to mechanical failure.										
1048.5	26														
	28		4.6	0.4											
1043.5	30				BOTTOM OF BORING										

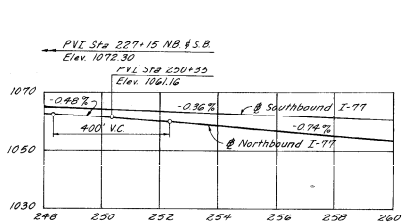
FED. RD. DIVISION	STATE	PROJECT	
2	OHIO		

320
382

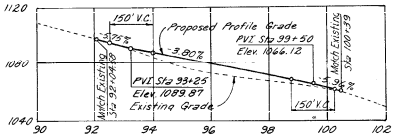
CUYAHOGA COUNTY
CUY- 77-4.07



PLAN
Scale: 1"=20'-0"



PROFILE I-77
Scale: Horiz. 1"=200'
Vert. 1"=20'



PROFILE WALLINGS ROAD
Scale: Horiz. 1"=200'
Vert. 1"=40'

PROPOSED STRUCTURE
TYPE: Continuous steel beam with reinforced concrete deck and substructure.
SPANS: 57'-6", 2 @ 82'-0" and 57'-6"
ROADWAY: 28'-0" curb/curb with two 4'-0" sidewalks
LOAD FREQUENCY: CF400(57)
SKEW: None
WEARING SURFACE: 1" Monolithic Concrete
APPROACH SLABS: AS-1-54 (25' Long)
ALIGNMENT: Tangent

TRAFFIC DATA:
1986 - 7,382 A.D.T. (Total both ways)

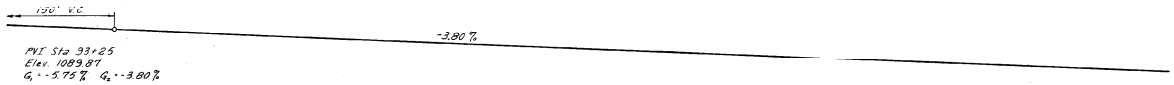
FOUNDATION DATA:
The abutments and piers shall be founded on spread footings with an allowable bearing pressure of 5 tons per sq. ft.

MICROFILMED
APR 18 1984

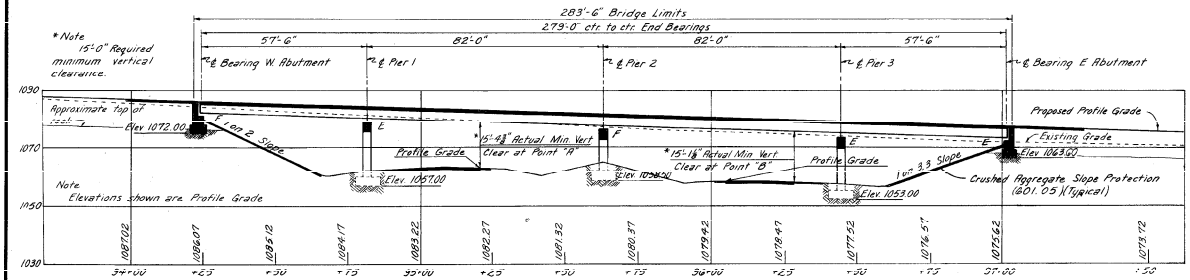
* Note: 8⅝" O.D. pipe sleeves to be furnished by The East Ohio Gas Co. and installed by the bridge contractor. The bridge contractor shall notify The East Ohio Gas Co. about seven days in advance of his need for installation.

CURVE DATA ℄ I-77

P.I. Sta. 249+15.50
Δ = 4°46'30"
°Dc 0°28'100" Rt
R = 12,277.67
L = 1023.19'
T = 511.89'
E = 10.67'



PROFILE



ELEVATION

ESTIMATED QUANTITIES

ITEM	TOTAL	UNIT	DESCRIPTION	CUY-77-0480 ABUTMENTS	PIERS	H.N.T.B. BRIDGE NO. 13 SUPERSTRUCTURE	GENERAL		
503	175	Cu. Yd.	Unclassified Excavation	175					
503	166	Cu. Yd.	Shale Excavation	76	90				
509	123,309	Pounds	Reinforcing Steel	10,519	21,891	90,899			
511	382	Cu. Yd.	Class "C" Concrete, Superstructure			382			
511	72	Cu. Yd.	Class "C" Concrete, Pier Caps and Columns		72				
511	123	Cu. Yd.	Class "E" Concrete, Abutment (Above footing)	123					
511	104	Cu. Yd.	Class "E" Concrete, Footings	47	57				
513	*316,700	Pounds	Structural Steel			316,700			
514	*316,700	Pounds	Field Painting of Structural Steel			316,700			
517	621.50	Lin. Ft.	Bridge Railing Type 2	60.67		560.83			
518	37	Cu. Yd.	Porous Backfill	37					
518	56	Lin. Ft.	6" Helical C.M.P. (Section 707.06) Non-Perforated	56					
518	60	Lin. Ft.	6" Perforated Helical C.M.P. (Section 707.06) Including Specials	60					
518	6	Each	Scuppers, Including Supports			6			
601	437	Sq. Yd.	Crushed Aggregate Slope Protection	437					
625	For Electrical Equipment Quantities see sheet 284								
808	382	Units	Water-Reducing, Set-Retarding Admixture			382			
925	1388	Sq. Yd.	Concrete Surface Treatment				1388		
928	56	Lin. Ft.	Joint Sealer (end dam)			56			

*Includes 427 pounds of structural steel to be paid for by East Ohio Gas Co.

H.N.T.B. BR. NO. 13

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

SITE PLAN

I-77 UNDER WALLINGS ROAD

BR. NO. CUY-77-0480 STA. 94+21.83 STA. 97+05.33

CUYAHOGA COUNTY OHIO

DRAWN C.K.B.	TRACED	CHECKED L.J.D.	REVIEWED W.C.	REVISED
DATE 8-6-83	DATE	DATE 8-24-83	DATE 11-23-83	SHEET 320

Piers 1 & 3 Calculations

Date: October 22, 2021
Project Number: 078832.01
Project Name: CUY-77-04.79/ Wallings Road
Client: Osborn Engineering

Computed By: Jalal Fatemi
Checked By: Alan J. Esser, PE, DGE

Service and strength limit states loads provided by Osborn Engineering:

Wallings Road Bridge			
		Service	Strength
Abutments	Max Vert Stress (toe)	3.6 ksf	5.4 ksf
	Max Vert Stress (heel)	4.9 ksf	6.7 ksf
	Max Vert. Load	26.3 klf	37.7 klf
	Max Horiz. Load	3.2 klf	4.8 klf
Pier 1	Max Vert.	500.6 k	650 k
	Mx (longitudinal)	1081.4 k-ft	1056.1 k-ft
	Mz (transverse)	149.5 k-ft	152.7 k-ft
Pier 2	Max Vert.	518.2 k	672.4 k
	Mx (longitudinal)	790.8 k-ft	948.6 k-ft
	Mz (transverse)	69.1 k-ft	92.5 k-ft
Pier 3	Max Vert.	500.6 k	650 k
	Mx (longitudinal)	1081.4 k-ft	1056.1 k-ft
	Mz (transverse)	149.5 k-ft	152.7 k-ft

Bearing Resistance:

$M_x := 1056.1\ ft \cdot kip$ $\quad M_z := 152.7\ ft \cdot kip$ $\quad V := 650\ kip$ Factored

$e_x := \frac{M_x}{V} = 1.625\ ft$ $\quad e_z := \frac{M_z}{V} = 0.235\ ft$ Factored

$L := 12\ ft$ $\quad L' := L - 2 \cdot e_z = 11.53\ ft$

From Foundation Plan sheet

$B := 10\ ft$ $\quad B' := B - 2 \cdot e_x = 6.75\ ft$

$D_f := 4\ ft$ Assumed

$RMR := 29$ (q_u=1-5, 1; RQD= 30%, 5; j_s=5; j_c=11; g_w=7)

$\phi' := \left(\frac{RMR}{2} + 5\right) \cdot 1\ deg = 19.5\ deg$ Bieniawski (1989)

$c' := (0.104 \cdot RMR) \cdot 1\ ksf = 3.016\ ksf$

$\gamma_q := 125\ pcf$ $\quad \gamma_f := 145\ pcf$

$$q_u := \left(\frac{1}{2} \cdot \gamma_f \cdot B' \cdot N_\gamma \cdot s_\gamma \cdot i_\gamma \cdot C_{w\gamma}\right) + \left(\gamma_q \cdot D_f \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot C_{wq}\right) + \left(c' \cdot N_c \cdot s_c \cdot i_c\right)$$

AASHTO LRFD Article 10.6.3.1.2a

$N_\gamma := 5.1$ $\quad N_q := 6.1$ $\quad N_c := 14.3$

$i_\gamma := 1$ $\quad i_q := 1$ $\quad i_c := 1$

Created with PTC Mathcad Express. See www.mathcad.com for more information.

$s_c := 1 + \left(\frac{B'}{L'}\right) \cdot \frac{N_q}{N_c} = 1.25$

$s_\gamma := 1 - 0.4 \left(\frac{B'}{L'}\right) = 0.766$

$s_q := 1 + \left(\frac{B'}{L'} \tan(\phi')\right) = 1.207$

$d_q := 1 + 2 \tan(\phi') \left(1 - \sin(\phi')\right)^2 \left(\operatorname{atan}\left(\frac{D_f}{B'}\right)\right) = 1.168$

$C_{wq} := 0.5 \quad C_{w\gamma} := 0.5$

$q_u := \left(\frac{1}{2} \cdot \gamma_f \cdot B' \cdot N_\gamma \cdot s_\gamma \cdot i_\gamma \cdot C_{w\gamma}\right) + \left(\gamma_q \cdot D_f \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot C_{wq}\right) + \left(c' \cdot N_c \cdot s_c \cdot i_c\right) = 57.006\ ksf$

$\varphi_b := 0.45$ AASHTO LRFD Table 10.5.5.2.2-1

$q_R := \varphi_b \cdot q_u = 25.653\ ksf$

Stress Distribution

$\sigma_{max} := \frac{V}{B \cdot L}\left(1 + 6 \cdot \frac{e_x}{B} + 6 \cdot \frac{e_z}{L}\right) = 11.333\ ksf$ Less than 25.6 ksf ---------> OK

$\sigma_{min} := \frac{V}{B \cdot L}\left(1 - 6 \cdot \frac{e_x}{B} - 6 \cdot \frac{e_z}{L}\right) = -0.5\ ksf$ Negligible uplift

Overturning: AASHTO LRFD Article 10.6.3.3

$e_x = 1.625\ ft \quad 0.45 \cdot B = 4.5\ ft$ ex < 0.45 B ---------> OK for overturning in x direction

$e_z = 0.235\ ft \quad 0.45 \cdot L = 5.4\ ft$ ez < 0.45 L ---------> OK for overturning in z direction

Settlement: AASHTO LRFD Article 10.6.2.4.4

$GSI := 29$ Per rock cores

$q_u := 6.9$ Average qu = 1000 psi = 6.9 MPa

$E_m := \left(\frac{q_u}{100}\right)^{0.5} \cdot 10^{\frac{GSI - 10}{40}} \cdot 1\ GPa = \left(1.638 \cdot 10^4\right)\ ksf$ Rock mass modulus

$\nu := 0.1$ Assummed for shale, AASHTO LRFD Table C10.4.6.5-2

Created with PTC Mathcad Express. See www.mathcad.com for more information.

Piers 1 & 3 Calculations

$M'_x := 1081.4\ ft \cdot kip$ $\quad M'_z := 149.5\ ft \cdot kip$ $\quad V' := 500.6\ kip$ Unfactored

$e'_x := \frac{M'_x}{V'} = 2.16\ ft$ $\quad e'_z := \frac{M'_z}{V'} = 0.299\ ft$ Unfactored

$L'' := L - 2 \cdot e'_z = 11.403\ ft$

$B'' := B - 2 \cdot e'_x = 5.68\ ft$

$\beta_z := \frac{1.10 - 1.08}{2 - 1} \cdot \left(\frac{L''}{B''} - 1\right) + 1.08 = 1.1$ Assuming rigid footing, AASHTO LRFD Table 10.6.2.4.4-1

$I_p := \frac{\left(\frac{L''}{B''}\right)^{0.5}}{\beta_z} = 1.288$

$q_0 := \frac{V'}{B'' \cdot L''} = 7.73\ ksf$

$\rho := q_0 \cdot \left(1 - \nu^2\right) \cdot \frac{B'' \cdot I_p}{144 \cdot E_m} = \left(2.848 \cdot 10^{-4}\right)\ in$

Created with PTC Mathcad Express. See www.mathcad.com for more information.

Pier 2 Calculations

Date: October 22, 2021
Project Number: 078832.01
Project Name: CUY-77-04.79/ Wallings Road
Client: Osborn Engineering

Computed By: Jalal Fatemi
Checked By: Alan J. Esser, PE, DGE

Service and strength limit states loads provided by Osborn Engineering:

Wallings Road Bridge			
		Service	Strength
Abutments	Max Vert Stress (toe)	3.6 ksf	5.4 ksf
	Max Vert Stress (heel)	4.9 ksf	6.7 ksf
	Max Vert. Load	26.3 klf	37.7 klf
	Max Horiz. Load	3.2 klf	4.8 klf
Pier 1	Max Vert.	500.6 k	650 k
	Mx (longitudinal)	1081.4 k-ft	1056.1 k-ft
	Mz (transverse)	149.5 k-ft	152.7 k-ft
Pier 2	Max Vert.	518.2 k	672.4 k
	Mx (longitudinal)	790.8 k-ft	948.6 k-ft
	Mz (transverse)	69.1 k-ft	92.5 k-ft
Pier 3	Max Vert.	500.6 k	650 k
	Mx (longitudinal)	1081.4 k-ft	1056.1 k-ft
	Mz (transverse)	149.5 k-ft	152.7 k-ft

Bearing Resistance:

$M_x := 948.6\ ft \cdot kip$ $\quad M_z := 92.5\ ft \cdot kip$ $\quad V := 672.4\ kip$ Factored

$e_x := \frac{M_x}{V} = 1.41\ ft$ $\quad e_z := \frac{M_z}{V} = 0.14\ ft$ Factored

$L := 12\ ft$ $\quad L' := L - 2 \cdot e_z = 11.725\ ft$

From Foundation Plan sheet

$B := 10\ ft$ $\quad B' := B - 2 \cdot e_x = 7.178\ ft$

$D_f := 4\ ft$ Assumed

$RMR := 29$ (q_u=1-5, 1; RQD= 30%, 5; j_s=5; j_c=11; g_w=7)

$\phi' := \left(\frac{RMR}{2} + 5\right) \cdot 1\ deg = 19.5\ deg$ Bieniawski (1989)

$c' := (0.104 \cdot RMR) \cdot 1\ ksf = 3.016\ ksf$

$\gamma_q := 125\ pcf$ $\quad \gamma_f := 145\ pcf$

$$q_u := \left(\frac{1}{2} \cdot \gamma_f \cdot B' \cdot N_\gamma \cdot s_\gamma \cdot i_\gamma \cdot C_{w\gamma}\right) + \left(\gamma_q \cdot D_f \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot C_{wq}\right) + \left(c' \cdot N_c \cdot s_c \cdot i_c\right)$$

AASHTO LRFD Article 10.6.3.1.2a

$N_\gamma := 5.1$ $\quad N_q := 6.1$ $\quad N_c := 14.3$

$i_\gamma := 1$ $\quad i_q := 1$ $\quad i_c := 1$

Created with PTC Mathcad Express. See www.mathcad.com for more information.

Pier 2 Calculations

$s_c := 1 + \left(\frac{B'}{L'}\right) \cdot \frac{N_q}{N_c} = 1.261$

$s_\gamma := 1 - 0.4 \left(\frac{B'}{L'}\right) = 0.755$

$s_q := 1 + \left(\frac{B'}{L'} \tan(\phi')\right) = 1.217$

$d_q := 1 + 2 \tan(\phi') \left(1 - \sin(\phi')\right)^2 \left(\operatorname{atan}\left(\frac{D_f}{B'}\right)\right) = 1.16$

$C_{wq} := 0.5 \quad C_{w\gamma} := 0.5$

$q_u := \left(\frac{1}{2} \cdot \gamma_f \cdot B' \cdot N_\gamma \cdot s_\gamma \cdot i_\gamma \cdot C_{w\gamma}\right) + \left(\gamma_q \cdot D_f \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot C_{wq}\right) + \left(c' \cdot N_c \cdot s_c \cdot i_c\right) = 57.547\ ksf$

$\varphi_b := 0.45$ AASHTO LRFD Table 10.5.5.2.2-1

$q_R := \varphi_b \cdot q_u = 25.896\ ksf$

Stress Distribution

$\sigma_{max} := \frac{V}{B \cdot L}\left(1 + 6 \cdot \frac{e_x}{B} + 6 \cdot \frac{e_z}{L}\right) = 10.732\ ksf$ Less than 25.9 ksf ---------> OK

$\sigma_{min} := \frac{V}{B \cdot L}\left(1 - 6 \cdot \frac{e_x}{B} - 6 \cdot \frac{e_z}{L}\right) = 0.475\ ksf$

Overturning: AASHTO LRFD Article 10.6.3.3

$e_x = 1.411\ ft \quad 0.45 \cdot B = 4.5\ ft$ ex < 0.45 B ---------> OK for overturning in x direction

$e_z = 0.138\ ft \quad 0.45 \cdot L = 5.4\ ft$ ez < 0.45 L ---------> OK for overturning in z direction

Settlement: AASHTO LRFD Article 10.6.2.4.4

$GSI := 29$ Per rock cores

$q_u := 6.9$ Average qu = 1000 psi = 6.9 MPa

$E_m := \left(\frac{q_u}{100}\right)^{0.5} \cdot 10^{\frac{GSI - 10}{40}} \cdot 1\ GPa = \left(1.638 \cdot 10^4\right)\ ksf$ Rock mass modulus

$\nu := 0.1$ Assummed for shale, AASHTO LRFD Table C10.4.6.5-2

Created with PTC Mathcad Express. See www.mathcad.com for more information.

Pier 2 Calculations

$M'_x := 790.8\ ft \cdot kip$ $\quad M'_z := 69.1\ ft \cdot kip$ $\quad V' := 518.2\ kip$ Unfactored

$e'_x := \frac{M'_x}{V'} = 1.526\ ft$ $\quad e'_z := \frac{M'_z}{V'} = 0.133\ ft$ Unfactored

$L'' := L - 2 \cdot e'_z = 11.733\ ft$

$B'' := B - 2 \cdot e'_x = 6.948\ ft$

$\beta_z := \frac{1.10 - 1.08}{2 - 1} \cdot \left(\frac{L''}{B''} - 1\right) + 1.08 = 1.094$ Assuming rigid footing, AASHTO LRFD Table 10.6.2.4.4-1

$I_p := \frac{\left(\frac{L''}{B''}\right)^{0.5}}{\beta_z} = 1.188$

$q_0 := \frac{V'}{B'' \cdot L''} = 6.357\ ksf$

$\rho := q_0 \cdot \left(1 - \nu^2\right) \cdot \frac{B'' \cdot I_p}{144 \cdot E_m} = \left(2.643 \cdot 10^{-4}\right)\ in$

Created with PTC Mathcad Express. See www.mathcad.com for more information.

Rear Abutment Calculations

Date: October 22, 2021
Project Number: 078832.01
Project Name: CUY-77-04.79/ Wallings Road
Client: Osborn Engineering

Computed By: Jalal Fatemi
Checked By: Alan J. Esser, PE, DGE

Service and strength limit states loads provided by Osborn Engineering:

Rear abutment foundation elevation: 1071.35 feet

Wallings Road Bridge			
		Service	Strength
Abutments	Max Vert Stress (toe)	3.6 ksf	5.4 ksf
	Max Vert Stress (heel)	4.9 ksf	6.7 ksf
	Max Vert. Load	26.3 klf	37.7 klf
	Max Horiz. Load	3.2 klf	4.8 klf
Pier 1	Max Vert.	500.6 k	650 k
	Mx (longitudinal)	1081.4 k-ft	1056.1 k-ft
	Mz (transverse)	149.5 k-ft	152.7 k-ft
Pier 2	Max Vert.	518.2 k	672.4 k
	Mx (longitudinal)	790.8 k-ft	948.6 k-ft
	Mz (transverse)	69.1 k-ft	92.5 k-ft
Pier 3	Max Vert.	500.6 k	650 k
	Mx (longitudinal)	1081.4 k-ft	1056.1 k-ft
	Mz (transverse)	149.5 k-ft	152.7 k-ft

Bearing Resistance:

$B := 6.25\ ft$ $\quad L := 66.67\ ft$

$\sigma_{toe} := 5.4\ ksf$ $\quad V_l := 37.7\ klf$ $\quad V := V_l \cdot L = (2.513 \cdot 10^3)\ kip$ Factored

$\sigma_{heel} := 6.7\ ksf$ $\quad H_l := 4.8\ klf$ $\quad H := H_l \cdot L = 320.016\ kip$ Factored

$V'_l := 26.3\ klf$ $\quad V' := V'_l \cdot L = (1.753 \cdot 10^3)\ kip$ Unfactored

$H'_l := 3.2\ klf$ $\quad H' := H'_l \cdot L = 213.344\ kip$ Unfactored

$M := 0.5 \cdot (\sigma_{heel} - \sigma_{toe}) \cdot \frac{B^2}{6} \cdot L = 282.132\ ft \cdot kip$ Factored

$e := \frac{M}{V} = 0.112\ ft$ $\quad B' := B - 2 \cdot e = 6.026\ ft$ $\quad L' := L = 66.67\ ft$

$D_f := 0\ ft$ Conservatively assumed

$RMR := 29$ (q_u = 1 to 5 MPa, 1; RQD= 8%, 5; j_s=5; j_c=11; g_w=7) Bieniawski (1989)

$\phi' := \left(\frac{RMR}{2} + 5\right) \cdot 1\ deg = 19.5\ deg$

$c' := (0.104 \cdot RMR) \cdot 1\ ksf = 3.016\ ksf$

$\gamma_q := 125\ pcf$ $\quad \gamma_f := 145\ pcf$

Created with PTC Mathcad Express. See www.mathcad.com for more information.

Rear Abutment Calculations

$$q_u := \left(\frac{1}{2} \cdot \gamma_f \cdot B' \cdot \boxed{N_\gamma} \cdot s_\gamma \cdot i_\gamma \cdot C_{w\gamma}\right) + \left(\gamma_q \cdot D_f \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot C_{wq}\right) + \left(c' \cdot N_c \cdot s_c \cdot i_c\right)$$

$N_\gamma := 5.1$ $\quad N_q := 6.1$ $\quad N_c := 14.3$ $\quad$ AASHTO LRFD Article 10.6.3.1.2a

$\theta := 90\ deg$ $\quad n := \left(\frac{2 + \frac{L'}{B'}}{1 + \frac{L'}{B'}}\right) \cdot (\cos(\theta))^2 + \left(\frac{2 + \frac{B'}{L'}}{1 + \frac{B'}{L'}}\right) \cdot (\sin(\theta))^2 = 1.917$ $\quad$ AASHTO LRFD Figure C10.6.3.1.2a-1

$$i_q := \left(1 - \frac{H'}{V' + c' \cdot B' \cdot L' \cdot \cot(\phi')}\right)^n = 0.922 \qquad i_c := i_q - \left(\frac{1 - i_q}{N_q - 1}\right) = 0.907$$

$$i_\gamma := \left(1 - \frac{H'}{V' + c' \cdot B' \cdot L' \cdot \cot(\phi')}\right)^{n+1} = 0.884$$

$$s_c := 1 + \left(\frac{B'}{L'}\right) \cdot \frac{N_q}{N_c} = 1.039 \qquad s_q := 1 + \left(\frac{B'}{L'} \tan(\phi')\right) = 1.032$$

$$s_\gamma := 1 - 0.4 \left(\frac{B'}{L'}\right) = 0.964$$

$$d_q := 1 + 2 \tan(\phi') (1 - \sin(\phi'))^2 \left(\operatorname{atan}\left(\frac{D_f}{B'}\right)\right) = 1$$

$C_{wq} := 0.5$ $\quad C_{w\gamma} := 0.5$ $\quad$ Conservatively assumed

$$q_n := \left(\frac{1}{2} \cdot \gamma_f \cdot B' \cdot N_\gamma \cdot s_\gamma \cdot i_\gamma \cdot C_{w\gamma}\right) + \left(\gamma_q \cdot D_f \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot C_{wq}\right) + \left(c' \cdot N_c \cdot s_c \cdot i_c\right) = 41.587\ ksf$$

$H_s := 1071.35\ ft - (1056.35 + 3)\ ft = 12\ ft$ $\quad$ AASHTO LRFD Figure 10.6.3.1.2c-1

$b := 16\ ft$ $\quad \beta := \operatorname{atan}\left(\frac{1}{2}\right) = 26.565\ deg$ $\quad$ Conservatively assumed 30 degrees

$$N_s := \frac{\gamma_f \cdot H_s}{c'} = 0.577 \qquad \frac{B'}{H_s} = 0.502 \qquad \frac{b}{B'} = 2.655$$

$RC_{BC} := 0.90$ $\quad$ Conservatively selected from Table 10.6.3.1.2c-2

$\varphi_b := 0.45$ $\quad$ AASHTO LRFD Table 10.5.5.2.2-1

$q_R := \varphi_b \cdot RC_{BC} \cdot q_n = 16.843\ ksf$ $\quad$ Greater than $\sigma_{heel} = 6.7\ ksf$ ---------> OK

Created with PTC Mathcad Express. See www.mathcad.com for more information.

Rear Abutment Calculations

Overturning: AASHTO LRFD Article 10.6.3.3 or 11.6.3.3

$e = 0.112\ ft$ $0.45 \cdot B = 2.813\ ft$ e < 0.45 B ----------> OK for overturning

Sliding: AASHTO LRFD Article 10.6.3.4

$\varphi_\tau := 0.9$ ODOT BDM 2004 Section 305.2.4

$C := 1$ Concrete cast against soil

$V_{service} := 26.3\ klf$

$R_{\tau 1} := C \cdot V_{service} \cdot \tan(\phi') = 9.313\ klf$

$\varphi R_n := \varphi_\tau \cdot R_{\tau 1} = 8.382\ klf$ Greater than 4.8 klf --------> OK

If footing is supported by a minimum 6 inches of compacted granular material:

$S_u := c' = 3.016\ ksf$

$\sigma_{min} := 0.5 \cdot 3.6\ ksf = 1.8\ ksf$ less than Su --------> OK

$\sigma_{max} := 0.5 \cdot 4.9\ ksf = 2.45\ ksf$ less than Su --------> OK

$R_{\tau 2} := \frac{\sigma_{max} + \sigma_{min}}{2} \cdot B' = 12.804\ klf$ AASHTO LRFD Figure 10.6.3.4-1

$\varphi R_n := \varphi_\tau \cdot R_{\tau 2} = 11.524\ klf$ Greater than $H_l = 4.8\ klf$ --------> OK

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Forward Abutment Calculations

Date: October 22, 2021
Project Number: 078832.01
Project Name: CUY-77-04.79/ Wallings Road
Client: Osborn Engineering

Computed By: Jalal Fatemi
Checked By: Alan J. Esser, PE, DGE

Service and strength limit states loads provided by Osborn Engineering:

Forward abutment foundation elevation: 1062.95 feet

Wallings Road Bridge			
		Service	Strength
Abutments	Max Vert Stress (toe)	3.6 ksf	5.4 ksf
	Max Vert Stress (heel)	4.9 ksf	6.7 ksf
	Max Vert. Load	26.3 klf	37.7 klf
	Max Horiz. Load	3.2 klf	4.8 klf
Pier 1	Max Vert.	500.6 k	650 k
	Mx (longitudinal)	1081.4 k-ft	1056.1 k-ft
	Mz (transverse)	149.5 k-ft	152.7 k-ft
Pier 2	Max Vert.	518.2 k	672.4 k
	Mx (longitudinal)	790.8 k-ft	948.6 k-ft
	Mz (transverse)	69.1 k-ft	92.5 k-ft
Pier 3	Max Vert.	500.6 k	650 k
	Mx (longitudinal)	1081.4 k-ft	1056.1 k-ft
	Mz (transverse)	149.5 k-ft	152.7 k-ft

Bearing Resistance:

$B := 6.25\ ft$ $\quad L := 77.67\ ft$ $\quad$ From Foundation Plan sheet

$\sigma_{toe} := 5.4\ ksf$ $\quad V_l := 37.7\ klf$ $\quad V := V_l \cdot L = (2.928 \cdot 10^3)\ kip$ $\quad$ Factored

$\sigma_{heel} := 6.7\ ksf$ $\quad H_l := 4.8\ klf$ $\quad H := H_l \cdot L = 372.816\ kip$ $\quad$ Factored

$V'_l := 26.3\ klf$ $\quad V' := V'_l \cdot L = (2.043 \cdot 10^3)\ kip$ $\quad$ Unfactored

$H'_l := 3.2\ klf$ $\quad H' := H'_l \cdot L = 248.544\ kip$ $\quad$ Unfactored

$M := 0.5 \cdot (\sigma_{heel} - \sigma_{toe}) \cdot \frac{B^2}{6} \cdot L = 328.682\ ft \cdot kip$ $\quad$ Factored

$e := \frac{M}{V} = 0.112\ ft$ $\quad B' := B - 2 \cdot e = 6.026\ ft$ $\quad L' := L = 77.67\ ft$

$D_f := 0\ ft$ $\quad$ Conservatively assumed

$RMR := 28$ (q_u < 1 MPa, 0; RQD= 0%, 5; j_s=5; j_c=11; g_w=7) $\quad$ Bieniawski (1989)

$\phi' := \left(\frac{RMR}{2} + 5\right) \cdot 1\ deg = 19\ deg$

$c' := (0.104 \cdot RMR) \cdot 1\ ksf = 2.912\ ksf$

$\gamma_q := 125\ pcf$ $\quad \gamma_f := 145\ pcf$

Created with PTC Mathcad Express. See www.mathcad.com for more information.

Forward Abutment Calculations

$$q_u := \left(\frac{1}{2} \cdot \gamma_f \cdot B' \cdot \boxed{N_\gamma} \cdot s_\gamma \cdot i_\gamma \cdot C_{w\gamma}\right) + \left(\gamma_q \cdot D_f \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot C_{wq}\right) + \left(c' \cdot N_c \cdot s_c \cdot i_c\right)$$

$N_\gamma := 4.7$ $\quad N_q := 5.8$ $\quad N_c := 13.9$ $\quad$ AASHTO LRFD Article 10.6.3.1.2a

$\theta := 90\ deg$ $\quad n := \left(\frac{2+\frac{L'}{B'}}{1+\frac{L'}{B'}}\right) \cdot (\cos(\theta))^2 + \left(\frac{2+\frac{B'}{L'}}{1+\frac{B'}{L'}}\right) \cdot (\sin(\theta))^2 = 1.928$ $\quad$ AASHTO LRFD Figure C10.6.3.1.2a-1

$$i_q := \left(1 - \frac{H'}{V' + c' \cdot B' \cdot L' \cdot \cot(\phi')}\right)^n = 0.922 \qquad i_c := i_q - \left(\frac{1-i_q}{N_q - 1}\right) = 0.905$$

$$i_\gamma := \left(1 - \frac{H'}{V' + c' \cdot B' \cdot L' \cdot \cot(\phi')}\right)^{n+1} = 0.884$$

$$s_c := 1 + \left(\frac{B'}{L'}\right) \cdot \frac{N_q}{N_c} = 1.032 \qquad s_q := 1 + \left(\frac{B'}{L'} \tan(\phi')\right) = 1.027$$

$$s_\gamma := 1 - 0.4 \left(\frac{B'}{L'}\right) = 0.969$$

$$d_q := 1 + 2 \tan(\phi') (1 - \sin(\phi'))^2 \left(\operatorname{atan}\left(\frac{D_f}{B'}\right)\right) = 1$$

$C_{wq} := 0.5$ $\quad C_{w\gamma} := 0.5$ $\quad$ Conservatively assumed

$$q_n := \left(\frac{1}{2} \cdot \gamma_f \cdot B' \cdot N_\gamma \cdot s_\gamma \cdot i_\gamma \cdot C_{w\gamma}\right) + \left(\gamma_q \cdot D_f \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot C_{wq}\right) + \left(c' \cdot N_c \cdot s_c \cdot i_c\right) = 38.711\ ksf$$

$H_s := 1062.95\ ft - (1052.35 + 3)\ ft = 7.6\ ft$ $\quad$ AASHTO LRFD Figure 10.6.3.1.2c-1

$b := 16\ ft$ $\quad \beta := \operatorname{atan}\left(\frac{1}{3}\right) = 18.435\ deg$ $\quad$ Conservatively assumed 20 degrees

$N_s := \frac{\gamma_f \cdot H_s}{c'} = 0.378$ $\quad \frac{B'}{H_s} = 0.793$ $\quad \frac{b}{B'} = 2.655$

$RC_{BC} := 0.95$ $\quad$ Conservatively selected from Table 10.6.3.1.2c-2

$\varphi_b := 0.45$ $\quad$ AASHTO LRFD Table 10.5.5.2.2-1

$q_R := \varphi_b \cdot RC_{BC} \cdot q_n = 16.549\ ksf$ $\quad$ Greater than $\sigma_{heel} = 6.7\ ksf$ ---------> OK

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Forward Abutment Calculations

Overturning: AASHTO LRFD Article 10.6.3.3 or 11.6.3.3

$e = 0.112\ ft$ $0.45 \cdot B = 2.813\ ft$ e < 0.45 B ----------> OK for overturning

Sliding: AASHTO LRFD Article 10.6.3.4

$\varphi_\tau := 0.9$ ODOT BDM 2004 Section 305.2.4

$C := 1$ Concrete cast against soil

$V_{service} := 26.3\ klf$

$R_{\tau 1} := C \cdot V_{service} \cdot \tan(\phi') = 9.056\ klf$

$\varphi R_n := \varphi_\tau \cdot R_{\tau 1} = 8.15\ klf$ Greater than 4.8 klf --------> OK

If footing is supported by a minimum 6 inches of compacted granular material:

$S_u := c' = 2.912\ ksf$

$\sigma_{min} := 0.5 \cdot 3.6\ ksf = 1.8\ ksf$ less than Su --------> OK

$\sigma_{max} := 0.5 \cdot 4.9\ ksf = 2.45\ ksf$ less than Su --------> OK

$R_{\tau 2} := \frac{\sigma_{max} + \sigma_{min}}{2} \cdot B' = 12.804\ klf$ AASHTO LRFD Figure 10.6.3.4-1

$\varphi R_n := \varphi_\tau \cdot R_{\tau 2} = 11.524\ klf$ Greater than $H_l = 4.8\ klf$ --------> OK

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Abutments Earth Pressure

Date: October 22, 2021
Project Number: 078832.01
Project Name: CUY-77-04.79/ Wallings Road
Client: Osborn Engineering

Computed By: Jalal Fatemi
Checked By: Alan J. Esser, PE, DGE

Abutments earth pressure: AASHTO LRFD Article 3.11.5.3

Typical Values for Backfill:

$\gamma_{bf} := 120\ pcf \quad \phi' := 30\ deg \quad \delta := 20\ deg \quad \theta := 90\ deg \quad \beta := 0\ deg$

$$\Gamma := \left(1 + \left(\frac{\sin(\phi' + \delta) \cdot \sin(\phi' - \beta)}{\sin(\theta - \delta) \cdot \sin(\theta + \beta)}\right)^{0.5}\right)^2 = 2.684$$

$$K_a := \frac{(\sin(\theta + \phi'))^2}{\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta)} = 0.297$$

$\gamma_{eq} := K_a \cdot \gamma_{bf} = 35.678\ pcf$ Use 36 pcf

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Seismic Site Class

Date: October 22, 2021
Project Number: 078832.01
Project Name: CUY-77-04.79/ Wallings Road
Client: Osborn Engineering

Computed By: Jalal Fatemi
Checked By: Alan J. Esser, PE, DGE

$GSI := 35$ Per rock cores $\gamma := 145\ pcf$ $g := 32.2\ \frac{ft}{s^2}$

$q_u := 6.9$ Average qu = 1000 psi = 6.9 MPa

$E_m := \left(\frac{q_u}{100}\right)^{0.5} \cdot 10^{\frac{GSI-10}{40}} \cdot 1\ GPa = 1.108\ GPa$ Rock mass modulus

$\nu := 0.1$ Assummed for shale

$G_m := \frac{E_m}{2\ (1+\nu)} = 0.504\ GPa$ Shear Modulus

$\rho := \frac{\gamma}{g} = 144.883\ \frac{lb}{ft^3}$ $\rho = 4.503\ \frac{slug}{ft^3}$

$V_s := \left(\frac{G_m}{\rho}\right)^{0.5} = \left(1.528 \cdot 10^3\right)\ \frac{ft}{s}$ Applies to top 20' of shale bedrock

Deeper rock will have a higher Vs.
Typical range for soft shale: 1-2 km/sec = 3280-6560 ft/sec
Assuming Vs= 3280 ft/s
Average shear wave velocity in the top 100 feet is:

$$V_{s30} := \frac{100\ ft}{\frac{20\ ft}{V_s} + \frac{80\ ft}{3280\ \frac{ft}{s}}} = \left(2.668 \cdot 10^3\right)\ \frac{ft}{s}$$

Site Class B per AASHTO LRFD Table 3.10.3.1-1

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APPENDIX B

IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL-ENGINEERING REPORT

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Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept*

responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are not final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

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Passionate People Building
and Revitalizing our World

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