Division of Engineering Research on Call Agreement 31796

Task 7 – Service Evaluation of Highway Structures with Soil-Bearing Spread Footings

Jamal Nusairat, Bashar Tarawneh, Shad Sargand, Kevin White

for the Ohio Department of Transportation
Office of Statewide Planning and Research

and the
United States Department of Transportation
Federal Highway Administration

February 2021

Final Report

RESEARCH

U.S. Department of Transportation
Federal Highway Administration

E.L. ROBINSON
ENGINEERING

OHIO UNIVERSITY
1804

RUSS COLLEGE OF ENGINEERING
AND TECHNOLOGY

1. Report No. FHWA/OH-2021-04	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Division of Engineering Research on Call Services **Service Evaluation of Highway Structures with Soil-Bearing Spread Footings**		5. Report Date February 2021 6. Performing Organization Code
7. Author(s) Jamal Nusairat (ORCID 0000-0001-9890-2632), Bashar Tarawneh (ORCID 0000-0002-9745-4812), Shad M. Sargand (ORCID 0000-0002-1633-1045), Kevin White (ORCID 0000-0002-2902-2524)		8. Performing Organization Report No.
9. Performing Organization Name and Address E.L. Robinson Engineering of Ohio Company 950 Goodale Blvd, Suite 180 Grandview, OH 43212 and Ohio Research Institute for Transportation and the Environment (ORITE) Room 141, Stocker Center Ohio University Athens OH 45701-2979		10. Work Unit No. (TRAIS) 11. Contract or Grant No. State Job No. 135786
12. Sponsoring Agency Name and Address Ohio Department of Transportation Office of Statewide Planning and Research 1980 West Broad St. Columbus, OH 43223		13. Type of Report and Period Covered Final Report 14. Sponsoring Agency Code
15. Supplementary Notes Prepared in cooperation with the Ohio Department of Transportation (ODOT) and the U.S. Department of Transportation, Federal Highway Administration		

16. Abstract

The Ohio Department of Transportation (ODOT) needs to evaluate highway structures' performance supported on spread footings bearing on soils. This evaluation's outcomes are recommendations for future use of spread footings and potential limitations on their use. The project team completed the following tasks:

1. Reviewed the documented performance data.
2. Evaluated the performance of existing footings and compare to the bridge performance criteria.
3. Reviewed the calculated settlement.
4. Provided an estimate of the structure tolerable settlement at the foundation location.
5. Compared the measured settlement to the predicted long-term settlement.
6. Compared performance to soil conditions and calculated bearing pressures.
7. Reviewed IRI (International Roughness Index) and differential settlement between substructures.

The data collected and the field review of all four sites included in this research showed that spread footing supporting structures perform as intended, and there were no signs of settlements and rideability issues. Spread footings constructed following the manuals and procedures by ODOT will perform satisfactorily. The requirements for monitoring of the settlement using rods specified in the plans need to be augmented with survey targets at the substructure units above finished ground for comparison and verification of plumbness of the substructure unit.

17. Key Words	18. Distribution Statement
Spread footings, Highway structures, Settlement, Performance, IRI	No Restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	xxx	

Form DOT F 1700.7 (8-72) Reproduction of completed pages authorized

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Volumes greater than 1000 L shall be shown in m^3.				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(°F-32)/9 or (°F-32)/1.8	Celsius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2 or psi	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celsius temperature	1.8°C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2 or psi

* SI is the symbol for the International Symbol of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised September 1993)

Division of Engineering Research on Call Agreement 31796

Task 7 – Service Evaluation of Highway Structures with Soil-Bearing Spread Footings

Jamal Nusairat[1]
Bashar Tarawneh[1]
Shad M. Sargand[2]
Kevin White[1]

[1]E. L. Robinson Engineering of Ohio Company
950 Goodale Blvd, Suite 180
Grandview Heights, OH 43212

[2]Ohio Research Institute for Transportation and the Environment (ORITE),
Civil Engineering Department
Russ College of Engineering and Technology
Ohio University

Prepared in cooperation with the
Ohio Department of Transportation
and the
U.S. Department of Transportation, Federal Highway Administration

Final Report

February 2021

Credits and Acknowledgments

Prepared in cooperation with the Ohio Department of Transportation

and the U.S. Department of Transportation, Federal Highway Administration

Acknowledgements

The authors are grateful to the Ohio Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration for sponsoring the investigation described in this report. The authors also would like to express their special thanks of gratitude to the ODOT's Technical Advisory Committee members Amal Goza, Alexander Dettloff, and Chris Merklin, for their insight, direction, and effort to finish this work. Their time and help are highly appreciated.

Amal Goza is noted for her involvement in the project from initiation to completion. She collected all existing data and plans for the structures involved in this study and verified their field conditions. Her efforts, during the study and during the report review, are greatly appreciated and were instrumental in the success of this project.

Division of Engineering Research on Call Agreement 31796

Task 7 – Service Evaluation of Highway Structures with Soil-Bearing Spread Footings

Jamal Nusairat, Ph.D., P.E.
Bashar Tarawneh, Ph.D., P.E.
Kevin White, Ph.D., P.E.
E.L. Robinson Engineering of Ohio Company
950 Goodale Blvd, Suite 180
Grandview Heights, OH 43212

Shad M. Sargand (Russ Professor)
Ohio Research Institute for Transportation and the Environment (ORITE)
Room 141, Stocker Center
Ohio University
Athens OH 45701-2729

February 2021

Table of Contents

Appendices

Appendix 1: Structures Plans

Appendix 2: Soil Profiles and Geotechnical Report Recommendations

Appendix 3: Settlement Calculations for All Structures

CHAPTER 1: INTRODUCTION

1.1 Scope of Work

Spread footings bearing on soil are becoming an attractive alternative for supporting highway structures. Spread footings bearing on soil have many advantages compared to deep foundations, primarily low cost, fast construction, and environmentally friendly. Highway structures with soil-bearing spread footings are underutilized due to limited performance data and overestimation of settlements. However, to encourage their utilization, well-documented, comprehensive case histories must be established and made available to the Bridge and geotechnical engineers. Despite previous shallow foundations studies' success, more research is needed to evaluate the performance of spread footings as a highway bridge foundation.

The Ohio Department of Transportation (ODOT) needs to evaluate highway structures' performance supported on spread footings bearing on soils. This evaluation's outcomes are recommendations for future use of spread footings and potential limitations on their use. The project team completed the following tasks to meet the project goals:

1. Reviewed the documented performance data.
2. Evaluated the performance of existing footings and compared to the bridge performance criteria.
3. Reviewed the calculated settlement.
4. Provided an estimate of the structure tolerable settlement at the foundation location.
5. Compared the measured settlement to the predicted long-term settlement.
6. Compared performance to soil conditions and calculated bearing pressures.
7. Reviewed IRI (International Roughness Index) and differential settlement between substructures.

1.2 Outline of the Report

Chapter 2 presents the results of an extensive literature review carried out as part of the current study. This chapter's content is arranged by topics such as advantages of using spread footings, ODOT's experience with spread footings, service evaluation, and performance of spread footings.

Chapter 3 summarizes highway structures' performance supported on spread footings. Four projects are presented, namely, MAH-680-2.83, CUY-77-14.35, FAI-33-13.09, and CUY/SUM-271-0.00/14.67. Survey monitoring data is presented for spread footings at the end of construction and the recently surveyed monuments under this study. In addition, a comparison is made between the estimated and measured settlements. Hough (1959) is used to estimate the settlements of spread footing on cohesionless soil as specified in section 10.6.2.4 of the AASHTO LRFD Bridge Specifications.

Chapter 4 presents collected profile data for CUY/SUM-271-0.00/14.87: CUY/SUM-480-29.58/0.00 and CUY-77-14.35. The data were examined to assess the effect of retaining walls, constructed on spread footings, on ride quality, in terms of the International Roughness Index

(IRI). The profiles presented included the pavement adjacent to walls, approach slabs, and the bridge deck. This chapter also introduces the differential settlements between substructures.

Chapter 5 draws important findings reached while performing this study and provided recommendations for future work.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

The literature search focuses on the service evaluation, previous performance prediction methods, and the advantages of using spread footings to support highway structures. It also discusses the current practices within different state DOTs, in addition to the guidance of other agencies and organizations, including the Federal Highway Administration (FHWA), American Association of State Highway and Transportation Officials (AASHTO).

Soil-bearing footings have been used successfully to support highway bridges by several state DOT's. Bridge engineers often are very hesitant to recommend spread footings because of (Sargand et al. 1999):

1. The AASHTO Bridge Design Specifications.
2. A lack of tolerable movement guidelines for spreads footings.
3. A common belief that spread footings settle much more than deep foundations and require higher maintenance costs.
4. Uncertainty in the selection of performance prediction methods.
5. Uncertainty in the properties of subsoils that are used in the settlement prediction methods.

Data from successful case histories must be documented and shared by civil engineering professionals to promote the use of spread footings for highway bridge structures. Understanding settlement and other behaviour of spread footing foundations under various loading and environmental conditions associated with highway bridge structures is essential in encouraging their use in highway bridge construction. Also, further verification of the performance prediction methods through such case histories contributes to increased use of the spread footing foundation (Sargand et. al. 1999).

Conventionally in the United States, highway structures were supported conservatively by either shallow spread footings on bedrock or pile foundations (or drilled shafts) bearing on bedrock or very dense/stiff soil formations to ensure their long-term serviceability. However, in recent years spread footings on soil have become an attractive alternative for these structures where subsurface soil conditions are suitable. For example, sites consisting mainly of granular soils or well-consolidated cohesive soils may be ideal for the spread footing use. Shallow spread footing foundations generally require less time and cost to construct than pile foundations (Sargand et al. 1999).

2.2 Advantages of using Spread Footings

Spread footings on soil are becoming an attractive alternative for supporting highway structures because they have many advantages compared to deep foundations. The advantages included cost-saving, expedited construction, simple design, environmentally friendly, and less maintenance (FHWA, 2002, 2006, 2010, and 2001).

Bridge approaches are generally constructed with reinforced concrete slabs that connect the bridge deck to the adjacent paved roadway. The slab is usually supported on one side by the bridge abutment and on the other side by the embankment. Their function provides a smooth and safe transition of vehicles from the roadway pavements to bridge structures and vice versa. However, complaints about the ride quality of bridge approach slabs still need to be resolved. The complaints usually involve a “bump” that motorists feel when they approach or leave bridges (Cai et al. ,2005). This problem is commonly referred to as the bump at the end of the bridge.

Spread footings are normally considered in conditions where deep foundation installation is not possible, for example to a) accommodate the presence of aquifers, underground structures such as utilities and obstructions beneath foundations; and b) generate less noise, ground vibrations, and movements of nearby structures. Construction of spread footings uses common materials, and can be constructed with readily available labor, simple and small equipment. The construction process is often easier, faster, and its quality control is simple compared to deep foundations. Because of these advantages, construction of spread footings is usually supposed to provide a safe work environment and fewer claims. Finally, the use of spread footing alleviates the bridge bump problem (Abu-Hejleh et al., 2006).

The use of spread footings may not be suitable or economical under certain design conditions, for example, presence of deep soft soil near the ground surface or very high lateral loads (e.g., due to a major earthquake), and at sites with large scour or liquefaction depths (Abu-Hejleh et al., 2014).

Shallow foundations are typically at least 30% more economical than deep foundations and can be utilized to support many different civil engineering structures (Masada and Sargand 2009). Foundations contribute a significant influence on the construction cost of concrete bridges; their cost ranges from 19 to 27% of the total bridge construction cost, depending on the construction method used and the bridge design system (Fragkakis and Lambropoulos 2004). Based on a sample of 19 concrete highway bridges built in Switzerland between 1958 and 1985, Menn (1990) concluded that foundations contribute 18% of the total bridge construction cost.

There are approximately 600,000 bridges in the United States. If those bridges had to be replaced by new bridges it would cost approximately $300 billion. Therefore, the average cost of a bridge is $500,000. About 50% of that cost is for the foundation. For such an average bridge the difference in cost between shallow foundations and deep foundations is $90,000 (Briaud, 1993).

Each year 6,000 new bridges are built for a yearly national bridge budget of $3 billion. Approximately 85% of the existing 600,000 bridges in the inventory are over water. This percentage is probably more like 50% when considering the bridges built in the last few years. If one assumes that all 6,000 bridges built yearly are on deep foundations and assumes that all bridges that are not over water can be placed on shallow foundations, the numbers above indicate a yearly saving to taxpayers of 90,000 x 6000 x 0.5 = $270 million. Even if the saving is only a fraction of this number, say 100 million, the potential saving is significant. If 5% of the potential saving is invested in research, a $5 million budget per year is not unreasonable to make serious progress towards this economic goal (Briaud, 1997).

2.3 DOT’s Experience with Spread Footings

A national FHWA survey of the geotechnical practices of the state DOTs was developed and distributed in 2007 (Abu-Hejleh et al. 2014). Forty-four (44) states responded to this survey. Survey results indicated that the average distribution of bridge foundation types considered by State DOTs across the United States is approximately 24% spread footings (11.5% founded on soils, 12.5% founded on rock) and 76% deep foundations (56.5% driven piles and 19.5% drilled shafts). The FHWA national survey identified the states with significant and moderate use (>10%) of spread footings on soils to support highway bridges and the states with limited or no use (<5%). Based on this survey, the FHWA concluded that some State DOTs could save time and cost if spread footings bearing on soils are used when appropriate to support bridges.

Table 1.2 presents some of the results from the FHWA national survey. The use of spread footings is 50% in the Northeast, 30% in the Southwest, 20% in the Northwest, and 10% in the Midwest. The Southeast region as well as some states in other regions reported no or limited use. The survey also reported that State DOTs have safely, and economically constructed highway bridges supported on spread footings bearing on competent and improved natural soils and engineered granular and MSE fills. Engineered granular fill is defined as a high-quality granular soil selected and constructed to meet certain material and construction specifications (also called "compacted structural fill" and "compacted granular soil").

Table 1.2: Lead States in Deploying Spread Footings for Bridges (2007 National Survey)

States	**Spread Footings (%)**		**Deep Foundations (%)**	
	Soil	Rock	Driven Piles	Drilled Shafts
Northeast States				
Connecticut	50	25	20	5
Vermont	40	10	45	5
Massachusetts	35	15	20	27
New Hampshire	30	30	30	10
New York	30	15	47	3
New Jersey	30	20	40	5
Southwest States				
New Mexico	30	10	30	30
Nevada	25	3	18	54
Northwest States				
Idaho	20	10	60	10
Oregon	20	10	60	10
Midwest States				
Michigan	10	5	80	5

2.4 Service Evaluation and Performance of Spread Footings

Many performance prediction methods have been proposed to estimate spread footing behavior (bearing capacity and settlement) based on the standard penetration test (SPT) or the cone penetration test (CPT) data. Examples of these include the work by Hough (1959), Alpan (1964), Meyerhof (1965), Terzaghi and Peck (1967), D'Appolonia et al. (1968), Peck and Bazaraa (1969), Schmertmann (1970), and Schmertmann et al. (1978), as presented by Sargand and Masada (2006).

Sargand and Masada (2006) instrumented four spread footings constructed at two interstate highway construction sites in Ohio with modern sensors and monitored through construction stages and beyond. The spread footing design methods presented in the AASHTO LRFD Bridge Design Specifications (2004) were validated based on the field performance data collected during the research. Twelve SPT-based settlement prediction methods, for footings resting on cohesionless or slightly cohesive soils, were evaluated considering the field performance data. General performance analysis of spread footing foundations at bridge construction sites was made to draw some guidelines concerning the use of spread footings for supporting highway bridge structures. Detailed cost comparisons were made between spread footing and pile foundation options. Overall, the results of the research project indicated that: a) spread footing can be a viable option as a bridge foundation; and b) the design methods presented in the AASHTO LRFD Bridge Design Specifications (2004) appear to be satisfactory.

Sargand et al. (1997) instrumented and monitored over fifty spread footings at five highway bridge construction sites in Ohio. Bridges A through C were constructed over predominantly cohesionless (A-2, A-3, A-4) subsoils, while Bridges D and E were built at sites consisting mostly of cohesive (A-6, A-7-6) soils. At the Bridge A construction site, the uncorrected SPT N value varied from about 20 blows per foot (bpf) at the base of footing to 100+ bpf at depths reaching 20 to 30 ft below the footing. At the site of Bridge B, the uncorrected SPT N value stayed relatively constant around 50 bpf below the foundation depth. Under the footings of Bridge C, the uncorrected SPT N value increased from 14 to 20 bpf within 30 ft. The SPT N values recorded at the Bridge D site varied from as low as 40 bpf at the base of footing to 100+ bpf at depths reaching 20 to 30 ft below the footing. The uncorrected SPT N value started at 13 bpf and gradually increased to 30 bpf and higher at the Bridge E site. The spread footings' overall settlement among all the footings ranged from 0.19 to 1.43 inches, with an average of 0.78 inches. Typically, about 70% of the total settlement took place before the deck construction. None of the footings experienced any significant differential movement problems. Limited data collected at the sites within 6 months after the bridge opening showed that the additional settlement induced by the live load application ranged between 0.05 and 0.5 inches, with an average of 0.17 inches.

Baus (1992) monitored the settlement of 12 spread footings at three highway bridge sites in South Carolina. Total settlement varied from 0.4–2.2 in. He compared the maximum settlement measured in the field to estimates made by six prediction methods (Alpan, Hough, Meyerhof, Peck-Bazaraa, Buisman-De Beer, and Schmertmann methods). He concluded that the methods by Peck and Bazaraa (1969) and Hough (1959) provided better settlement predictions.

DiMillio (1982) surveyed the conditions of 148 bridges supported by spread footings on compacted fill in Washington. All bridges were in good condition, and none exhibited any safety or functional problems. He found that the bridges could tolerate easily differential settlement of one to three inches without severe distress. He estimated those spread footings were 50–60% less expensive than pile foundations.

Moulton et al. (1982) examined the tolerable movement of bridges. He reviewed movements and damages data for 204 bridges on both spread footings and piles. His study revealed that the average vertical movement of abutments was more than 4 inches regardless of the foundation type, and the average horizontal movement was more than 2.5 inches.

Keene (1978) investigated some case histories of spread footing used in Connecticut. He observed that in some cases, a post-construction settlement of two to three inches occurred without any damage to the bridge structures. He stressed the importance of ‘‘staged’’ construction before the superstructure placement to minimize post-construction settlement.

Walkinshaw (1978) reviewed the field performance of 35 bridges in the western states that were supported by spread footings. He stated that poor riding quality resulted when vertical settlement exceeded 2.5 inches.

Gifford et al. (1987) reported a study on the settlement performance of 21 bridge spread footings on cohesionless soils. The overall settlement of these spread footings ranged from 0.02 to 2.72 inches, with an average value of 0.61 inch. Approximately 70% of the total settlement occurred before the placement of the bridge deck. They evaluated six settlement prediction methods for sands (Burland-Burbridge, D’Appolonia, Hough, Peck-Bazaraa, Peck-Bazaraa-Ladd, and Schmertmann methods). They concluded that the methods by D’Appolonia and Burland-Burbridge were more accurate. The methods by Peck and Bazaraa typically underpredicted the field settlement, and the methods by Hough and Schmertmann overpredicted the field settlement.

CHAPTER 3: PERFORMANCE OF HIGHWAY STRUCTURES SUPPORTED ON SPREAD FOOTINGS BEARING ON SOILS IN OHIO

3.1 General

This chapter presents background information/data for four sites (MAH-680-2.83, CUY-77-14.35, FAI-33-13.09, and CUY/SUM-271-00.00/14.87), where spread footings monitored during different phases of construction and surveyed during this task between October 13 and 14, 2020.

3.2 MAH-680-2.83

This is a four-span bridge (MAH-680-0283) carrying Vestal Road (Rd) over Interstate 680 (IR-680) in Mahoning County in northwestern Youngstown, Ohio. In 2016 a rehabilitation work was performed for the Bridge. The work consisted of removing the existing superstructure and the three existing piers and raising the existing abutment seats. Therefore, new spread footings were constructed for the piers only. Existing spread footings at the abutments were used. The Bridge is four-span continuous painted steel girders with reinforced concrete deck on new semi-integral abutments, new bearings, and new cap and column piers founded on spread footings. Bridge plans are included in Appendix 1. Soil profiles and Geotechnical Report including recommendations are presented in Appendix 2.

Site stratigraphy consists of hard silt and clay (A-4a) near the ground surface to an elevation of about 932 ft. where a dense sand and gravel or sand layer (A-1-b, A-3a) 5 to 15 ft. in thickness was encountered. Below this was another hard layer of silt/ sand mixture (A-4a) and a hard silt layer beneath it. Groundwater was not encountered. The bottom of footing elevation for pier 1,2 and 3 are 940 ft., 939 ft, and 937 ft., respectively.

On October 6, 2020, a site visit was performed by Jamal Nusairat and Dave Traini of E.L. Robinson Engineering to the MAH-680-0283 bridge. The deck and sidewalk were inspected for deck cracking, indicating relative settlement of piers, and no indications were found. Piers were found to be in good condition; no settlement was observed. One of the PVC pipes over the settlement pins was observed to be broken off at the groundline. The existing bridge foundation founded on spread footing functions as designed, and no signs of any settlements. Figure 3.1 shows a picture of the MAH-680-0283 bridge.

Piers spread footings were monitored after pouring the footings' concrete, before and after beam placement, after pouring the deck's concrete, and after project completion. Recently, under this task, piers footings were surveyed. Table 3.1 presents the recorded monitoring data for the right and left monuments of each pier.

At project completion, the measured settlements ranged from 0.0 to 0.48 inches. Recently under this task, the final measured settlements ranged from -0.24 to 0.72 inches. It should be noted that the minus values appear due to elevation reading tolerable error and refer to no settlement. After approximately four years, settlements did not change significantly. The settlement was within tolerable limits for these span lengths. The measured settlement to girder length ratio was 0.00067. This is well below the acceptable limits of 0.004 as documented by Felix Yokel (1990).

The measured settlement under this task was negative for some of the substructure units. The negative settlement can result from either settlement of the benchmark used or measurement error as the surveying accuracy is to the nearest 1/8 inch.

Figure 3.1 The MAH-680-0283 Bridge

Table 3.1: MAH-680-2.83 Footings Settlement Monitoring Data

Stage	**Elevation of Left Monument (ft.)**			**Elevation of Right Monument (ft.)**			**Date**
	Pier 1	**Pier 2**	**Pier 3**	**Pier 1**	**Pier 2**	**Pier 3**	
After Footings Poured	943.97	943.23	940.97	943.92	943.13	940.82	2016-05-31
Before Beams	943.93	943.2	940.97	943.91	943.13	940.82	2016-06-28
After Beams	943.92	943.19	940.96	943.92	943.12	940.81	2016-07-28
After Deck Pour	943.93	943.2	940.97	943.94	943.13	940.83	2016-10-05
Project Completion	943.93	943.19	940.96	943.92	943.13	940.83	2016-11-12
This Task	943.93	943.17	940.98	943.94	943.14	940.84	2020-10-13
Measured Sett. at Project Completion (Inches)	**0.48**	**0.48**	**0.12**	**0.0**	**0.0**	**0.12**	
Measured Settlements under this Task (Inches)	**0.48**	**0.72**	**-0.12**	**0.24**	**-0.12**	**-0.24**	

3.3 CUY-77-14.35

The project consists of four cast-in-place (CIP) concrete cantilever walls (Wall 1,2,3 and 4) and a bridge replacement for Bridge No. CUY-1433 L&R over I.R. 490 and Ramps. The bridge is a three-span continuous steel hybrid girder composite with a reinforced concrete deck on reinforced concrete piers and semi-integral abutments. The bridge is supported on 16 inches cast-in-place reinforced concrete piles. Settlements were monitored for walls 1 and 4, the left rear abutment wing wall (Wall 2), and the right forward abutment wing wall (Wall 3). Walls plans are included in Appendix 1. Soil profiles and Geotechnical Report geotechnical recommendations are presented in Appendix 2.

On October 6, 2020, a site visit was performed by Jamal Nusairat and Dave Traini of E.L. Robinson Engineering. The cantilevered cast-in-place concrete wing walls were inspected and founded to be plumb vertically; no leaning or sliding of the walls was observed. The existing bridge retaining walls foundation founded on spread footing is functioning as designed. Figure 3.2 shows the Right Forward Abutment Wing Wall (Wall 3).

Figure 3.2 The Right Forward Abutment Wing Wall (Wall 3)

3.3.1 CUY-77-14.35 Soil Profile

Wall 1 (IR 77 Sta. 72+19.25 to Sta. 74+20.54) – Boring BB-104

Material visually identified as fill was encountered below the pavement to a depth of 25.5 feet and consisted of medium-dense to hard sandy silt (A-4a) and dense coarse and fine sand (A-3a).

Natural soils were encountered below the fill to the termination depth of 40.0 feet and consisted of medium dense fine sand (A-3). Groundwater seepage was not encountered, and the boring appeared to be dry at the completion of drilling.

Wall 4 (IR 77 Sta. 80+34.56 to Sta. 82+71.00) – Boring BB-112

Material visually identified as fill was encountered below the pavement to a depth of 23.0 feet and consisted of medium-dense sandy silt (A-4a) and dense gravel with sand (A-1-b). Natural soils were encountered below the fill to the termination depth of 40.0 feet and consisted of soft to medium-stiff silt and clay (A-6a), medium-dense to dense coarse and fine sand (A-3a), and dense fine sand (A-3). No groundwater seepage was encountered, and the boring appeared to be dry at the completion of drilling.

Bridge No. CUY-1433 L&R over I.R. 490 and Ramps Wing Walls

Left Rear Abutment wing wall-Boring BB-106 (Wall 2)

Material visually identified as fill was encountered below the pavement to a depth of 32.0 feet and consisted of medium-dense to very-dense gravel with sand (A-1-b) and dense coarse and fine sand (A-3a). Natural soils were encountered below the fill to the termination depth of 90.0 feet and consisted of medium-dense fine sand (A-3), medium-dense to very-dense coarse and fine sand (A-3a), stiff to very-stiff silt (A-4b), and medium-stiff to stiff silty-clay (A-6b). Groundwater seepage was encountered at a depth of 53.5 feet and groundwater was encountered at a depth of 58.5 feet.

Right forward abutment wing wall Boring-BB-111 (Wall 3)

Material visually identified as fill was encountered below the pavement to a depth of 32.0 feet and consisted of dense sandy-silt (A-4a), dense silt (A-4b), medium-dense fine sand (A-3) and dense to very-dense coarse and fine sand (A-3a). Natural soils were encountered below the fill to the termination depth of 90.0 feet and consisted of medium-dense to very-dense fine sand (A-3), medium-stiff to dense sandy silt (A-4a), and dense silt (A-4b). Groundwater seepage was encountered at a depth of 58.5 feet and water was measured at a depth of 68.5 feet at the completion of drilling.

3.3.2 CUY-77-14.35 Settlement Monitoring Data

Spread footings were monitored after pouring the footings' concrete, and after project completion. Recently, under this task, footings were surveyed. Table 3.2 presents the recorded monitoring data for the monuments of the left rear abutment wing wall (Wall 2) and the right forward abutment wing wall (Wall 3). At project completion, the measured settlements are about 0.24 inches for Wall 2 and Wall 3. Recently under this task, the measured settlements ranged from 0.24 to 1.44 inches for Wall 2 and from 0.6 to 0.72 inches for Wall 3. It should be noted that settlements did not change significantly except for Wall 2 monument 2.

Table 3.2: Bridge No. CUY-77-1433 Wing Walls Settlement Monitoring Data

Stage	Elevation (ft.)				Date
	Left Rear Abutment Wing Wall (Wall 2)		Right Forward Abutment Wing Wall (Wall 3)		
	Monument-1	Monument-2	Monument-1	Monument-2	
	75+18.48, 62' LT	74+63.86, 62' LT	79+37.02, 62' RT	79+87.68, 62' RT	
After Footing Concrete Placed	673.64	673.68	680.38	680.39	N. A
Project Completion	673.62	673.66	680.36	680.37	N. A
Recent, this Task	673.62	673.56	680.32	680.34	2020-10-13
Measured Sett. at Project Completion (Inches)	**0.24**	**0.24**	**0.24**	**0.24**	
Measured Settlements under this Task (Inches)	**0.24**	**1.44**	**0.72**	**0.6**	

Table 3.3 presents the recorded monitoring data for the monuments of Wall 1 and 4. At project completion, the measured settlements ranged from 0.0 to 0.12 inches for Wall 1 and Wall 4. Recently under this task, the measured settlements ranged from 0.12 to 0.24 inches for Wall 1 and from 4.92 to 7.68 inches for Wall 4. Settlements did not change significantly for Wall 1. However, Wall 4 experienced excessive settlements due the existed 1.8 to 3.5-foot layer of soft to medium-stiff silt and clay (A-6a) which was encountered immediately beneath the fill material at elevation 665.9 ft. This layer is located 10.6 feet below the bottom of Wall 4 footing. The wall was inspected and founded to be plumb vertically; no leaning or sliding of the walls was observed. Therefore, the benchmark may have a problem.

Table 3.3: Wall 1 and 4 Settlement Monitoring Data

Stage	Elevation (ft.)				Date
	Wall 1		Wall 4		
	Monument-1	Monument-2	Monument-1	Monument-2	
	73+25.50, 64' RT	74+17.86, 65' RT	80+37.14, 62' LT	81+84.50, 62' LT	
After Footing Concrete Placed	674.22	674.18	679.53	681.27	N. A
Project Completion	674.23	674.18	679.52	681.27	N. A
Recent, this Task	674.21	674.16	679.12	680.63	2020-10-13
Measured Sett. at Project Completion (Inches)	**0.12**	**0.0**	**0.12**	**0.0**	
Measured Settlements under this Task (Inches)	**0.12**	**0.24**	**4.92**	**7.68**	

3.4 FAI-33-13.09

This is a four-span bridge (FAI-33-1309) carrying Delmont Road over U.S. Route 33 (Lancaster Bypass) west of Lancaster in Fairfield County, Ohio. The Bridge is a four-Span continuous composite steel girder bridge with semi-integral type abutments and cap and columns type piers and on spread footings. Bridge plans are included in Appendix 1. Soil profiles and Geotechnical Report geotechnical recommendations are presented in Appendix 2.

On October 5, 2020, a site visit was performed to FAI-33-1309 bridge by Jamal Nusairat and Dave Traini of E.L. Robinson Engineering, and Chris Merklin of ODOT. The team inspected the embankment slopes, roadway settlement at abutments, concrete deck, and relative substructure orientation. The team found no evidence of settlements. The soils around the piers were found to be soft, but this was due to drainage. Some of the PVC caps were missing or damaged due to vandalism. The existing bridge foundations founded on spread footing are functioning as designed.

Figure 3.3 The FAI-33-1309 Bridge

Field exploration was performed using five boreholes per the original geotechnical report. Each one of the five borings first encountered between 3 and 12 inches of topsoil. Underlying the topsoil, the five borings typically encountered cohesive soils consisting of stiff to hard silt and clay (A-6a) and silty clay (A-6b) to depths of between 10.5 and 20.5 feet. Some of these soils were organic in nature. Underlying these cohesive soils, each of the five borings generally encountered medium dense to very dense non-cohesive soils, including gravel with sand (A-1-b), gravel with sand and silt (A-2-4), fine sand (A-3), and coarse and fine sand (A-3a). These soils were encountered to the completion depths of the borings in boring B-33 and B-34 and were encountered to depths of between 49.5 and 62 feet in borings B-30, B-31, and B-32, where bedrock was encountered. It should be noted that material classified as silt (A-4b) was encountered in boring B-34. However, this material was encountered at depths of greater than 50 feet. Bedrock was encountered in borings

B-30, B-31, and B-32 at depths of between 49.5 and 62 feet. The bedrock consisted of medium-hard broken sandstone with RQDs of between 30% and 50%. Water seepage was encountered at depths of between 7.2 and 17 feet.

Spread footings were monitored after pouring the footings' concrete, before and after beams placement, after pouring the deck's concrete, and after project completion. Recently, under this task, footings were surveyed. Table 3.4 presents the recorded monitoring data for the right and left monuments of each pier. At project completion, the measured settlements ranged from 0.36 to 1.2 inches. Recently under this task, the final measured settlements ranged from 0.12 to 2.52 inches. After approximately 19 years, settlements did not change significantly except for the rear abutment. It should be noted that monuments could not be located at the right rear abutment, left and right pier 3. As at these locations survey monument information was not available, the survey crew established temporary benchmarks which were tied into two ODOT benchmarks via GPS observations.

Table 3.4: FAI-33-13.09 Footings Settlement Monitoring Data

Stage	Elevation of Left Monument (ft.)					Elevation of Right Monument (ft.)					Date
	Rear Abut.	Pier 1	Pier 2	Pier 3	FR. Abut.	Rear Abut.	Pier 1	Pier 2	Pier 3	FR. Abut.	
After Footings Poured	915.12	899.42	901.37	897.64	915.83	915.21	899.49	901.39	897.82	915.89	Between 06-05 and 07-12-2001
Before Beams	915.12	899.42	901.36	897.64	915.81	915.21	899.49	901.39	897.82	915.87	2001-10-19
After Beams	915.03	899.4	901.36	897.63	915.78	915.15	899.47	901.38	897.8	915.84	2002-03-21
After Deck Pour	915.06	899.38	901.35	897.62	915.8	915.13	899.45	901.36	897.79	915.86	2002-04-26
Project Completion	915.04	899.38	901.34	897.61	915.79	915.11	N. A[a]	901.36	897.79	915.85	2002-08-05
Recent, this Task	914.91	899.43	901.36	898.1	915.75	915.56	899.46	901.37	898.41	915.83	2020-10-14
Measured Sett. at Project Completion (Inches)	**0.96**	**0.48**	**0.36**	**0.36**	**0.48**	**1.2**	N. A[a]	**0.36**	**0.36**	**0.48**	
Measured Settlements under this Task (Inches)	**2.52**	**-0.12**	**0.12**	**-5.52[b]**	**0.96**	**-4.2[b]**	**0.36**	**0.24**	**-7.08[b]**	**0.72**	

[a] Not Available, [b] Monuments could not be located.

3.5 CUY/SUM-271-00.00/14.87

The CUY/SUM-271-00.00/14.87 and CUY/SUM-480-29.58/00.00 project calls for the design and construction of three (3) new retaining walls identified as RW-1 (WS1), RW-2 (SW1), and RW-3 (WS2) in Summit/Cuyahoga Counties, Ohio. These retaining walls were constructed in association with constructing two additional Lanes identified as S-W and W-S located along the outside shoulders of IR-271 SB and NB between the Summit County Line and Alexander Road. Settlements were monitored for RW-1 (WS1). Wall plans are included in Appendix 1. Soil profiles and Geotechnical Report geotechnical recommendations are presented in Appendix 2.

On October 6, 2020, a site visit was performed by Jamal Nusairat and Dave Traini of E.L. Robinson Engineering. The cantilevered cast-in-place concrete wall was inspected and founded to be plumb vertically; no leaning or sliding of the walls was observed. The existing retaining wall foundation founded on spread footing is functioning as designed. Figure 3.5 shows RW-1 (WS1).

Figure 3.5: RW-1 (WS1)

Field exploration was performed using two boreholes per the original geotechnical report. The subsurface soils encountered in both test borings were predominantly cohesive in nature and consisted of both fill materials and natural soils. The fill materials located above the natural soils consisted of silt and clay (A-6a) and silty clay (A-6b). The fill material's approximate thickness was 8.5 feet in boring test B-007-1-13 and 3.5 feet in boring test B-007-4-13. Natural soils encountered above bedrock in boring test B-007-4-13 and to the termination depth in boring test B-007-1-13 consisted of sandy silt (A-4a), silt, and clay (A-6a), non-plastic sandy silt (A-4a), and coarse and fine sand (A-3a). Bedrock consisting of gray, severely to highly weathered shale was encountered at an approximate depth of 59.8 feet in boring test B-007-4-13. The consistency of the cohesive soils ranged from "medium stiff" to "hard" but was generally "very stiff". The relative density of the non-cohesive soils ranged from "dense" to "very dense".

Table 3.5 presents the recorded monitoring data for RW-1 (WS1). At project completion, the wall did not experience any settlements based on the provided data. Settlements data were not collected for this recently constructed wall under this task.

It should be noted that consolidation settlement calculations require soil parameters that are not included in the provided soil reports. The only soil report that provided such information is CUY/SUM 271-00.00/14.87 RW-1, as presented in Appendix 3. Therefore, the estimated settlement for this wall is the total settlements.

Table 3.5: RW-1 (WS1) Settlement Monitoring Data

Stage	Elevation (ft.)						Date
	Sta. 3243+76	**Sta. 3244+46.6**	**Sta. 3245+29.82**	**Sta. 3245+85.3**	**Sta. 3246+40.78**	**Sta. 3247+24.00**	
After Footing Concrete is Placed	1038.55	1038.475	1038.55	1038.505	1038.485	1037.99	1/8/2019
After Wall Concrete is Placed and backfilled	1038.55	1038.475	1038.55	1038.505	1038.485	1037.99	7/26/2019
Project Completion	1038.55	1038.475	1038.55	1038.505	1038.485	1037.99	10/30/2019
Recent, this Task	N. A[a]	N. A[a]	N. A[a]	N. A[a]	N. A[a]	N. A[a]	
Measured Sett. at Project Completion (Inches)	**N.A**	**N.A**	**N.A**	**N.A**	**N.A**	**N.A**	
Measured Settlements under this Task (Inches)	N. A[a]	N. A[a]	N. A[a]	N. A[a]	N. A[a]	N. A[a]	

[a] Not Available, settlements data were not collected under this task

3.6 Comparison Between Estimated and Measured Settlements

In this section a comparison is made between the estimated and measured settlements. Measured settlements are presented in the previous sections of this report. Hough (1959) is used to estimate the settlements of spread footing on cohesionless soil as specified in section 10.6.2.4 of the AASHTO LRFD Bridge Design Specifications.

Spread footing settlements should be estimated using computational methods based on the results of laboratory or insitu testing, or both. The soil parameters used in the computations should be chosen to reflect the loading history of the ground, the construction sequence, and the effects of soil layering.

Generally conservative settlement estimates may be obtained using the elastic half-space procedure or the empirical method by Hough (1959). The Hough method has several advantages over other methods used to estimate settlement in cohesionless soil deposits, including express consideration of soil layering and the zone of stress influence beneath a footing of finite size. The subsurface soil profile should be subdivided into layers based on stratigraphy to a depth of about three times the footing width. The maximum layer thickness should be about 10 ft. Settlements of cohesionless soil can be estimated using Hough (1959) method as explained in the equation (3.1 and 3.2).

$$S_e = \sum_{i=1}^{n} \Delta H_i \quad \text{...} \quad (3.1)$$

$$\Delta H_i = H_c \frac{1}{C'} \log\left(\frac{\sigma_o' + \Delta\sigma_v}{\sigma_o'}\right) \quad \text{..} \quad (3.2)$$

Where:

n= Number of soil layers within zone of stress influence of the footing

ΔH_i= Elastic settlement of layer i (ft)

H_c= Initial height of layer i (ft)

C'= Bearing Capacity Index, from Figure 10.6.2.4.2-1 of the AASHTO LRFD Bridge Specifications.

σ_o'= Initial average effective stress of the subdivided soil layer.

$\Delta\sigma_v$ = Vertical stress increase in the subdivided soil layer due to applied foundation load.

Settlement calculations for all structures are presented in Appendix 3. Table 3.6 presents a comparison between estimated and measured settlements. Comparison is made between the estimated and measured settlements at the end of construction and the recently measured values. It should be noted that average measured settlements were calculated by taking the average of two monuments readings for each footing. Current measured settlements of FAI-33-13.09 pier 3 was ignored because survey monument information was not available.

It can be noted that Hough's method tends to over predict the immediate settlements (measured at the end of construction). It also over predicts the current measured settlements for most of the footings. However, Hough's method significantly under predicts the current measured settlements for some footings where cohesive soil (A-6a) layers exist, such as CUY-77-14.35 Wall 4 and rear abutment of FAI-33-13.09. Therefore, it is important to calculate the long-term settlements when cohesive soils exist.

Per the Geotechnical report of CUY/SUM-271-00.00/14.87, the recently constructed Wall RW-1 (WS1) is expected to experience an immediate settlement of 0.26 inches and consolidation settlements of 0.52 inches. The wall is expected to experience a total settlement of 0.78 inches. Consolidation settlements are very important because of the existed cohesive soil below the wall's footing.

Table 3.6 Comparison Between Estimated and Measured Settlements

Project	Structure	Estimated Settlements (Inches)	Ave. Measured Settlements (Inches)		(Estimated/Measured) Settlement	
			End of Construction	Current Settlement	End of Construction	Current Settlement
MAH-680-2.83	Pier 1	0.69	0.24	0.36	2.88	1.92
	Pier 2	0.5	0.24	0.48	2.08	1.04
	Pier 3	0.47	0.12	0.18	3.92	2.61
CUY-77-14.35	Wall 1	0.69	0.06	0.36	11.50	1.92
	Wall 4	0.54	0.06	6.3	9.00	0.09
	LT. RA.Wing Wall (Wall 2)	0.89	0.24	0.84	3.71	1.06
	RT. FA.Wing Wall (Wall 3)	0.76	0.24	0.66	3.17	1.15
FAI-33-13.09	Rear Abut.	0.91	0.96	2.52	0.95	0.36
	Pier 1	1.03	0.48	0.36	2.15	2.86
	Pier 2	1.3	0.36	0.18	3.61	7.22
	Pier 3	1.03	0.36	N. A	2.86	N. A
	FR. Abut.	0.73	0.48	0.84	1.52	0.87
CUY/SUM-271-00.00/14.87	RW-1 (WS1)	0.78	0.0	N. A	N. A	N. A

CHAPTER 4: REVIEW OF IRI DATA AND DIFFERENTIAL SETTLEMENT BETWEEN SUBSTRUCTURES

4.1 General

This chapter presents collected profile data for CUY/SUM-271-0.00/14.67: CUY/SUM-480-29.58/0.00 and CUY-77-14.35. The data were examined to assess the effect of retaining walls, constructed on spread footings, on ride quality, in terms of the International Roughness Index (IRI). The profiles presented included the pavement adjacent to walls, approach slabs, and the bridge deck. This chapter also presents the differential settlements between substructures.

4.2 Collected Profile Data

Profile data collected on CUY/SUM-271-0.00/14.67:CUY/SUM-480-29.58/0.00 and CUY-77-14.35 was analyzed to evaluate the effect of walls constructed on spread footers on ride quality, in terms of International Roughness Index (IRI).

Both sites are in Cleveland, Ohio. The CUY/SUM-271 site is located at the Summit County and Cuyahoga County line, at the southern I-271 and I-480 interchange as shown in Figure 4.1. The location and length of the profile runs are shown in white.

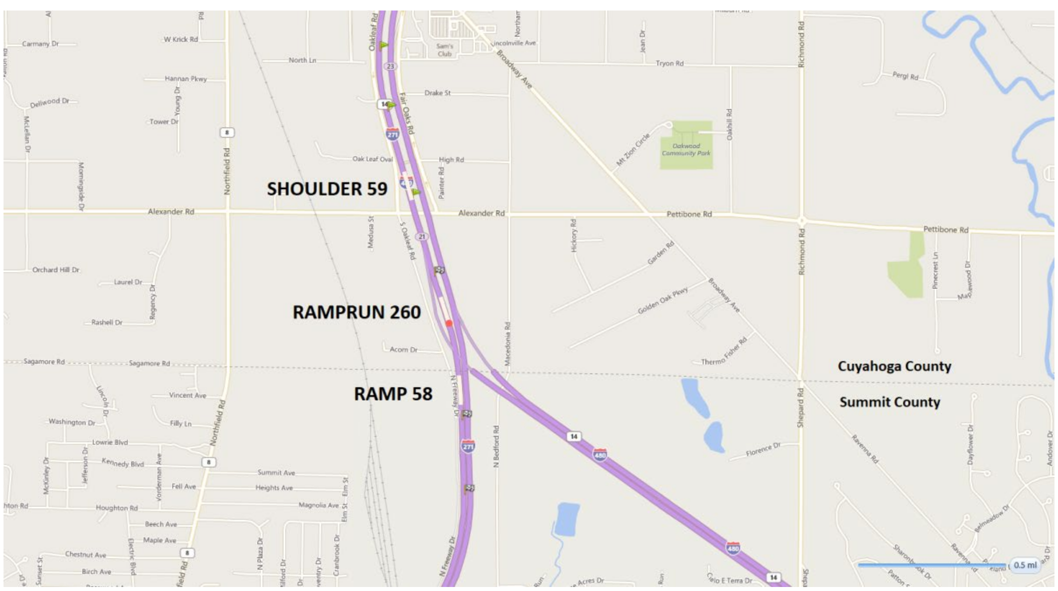


Figure 4.1 CUY/SUM-271-0.00/14.67
CUY/SUM-480-29.58/0.00

Three profiles were provided for the CUY/SUM-271/480 section: Shoulder 59, Ramp 260, and Ramp 58. A 353’ long wall on spread footer was constructed between the southbound lanes and an entrance ramp on the Ramp 58 section (Figures 4.2 and 4.3).

Figure 4.2 Picture of I-271 Wall (Google Maps)

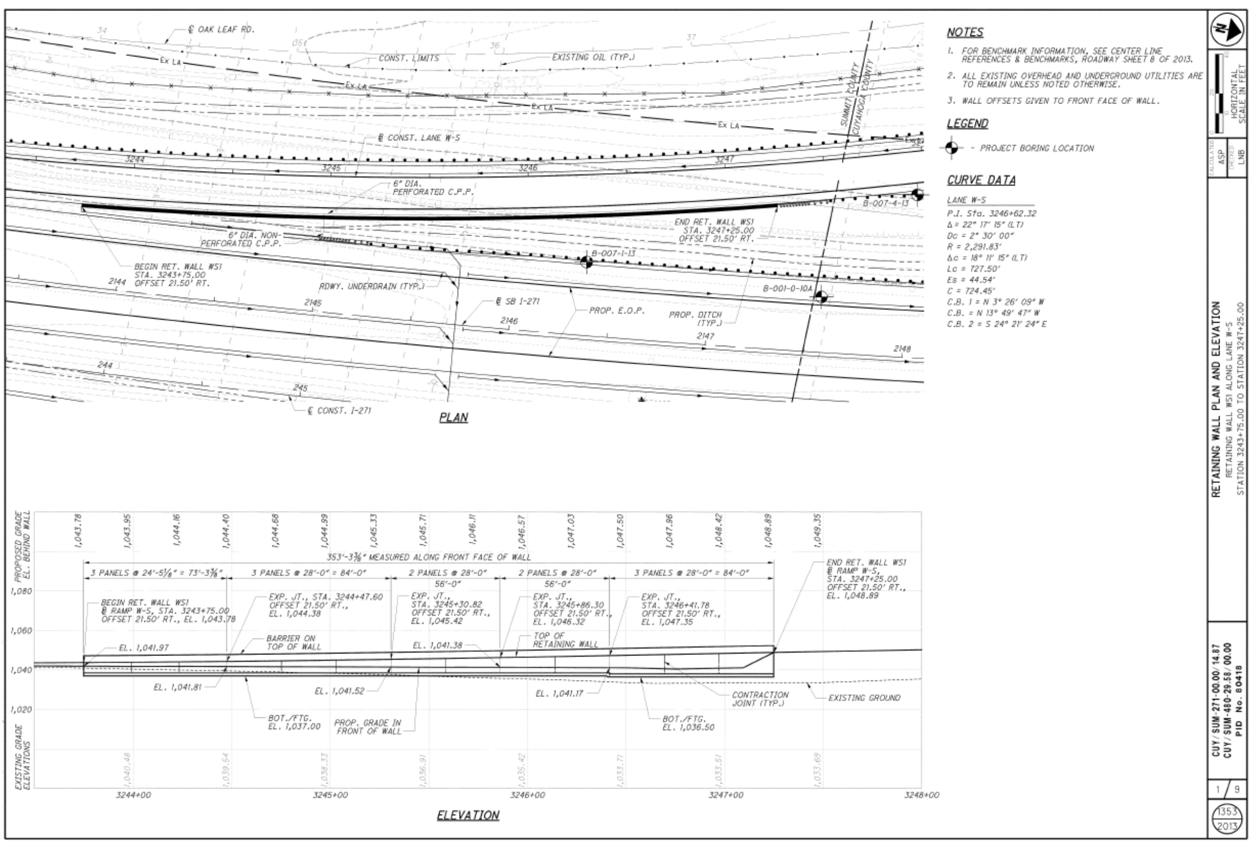


Figure 4.3 Plan Details, I-271 Wall

The CUY-77 is located at the interchange of I-77 and I-490 as shown in Figure 4.4. The profile run on I-77 was one continuous run shown in white in the Figure. The walls at this location were extensions of the bridge wingwalls and are circled in red in Figure 4.5. Wall 4 is shown in Figure 4.6.

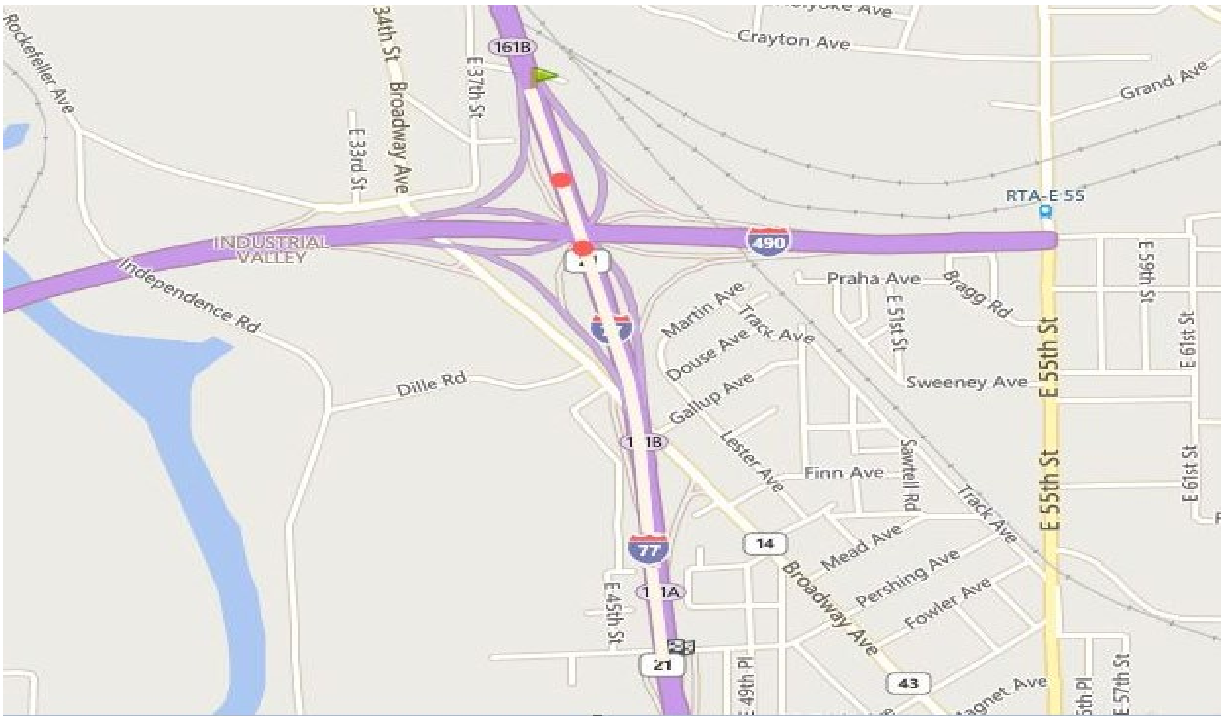


Figure 4.5 CUY-77-14.35

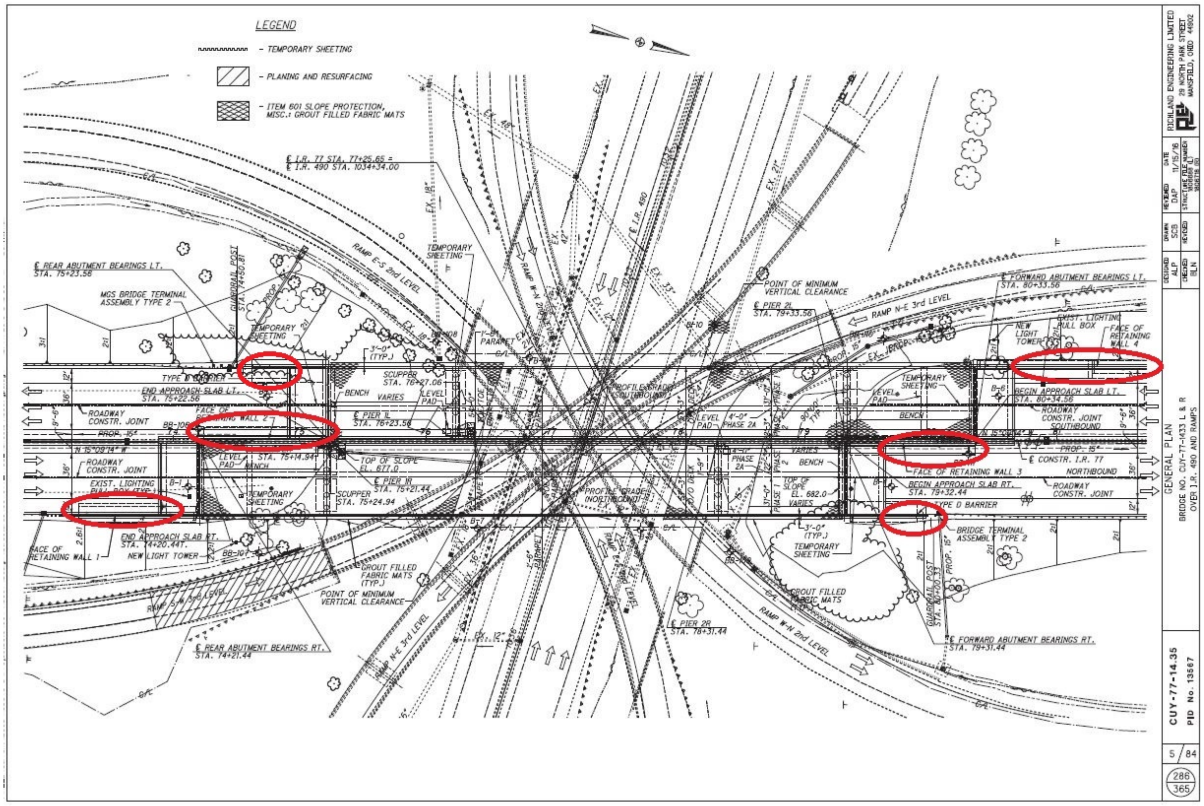


Figure 4.5 Plan details, CUY-77 Walls

Figure 4.6 CUY-77-14.35 Wall 4 (Google Maps)

ProVal 3.61.34 was used to analyze the profiles. ProVal is a computer application developed by the Transtec Group to view and analyze pavement profiles (www.roadprofile.com). The ride quality module was used to analyze the data. This module allows sections to be divided into fixed intervals and the IRI determined for each interval. A fixed length of 17.6’ was chosen to provide enough subsections at each site to allow a comparison between sites. Figure 4.7 shows the IRI plot for Ramp 58 after division into 17.6' section. The sections on I-271 were processed as received.

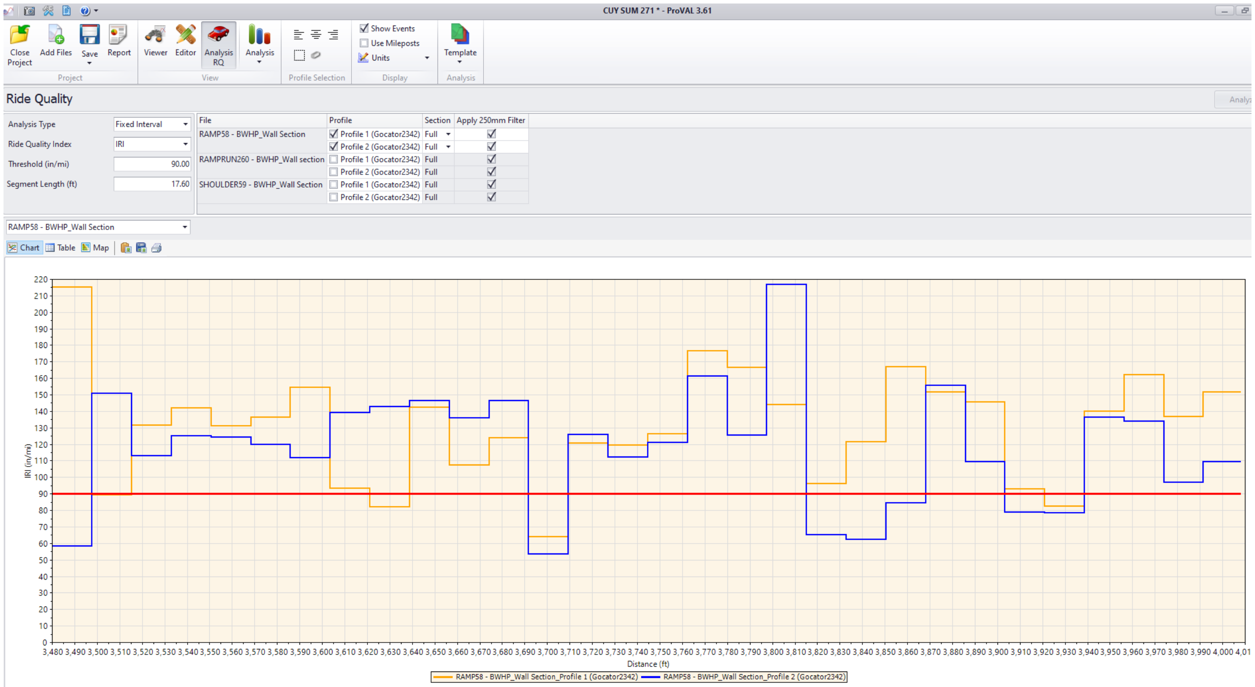


Figure 4.7 RAMP58 IRI plot in ProVal

The profiles provided for I-77 were continuous runs which included the pavement without walls, pavement adjacent to walls, approach slabs, and the bridge deck. This data was processed by identifying the limits of the approach slabs (see Figure 4.8) in the profile, deleting the profile of the approach slab and bridge deck, and dividing the data into pavement adjacent to walls and pavement without walls.

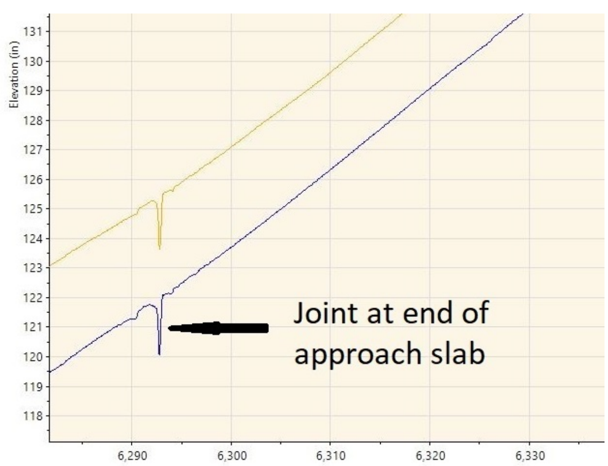


Figure 4.8 Identifying location of approach slab in profile trace

Box plots were used to evaluate the effect of retaining walls on spread footings on pavement ride quality. Figures 4.9 and 4.10 are box plots IRI for sections with and without walls for CUY-271 and CUY-77, respectively. The bottom and top of the box represents the 1st and 3rd quartiles, respectively. The line inside the box represents the median value and the diamond inside the box represents the mean value. The two lines extending from the box represents values outside the 1st and 3rd quartile and the horizontal bars on the end of the vertical lines represent the minimum and maximum values. Box plots are useful for determining the spread and skew of the data. When comparing IRI for the sections, if the boxes do not overlap, there is a difference in the IRI for the sections. If the boxes overlap, but do not include both medians, there is likely a difference in the two IRI values. If the boxes overlap and include both medians, both sections are considered to have the same IRI values.

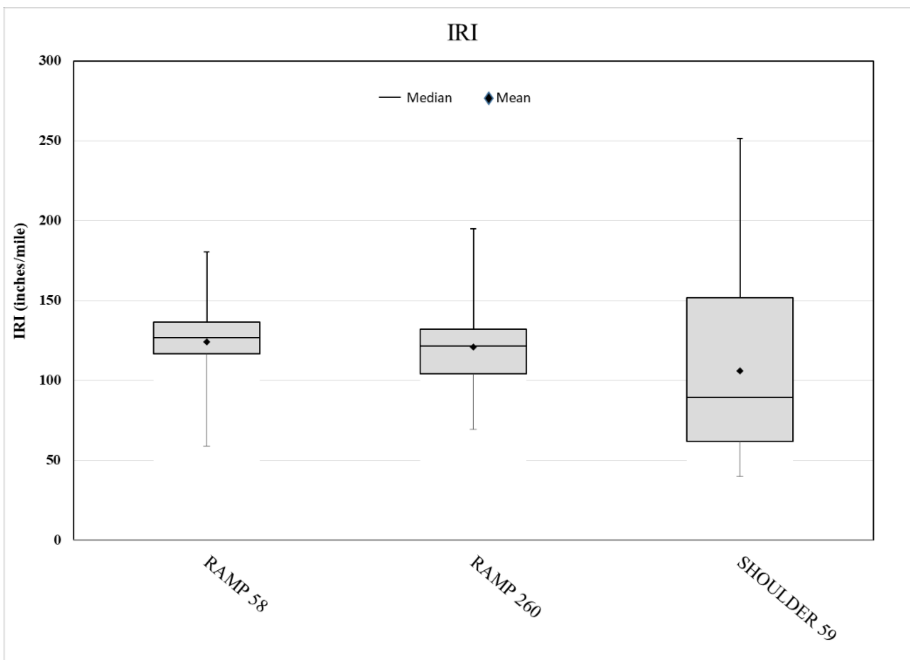


Figure 4.9 CUY 271 Box Plot

The results in Figure 4.9 would indicate the IRI values on the section with the wall, RAMP 58, is statistically the same as the IRI value for the RAMP 260 section. However, the IRI values of SHOULDER 59 is likely different than the IRI values on RAMP 58 and RAMP 260 and has a better ride number.

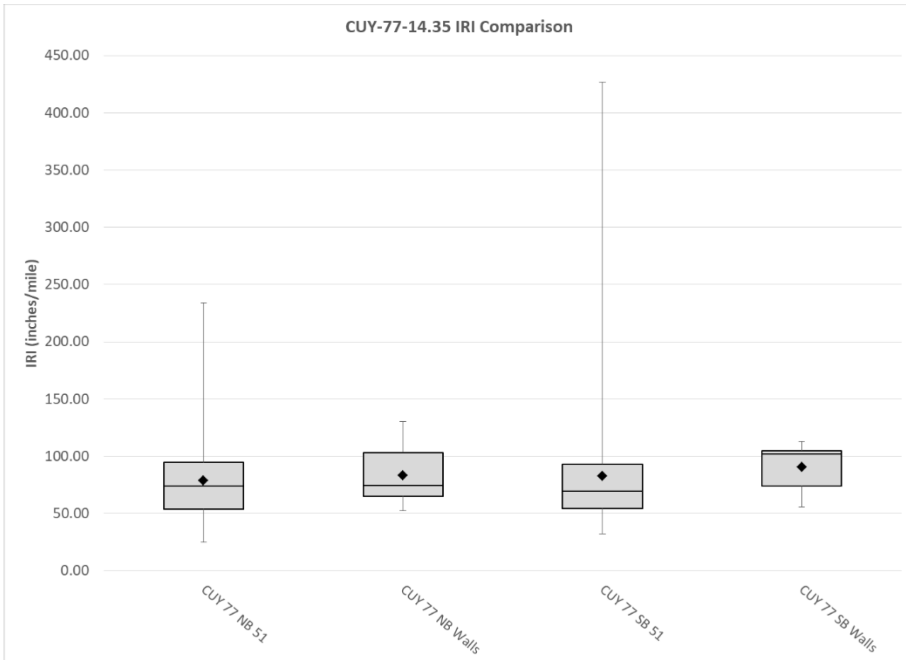


Figure 4.10. CUY-77 Box Plot

The results in Figure 4.10 would indicate the IRI values on the sections with and without the wall in the southbound direction are statistically the same. In the northbound direction, the IRI values on the sections with and without the wall in the southbound direction are statistically different, the section with the wall having the worst ride number.

The inconsistent results between CUY-77 and CUY-271 would suggest there are factors other than the presence of the wall, i.e. pavement construction sequence, quality of construction material, etc., which are affecting IRI values and, based on IRI, the effect of the walls on pavement ride quality is inconclusive.

4.3 Differential Settlements between Substructures

Differential settlements between substructures are presented in Table 4.1. The table shows differential settlements at the end of construction and the current differential settlements based on recent survey performed in this study.

MAH-680-0283, at the end of construction, did not experience any differential settlements between pier 1 and 2 but it experienced 0.12 inches between pier 2 and 3. Based on recent settlement monitoring data, the structure experienced 0.12 inches between pier 1 and 2; and 0.3 inches between pier 2 and 3. It should be noted that those values are within tolerable settlement limits.

FAI-33-1309 experienced tolerable differential settlements of (0.0 to 0.48) inches at the end of construction. However, recent settlement monitoring survey showed differential settlements of (0.12 to 2.16) inches, the values at the piers less than those at the end of construction. This may be due to survey issues. It should be noted that differential settlements of 2.16 inches between the rear abutment and pier 1 is recorded recently, because of the 2.52 inches current settlements at the rear abutment. This may be due to the encountered cohesive soils consisting of stiff to hard silt and clay (A-6a) and silty clay (A-6b) at deeper layers below the rear abutment. There are no signs of this differential settlement at the rear abutment approach.

Table 4.1 Differential Settlements between Substructures

Structure No.	**Substructure**	**Ave. Measured Settlements (Inches)**		**Differential Settlements between Substructures (Inches)**		
		End of Construction	**Current Settlement**	**Two Substructures**	**Based on End of Construction**	**Based on Current Settlement**
MAH-680-0283	Pier 1	0.24	0.36	Pier 1 and 2	0.0	0.12
	Pier 2	0.24	0.48	Pier 2 and 3	0.12	0.30
	Pier 3	0.12	0.18	----------	----------	----------
FAI-33-1309	Rear Abut.	0.96	2.52	Rear Abut. and Pier 1	0.48	2.16
	Pier 1	0.48	0.36	Pier 1 and 2	0.12	0.18
	Pier 2	0.36	0.18	Pier 2 and 3	0.0	N. A
	Pier 3	0.36	N. A	P3 and FR. Abut. = 0.12	0.12	N. A
	FR. Abut.	0.48	0.84	----------	----------	----------

CHAPTER 5: FINDINGS AND RECOMMENDATIONS

5.1 Findings

The following summarizes the findings of this study:

- Spread footings were monitored after pouring the footings' concrete, before and after beams placement, after pouring the deck's concrete, and after project completion. Recently, under this task, footings were surveyed to get the final settlements values. Generally, spread footing performed well except in some locations where cohesive soils exist, or there was an issue with the survey benchmark (either it settled or could not be located).

- The empirical settlement prediction method proposed by Hough (1959) should be utilized when a single value cannot represent the elastic modulus of the sandy subsoil layers (i.e. when the corrected SPT-N value varied significantly with depth over the depth of influence).

- The settlement of spread footings on cohesionless soil can be estimated using the method proposed by Hough (1959) as specified in section 10.6.2.4 of the AASHTO LRFD Bridge Design Specifications. Based on the results presented in Chapter 3, this method can be used with some confidence.

- It can be concluded that Hough's method tends to over predict the immediate settlements (measured at the end of construction). It also over predicts the current measured settlements for most of the footings. However, Hough's method significantly underpredicts the current measured settlements for some footings where cohesive soil (A-6a) layers exist. Therefore, it is important to calculate the long-term settlements when cohesive soil exists.

- The collected data at the sites illustrated that the spread footings could be used to support the highway structures satisfactorily, given that granular subsurface conditions are adequate (i.e., the corrected SPT-N value is larger than 20 blows/ft).

- All structures in this study experienced tolerable differential settlements between substructures.

- The IRI inconsistent results between CUY-77 and CUY-271 would suggest there are factors other than the presence of the wall, i.e., pavement construction sequence, quality of construction material, etc., which are affecting IRI values and, based on IRI, the effect of the walls on pavement ride quality is inconclusive.

5.2 Recommendations

- Reference monuments for substructure units founded on spread footing on soil are important to collect and document historical performance data. Also, to assure the foundation bearing material is performing as designed. Having such data is essential to improve the reliability of spread footings and evaluate settlement prediction methods' accuracy.

- Reference monuments, for substructure units founded on spread footing foundations on soil, should be surveyed annually as part of the annual bridge inspection. This way spread footing design can be verified, performance can be investigated, and data reviewed to take any necessary action.

- Coordinates of monuments and reference benchmarks should be provided in the as-built plans to assure accurate readings of the settlements in the future.

- It is recommended to install target points on substructures so that elevation data could be more readily collected. Field review indicated damaged PVC pipes, missing PVC pipes, clogged holes, and PVC pipes' vandalism. Targets would be more readily seen, and if placed, vandalism is deterred. Relative settlement measurement could be obtained with simple optical survey equipment.

- When proposing spread footings, the designer should pay attention to the subsurface investigation and accurately estimate the consolidation settlement for spread footings resting on saturated cohesive soils.

- ODOT Bridge Design Manual (BDM) states that "*All spread footings at all substructure units, not founded on bedrock, are to have elevation reference monuments constructed in the footings. This is for the purpose of measuring footing elevations during and after construction for the purpose of documenting the performance of the spread footings, both short term and long term*". The BDM should provide more details about the long-term measurement of footing elevations. We recommend measuring footing elevation annually, and the collected data should be stored in a database to track the long-term performance of structures.

- Total expected settlements (immediate and long-term) are usually calculated and presented in the Geotechnical Report. Total expected settlements should be provided in the Spread Foundation Plan Note to be compared with measured values in the future. Therefore, its recommended to include this statement in the Spread Foundation Plan Note, "*The footing is expected to experience an immediate settlement of ……..inches and consolidation settlements of ……inches.*

REFERENCES

- AASHTO (2017). "LRFD Bridge Design Specifications". Section 10.6 (Spread Footings).

- Abu-Hejleh, N., Hanneman, D., White, D.J., Wang, T., and Ksouri, I. (2006). "Flowfill and MSE Bridge Approaches: Performance, Cost, and Recommendations for Improvements." Report No. CDOT-DTD-R-2006-2, Colorado Department of Transportation.

- Abu-Hejleh, N., Mohamed, K., & Alzamora, D. (2014). Recommendations for the Design of Spread Footings on Soils to Support Highway Bridges. In Geo-Congress 2014: Geo-characterization and Modeling for Sustainability (pp. 1998-2009).

- Baus, R. L. (1992). "Spread Footing Performance Evaluation in South Carolina." Report No. F92-102, South Carolina Department of Highways and Public Transportation and the Federal Highway Administration, Columbia, SC.

- Briaud, J.-L. (1993). "The National Geotechnical Experimentation Sites at Texas A&M University: Data Collected Until 1992", Report to the Federal Highway Administration and the National Science Foundation, Civil Engineering, Texas A&M University.

- Cai, C.S., Voyiadjis, G.Z., Shi, X., 2005. "Determination of Interaction between Bridge Concrete Approach Slab and Embankment Settlement," Final Report. Louisiana Transportation Research Center (LTRC), Louisiana Department of Transportation and Development (LADOTD), Baton Rouge, LA, p. 150. Report No. FHWA/LA.05/403.

- DiMillio, A. F. (1982). "Performance of Highway Bridge Abutment Supported By Spread Footings on Compacted Fill." Report No. FHWA/RD-81/184, FHWA Staff Study, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C.

- FHWA (2001). "Shallow Foundations: Reference Manual." FHWA-NHI-01-023, Authors: Munfakh, G., Arman, A., Collin, J.G., Hung, J.C.-J., and Brouillette, R.P., Federal Highway Administration, U.S. Department of Transportation, Washington, D.C. In 2012, the training course was updated in accordance with the AASHTO LRFD Bridge Design Specifications, 5th Edition, 2010.

- FHWA (2002). "Geotechnical Engineering Circular 6, Shallow Foundations." FHWA SA-02-054, Author: Kimmerling, R.E., Federal Highway Administration, U.S. Department of Transportation, Washington, D.C.

- FHWA (2006). "Soils and Foundations: Reference Manual." FHWA-NHI-06-088, Authors: Samtani, N.C., and Nowatzki, E.A., Federal Highway Administration, U.S. Department of Transportation, Washington, D.C.

- FHWA (2010). "Selection of Spread Footings on Soils to Support Highway Bridge Structures." FHWA-RC/TD-10-001, Authors: Samtani, N.C., and Mertz, D.R. Federal Highway Administration, U.S. Department of Transportation, Washington, D.C.

- Fragkakis, N. P., & Lambropoulos, S. (2004). A quantity and cost estimate model for concrete road bridges. Tech. Chron. Science Journal, TCG, 1, 24.

- Gifford, D. G., Kraemer, S. R., Wheeler, J. R., and McKown, A. F. (1987). "Spread Footings for Highway Bridges." Report No. FHWA/RD-86/185, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., 229 pp.

- Keene, P. (1978). "Tolerable Movements of Bridge Foundations." Transportation Research Record (Journal of Transportation Research Board), No. 678, National Research Council, Washington, D.C., pp. 1-6.

- Masada, T., & Sargand, S. (2009). Latest Highway Bridge Spread Footing Research Findings in Ohio. In Contemporary Topics in Ground Modification, Problem Soils, and Geo-Support (pp. 488-495).

- Menn, C. (1990). Prestressed concrete bridges, Birkhauser, Germany. Montgomery, D. C., Peck, E. A., and Vining, G. G. (2006). Introduction to linear regression analysis, Wiley, New York.

- Moulton, L. K., Ganga Rao, H. V. S., and Halvorson, G. T. (1982). "Tolerable Movement Criteria for Highway Bridges." Vol. 1. Interim Report. Report No. FHWA/RD-81/162, Federal Highway Administration, Washington, D.C., 117 pp.

- Peck, R. B., and Bazaraa, A. R. S. (1969). "Discussion of 'Settlement of Spread Footings from SPT Values.' by D'Appolonia, D'Appolonia, and Brissette" Journal of Soil Mechanics and Foundation Division, American Society of Civil Engineers (ASCE), Vol. 95, No. 3, pp. 905-909.

- Sargand, S. M., Hazen, G. A., and Masada, T. (1997). "Field and Laboratory Performance Evaluations of Spread Footings for Highway Bridges." Report No. FHWA/OH-98/017, Final Report to Ohio Dept. of Transportation, Civil Engineering Dept., Ohio University, Athens, OH, 234+ pp.

- Sargand, S. M., and Masada, T. (April 2006). "Further Use of Spread Footing Foundations for Highway Bridges". State Job Number 14747(0) Final Report FHWA-OH-2006/8 Final Report to Ohio Dept. of Transportation, 332 p.

- Sargand, S. M., Masada, T., and Engle, R. (1999), "Spread Footing Foundation for Highway Bridge Applications." *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 125, No. 5, American Society of Civil Engineers (ASCE), Reston, VA, pp. 373-382.

- Walkinshaw, J. L. (1978). "Survey of Bridge Movements in the Western United States." Transportation Research Record (Journal of Transportation Research Board), No. 678, National Research Council, Washington, D.C., pp. 6-11.

- Yokel, F. Y. (1990). Proposed Design Criteria for Shallow Bridge Foundations. US Department of Commerce, National Institute of Standards and Technology.

Appendices

APPENDIX 1:

STRUCTURES PLANS

Pages 44-64

℄ CONSTRUCTION VESTAL ROAD
N 64° 12' 19" E

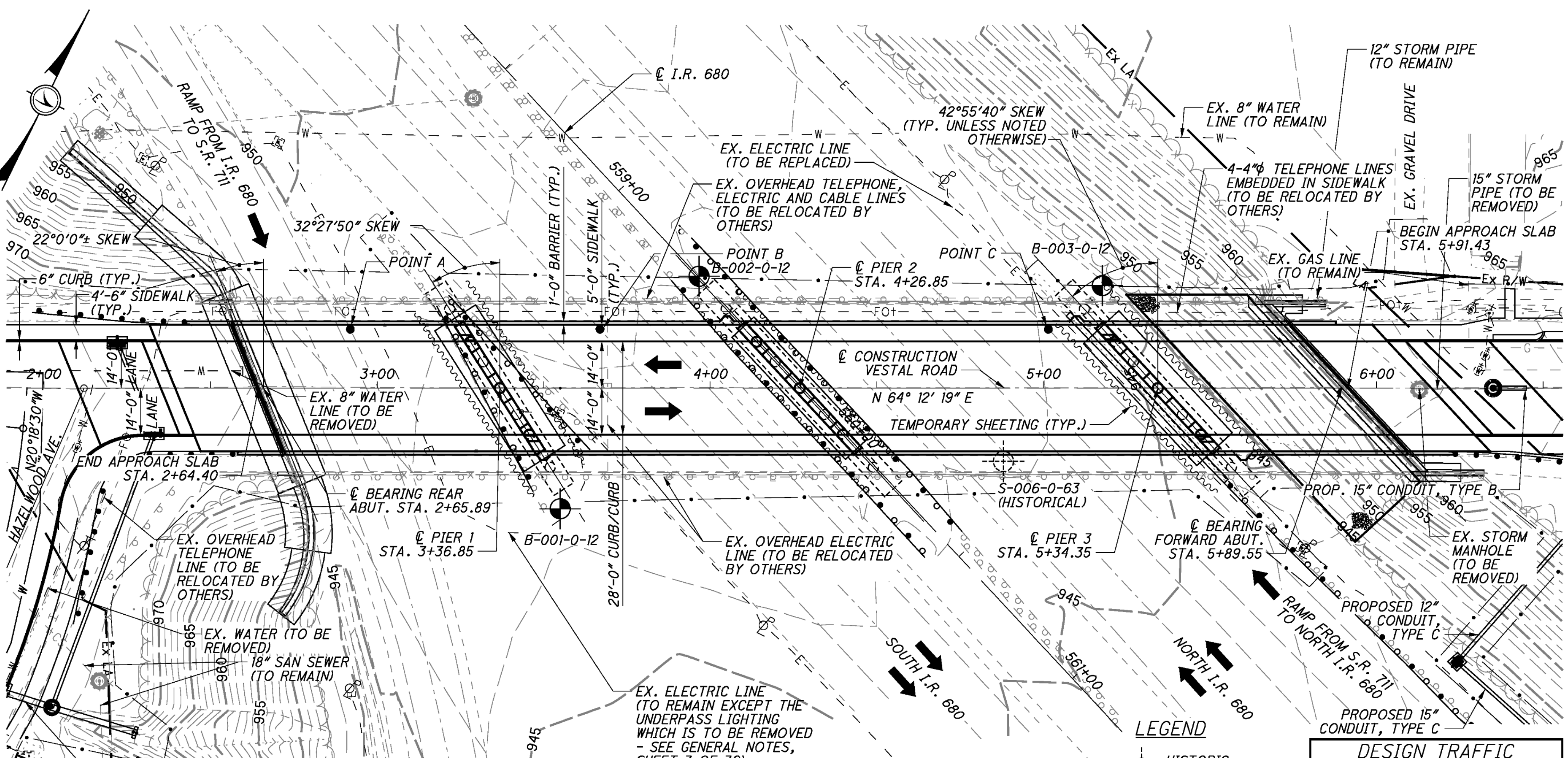
NOTES

EARTHWORK LIMITS SHOWN ARE APPROXIMATE. ACTUAL SLOPES CONFORM TO PLAN CROSS SECTIONS.

THE PROPOSED PROFILE GRADE IS WITHIN THE BRIDGE LIMITS. SEE ROADWAY PLANS FOR PAVEMENT ELEVATIONS BEYOND BRIDGE LIMITS.

ALL EXISTING SUBSTRUCTURE FOOTING ELEVATIONS ARE TAKEN FROM THE EXISTING PLANS AND HAVE BEEN ADJUSTED FOR APPROPRIATE DATUM ADJUSTMENT OF -0.69 FEET.

FOR MINIMUM LATERAL CLEARANCE DIMENSIONS FROM THE SUBSTRUCTURE UNDERNEATH THE BRIDGE, SEE THE GENERAL PLAN SHEET.



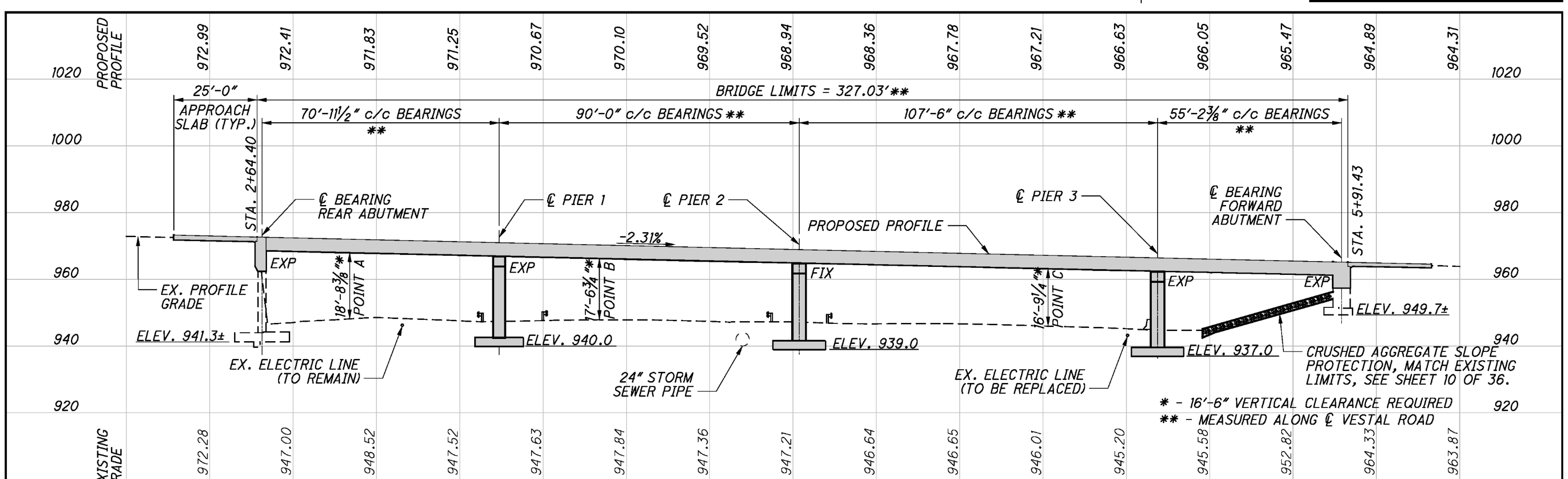
PLAN

LEGEND

HISTORIC BORING LOCATION

PROJECT BORING LOCATION

DESIGN TRAFFIC	
VESTAL ADT 2017	2800
VESTAL ADTT 2017	84
VESTAL ADT 2037	3300
VESTAL ADTT 2037	99



PROPOSED PROFILE ALONG ℄ CONSTRUCTION VESTAL ROAD

BENCHMARK DATA

BM#15, NORTHING: 534171.7908, EASTING: 2464756.8956, ELEV. 981.717, TOP OF ARROW MARKED BOLT ON TOP FLANGE OF FIRE HYDRANT
BM#27, NORTHING: 534445.2016, EASTING: 2465308.7332, ELEV. 966.532, TOP OF ARROW MARKED BOLT ON TOP FLANGE OF FIRE HYDRANT
BM #34, NORTHING: 534572.0906, EASTING: 2465591.4382, ELEV. 958.390, TOP OF "X" MARKED BOLT ON TOP FLANGE OF FIRE HYDRANT

EXISTING STRUCTURE

TYPE: 4 SPAN CONTINUOUS STEEL GIRDERS HAVING A HINGE WITH REINFORCED CONCRETE DECK ON CAP AND COLUMN PIERS AND STUB TYPE ABUTMENT WITH BOTH BEING FOUNDED ON SPREAD FOOTINGS

SPANS: 71'-0"±, 90'-0"±, 107'-6"±, 54'-0"± c/c BEARINGS (ALONG ℄ VESTAL ROAD)

ROADWAY: 40'-0" ROADAY WIDTH F/F CURB AND TWO 5'-0" SIDEWALKS WITH TWO 1'-0" BARRIERS

LOADING: C.F. 400 (57)

SKEW: VARIES (SEE PLAN VIEW)

APPROACH SLABS: AS-1-54 (25'-0" LONG)

ALIGNMENT: TANGENT

CROWN: 0.016± FT/FT

STRUCTURAL FILE NUMBER: 5006511

DATE BUILT: 1967

WEARING SURFACE: LATEX MODIFIED CONCRETE OVERLAY

DISPOSITION: REHABILITATION

PROPOSED STRUCTURE

PROPOSED WORK: REMOVE EXISTING SUPERSTRUCTURE AND THE THREE EXISTING PIERS AND RAISE THE ABUTMENT SEATS. ABUTMENTS ARE TO BE MODIFIED TO SEMI-INTEGRAL AND CONSTRUCT NEW PIERS THAT WILL BE CAP AND COLUMN. CONSTRUCT NEW SUPERSTRUCTURE.

TYPE: 4 SPAN CONTINUOUS PAINTED STEEL GIRDERS WITH REINFORCED CONCRETE DECK ON NEW SEMI-INTEGRAL ABUTMENTS, NEW BEARINGS AND NEW CAP AND COLUMN PIERS FOUNDED ON SPREAD FOOTINGS

SPAN: 70'-11½", 90'-0", 107'-6", 55'-2⅜" c/c BEARINGS (ALONG ℄ VESTAL ROAD)

ROADWAY: 28'-0" ROADWAY WIDTH F/F CURB AND TWO 5'-0" SIDEWALKS WITH TWO 1'-0" BARRIERS

LOADING: SUPERSTRUCTURE AND PIERS: HS-25 WITH 60 PSF FUTURE WEARING SURFACE

SKEW: VARIES (SEE PLAN VIEW)

APPROACH SLABS: 25'-0" LONG (AS-1-81)

WEARING SURFACE: 1" MONOLITHIC WEARING SURFACE

ALIGNMENT: TANGENT

CROWN: 0.016 FT/FT

COORDINATES: LATITUDE N41°07'09"
LONGITUDE W80°41'50"

STRUCTURE FILE NUMBER: 5006511

DESIGN AGENCY
McCormick Taylor

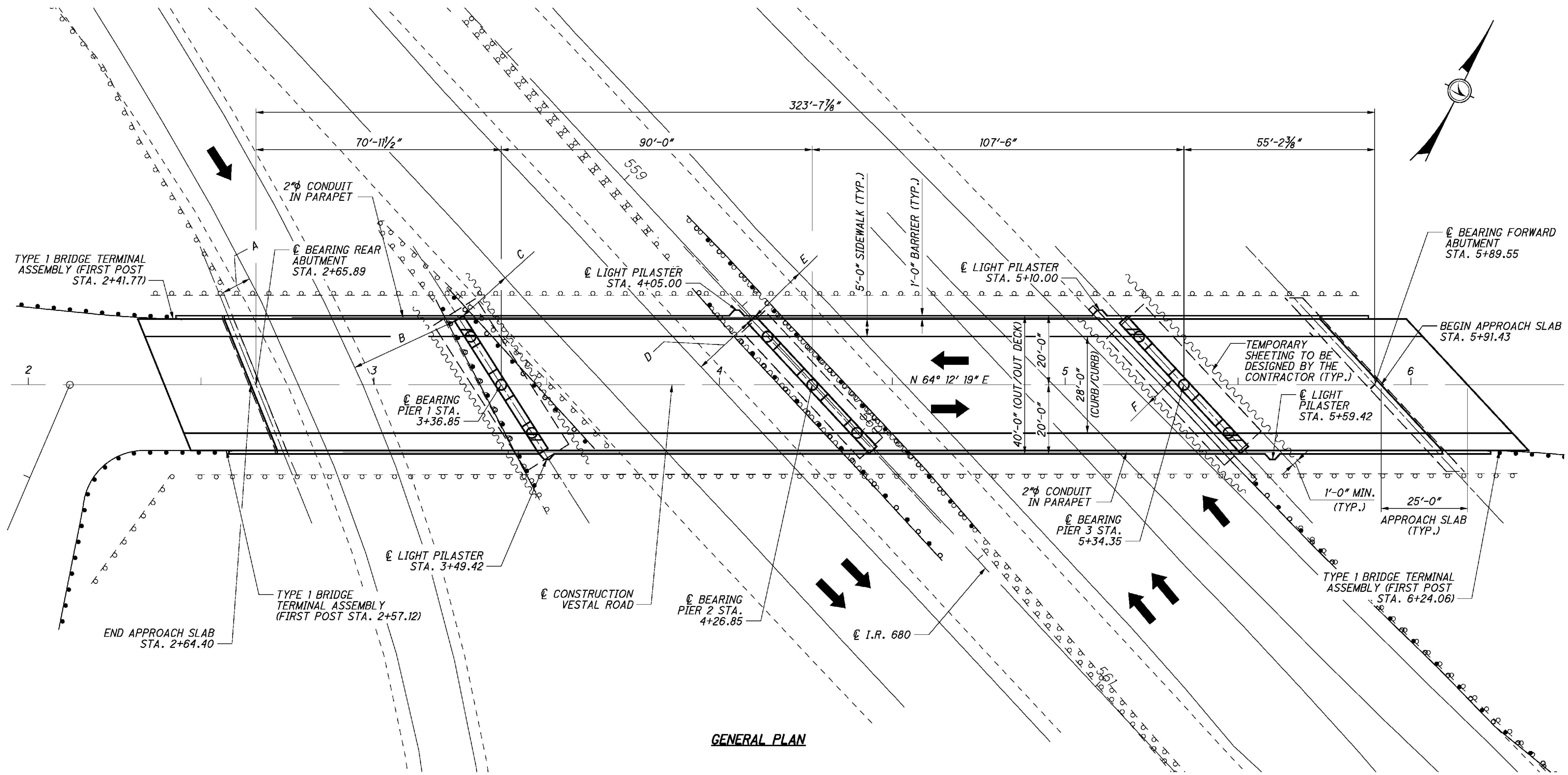
DESIGNED	CHECKED	DRAWN	REVISED	REVIEWED	DATE
JFM	CCJ	JFM		CCJ	04/14

STRUCTURE FILE NUMBER 5006511

MAHONING COUNTY
STA. 2+64.40
STA. 5+91.43

SITE PLAN
BRIDGE NO. MAH-680-0282
VESTAL ROAD OVER I.R. 680

MAH-680-2.83
PID No. 82941



GENERAL PLAN

LATERAL OFFSETS		
LOCATION	PROVIDED	REQUIRED
A	8'-6⅜"	6'-0"
B	31'-10⅛"	5'-6"
C	16'-3⅝"	14'-6"
D	18'-6¼"	9'-0"
E	17'-8¼"	9'-0"
F	9'-8⅜"	7'-8"

NOTES:

1. FOR LIGHT PILASTER DETAILS, SEE SHEET 28 OF 36 AND ODOT STANDARD DRAWING HL-20.14.

DESIGN AGENCY
McCormick Taylor
Engineers & Planners Since 1946
445 HUTCHINSON AVENUE SUITE 540 COLUMBUS, OH 43235

DESIGNED	DRAWN	REVIEWED	DATE
JFM	KWR/JFM	CCJ	04/14
CHECKED	REVISED	STRUCTURE FILE NUMBER	
CCJ		5006511	

GENERAL PLAN
BRIDGE NO. MAH-680-0282
VESTAL ROAD OVER I.R. 680

MAH-680-2.83
PID No. 82941

2/36

47/87

T:\Projects\All ODOT Setup\MAH\82941\Structures\Sheets\82941GP001.dgn 6/12/2015 9:52:39 AM dh

STRUCTURE GENERAL NOTES

REFER TO THE FOLLOWING STANDARD BRIDGE DRAWINGS:

AS-1-81	REVISED	01-18-13
BR-2-98	REVISED	07-20-12
GSD-1-96	REVISED	07-19-02
SICD-1-96	REVISED	07-18-14
VPF-1-90	REVISED	04-15-11

AND TO THE FOLLOWING SUPPLEMENTAL SPECIFICATIONS:

800 DATED 10/16/15

DESIGN SPECIFICATIONS:
THIS STRUCTURE CONFORMS TO "STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES" ADOPTED BY THE AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS, 17th EDITION, 2002, AND THE ODOT BRIDGE DESIGN MANUAL, 2004.

DESIGN LOADING:
HS25, CASE II, THE ALTERNATE MILITARY LOADING AND FUTURE WEARING SURFACE (FWS) OF 60 PSF.

DESIGN DATA:
CONCRETE CLASS QC2 - COMPRESSIVE STRENGTH 4.5 KSI (SUPERSTRUCTURE)
CONCRETE CLASS QC1 - COMPRESSIVE STRENGTH 4.0 KSI (SUBSTRUCTURE)
REINFORCING STEEL - ASTM A615 OR A996, GRADE 60, Fy = 60,000 PSI
SPIRAL REINFORCEMENT - ASTM A82 OR A615

STRUCTURAL STEEL
ASTM A709 GRADE 50 - YIELD STRENGTH 50,000 P.S.I.

DECK PROTECTION METHOD:
EPOXY COATED REINFORCING STEEL, 2½" CONCRETE COVER, CLASS QC2 CONCRETE

MONOLITHIC WEARING SURFACE:
MONOLITHIC WEARING SURFACE IS ASSUMED, FOR DESIGN PURPOSES, TO BE 1 INCH THICK.

ITEM 202-PORTIONS OF STRUCTURE REMOVED, OVER 20 FOOT SPAN, AS PER PLAN:
REMOVAL LIMITS:
SUPERSTRUCTURE - REMOVE IN ITS ENTIRETY
REAR ABUTMENT- REMOVE SEAT AND BACKWALL TO ELEV. 962.3
PIER 1 - REMOVE IN ITS ENTIRETY
PIER 2 - REMOVE IN ITS ENTIRETY
PIER 3 - REMOVE IN ITS ENTIRETY
FORWARD ABUTMENT - REMOVE BACKWALL TO ELEV. 957.9 TO 958.9 (VARIES)
FORWARD ABUTMENT WINGWALLS - REMOVE PORTIONS OF WINGWALLS. SEE SHEET 7 OF 36 FOR REMOVAL LIMITS.

THIS ITEM SHALL INCLUDE THE ELEMENTS INDICATED IN THE PLANS AND GENERAL NOTES. ITEMS TO BE REMOVED INCLUDE ALL EXISTING MATERIALS BEING REPLACED BY NEW CONSTRUCTION AND MISCELLANEOUS ITEMS THAT ARE NOT SHOWN TO BE INCORPORATED INTO THE FINAL CONSTRUCTION AND ARE DIRECTED TO BE REMOVED BY THE ENGINEER. THE USE OF EXPLOSIVES, HEADACHE BALLS AND/OR HOE-RAMS WILL NOT BE PERMITTED. THE METHOD OF REMOVAL AND THE WEIGHT OF HAMMER SHALL BE APPROVED BY THE ENGINEER. PERFORM ALL WORK IN A MANNER THAT WILL NOT CUT, ELONGATE OR DAMAGE THE EXISTING REINFORCING STEEL TO BE PRESERVED. CHIPPING HAMMERS SHALL NOT BE HEAVIER THAN THE NOMINAL 90-POUND CLASS. PNEUMATIC HAMMERS SHALL NOT BE PLACED IN DIRECT CONTACT WITH REINFORCING STEEL THAT IS TO BE RETAINED IN THE REBUILT STRUCTURE. SUBMIT CONSTRUCTION PLANS ACCORDING TO CMS 501.05.

SUBSTRUCTURE CONCRETE REMOVAL: REMOVE CONCRETE BY MEANS OF APPROVED PNEUMATIC HAMMERS EMPLOYING POINTED AND BLUNT CHISEL TOOLS. HYDRAULIC HOE-RAM TYPE HAMMERS WILL NOT BE PERMITTED. THE WEIGHT OF THE HAMMER SHALL NOT BE MORE THAN 35 POUNDS FOR REMOVAL WITHIN 18 INCHES OF PORTIONS TO BE PRESERVED. OUTSIDE THE 18 INCH LIMIT, THE CONTRACTOR MAY USE HAMMERS NOT EXCEEDING 90 POUNDS UPON THE APPROVAL OF THE ENGINEER. DO NOT PLACE PNEUMATIC HAMMERS IN DIRECT CONTACT WITH REINFORCING STEEL THAT IS TO BE RETAINED IN THE REBUILT STRUCTURE.

WEEP HOLES ARE TO BE THOROUGHLY CLEANED BY VACUUM-VETTING, PAYMENT IS INCLUDED WITH ITEM-202 - PORTIONS OF STRUCTURES REMOVED, OVER 20 FOOT SPAN, AS PER PLAN.

CUT LINE CONSTRUCTION JOINT PREPARATION: SAW CUT BOUNDARIES OF PROPOSED CONCRETE REMOVALS 1 INCH DEEP. REMOVE CONCRETE TO A ROUGH SURFACE. LEAVE THE EXISTING REINFORCING STEEL, IF REQUIRED IN THE PLANS, IN PLACE. INSTALL DOWEL BARS IF SPECIFIED. PRIOR TO CONCRETE PLACEMENT ABRASIVELY CLEAN JOINT SURFACES AND EXISTING EXPOSED REINFORCEMENT TO REMOVE LOOSE AND DISINTEGRATED CONCRETE AND LOOSE RUST. THOROUGHLY CLEAN THE JOINT SURFACE AND EXPOSED REINFORCEMENT OF ALL DIRT, DUST, RUST OR OTHER FOREIGN MATERIAL BY THE USE OF WATER, AIR UNDER PRESSURE, OR OTHER METHODS THAT PRODUCE SATISFACTORY RESULTS. EXISTING REINFORCING STEEL DOES NOT HAVE TO HAVE A BRIGHT STEEL FINISH, BUT REMOVE ALL PACK AND LOOSE RUST. THOROUGHLY DRENCH EXISTING CONCRETE SURFACES WITH CLEAN WATER AND ALLOW TO DRY TO A DAMP CONDITION BEFORE PLACING CONCRETE.

UNDERPASS LIGHTING: THE EXISTING UNDERPASS LIGHTING WILL BE REMOVED. THERE IS AN ELECTRIC LINE ATTACHED TO PIER 1 THAT IS USED FOR THIS LIGHTING. THE ELECTRIC SHALL BE DISCONNECTED AND THE ELECTRIC LINE CAPPED BELOW GRADE AND OUT OF THE PROPOSED PIER WORK LIMITS, OR REMOVED ENTIRELY. THE CONTRACTOR SHALL COORDINATE WITH ODOT DISTRICT 4 TO DISCONNECT THE ELECTRIC. PAYMENT FOR DISCONNECTING AND CAPPING THE ELECTRIC SHALL BE INCLUDED WITH ITEM 202-PORTIONS OF STRUCTURES REMOVED, OVER 20 FOOT SPAN, AS PER PLAN.

EXISTING STRUCTURE VERIFICATION:
DETAILS AND DIMENSIONS SHOWN ON THESE PLANS PERTAINING TO THE EXISTING STRUCTURE HAVE BEEN OBTAINED FROM PLANS OF THE EXISTING STRUCTURE AND FROM FIELD OBSERVATIONS AND MEASUREMENTS. CONSEQUENTLY, THEY ARE INDICATIVE OF THE EXISTING STRUCTURE AND THE PROPOSED WORK BUT THEY SHALL BE CONSIDERED TENTATIVE AND APPROXIMATE. THE CONTRACTOR IS REFERRED TO CMS SECTIONS 102.05 AND 105.02.

BASE CONTRACT BID PRICES UPON A RECOGNITION OF THE UNCERTAINTIES DESCRIBED ABOVE AND UPON A PREBID EXAMINATION OF THE EXISTING STRUCTURE. HOWEVER, THE DEPARTMENT WILL PAY FOR ALL PROJECT WORK BASED UPON ACTUAL DETAILS AND DIMENSIONS THAT HAVE BEEN VERIFIED IN THE FIELD.

DECK PLACEMENT DESIGN ASSUMPTIONS:
THE FOLLOWING ASSUMPTIONS OF CONSTRUCTION MEANS AND METHODS WERE MADE FOR THE ANALYSIS AND DESIGN OF THE SUPERSTRUCTURE. THE CONTRACTOR IS RESPONSIBLE FOR THE DESIGN OF THE FALSEWORK SUPPORT SYSTEM WITHIN THESE PARAMETERS AND WILL ASSUME RESPONSIBILITY FOR SUPERSTRUCTURE ANALYSIS FOR DEVIATION FROM THESE DESIGN ASSUMPTIONS.

AN EIGHT WHEEL FINISHING MACHINE WITH A MAXIMUM WHEEL LOAD OF 1.32 KIPS FOR A TOTAL MACHINE LOAD OF 10.6 KIPS.

A MINIMUM OUT-TO-OUT WHEEL SPACING AT EACH END OF THE MACHINE OF 103".

A MAXIMUM SPACING OF OVERHANG FALSEWORK BRACKETS OF 48".

A MAXIMUM DISTANCE FROM THE CENTERLINE OF THE FASCIA GIRDER TO THE FACE OF THE SAFETY HANDRAIL OF 65".

DECK POUR SEQUENCE
THE BRIDGE DECK SHALL BE POURED BY STARTING FROM THE FORWARD ABUTMENT AND CONTINUING TOWARDS THE REAR ABUTMENT. POURING THE DECK IN THIS MANNER WILL MINIMIZE POTENTIAL UPLIFT FORCES DURING POURING.

FOUNDATION BEARING PRESSURE
PIER FOOTINGS, AS DESIGNED, PRODUCE A MAXIMUM BEARING PRESSURE OF 2.5 TONS PER SQUARE FOOT. THE ALLOWABLE BEARING PRESSURE IS 3.5 TONS PER SQUARE FOOT.

UTILITIES
THE UTILITIES SHALL BEAR ALL EXPENSE INVOLVED IN RELOCATING AND/OR INSTALLING THE AFFECTED UTILITY LINES. THE CONTRACTOR AND UTILITIES ARE TO COOPERATE BY ARRANGING THEIR WORK IN SUCH A MANNER THAT INCONVENIENCE TO EITHER WILL BE HELD TO A MINIMUM.

DOMNINION EAST OHIO PIPELINE(S): IT IS THE CONTRACTOR'S RESPONSIBILITY TO MAINTAIN THE LATERAL AND SUBADJACENT SUPPORT OF DOMINION'S PIPELINE(S), IN COMPLIANCE TO 29 CFR, PART 1926, SUBPART P (SAFE EXCAVATION AND SHORING). ONE-FOOT MINIMUM VERTICAL AND HORZONTAL CLEARANCE MUST BE MAINTAINED BETWEEN DOMINION EAST OHIO'S (DEO) EXISTING PIPELINE(S) AND ALL OTHER IMPROVEMENTS. EXTREME CARE SHOULD BE TAKEN NOT TO HARM ANY DEO FACILITY (PIPELINES, ETC) OR APPURTENANCE (PIPE COATING, TRACER WIRE, CATHODIC PROJECTION TEST STATION WIRES & DEVICES, VALVE BOXES, ETC.). DEO FACILITIES MUST BE PROTECTED WITH A TARP DURING BRIDGE CONSTRUCTION. THE CONTRACTOR WILL BE RESPONSIBLE AND LIABLE FOR ENSURING THAT ALL DEO EXISTING FACILITIES, ABOVE AND BELOW GROUND, REMAIN UNDAMAGED, ACCESSIBLE AND IN WORKING ORDER. THE CROSSING OF DEO'S PIPELINE WITH ANOTHER STEEL FACILITY MAY CREATE A POTENTIAL CORROSION ISSSUE FOR THE PROPOSED FACILITY AND THE EXISTING DEO FACILITY. PLEASE CONTACT DOMINION'S CORROSION DEPARTMENT: DAVE CUTLIP (330-266-2121), RICK MCDONALD (330-266-2122).

GRADING AT PROPOSED PIERS
SOIL SURROUNDING PIERS THAT IS DISTURBED BY NEW PIER CONSTRUCTION SHALL BE REGRADED SUCH THAT PROPOSED GRADING AT ALL PIER LOCATIONS MATCHES EXISTING CONDITIONS, INCLUDING ANY DRAINAGE DITCHES. PAYMENT FOR GRADING AT PIERS SHALL BE INCLUDED WITH ITEM 503 - UNCLASSIFIED EXCAVATION, AS PER PLAN.
SEEDING AND MULCHING QUANTITIES ARE INCLUDED IN ROADWAY PLANS FOR WORK ALONG I-680.

ITEM 511 - CLASS QC1 CONCRETE WITH QC/QA, FOOTING, AS PER PLAN
IN ADDITION TO THE REQUIREMENTS OF ITEM 511 - CLASS QC1 CONCRETE WITH QC/QA, FOOTING, AS PER PLAN, INSTALL A REFERENCE MONUMENT AT EACH SPREAD FOOTING. THE REFERENCE MONUMENT SHALL CONSIST OF A #8 OR LARGER EPOXY COATED REBAR EMBEDDED AT LEAST 6" INTO THE FOOTING AND EXTENDED VERTICALLY 4 TO 6 INCHES ABOVE THE TOP OF THE FOOTING. INSTALL A SIX INCH DIAMETER, SCHEDULE 40, PLASTIC PIPE AROUND THE REFERENCE MONUMENT. CENTER THE PIPE ON THE REFERENCE MONUMENT AND PLACE THE PIPE VERTICAL WITH ITS TOP AT THE FINISHED GRADE. THE PIPE SHALL HAVE A REMOVABLE, SCHEDULE 40, PLASTIC CAP. PERMANENTLY ATTACH THE BOTTOM OF THE PIPE TO THE TOP OF THE FOOTING.

ESTABLISH A BENCHMARK TO DETERMINE THE ELEVATIONS OF THE REFERENCE MONUMENTS AT VARIOUS MONITORING PERIODS THROUGHOUT THE LENGTH OF THE CONSTRUCTION PROJECT. THE BENCHMARK SHALL BE THE SAME THROUGHOUT THE PROJECT AND SHALL BE INDEPENDENT OF ALL STRUCTURES.

RECORD THE ELEVATION OF EACH REFERENCE MONUMENT AT EACH MONITORING PERIOD SHOWN IN THE TABLE BELOW.

THE ORIGINAL COMPLETED TABLES WILL BECOME PART OF THE DISTRICT'S PROJECT PLAN RECORDS. SEND A COPY OF THE COMPLETED TABLES TO THE OFFICE OF STRUCTURAL ENGINEERING.

PROJECT NUMBER: 82941		
MAXIMUM FACTORED BEARING PRESSURE: 2.5 TONS PER SQUARE FOOT		
BRIDGE NUMBER: MAH-680-0282		
STRUCTURE FILE NUMBER: 5006511		
BENCHMARK LOCATION:		
FOOTING LOCATION: PIER 1, PIER 2, AND PIER 3.		
MONITORING PERIOD	LEFT MONUMENT	RIGHT MONUMENT
AFTER FOOTING CONCRETE IS PLACED		
BEFORE PLACEMENT OF SUPERSTRUCTURE MEMBERS		
BEFORE DECK PLACEMENT		
AFTER DECK PLACEMENT		
PROJECT COMPLETION		

ITEM 519 - PATCHING CONCRETE STRUCTURES, AS PER PLAN
PRIOR TO THE SURFACE CLEANING SPECIFIED IN 519.04 AND WITHIN 24 HOURS OF PLACING PATCHING MATERIAL, BLAST CLEAN ALL SURFACES TO BE PATCHED INCLUDING THE EXPOSED REINFORCING STEEL. ACCEPTABLE METHODS INCLUDE HIGH-PRESSURE WATER BLASTING WITH OR WITHOUT ABRASIVES IN THE WATER, ABRASIVE BLASTING WITH CONTAINMENT, OR VACUUM ABRASIVE BLASTING.

ASBESTOS NOTIFICATION
AN ASBESTOS SURVEY OF THE VESTAL ROAD BRIDGE (MAH-680-2.83; SFN 5006511) OVER INTERSTATE ROUTE 680 SCHEDULED FOR REHABILITATION WAS CONDUCTED BY A CERTIFIED ASBESTOS HAZARD EVALUATION SPECIALIST. THE SURVEY DETERMINED THAT NO ASBESTOS IS PRESENT ON THE BRIDGE STRUCTURE.

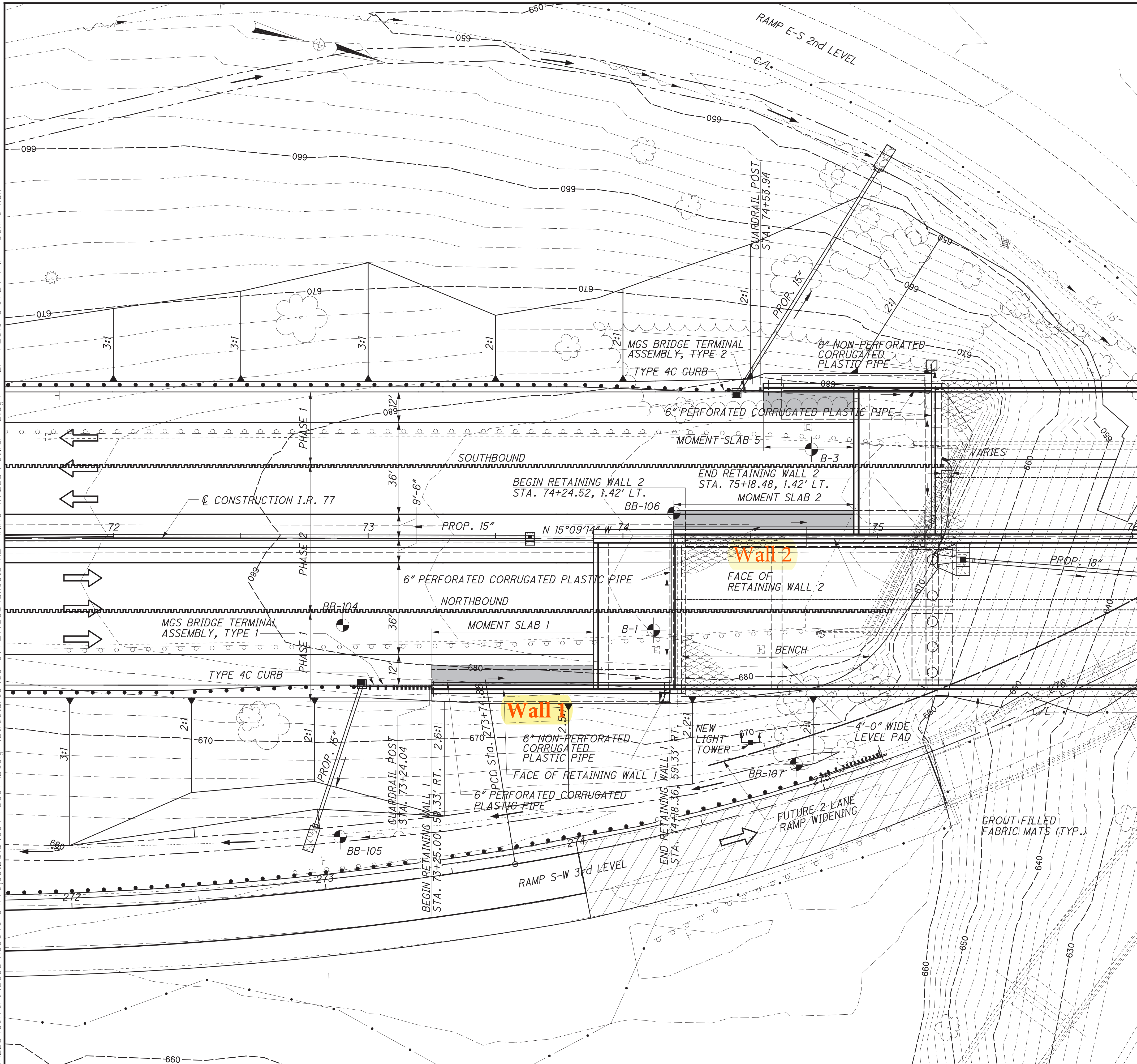
A COPY OF THE OHIO ENVIRONMENTAL PROTECTION AGENCY (OEPA) NOTIFICATION OF DEMOLITION AND RENOVATION FORM, PARTIALLY COMPLETED BY THE BRIDGE OWNER, WILL BE PROVIDED TO THE SUCCESSFUL BIDDER. THE CONTRACTOR SHALL COMPLETE AND SIGN THE FORM AND SUBMIT IT TO:

MAHONING-TRUMBULL AIR POLLUTION CONTROL
345 OAK HILL AVE., SUITE 200
YOUNGSTOWN, OH 44502
TARA CIOFFI
(330) 743-3333
FAX: (330) 743-3960

AT LEAST TEN (10) WORKING DAYS PRIOR TO THE START OF ANY DEMOLITION AND/OR REHABILITATION. THE CONTRACTOR SHALL PROVIDE A COPY OF THE COMPLETED AND SIGNED FORM TO THE ENGINEER. INFORMATION REQUIRED ON THE FORM WILL INCLUDE: 1) THE CONTRACTOR'S NAME AND ADDRESS, 2) THE SCHEDULED DATES FOR THE START AND COMPLETION OF BRIDGE REMOVAL, AND 3) A DESCRIPTION OF THE PLANNED DEMOLITION WORK AND THE METHOD(S) TO BE USED. THE OEPA FORM IS AVAILABLE FOR INSPECTION AT THE ODOT DISTRICT 4 OFFICE, 2088 S. ARLINGTON RD, AKRON, OH 44306.

BASIS FOR PAYMENT - THE CONTRACTOR SHALL FURNISH ALL FEES, LABOR, AND MATERIAL NECESSARY TO COMPLETE AND SUBMIT THE OEPA NOTIFICATION FORM. PAYMENT FOR THIS WORK SHALL BE INCLUDED IN ITEM 202 - STRUCTURE REMOVED, OVER 20 FOOT SPAN, AS PER PLAN.

FINAL COAT OF PAINT
STRUCTURAL STEEL FINISH COAT COLOR FOR THE BRIDGE SHALL MATCH FEDERAL STANDARD FS-595B COLOR NO. 15526 (LIGHT BLUE).



BENCH MARK 2 (SV 2003) SURVEY MARKER FOUND WITH 2" ALUMINUM CAP STAMPED "BARR & PREVOST" STA. 74+62.90, 281.76' LT. EL. 648.40

LEGEND

- TEMPORARY SHEETING
- MOMENT SLAB
- PLANING AND RESURFACING
- ITEM 601 SLOPE PROTECTION, MISC.: GROUT FILLED FABRIC MATS

NOTES

RETAINING WALL 1: SEE SHEET 4/22.

RETAINING WALL 2: SEE SHEET 5/22.

MOMENT SLAB 1: SEE SHEET 11/22.

MOMENT SLAB 2: SEE SHEET 12/22.

MOMENT SLAB 5: SEE SHEET 15/22.

EARTHWORK LIMITS SHOWN ARE APPROXIMATE. ACTUAL SLOPES SHALL CONFORM TO PLAN CROSS SECTIONS.

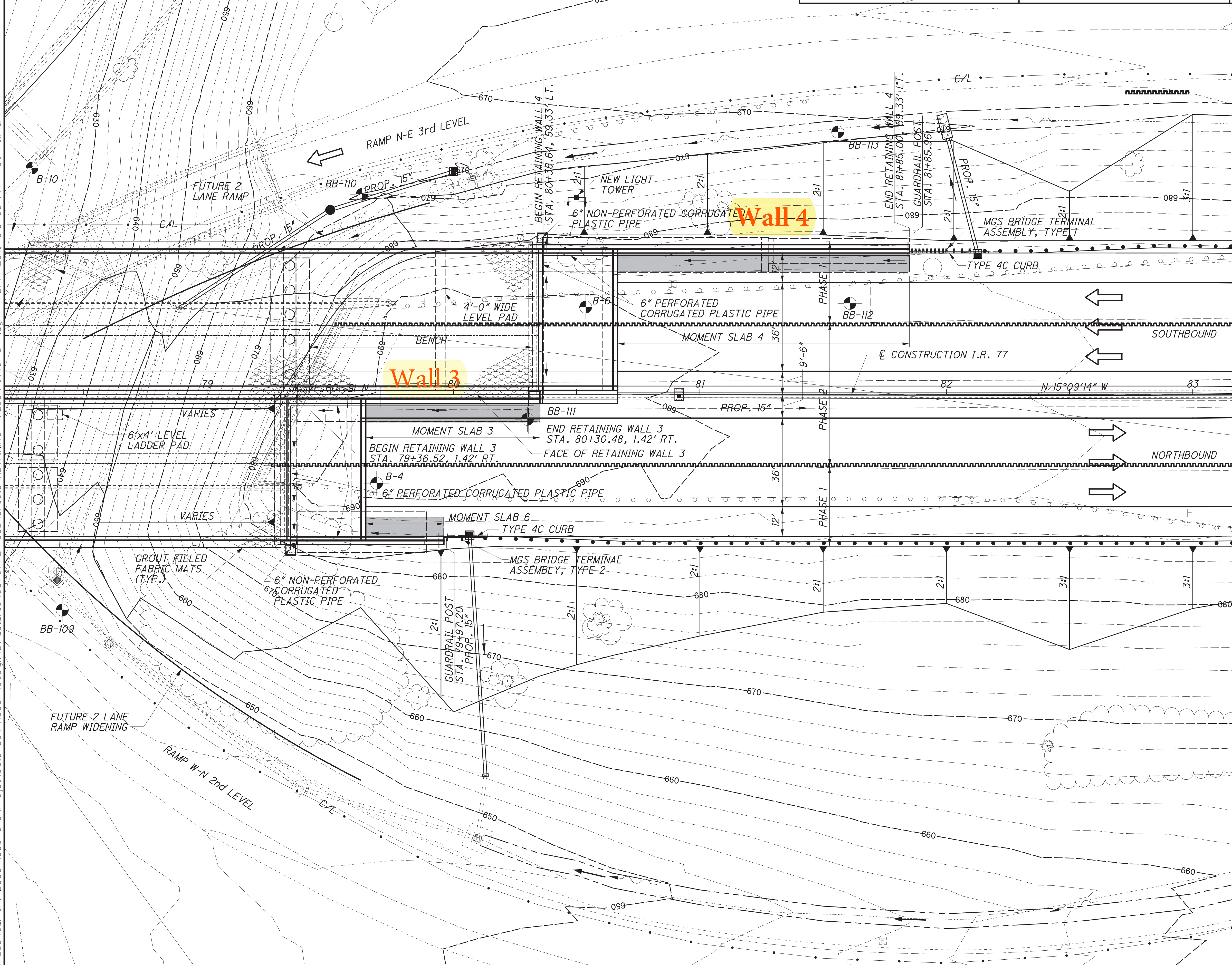
RICHLAND ENGINEERING LIMITED
29 NORTH PARK STREET
MANSFIELD, OHIO 44902

DESIGNED BLN | CHECKED DLR | DRAWN JLS | REVIEWED DAP | DATE 11/15/16

RETAINING WALLS 1 AND 2 SITE PLAN

CUY-77-14.35
PID No. 13567

BENCH MARK 3 (132) 5/8" REBAR FOUND STA. 79+51.45, 151.16' LT. EL. 668.73	BENCH MARK 4 TOP WEST BOLT ODOT LIGHT POLE #BOW28 STA. 80+12, 288' RT. EL. 649.75	BENCH MARK 5 (SV 2000) SURVEY MARKER SET WITH CAP STAMPED "RICHLAND ENG. TRAVERSE PT." STA. 82+88.39, 56.45' LT. EL. 687.04



LEGEND

- TEMPORARY SHEETING
- MOMENT SLAB
- ITEM 601 SLOPE PROTECTION, MISC.: GROUT FILLED FABRIC MATS

NOTES

RETAINING WALL 3: SEE SHEET 6/22.

RETAINING WALL 4: SEE SHEETS 7/22 AND 8/22.

MOMENT SLAB 3: SEE SHEET 12/22.

MOMENT SLAB 4: SEE SHEET 13/22.

MOMENT SLAB 6: SEE SHEET 16/22.

ADDITIONAL NOTES: SEE SHEET 1/22.

RICHLAND ENGINEERING LIMITED
29 NORTH PARK STREET
MANSFIELD, OHIO 44902

DATE 11/15/16 | REVIEWED DAP | DRAWN JLS | REVISED | DESIGNED BLN | CHECKED DLR

RETAINING WALLS 3 AND 4 SITE PLAN

CUY-77-14.35
PID No. 13567

ESTIMATED QUANTITIES (FUNDING PARTICIPATION)

CALCULATED JLS DATED 1/16
CHECKED JSD DATED 1/16

FUNDING PARTICIPATION 01/BRO/BR	FUNDING PARTICIPATION 02/BRO/BR	ITEM	ITEM EXT.	TOTAL	UNIT	DESCRIPTION	RETAINING WALL 1	RETAINING WALL 2	RETAINING WALL 3	RETAINING WALL 4	MOMENT SLAB WITH SINGLE SLOPE RAILING OR MEDIAN BARRIER 1	2	3	4	5	6	SEE SHEET
LS	LS	503	11100	LS		COFFERDAMS AND EXCAVATION BRACING											
LS	LS	503	21301	LS		UNCLASSIFIED EXCAVATION, AS PER PLAN											3/22
85,432	42,079	509	10000	127,511	POUND	EPOXY COATED REINFORCING STEEL	22,032	31,243	30,872	43,364							
561	276	511	46213	837	CY	CLASS QC1 CONCRETE WITH QC/QA, RETAINING/WINGWALL INCLUDING FOOTING, AS PER PLAN	176	186	187	288							3-9/22
167	82	511	53010	249	CY	CLASS QC1 CONCRETE, MISC.: MOMENT SLAB INCLUDING RAILING AND MEDIAN BARRIER					38	49	49	72	22	19	3/22
673	332	512	10100	1005	SY	SEALING OF CONCRETE SURFACES (EPOXY-URETHANE)	108	158	163	190	56	83	83	105	31	28	
38	19	512	33000	57	SY	TYPE 2 WATERPROOFING	10	15	15	17							
615	303	516	13900	918	SF	2" PREFORMED EXPANSION JOINT FILLER	174	177	177	284	17	19	19	17	17	17	
279	138	518	21200	417	CY	POROUS BACKFILL WITH GEOTEXTILE FABRIC	76	105	107	129							
305	150	518	40000	455	FT	6" PERFORATED CORRUGATED PLASTIC PIPE	96	105	103	151							
15	7	518	40010	22	FT	6" NON-PERFORATED CORRUGATED PLASTIC PIPE, INCLUDING SPECIALS	11			11							

Participation Codes:
01/BRO/BR - Existing Lanes (90/10)-67%
02/BRO/BR - Added Capacity (80/20)-33%

GENERAL NOTES

REFER TO THE FOLLOWING STANDARD DRAWING:
SBR-1-13 (REV 01-17-2014)
SBR-2-13 (REV 01-17-2014)

DESIGN SPECIFICATIONS: THIS STRUCTURE CONFORMS TO THE LRFD BRIDGE DESIGN SPECIFICATIONS ADOPTED BY THE AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS, 2012, SIXTH EDITION, INCLUDING INTERIM SPECIFICATIONS AND THE ODOT BRIDGE DESIGN MANUAL, 2007.

DESIGN DATA:

OPERATIONAL IMPORTANCE: A LOAD MODIFIER OF 1.05 HAS BEEN ASSUMED FOR THE DESIGN OF THE RETAINING WALLS IN ACCORDANCE WITH THE AASHTO LRFD BRIDGE DESIGN SPECIFICATION, ARTICLE 1.3.5, AND ODOT BRIDGE DESIGN MANUAL, 2007.

CONCRETE CLASS QC1 - COMPRESSIVE STRENGTH 4.0 KSI (WALLS AND FOOTINGS)

REINFORCING STEEL - MINIMUM YIELD STRENGTH 60 KSI

FOUNDATION BEARING PRESSURE:

WALL NUMBER	MAXIMUM SERVICE LOAD PRESSURE	MAXIMUM STRENGTH LOAD PRESSURE	FACTORED BEARING RESISTANCE SERVICE LIMIT	FACTORED BEARING RESISTANCE STRENGTH LIMIT
1	2.5 KSF	3.5 KSF	6.0 KSF	26.2 KSF
2	2.9 KSF	4.0 KSF	6.0 KSF	22.6 KSF
3	2.9 KSF	4.0 KSF	6.0 KSF	24.8 KSF
4	2.9 KSF	4.0 KSF	6.0 KSF	21.2 KSF

MOMENT SLAB AND RAILING OR MEDIAN BARRIER: ITEM 511 CLASS QC1 CONCRETE, MISC.: MOMENT SLAB INCLUDING RAILING AND MEDIAN BARRIER SHALL INCLUDE ALL MATERIALS INCLUDING EXCAVATION, CONCRETE, EPOXY REINFORCING STEEL AND THAT ARE REQUIRED FOR THE RAILING OR MEDIAN BARRIER DEFLECTION JOINTS PER PLAN DETAILS.

ITEM 503 - UNCLASSIFIED EXCAVATION, AS PER PLAN
UNCLASSIFIED EXCAVATION SHALL BE IN ACCORDANCE WITH CMS ITEM 503 EXCEPT THAT THE BACKFILL MATERIAL UNDER THE APPROACH SLABS AND MOMENT SLABS SHALL BE MATERIAL CONFORMING TO CMS 703.17 (CMS 304 MATERIAL) AND MEET THE COMPACTION REQUIREMENTS OF CMS 304.05. IN ADDITION, THE BACKFILL MATERIAL SHALL BE PLACED AND COMPACTED IN 6" LIFTS. EXCAVATION OF THE EXISTING POROUS BACKFILL SHALL BE INCLUDED IN THIS ITEM.

ITEM 511 - CLASS QC1 CONCRETE WITH QC/QA, AS PER PLAN, RETAINING WALL AND FOOTING
IN ADDITION TO THE REQUIREMENTS OF 511, INSTALL A REFERENCE MONUMENT IN THE RETAINING WALL FOOTING AT THE LOCATIONS SHOWN IN THE TABLE BELOW. THE REFERENCE MONUMENT SHALL CONSIST OF A #8, OR LARGER, EPOXY COATED REBAR EMBEDDED AT LEAST 6" INTO THE FOOTING AND EXTENDED VERTICALLY 4 TO 6 INCHES ABOVE THE TOP OF THE FOOTING. INSTALL A SIX INCH DIAMETER, SCHEDULE 40, PLASTIC PIPE AROUND THE REFERENCE MONUMENT. CENTER THE PIPE ON THE REFERENCE MONUMENT AND PLACE THE PIPE VERTICAL WITH ITS TOP AT THE FINISHED GRADE. THE PIPE SHALL HAVE A REMOVABLE, SCHEDULE 40, PLASTIC CAP. PERMANENTLY ATTACH THE BOTTOM OF THE PIPE TO THE TOP OF THE FOOTING.

ESTABLISH A BENCH MARK TO DETERMINE THE ELEVATIONS OF THE REFERENCE MONUMENTS AT VARIOUS MONITORING PERIODS THROUGHOUT THE LENGTH OF THE CONSTRUCTION PROJECT. THE BENCHMARK SHALL BE THE SAME THROUGHOUT THE PROJECT AND SHALL BE INDEPENDENT OF ALL STRUCTURES.

RECORD THE ELEVATION OF EACH REFERENCE MONUMENT AT EACH MONITORING PERIOD SHOWN IN THE TABLE BELOW.

THE ORIGINAL COMPLETED TABLES WILL BECOME PART OF THE DISTRICT'S PROJECT PLAN RECORDS.

INDEX OF SHEETS

	PROJECT NUMBER: CUY-77-14.35, PID: 13567	MAXIMUM FACTORED BEARING PRESSURE: 3.5 KIPS PER SQ. FT.	PROJECT NUMBER: CUY-77-14.35, PID: 13567	MAXIMUM FACTORED BEARING PRESSURE: 4.0 KIPS PER SQ. FT.	PROJECT NUMBER: CUY-77-14.35, PID: 13567	MAXIMUM FACTORED BEARING PRESSURE: 4.0 KIPS PER SQ. FT.	PROJECT NUMBER: CUY-77-14.35, PID: 13567	MAXIMUM FACTORED BEARING PRESSURE: 4.0 KIPS PER SQ. FT.
	WALL NUMBER: 1		WALL NUMBER: 2		WALL NUMBER: 3		WALL NUMBER: 4	
	BENCHMARK LOCATION: LP 74+49.4-81.1 RT 669.49		BENCHMARK LOCATION:		BENCHMARK LOCATION:		BENCHMARK LOCATION: LP 80+51.3-84.5 LT 674.37	
MONITORING PERIOD:	MONUMENT #1 RETAINING WALL 1 STA. 73+25.50 OFFSET: 62' RT.	MONUMENT #2 RETAINING WALL 1 STA. 74+17.86 OFFSET: 62' RT.	MONUMENT #1 RETAINING WALL 2 STA. 74+25.02 OFFSET: 1'-0" RT.	MONUMENT #2 RETAINING WALL 2 STA. 75+17.98 OFFSET: 1'-0" RT.	MONUMENT #1 RETAINING WALL 3 STA. 79+37.02 OFFSET: 1'-0" LT.	MONUMENT #2 RETAINING WALL 3 STA. 80+29.98 OFFSET: 1'-0" LT.	MONUMENT #1 RETAINING WALL 4 STA. 80+37.14 OFFSET: 62' LT.	MONUMENT #2 RETAINING WALL 4 STA. 81+84.50 OFFSET: 62' LT.
AFTER FOOTING CONCRETE IS PLACED:	674.222	674.182					679.53	681.27
AFTER STEM CONCRETE IS PLACED:	674.222	674.182					679.53	681.27
PROJECT COMPLETION:	674.23	674.182					679.52	681.27

6" PIPE LEAN

6" PIPE LEAN 11.3°

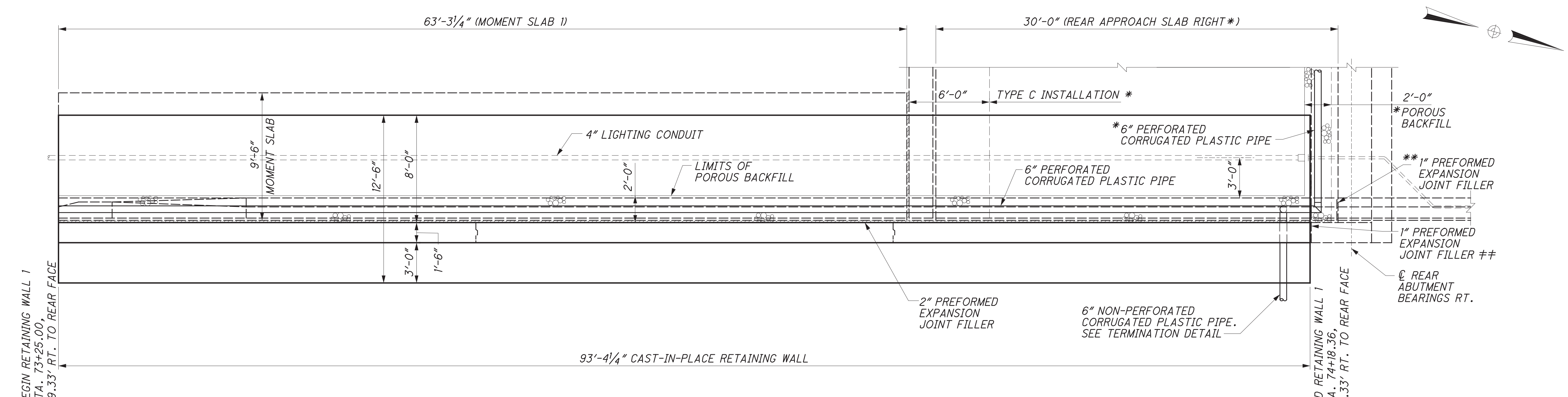
F:\2005\105040-5\ProjectData\CUY\13567\Design\Structures\CUY077_1435L\Sheets\RETAINING WALLS\13567ecb.dgn 5/16/2017 3:33:19 PM SteveMorr

RICHLAND ENGINEERING LIMITED 29 NORTH PARK STREET MANSFIELD, OHIO 44902

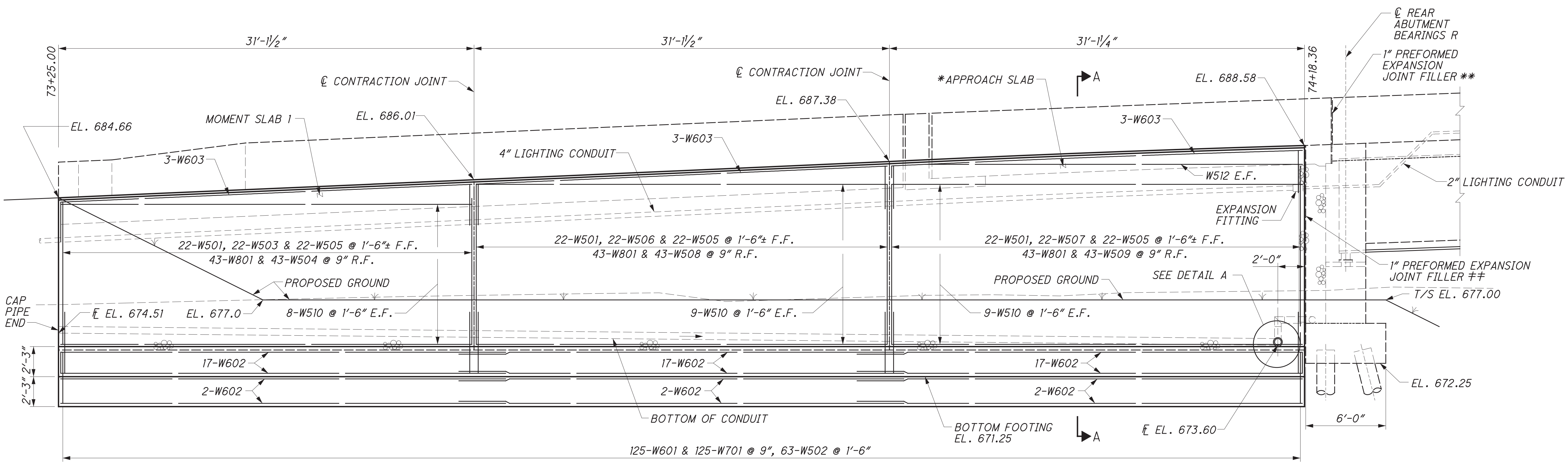
DESIGNED BLN CHECKED DLR DRAWN JLS REVISED REVIEWED DAP DATE 11/15/16

ESTIMATED QUANTITIES AND GENERAL NOTES

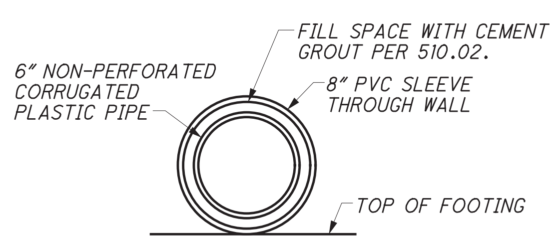
CUY-77-14.35 PID No. 13567



PLAN - RETAINING WALL 1



ELEVATION - RETAINING WALL 1



DETAIL A

LEGEND

* TO BE INCLUDED WITH STRUCTURE QUANTITIES FOR PAYMENT.

** TO BE INCLUDED WITH ITEM 526 - REINFORCED CONCRETE APPROACH SLABS (T=17"), AS PER PLAN FOR PAYMENT.

TO BE INCLUDED WITH ITEM 511 - CLASS QC1 CONCRETE WITH QC/QA, AS PER PLAN, RETAINING WALL AND FOOTING FOR PAYMENT.

NOTES

REINFORCING STEEL SPLICE LENGTHS SHALL BE 2'-1" FOR VERTICAL #5 BARS AND 3'-6" FOR HORIZONTAL #6 BARS.

PIPE TERMINATION DETAIL: SEE SHEET 10/22.

SECTION A-A: SEE SHEET 9/22.

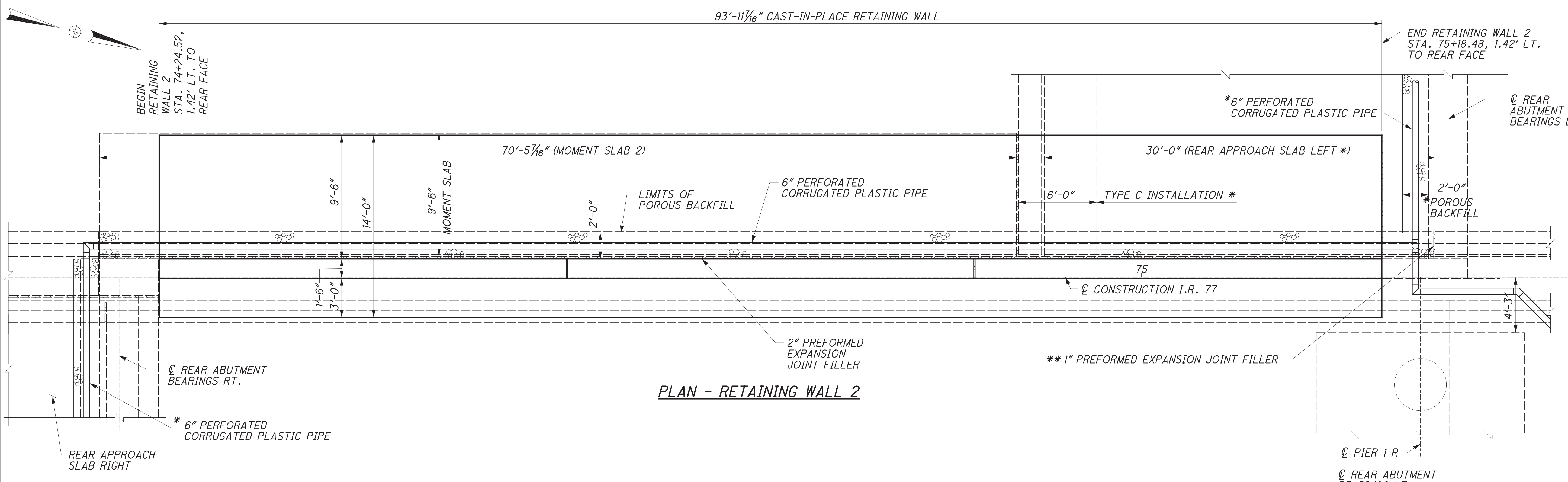
CONTRACTION JOINT DETAIL SEE SHEET 9/22.

RICHLAND ENGINEERING LIMITED
29 NORTH PARK STREET
MANSFIELD, OHIO 44902

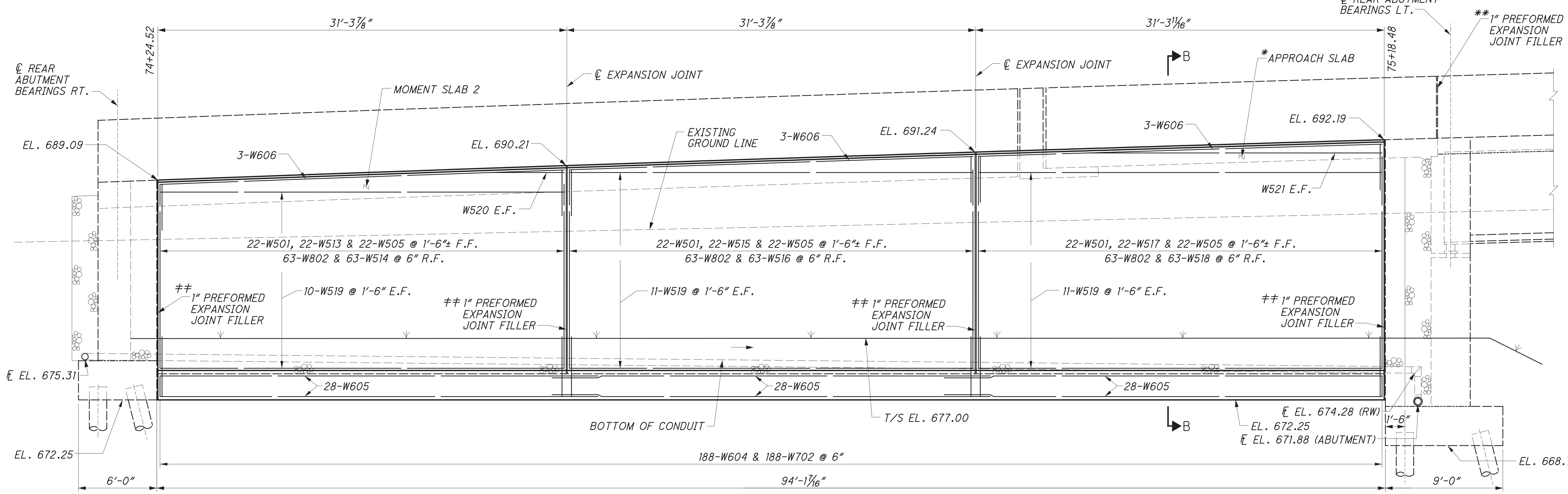
DESIGNED	DRAWN	REVIEWED	DATE
BLN	JLS	DAP	11/15/16
CHECKED	REVISED		
DLR			

RETAINING WALL 1

CUY-77-14.35
PID No. 13567



PLAN - RETAINING WALL 2



ELEVATION - RETAINING WALL 2

LEGEND

* TO BE INCLUDED WITH STRUCTURE QUANTITIES FOR PAYMENT.

** TO BE INCLUDED WITH ITEM 526 - REINFORCED CONCRETE APPROACH SLABS (T=17"), AS PER PLAN FOR PAYMENT.

‡‡ TO BE INCLUDED WITH ITEM 511 - CLASS QC1 CONCRETE WITH QC/QA, AS PER PLAN, RETAINING WALL AND FOOTING FOR PAYMENT.

NOTES

ADDITIONAL NOTES: SEE SHEET 4/22.

SECTION B-B: SEE SHEET 9/22.

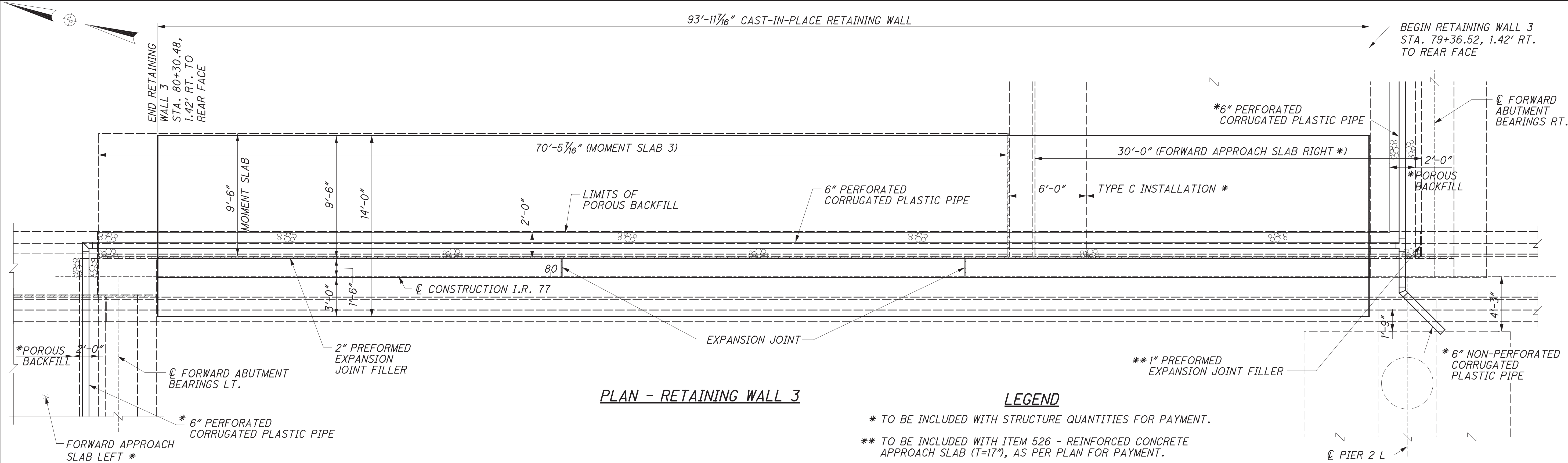
RICHLAND ENGINEERING LIMITED
29 NORTH PARK STREET
MANSFIELD, OHIO 44902

DESIGNED BLN	CHECKED DLR
DRAWN JLS	REVISED
REVIEWED DAP	DATE 11/15/16
STRUCTURE FILE NUMBER	

RETAINING WALL 2

CUY-77-14.35

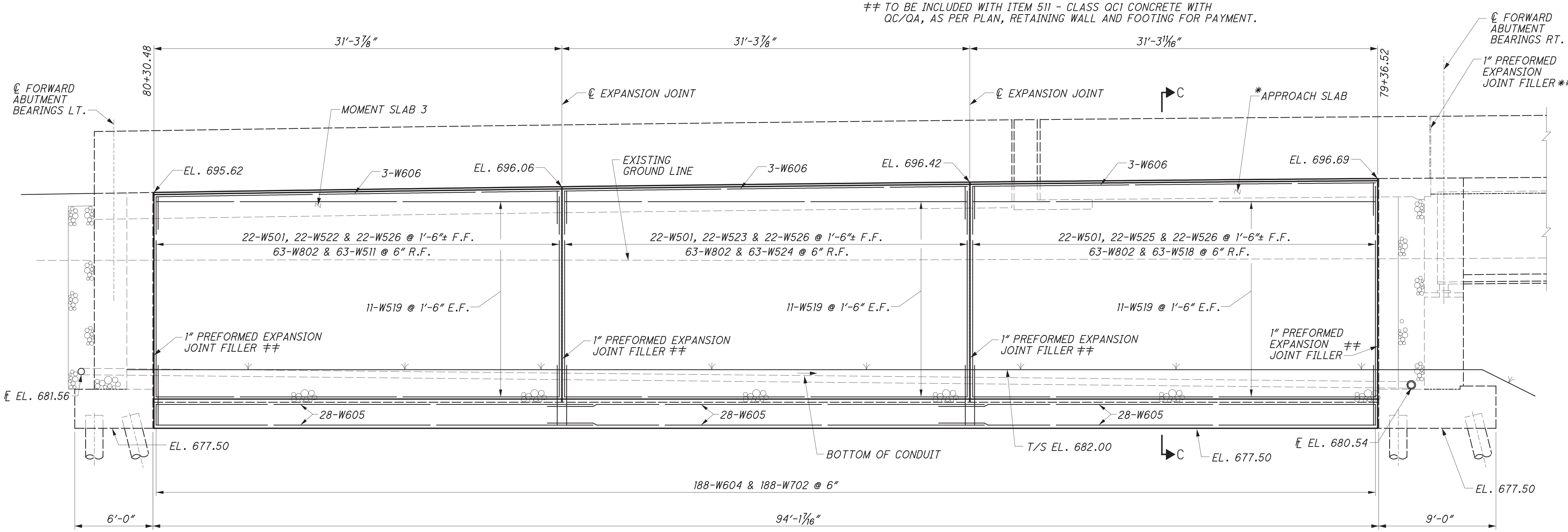
PID No. 13567



PLAN - RETAINING WALL 3

LEGEND

* * TO BE INCLUDED WITH STRUCTURE QUANTITIES FOR PAYMENT.
* ** TO BE INCLUDED WITH ITEM 526 - REINFORCED CONCRETE APPROACH SLAB (T=17"), AS PER PLAN FOR PAYMENT.
* ‡‡ TO BE INCLUDED WITH ITEM 511 - CLASS QC1 CONCRETE WITH QC/QA, AS PER PLAN, RETAINING WALL AND FOOTING FOR PAYMENT.



ELEVATION - RETAINING WALL 3

NOTES

ADDITIONAL NOTES: SEE SHEET 4/22.

SECTION C-C: SEE SHEET 9/22.

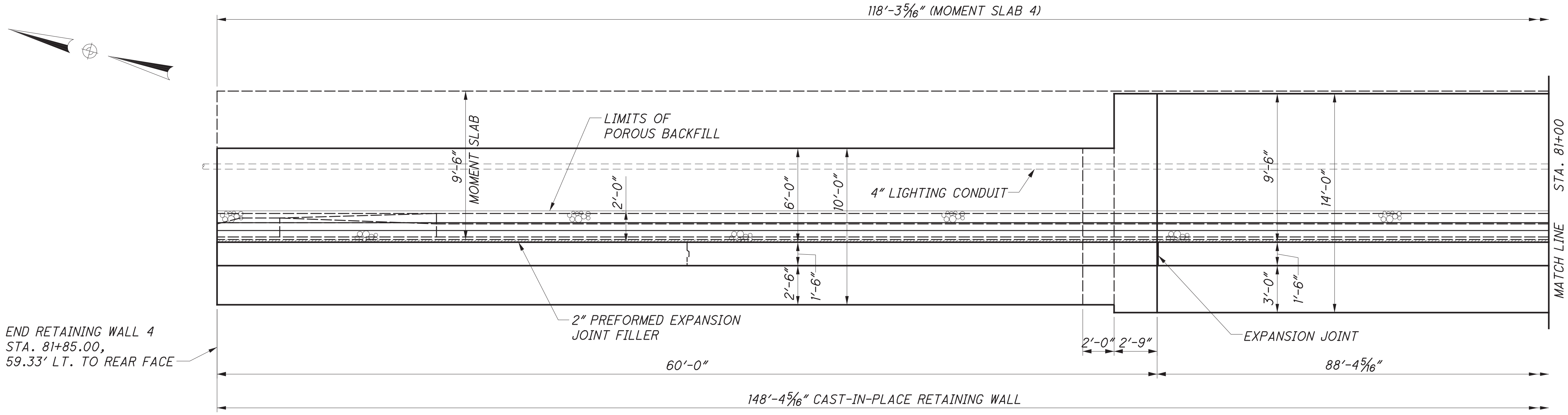
EXPANSION JOINT DETAIL: SEE SHEET 9/22.

RICHLAND ENGINEERING LIMITED
29 NORTH PARK STREET
MANSFIELD, OHIO 44902

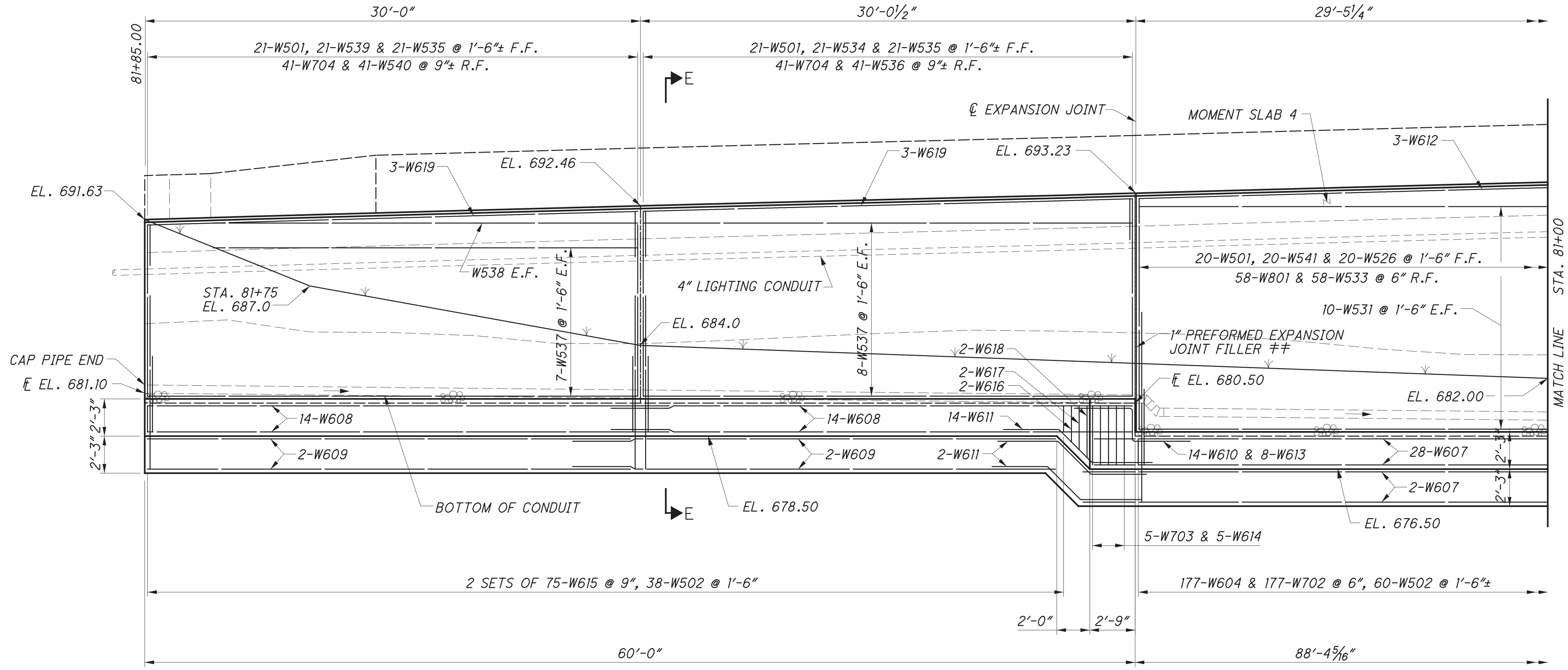
DATE	REVIEWED	DRAWN	REVISED	DESIGNED	CHECKED
11/15/16	DAP	JLS		BLN	DLR

RETAINING WALL 3

CUY-77-14.35
PID No. 13567



PLAN - RETAINING WALL 4



LEGEND

ELEVATION - RETAINING WALL 4

* TO BE INCLUDED WITH STRUCTURE QUANTITIES FOR PAYMENT.

** TO BE INCLUDED WITH ITEM 526 - REINFORCED CONCRETE APPROACH SLABS (T=17″), AS PER PLAN FOR PAYMENT.

‡‡ TO BE INCLUDED WITH ITEM 511 - CLASS QC1 CONCRETE WITH QC/QA, AS PER PLAN, RETAINING WALL AND FOOTING FOR PAYMENT.

NOTES

ADDITIONAL NOTES: SEE SHEET 4/22.

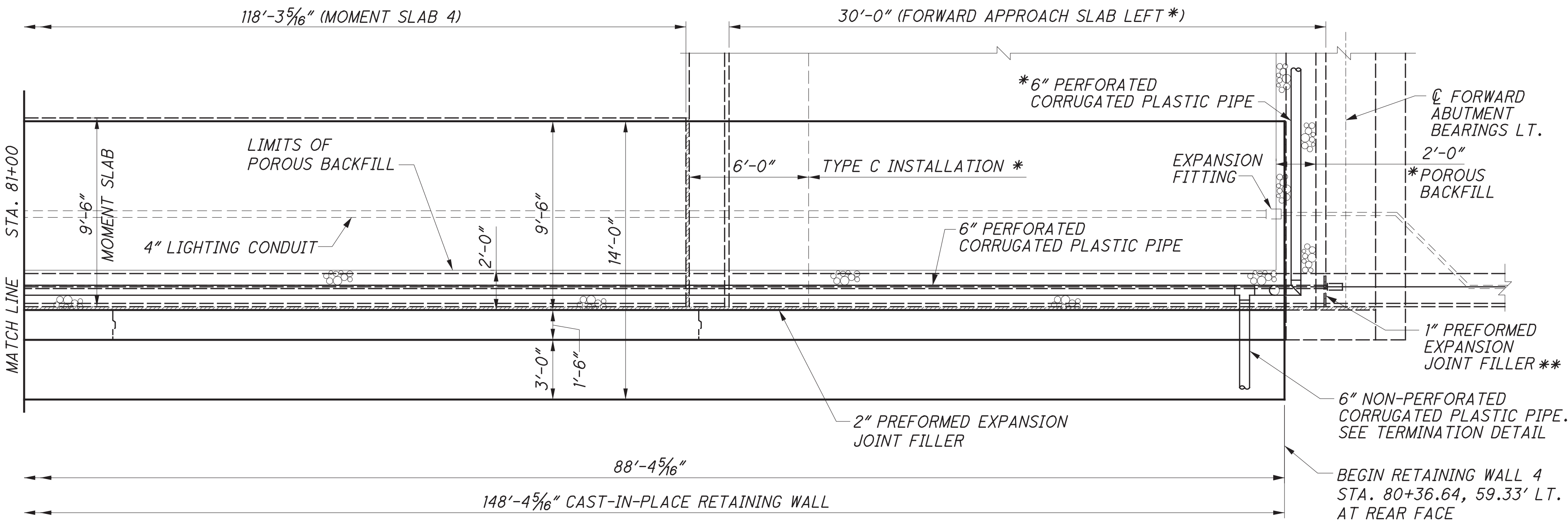
SECTION E-E: SEE SHEET 10/22.

RICHLAND ENGINEERING LIMITED
29 NORTH PARK STREET
MANSFIELD, OHIO 44902

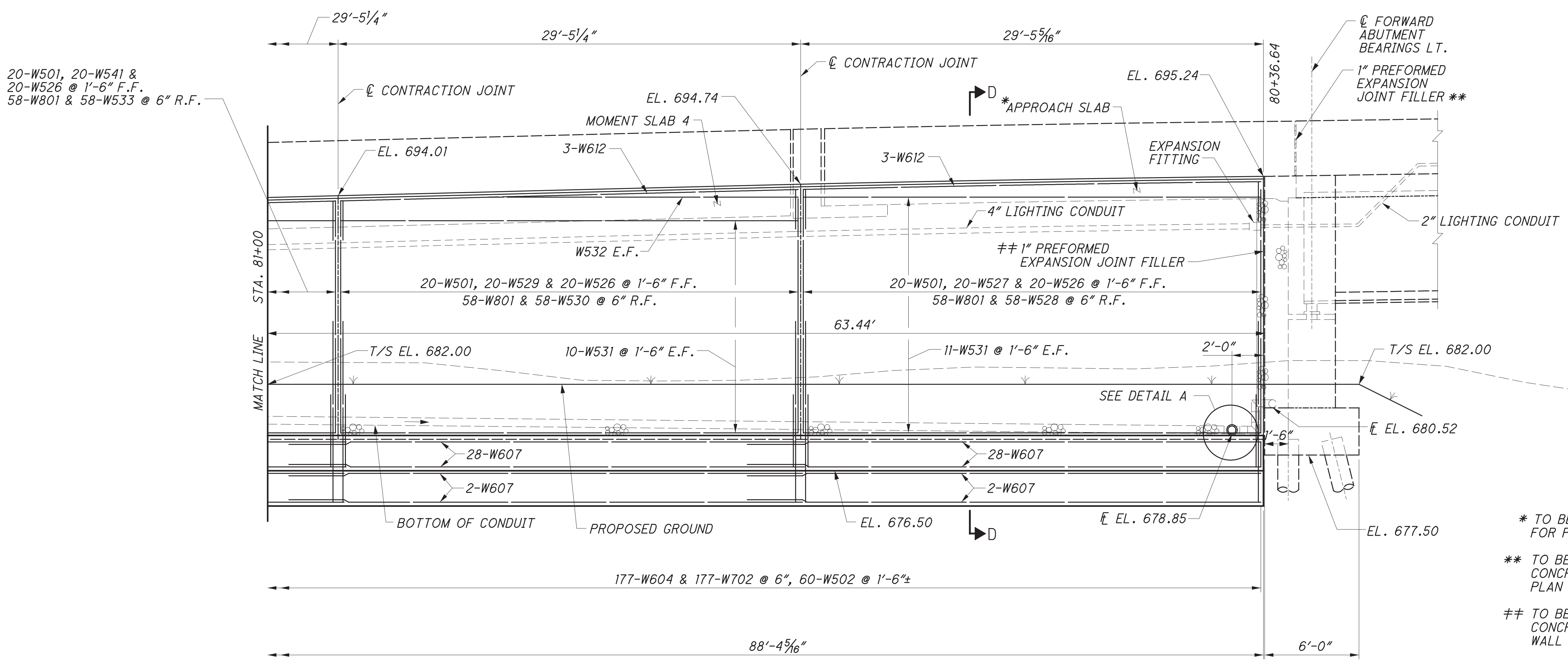
DESIGNED BLN	DRAWN JLS	REVIEWED DAP	DATE 11/15/16
CHECKED DLR	REVISED		

RETAINING WALL 4 - 1

CUY-77-14.35
PID No. 13567



PLAN - RETAINING WALL 4



ELEVATION - RETAINING WALL 4

LEGEND

* TO BE INCLUDED WITH STRUCTURE QUANTITIES FOR PAYMENT.

** TO BE INCLUDED WITH ITEM 526 - REINFORCED CONCRETE APPROACH SLABS (T=17″), AS PER PLAN FOR PAYMENT.

‡‡ TO BE INCLUDED WITH ITEM 511 - CLASS QC1 CONCRETE WITH QC/QA, AS PER PLAN, RETAINING WALL AND FOOTING FOR PAYMENT.

NOTES

ADDITIONAL NOTES: SEE SHEET 4/22.

SECTION D-D: SEE SHEET 10/22.

PIPE TERMINATION DETAIL: SEE SHEET 10/22.

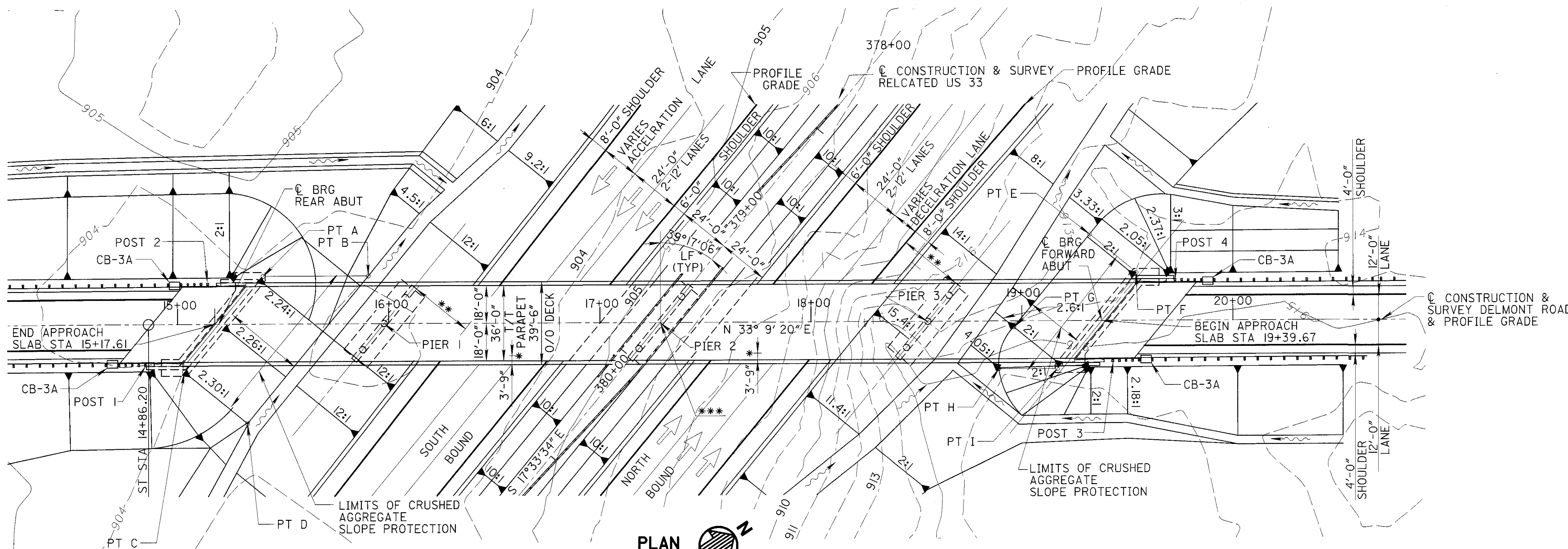
DETAIL A: SEE SHEET 4/22.

RICHLAND ENGINEERING LIMITED
29 NORTH PARK STREET
MANSFIELD, OHIO 44902

DESIGNED	DRAWN	REVIEWED	DATE
BLN	JLS	DAP	11/15/16
CHECKED	REVISED		
DLR			

RETAINING WALL 4 - 2

CUY-77-14.35
PID No. 13567



PLAN

NOTES:

1. EARTHWORK LIMITS SHOWN ARE APPROXIMATE ACTUAL SLOPES SHALL CONFORM TO PLAN CROSS SECTIONS.
2. BORING LOCATIONS (NOT SHOWN HERE) ARE LOCATED AT INTERSECTION OF SUB-STRUCTURE BEARINGS AND ℄ DELMONT ROAD

LEGEND:

* MINIMUM VERTICAL CLEARANCE
16'-9⅞" ACTUAL SOUTHBOUND
16'-6" REQUIRED
17'-5¼" ACTUAL NORTHBOUND
16'-6" REQUIRED

** MINIMUM HORIZONTAL CLEARANCE
30'-0¼" ACTUAL SOUTHBOUND
30'-0" REQUIRED
30'-4" ACTUAL NORTHBOUND
30'-0" REQUIRED

*** ℄ PIER 2, STA 17+28.64 DELMONT ROAD = STA 379+68.18 RELOCATED US 33

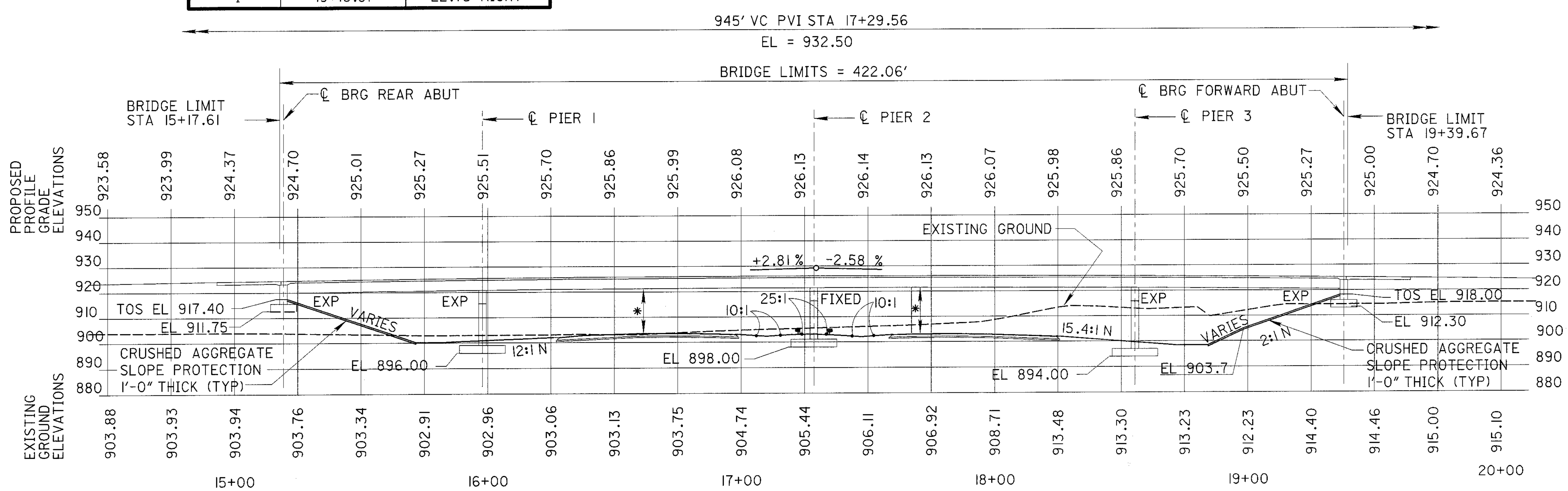
ABUT = ABUTMENT
ADT = AVERAGE DAILY TRAFFIC
ADTT = AVERAGE DAILY TRUCK TRAFFIC
BRG = BEARING
CB-3A = CATCH BASIN TYPE 3A
C/C = CENTER TO CENTER
EL = ELEVATION
EXP = EXPANSION JOINT
LF = LEFT FORWARD
LT = LEFT
N = NORMAL
O/O = OUT TO OUT
PI = POINT OF INTERSECTION
STA = STATION
ST = SPIRAL TANGENT
T/T = TOE TO TOE
TOS = TOP OF SLOPE
VC = VERTICAL CURVE
VPI = VERTICAL POINT OF INTERSECTION

SLOPE PROTECTION LIMITS		
POINT (PT)	STATION	OFFSET
A	15+40.48	22.75' LEFT
B	15+90.34	22.75' LEFT
C	15+03.19	22.75' RIGHT
D	15+33.15	47.24' RIGHT
E	19+32.04	41.07' LEFT
F	19+54.09	22.75' LEFT
G	18+96.09	2.78' RIGHT
H	18+89.78	22.75' RIGHT
I	19+16.81	22.75' RIGHT

SUBSTRUCTURE @ ℄ DELMONT ROAD	STATION
℄ BRG REAR ABUTMENT	15+19.39
℄ PIER 1	15+97.81
℄ PIER 2	17+28.64
℄ PIER 3	18+55.52
℄ BRG FORWARD ABUTMENT	19+37.89

POST NUMBER	FIRST POST GUARD RAIL	STATION #
POST 1	RIGHT REAR ABUT	14+84.74
POST 2	LEFT REAR ABUT	15+14.19
POST 3	RIGHT FORWARD ABUT	19+43.09
POST 4	LEFT FORWARD ABUT	19+72.54

\# OFFSET IS LOCATED AT FACE OF THE GUARDRAIL



PROFILE

ALONG ℄ CONSTRUCTION & SURVEY DELMONT ROAD

BENCHMARK

BENCHMARKS WILL BE PROVIDED BY THE STATE AT THE PRE-CONSTRUCTION MEETING

LOCATION

LATITUDE: N 39°42'49"
LONGITUDE: W 82°40'25"
USGS QUADRANGLE: AMANDA

TRAFFIC DATA

DELMONT ROAD:

2001 ADT = 919 ADTT = 9
2021 ADT = 1290 ADTT = 13

US 33:

2001 ADT NORTHBOUND = 9980 ADTT NORTHBOUND = 840
SOUTHBOUND = 9890 SOUTHBOUND = 840
2021 ADT NORTHBOUND = 13710 ADTT NORTHBOUND = 1300
SOUTHBOUND = 12900 SOUTHBOUND = 1300

PROPOSED STRUCTURE DATA

TYPE: FOUR-SPAN CONTINUOUS HPS 70W COMPOSITE STEEL GIRDER WITH SEMI-INTEGRAL TYPE ABUTMENTS, CAP AND COLUMN TYPE PIERS & ON SPREAD FOOTINGS
SPAN: 78'-5", 130'-10", 126'-10½", 82'-4½" C/C BEARINGS ALONG ℄ CONSTRUCTION & SURVEY DELMONT ROAD.
ROADWAY: 36'-0" T/T PARAPET
SKEW: 39°17'06" LF
DESIGN LOADING: HS- 25 (CASE II) AND THE ALTRNATE MILITARY LOADING
WEARING SURFACE: MONOLITHIC CONCRETE
ALIGNMENT: TANGENT
APPROACH SLABS: 30'-0" LONG

DLZ

DESIGNED PHB | CHECKED JCW | DRAWN BDB | REVISED | REVIEWED DOR | DATE 10-12-00 | STRUCTURE FILE NUMBER 2301067

FAIRFIELD COUNTY
STA. 15+17.61
STA. 19+39.67

SITE PLAN
BRIDGE NO. FAI-33-1309
DELMONT ROAD OVER US33 BYPASS

FAI-33-7.31

1834
2027

FILE: M:\proj\992\1009.00\9373\bridge\1309\1309sp01.dgn
DATE: 21-Nov-00 11:47

GENERAL BRIDGE NOTES:

REFERENCE SHALL BE MADE TO STANDARD DRAWINGS:

SICD-1-96M DATED 02/12/97
GSD-1-96 DATED 02/12/97

AND TO THE SUPPLEMENTAL SPECIFICATIONS:

816 DATED 04/21/97 899 DATED 10/21/98
842 DATED 01/06/99 910 DATED 07/28/98
846 DATED 09/09/97
848 DATED 06/30/98
863 DATED 10/12/99
893 DATED 10/12/99

DESIGN SPECIFICATIONS
THIS STRUCTURE CONFORMS TO "STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES" ADOPTED BY THE AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION TRANSPORTATION OFFICIALS, 1996, INCLUDING THE 1997, 1998 AND 1999 INTERIM SPECIFICATIONS AND THE ODOT BRIDGE DESIGN MANUAL.

DESIGN LOADING
HS25, CASE II AND THE ALTERNATE MILITARY LOADING.
FUTURE WEARING SURFACE (FWS) OF 60 LBS/FT^2.

DESIGN STRENGTHS
CONCRETE CLASS C (SUBSTRUCTURE) - COMPRESSIVE STRENGTH 4000 psi
CONCRETE CLASS S (SUPERSTRUCTURE) - COMPRESSIVE STRENGTH 4500 psi
REINFORCING STEEL - ASTM A615, A616, OR A617
GRADE 60 MINIMUM YIELD STRENGTH 60,000 psi.
SPIRAL REINFORCEMENT MAY BE PLAIN BARS, ASTM A82 OR A615
STRUCTURAL STEEL (FLANGES ONLY) - ASTM A709 GRADE HPS70W TMCP STEEL
OTHER STRUCTURAL STEEL - ASTM A588/A709 GRADE 50 - YIELD STRENGTH 50,000 PSI

DECK PROTECTION METHOD
EPOXY COATED REINFORCING STEEL, ALL REINFORCING
2-1/2" CONCRETE COVER AT TOP OF DECK

MONOLITHIC WEARING SURFACE IS ASSUMED, FOR DESIGN PURPOSES, TO BE 1" THICK.

ITEM 203 EMBANKMENT, AS PER PLAN

SEE GENERAL NOTES SHEET 63/2027 FOR ITEM 203 EMBANKMENT, AS PER PLAN A NOTE.

ABUTMENT CONSTRUCTION CONSTRAINTS

PRIOR TO CONSTRUCTION OF THE ABUTMENT SPREAD FOOTINGS, THE SPILL THROUGH SLOPES AND THE BRIDGE APPROACH EMBANKMENTS BEHIND THE ABUTMENT SHALL BE CONSTRUCTED UP TO THE LEVEL OF THE SUBGRADE ELEVATION FOR A MINIMUM DISTANCE OF 200 FEET BEHIND EACH ABUTMENT. THE EXCAVATION FOR THE ABUTMENT FOOTINGS SHALL NOT BEGIN UNTIL AFTER THE ABOVE-REQUIRED EMBANKMENT HAS BEEN CONSTRUCTED AND A MINIMUM WAITING PERIOD OF 30 DAYS HAS ELAPSED AFTER THE COMPLETION OF THE CONSTRUCTED BRIDGE APPROACH EMBANKMENT

UTILITY LINES:
ALL EXPENSES INVOLVED IN REINSTALLING THE AFFECTED UTILITY LINES SHALL BE BORNE BY THE UTILITIES. THE CONTRACTOR AND UTILITIES ARE TO COOPERATE BY ARRANGING THEIR WORK IN SUCH A MANNER THAT INCONVENIENCE TO EITHER WILL BE HELD TO A MINIMUM.

FOUNDATION BEARING PRESSURE

ABUTMENT FOOTINGS, AS DESIGNED, PRODUCE A MAXIMUM BEARING PRESSURE OF 1.86 TONS PER SQUARE FOOT. THE ALLOWABLE BEARING PRESSURE IS 2.0 TONS PER SQUARE FOOT.

PIER FOOTINGS, AS DESIGNED, PRODUCE A MAXIMUM BEARING PRESSURE OF 2.93 TONS PER SQUARE FOOT. THE ALLOWABLE BEARING PRESSURE IS 3.0 TONS PER SQUARE FOOT.

ITEM 516 SEMI-INTEGRAL ABUTMENT EXPANSION JOINT SEAL, AS PER PLAN:
INSTALL A 3-FOOT WIDE STRIP, 3/32-INCH THICK, GENERAL PURPOSE, HEAVY DUTY NEOPRENE SHEET WITH NYLON FABRIC REINFORCEMENT AT LOCATIONS SHOWN ON THE PLANS. SECURE THE 3-FOOT WIDE NEOPRENE SHEETING TO THE CONCRETE WITH 1-1/4" x 3/32" (LENGTH x SHANK DIAMETER) #10 GALVANIZED BUTTON HEAD SPIKE THROUGH A 1" OUTSIDE DIAMETER, #10 GAGE GALVANIZED WASHER. MAXIMUM FASTENER SPACING IS 9 INCHES. OTHER SIMILAR GALVANIZED DEVICES WHICH WILL NOT DAMAGE EITHER THE NEOPRENE OR THE CONCRETE MAY BE USED SUBJECT TO THE APPROVAL OF THE ENGINEER.

CENTER THE NEOPRENE STRIPS ON ALL JOINTS. FOR HORIZONTAL JOINTS, SECURE THE HORIZONTAL NEOPRENE STRIP BY USING A SINGLE LINE OF FASTENERS, STARTING AT 6 INCHES (+/-) FROM THE TOP OF THE NEOPRENE STRIP. FOR THE VERTICAL JOINTS SECURE THE VERTICAL NEOPRENE STRIP BY USING A SINGLE VERTICAL LINE OF FASTENERS, STARTING AT 6 INCHES (+/-) FROM THE VERTICAL EDGE OF THE NEOPRENE STRIP NEAREST TO THE CENTERLINE OF ROADWAY. FOR VERTICAL JOINTS, INSTALL 2 ADDITIONAL FASTENERS, AT 6 INCHES CENTER TO CENTER, ACROSS THE TOP OF THE NEOPRENE STRIP ON THE SAME SIDE OF THE VERTICAL JOINT AS WHERE THE SINGLE VERTICAL ROW OF FASTENERS IS LOCATED.

THE VERTICAL NEOPRENE STRIPS SHOULD COMPLETELY OVERLAP THE HORIZONTAL STRIPS. LAPS IN THE LENGTH OF THE HORIZONTAL STRIPS DUE TO MATERIAL MANUFACTURING SHALL BE AT LEAST ONE FOOT IN LENGTH, IF NOT VULCANIZED OR ADHESIVE BONDED. NO LAPS ARE ACCEPTABLE IN VERTICALLY INSTALLED NEOPRENE STRIPS.

THE NEOPRENE SHEETING SHALL BE 3/32-INCH THICK GENERAL PURPOSE, HEAVY DUTY NEOPRENE SHEET WITH NYLON FABRIC REINFORCEMENT. THE SHEETING SHALL BE "FAIRPRENE NUMBER NN-0003", BY E.I. DUPONT DE NEMOURS AND COMPANY, INC., "WINGPRENE" BY GOODYEAR TIRE & RUBBER COMPANY, OR AN APPROVED ALTERNATE. THE NEOPRENE SHEETING SHALL CONFORM TO THE FOLLOWING:

DESCRIPTION OF TEST	ASTM METHOD	REQUIREMENT
THICKNESS, INCHES	D751	0.094 +/- 0.01
BREAKING STRENGTH, GRAB WXF, LBS, MINIMUM	D751	700 x 700
ADHESIVE 1" STRIP, 2" MINIMUM, LBS, MINIMUM	D751	9
BURST STRENGTH (MULLEN) psi, MINIMUM	D751	1400
HEAT AGING 70 HOURS, T 212°F, 180° BEND WITHOUT CRACKING	D2136	NO CRACKING OF COATING
LOW TEMPERATURE BRITTLENESS 1 HOUR AT -40°F, BEND AROUND 1/4" MANDREL	D2136	NO CRACKING OF COATING

IN LIEU OF THE NEOPRENE SHEETING THE CONTRACTOR MAY CHOOSE TO SUPPLY TYPE 3 MEMBRANE, 711.29.

PAYMENT FOR LABOR, MATERIALS, AND INSTALLATION OF THESE ITEMS SHALL BE INCLUDED IN ITEM 516, SEMI-INTEGRAL EXPANSION JOINT SEAL, AS PER PLAN.

CONCRETE PARAPETS
AS SOON AS A CONCRETE SAW CAN BE OPERATED WITHOUT DAMAGING THE FRESHLY PLACED CONCRETE, 1 1/4" DEEP CONTROL JOINTS SHALL BE SAWED INTO THE PERIMETER OF THE CONCRETE PARAPET. THE SAW CUT SHALL BE MADE IN THE COMPLETE CIRCUMFERENCE OF THE PARAPET, STARTING AND ENDING AT THE ELEVATION OF THE CONCRETE DECK. THE SAWCUTS SHALL BE PLACED AT A MINIMUM OF 6 FEET AND A MAXIMUM OF 10 FEET CENTERS. THE USE OF AN EDGE GUIDE, FENCE, OR JIG IS REQUIRED TO INSURE THAT THE CUT JOINT IS STRAIGHT, TRUE, AND ALIGNED ON ALL FACES OF THE PARAPET. THE JOINT WIDTH SHALL BE THE WIDTH OF THE SAW BLADE, A NOMINAL WIDTH OF 1/4 INCH. THE PERIMETER OF THE DEFLECTION CONTROL JOINT SHALL BE SEALED WITH A CAULKING MATERIAL CONFORMING TO FEDERAL SPECIFICATION, TT-S-00227E TO A MINIMUM DEPTH OF 1 INCH. THE BOTTOM 1/2 INCH OF THE INSIDE AND OUTSIDE FACE SHOULD BE LEFT UNSEALED TO ALLOW WATER TO ESCAPE.

CONVERSION OF STANDARD CONSTRUCTION DRAWINGS
SOME OF THE STANDARD BRIDGE DRAWINGS REFERENCED IN THIS PLAN ARE METRIC. ANY CONVERSION OF DIMENSIONS REQUIRED TO CONSTRUCT THE ITEMS SHOWN ON THE STANDARDS SHALL BE THE RESPONSIBILITY OF THE CONTRACTOR. CONVERSIONS SHALL BE MADE USING THE SI (METRIC) TO ENGLISH CONVERSION FACTORS PROVIDED IN SECTION 109.011 OF THE 1997 CONSTRUCTION AND MATERIALS SPECIFICATION. THE APPENDIX OF ASTM E380 SHALL BE UTILIZED FOR ANY ADDITIONAL CONVERSIONS REQUIRED. CONVERSIONS SHALL BE APPROPRIATELY PRECISE AND SHALL REFLECT STANDARD INDUSTRY ENGLISH VALUES WHERE SUITABLE.

SEALING OF CONCRETE SURFACES (EPOXY-URETHANE)

ALL CONCRETE SURFACES, INCLUDING PARAPETS, ABUTMENTS, WINGWALLS, AND PIERS AS DETAILED IN THE PLANS SHALL BE SEALED WITH LIGHT NEUTRAL FEDERAL COLOR NO. 17778.

FIELD PAINTING OF NEW STEEL, INTERMEDIATE AND FINISH COAT, SYSTEM IZEU, AS PER PLAN

STRUCTURAL STEEL SURFACES ON THE GIRDERS AND BEARING LOAD PLATES AS DETAILED IN THE PLANS, SHALL BE PAINTED WITH DARK NEUTRAL, FEDERAL COLOR NO. 10324. SEE SHEETS 12/22, 14/22, AND 20/22 FOR LOCATIONS.

REINFORCING STEEL:

THE REINFORCING STEEL LIST SHOWN IN THE PLANS IS FOR INFORMATION ONLY. IF THE REINFORCING STEEL LIST PROVIDED IN THE PLANS IS USED IT SHALL BE VERIFIED BY THE CONTRACTOR. ANY REVISIONS IN THE REINFORCING STEEL LIST AS SHOWN IN THE PLANS WILL NOT BE A REASON FOR AJUSTMENT IN THE BID PRICE FOR THE STRUCTURAL CONCRETE.

ITEM 842, CLASS C CONCRETE FOOTING, AS PER PLAN

IN ADDITION TO THE REQUIREMENTS OF ITEM 842, REFERENCE MONUMENTS SHALL BE INSTALLED IN EACH PIER AND ABUTMENT SPREAD FOOTING. EACH SPREAD FOOTING SHALL HAVE TWO REFERENCE MONUMENTS INSTALLED, ONE AT EACH END OF THE FOOTING. THE REFERENCE MONUMENT SHALL CONSIST OF A #8, OR LARGER, EPOXY COATED REBAR. IT SHALL BE EMBEDDED INTO THE FOOTING AT LEAST 6" AND EXTEND VERTICALLY 4 TO 6 INCHES ABOVE THE TOP OF THE FOOTING. A SIX INCH DIAMETER, SCHEDULE 40, PLASTIC PIPE SHALL BE INSTALLED AROUND THE REFERENCE MONUMENT; SHALL BE VERTICAL; AND THE TOP OF THE PIPE SHALL BE AT THE FINISHED GRADE. THE PIPE SHALL HAVE A REMOVABLE, SCHEDULE 40, PLASTIC CAP. THE PIPE SHALL BE CENTERED ON THE REFERENCE MONUMENT. THE BOTTOM OF THE PIPE SHALL BE PERMANENTLY ATTACHED TO THE TOP OF THE FOOTING.

THE BELOW TABLE SHALL BE COMPLETED FOR EACH INSTALLED REFERENCE MONUMENT.
THE CONTRACTOR SHALL ESTABLISH A BENCHMARK FOR DETERMINING ELEVATIONS FOR THE BELOW TABLE. THE BENCHMARK SHALL BE THE SAME THROUGHOUT THE PROJECT AND SHALL BE INDEPENDENT OF ALL STRUCTURES.
COMPLETED TABLES SHALL BECOME PART OF THE DISTRICT'S PROJECT PLAN RECORDS AND A COPY SHALL BE SENT TO THE OFFICE OF STRUCTURAL ENGINEERING.

SPREAD FOOTING REFERENCE MONUMENTS TABLE

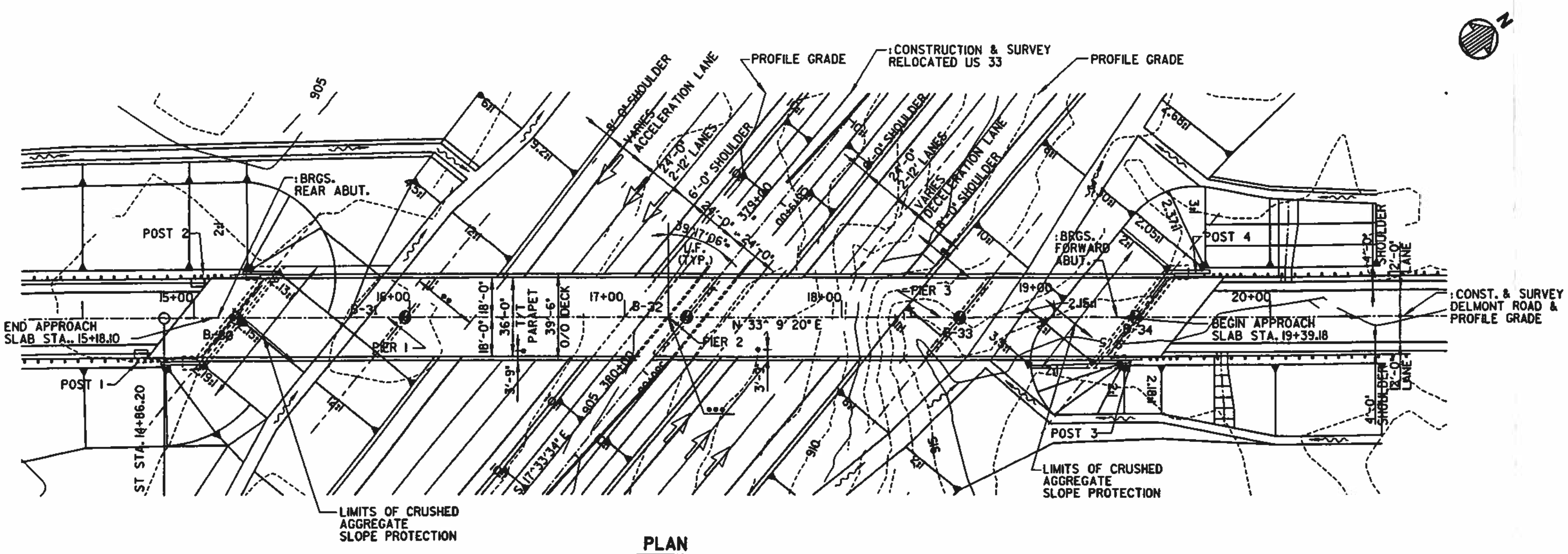
PROJECT No. FAI-33-7.31	BRIDGE No. FAI-33-1309		STRUCTURE FILE No. 2301067	INDEPENDANT BENCHMARK LOC:			
	DATE:						
MAXIMUM BEARING PRESSURE TONS/SQ FT	MONUMENT LOCATION		AFTER FOOTING CONCRETE IS PLACED	BEFORE PLACEMENT OF SUPERSTRUCTURE MEMBERS	BEFORE DECK PLACEMENT	AFTER DECK PLACEMENT	PROJECT COMPLETED
1.86	REAR ABUT	LEFT MON.					
		RIGHT MON.					
2.72	PIER 1	LEFT MON.					
		RIGHT MON.					
2.93	PIER 2	LEFT MON.					
		RIGHT MON.					
2.72	PIER 3	LEFT MON.					
		RIGHT MON.					
1.86	FORWARD ABUT	LEFT MON.					
		RIGHT MON.					

ITEM 842, CLASS C CONCRETE, PIER ABOVE FOOTING, AS PER PLAN

BEFORE THE STRUCTURAL STEEL IS PLACED, THE CONTRACTOR SHALL WRAP EACH PIER CAP AND COLUMN WITH POLYETHYLENE SHEETING TO PREVENT STAINING FROM WATER RUNOFF DURING CONSTRUCTION. THE SHEETING SHALL MEET THE REQUIREMENTS OF 705.06 AND REMAIN IN PLACE UNTIL AFTER THE DECK HAS BEEN CONSTRUCTED. NO POLYETHYLENE SHALL BE PLACED BETWEEN THE ELASTOMERIC BEARING PADS AND THE CONCRETE PIER CAP. THE COST FOR THIS ITEM SHALL BE INCLUDED WITH ITEM 842, CLASS C CONCRETE, PIER CAP AND COLUMN ABOVE FOOTING, AS PER PLAN.

DLZ
620 HUNTLEY ROAD • COLUMBUS, OHIO 43229
REVIEWED DOR DATE 10-12-00
STRUCTURE FILE NUMBER 2301067
DRAWN RJ REVISED
DESIGNED PHB CHECKED VKH
GENERAL NOTES
BRIDGE NO. FAI-33-1309
DELMONT ROAD OVER US33 BYPASS
FAI-33-7.31

FILE: M:\proj\1992\1009.00\19373\bridge\1309\1309sn01.dgn
DATE: 21-Nov-00 11:47

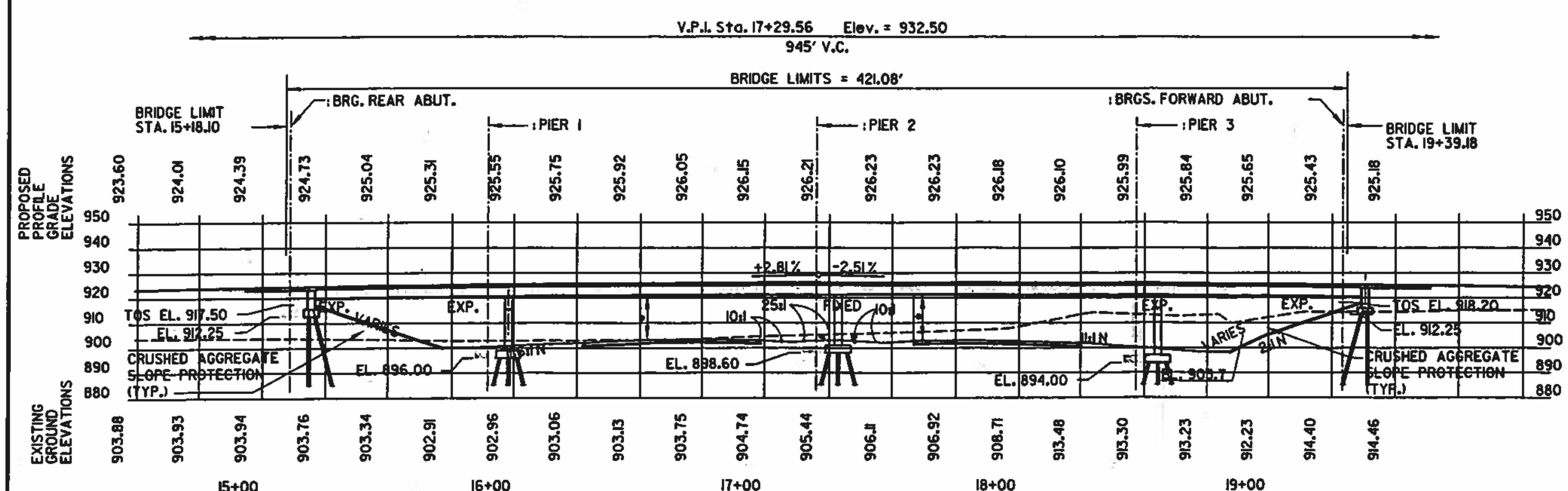


PLAN

SUBSTRUCTURE @ :DELMONT ROAD	STATION	* PILE LENGTH
:BRGS. REAR ABUTMENT	15+19.39	52.4'
:PIER 1	15+97.89	36.7'
:PIER 2	17+28.64	40.8'
:PIER 3	18+55.39	33.9'
:BRGS. FORWARD ABUTMENT	19+37.89	48.2'

* ALL PILE TYPES 12" Ø CIP TYP.

POST NUMBER	FIRST POST GUARD RAIL	STATION
POST 1	RIGHT REAR ABUT.	14+83.22
POST 2	LEFT REAR ABUT.	15+12.66
POST 3	RIGHT FORWARD ABUT.	19+44.61
POST 4	LEFT FORWARD ABUT.	19+74.05



PROFILE

ALONG :CONSTRUCTION & SURVEY DELMONT ROAD

NOTES:

1. EARTHWORK LIMITS SHOWN ARE APPROXIMATE ACTUAL SLOPES SHALL CONFORM TO PLAN CROSS SECTIONS.
2. ⊕= BORING LOCATIONS.

LEGEND

- * MINIMUM VERTICAL CLEARANCE
  - 16'-10¼" ACTUAL SOUTHBOUND
  - 16'-6" REQUIRED
  - 17'-6¼" ACTUAL NORTHBOUND
  - 16'-6" REQUIRED
- ** MINIMUM HORIZONTAL CLEARANCE
  - 30'-3" ACTUAL SOUTHBOUND
  - 30'-0" REQUIRED
  - 30'-0¼" ACTUAL NORTHBOUND
  - 30'-0" REQUIRED
- *** :PIER 2, STA. 17+28.64 DELMONT RD. = STA. 379+68.18 RELOCATED US 33

ABUT. = ABUTMENT
ADT = AVERAGE DAILY TRAFFIC
ADTT = AVERAGE DAILY TRUCK TRAFFIC
BRG. = BEARING
CB-3 = CATCH BASIN TYPE 3
C/C = CENTER TO CENTER
EL = ELEVATION
EXP = EXPANSION JOINT
LT. = LEFT
N = NORMAL
O/O = OUT TO OUT
P.I. = POINT OF INTERSECTION
STA. = STATION
S.T. = SPIRAL TANGENT
T/T = TOE TO TOE
TOS = TOP OF SLOPE
V.C. = VERTICAL CURVE
V.P.I. = VERTICAL POINT OF INTERSECTION

BENCHMARK

BENCHMARK:

LOCATION

LATITUDE: N 39°42'49"
LONGITUDE: W 82°40'25"
USGS QUADRANGLE: AMANDA

TRAFFIC DATA

DELMONT ROAD:

2001 ADT = 919 ADTT = 9
2021 ADT = 1290 ADTT = 13

US 33:

2001 ADT NORTHBOUND = 9980 ADTT NORTHBOUND = 840
SOUTHBOUND = 9890 SOUTHBOUND = 840
2021 ADT NORTHBOUND = 13710 ADTT NORTHBOUND = 1300
SOUTHBOUND = 12900 SOUTHBOUND = 1300

PROPOSED STRUCTURE DATA

TYPE: FOUR-SPAN CONTINUOUS HPS 70W COMPOSITE STEEL GIRDER WITH SEMI-INTEGRAL TYPE ABUTMENTS, CAP AND COLUMN TYPE PIERS.
SPAN: 78'-6", 130'-9", 126'-9", 82'-6" C/C BEARINGS ALONG DELMONT ROAD.
ROADWAY: 36'-0" T/T PARAPET
SKEW: 39°17'06" L.F.
DESIGN LOADING: HS- 20 (CASE II) AND THE ALTERNATE MILITARY LOADING
WEARING SURFACE: MONOLITHIC CONCRETE
ALIGNMENT: TANGENT
APPROACH SLABS: AS-1-81 (25'-0" LONG)

DODSON-STILSON INC.
DESIGN AGENCY
ENGINEERS • ARCHITECTS • SCIENTISTS

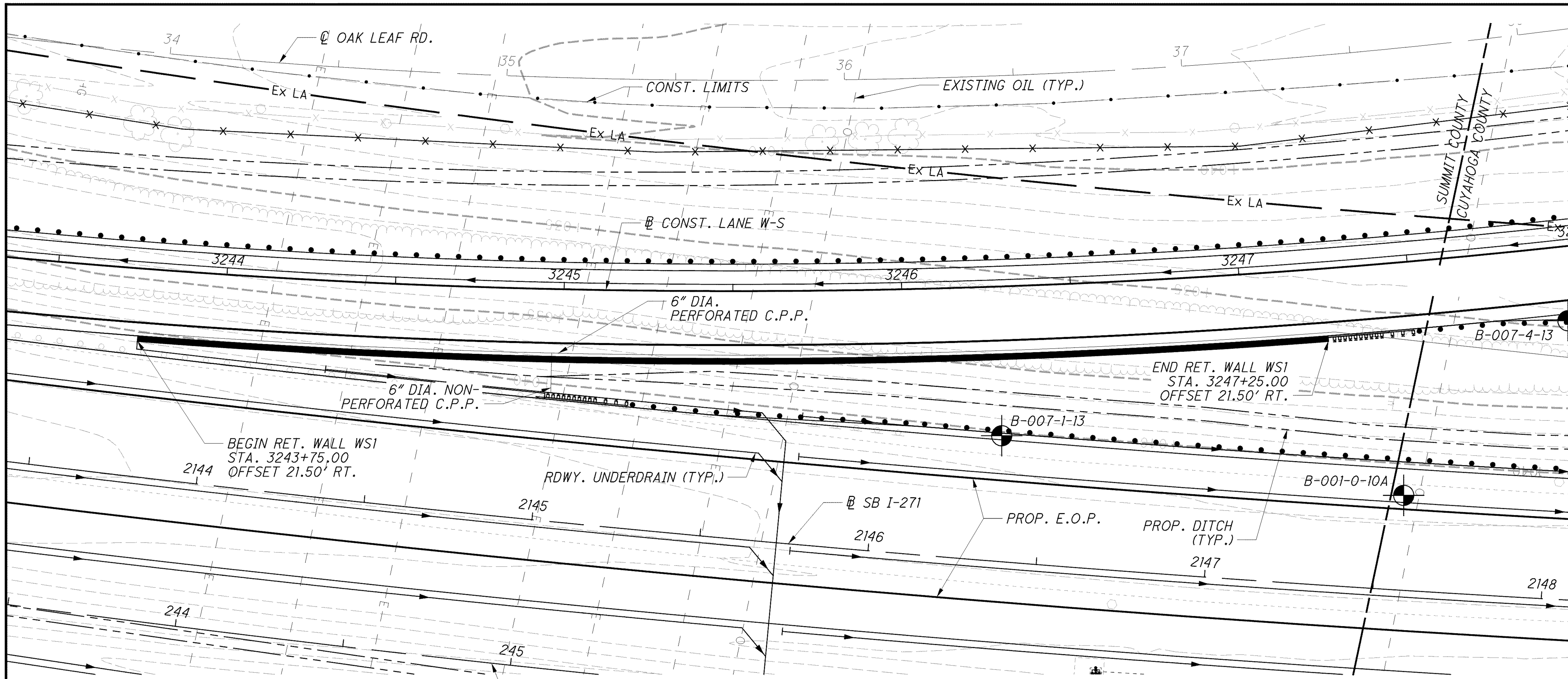
DATE | REVIEWED | STRUCTURE FILE NUMBER 2301067 | DRAWN | REVISED | DESIGNED | CHECKED

FAIRFIELD COUNTY
STA. 15+18.10
STA. 19+39.18

SITE PLAN

BRIDGE NO. FAI-33-1309
DELMONT ROAD OVER US33 BYPASS

FAI-33-7.08



PLAN

NOTES

1. FOR BENCHMARK INFORMATION, SEE CENTER LINE REFERENCES & BENCHMARKS, ROADWAY SHEET 8 OF 2013.
2. ALL EXISTING OVERHEAD AND UNDERGROUND UTILITIES ARE TO REMAIN UNLESS NOTED OTHERWISE.
3. WALL OFFSETS GIVEN TO FRONT FACE OF WALL.

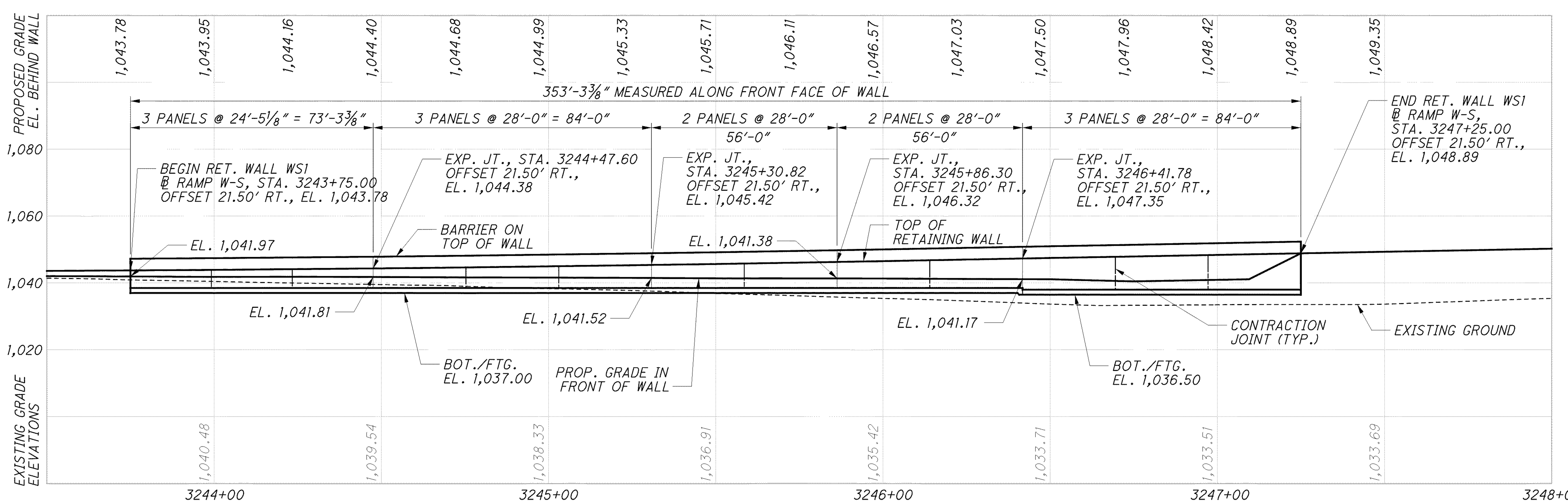
LEGEND

⊕ - PROJECT BORING LOCATION

CURVE DATA

LANE W-S

P.I. Sta. 3246+62.32
Δ = 22° 17' 15" (LT)
Dc = 2° 30' 00"
R = 2,291.83'
Δc = 18° 11' 15" (LT)
Lc = 727.50'
Es = 44.54'
C = 724.45'
C.B. 1 = N 3° 26' 09" W
C.B. = N 13° 49' 47" W
C.B. 2 = S 24° 21' 24" E



ELEVATION

HORIZONTAL SCALE IN FEET 0 20 40

CALCULATED ASP CHECKED LNB

RETAINING WALL PLAN AND ELEVATION
RETAINING WALL WS1 ALONG LANE W-S
STATION 3243+75.00 TO STATION 3247+25.00

CUY/SUM-271-00.00/14.87
CUY/SUM-480-29.58/00.00
PID No. 80418

GENERAL NOTES

REFER TO THE FOLLOWING STANDARD BRIDGE DRAWINGS:

SBR-1-13 DATED (REVISED) 1-17-14

AND TO THE FOLLOWING SUPPLEMENTAL SPECIFICATION:

800 DATED 07-18-14

DESIGN SPECIFICATIONS:

THIS STRUCTURE CONFORMS TO THE "LRFD BRIDGE DESIGN SPECIFICATIONS" ADOPTED BY THE AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS, 6TH EDITION, INCLUDING THE 2013 INTERIM SPECIFICATIONS, AND THE ODOT BRIDGE DESIGN MANUAL, 2007, INCLUDING INTERIM SPECIFICATIONS.

DESIGN STRESSES:

CONCRETE CLASS QC1: (RETAINING WALLS) - COMPRESSIVE STRENGTH 4 KSI
CONCRETE CLASS QC2: (PARAPET) - COMPRESSIZE STRENGTH 4.5 KSI
REINFORCING STEEL - MINIMUM YIELD STRENGTH 60 KSI

SPREAD FOOTING FOUNDATIONS:
SPREAD FOOTING FOUNDATIONS:

STA. 3243+75.00 TO STA. 3244+47.60
FOUNDATION BEARING RESISTANCE: THE RETAINING WALL FOOTING BETWEEN THE ABOVE STATIONING, AS DESIGNED, PRODUCES A MAXIMUM SERVICE LOAD PRESSURE OF 0.94 KIPS PER SQUARE FOOT AND A MAXIMUM STRENGTH LOAD PRESSURE OF 1.27 KIPS PER SQUARE FOOT. THE FACTORED BEARING RESISTANCE IS 8.60 KIPS PER SQUARE FOOT.

STA. 3244+47.60 TO STA. 3245+30.82
FOUNDATION BEARING RESISTANCE: THE RETAINING WALL FOOTING BETWEEN THE ABOVE STATIONING, AS DESIGNED, PRODUCES A MAXIMUM SERVICE LOAD PRESSURE OF 1.08 KIPS PER SQUARE FOOT AND A MAXIMUM STRENGTH LOAD PRESSURE OF 1.46 KIPS PER SQUARE FOOT. THE FACTORED BEARING RESISTANCE IS 8.60 KIPS PER SQUARE FOOT.

STA. 3245+30.82 TO STA. 3245+86.30
FOUNDATION BEARING RESISTANCE: THE RETAINING WALL FOOTING BETWEEN THE ABOVE STATIONING, AS DESIGNED, PRODUCES A MAXIMUM SERVICE LOAD PRESSURE OF 1.20 KIPS PER SQUARE FOOT AND A MAXIMUM STRENGTH LOAD PRESSURE OF 1.65 KIPS PER SQUARE FOOT. THE FACTORED BEARING RESISTANCE IS 8.60 KIPS PER SQUARE FOOT.

STA. 3245+86.30 TO STA. 3246+41.78
FOUNDATION BEARING RESISTANCE: THE RETAINING WALL FOOTING BETWEEN THE ABOVE STATIONING, AS DESIGNED, PRODUCES A MAXIMUM SERVICE LOAD PRESSURE OF 1.35 KIPS PER SQUARE FOOT AND A MAXIMUM STRENGTH LOAD PRESSURE OF 1.87 KIPS PER SQUARE FOOT. THE FACTORED BEARING RESISTANCE IS 8.60 KIPS PER SQUARE FOOT.

STA. 3246+41.78 TO STA. 3247+25.00
FOUNDATION BEARING RESISTANCE: THE RETAINING WALL FOOTING BETWEEN THE ABOVE STATIONING, AS DESIGNED PRODUCES A MAXIMUM SERVICE LOAD PRESSURE OF 1.71 KIPS PER SQUARE FOOT AND A MAXIMUM STRENGTH LOAD PRESSURE OF 2.42 KIPS PER SQUARE FOOT. THE FACTORED BEARING RESISTANCE IS 8.60 KIPS PER SQUARE FOOT.

ESTIMATED QUANTITIES

FUNDING 01/IMS/PV	FUNDING 02/IMS/BR	ITEM	EXT.	TOTAL	UNIT	DESCRIPTION	REF. SHT NUMBER
949		304	20000	949	CY	AGGREGATE BASE	
887		503	21100	887	CY	UNCLASSIFIED EXCAVATION	
43,732		509	10000	43,732	LB	EPOXY COATED REINFORCING STEEL	
149		511	46010	149	CY	CLASS QC1 CONCRETE	
187		511	46513	187	CY	CLASS QC1 CONCRETE WITH QC/QA, FOOTING, AS PER PLAN	2
56		511	53012	56	CY	CLASS QC2 CONCRETE, MISC.: PARAPET	
500		512	10100	500	SY	SEALING OF CONCRETE SURFACES (EPOXY-URETHANE)	
34		512	33000	34	SY	TYPE 2 WATERPROOFING	
62		516	13600	62	SF	1" PREFORMED EXPANSION JOINT FILLER	
176		518	21200	176	CY	POROUS BACKFILL WITH FILTER FABRIC	
353		518	40000	353	FT	6" PERFORATED CORRUGATED PLASTIC PIPE	
198		518	40010	198	FT	6" NON-PERFORATED CORRUGATED PLASTIC PIPE, INCLUDING SPECIALS	

ITEM 511 - CLASS QC1 CONCRETE WITH QC/QA, FOOTING, AS PER PLAN:

IN ADDITION TO THE REQUIREMENTS OF ITEM 511, INSTALL REFERENCE MONUMENTS IN THE RETAINING WALL FOOTING AT THE LOCATIONS SHOWN IN THE TABLE BELOW.

THE REFERENCE MONUMENT SHALL CONSIST OF A #8, OR LARGER, EPOXY COATED REBAR EMBEDDED AT LEAST 6" INTO THE FOOTING AND EXTENDED VERTICALLY 4 TO 6 INCHES ABOVE THE TOP OF THE FOOTING. INSTALL A SIX INCH DIAMETER, SCHEDULE 40, PLASTIC PIPE AROUND THE REFERENCE MONUMENT. CENTER THE PIPE ON THE REFERENCE MONUMENT AND PLACE THE PIPE VERTICAL WITH ITS TOP AT THE FINISHED GRADE. THE PIPE SHALL HAVE A REMOVABLE, SCHEDULE 40, PLASTIC CAP. PERMANENTLY ATTACH THE BOTTOM OF THE PIPE TO THE TOP OF THE FOOTING.

ESTABLISH A BENCHMARK TO DETERMINE THE ELEVATIONS OF THE REFERENCE MONUMENTS AT VARIOUS MONITORING PERIODS THROUGHOUT THE LENGTH OF THE CONSTRUCTION PROJECT. THE BENCHMARK SHALL BE THE SAME THROUGHOUT THE PROJECT AND SHALL BE INDEPENDENT OF ALL STRUCTURES.

RECORD THE ELEVATION OF EACH REFERENCE MONUMENT AT EACH MONITORING PERIOD SHOWN IN THE TABLE BELOW.

THE ORIGINAL COMPLETED TABLES WILL BECOME PART OF THE DISTRICT'S PROJECT PLAN RECORDS. SEND A COPY OF THE COMPLETED TABLES TO THE OFFICE OF STRUCTURAL ENGINEERING.

ITEM 512 - SEALING OF CONCRETE SURFACES (EPOXY-URETHANE):

THE COLOR OF THE FINISH COAT FOR ALL SURFACES SHALL BE FEDERAL COLOR NO. 595B-27722 (BUFF).

ABBREVIATIONS

℄	= BASELINE
BOT.	= BOTTOM
℄	= CENTERLINE
C.J.	= CONSTRUCTION JOINT
CLR.	= CLEARANCE
CONC.	= CONCRETE
CONST.	= CONSTRUCTION
C.P.P.	= CORRUGATED PLASTIC PIPE
DIA.	= DIAMETER
DWG.	= DRAWING
E.F.	= EACH FACE
EL.	= ELEVATION
ELEC.	= ELECTRIC
E.O.	= EDGE OF
E.O.P.	= EDGE OF PAVEMENT
EXIST.	= EXISTING
EXP.	= EXPANSION
F.F.	= FAR FACE
FTG.	= FOOTING
JT.	= JOINT
MAX.	= MAXIMUM
MIN.	= MINIMUM
N.F.	= NEAR FACE
PCC	= POINT OF COMPOUND CURVATURE
PEJF	= PREFORMED EXPANSION JOINT FILLER
P.I.	= POINT OF INTERSECTION
PROP.	= PROPOSED
PT	= POINT OF TANGENCY
RDWY.	= ROADWAY
RET.	= RETAINING
RT.	= RIGHT
SAN.	= SANITARY SEWER
SC	= POINT OF CHANGE FROM SPIRAL TO CIRCULAR CURVE
SER.	= SERIES
SHLDR.	= SHOULDER
SPA.	= SPACED
STA.	= STATION
STD.	= STANDARD
STM.	= STORM SEWER
T/WALL	= TOP OF WALL
TYP.	= TYPICAL
1H:1V	= 1 HORIZONTAL TO 1 VERTICAL

PROJECT NUMBER: CUY-271-0.00, PID: 80418 WALL NUMBER: WS1	MAXIMUM FACTORED BEARING PRESSURE: 2.42 KIPS PER SQ. FT.					
BENCHMARK LOCATION:						
MONITORING PERIOD	STA. 3243+76.00	STA. 3244+46.60	STA. 3245+29.82	STA. 3245+85.30	STA. 3246+40.78	STA. 3247+24.00
AFTER FOOTING CONCRETE IS PLACED:	1038.550	1038.475	1038.555	1038.505	1038.485	1037.990
AFTER WALL CONCRETE IS PLACED AND BACKFILLED:	1038.550	1038.475	1038.555	1038.505	1038.485	1037.990
PROJECT PHASE 2F COMPLETION:	1038.550	1038.475	1038.555	1038.505	1038.485	1037.990
	#1	#2	#3	#4	#5	#6

1/8/19 7/26/19

10/30/19

10/30/19

CALCULATED ASP
CHECKED LNB

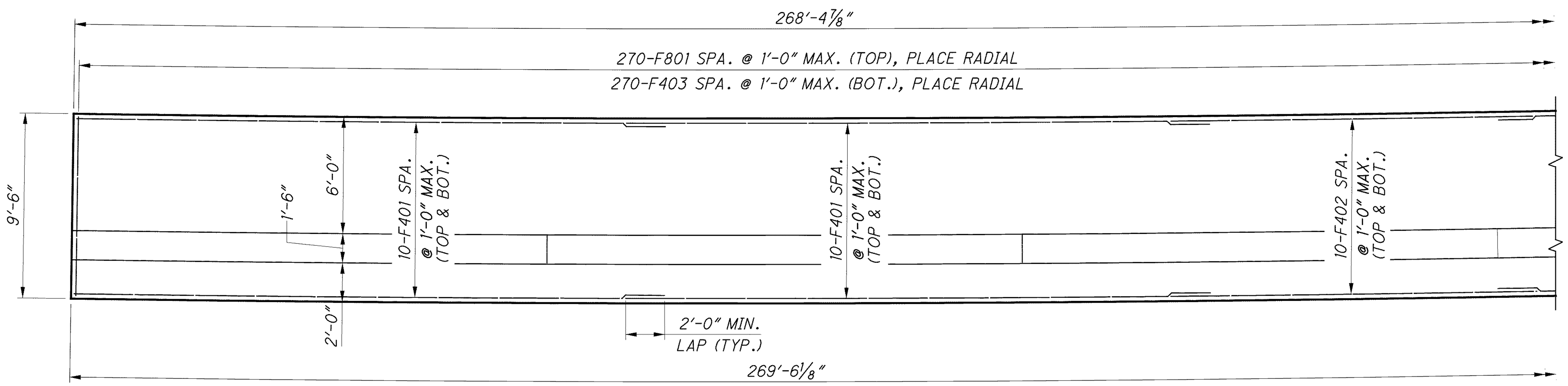
GENERAL NOTES & ESTIMATED QUANTITIES
RETAINING WALL WS1 ALONG LANE W-S
STATION 3243+75.00 TO STATION 3247+25.00

CUY/SUM-271-00.00/14.87
CUY/SUM-480-29.58/00.00
PID No. 80418

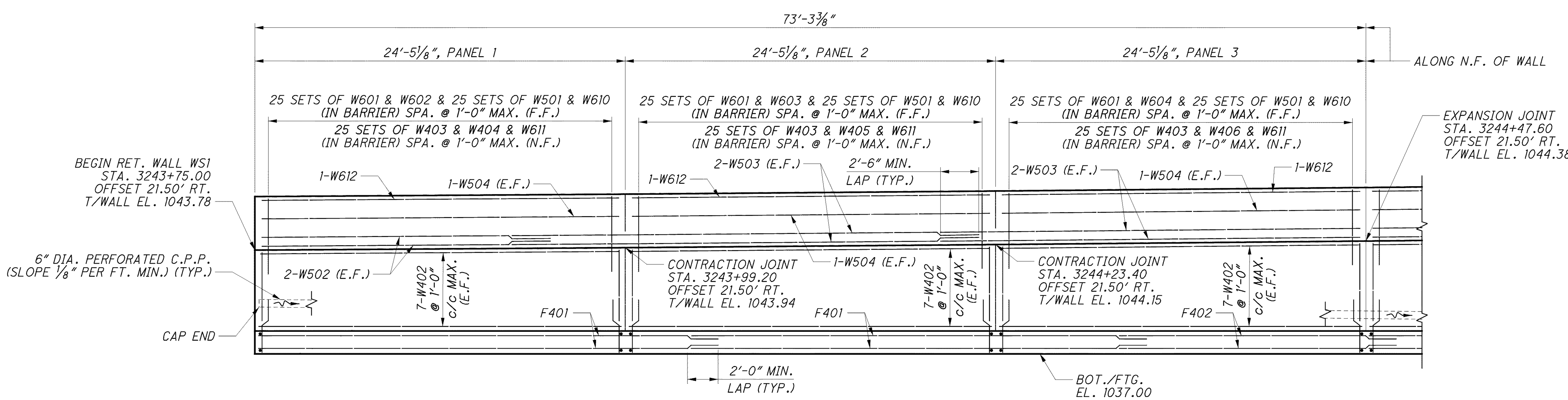
2/9

1354
2013

...\sheets\80418_WS1WN001.dgn (Sheet) 9/30/2014 6:41:13 AM dshaner



FOOTING PLAN



ELEVATION

NOTES

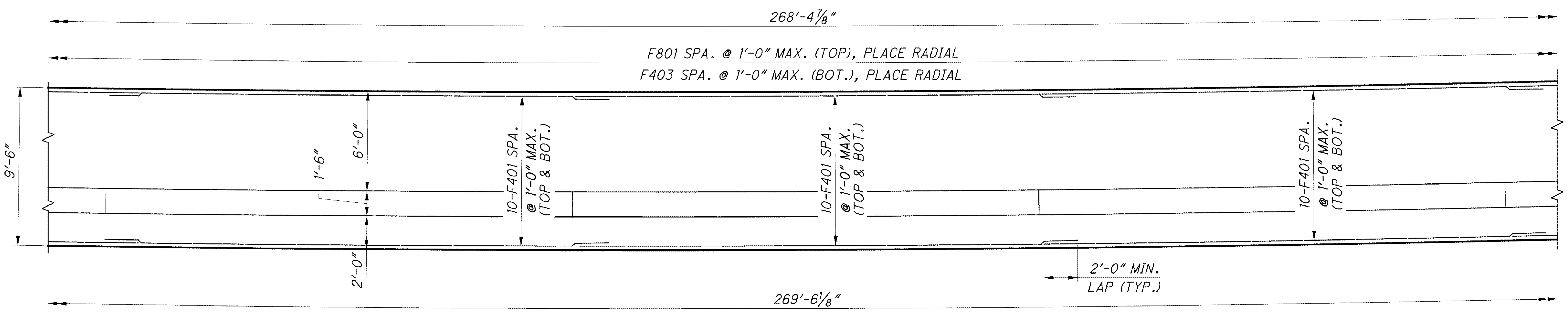
1. FOR EXPANSION AND CONTRACTION JOINTS, SEE SHEET 8/9

CALCULATED ASP
CHECKED LNB

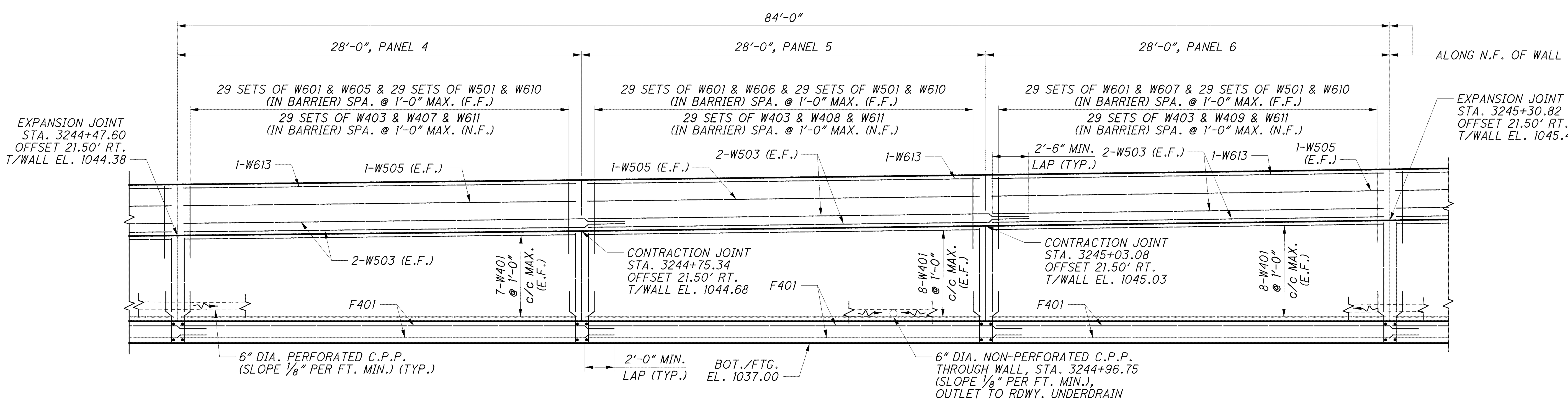
RETAINING WALL DETAILS - 1
RETAINING WALL WS1 ALONG LANE W-S
STATION 3243+75.00 TO STATION 3247+25.00

CUY/SUM-271-00.00/14.87
CUY/SUM-480-29.58/00.00
PID No. 80418

1355
2013



FOOTING PLAN



ELEVATION

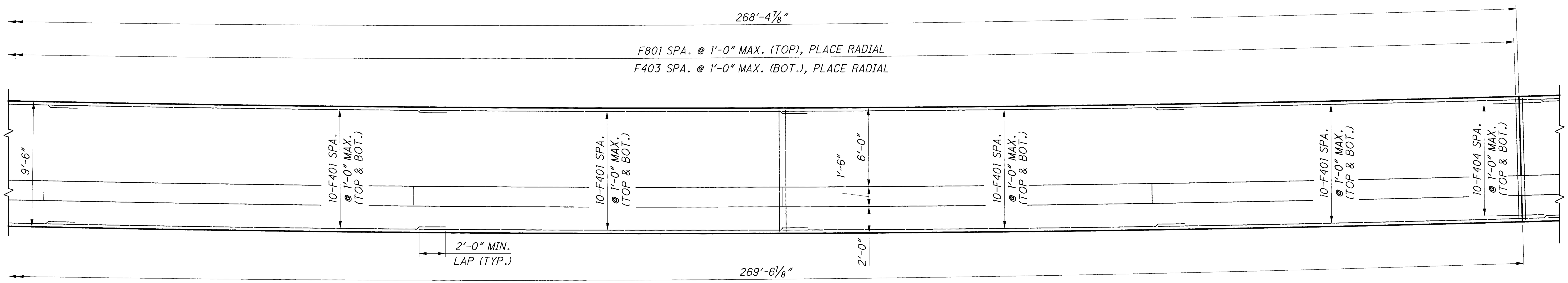
NOTES

1. FOR EXPANSION AND CONTRACTION JOINTS, SEE SHEET 8/9

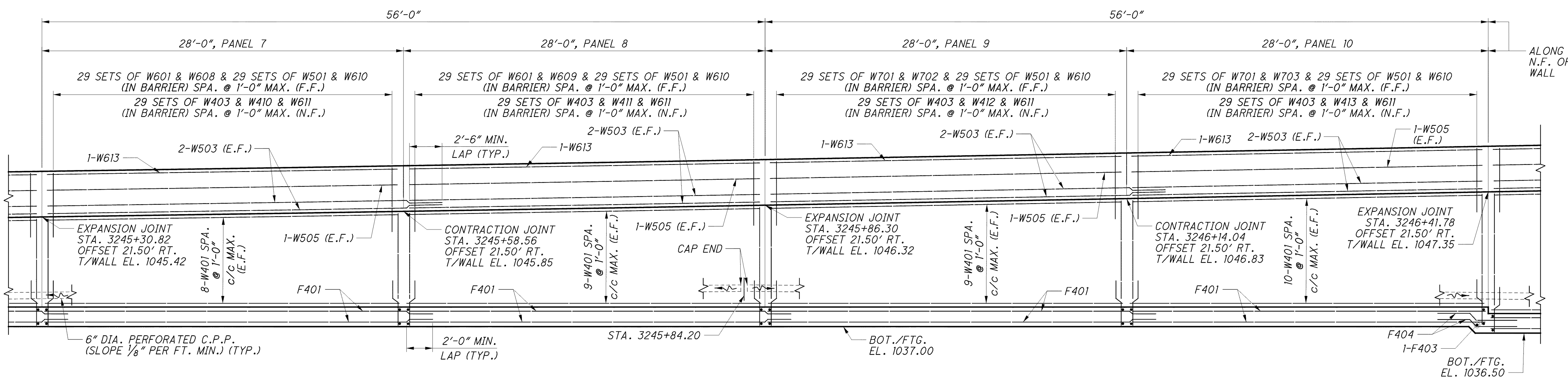
CALCULATED ASP
CHECKED LNB

RETAINING WALL DETAILS - 2
RETAINING WALL WS1 ALONG LANE W-S
STATION 3243+75.00 TO STATION 3247+25.00

CUY/SUM-271-00.00/14.87
CUY/SUM-480-29.58/00.00
PID No. 80418



FOOTING PLAN



ELEVATION

NOTES

1. FOR EXPANSION AND CONTRACTION JOINTS, SEE SHEET 8/9

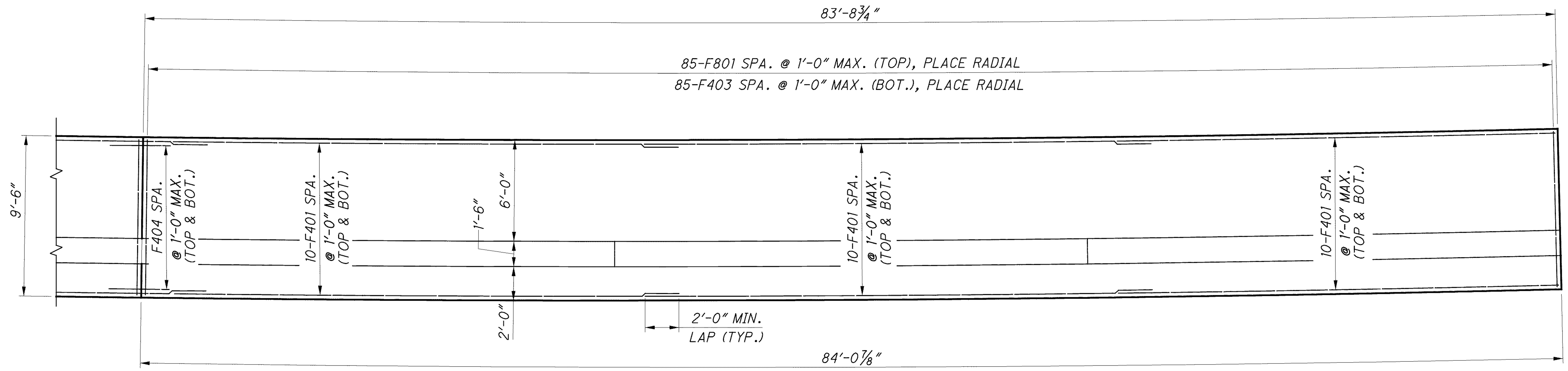
CALCULATED ASP
CHECKED LNB

RETAINING WALL DETAILS - 3
RETAINING WALL WS1 ALONG LANE W-S
STATION 3243+75.00 TO STATION 3247+25.00

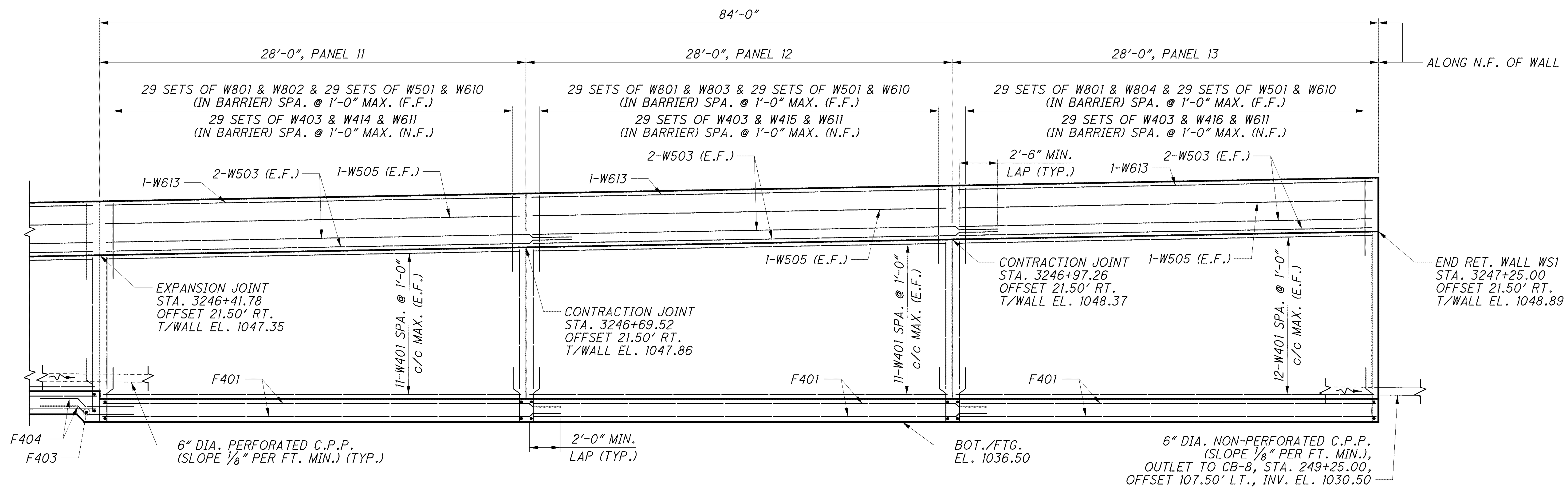
CUY/SUM-271-00.00/14.87
CUY/SUM-480-29.58/00.00
PID No. 80418

5/9

1357/2013



FOOTING PLAN



ELEVATION

NOTES

1. FOR EXPANSION AND CONTRACTION JOINTS, SEE SHEET 8/9

CALCULATED ASP
CHECKED LNB

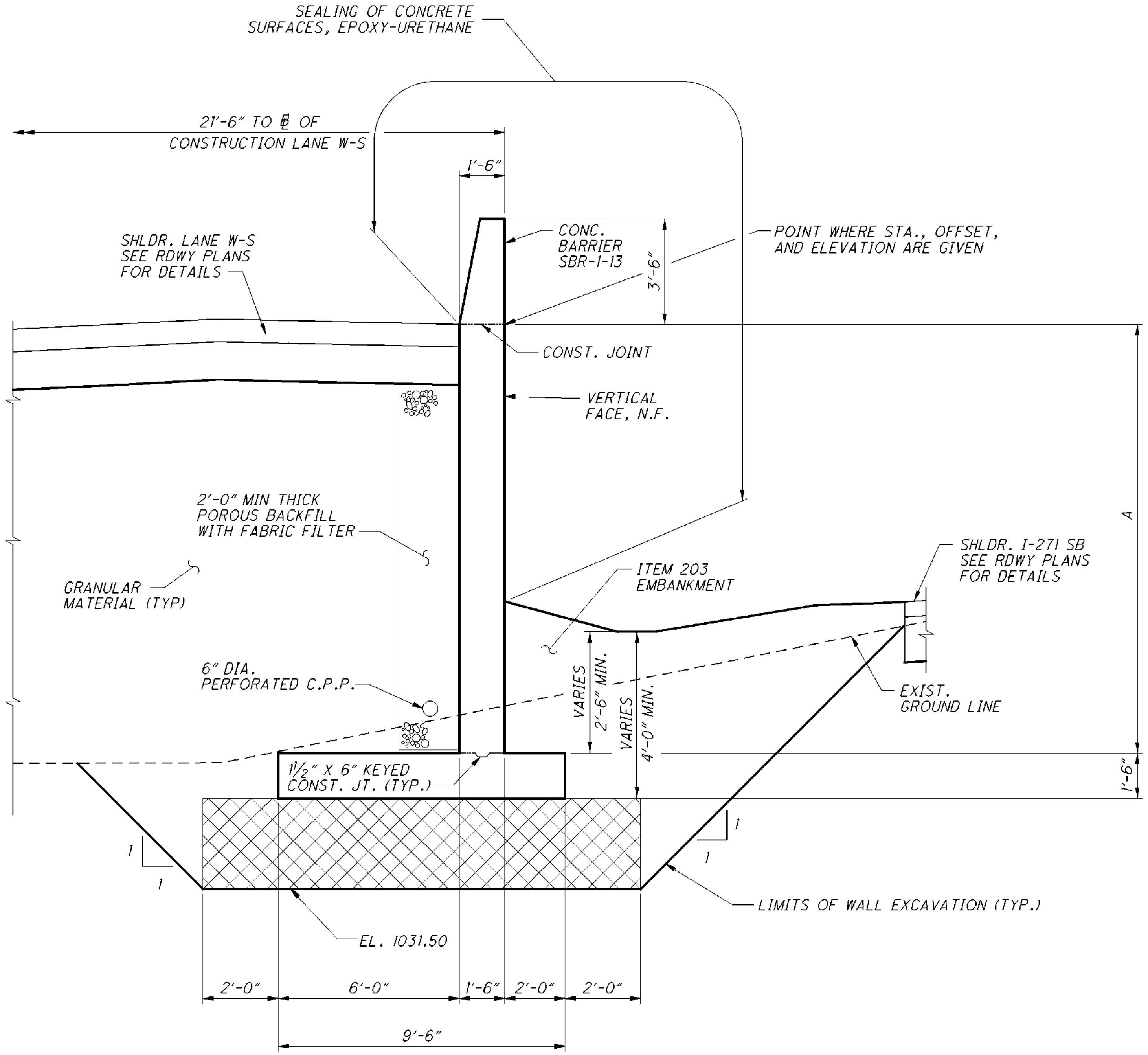
RETAINING WALL DETAILS - 4
RETAINING WALL WS1 ALONG LANE W-S
STATION 3243+75.00 TO STATION 3247+25.00

CUY/SUM-271-00.00/14.87
CUY/SUM-480-29.58/00.00
PID No. 80418

6/9

1358
2013

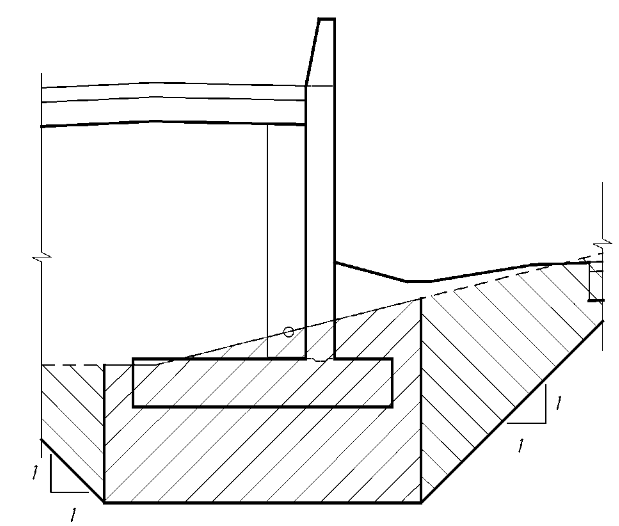
...\sheets\80418_WS1WE004.dgn (Sheet) 9/30/2014 6:41:23 AM dshaner



TYPICAL SECTION

LOOKING UPSTATION

WALL DIMENSIONS		
	A	
	MIN.	MAX.
PANEL 1	5'-3⅜"	5'-5⅜"
PANEL 2	5'-5⅜"	5'-7¾"
PANEL 3	5'-7¾"	5'-10½"
PANEL 4	5'-10½"	6'-2¼"
PANEL 5	6'-2¼"	6'-6⅜"
PANEL 6	6'-6⅜"	6'-11"
PANEL 7	6'-11"	7'-4¼"
PANEL 8	7'-4¼"	7'-9⅞"
PANEL 9	7'-9⅞"	8'-4"
PANEL 10	8'-4"	8'-10⅛"
PANEL 11	9'-4⅛"	9'-10¼"
PANEL 12	9'-10¼"	10'-4⅜"
PANEL 13	10'-4⅜"	10'-10⅝"



APPROXIMATE RETAINING WALL WORK LIMITS

STA. 3243+75.00 TO STA. 3247+25.00

LEGEND

DENOTES LIMITS FOR ITEM 203 - EXCAVATION

DENOTES LIMITS FOR ITEM 304 - AGGREGATE BASE

DENOTES LIMITS FOR ITEM 503 - UNCLASSIFIED EXCAVATION

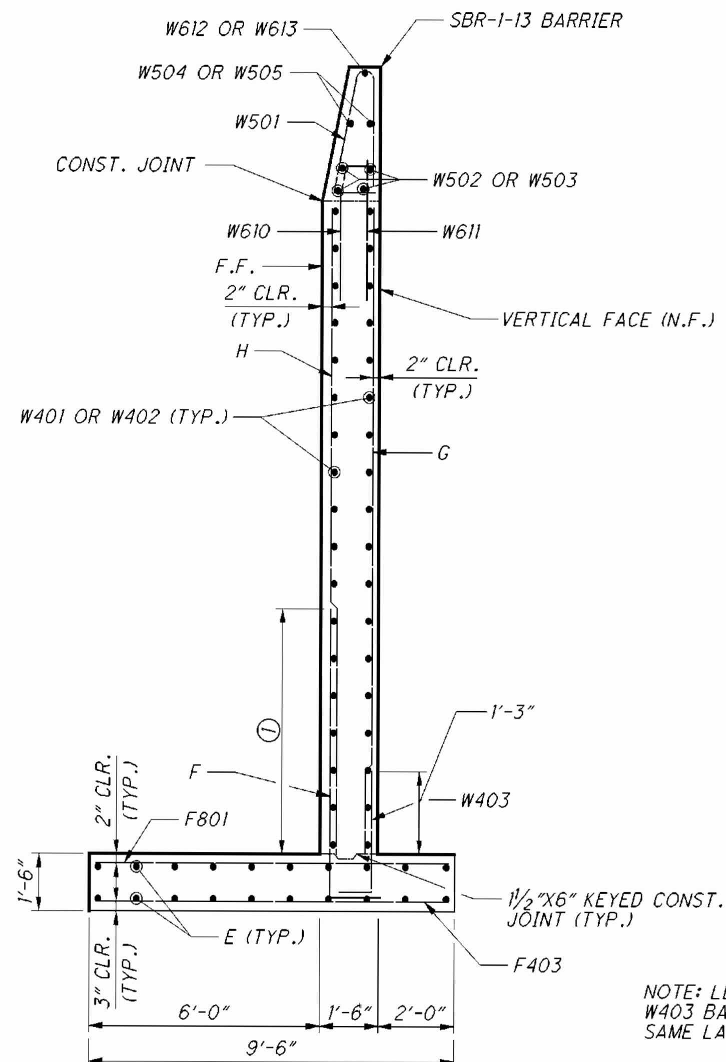
REV. BY	DATE	DESCRIPTION
ARA	4/7/2016	1 - REVISION 01

CALCULATED ASP CHECKED LNB

RETAINING WALL CROSS SECTION & MISC. DETAILS
RETAINING WALL WS1 ALONG LANE W-S
STATION 3243+75.00 TO STATION 3247+25.00

CUY/SUM-271-00.00/14.87
CUY/SUM-480-29.58/00.00
PID No. 80418

1359
2013



TYPICAL SECTION

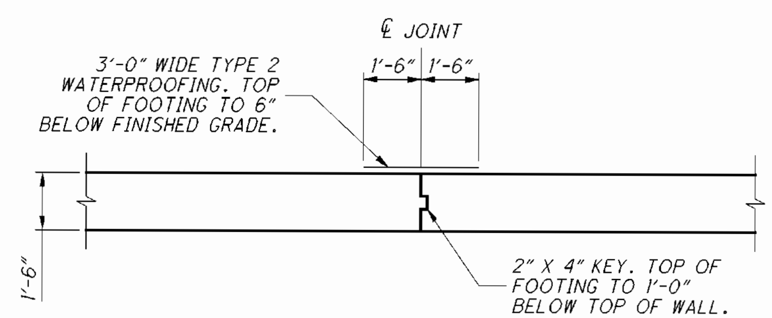
LOOKING UPSTATION
SHOWING REINFORCING

NOTE: LEGS OF "F" BARS AND W403 BARS SHALL BE ON SAME LAYER AS F403 BARS

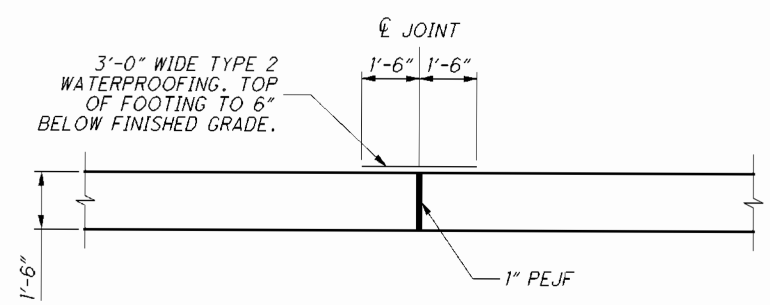
1 MIN. BAR SPLICES
#6 = 2'-11"
#7 = 3'-8"
#8 = 4'-11"

WALL REINFORCEMENT

	E	F	G	H
PANEL 1	F401	W601	W404	W602
PANEL 2	F401	W601	W405	W603
PANEL 3	F402	W601	W406	W604
PANEL 4	F401	W601	W407	W605
PANEL 5	F401	W601	W408	W606
PANEL 6	F401	W601	W409	W607
PANEL 7	F401	W601	W410	W608
PANEL 8	F401	W601	W411	W609
PANEL 9	F401	W701	W412	W702
PANEL 10	F401	W701	W413	W703
PANEL 11	F401	W801	W414	W802
PANEL 12	F401	W801	W415	W803
PANEL 13	F401	W801	W416	W804



CONTRACTION JOINT DETAIL



EXPANSION JOINT DETAIL

REV. BY	DATE	DESCRIPTION
ARA	4/7/2016	1 - REVISION 01

CALCULATED ASP
CHECKED LNB

TYPICAL REINFORCING SECTION & MISC. DETAILS
RETAINING WALL WS1 ALONG LANE W-S
STATION 3243+75.00 TO STATION 3247+25.00

CUY/SUM-271-00.00/14.87
CUY/SUM-480-29.58/00.00
PID No. 80418

...\sheets\80418_WS1WY001.dgn (Sheet) 4/7/2016 4:33:16 PM aadkins

APPENDIX 2:

SOIL PROFILES AND GEOTECHNICAL REPORT RECOMMENDATIONS

Pages 66-153

PROJECT DESCRIPTION

THE PROJECT CONSISTS OF THE RECONSTRUCTION OF A FOUR-SPAN STRUCTURE (MAH-680-0283) CARRYING VESTAL ROAD OVER INTERSTATE 680 (IR-680) IN MAHONING COUNTY IN NORTHWESTERN YOUNGSTOWN, OHIO. THE REPLACEMENT BRIDGE DECK WILL BE CONTINUOUS, STEEL ROLLED BEAMED CONSTRUCTED WITH A REINFORCED CONCRETE DECK. THE NEW BRIDGE WILL BE 40 FT IN WIDTH, 12 FT NARROWER THAN THE EXISTING ONE, BUT THE SAME LENGTH AT 327 FT. THE EXISTING ABUTMENTS WILL BE REUSED, BUT THE PIER FOUNDATIONS WILL BE REPLACED.

HISTORIC RECORDS

THE PRIMARY SOURCE OF EXISTING GEOTECHNICAL DATA USED TO SUPPORT THIS STUDY WERE ODOT'S ARCHIVE OF GEOTECHNICAL EXPLORATION REPORTS (STATE OF OHIO, FALCON, 2012). 1963 DESIGN DRAWINGS FOR THE EXISTING BRIDGE WERE LOCATED AND REVIEWED, AS WELL AS A BORING LOG AND SOIL TEST DATA FOR THE STRUCTURE (BORING S-006-0-63).

GEOLOGY

THE SITE IS LOCATED IN THE KILLBUCK-GLACIATED PITTSBURGH PLATEAU PHYSIOGRAPHIC REGION OF OHIO, ONE OF THE GLACIATED ALLEGHENY PLATEAUS. THIS AREA IS CHARACTERIZED BY GENERALLY FLAT UPLANDS DISSECTED BY STEEP VALLEYS THAT CREATE LOCAL RELIEF ON THE ORDER OF 200 FT. THE HILLTOPS ARE COVERED IN VARIABLE AMOUNTS OF WISCONSINAN-AGE CLAY AND LOAM TILL THAT OVERLIES MISSISSIPPIAN AND PENNSYLVANIANAGE SHALES, SANDSTONES, CONGLOMERATES AND COALS. BEDROCK AT THE BRIDGE SITE IS MAPPED AS PENNSYLVANIAN ALLEGHENY AND POTTSVILLE GROUPS AND AT AN ESTIMATED ELEVATION OF ABOUT 955 FT. THE SITE IS LOCATED WITHIN, AND ON THE EDGE OF THE RIGHT BANK OF, THE GLACIAL VALLEY OF THE MAHONING RIVER WHICH TODAY OCCUPIES ONLY A SMALL PORTION OF ITS VALLEY FLOOR. THE BEDROCK WALLS DROP STEEPLY FROM ABOUT ELEVATION 1,000 FT TO THE FLOOR OF A BURIED VALLEY AT ELEVATION ABOUT 800 FT. SURFICIAL GEOLOGY MAPPING IN THE AREA OF THE SITE SHOWS THAT THE BURIED VALLEY CONTAINS UP TO 30 FT OF SAND AND GRAVEL CAPPED WITH ABOUT 50 FT OF WISCONSINAN-AGE TILL.

RECONNAISSANCE

A FIELD RECONNAISSANCE OF THE MAH-680-0283 BRIDGE SITE WAS CONDUCTED ON NOVEMBER 7, 2012 TO OBSERVE THE EXISTING STRUCTURE. THE FOUR-SPAN STRUCTURE CARRIES VESTAL ROAD OVER IR-680 NEAR THE SR-711 INTERCHANGE. THE FIRST SPAN ACCOMMODATES THE SR-711 OFF RAMP FROM IR-680 SB. IR-680 IS IN CUT; THE REAR ABUTMENT IS SUPPORTED ON A REINFORCED CONCRETE RETAINING WALL AND THE FORWARD ABUTMENT ON A SPILL-THROUGH SLOPE. THE ABUTMENT SUPPORTS APPEARED TO BE PERFORMING WELL FROM A GEOTECHNICAL STANDPOINT WITH NO OBVIOUS SIGNS OF INSTABILITY OR EXCESSIVE SETTLEMENT. THE PIERS ALSO APPEAR TO BE PERFORMING WELL WITH NO APPARENT SIGNS OF GEOTECHNICAL RELATED DISTRESS SUCH AS DIFFERENTIAL SETTLEMENT OR LATERAL DISPLACEMENT. THE PAVEMENT ON VESTAL ROAD IS BADLY DETERIORATED AT BOTH ENDS OF THE BRIDGE. THIS IS MOST LIKELY TO BE FATIGUE CRACKING DUE TO LONG SERVICE RATHER THAN SUBGRADE FAILURE.

SUBSURFACE EXPLORATION

THE SUBSURFACE EXPLORATION FOR THE BRIDGE INCLUDED 3 PIER AND ABUTMENT BORINGS DRILLED TO DEPTHS OF BETWEEN 40 AND 45 FT. DRILLING OCCURED FROM DECEMBER 18, 2012 TO JANUARY 6, 2013 USING A TRUCK-MOUNTED MOBILE B-61 ORV RIG WITH 3.25-INCH DIAMETER HOLLOW STEM AUGERS. SOIL SAMPLES WERE RECOVERED AT INTERVALS OF 2.5 FEET (0-20 FT) AND 5.0 FT (BELOW 20 FT) USING A SPLIT SPOON SAMPLER (AASHTO T-206 "STANDARD METHOD FOR PENETRATION TEST AND SPLIT BARREL SAMPLING OF SOILS.") AND PLACED IN SEALED JARS. THE STANDARD PENETRATION TEST (SPT) WAS CONDUCTED USING AN AUTO-HAMMER THAT HAS BEEN CALIBRATED AS 89% EFFICIENT ON JUNE 30, 2011.

EXPLORATION FINDINGS

THE BORINGS ENCOUNTERED SOIL CONDITIONS CONSISTENT WITH THE GEOLOGICAL MODEL FOR THE SITE, AND WITH THE FINDINGS OF EARLIER EXPLORATIONS. SITE STRATIGRAPHY CONSISTS OF HARD SILT AND CLAY (A-4a) NEAR THE GROUND SURFACE TO AN ELEVATION OF ABOUT 932 FT WHERE A DENSE SAND AND GRAVEL OR SAND LAYER (A-1-b, A-3a) 5-15 FT IN THICKNESS WAS ENCOUNTERED. BELOW THIS WAS ANOTHER HARD LAYER OF SILT/SAND MIXTURE (A-4a), AND BENEATH IT A HARD SILT LAYER. NO FREE GROUND WATER WAS ENCOUNTERED TO THE DEPTHS DRILLED. HOWEVER, THERE IS A NOTABLE INCREASE IN MOISTURE CONTENT OF SAMPLES FROM THE BOTTOM OF TWO OF THE BORINGS SUGGESTING THAT THEY MAY BE SATURATED. OVERALL, MOISTURE CONTENTS ARE VERY LOW (LESS THAN 10%) GIVEN THE FINE GRAINED NATURE OF MANY OF THE SAMPLES. ONLY A SINGLE SAMPLE AT THE SURFACE IN TWO OF THE BORINGS PRODUCED SPT N-VALUES OF LESS THAN 50 BLOWS/FT . ALL THE REMAINDER WERE WELL IN EXCESS OF 50 AND, IN MANY CASES, LIKELY TO BE 100 OR MORE.

SPECIFICATIONS

THIS GEOTECHNICAL EXPLORATION WAS PERFORMED IN ACCORDANCE WITH THE STATE OF OHIO, DEPARTMENT OF TRANSPORTATION, OFFICE OF GEOTECHNICAL ENGINEERING, SPECIFICATIONS FOR GEOTECHNICAL EXPLORATIONS, DATED JANUARY 2012.

AVAILABLE INFORMATION

ALL AVAILABLE SOIL AND BEDROCK INFORMATION THAT CAN BE CONVENIENTLY SHOWN ON THE GEOTECHNICAL EXPLORATION SHEETS HAS BEEN SO REPORTED. ADDITIONAL EXPLORATIONS MAY HAVE BEEN MADE TO STUDY SOME SPECIAL ASPECT OF THE PROJECT. COPIES OF THIS DATA, IF ANY, MAY BE INSPECTED IN THE DISTRICT DEPUTY DIRECTOR'S OFFICE, THE OFFICE OF GEOTECHNICAL ENGINEERING AT 1600 WEST BROAD STREET OR THE OFFICE OF STRUCTURAL ENGINEERING AT 1980 WEST BROAD STREET.

LEGEND

DESCRIPTION	ODOT CLASS	CLASSIFIED MECH.	CLASSIFIED VISUAL
GRAVEL AND/OR STONE FRAGMENTS	*A-1-a*	*1*	*1*
GRAVEL AND/OR STONE FRAGMENTS WITH SAND	*A-1-b*	*1*	*2*
COARSE AND FINE SAND	*A-3a*	*3*	*5*
SANDY SILT	*A-4a*	*8*	*18*
SILT	*A-4b*	*2*	*2*
	TOTAL	*15*	*28*

PAVEMENT OR BASE = X = APPROXIMATE THICKNESS	*VISUAL*
SOD AND TOPSOIL = X = APPROXIMATE THICKNESS	*VISUAL*

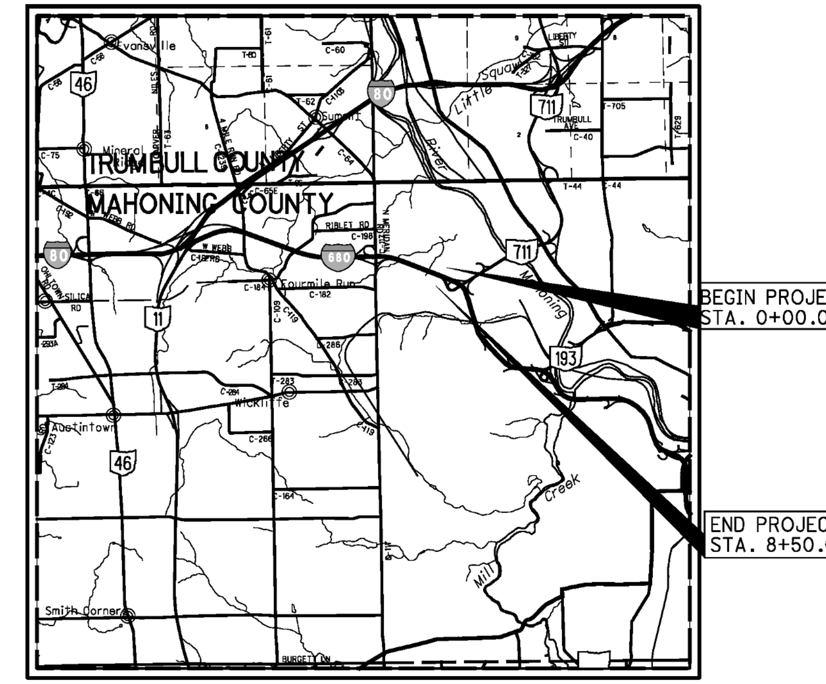
PROJECT BORING LOCATION - PLAN VIEW.

HISTORIC BORING LOCATION - PLAN VIEW.

DRIVE SAMPLE AND/OR ROCK CORE BORING PLOTTED TO VERTICAL SCALE ONLY. HORIZONTAL BAR INDICATES A CHANGE IN STRATIGRAPHY.

WC	INDICATES WATER CONTENT IN PERCENT.
N_{60}	INDICATES STANDARD PENETRATION RESISTANCE NORMALIZED TO 60% DRILL ROD ENERGY RATIO.
X/Y	NUMBER OF BLOWS FOR STANDARD PENETRATION TEST (SPT): X= NUMBER OF BLOWS FOR FIRST 6 INCHES. Y= NUMBER OF BLOWS FOR SECOND 6 INCHES.
W—	INDICATES FREE WATER ELEVATION.
SS	INDICATES A SPLIT SPOON SAMPLE.
NP	INDICATES A NON-PLASTIC SAMPLE.

HISTORIC BORING DESCRIPTIONS	ODOT CLASS	CLASSIFIED MECH.	CLASSIFIED VISUAL
GRAVEL AND/OR STONE FRAGMENTS WITH SAND	*A-1-b*	*3*	-
COARSE AND FINE SAND	*A-3a*	*1*	-
SANDY SILT	*A-4a*	*9*	-
SILT AND CLAY	*A-6a*	*1*	-
	TOTAL	*14*	-



LOCATION MAP

SCALE IN MILES

0 1 2 3 4

N

PARTICLE SIZE DEFINITIONS

12"	3"	2.0 mm	0.42 mm	0.074 mm	0.005 mm	
BOULDERS	COBBLES	GRAVEL	COARSE SAND	FINE SAND	SILT	CLAY
		No. 10 SIEVE	No. 40 SIEVE	No. 200 SIEVE		

RECON. - SE 11/7/12

DRILLING - RW 12/18/12 - 1/16/13

DRAWN - CHE 4/10-18/13

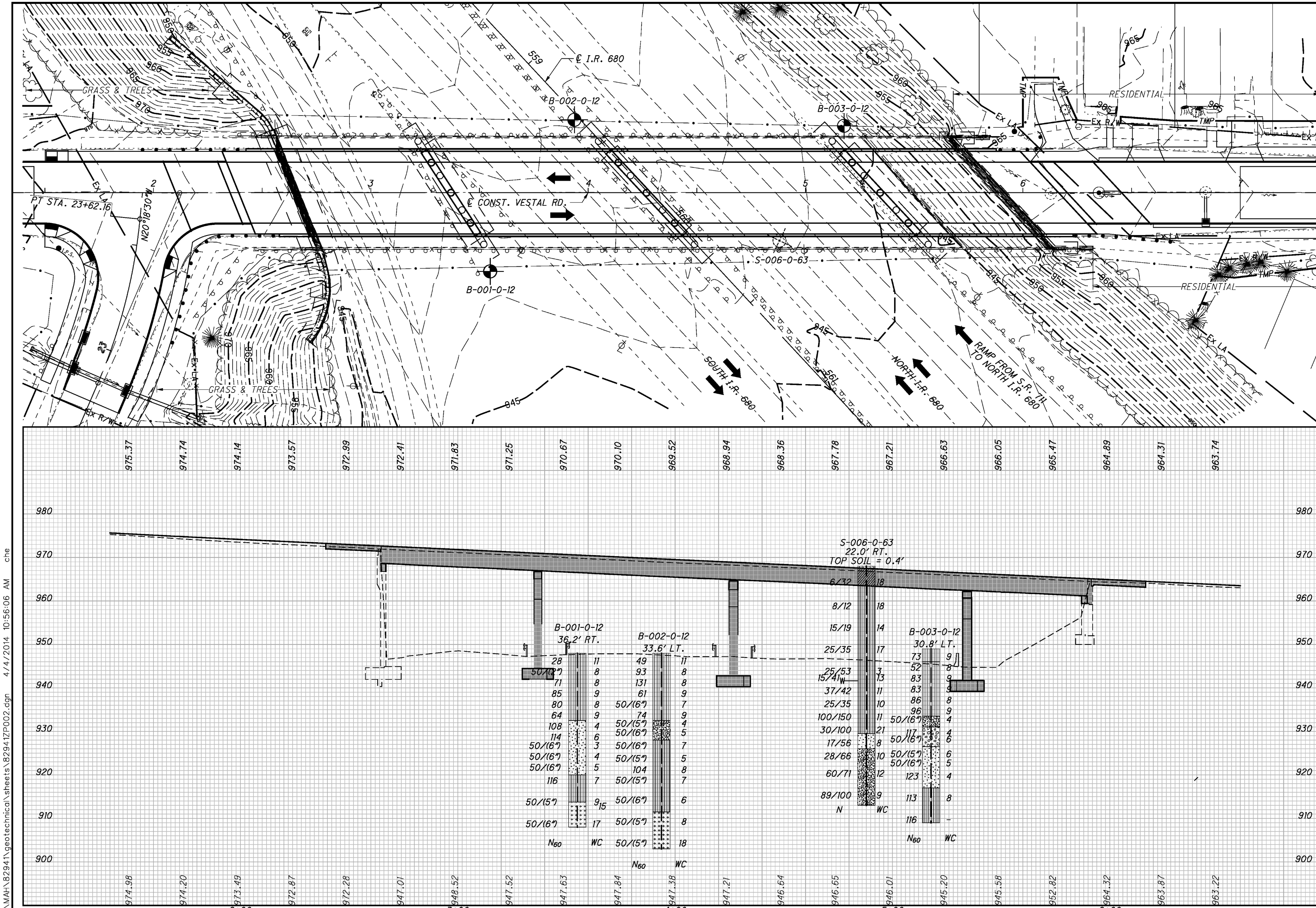
REVIEWED - SE 4/1/14

DESIGN AGENCY
BARR & PREVOST
2800 CORPORATE EXCHANGE DR., STE 240
COLUMBUS, OH 43231
(614) 714-0270 FAX (614) 714-0323

PID NO. 82941

STRUCTURE FOUNDATION EXPLORATION
BRIDGE MAH-680-0283 OVER IR 680

MAH-680-2.83



STRUCTURE FOUNDATION EXPLORATION
BRIDGE NO. MAH-680-0283 OVER IR 680

MAH-680-2.83

HORIZONTAL SCALE IN FEET

DRAWN CMH

CHECKED SE

2 / 6

F:\MAH\82941\geotechnical\sheets\82941ZP002.dgn 4/4/2014 10:56:06 AM che

F:\MAH\82941\geotechnical\sheets\82941ZL001.dgn 4/4/2014 10:56:07 AM che

PROJECT: MAH-680-02.83 TYPE: STRUCTURE PID: 82941 BR ID: START: 12/20/12 END: 1/6/13	DRILLING FIRM / OPERATOR: B&P / R.WEBB SAMPLING FIRM / LOGGER: B&P / P.MCKINLEY DRILLING METHOD: 3.25" HSA SAMPLING METHOD: SPT	DRILL RIG: MOBILE B61 TRUCK HAMMER: DIEDRICH AUTOMATIC CALIBRATION DATE: 6/30/11 ENERGY RATIO (%): 89	STATION / OFFSET: 3+54.97, 36.2 RT ALIGNMENT: CLSVVESTAL ELEVATION: 948.0 (MSL), EOB: 40.0 ft. COORD: 41.119054580, -80.697518289

EXPLORATION ID B-001-0-12 — PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
HARD, GRAY-BROWN, **SANDY SILT**, LITTLE CLAY, TRACE GRAVEL, DAMP	948.0																
			5 8 11	28	89	SS-1	4.5+	-	-	-	-	-	-	-	-	11	A-4a (V)
SS-2 NO RECOVERY			50/2"	-	0	SS-2	-	-	-	-	-	-	-	-	-	8	
@6.0'; SS-3 CHANGES TO GRAY			18 23 25	71	100	SS-3	4.5+	5	10	23	43	19	19	14	5	8	A-4a (5)
			21 27 30	85	100	SS-4	4.5+	-	-	-	-	-	-	-	-	9	A-4a (V)
@11.0'; SS-5 SOME CLAY			21 26 28	80	94	SS-5	4.5+	7	10	14	44	25	21	13	8	8	A-4a (7)
			15 20 23	64	100	SS-6	4.5+	-	-	-	-	-	-	-	-	9	A-4a (V)
VERY DENSE, BROWN WITH GRAY, **COARSE AND FINE SAND**, LITTLE SILT, TRACE CLAY, TRACE GRAVEL, DAMP	932.5																
			14 27 46	108	100	SS-7	-	-	-	-	-	-	-	-	-	4	A-3a (V)
			18 30 47	114	94	SS-8	-	5	10	66	14	5	NP	NP	NP	6	A-3a (0)
			21 50/6"	-	96	SS-9	-	-	-	-	-	-	-	-	-	3	A-3a (V)
			25 50/6"	-	100	SS-10	-	-	-	-	-	-	-	-	-	4	A-3a (V)
			38 50/6"	-	96	SS-11	-	0	2	79	16	3	NP	NP	NP	5	A-3a (0)
VERY DENSE, BROWN, **SANDY SILT**, TRACE CLAY, DAMP	920.0																
			30 35 43	116	94	SS-12	-	-	-	-	-	-	-	-	-	7	A-4a (V)
			17 38 50/5"	-	94	SS-13	-	-	-	-	-	-	-	-	-	9	A-4a (V)
VERY DENSE, BROWN, **SILT**, LITTLE SAND, TRACE CLAY, MOIST	913.8						-	-	-	-	-	-	-	-	-	15	A-4b (V)
			31 42 50/6"	-	91	SS-14	-	0	0	13	80	7	NP	NP	NP	17	A-4b (8)
	908.0	EOB 40															

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. CAVE DEPTH 20.0'.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: SHOVELED SOIL CUTTINGS

STANDARD ODOT SOIL BORING LOG (11 X 17) - OH DOT.GDT - 2/10/14 11:46 - X:\ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\MAH-680-02.83\MAH-680-02.83.GPJ

PROJECT: MAH-680-02.83 TYPE: STRUCTURE PID: 82941 BR ID: START: 12/18/12 END: 12/19/12	DRILLING FIRM / OPERATOR: B&P / R.WEBB SAMPLING FIRM / LOGGER: B&P / P.MCKINLEY DRILLING METHOD: 3.25" HSA SAMPLING METHOD: SPT	DRILL RIG: MOBILE B61 TRUCK HAMMER: DIEDRICH AUTOMATIC CALIBRATION DATE: 6/30/11 ENERGY RATIO (%): 89	STATION / OFFSET: 3+96.64, 33.6 LT ALIGNMENT: CLSVVESTAL ELEVATION: 948.0 (MSL) EOB: 45.0 ft. COORD: 41.119272704, -80.697496045	EXPLORATION ID B-002-0-12 PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 948.0	DEPTHS
HARD, GRAY, **SANDY SILT**, LITTLE CLAY, LITTLE GRAVEL, DAMP	932.5	0 – 15.5
VERY DENSE, BROWN CHANGING TO BROWN AND GRAY, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, DAMP	928.0	15.5 – 20.0
VERY DENSE, BROWN, **SANDY SILT**, TRACE CLAY, DAMP	911.5	20.0 – 36.5
VERY DENSE, BROWN AND GRAY CHANGING TO GRAY, **SILT**, "AND" SAND, TRACE CLAY, DAMP	903.0	36.5 – 45.0 EOB

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
8 11 22	49	78	SS-1	4.5+	-	-	-	-	-	-	-	-	11	A-4a (V)
18 33 30	93	83	SS-2	4.5+	11	9	18	43	19	20	16	4	8	A-4a (5)
15 43 45	131	83	SS-3	4.5+	-	-	-	-	-	-	-	-	8	A-4a (V)
7 18 23	61	83	SS-4	4.5+	-	-	-	-	-	-	-	-	9	A-4a (V)
30 33 50/6"	-	86	SS-5	4.5+	14	12	18	36	20	20	14	6	7	A-4a (4)
8 23 27	74	78	SS-6	4.5+	-	-	-	-	-	-	-	-	9	A-4a (V)
41 50/5"	-	76	SS-7	-	35	30	23	10	2	NP	NP	NP	4	A-1-b (0)
50/6"	-	91	SS-8	-	-	-	-	-	-	-	-	-	5	A-1-b (V)
35 50/6"	-	43	SS-9	-	-	-	-	-	-	-	-	-	7	A-4a (V)
28 37 50/5"	-	53	SS-10	-	-	-	-	-	-	-	-	-	5	A-4a (V)
24 34 36	104	67	SS-11	-	-	-	-	-	-	-	-	-	8	A-4a (V)
16 37 50/5"	-	76	SS-12	-	0	0	59	35	6	NP	NP	NP	7	A-4a (1)
37 50/6"	-	113	SS-13	-	-	-	-	-	-	-	-	-	6	A-4a (V)
36 50/5"	-	82	SS-14	-	0	0	41	50	9	NP	NP	NP	8	A-4b (5)
30 50/5"	-	86	SS-15	-	-	-	-	-	-	-	-	-	18	A-4b (V)

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. CAVE DEPTH 41.0'.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: SHOVELED SOIL CUTTINGS

STANDARD ODOT SOIL BORING LOG (11 X 17) - OH DOT.GDT - 2/10/14 11:46 - X:\ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\MAH-680-02.83\MAH-680-02.83.GPJ

PROJECT: MAH-680-02.83 TYPE: STRUCTURE PID: 82941 BR ID: START: 12/19/12 END: 12/19/12	DRILLING FIRM / OPERATOR: B&P / R.WEBB SAMPLING FIRM / LOGGER: B&P / P.MCKINLEY DRILLING METHOD: 3.25" HSA SAMPLING METHOD: SPT	DRILL RIG: MOBILE B61 TRUCK HAMMER: DIEDRICH AUTOMATIC CALIBRATION DATE: 6/30/11 ENERGY RATIO (%): 89	STATION / OFFSET: 5+17.69, 30.8 LT ALIGNMENT: CLSVVESTAL ELEVATION: 949.0 (MSL), EOB: 40.0 ft. COORD: 41.119407621, -80.697082543	EXPLORATION ID B-003-0-12 PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 949.0	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)
HARD, GRAY AND BROWN, **SANDY SILT**, LITTLE CLAY, LITTLE GRAVEL, CONTAINS FEW ROOTS, DAMP **(POSSIBLE FILL)**		9 21 28	73	89	SS-1	4.5+	-	-	-	-	-	-	-	-	9	A-4a (V)
	946.0															
HARD, GRAY, **SANDY SILT**, LITTLE CLAY, SOME TO LITTLE GRAVEL, DAMP		17 18 17	52	61	SS-2	4.5+	26	10	17	34	13	20	14	6	8	A-4a (2)
		24 28 28	83	100	SS-3	4.5+	-	-	-	-	-	-	-	-	9	A-4a (V)
		14 26 30	83	83	SS-4	4.5+	-	-	-	-	-	-	-	-	9	A-4a (V)
		16 30 28	86	89	SS-5	4.5+	13	11	20	39	17	20	13	7	8	A-4a (4)
		18 28 37	96	94	SS-6	4.5+	-	-	-	-	-	-	-	-	9	A-4a (V)
	933.5															
VERY DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, DAMP		30 50/6"	-	78	SS-7	-	-	-	-	-	-	-	-	-	4	A-1-b (V)
	931.0															
VERY DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, SOME SAND, LITTLE SILT, TRACE CLAY, DAMP		28 40 39	117	89	SS-8	-	57	16	12	11	4	NP	NP	NP	4	A-1-a (0)
		50/6"	-	91	SS-9	-	-	-	-	-	-	-	-	-	6	A-1-a (V)
	926.5															
VERY DENSE, BROWN, **COARSE AND FINE SAND**, LITTLE SILT, TRACE CLAY, DAMP		28 46 50/5"	-	97	SS-10	-	-	-	-	-	-	-	-	-	6	A-3a (V)
		38 50/6"	-	78	SS-11	-	0	1	81	14	4	NP	NP	NP	5	A-3a (0)
		24 34 49	123	89	SS-12	-	-	-	-	-	-	-	-	-	4	A-3a (V)
	917.0															
VERY DENSE, BROWN AND GRAY, **SANDY SILT**, TRACE CLAY, DAMP		26 34 42	113	94	SS-13	-	0	0	48	47	5	NP	NP	NP	8	A-4a (3)
		18 36 42	116	117	SS-14	-	-	-	-	-	-	-	-	-	-	A-4a (V)
	909.0	EOB — 40														

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. CAVE DEPTH NOT RECORDED.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: SHOVELED SOIL CUTTINGS

STANDARD ODOT SOIL BORING LOG (11 X 17) - OH DOT.GDT - 2/10/14 11:51 - X:\ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\MAH-680-02.83\MAH-680-02.83.GPJ

FINAL REPORT
STRUCTURE FOUNDATION EXPLORATION
MAH-680-02.83
RECONSTRUCTION OF BRIDGE MAH-680-0283
MAHONING COUNTY, OHIO
PID#: 82941

For:

McCormick Taylor

445 Hutchinson Ave., Suite 540
Columbus, Ohio 43235

Submitted by:

BARR & PREVOST
ENGINEERING | TESTING | SURVEYING

2800 Corporate Exchange Drive, Suite 240
Columbus, OH 43231
Voice: 614.714.0270
Fax: 614.714.0323

April 2, 2014

FINAL REPORT
STRUCTURE FOUNDATION EXPLORATION
MAH-680-02.83
RECONSTRUCTION OF BRIDGE MAH-680-0283
MAHONING COUNTY, OHIO
PID#: 82941

EXECUTIVE SUMMARY

This report presents the results of a structure foundation exploration for the MAH-680-02.83 project, which consists of reconstruction of a four-span bridge (MAH-680-0283) carrying Vestal Road (Rd) over Interstate 680 (IR-680) in Mahoning County in northwestern Youngstown, Ohio. This exploration included drilling 3 pier borings to characterize structure foundation conditions at the piers, laboratory testing of soil samples and engineering analysis to assess foundation design requirements for the piers.

Subsurface conditions are fairly uniform and consistent with the geological model for the project, consisting of very hard glacial till composed of sandy silt (A-4a), sand and gravel (A-1-6 and A-3a) and silt (A-4b) from the ground surface to 45 feet (ft).

Existing piers are supported on shallow foundations (spread footings), and it is recommended that the replacement be supported on similar foundations.

1. INTRODUCTION

1.1. General

This report presents the results of a structure foundation exploration for the MAH-680-02.83 project, which consists of reconstruction of a four-span bridge (MAH-680-0283) carrying Vestal Rd over IR-680 in Mahoning County in northwestern Youngstown, Ohio (Exhibit 1).

The replacement bridge deck will be continuous, steel rolled beamed constructed with a reinforced concrete deck. Its width and roadway will be narrower than the existing 40 ft versus 52 ft 4 inches, but the length will be the same: 327 ft. Existing piers are supported on shallow spread footings. It is planned to reuse the existing abutments, but to replace the existing piers.

The exploration was conducted in general accordance with Barr & Prevost Inc.'s (B&P) proposal to

McCormick Taylor, dated August 20, 2012. The bridge rehabilitation is being designed using the Load Factor Design (LFD) method as set forth in the American Association of State Highway and Transportation Officials (AASHTO) Publication "AASHTO Standard Specifications for Highway Bridges 17th Edition" (AASHTO, 2002) and ODOT Bridge Design Manual (BDM), (2004). The exploration was conducted in general accordance with the provisions of ODOT's "Specifications for Geotechnical Explorations" (SGE) (ODOT, 2012).

The geotechnical exploration included drilling 3 pier borings to characterize structure foundation conditions, laboratory testing of soil samples, and engineering analysis to assess foundation design requirements for the piers.

1.2. Design Basis

As indicated above, the bridge rehabilitation will reuse the existing abutments and replace only the deck and piers. This exploration addresses only the design of foundations for the three piers.

2. GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1. Physiography

The site is located in the Killbuck-Glaciated Pittsburgh Plateau Physiographic Region of Ohio, one of the Glaciated Allegheny Plateaus (Brockman, 1998). This area is characterized by generally flat uplands dissected by steep valleys that create local relief on the order of 200 ft. The hilltops are covered in variable amounts of Wisconsinan-age clay and loam till that overlies Mississippian and Pennsylvanian-age shales, sandstones, conglomerates and coals.

2.2. Geology

Bedrock at the bridge site is mapped as Pennsylvanian Allegheny and Pottsville Groups and at an estimated elevation of ~ 955 ft (Slucher, 1996 and Slucher et.al., 1996). With a natural ground surface elevation of ~970-975 ft, the bedrock would be expected within 25 ft of the surface. Nearby historical borings suggest that this depth is, in fact greater than 55 ft.

The site is located within, and on the edge of the right bank of, the glacial valley of the Mahoning River which today occupies only a small portion of its valley floor. The bedrock walls drop steeply from about elevation 1,000 ft to the floor of a buried valley at elevation ~800 ft. Varying amounts of glacial outwash and till mask this prominent bedrock structure. Recent surficial geology mapping in the area of the site shows that the buried valley contains up to 30 ft of sand and gravel capped with about 50 ft of Wisconsinan-age till. A nearby historical boring revealed conditions consistent with this profile: about 5 ft of clay (A-6a) overlying 30 ft of hard till (sandy silt - A-4a), in turn overlying 20 ft of dense sand and gravel (A-1-b).

2.3. Soils

The USDA Natural Resource Conservation Service has mapped the soils at the project location as Ellsworth-Urban land complex and is rated very limited for local roads and streets due to low strength, shrink-swell, frost action and depth to saturated zone (U.S. Department of Agriculture, 2012).

2.4. Seismicity

Earthquake hazard analysis in this part of the country is dominated by proximity to the New Madrid Fault Zone (NMFZ) approximately 570 miles to the southwest. Possible future movements along this fault could generate earthquakes of magnitude 7.0-8.0 with a recurrence period of 500-1,500 years (USGS, 2008). The resulting ground motion would be experienced over a wide area, with most of Ohio located within the possible zone of influence. A cluster of earthquake epicenters representing events of lesser magnitude is located due north near the shore of Lake Erie (ODNR, 2012[(1)]), but no causative faults have been mapped to date. One event, with a magnitude in excess of 5.0, and 11 events with magnitudes less than 5.0, were recorded in the Youngstown area between 2011 and 2012. While these may have been caused by fracking operations from shale gas and liquids extraction, these represent another potential earthquake source area that is contributory to overall seismic risk.

2.5. Hydrology/Hydrogeology

The local hydrogeologic regime is dominated by the valley of the Mahoning River (0.6 mile to the east), which flows southward to the Beaver River in Pennsylvania before becoming tributary to the Ohio River. The project location is outside the area designated by FEMA (U.S. Department of Homeland Security, 2009) as having a 1% annual chance flood (100-year flood).

The elevation of the Mahoning River is likely to be representative of the regional groundwater system that, near the site, is at about elevation 840 ft (U.S. Department of the Interior, 1994). The surficial glacial drift on the uplands and the shales comprising the Cuyahoga Formation are poor water-bearing materials and typically only support modest ground water flows in fractures and joints.

Water in the glacial outwash deposits may have been viable as a ground water resource, depending on quality and demand. A well drilled nearby (800 ft south) in 1954 penetrated 165 ft of brown sand and sandstone before encountering shale and water at about 173 ft (ODNR, 2012[(2)]). The well was advanced 175 ft into shale.

Groundwater is not likely to be encountered in significant amounts above the level of the Mahoning River unless it is contained in isolated lenses of sand or gravel within the glacial till.

2.6. Mining and Oil/Gas Production

The nearest mapped abandoned underground mine to the bridge site is over 4,000 ft southwest. No evidence of mining, or abandoned mines, at or beneath the bridge site was found (ODNR, 2012[(3)]). While numerous oil and gas wells are located throughout the Youngstown area, none are mapped at or adjacent to the bridge site by the Ohio Geological Survey (ODNR, 2012[(4)]).

2.7. Site Reconnaissance

A field reconnaissance of the MAH-680-0283 bridge site was conducted on November 7, 2012 to observe the existing structure. The four-span structure carries Vestal Rd over IR-680 near the SR-711 interchange. The first span is required to accommodate the SR-711 off ramp from IR-680 SB (Photograph 1).

Photograph 1: Existing Four-Span Structure Looking East

IR-680 is in cut; the rear abutment is supported on a reinforced concrete retaining wall and the forward abutment on a spill-through slope (Photographs 2 & 3).

Photograph 2: Reinforced Concrete Retaining Wall at Rear Abutment

Photograph 3: Spill-through Slope at Forward Abutment

The abutment supports appeared to be performing well from a geotechnical standpoint with no obvious signs of instability or excessive settlement. The piers also appear to be performing well with no apparent signs of geotechnical related distress such as differential settlement or lateral displacement (Photograph 1).

The pavement on Vestal Rd is badly deteriorated at both ends of the bridge. This is most likely to be fatigue cracking due to long service rather than subgrade failure.

3. EXPLORATION

3.1. Historical Records

The primary source of existing geotechnical data used to support this study were ODOT's archive of geotechnical exploration reports (State of Ohio, FALCON, 2012). 1963 design drawings for the existing bridge were located and reviewed, as well as a boring log and soil test data for the structure (Boring S-006-0-63), and a nearby roadway boring. These were drilled to support the design of projects MAH-18-15.50 and Bridge MAH-18-1629.

3.2. Exploration Program

The subsurface exploration included 3 borings drilled to depths of 40-45 ft below ground surface (bgs). Locations and elevations are shown on Table 1. B&P drilled the borings between December 18, 2012 and January 6, 2013 using a truck-mounted, Mobile B-61 ORV rig with 3.25-inch diameter hollow stem augers. Soil samples were recovered at intervals of 2.5 ft (0-20 ft) and 5.0 ft (below 20 ft) using a split spoon sampler (AASHTO T-206 “Standard Method for Penetration Test and Split Barrel Sampling of Soils”) and placed in sealed jars. The standard penetration test (SPT) was conducted using an auto-hammer that has been calibrated as 89% efficient.

Field boring logs were prepared by the driller, lithological description, and standard penetration test results recorded as blows per 6-inch increment of penetration. Groundwater observations were recorded during the investigation where encountered. Field penetrometer testing was conducted on a majority of cohesive SPT samples prior to removal from the sampler. Each boring was backfilled with either soil cuttings or bentonite and cuttings as indicated on the logs of borings.

As-drilled boring locations are shown on Exhibit 1, and summaries of the drilling information are presented in Table 1. Logs of the borings are presented in Appendix A. Sheets comprising the Structure Foundation Exploration are presented in Appendix C and depict boring locations and logs.

Table 1: Boring Summary

Boring Number	Boring Location (Station, Offset)	Approximate Surface Elevation (NGDV-ft)	Depth (ft)	Bottom of Hole Elevation (ft)	Structure
B-001-0-12	3+54.97, 36.2 RT	948.0	40	908.0	Pier 1
B-002-0-12	3+96.64, 33.6 LT	948.0	45	903.0	Pier 2
B-003-0-12	5+17.69, 30.8 LT	949.0	40	909.0	Pier 3

3.3. Laboratory Testing

Data from the laboratory-testing program were incorporated onto the logs of borings (Appendix A). Soil samples are retained at the laboratory for 60 days following report submittal, after which time they will be discarded.

3.3.1. *Classification Testing*

Natural moisture content tests were performed on all soil samples. Representative soil samples were selected for index property (Atterberg Limits) and gradation testing for classification purposes. The results are presented on the log of borings.

Mechanical soil classification (Plastic Limit, Liquid Limit and gradation testing) was conducted on approximately 30% of the samples.

Final classification of soil strata in accordance with AASHTO M-145 "Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes," as modified by ODOT "Classification of Soils" was made once laboratory test results became available.

3.3.2. *Standard Penetration Test Results*

Standard Penetration Tests and split-barrel (commonly known as split-spoon) sampling of soils were performed at 2.5 or 5.0-ft intervals in all borings. The hammer corrected SPT N-values were all very high, with values typically in excess of 50 [blows per foot (bpf)]. The SPT N-values and laboratory test results, which include natural moisture content, fines (silt and clay) content, and engineering classifications, are presented on the logs of borings (Appendix A). N-values were adjusted to account for the high efficiency (89%) hammer used in the test. The resulting N_{60} values are also shown on the logs of borings.

4. FINDINGS

4.1. General

The borings encountered soil conditions consistent with the geological model for the site, and with the findings of the 1963 exploration. They produced samples at discrete locations within the subsurface environment beneath the site, and test data, whether insitu or exsitu, are representative only of those locations. To evaluate the general characteristics of the subsurface units and to develop generalized geotechnical design parameters, the data produced during field and laboratory testing were evaluated by observation supported by reference to published engineering correlations. The following description of the subsurface conditions is based on interpretation of the current and historical field explorations, the results of laboratory testing, and consideration of the geological history of the site.

4.2. Stratigraphy

A single boring drilled as part of the geotechnical exploration for the existing structure (Michael Baker Jr., ~1963) shows an existing ground surface of ~968 ft below which ~21 ft of cut was to be made. Soils in the base of the cut were silt and sand mixtures (A-4a) and consistently hard (N > 50) to an elevation of 929 ft below which dense sand and gravel was encountered (A-1-b). The results of the current exploration are consistent with this stratigraphy. Hard silt and clay (A-4a) was found near the ground surface to an elevation of ~932 ft where a dense sand and gravel or sand layer (A-1-b, A-3a) 5-15 ft in thickness was encountered. Below this was another hard layer of silt/sand mixture (A-4a), and beneath it a hard silt layer. No free groundwater was encountered to the depths drilled. However, there is a notable increase in moisture content of samples from the bottom of two of the borings suggesting that they may be saturated. Overall, moisture contents are very low (less than 10%) given the fine grained nature of many of the samples.

Only a singe sample at the surface in two of the borings produced SPT N-values of less than 50 bpf. All the remainder were well in excess of 50 and, in many cases, likely to be 100 or more. The test protocol requires the test to be terminated at a blow count of 50 for an individual 6-inch increment, or a total of 100.

4.3. Soils

For purpose of analysis the soil conditions have been generalized into the stratigraphic column described in Table 2.

Table 2: Generalized Geotechnical Profile

Stratum	Material	ODOT Classification	Elevation (base of stratum-ft)
1	sandy silt	A-4a	932
2	gravel and sand	A-1-b	927
3	sand	A-3a	920
4	sandy silt	A-4a	910
5	silt	A-4b	903

Properties of the soils for use in engineering analysis are presented in Table 3 and have been estimated based on their index properties, N_{60} and HP results using correlations provided in published engineering manuals, research reports and guidance documents, as indicated.

Table 3: Soils Physical Properties

Soil Type	Description	Property	Value	Source
1	sandy silt	N_{60}	80	App A
		S_u	4,000 psf	App A
		γ(bulk)	130 pcf	estimate
		c'	0	-
		φ'	36º	NHI-06-088 5-22/23
2	gravel and sand	N_{60}	100	App A
		S_u	0 psf	-
		γ(bulk)	130 pcf	estimate
		c'	0	-
		φ'	45º	NHI-06-088 5-22/23
3	sand	N_{60}	100	App A
		S_u	0 psf	-
		γ(bulk)	130 pcf	Estimate
		c'	0	-
		φ'	40º	NHI-06-088 5-22/23
4 and 5	sandy silt, silt	N_{60}	100	App A
		S_u	4,000 psf	App A
		γ(bulk)	130 pcf	Estimate
		c'	0	-
		φ'	36º	NHI-06-088 5-22/23

The existing piers are founded at an elevation of 959.2-942.5 ft, placing them within hard glacial till (A-4a). The properties of glacially deposited materials can vary widely depending on the mode of deposition. This appears to be a basal till, laid down under an ice sheet and, as a result, is significantly over-consolidated. Its strength properties have been estimated using field test results and published correlations with various indicator parameters including N_{60} (blow counts), void ratio and hand penetrometer tests.

The undrained shear strength has been correlated with N_{60} values for a variety of basal tills (ICE, 2012). The results of laboratory testing suggest a factor of 4.1-7.0 x N_{60} (measured in kPa: 1 kPa = 21 psf), although it is claimed that experience in the field justifies a higher value, and a factor of 9 is recommended for foundation design. This method would yield shear strength values of 6,720-18,900 psf for the tills at the project site.

The hand penetrometer is of limited value for this material as it has a maximum scale value of 4.5 tsf (compressive strength) that correlates with a 4,500 psf shear strength. Values below this are useful data points, but in hard till the great majority of points are simply reported as '>4.5 (tsf)'. Similarly, the SGE-based correlations suggest a minimum of 4,000 psf if $N_{60} >= 30$, unless hand penetrometer results suggest a lower value.

While it is very likely that the average shear strength of the hard glacial till is well in excess of 4,000 psf, this value was selected as conservative for use in design.

4.4. Bedrock

No bedrock was encountered during drilling.

5. ANALYSIS AND RECOMMENDATIONS

These recommendations are based on a review of existing data, field and laboratory testing results, and engineering analysis and judgment. If any element of the project evolves to be significantly different than is described herein, these recommendations should be reviewed by a geotechnical engineer to assess their continuing validity before they are incorporated into the design.

The existing piers are supported on shallow foundations (spread footings) founded at a depth in the range of 5.5-9 ft below ground surface. These are ~ 6.5 ft-7.5 ft in width and ~ 64-73 ft long and designed for a maximum bearing pressure of 2.9 tsf.

The proposed structure will be approximately 20% narrower than the existing one indicating a commensurate reduction in dead load. Given the apparent satisfactory performance of the existing foundations, it is recommended that similar foundations be used. The following analysis considers the bearing resistance of shallow foundations, using the smaller existing footing as a model (Pier 1).

5.1. Bearing Resistance

The resistance of shallow foundations to vertical loading at the strength limit state was assessed using the procedure described in AASHTO 2002 and BDM. The foundation soils were modeled first as a cohesive material (high shear strength, low friction) and then as a non-cohesive soil (high friction, low cohesion). As indicated above, the properties of hard glacial till are difficult to quantify accurately, and a parametric approach provides a bounding analysis, the results of which are summarized in Table 4. The spreadsheet output is provided in Appendix B.

Table 4: Results of Bearing Resistance Analysis

Soil/Parameter	Shear Strength (ksf)	Friction angle (º)	Ultimate Capacity (ksf)	Allowable Capacity (ksf)
high shear strength	4.0	0	21.7	7.2
high friction	0	36	51.8	17.3

The results indicate a significant spread between the two assumed material types. For preliminary engineering, an allowable bearing capacity of 7 ksf is recommended. It should be noted that this analysis assumes no load eccentricity. Further, the results are dependent on the foundation size and depth of embedment. When the preliminary loadings have been determined, a more detailed analysis of the foundation bearing resistance should be made.

5.2. Settlement

The glacial till upon which the foundations bear is highly over-consolidated as indicated by the consistent high strength. Compressibility is correspondingly low. The area occupied by the piers was cut about 20 ft during construction further reducing the effective stress on the soils and, finally, the bridge has been in place for about 50 years giving ample opportunity for any settlement of the existing foundations to occur.

AASHTO provides a presumptive bearing resistance for shallow foundations constructed on dense glacial till at the service limit state of 16-24 ksf (C10.6.2.6.1-1). It is recommended that a factored resistance of 20 ksf be used (resistance factor for service limit state = 1.0). This presumptive value considers the possibility of 1 inch of settlement, most of which will occur during construction.

5.3. Seismic Load Evaluations and Liquefaction Potential

The seismic hazard at the site has been characterized in terms of an acceleration response spectrum using parameters provided by AASHTO that are believed to be representative of a 1 in 1,000 year return period event (7% probability of being exceeded in approximately 75 years). Variables used in the development of the response spectrum are found on maps provided in AASHTO, 2002, Division 1A, Section 3, Figure 3.3A, Section 3.10 (Figures 3.10.2.1-1, -2 and -3) or computed on a site-specific basis using USGS, 2008:

Peak Ground Acceleration (PGA)	0.039 g
Short term (0.2 second) Spectral Acceleration Coefficient (S_s)	0.085 g
Long term (1 second) Spectral Acceleration Coefficient (S_l)	0.031 g

and tabulated Site Class Definitions (Table 3.10.3.1-1):

Site Class	C

based on predominance of hard glacial till as the subsurface material.

These parameters lead to tabulated site factors that are used to compute the response spectrum (Tables 3.10.3.2-1, -2 and -3):

$$F_{PGA} = 1.2$$
$$F_a = 1.2$$
$$F_v = 1.7$$

The resulting response spectrum produces a spectral acceleration at a period of 1 second (S_{Dl}) of 0.053 that meets the criteria for a bridge in Seismic Performance Zone 1 (Table 3.10.6-1), the least restrictive.

Liquefaction of saturated, loose cohesionless soil may occur in response to vibration such as occurs during earthquakes, however, cohesionless soils present beneath the site are both dense and unsaturated.

6. QUALIFICATIONS

This investigation was performed in accordance with accepted geotechnical engineering practice for the purpose of characterizing the subsurface conditions at the MAH-680-0283 bridge rehabilitation site, performing geotechnical engineering analyses, and providing recommendations for foundation design of bridge piers only. The analyses and recommendations submitted in this report are based upon the data obtained from the borings drilled at the locations shown on Exhibit 1, and as presented on the Logs of Borings (Appendix A). This report does not reflect any variations that may occur between the borings or elsewhere on the site, or variations whose nature and extent may not become evident until a later stage of construction. In the event that any changes in the nature, design or location of the proposed piers are made, the conclusions and recommendations contained in this report should not be considered valid until they are reviewed, and the conclusions and recommendations have been modified or verified in writing by a geotechnical engineer.

It has been a pleasure to be of service to McCormick Taylor in performing this geotechnical exploration for the MAH-680-02.83 project.

Respectfully Submitted,

Barr & Prevost

STATE OF OHIO
STUART
EDWARDS
E-47218
REGISTERED
PROFESSIONL ENGINEER

4/2/14

Enoch Chipukaizer
Principal

Stuart Edwards, P.E.
Geotechnical Engineer

SUPPLEMENTAL GEOTECHNICAL ANALYSIS
CUY-77-14.35 BRIDGE REPLACEMENT
CLEVELAND, OHIO

S&ME Project No. 1179-15-006

Prepared for:
Richland Engineering, Limited
29 Park Street North
Mansfield, Ohio 44902

Prepared by:
S&ME, Inc.
Cleveland, Ohio

Revised May 16, 2016

sufficiently saturated that additional moisture conditioning is impractical, the material should either be dried back and re-compacted or be wasted. Therefore, it is recommended that moisture conditioning only be performed when extended periods of suitable weather are anticipated, and that only the amount of borrow soil be exposed that may be moisture conditioned and properly compacted during suitable weather periods.

A few samples had moisture contents more than 3% above their estimated optimum moisture contents. As such, if on-site soils will be reused as compacted fill, the contractor should be prepared to moisture condition (dry back) those soil prior to re-compaction.

Groundwater Considerations for Roadway Construction
Based upon observations made during our investigation, significant groundwater problems are not anticipated in most areas during earthwork operations associated with the roadway construction. It should be possible to remove limited volumes of surface water with sump pumps in sump pits. Note that groundwater levels could vary with seasonal fluctuations in precipitation.

Roadway subgrades should be graded to prevent surface runoff from pooling on any cohesive soils during construction as exposure of cohesive soils to moisture will result in a decrease in strength and an increase in compressibility. Soil softened by standing water or disturbed by construction activities should be removed before proceeding with construction.

As previously discussed, the standing water adjacent to any new or widened pavement subgrade areas should be removed prior to earthwork activities and any underlying soils that have been softened or loosened by standing water should be remediated.

4.0 CIP RETAINING WALL FOUNDATION ANALYSIS

The following subsurface information was used to perform the LRFD analysis for the spread footings supporting the four (4) retaining walls:

Wall 1 (IR 77 Sta. 72+19.25 to Sta. 74+20.54) – Boring BB-104

Approximately 16 inches of asphalt and concrete pavement was encountered at the ground surface in Boring BB-104. Material visually identified as fill was encountered below the pavement to a depth of 25.5 feet and consisted of medium-dense to hard SANDY SILT (A-4a) and dense COARSE AND FINE SAND (A-3a). Natural soils were encountered below the fill to the termination depth of 40.0 feet and consisted of medium-dense FINE SAND (A-3). Groundwater seepage was not encountered and the boring appeared to be dry at the completion of drilling.

Wall 2 (IR 77 Sta. 74+22.44 to Sta. 75+22.06) – Boring BB-106

Approximately 14 inches of asphalt and concrete pavement was encountered at the ground surface in Boring BB-106. Material visually identified as fill was encountered below the pavement to a depth of 32.0 feet and consisted of medium-dense to very-dense

GRAVEL WITH SAND (A-1-b) and dense COARSE AND FINE SAND (A-3a). Natural soils were encountered below the fill to the termination depth of 90.0 feet and consisted of medium-dense FINE SAND (A-3), medium-dense to very-dense COARSE AND FINE SAND (A-3a), stiff to very-stiff SILT (A-4b), and medium-stiff to stiff SITLY-CLAY (A-6b). Groundwater seepage was encountered at a depth of 53.5 feet and groundwater was encountered at a depth of 58.5 feet.

Wall 3 (IR 77 Sta. 79+32.94 to Sta. 80+32.56) – Boring BB-111

Approximately 16 inches of asphalt pavement was encountered at the ground surface in Boring BB-111. Material visually identified as fill was encountered below the pavement to a depth of 32.0 feet and consisted of dense SANDY-SILT (A-4a), dense SILT (A-4b), medium-dense FINE SAND (A-3) and dense to very-dense COARSE AND FINE SAND (A-3a). Natural soils were encountered below the fill to the termination depth of 90.0 feet and consisted of medium-dense to very-dense FINE SAND (A-3), medium-stiff to dense SANDY SILT (A-4a), and dense SILT (A-4b). Groundwater seepage was encountered at a depth of 58.5 feet and water was measured at a depth of 68.5 feet at the completion of drilling.

Wall 4 (IR 77 Sta. 80+34.56 to Sta. 82+71.00) – Boring BB-112

Approximately 15 inches of asphalt and concrete pavement was encountered at the ground surface in Boring BB-112. Material visually identified as fill was encountered below the pavement to a depth of 23.0 feet and consisted of medium-dense SANDY-SILT (A-4a) and dense GRAVEL WITH SAND (A-1-b). Natural soils were encountered below the fill to the termination depth of 40.0 feet and consisted of soft to medium-stiff SILT AND CLAY (A-6a), medium-dense to dense COARSE AND FINE SAND (A-3a), and dense FINE SAND (A-3). No groundwater seepage was encountered and the boring appeared to be dry at the completion of drilling.

Please note that four additional borings (BB-105, BB-107, BB-110, and BB-113) were performed in the general vicinity of the wall alignments. Please refer to the 2006 report for the boring logs and refer to the 2008 Stage 2 plans for the boring locations with respect to the proposed construction.

Based on the available information for the proposed CIP Retaining Walls from the Retaining Wall plans, dated October 20, 2015, and our analysis of the conditions encountered in the vicinity of the proposed construction, the following recommendations have been developed based on the AASHTO LRFD Bridge Design Specifications, Sixth Edition (AASHTO LRFD). Soil parameters that were employed in the engineering analyses are included on the calculation output shown as Plates 1 through 9 in Appendix B. External stability calculations performed include analyses of Strength I Event loading for sliding (along the base of the footing), eccentricity (overturning) and bearing resistance. It was not within S&ME's scope of work to perform Service I Event analyses, including overall (global) stability and settlement. Collision/impact loading (Extreme II Event) was not included in the analyses as it was not provided to S&ME at the time of this submittal. A description of each of these analyses and a summary of our findings is presented in the following sections.

Sliding

The calculations for the sliding coefficient of friction identified in the last column of Table 2 are shown on Plates 6 through 9 of Appendix B. These calculations use the formulas in section 10.6.3.4 AASHTO LRFD Bridge Design Specifications (6th Edition).

Eccentricity (Overturning)

The results of the eccentricity analyses for the CIP Wall indicate that a footing width sufficient to satisfy the sliding stability analysis for each panel will maintain the eccentricity within the middle two-thirds of the base wall footing width in accordance with AASHTO criteria.

Factored Bearing Resistance (Service and Strength Limit States)

Applied bearing pressures on the foundations of the CIP walls for the wall geometry provided on the REL Retaining Wall plans, dated October 20, 2015, were calculated based on the estimated unit weight of a drained backfill soil of 120 pcf being placed and compacted behind the wall. Since the CIP walls have variable heights at various locations along the walls, the factored bearing pressure changes between locations.

Table 2 summarizes the preliminary recommended nominal and factored bearing resistances (q_n and q_R) for the service and strength limit states for spread foundations bearing on the natural dense granular soil. In order to achieve the recommended factored bearing resistances provided in Table 2, the bearing surfaces should also be carefully cleaned prior to placement of concrete. Service limit factored bearing resistance values were obtained from Table C10.6.2.6.1-1 of the AASHTO LRFD Bridge Design Specifications (6th Edition). The calculations for the nominal and factored bearing resistance indentified in Table 2 are shown on Plates 1 through 5 of Appendix B.

Table 2: Summary of LRFD CIP Retaining Wall Foundation Design Parameters

Wall Number	Station	Reference Boring	Nominal Bearing Resistance q_n (ksf)	Factored Bearing Resistance q_r (ksf)		Sliding Coefficient of Friction
				Service Limit	Strength Limit	
1	73+72	BB-104	58.1	6.0	26.2	0.67
2	74+71	BB-106	50.2	6.0	22.6	0.67
3	79+83	BB-111	55.1	6.0	24.8	0.67
4	80+51	BB-112	55.8	6.0	25.1	0.67
4	81+55	BB-112	47.0	6.0	21.2	0.67

4.1 Lateral Earth Pressures

The proposed CIP wall must be designed to withstand lateral earth pressures, as well as hydrostatic pressures, that may develop behind the structure. The magnitude of the lateral earth pressures varies on the basis of soil type, permissible wall movement, and the configuration of the backfill.

To minimize lateral earth pressures, the zone behind the CIP wall should be backfilled with granular soil, and the backfill should be effectively drained. For effective drainage, a zone of free-draining gravel (similar to ODOT Item 703.16.C.5 Granular Material Type E, No. 57 stone) should be used directly behind the wall for a minimum thickness of 24 inches. This drainage area should be separated from the adjacent backfill by a filter fabric which will prevent the infiltration of fine material into the free-draining gravel. Other alternative products may also be available for use in lieu of the zone of gravel. These products must also be protected from clogging of fine particles from the adjacent backfill. This granular zone (or alternative product) should drain to either weepholes or a pipe, so that hydrostatic pressures do not develop against the wall.

The type of backfill beyond the free-draining granular zone (or material), however, will govern the magnitude of the pressure to be used for structural design. Pressures of a relatively low magnitude will be developed by the use of granular backfill, whereas a cohesive (clay) backfill will result in the development of much higher pressures.

It is recommended that granular backfill be used behind the CIP wall. The backfill should be placed in a wedge formed by the back of the structure and a line rising from the base of the wall foundation at an angle no greater than 60^o from horizontal. Over-compaction in areas directly behind the walls should be avoided, as this might cause damage to the structure.

If proper drainage is provided and compacted granular backfill is provided as described above, an equivalent fluid unit weight of 35 lb/ft^3 (pcf) may be used if a wall movement equivalent to 0.25 percent of the height of the abutment or wingwall (H) is allowed to occur. Such movement is considered sufficient to mobilize an active earth pressure condition. In this case, the resultant lateral force should be taken as acting at 0.33H (AASHTO LRFD Article 3.11.5). If this movement is not anticipated or cannot occur, it is recommended that an “at-rest” equivalent fluid unit weight of 55 pcf be used.

Compacted cohesive materials tend alternatively to shrink, expand and creep over periods of time and create significant lateral pressures on any adjacent structures. Cohesive materials also require a greater amount of movement to mobilize an active earth pressure condition. For these reasons, we do not recommend using cohesive backfill behind the retaining walls.

The structures must also be designed to withstand the surcharge effect of traffic in addition to the vertical load resulting from the weight of any fill and pavement to be placed over the structures. To estimate vertical loading, a total unit weight of 125 pcf and 135 pcf may be used for compacted granular and cohesive soil, respectively.

4.2 General Construction and Groundwater Recommendations

Based on observed groundwater levels encountered during drilling, and groundwater measurements obtained at the end of drilling, it is not anticipated that any excavations for the abutments or retaining wall foundations will likely encounter significant groundwater seepage. However, excavations for the pier foundations may require minor dewatering

S&ME ENGINEERING INTEGRITY.

Version 2.0 (7/7/15)

Project No	1179-15-006		Sheet	1	of	1
Client	Richland Engineering	**Calc. By**	KAH	**Date**	3/31/16	
Project	CUY-77-14.35 Bridge Replacement	**Check By**	EAA	**Date**	4/1/16	
Desc.	Retaining Wall Footing					
	Wall 1, Sta 73+72					

LRFD BEARING RESISTANCE CALCULATION

SOIL PARAMETERS

Structure Location	Boring ID	Soil Layer	Depth (ft)	Description	SPT N (lb/ft)	D_w (ft)	γ_m (pcf)	w_n (%)	Φ (deg.)	C (psf)
STA 73+72	BB-104	1	11	Coarse and Fine Sand	43	57	135	7	34	0

FOOTING / BEARING RESISTANCE COEFFICIENTS

Structure Location	D_f (ft)	B (ft)	L (ft)	Nc (1)	Nq (1)	Nγ (1)	Sc (2)	Sq (2)	Sγ (2)	Dq (3)	Cwq (4)	Cwγ (4)
STA 73+72	5.75	12.5	31.1	42.20	29.40	41.10	1.280	1.271	0.839	1.0	1.0	1.0

NOMINAL BEARING RESISTANCE

Structure Location	q_N (ksf)
STA 73+72	58.1

$$q_N = cN_c s_c i_c + \gamma D_f N_q s_q d_q i_q C_{wq} + \frac{1}{2}\gamma B N_\gamma s_\gamma i_\gamma C_{w\gamma}$$

BEARING RESISTANCE FACTORS

Limit State	Resistance Factor	
Service	1.0	Article 10.5.5.1
Strength	0.5	Table 10.5.5.2.2-1 (cohesive)
Strength	0.45	Table 10.5.5.2.2-1 (non-cohesive)

FACTORED BEARING RESISTANCE

Limit State	q_R (ksf) Footing	
Service	6.0	Table C10.6.2.6.1-1
Strength	26.2	

REFERENCES

AASHTO LRFD Bridge Design Specifications, 6th Edition, Section 10: Foundations.

1. Bearing Capacity Factors Nc, Nq, and Nγ obtained from Table 10.6.3.1.2a-1.
2. Shape Correction Factors Sc, Sq, and Sγ obtained from Table 10.6.3.1.2a-3.
3. Depth Correction Factor Dq obtained from Table 10.6.3.1.2a-4.
4. Groundwater Correction Coefficients Cwq and Cwγ obtained from Table 10.6.3.1.2a-2.

S&ME ENGINEERING INTEGRITY.

Version 2.0 (7/7/15)

Project No	1179-15-006		Sheet	1	of	1
Client	Richland Engineering		**Calc. By**	KAH	**Date**	3/31/16
Project	CUY-77-14.35 Bridge Replacement		**Check By**	EAA	**Date**	4/1/16
Desc.	Retaining Wall Footing					
	Wall 2, Sta 74+71					

LRFD BEARING RESISTANCE CALCULATION

SOIL PARAMETERS

Structure Location	Boring ID	Soil Layer	Depth (ft)	Description	SPT N (lb/ft)	D_w (ft)	γ_m (pcf)	w_n (%)	Φ (deg.)	C (psf)
STA 74+71	BB-106	1	11	Gravel with Sand	39	51	120	10	34	0

FOOTING / BEARING RESISTANCE COEFFICIENTS

Structure Location	D_f (ft)	B (ft)	L (ft)	Nc (1)	Nq (1)	Nγ (1)	Sc (2)	Sq (2)	Sγ (2)	Dq (3)	Cwq (4)	Cwγ (4)
STA 74+71	4.75	14	31.3	42.20	29.40	41.10	1.312	1.302	0.821	1.0	1.0	1.0

NOMINAL BEARING RESISTANCE

Structure Location	q_N (ksf)
STA 74+71	50.2

$$q_N = cN_c s_c i_c + \gamma D_f N_q s_q d_q i_q C_{wq} + \frac{1}{2}\gamma B N_\gamma s_\gamma i_\gamma C_{w\gamma}$$

BEARING RESISTANCE FACTORS

Limit State	Resistance Factor	
Service	1.0	Article 10.5.5.1
Strength	0.5	Table 10.5.5.2.2-1 (cohesive)
Strength	0.45	Table 10.5.5.2.2-1 (non-cohesive)

FACTORED BEARING RESISTANCE

Limit State	q_R (ksf) Footing	
Service	6.0	Table C10.6.2.6.1-1
Strength	22.6	

REFERENCES

AASHTO LRFD Bridge Design Specifications, 6th Edition, Section 10: Foundations.

1. Bearing Capacity Factors Nc, Nq, and Nγ obtained from Table 10.6.3.1.2a-1.
2. Shape Correction Factors Sc, Sq, and Sγ obtained from Table 10.6.3.1.2a-3.
3. Depth Correction Factor Dq obtained from Table 10.6.3.1.2a-4.
4. Groundwater Correction Coefficients Cwq and Cwγ obtained from Table 10.6.3.1.2a-2.

S&ME ENGINEERING INTEGRITY.

Version 2.0 (7/7/15)

Project No	1179-15-006		Sheet	1	of 1
Client	Richland Engineering	**Calc. By**	KAH	**Date**	3/31/16
Project	CUY-77-14.35 Bridge Replacement	**Check By**	EAA	**Date**	4/1/16
Desc.	Retaining Wall Footing				
	Wall 3, Sta 79+83				

LRFD BEARING RESISTANCE CALCULATION

SOIL PARAMETERS

Structure Location	Boring ID	Soil Layer	Depth (ft)	Description	SPT N (lb/ft)	D_w (ft)	γ_m (pcf)	w_n (%)	Φ (deg.)	C (psf)
STA 79+83	BB-111	1	7.7	Coarse and Fine Sand	47	60	135	11	34	0

FOOTING / BEARING RESISTANCE COEFFICIENTS

Structure Location	D_f (ft)	B (ft)	L (ft)	Nc (1)	Nq (1)	Nγ (1)	Sc (2)	Sq (2)	Sγ (2)	Dq (3)	Cwq (4)	Cwγ (4)
STA 79+83	4.5	14	31.3	42.20	29.40	41.10	1.312	1.302	0.821	1.0	1.0	1.0

NOMINAL BEARING RESISTANCE

Structure Location	q_N (ksf)
STA 79+83	55.1

$$q_N = cN_c s_c i_c + \gamma D_f N_q s_q d_q i_q C_{wq} + \frac{1}{2}\gamma B N_\gamma s_\gamma i_\gamma C_{w\gamma}$$

BEARING RESISTANCE FACTORS

Limit State	Resistance Factor	
Service	1.0	Article 10.5.5.1
Strength	0.5	Table 10.5.5.2.2-1 (cohesive)
Strength	0.45	Table 10.5.5.2.2-1 (non-cohesive)

FACTORED BEARING RESISTANCE

Limit State	q_R (ksf) Footing
Service	6.0
Strength	24.8

Table C10.6.2.6.1-1

REFERENCES

AASHTO LRFD Bridge Design Specifications, 6th Edition, Section 10: Foundations.

1. Bearing Capacity Factors Nc, Nq, and Nγ obtained from Table 10.6.3.1.2a-1.
2. Shape Correction Factors Sc, Sq, and Sγ obtained from Table 10.6.3.1.2a-3.
3. Depth Correction Factor Dq obtained from Table 10.6.3.1.2a-4.
4. Groundwater Correction Coefficients Cwq and Cwγ obtained from Table 10.6.3.1.2a-2.

S&ME ENGINEERING INTEGRITY.

Version 2.0 (7/7/15)

Project No	1179-15-006		Sheet	1	of 1
Client	Richland Engineering	**Calc. By**	KAH	**Date**	3/31/16
Project	CUY-77-14.35 Bridge Replacement	**Check By**	EAA	**Date**	4/1/16
Desc.	Retaining Wall Footing				
	Wall 4, Sta 81+55				

LRFD BEARING RESISTANCE CALCULATION

SOIL PARAMETERS

Structure Location	Boring ID	Soil Layer	Depth (ft)	Description	SPT N (lb/ft)	D_w (ft)	γ_m (pcf)	w_n (%)	Φ (deg.)	C (psf)
STA 81+55	BB-112	1	12.6	Gravel with Sand	43	61.5	125	10	34	0

FOOTING / BEARING RESISTANCE COEFFICIENTS

Structure Location	D_f (ft)	B (ft)	L (ft)	Nc (1)	Nq (1)	Nγ (1)	Sc (2)	Sq (2)	Sγ (2)	Dq (3)	Cwq (4)	Cwγ (4)
STA 81+55	5.5	10	30	42.20	29.40	41.10	1.232	1.225	0.867	1.0	1.0	1.0

NOMINAL BEARING RESISTANCE

Structure Location	q_N (ksf)
STA 81+55	47.0

$$q_N = cN_c s_c i_c + \gamma D_f N_q s_q d_q i_q C_{wq} + \frac{1}{2}\gamma B N_\gamma s_\gamma i_\gamma C_{w\gamma}$$

BEARING RESISTANCE FACTORS

Limit State	Resistance Factor	
Service	1.0	Article 10.5.5.1
Strength	0.5	Table 10.5.5.2.2-1 (cohesive)
Strength	0.45	Table 10.5.5.2.2-1 (non-cohesive)

FACTORED BEARING RESISTANCE

Limit State	q_R (ksf) Footing	
Service	6.0	Table C10.6.2.6.1-1
Strength	21.2	

REFERENCES

AASHTO LRFD Bridge Design Specifications, 6th Edition, Section 10: Foundations.

1. Bearing Capacity Factors Nc, Nq, and Nγ obtained from Table 10.6.3.1.2a-1.
2. Shape Correction Factors Sc, Sq, and Sγ obtained from Table 10.6.3.1.2a-3.
3. Depth Correction Factor Dq obtained from Table 10.6.3.1.2a-4.
4. Groundwater Correction Coefficients Cwq and Cwγ obtained from Table 10.6.3.1.2a-2.

S&ME ENGINEERING INTEGRITY.

Version 2.0 (7/7/15)

Project No	1179-15-006		Sheet	1	of	1
Client	Richland Engineering	**Calc. By**	KAH	**Date**	3/31/16	
Project	CUY-77-14.35 Bridge Replacement	**Check By**	EAA	**Date**	4/1/16	
Desc.	Retaining Wall Footing					
	Wall 4, Sta 80+51					

LRFD BEARING RESISTANCE CALCULATION

SOIL PARAMETERS

Structure Location	Boring ID	Soil Layer	Depth (ft)	Description	SPT N (lb/ft)	D_w (ft)	γ_m (pcf)	w_n (%)	Φ (deg.)	C (psf)
STA 80+51	BB-112	1	10.6	Gravel with Sand	43	62.5	125	10	34	0

FOOTING / BEARING RESISTANCE COEFFICIENTS

Structure Location	D_f (ft)	B (ft)	L (ft)	Nc (1)	Nq (1)	Nγ (1)	Sc (2)	Sq (2)	Sγ (2)	Dq (3)	Cwq (4)	Cwγ (4)
STA 80+51	5.5	14	30	42.20	29.40	41.10	1.325	1.315	0.813	1.0	1.0	1.0

NOMINAL BEARING RESISTANCE

Structure Location	q_N (ksf)
STA 80+51	55.8

$$q_N = cN_c s_c i_c + \gamma D_f N_q s_q d_q i_q C_{wq} + \frac{1}{2}\gamma B N_\gamma s_\gamma i_\gamma C_{w\gamma}$$

BEARING RESISTANCE FACTORS

Limit State	Resistance Factor	
Service	1.0	Article 10.5.5.1
Strength	0.5	Table 10.5.5.2.2-1 (cohesive)
Strength	0.45	Table 10.5.5.2.2-1 (non-cohesive)

FACTORED BEARING RESISTANCE

Limit State	q_R (ksf) Footing
Service	6.0
Strength	25.1

Table C10.6.2.6.1-1

REFERENCES

AASHTO LRFD Bridge Design Specifications, 6th Edition, Section 10: Foundations.

1. Bearing Capacity Factors Nc, Nq, and Nγ obtained from Table 10.6.3.1.2a-1.
2. Shape Correction Factors Sc, Sq, and Sγ obtained from Table 10.6.3.1.2a-3.
3. Depth Correction Factor Dq obtained from Table 10.6.3.1.2a-4.
4. Groundwater Correction Coefficients Cwq and Cwγ obtained from Table 10.6.3.1.2a-2.

S&ME ENGINEERING INTEGRITY.

Version 2.0
(7/7/2015)

Project No	1179-15-006	Sheet	1	of	1
Client	Richland Engineering	**Calc. By**	KAH	**Date**	3/31/16
Project	CUY-77-14.35 Bridge Replacement	**Check By**	EAA	**Date**	4/1/16
Desc.	Retaining Wall Footing				
	Wall 1				

LRFD SLIDING PARAMETERS

Footing @ Sta 73+72:

Reference Boring: BB-104

D_f = 5.75 ft below existing grade, bearing in: Coarse and Fine Sand (A-3a)

For Cohesionless Soils (i.e. $\phi_f > 0$):

N = 43 bpf

ϕ_f = 34 degrees

Coefficient of Friction = tan δ

= tan(ϕ_f) for concrete cast against soil

= 0.8 tan(ϕ_f) for precast concrete footing

Headwall Type: Cast-in-Place

tan δ = tan (34)

tan δ = **0.67**

REFERENCES

AASHTO LRFD Bridge Design Specifications, 6th Edition, Section 10: Foundations.

10.6.3.4 Failure by Sliding

S&ME ENGINEERING INTEGRITY.	**Project No**	1179-15-006		Sheet	1	of 1
	Client	Richland Engineering	**Calc. By**	KAH	**Date**	3/31/16
	Project	CUY-77-14.35 Bridge Replacement	**Check By**	EAA	**Date**	4/1/16
Version 2.0	**Desc.**	Retaining Wall Footing				
(7/7/2015)		Wall 2				

LRFD SLIDING PARAMETERS

Footing @ Sta 74+71:

Reference Boring: BB-106

D_f = 4.75 ft below existing grade, bearing in: Gravel with Sand (A-1-b)

For Cohesionless Soils (i.e. $\phi_f > 0$):

N = 39 bpf

ϕ_f = 34 degrees

Coefficient of Friction = tan δ

= tan(ϕ_f) for concrete cast against soil

= 0.8 tan(ϕ_f) for precast concrete footing

Headwall Type: Cast-in-Place

tan δ = tan (34)

tan δ = **0.67**

REFERENCES

AASHTO LRFD Bridge Design Specifications, 6th Edition, Section 10: Foundations.

10.6.3.4 Failure by Sliding

Project No	1179-15-006		Sheet	1	of	1
Client	Richland Engineering	**Calc. By**	KAH		**Date**	3/31/16
Project	CUY-77-14.35 Bridge Replacement	**Check By**	EAA		**Date**	4/1/16
Desc.	Retaining Wall Footing					
	Wall 3					

Version 2.0
(7/7/2015)

LRFD SLIDING PARAMETERS

Footing @ Sta 79+83:

Reference Boring: BB-111

D_f = 4.5 ft below existing grade, bearing in: Coarse and Fine Sand (A-3a)

For Cohesionless Soils (i.e. $\phi_f > 0$):

N = 47 bpf
ϕ_f = 34 degrees

Coefficient of Friction = tan δ
= tan(ϕ_f) for concrete cast against soil
= 0.8 tan(ϕ_f) for precast concrete footing

Headwall Type: Cast-in-Place

tan δ = tan (34)
tan δ = **0.67**

REFERENCES

AASHTO LRFD Bridge Design Specifications, 6th Edition, Section 10: Foundations.
10.6.3.4 Failure by Sliding

S&ME ENGINEERING INTEGRITY.

Version 2.0 (7/7/2015)

Project No	1179-15-006	Sheet	1	of	1
Client	Richland Engineering	**Calc. By**	KAH	**Date**	3/31/16
Project	CUY-77-14.35 Bridge Replacement	**Check By**	EAA	**Date**	4/1/16
Desc.	Retaining Wall Footing				
	Wall 4				

LRFD SLIDING PARAMETERS

Footing @ Sta 81+55:

Reference Boring: BB-112

D_f = 5.5 ft below existing grade, bearing in: Gravel with Sand (A-1-b)

For Cohesionless Soils (i.e. $\phi_f > 0$):

N = 43 bpf

ϕ_f = 34 degrees

Coefficient of Friction = tan δ

= tan(ϕ_f) for concrete cast against soil

= 0.8 tan(ϕ_f) for precast concrete footing

Headwall Type: Cast-in-Place

tan δ = tan (34)

tan δ = **0.67**

Footing @ Sta 80+51:

Reference Boring: BB-112

D_f = 5.5 ft below existing grade, bearing in: Gravel with Sand (A-1-b)

For Cohesionless Soils (i.e. $\phi_f > 0$):

N = 43 bpf

ϕ_f = 34 degrees

Coefficient of Friction = tan δ

tan(ϕ_f) for concrete cast against soil

0.8 tan(ϕ_f) for precast concrete footing

Headwall Type: Cast-in-Place

tan δ = tan (34)

tan δ = **0.67**

REFERENCES

AASHTO LRFD Bridge Design Specifications, 6th Edition, Section 10: Foundations.

10.6.3.4 Failure by Sliding

BBCM

LOG OF BORING NO. BB-104
CUY-77-14.35
CLEVELAND, OHIO

BBC&M JOB: 012 00851.301

TYPE: 3-1/4" I.D. Hollow-stem Auger
2" O.D. Split-barrel Sampler

LOCATION: IR - 77
Sta. 72+90
34' Rt. of Centerline

COMPLETION DEPTH: 40.0'
ELEVATION: 680.8
DATE: 12/14/05

Elev. (feet)	Depth (feet)	Std. Pen. / RQD	Hand Pen. (tsf)	Rec./Loss (feet)	CLASSIFICATION: DESCRIPTION	Samp. No.	% AGG.	% C.S.	% F.S.	% SILT	% CLAY	LL	PI	WC	ODOT Class
680.2	0.6				ASPHALT - 6 3/4 INCHES										
679.5	1.3				CONCRETE - 9 INCHES										
677.8	3.0	11/11/11			SANDY SILT (FILL): Medium-dense brown fine to coarse sand, some silt, little clay, trace fine to coarse gravel.	1	8	14	33	28	17	NP	NP	11	A-4a(2)
676.3	4.5	11/14/22			COARSE AND FINE SAND (FILL): Dense brown and gray fine to coarse sand, little fine to coarse gravel, little silt, trace clay, contains few brick fragments.	2								13	Est. A-3a
674.8	6.0	8/7/11	4.5+		SANDY SILT (FILL): Hard brown and dark gray silt, some clay, "and" fine to coarse sand, trace fine gravel, slightly organic.	3	5	14	31	27	23	21	6	13	A-4a(3)
		8/15/28				4								7	Est. A-3a
		18/22/26			COARSE AND FINE SAND (FILL): Dense gray and brown fine to coarse sand, trace to little fine to coarse gravel, trace to little silt, trace clay, contains few brick fragments, slightly organic.	5									Est. A-3a
		16/22/21				6									Est. A-3a
		9/17/19				7									Est. A-3a
		14/26/22				8									Est. A-3a
660.3	20.5	21/21/17				9									Est. A-3a
657.3	23.5	13/13/23	4.5+		SANDY SILT (FILL): Hard brown silt, some clay , some to "and" fine to coarse sand, trace to little fine gravel, slightly organic.	10								13	Est. A-4a
655.3	25.5	14/24/22			SANDY SILT (FILL): Dense brown fine to coarse sand, some silt, little clay, trace to little fine gravel.	11									Est. A-4a
		12/14/17			FINE SAND: Medium-dense brown fine sand, little silt, trace coarse sand, trace clay.	12									Est. A-3
		5/6/7				13									Est. A-3
		9/12/13				14									Est. A-3
640.8	40.0	9/11/12				15									Est. A-3

NOTES:
- No seepage encountered.

WATER LEVEL:	"Dry"	"Dry"		
WATER NOTE:	Inside HSA - At Completion	Augers Pulled - Caved at 31.0'		
DATE:	12/14/05	12/14/05		

BBCM

LOG OF BORING NO. BB-105
CUY-77-14.35
CLEVELAND, OHIO

BBC&M JOB: 012 00851.301

TYPE: 3-1/4" I.D. Hollow-stem Auger
2" O.D. Split-barrel Sampler

LOCATION: IR - 77
Sta. 72+89
117' Rt. of Centerline

COMPLETION DEPTH: 20.0'
ELEVATION: 660.8
DATE: 12/21/05

Elev. (feet)	Depth (feet)	Std. Pen. / RQD	Hand Pen. (tsf)	Rec./Loss (feet)	CLASSIFICATION: DESCRIPTION	Samp. No.	% AGG.	% C.S.	% F.S.	% SILT	% CLAY	LL	PI	WC	ODOT Class
659.8	1.0				TOPSOIL - 12 INCHES										
658.3	2.5	5/12/12	1.5-4.0		SILT AND CLAY (FILL): Stiff to very-stiff brown clay, some silt, "and" fine to coarse sand, trace fine to coarse gravel.	1	8	18	24	22	28	27	13	14	A-6a(4)
		5/8/9			COARSE AND FINE SAND (FILL): Medium-dense brown fine to coarse sand, little fine to coarse gravel, trace silt.	2									Est. A-3a
652.8	8.0	6/8/9				3									Est. A-3a
		6/4/4			FINE SAND: Loose to medium-dense brown fine to coarse sand, trace silt, trace fine gravel.	4	3	38	55	4		NP	NP	9	A-3(0)
		6/9/11				5									Est. A-3
640.8	20.0	4/7/9				6									Est. A-3

NOTES:
- No seepage encountered.

WATER LEVEL:	"Dry"	"Dry"		
WATER NOTE:	Inside HSA - At Completion	Augers Pulled - Caved at 15.0'		
DATE:	12/21/05	12/21/05		

BBCM

CALCULATED	CHECKED BY PSL
DATE 3/24/06	REVISED 11/21/07
REVIEWED BY JLS	
DRAWN BY BLR	

STRUCTURE FOUNDATION INVESTIGATION
BRIDGE NO. CUY-77-1433 OVER I.R. 490 AND RAMPS

CUY-77-14.35
PID No. 13567

6 / 26

10 / 30

BBCM

LOG OF BORING NO. BB-112
CUY-77-14.35
CLEVELAND, OHIO

BBC&M JOB: 012.00851.301

TYPE: 3-1/4" I.D. Hollow-stem Auger
2" O.D. Split-barrel Sampler   3" O.D. Shelby Tube Sampler

LOCATION: IR - 77
Sta. 81+61
37' Lt. of Centerline

COMPLETION DEPTH: 40.0'
ELEVATION: 688.9
DATE: 12/16/05

Elev. (feet)	Depth (feet)	Std. Pen. / RQD	Hand Pen. (tsf)	Rec./Loss (feet)	CLASSIFICATION: DESCRIPTION	Samp. No.	% AGG.	% C.S.	% F.S.	% SILT	% CLAY	LL	PI	WC	ODOT Class
688.9	0														
688.4	0.5				ASPHALT - 6 INCHES										
687.6	1.3				CONCRETE - 9 INCHES										
		5/10/14			SANDY SILT (FILL): Medium-dense brown silt, some clay, some fine to coarse sand, trace fine gravel.	1	3	10	24	40	23	NP	NP	15	A-4a(6)
685.9	3.0	14/14/18			GRAVEL WITH SAND (FILL): Dense brown fine to coarse sand, some fine to coarse gravel, little silt, trace to little clay, contains few brick fragments.	2	24	27	26	23		NP	NP	10	A-1-b(0)
		13/15/20				3								9	Est. A-1-b
		18/18/22				4								11	Est. A-1-b
		13/15/16				5									Est. A-1-b
		10/19/20				6									Est. A-1-b
		17/23/22				7									Est. A-1-b
		15/23/26				8									Est. A-1-b
		15/18/19				9									Est. A-1-b
		15/22/23				10									Est. A-1-b
665.9	23.0	2/2/2	0.25-0.75		SILT AND CLAY: Soft to medium-stiff gray silt, "and" clay, trace fine to coarse sand, contains few silt and fine sand seams, slightly organic.	11	0	1	4	53	42	34	11	31	A-6a(8)
		P	0.25			12	1	1	3	49	46	34	13	28	A-6a(9)
662.4	26.5	10/14/18			COARSE AND FINE SAND: Medium-dense to dense brown fine to coarse sand, trace fine gravel, trace silt, trace clay.	13									Est. A-3a
		8/10/12				14									Est. A-3a
651.9	37.0	13/15/16			FINE SAND: Dense brown fine sand, trace coarse sand, trace silt, trace clay.	15									Est. A-3
648.9	40.0														

NOTES:
- No seepage encountered.
- Loss-on-Ignition test performed on Sample S-11, LOI=2.9%.

WATER LEVEL:	"Dry"	"Dry"		
WATER NOTE:	Inside HSA - At Completion	Augers Pulled - Caved at 32.0'		
DATE:	12/16/05	12/16/05		

BBCM

LOG OF BORING NO. BB-113
CUY-77-14.35
CLEVELAND, OHIO

BBC&M JOB: 012.00851.301

TYPE: 3-1/4" I.D. Hollow-stem Auger
2" O.D. Split-barrel Sampler   3" O.D. Shelby Tube Sampler

LOCATION: IR - 77
Sta. 81+56
107' Lt. of Centerline

COMPLETION DEPTH: 20.0'
ELEVATION: 669.5
DATE: 12/22/05

Elev. (feet)	Depth (feet)	Std. Pen. / RQD	Hand Pen. (tsf)	Rec./Loss (feet)	CLASSIFICATION: DESCRIPTION	Samp. No.	% AGG.	% C.S.	% F.S.	% SILT	% CLAY	LL	PI	WC	ODOT Class
668.9	0.6				TOPSOIL - 7 INCHES										
		4/5/6			COARSE AND FINE SAND (FILL): Medium-dense brown fine to coarse sand, trace silt, trace clay, trace fine gravel.	1									Est. A-3a
666.5	3.0	3/5/4			COARSE AND FINE SAND (POSSIBLE FILL): Loose brown fine to coarse sand, little silt, trace clay, trace fine gravel.	2	1	30	47	12	10	NP	NP	20	A-3a(0)
663.8	5.7	P	0.0-0.25			7	1	4	8	40	47	40	18	41	A-6b(11)
663.3	6.2	1/1/2	0.0-0.5		SILTY CLAY: Very-soft brown and gray silty clay, little fine to coarse sand, trace fine gravel, contains few fine sand and silt seams, contains decayed roots.	3	1	1	7	52	39	34	14	33	A-6a(10)
662.0	7.5	10/11/12			SILT AND CLAY: Very-soft to soft gray becoming brown silt, "and" clay, trace fine to coarse sand, trace fine gravel, contains many silt and fine sand lenses and seams.	4									Est. A-3a
					COARSE AND FINE SAND: Medium-dense brown fine to coarse sand, trace silt, trace clay, trace fine to coarse gravel.										
		8/10/11				5									Est. A-3a
		7/8/12				6									Est. A-3a
649.5	20.0														

NOTES:
- Encountered seepage at 4.5'.
- Offset borehole to attempt Shelby Tube, S-7 from 5.5' to 7.5'.

WATER LEVEL:	"Dry"	"Dry"		
WATER NOTE:	Inside HSA - At Completion	Augers Pulled - Caved at 15.6'		
DATE:	12/22/05	12/22/05		

BBCM

CALCULATED	CHECKED BY PSL
DATE 3/24/06	REVISED 11/21/07
REVIEWED BY JLS	
DRAWN BY BLR	

STRUCTURE FOUNDATION INVESTIGATION
BRIDGE NO. CUY-77-1433 OVER I.R. 490 AND RAMPS

CUY-77-14.35
PID No. 13567

BBCM

LOG OF BORING NO. BB-106
CUY-77-14.35
CLEVELAND, OHIO

TYPE: 3-1/4" I.D. Hollow-stem Auger
2" O.D. Split-barrel Sampler

LOCATION: IR - 77
Sta. 74+20
10' Lt. of Centerline

COMPLETION DEPTH: 90.0'
ELEVATION: 684.4
DATE: 12/5/05 - 12/6/05

Elev. (feet)	Depth (feet)	Std. Pen. / RQD	Hand Pen. (tsf)	Rec./Loss (feet)	CLASSIFICATION: DESCRIPTION	Samp. No.	% AGG.	% C.S.	% F.S.	% SILT	% CLAY	LL	PI	WC	ODOT Class
684.0	0.4				ASPHALT - 5 INCHES										
683.2	1.2				CONCRETE - 9 INCHES										
681.4	3.0	15/20/24			COARSE AND FINE SAND (FILL): Dense brown fine to coarse sand, little silt, trace clay, trace fine gravel, contains iron oxide staining.	1	7	21	48	14	10	NP	NP	8	A-3a(0)
		16/11/9			GRAVEL WITH SAND (FILL): Medium-dense to very-dense brown fine to coarse sand, little fine to coarse gravel, trace to little silt, trace clay, contains many brick fragments and few coal and concrete fragments.	2	16	34	29	13	8	NP	NP	8	A-1-b(0)
		10/12/17				3								10	Est. A-1-b
		9/17/21				4								8	Est. A-1-b
		16/16/16				5									Est. A-1-b
		10/22/17				6								10	Est. A-1-b
		19/26/30				7									Est. A-1-b
		15/21/19				8	18	33	32	10	7	NP	NP	11	A-1-b(0)
		17/30/23				9									Est. A-1-b
		41/17/15				10								10	Est. A-1-b
661.4	23.0														
		11/15/18			GRAVEL WITH SAND (POSSIBLE FILL): Dense to very-dense brown fine to coarse sand, little fine to coarse gravel, trace silt, trace clay.	11									Est. A-1-b
		17/22/32				12									Est. A-1-b
		15/16/18				13	19	53	21	3	4	NP	NP	4	A-1-b(0)
652.4	32.0														
		12/12/16			FINE SAND: Medium-dense brown fine sand, some coarse sand, trace silt, trace clay.	14									Est. A-3
		8/6/6				15								8	Est. A-3
		6/6/7				16	0	21	73	2	4	NP	NP	9	A-3(0)
		6/7/8				17								7	Est. A-3
632.4	52.0														
		11/9/10			COARSE AND FINE SAND: Medium-dense to very-dense brown and gray fine sand, little silt, trace clay.	18									Est. A-3a

WATER LEVEL:	58.5	"Dry"		
WATER NOTE:	Encountered	Augers Pulled - Caved at 35.0'		
DATE:	12/5/05	12/6/05		

BBCM

LOG OF BORING NO. BB-106
CUY-77-14.35
CLEVELAND, OHIO

TYPE: 3-1/4" I.D. Hollow-stem Auger
2" O.D. Split-barrel Sampler

LOCATION: IR - 77
Sta. 74+20
10' Lt. of Centerline

COMPLETION DEPTH: 90.0'
ELEVATION: 684.4
DATE: 12/5/05 - 12/6/05

Elev. (feet)	Depth (feet)	Std. Pen. / RQD	Hand Pen. (tsf)	Rec./Loss (feet)	CLASSIFICATION: DESCRIPTION	Samp. No.	% AGG.	% C.S.	% F.S.	% SILT	% CLAY	LL	PI	WC	ODOT Class
		7/8/11			COARSE AND FINE SAND: Medium-dense to very-dense brown and gray fine sand, little silt, trace clay.	19	0	0	79	17	4	NP	NP	27	A-3a(0)
		18/27/30				20								23	Est. A-3a
		14/21/19				21									Est. A-3a
613.4	71.0														
		24/23/38			SILT: Stiff to very-stiff gray silt, some clay, trace fine to coarse sand, trace fine gravel.	22	1	1	6	67	25	24	7	19	A-4b(8)
		14/17/24	1.5-4.0			23								23	Est. A-4b
602.4	82.0														
		11/11/12	0.5-1.5		SILTY CLAY: Medium-stiff to stiff gray silty clay, trace fine to coarse sand, contains many silt seams.	24								26	Est. A-6b
		6/9/12	0.75-1.5			25	0	1	2	37	60	37	16	26	A-6b(10)
594.4	90.0														

NOTES:
- Encountered seepage at 53.5'.
- Encountered water at 58.5'.
- After the augers were covered at the end of the day (12/5/05), 3' of heave was observed at 68.5' the next morning (12/6/05).
- Added water inside the HSA prior to drilling on 12/6/05 starting at 70.0'.

WATER LEVEL:	58.5	"Dry"		
WATER NOTE:	Encountered	Augers Pulled - Caved at 35.0'		
DATE:	12/5/05	12/6/05		

BBCM

CALCULATED	CHECKED BY PSL
DATE 3/24/06	REVISED 11/21/07
REVIEWED BY JLS	
DRAWN BY BLR	

STRUCTURE FOUNDATION INVESTIGATION
BRIDGE NO. CUY-77-1433 OVER I.R. 490 AND RAMPS

CUY-77-14.35
PID No. 13567

BBCM

LOG OF BORING NO. BB-111
CUY-77-14.35
CLEVELAND, OHIO

BBC&M JOB: 012.00851.301

TYPE: 3-1/4" I.D. Hollow-stem Auger
2" O.D. Split-barrel Sampler

LOCATION: IR - 77
Sta. 80+30
10' Rt. of Centerline

COMPLETION DEPTH: 90.0'
ELEVATION: 690.3
DATE: 12/7/05 - 12/13/05

Elev. (feet)	Depth (feet)	Std. Pen. / RQD	Hand Pen. (tsf)	Rec./Loss (feet)	CLASSIFICATION: DESCRIPTION	Samp. No.	% AGG.	% C.S.	% F.S.	% SILT	% CLAY	LL	PI	WC	ODOT Class
689.0	1.3				ASPHALT - 16 INCHES										
		16/16/26			SANDY SILT (FILL): Dense brown fine to coarse sand, some to "and" silt, little clay, trace fine gravel.	1	5	14	26	37	18	NP	NP	14	A-4a(4)
685.8	4.5	16/20/28				2	5	21	38	20	16	NP	NP	11	A-4a(0)
		19/27/27			COARSE AND FINE SAND (FILL): Dense to very-dense brown fine sand, little coarse sand, little silt, little clay, contains few brick fragments.	3								11	Est. A-3a
		9/14/27				4								11	Est. A-3a
		25/22/24				5									Est. A-3a
		15/25/27				6	9	11	49	19	12	NP	NP	10	A-3a(0)
		25/25/38				7									Est. A-3a
		20/26/16				8								12	Est. A-3a
669.8	20.5	23/34/30				9									Est. A-3a
667.3	23.0	16/17/22			SILT (FILL): Dense brown silt, little fine sand, little clay, trace coarse sand.	10	0	1	20	61	18	NP	NP	13	A-4b(8)
664.8	25.5	8/15/15			FINE SAND (FILL): Medium-dense gray and brown fine sand, trace silt.	11									Est. A-3
		13/21/28			COARSE AND FINE SAND (FILL): Dense brown and gray fine to coarse sand, trace fine gravel, trace silt, contains few brick and shale fragments.	12								6	Est. A-3a
658.3	32.0	22/22/27				13									Est. A-3a
		9/10/12			FINE SAND: Medium-dense to very-dense brown fine sand, little coarse sand, trace silt.	14								7	Est. A-3
		38/56				15									Est. A-3
		22/22/23				16									Est. A-3
		18/17/22				17	0	11	82	7		NP	NP	5	A-3(0)
		18/20/21				18									Est. A-3

WATER LEVEL: 68.5 | "Dry"
WATER NOTE: Inside HSA - At Completion | Augers Pulled - Caved at 31.0'
DATE: 12/13/05 | 12/13/05

BBCM

LOG OF BORING NO. BB-111
CUY-77-14.35
CLEVELAND, OHIO

BBC&M JOB: 012.00851.301

TYPE: 3-1/4" I.D. Hollow-stem Auger
2" O.D. Split-barrel Sampler

LOCATION: IR - 77
Sta. 80+30
10' Rt. of Centerline

COMPLETION DEPTH: 90.0'
ELEVATION: 690.3
DATE: 12/7/05 - 12/13/05

Elev. (feet)	Depth (feet)	Std. Pen. / RQD	Hand Pen. (tsf)	Rec./Loss (feet)	CLASSIFICATION: DESCRIPTION	Samp. No.	% AGG.	% C.S.	% F.S.	% SILT	% CLAY	LL	PI	WC	ODOT Class
		20/19/24			FINE SAND: Medium-dense to very-dense brown fine sand, little coarse sand, trace silt.	19								18	Est. A-3
623.3	67.0	24/35/38				20									Est. A-3
		20/16/17			SANDY SILT: Dense gray silt, "and" fine sand, little clay, contains few silty clay seams and pockets.	21								20	Est. A-4a
614.3	76.0	12/12/23				22									Est. A-4a
608.3	82.0	14/16/21			SILT: Dense brown and gray silt, little clay, little fine sand, contains a diagonal dark gray silty clay seam.	23	0	1	13	70	16	NP	NP	23	A-4b(8)
		9/12/12	0.5-1.0		SANDY SILT: Medium-stiff to stiff gray silt, some to "and" clay, little fine to coarse sand.	24								25	Est. A-4a
600.3	90.0	9/14/16	1.0-1.75			25									Est. A-4a

NOTES:
- Encountered seepage at 58.5'.
- Added water inside HSA at 60.0'.

WATER LEVEL: 68.5 | "Dry"
WATER NOTE: Inside HSA - At Completion | Augers Pulled - Caved at 31.0'
DATE: 12/13/05 | 12/13/05

BBCM

CALCULATED	CHECKED BY PSL	DATE 3/24/06	REVISED 11/21/07	REVIEWED BY JLS	DRAWN BY BLR

STRUCTURE FOUNDATION INVESTIGATION
BRIDGE NO. CUY-77-1433 OVER I.R. 490 AND RAMPS

CUY-77-14.35
PID No. 13567

12/26

16/30

BBC&M ENGINEERING, INC. DRAWING NO. ...\012-00851-301D060 130 STRUCTURE.dgn

REPORT

OF

SUBSURFACE EXPLORATION FOR

PROPOSED U.S. ROUTE 33 (LANCASTER) BYPASS:

PROPOSED RELOCATED DELMONT ROAD BRIDGE

OVER PROPOSED U.S. ROUTE 33 BYPASS

(BRIDGE NUMBER FAI-33-1309)

LANCASTER, OHIO

Prepared By:

DODSON-STILSON, INC.
6121 Huntley Road
Columbus, Ohio 43229

DSI Job No. 9921-302400

June 21, 2000

REPORT
OF
SUBSURFACE EXPLORATION FOR
PROPOSED U.S. ROUTE 33 (LANCASTER) BYPASS:
PROPOSED RELOCATED DELMONT ROAD BRIDGE
OVER PROPOSED U.S. ROUTE 33 BYPASS
(BRIDGE NUMBER FAI-33-1309)
LANCASTER, OHIO

1.0 INTRODUCTION

The project consists of constructing a new bridge where the proposed relocated Delmont Road is to cross over the proposed U.S. Route 33 (Lancaster) Bypass. The proposed bridge is designated as bridge number FAI-33-1309. It is understood that the proposed bridge is to be a four-span, continuous composite steel girder bridge with semi-integral type abutments and cap and column type piers. The site is located just west of Lancaster, Ohio and can be found on the USGS Amanda Quadrangle. The exploration presented in this report has been conducted essentially in accordance with the subsurface exploration scope of services in Dodson-Stilson, Inc.'s proposal dated January 6, 1999 and the subsequent modification to this proposal dated June 4, 1999.

The purpose of this exploration was to 1) determine the subsurface conditions to the depths of the borings, 2) evaluate the engineering characteristics of the subsurface materials, and 3) provide information to assist in the design of the bridge foundation.

The geotechnical engineer has planned and supervised the performance of the geotechnical engineering services, has considered the findings, and has prepared this report in accordance with generally accepted geotechnical engineering practices. No other warranties, either expressed or implied, are made as to the professional advice included in this report.

2.0 FIELD EXPLORATION

The field exploration consisted of drilling five borings, B-30 through B-34, for the proposed bridge foundation. The borings were drilled from January 10 to January 14, 2000, using a truck-mounted drill rig. Each of the borings was drilled to a depth of between 45 and 80 feet. Information concerning the drilling procedures is presented in the Appendix.

The boring locations were selected by representatives of the Structural Department at Dodson-Stilson, Inc. (DSI) and were staked in the field by DSI personnel. The borings were drilled as close to the selected locations as site conditions and utilities would permit. The as-drilled boring locations and ground surface elevations at the boring locations were also established by representatives of DSI. The location of each of the borings is shown on the Site Plan presented in the Appendix.

3.0 FINDINGS

3.1 Geology of the Site

Generalized geological references report that the site exists near the southern-most advancement of the Illinoian and Wisconsin glaciers. Overburden at the site is believed to be deposits of glacial end moraines, glacial ground moraines, and glacial outwash consisting of clay, silt, sand, and coarser fragments. In addition, the site is located near the edge of a prehistoric buried valley filled predominantly with sand and coarser fragments. The underlying bedrock at the site is believed to consist of shale and shaly sandstone of the Cuyahoga formation.

3.2 Subsurface Conditions

The following sections present the generalized subsurface conditions encountered by the borings. For more detailed information, please refer to the boring logs presented in the Appendix. The borings logs have been prepared on the basis of the driller's field record of drilling and sampling and the soil engineer's examination and visual classification of the samples procured. Also, moisture content and other test data developed in the laboratory are shown on the boring logs. Stratification lines indicating changes in soil composition, moisture, and color represent depths of changes as best as can be approximated by the drilling records, by sample recovery, and by examination of the samples. Depths of in-situ changes may differ somewhat from those estimated.

3.2.1 Soil Conditions

Each of the five borings first encountered between 3 and 12 inches of topsoil during drilling. Underlying the topsoil, the five borings typically encountered cohesive soils consisting stiff to hard silt and clay (A-6a) and silty clay (A-6b) to depths of between 10.5 and 20.5 feet. Some of these soils were organic in nature. Underlying these cohesive soils each of the five borings generally encountered medium dense to very dense non-cohesive soils including gravel with sand (A-1-b), gravel with sand and silt (R-2-4), fine sand (A-3), and coarse and fine sand (A-3a). These soils were encountered to the completion depths of the borings in borings B-33 and B-34 and were encountered to depths of between 49.5 and 62.0 feet in borings B-30, B-31, and B-32, where bedrock was encountered. It should be noted that material classified as silt (A-4b) was encountered in boring B-34. However, this material was encountered at depths of greater than 50 feet.

3.2.2 Bedrock Conditions

Bedrock was first encountered in borings B-30, B-31, and B-32 at depths of between 49.5 and 62.0 feet and confirmed by coring. The bedrock consisted of medium hard broken sandstone with Rock Quality Designations (RQD's) of between 30 and 50 percent.

3.2.3 Groundwater Conditions

Water seepage was first encountered in each of the five borings at depths of between 7.2 and 16.0 feet below the existing ground surface. At the completion of drilling there were measurable water levels in each of the five borings and the depth to water in these borings ranged from 4.2 to 6.0 feet. However, these final water level readings included water and mud used for the drilling operations and consequently may not reflect the actual groundwater conditions.

In addition, it should be emphasized that excess hydrostatic pressures were encountered in several of the borings during drilling. These excess hydrostatic pressures caused the water levels to rise rapidly in the boreholes and also caused the underlying soils to "heave" in the boreholes during drilling.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 Foundation Recommendations

Based on the findings of the borings, the subsurface conditions appear reasonably suitable to support the proposed structure on shallow spread footings. It is therefore recommended that the spread footings for the proposed piers be designed based on an allowable soil bearing capacity of 5000 pounds per square foot (psf). This is provided that the footings are founded at or below the elevations shown on the attached Site Plan.

In addition, it is understood that the proposed bridge abutments are to be supported on spread footings founded in new embankment fill and reportedly are to be designed based on an allowable soil bearing capacity of 4000 psf. This is considered acceptable provided the embankment material is selected and placed in such a manner so as to achieve the desired allowable soil bearing capacity.

Relative to the footings and footing excavations, the following additional recommendations are made:

1. All exterior footings should be founded deep enough for frost protection, considered to be 36 inches in this area.

2. Somewhat weaker soils were encountered near the planned footing elevation in boring B-31. Consequently, it is strongly recommended that the footing excavation bottoms be examined by the geotechnical engineer prior to placement of reinforcing steel and concrete, in order to determine the suitability of the supporting soils.

3. Embankment fill material should be selected and placed in such a manner so as to achieve the recommended abutment soil bearing pressure.

4. Excavations should be undercut to suitable bearing material if such material is not encountered at the planned footing level. Such undercuts may be backfilled with a lean mix concrete (1,500 psi @ 28 days) or footing concrete.

Alternately, steel cast-in-place (CIP) piles could be considered to support the proposed structure. Recommendations regarding CIP have not been included in this report but can be provided upon request.

4.2 Proposed Bridge Approach Embankment Construction Recommendations

Prior to subgrade preparation and new fill placement, all grass, weeds, topsoil, and organic soils should be overexcavated and removed from within and to 10 feet beyond the limits of the proposed embankments. The fill foundation should then be proofrolled with a heavy piece of rubber-tired construction equipment. Any areas that rut or deflect significantly should be overexcavated and replaced with suitable compacted material as required to develop stability.

Somewhat weaker soils were encountered at shallower depths in the majority of the borings. Consequently, it is strongly recommended that the fill foundation be examined by the geotechnical engineer prior to placement of any new fill in order to determine the suitability of the foundation soils.

It is understood that the proposed approach embankments will be up to 20 feet above existing grade. Post-construction settlement of the approach embankments should not be a concern provided the above recommendations are followed.

4.3 Excavation and Groundwater Considerations

Excavations deeper than five feet must be laid back or braced to protect workers entering the excavations. Please refer to current OSHA regulations regarding sloping and shoring requirements (29 CFR Part 1926).

Groundwater seepage was first encountered in each of the five borings at depths of between 7.2 and 16.0 feet. In addition, excess hydrostatic pressures were encountered in each of the five borings which caused the water levels to rise rapidly in the boreholes. It is anticipated that excavations shallower than 5 feet should not encounter significant water seepage. However, excavations extending below this depth may encounter significant amounts of groundwater seepage. In addition, groundwater conditions can vary seasonally and with the passage of time. Consequently, the contractor should be prepared to deal with potentially significant water seepage or surface water. All excavations should be maintained reasonably dry.

5.0 CLOSING REMARKS

We appreciate having the opportunity to be of service to you on this project. Please do not hesitate to call if you have any questions concerning our report.

Sincerely,

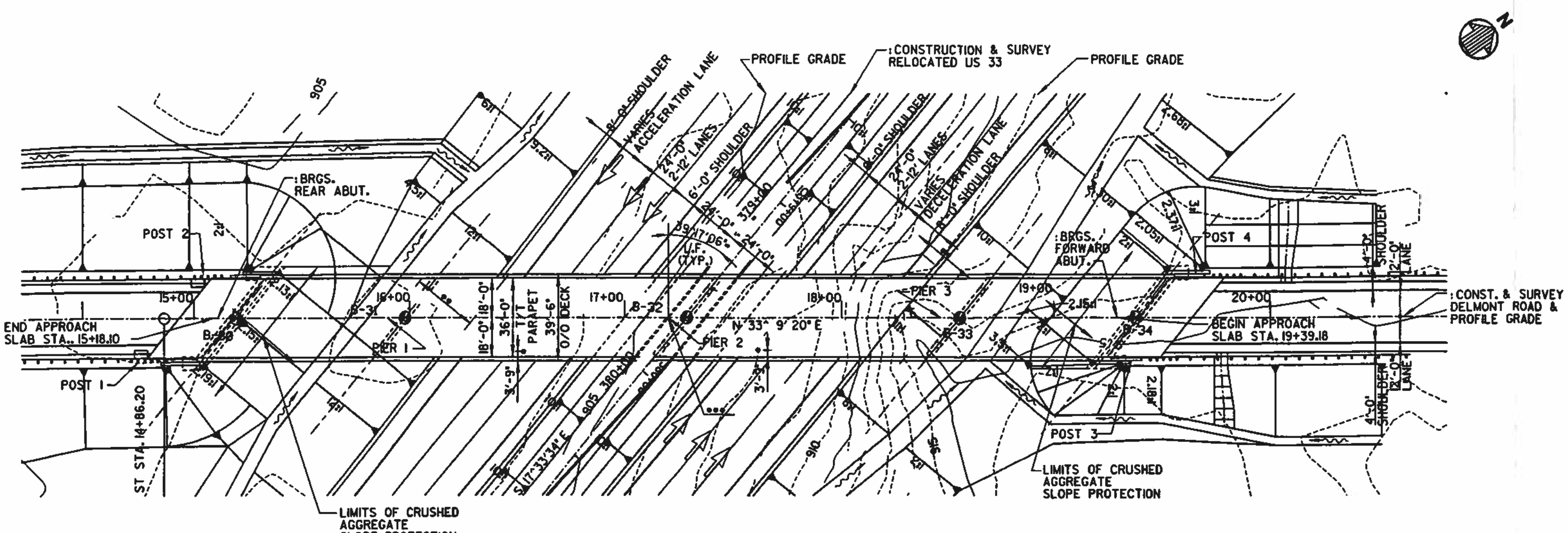
DODSON-STILSON, INC.

Timothy A. Hampshire, MSCE, P.E.
Staff Engineer

Arthur (Pete) Nix, P.E.
Senior Geotechnical Engineer

TAH/APN/cb

M:\proj\9921\3024.00\documents\Delmont-rpt.wpd

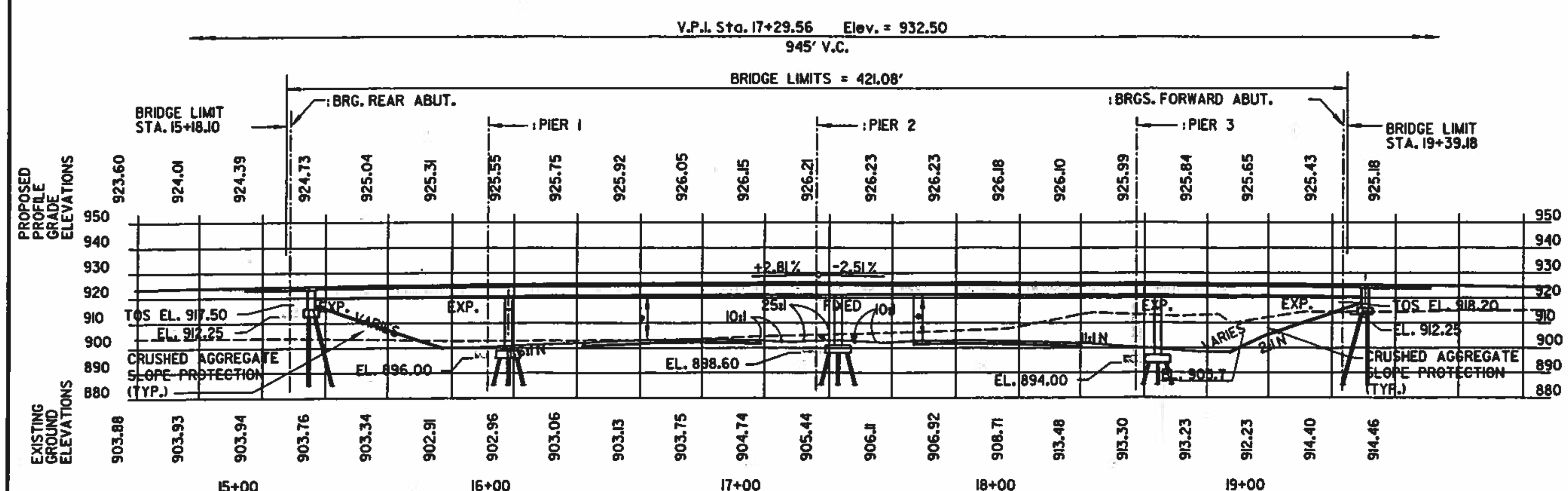


PLAN

SUBSTRUCTURE @ :DELMONT ROAD	STATION	* PILE LENGTH
:BRGS. REAR ABUTMENT	15+19.39	52.4'
:PIER 1	15+97.89	36.7'
:PIER 2	17+28.64	40.8'
:PIER 3	18+55.39	33.9'
:BRGS. FORWARD ABUTMENT	19+37.89	48.2'

* ALL PILE TYPES 12" Ø CIP TYP.

POST NUMBER	FIRST POST GUARD RAIL	STATION
POST 1	RIGHT REAR ABUT.	14+83.22
POST 2	LEFT REAR ABUT.	15+12.66
POST 3	RIGHT FORWARD ABUT.	19+44.61
POST 4	LEFT FORWARD ABUT.	19+74.05



PROFILE

ALONG :CONSTRUCTION & SURVEY DELMONT ROAD

NOTES:

1. EARTHWORK LIMITS SHOWN ARE APPROXIMATE ACTUAL SLOPES SHALL CONFORM TO PLAN CROSS SECTIONS.
2. ⊕= BORING LOCATIONS.

LEGEND

- * MINIMUM VERTICAL CLEARANCE
  - 16'-10¼" ACTUAL SOUTHBOUND
  - 16'-6" REQUIRED
  - 17'-6¼" ACTUAL NORTHBOUND
  - 16'-6" REQUIRED
- ** MINIMUM HORIZONTAL CLEARANCE
  - 30'-3" ACTUAL SOUTHBOUND
  - 30'-0" REQUIRED
  - 30'-0¼" ACTUAL NORTHBOUND
  - 30'-0" REQUIRED
- *** :PIER 2, STA. 17+28.64 DELMONT RD. = STA. 379+68.18 RELOCATED US 33

ABUT. = ABUTMENT
ADT = AVERAGE DAILY TRAFFIC
ADTT = AVERAGE DAILY TRUCK TRAFFIC
BRG. = BEARING
CB-3 = CATCH BASIN TYPE 3
C/C = CENTER TO CENTER
EL = ELEVATION
EXP = EXPANSION JOINT
LT. = LEFT
N = NORMAL
O/O = OUT TO OUT
P.I. = POINT OF INTERSECTION
STA. = STATION
S.T. = SPIRAL TANGENT
T/T = TOE TO TOE
TOS = TOP OF SLOPE
V.C. = VERTICAL CURVE
V.P.I. = VERTICAL POINT OF INTERSECTION

BENCHMARK

BENCHMARK:

LOCATION

LATITUDE: N 39°42'49"
LONGITUDE: W 82°40'25"
USGS QUADRANGLE: AMANDA

TRAFFIC DATA

DELMONT ROAD:

2001 ADT = 919 ADTT = 9
2021 ADT = 1290 ADTT = 13

US 33:

2001 ADT NORTHBOUND = 9980 ADTT NORTHBOUND = 840
SOUTHBOUND = 9890 SOUTHBOUND = 840
2021 ADT NORTHBOUND = 13710 ADTT NORTHBOUND = 1300
SOUTHBOUND = 12900 SOUTHBOUND = 1300

PROPOSED STRUCTURE DATA

TYPE: FOUR-SPAN CONTINUOUS HPS 70W COMPOSITE STEEL GIRDER WITH SEMI-INTEGRAL TYPE ABUTMENTS, CAP AND COLUMN TYPE PIERS.
SPAN: 78'-6", 130'-9", 126'-9", 82'-6" C/C BEARINGS ALONG DELMONT ROAD.
ROADWAY: 36'-0" T/T PARAPET
SKEW: 39°17'06" L.F.
DESIGN LOADING: HS- 20 (CASE II) AND THE ALTERNATE MILITARY LOADING
WEARING SURFACE: MONOLITHIC CONCRETE
ALIGNMENT: TANGENT
APPROACH SLABS: AS-1-81 (25'-0" LONG)

DODSON-STILSON INC.
DESIGN AGENCY
ENGINEERS • ARCHITECTS • SCIENTISTS

DESIGNED | CHECKED | DRAWN | REVISED | REVIEWED | DATE
STRUCTURE FILE NUMBER 2301067

FAIRFIELD COUNTY
STA. 15+18.10
STA. 19+39.18

SITE PLAN

BRIDGE NO. FAI-33-1309
DELMONT ROAD OVER US33 BYPASS

FAI-33-7.08

Client: DSI	Project: FAI-33-09.95 (Delmont Road over 33 Mainline)	Job No. 9921-3024.00
LOG OF: Boring B-30	Location: Sta. 15+19, Centerline (Delmont Road)	Date Drilled: 1/10/00

WATER OBSERVATIONS:

Water seepage at: 16.0'

Water level at completion: 5.8' (includes drilling mud)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	903.8												
1.0	902.8						Topsoil - 12"						
		4 5 6	8	1		3.0	Very stiff black SILTY CLAY (A-6b), little fine to coarse sand, trace gravel; organic; moist.						
5		5 4 5	8	2		2.5							
5.5	898.3												
		5 7 13	18	3		4.5+	Hard brown SILT AND CLAY (A-6a), some fine to coarse sand, trace gravel; damp.	8	9	---	17	41	25
8.0	895.8												
10		12 13 15	18	4		4.5+	Hard mottled brown and gray SILT AND CLAY (A-6a), some fine to coarse sand, little gravel; damp.						
10.5	893.3												
		5 8 9	18	5		4.5+	Hard dark gray SILT AND CLAY (A-6a), some fine to coarse sand, little gravel; contains occasional sand seams; damp.						
15		6 9 14	16	6		4.5+							
		9 12 12	14	7		4.5+							
18.0	885.8												
20		24 14 32	18	8A 8B		4.5+ 4.5+	Hard brown SILT AND CLAY (A-6a), "and" fine to coarse sand, little gravel; damp to moist.						
20.5	883.3												
22.0	881.8	14 10 14	18	9A 9B			Medium dense brown GRAVEL WITH SAND AND SILT (A-2-4), trace clay; wet.						
23.5	880.3						Medium dense gray FINE SAND (A-3), trace silt and clay; wet.						
25	878.8	15 18 16	18	10			Dense gray COARSE AND FINE SAND (A-3a), trace silt and clay; wet.						

STANDARD PENETRATION (N)

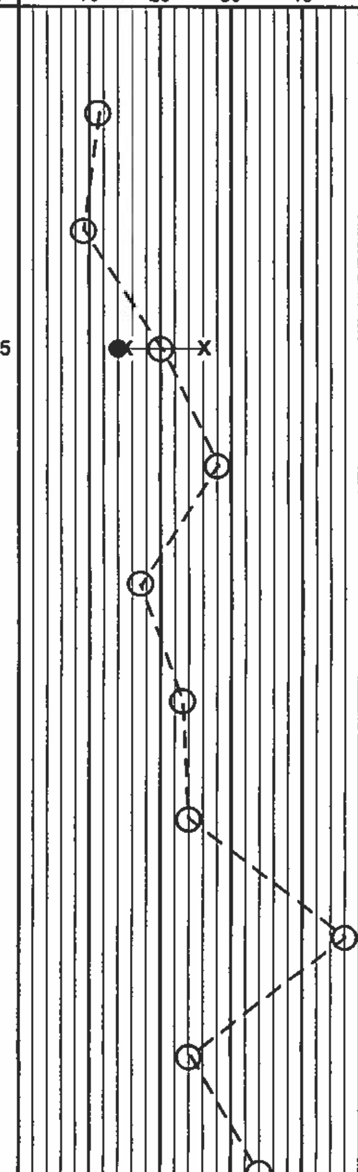
Blows per foot ○

10 20 30 40

Moisture Content - %

PL Natural LL

10 20 30 40



Client: DSI	Project: FAI-33-09.95 (Delmont Road over 33 Mainline)	Job No. 9921-3024.00
LOG OF: Boring B-30	Location: Sta. 15+19, Centerline (Delmont Road)	Date Drilled: 1/10/00

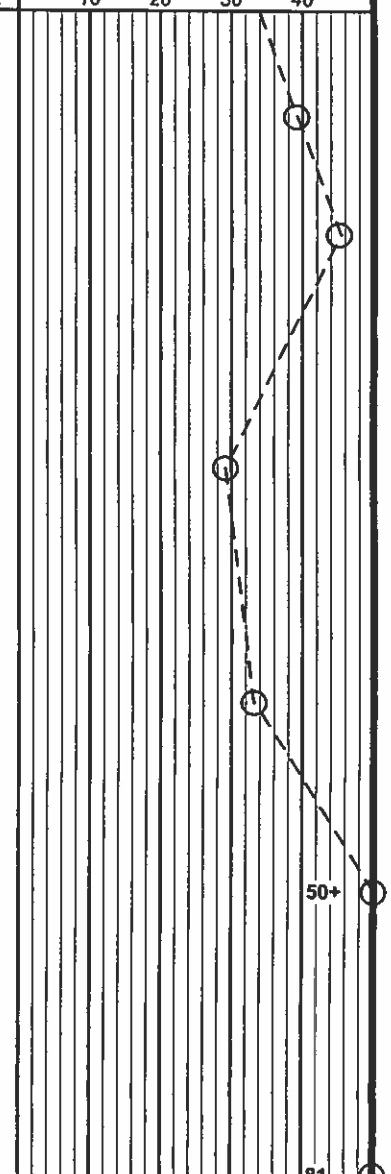
WATER OBSERVATIONS:

Water seepage at: 16.0'
Water level at completion: 5.8' (includes drilling mud)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
25	878.8						Dense brown GRAVEL WITH SAND AND SILT (A-2-4), trace clay wet.						
		9 16 23	18	11									
							@ 28.5', Driller's note: 1.5 feet of heave encountered.						
30		16 24 21	18	12			@ 30.0', Driller's note: began using mud while drilling.						
32.0	871.8						Medium dense to dense brown COARSE AND FINE SAND (A-3a), trace silt and clay; wet.						
35		12 15 14	18	13									
40		10 15 18	18	14									
							@ 42.0', Drillers note: auger refusal - began coring. Cored 1 foot through limestone boulder.						
		50/4	4	15			@ 43.5', limestone fragments - possible limestone cobbles or boulders.						
45													
50	853.8	40 46 35	14	16A 16B		2.5	@ 49.0', gray clay seam.						

STANDARD PENETRATION (N)
Blows per foot
10 20 30 40

Moisture Content - %
PL Natural LL
10 20 30 40



Client: DSI	Project: FAI-33-09.95 (Delmont Road over 33 Mainline)	Job No. 9921-3024.00
LOG OF: Boring B-30	Location: Sta. 15+19, Centerline (Delmont Road) Date Drilled: 1/10/00	

WATER OBSERVATIONS:
Water seepage at: 16.0'
Water level at completion: 5.8' (includes drilling mud)

STANDARD PENETRATION (N) — Blows per foot: 10, 20, 30, 40
Moisture Content - %: PL, Natural, LL: 10, 20, 30, 40

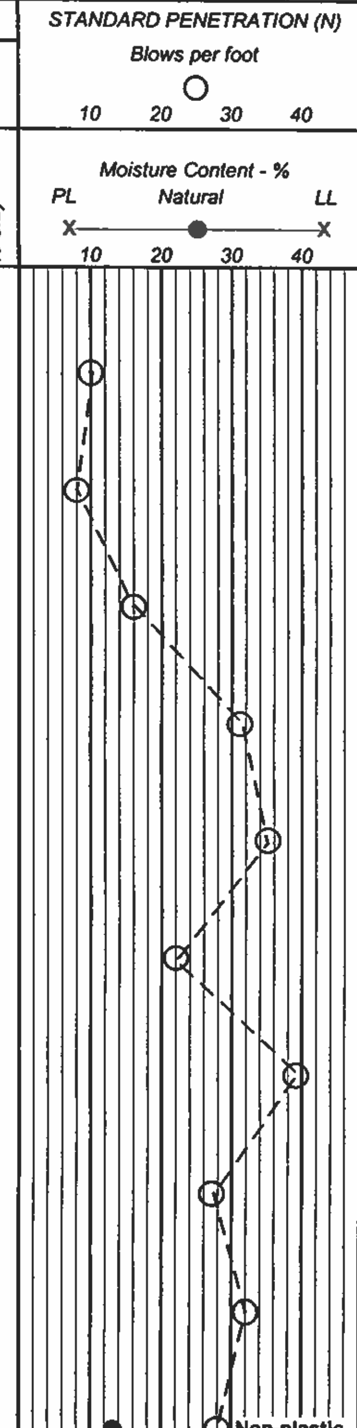
Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	Standard Penetration (N)
50	853.8						Dense brown COARSE AND FINE SAND (A-3a), trace silt clay; wet.							
51.0	852.8	50/1	1	17			Medium hard light brown to brown SANDSTONE; slightly weathered; broken.							50+
		Core 64"	Rec 64"	RQD 30%			@ 52.8', 53.2', 55.5', healed fractures.							
55														
56.3	847.5						Bottom of Boring - 56.3'							
60														
65														
70														
75														

Client: DSI | **Project:** FAI-33-09.95 (Delmont Road over 33 Mainline) | **Job No.** 9921-3024.00

LOG OF: Boring B-31 | **Location:** Sta. 15+98, Centerline (Delmont Road) | **Date Drilled:** 1/10/00 and 1/11/00

WATER OBSERVATIONS:
Water seepage at: 15.0'
Water level at completion: 4.2' (includes drilling mud)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press	Hand Penetro-meter (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	903.3												
1.0	902.3						Topsoil - 12"						
		5 5 5	12	1		2.5	Stiff to very stiff black SILTY CLAY (A-6b), trace fine to coarse sand, trace gravel; organic; moist.						
		3 4 4	6	2		2.0							
5 / 5.5	897.8												
		3 4 12	6	3		2.5	Stiff to very stiff mottled brown and gray SILTY CLAY (A-6b), some fine to coarse sand, little gravel; slightly organic; moist.						
		31 17 14	3	4		2.0							
10 / 10.5	892.8												
		14 14 21	18	5		3.5	Very stiff dark gray SILT AND CLAY (A-6a), some fine to coarse sand, little gravel; moist.						
13.0	890.3												
15		17 14 8	18	6			Medium dense brown GRAVEL WITH SAND AND SILT (A-2-4), trace clay; wet. @ 13.5', Driller's note: began using mud while drilling.						
15.5	887.8												
		11 18 21	14	7			Dense dark gray FINE SAND (A-3), trace silt and clay; wet.						
18.0	885.3												
20		8 12 15	16	8			Medium dense to dense brown COARSE AND FINE SAND (A-3a), trace silt and clay, trace gravel; wet. @ 18.5', Driller's note: 6 inches of heave encountered. @ 21.5', Driller's note: 6 inches of heave encountered.						
		7 14 18	18	9									
28.0	875.3												
25	878.3	10 14 14	18	10			Medium dense brown GRAVEL WITH SAND (A-1-b), trace silty clay; wet.	39	37	—	16	8	



Client: DSI

Project: FAI-33-09.95 (Delmont Road over 33 Mainline)

Job No. 9921-3024.00

LOG OF: Boring B-31

Location: Sta. 15+98, Centerline (Delmont Road)

Date Drilled: 1/10/00 and 1/11/00

WATER OBSERVATIONS:

Water seepage at: 15.0'

Water level at completion: 4.2' (includes drilling mud)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
25	878.3												
		6 14 21	18	11			Dense brown GRAVEL WITH SAND (A-1-b), trace silt and clay; wet.						
							@ 28.0', becomes very dense.						
30		18 24 31	18	12									
32.0	871.3												
35		8 20 19	18	13			Dense brown GRAVEL WITH SAND AND SILT (A-2-4), trace clay; wet.						
37.0	866.3												
40		12 20 21	18	14			Dense brown GRAVEL WITH SAND (A-1-b), trace silt and clay; wet.						
45		17 29 23	18	15									
		50/3	1	16									
49.5	853.8						Note: Bedrock encountered at 49.5'. See discription on next page.						
50	853.3												

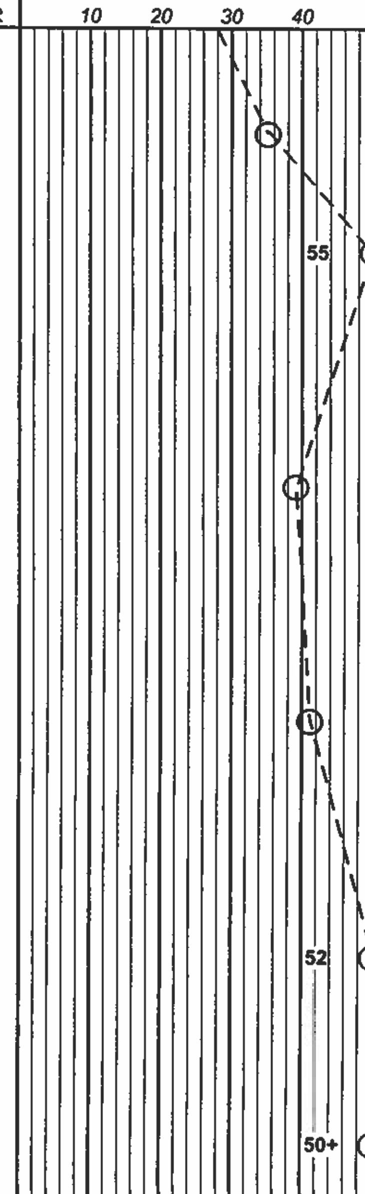
STANDARD PENETRATION (N)

Blows per foot

10 20 30 40

Moisture Content - %

PL Natural LL



Client: DSI		Project: FAI-33-09.95 (Delmont Road over 33 Mainline)	Job No. 9921-3024.00
LOG OF: Boring B-31	Location: Sta. 15+98, Centerline (Delmont Road)	Date Drilled: 1/10/00 and 1/11/00	

WATER OBSERVATIONS:
Water seepage at: 15.0'
Water level at completion: 4.2' (includes drilling mud)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Blows per foot 10 20 30 40 / Moisture Content - % PL Natural LL 10 20 30 40
50	853.3													
		Core 60"	Rec 37"	RQD 37%			Medium hard light gray SANDSTONE; slightly weathered; broken. @ 52.0', becomes light brown.							
54.5	848.8													
55							Bottom of Boring - 54.5'							
60														
65														
70														
75														

Client: DSI

Project: FAI-33-09.95 (Delmont Road over 33 Mainline)

Job No. 9921-3024.00

LOG OF: Boring B-32

Location: Sta. 17+29, Centerline (Delmont Road)

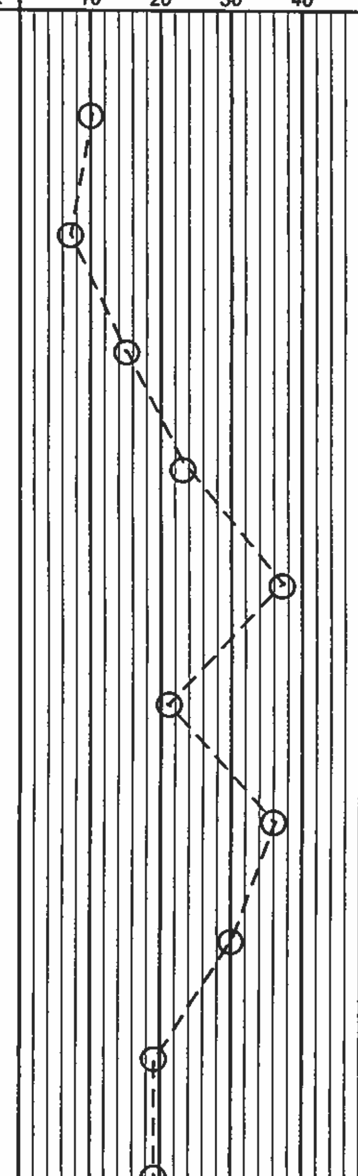
Date Drilled: 1/11/00 and 1/12/00

WATER OBSERVATIONS:
Water seepage at: 13.5'
Water level at completion: 6.0' (includes drilling mud)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press	Hand Penetro-meter (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	905.1												
1.0	904.1						Topsoil - 12"						
		4 / 5 / 5	16	1		2.5	Stiff to very stiff light brown SILT AND CLAY (A-6a), little to some fine to coarse sand, trace gravel; moist.						
5		3 / 3 / 4	8	2		1.5							
5.5	899.6												
		6 / 7 / 8	11	3		4.0	Very stiff to hard brown SILT AND CLAY (A-6a), some fine to coarse sand, trace gravel; contains sand seams; moist.						
8.0	897.1												
10		5 / 11 / 12	18	4		4.5+	Hard dark gray SILT AND CLAY (A-6a), some fine to coarse sand, little gravel; damp.						
10.5	894.6												
		12 / 17 / 20	16	5			Dense brown GRAVEL WITH SAND AND SILT (A-2-4), trace clay; wet.						
13.0	892.1												
15		14 / 11 / 10	6	6			Medium dense brown SANDY SILT (A-4a), trace clay, trace gravel; moist to wet. @ 13.5', Driller's note: began using mud while drilling.						
		19 / 20 / 16	6	7									
18.0	887.1												
20		14 / 16 / 14	12	8			Medium dense to dense brown GRAVEL WITH SAND (A-1-b), trace silt and clay; wet.						
20.5	884.6												
		10 / 9 / 10	18	9			Medium dense dark gray FINE SAND (A-3), trace silt and clay; wet.						
23.0	882.1												
25	880.1	8 / 9 / 10	18	10			Medium dense brown GRAVEL WITH SAND AND SILT (A-2-4), trace clay; wet.						

STANDARD PENETRATION (N)
Blows per foot
10 20 30 40

Moisture Content - %
PL Natural LL
10 20 30 40



Client: DSI | **Project:** FAI-33-09.95 (Delmont Road over 33 Mainline) | **Job No.** 9921-3024.00

LOG OF: Boring B-32 | **Location:** Sta. 17+29, Centerline (Delmont Road) | **Date Drilled:** 1/11/00 and 1/12/00

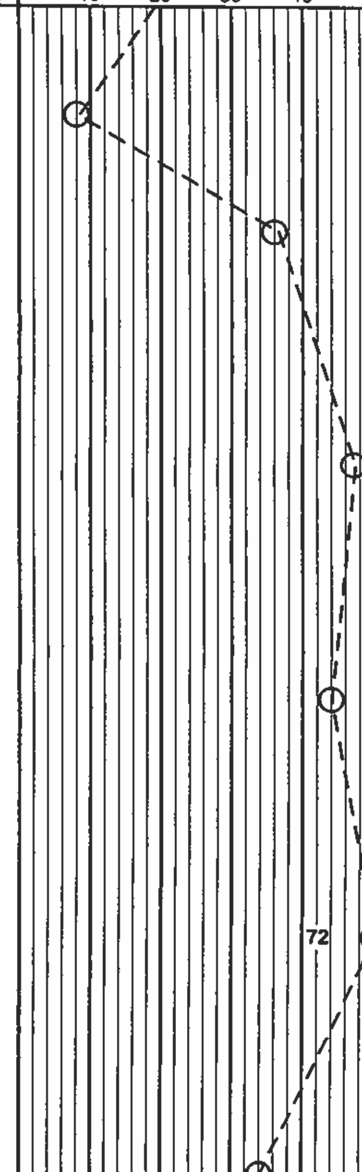
WATER OBSERVATIONS:
Water seepage at: 13.5'
Water level at completion: 6.0' (includes drilling mud)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
25	880.1												
		9 3 5	18	11			Loose brown SANDY SILT (A-4a), trace clay, trace gravel; wet.						
28.0	877.1												
30		16 14 22	18	12			Dense brown GRAVEL WITH SAND (A-1-b), trace silt and clay; wet.						
32.0	873.1												
35		17 21 26	18	13			Dense dark gray FINE SAND (A-3), trace silt and clay; wet.						
37.0	868.1												
40		12 19 25	18	14			Dense brown GRAVEL WITH SAND AND SILT (A-2-4), trace clay; wet.						
45		45 40 32	18	15			@ 45.0', silt and clay seam, limestone fragments.						
47.0	858.1												
50	855.1	14 15 19	18	16			Dense brown GRAVEL WITH SAND (A-1-b), trace silt and clay; wet.						

GRADATION

STANDARD PENETRATION (N)
Blows per foot
10 20 30 40

Moisture Content - %
PL Natural LL
10 20 30 40



Client: DSI | **Project:** FAI-33-09.95 (Delmont Road over 33 Mainline) | **Job No.** 9921-3024.00

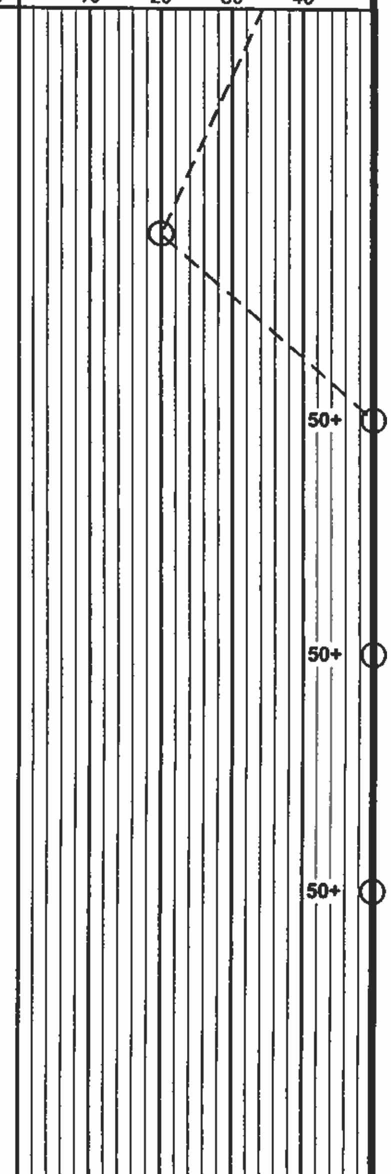
LOG OF: Boring B-32 | **Location:** Sta. 17+29, Centerline (Delmont Road) | **Date Drilled:** 1/11/00 and 1/12/00

WATER OBSERVATIONS:
Water seepage at: 13.5'
Water level at completion: 6.0' (includes drilling mud)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. (Drive)	Sample No. (Press)	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
50	855.1												
52.0	853.1						Dense brown GRAVEL WITH SAND (A-1-b), trace silt and clay; wet.						
55		10 7 13	18	17			Medium dense brown COARSE AND FINE SAND (A-3a), trace silt and clay; contains sandstone fragments; wet.						
		50/3	3	18			@ 58.5', limestone fragments, possible limestone cobbles or boulders.						
60													
62.0	843.1												
		50/4	4	19			Very soft brown SANDSTONE; fine to medium grained; severely weathered and deteriorated; very broken.						
65													
		50/2	2	20									
70													
72.5	832.6												
75	830.1						Medium hard dark brown SANDSTONE; slightly weathered; occasional gray clay seams; broken. @ 73.4' - 74.1', becomes light brown. @ 74.1' - 76.0', becomes dark brown.						

STANDARD PENETRATION (N)
Blows per foot ○
10 20 30 40

Moisture Content - %
PL Natural LL
X———●———X
10 20 30 40



Client: DSI	Project: FAI-33-09.95 (Delmont Road over 33 Mainline)	Job No. 9921-3024.00
LOG OF: Boring B-32	Location: Sta. 17+29, Centerline (Delmont Road)	Date Drilled: 1/11/00 and 1/12/00

WATER OBSERVATIONS:

Water seepage at: 13.5'

Water level at completion: 6.0' (includes drilling mud)

STANDARD PENETRATION (N) Blows per foot: 10 20 30 40

Moisture Content - %: PL, Natural, LL: 10 20 30 40

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
75	830.1												
		Core 60"	Rec 60"	RQD 50%			Medium hard dark brown SANDSTONE; slightly weathered; occasional gray clay seams; broken. @ 76.0' - 77.5', becomes light brown.						
77.5	827.6												
							Bottom of Boring - 77.5'						
80													
85													
90													
95													
100													

Client: DSI

Project: FAI-33-09.95 (Delmont Road over 33 Mainline)

Job No. 9921-3024.00

LOG OF: Boring B-33

Location: Sta. 18+55, Centerline (Delmont Road)

Date Drilled: 1/13/00

WATER OBSERVATIONS:

Water seepage at: 14.7'

Water level at completion: 6.0' (includes drilling mud)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press	Hand Penetrometer (tsf)	DESCRIPTION
0	914.3						
0.3	914.0						Topsoil - 3"
		4 / 7 / 7	12	1		2.5	Very stiff to hard brown SILTY CLAY (A-6b), little to some fine to coarse sand, trace to little gravel; damp to moist.
5		13 / 15 / 20	16	2		4.5+	
		6 / 11 / 13	11	3		4.5+	
8.0	906.3						
10		9 / 14 / 14	15	4		4.5+	Hard gray SILTY CLAY (A-6b), little to some fine to coarse sand, trace to little gravel; damp to moist.
		7 / 8 / 11	18	5		4.5	
							@ 13.0', becomes stiff.
15		5 / 6 / 7	14	6		1.5	@ 14.5', coarse and fine sand seam.
15.5	898.8						
		7 / 16 / 19	12	7			Dense gray GRAVEL WITH SAND (A-1-b), trace silt and clay; wet.
							@ 17.5', Driller's note: began using mud while drilling.
20		11 / 17 / 15	18	8			
		17 / 15 / 16	5	9			
25	889.3	10 / 16 / 24	18	10			

GRADATION					
% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay

STANDARD PENETRATION (N)

Blows per foot ○

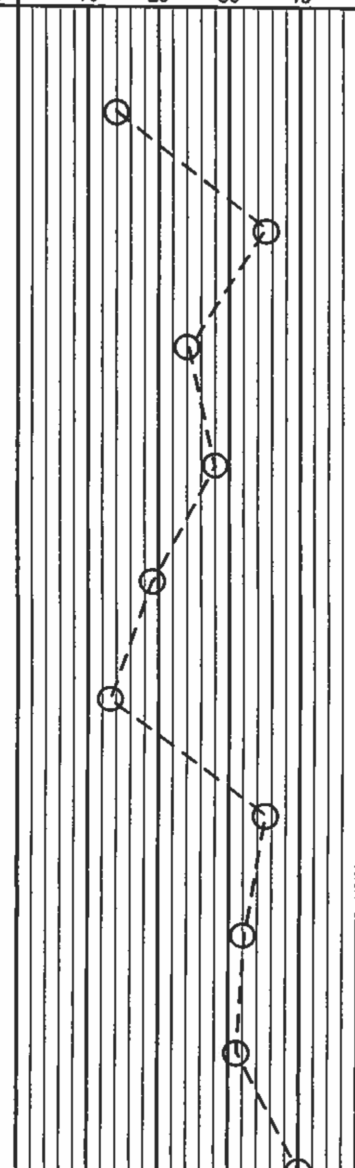
10 20 30 40

Moisture Content - %

PL Natural LL

X——●——X

10 20 30 40



Client: DSI | **Project:** FAI-33-09.95 (Delmont Road over 33 Mainline) | **Job No.** 9921-3024.00

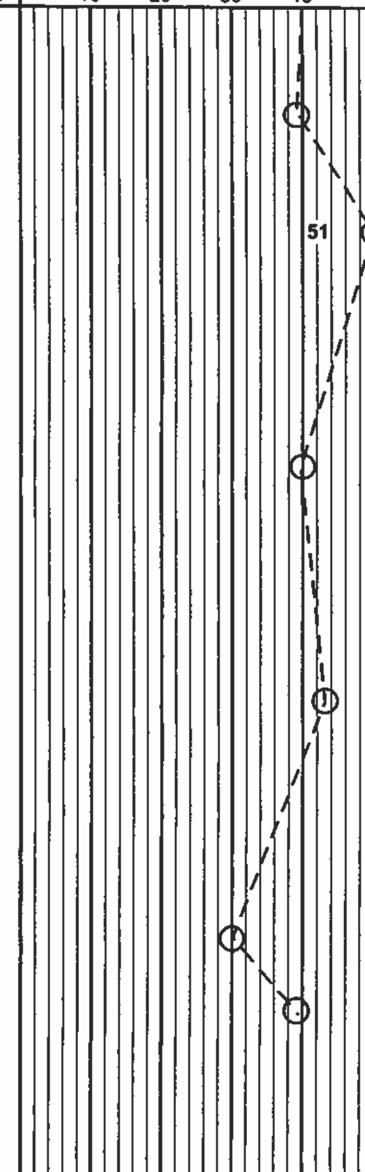
LOG OF: Boring B-33 | **Location:** Sta. 18+55, Centerline (Delmont Road) | **Date Drilled:** 1/13/00

WATER OBSERVATIONS:
Water seepage at: 14.7'
Water level at completion: 6.0' (includes drilling mud)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press	Hand Penetro-meter (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
25	889.3												
		15 19 20	18	11		---	Medium stiff to stiff gray SILTY CLAY (A-6b), some fine to coarse sand, little gravel; moist. @ 26.0', Driller's note: 2 feet of heave encountered.						
28.0	886.3												
30		14 20 31	18	12			Dense to very dense brown GRAVEL WITH SAND (A-1-b), trace silt and clay; wet.						
35		21 22 18	18	13									
40		21 21 22	18	14									
42.0	872.3												
45		9 13 17	15	15		1.5	Medium stiff to stiff gray SILTY CLAY (A-6b), little fine to coarse sand, trace gravel; moist.						
46.5	867.8	12 17 22	16	16		1.0							
							Bottom of Boring - 46.5'						
50													

STANDARD PENETRATION (N)
Blows per foot
10 20 30 40

Moisture Content - %
PL Natural LL
10 20 30 40



Client: DSI | **Project:** FAI-33-09.95 (Delmont Road over 33 Mainline) | **Job No.** 9921-3024.00

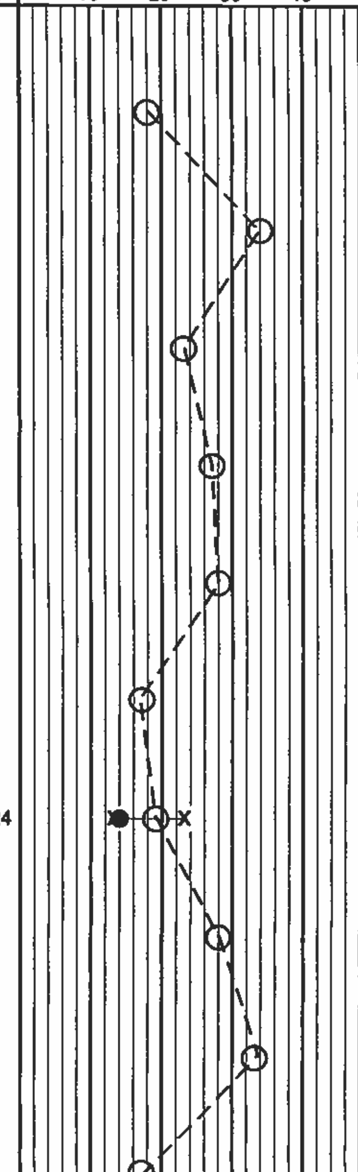
LOG OF: Boring B-34 | **Location:** Sta. 19+38, Centerline (Delmont Road) | **Date Drilled:** 1/13/00 and 1/14/00

WATER OBSERVATIONS:
Water seepage at: 7.2', 7.3', 21.7'
Water level at completion: 6.0' (includes drilling mud)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press	Hand Penetro-meter (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	915.3												
0.5	914.8						Topsoil - 6"						
		3, 4, 14	14	1		3.75	Very stiff to hard brown SILTY CLAY (A-6b), some fine to coarse sand, little gravel; contains trace sandstone fragments; damp to moist.						
5		12, 15, 19	17	2		4.5+							
		13, 10, 13	18	3		4.5							
8.0	907.3						Very stiff to hard gray SILTY CLAY (A-6b), some fine to coarse sand, little gravel; contains trace sandstone fragments; damp to moist.						
10		11, 13, 14	17	4		2.75							
		7, 11, 17	12	5		4.5							
							@ 13.0', becomes stiff to very stiff.						
15		6, 7, 10	18	6		1.5							
15.5	899.8						Medium dense gray SANDY SILT (A-4a), some clay, trace to little gravel; moist.						
		7, 10, 9	18	7		—		9	10	---	19	38	24
20		7, 14, 14	11	8		—							
							@ 20.5', becomes dense.						
		10, 15, 18	14	9		—							
23.0	892.3						Medium dense brown GRAVEL WITH SAND (A-1-b), trace silt and clay; wet.						
25	890.3	9, 8, 9	16	10									

STANDARD PENETRATION (N)
Blows per foot
10 20 30 40

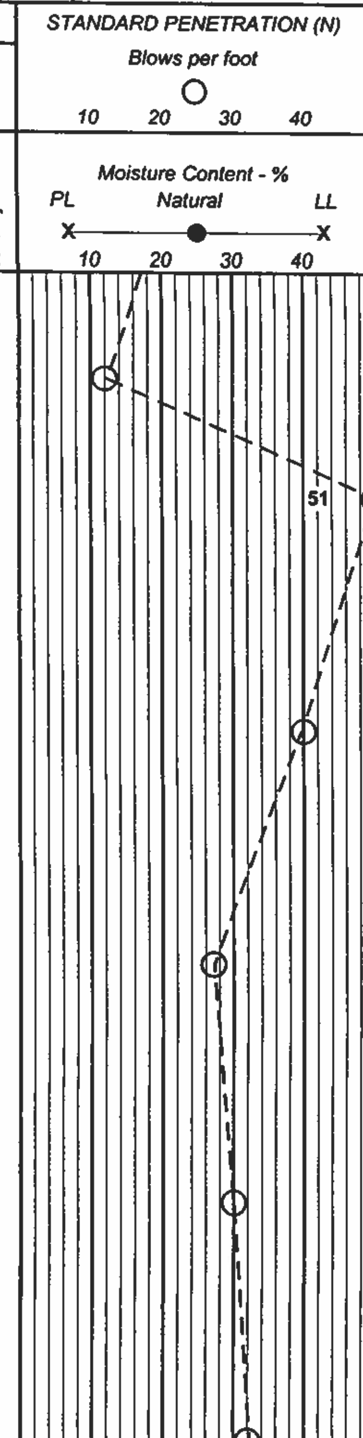
Moisture Content - %
PL Natural LL
10 20 30 40



Client: DSI	**Project:** FAI-33-09.95 (Delmont Road over 33 Mainline)	**Job No.** 9921-3024.00
LOG OF: Boring B-34	**Location:** Sta. 19+38, Centerline (Delmont Road)	**Date Drilled:** 1/13/00 and 1/14/00

WATER OBSERVATIONS:
Water seepage at: 7.2', 7.3', 21.7'
Water level at completion: 6.0' (includes drilling mud)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. (Drive)	Sample No. (Press)	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
25	890.3						Medium dense brown GRAVEL WITH SAND (A-1-b), trace silt and clay; wet.						
		14 / 6 / 6	18	11			@ 25.0', Driller's note: began using mud while drilling.						
28.0	887.3						Very dense gray FINE SAND (A-3), trace silt and clay, trace gravel; wet.						
30		16 / 25 / 26	18	12									
32.0	883.3						Medium dense to dense gray GRAVEL WITH SAND (A-1-b), trace silt and clay; wet.						
35		14 / 19 / 21	18	13									
40		11 / 9 / 18	18	14									
42.0	873.3						Very stiff gray SILTY CLAY (A-6b), little fine to coarse sand, trace gravel; moist.						
45		9 / 13 / 17	18	15		3.75							
							@ 47.0', becomes stiff.						
50	865.3	17 / 17 / 15	18	16A / 16B		1.5 / —							



Client: DSI

Project: FAI-33-09.95 (Delmont Road over 33 Mainline)

Job No. 9921-3024.00

LOG OF: Boring B-34

Location: Sta. 19+38, Centerline (Delmont Road)

Date Drilled: 1/13/00 and 1/14/00

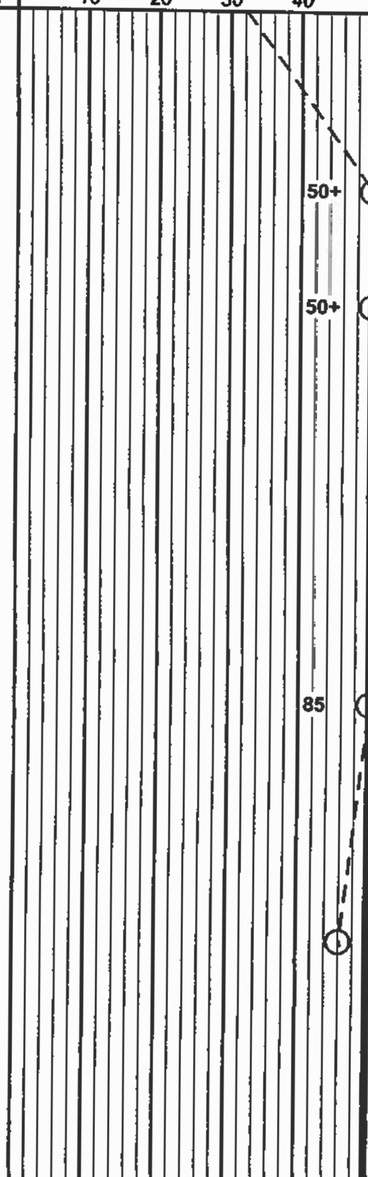
WATER OBSERVATIONS:

Water seepage at: 7.2', 7.3', 21.7'
Water level at completion: 6.0' (includes drilling mud)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
50	865.3						Dense to very dense gray SILT (A-4b), trace fine to coarse sand, trace clay; wet.						
		50/3	3	17			@ 53.5' - 56.0', limestone and sandstone fragments in samples; possible limestone and sandstone cobbles or boulders.						
55													
		50/2	2	18			@ 56.0', Drillers note: auger refusal, began coring. Cored 1 foot through sandstone boulder.						
60	855.3						Very dense gray GRAVEL WITH SAND (A-1-b), trace silt and clay; wet.						
65		46 40 45	16	19									
67.0	848.3						Dense gray COARSE AND FINE SAND (A-3a), trace silt and clay, trace gravel; wet.						
70	845.3	12 17 29	18	20			Bottom of Boring - 70.0'						
75													

STANDARD PENETRATION (N)
Blows per foot: 10 20 30 40

Moisture Content - %: PL, Natural, LL: 10 20 30 40



FINAL
STRUCTURE FOUNDATION EXPLORATION REPORT
FOR
CUY-271-0.00 - RETAINING WALLS
(PID NO. 80418)
SUMMIT/CUYAHOGA COUNTIES, OHIO
PGI PROJECT NOS. G13005G & G14004G

PREPARED FOR:

DLZ OHIO, INC.

PREPARED BY:

PRO GEOTECH, INC.

AUGUST 18, 2014

1.0 EXECUTIVE SUMMARY

This report has been prepared for the CUY/SUM-271-00.00/14.87 and CUY/SUM-480-29.58/00.00 project which calls for design and construction of three (3) new retaining walls indentified as RW-1 (WS1), RW-2 (SW1), and RW-3 (WS2) in Summit/Cuyahoga Counties, Ohio. These proposed retaining walls are to be constructed in association with the construction of two additional Lanes identified as S-W and W-S located along the outside shoulders of IR-271 SB and NB between the Summit County Line and Alexander Road. A total of twelve (12) test borings were advanced for retaining wall foundation design purposes. Two (2) test borings identified as B-007-1-13 and B-007-4-13 were advanced in the vicinity of proposed RW-1 to approximate depths of 50.0 feet and 63.9 feet below the existing ground surface. Two (2) test borings identified as B-007-2-13 and B-007-3-13 were advanced in the vicinity of proposed RW-2 to approximate depths of 38.9 feet and 48.9 feet below the existing ground surface. One (1) additional test boring identified as B-007-5-14 was drilled in the vicinity of proposed RW-2 to an approximate depth of 40.0 feet below the existing ground surface for Soil Nail Wall design purposes. Test boring B-007-5-14 was drilled by DLZ, laboratory testing of the soil samples, and preparation of the boring log was performed by DLZ. Eight (8) test borings identified as B-047-0-13 through and B-054-0-13 were advanced in the vicinity of proposed RW-3 to approximate depths ranging from 26.0 feet to 60.0 feet below the existing ground surface.

Retaining Wall RW-1: The subsurface soils encountered in both test borings were predominantly cohesive in nature and consisted of both fill materials and natural soils. The fill materials encountered above the natural soils consisted of silt and clay (A-6a) and silty clay (A-6b). The approximate thickness of the fill material was 8.5 feet in test boring B-007-1-13 and 3.5 feet in test boring B-007-4-13. Natural soils encountered above bedrock in test boring B-007-4-13 and to the termination depth in test boring B-007-1-13 consisted of sandy silt (A-4a), silt and clay (A-6a), non-plastic sandy silt (A-4a), and coarse and fine sand (A-3a). Bedrock consisting of gray, severely to highly weathered shale was encountered at an approximate depth of 59.8 feet in test boring B-007-4-13. The consistency of the cohesive soils ranged from "medium stiff" to "hard", but was generally "very stiff". The relative density of the non-cohesive soils ranged from “dense” to "very dense".

Retaining Wall RW-2: The subsurface soils encountered in all test borings were predominantly cohesive in nature and consisted of both fill materials and natural soils. The fill materials encountered above the natural soils consisted of silt and clay (A-6a), sandy silt (A-4a), silty clay (A-6b), and gravel with sand

(A-1-b). The approximate thicknesses of the fill materials range from 3.5 feet in test borings B-007-2-13 and B-007-3-13 to 31 feet in test boring B-007-5-14. Natural soils encountered above bedrock in the test borings consisted of silt and clay (A-6a), sandy silt (A-4a), silty clay (A-6b), non-plastic sandy silt (A-4a), and coarse and fine sand (A-3a). Bedrock consisted of gray, severely to highly weathered shale at approximate depths of 39.0 feet in test boring B-007-2-13 and 34.0 feet in test boring B-007-3-13. The consistency ranged from "medium stiff" to "hard", but was generally "very stiff" and the relative density was ranged from "loose" to "dense".

<u>Retaining Wall RW-3:</u> The subsurface soils encountered in the test borings were predominantly cohesive in nature and consisted of both fill materials and natural soils with the exception of B-049-0-13 which consisted entirely of natural soil. The fill materials encountered above the natural soils consisted of sandy silt (A-4a), silt and clay (A-6a) and silty clay (A-6b). The approximate thickness of the fill materials ranged from 3.5 feet to 6 feet and averaged 4.6 feet. Natural soils consisted of stone fragments with sand (A-1-b), coarse and fine sand (A-3a), both plastic and non-plastic sandy silt (A-4a), both plastic and non-plastic silt (A-4b), silt and clay (A-6a), and silty clay (A-6b). All of the test borings were terminated in natural soils, with the exception of B-054-0-13 which was terminated in bedrock consisting of gray, severely weathered shale at an approximate depth of 53.5 feet. Additionally, test boring B- 053-0-13 was terminated at auger refusal at 26.0 feet on what may have been a boulder. The consistency ranged from "very soft" to "hard", but was generally "very stiff" to "hard". The relative density ranged from "loose" to "very dense".

Retaining Wall Foundation Systems

<u>Retaining Wall RW-1</u>: Design information provided by DLZ personnel indicates that a Semi-Gravity Type Wall System will be used to retain the soils at the RW-1 location. Fill soils were encountered to the depth of 8.5 feet (at elevation of 1031.6 feet) in test boring B-007-1-13 and to the depth of 3.5 feet (at elevation of 1031.9 feet) in test boring B-007-4-13. The consistency of these fill cohesive soils encountered above the natural soils ranged from "medium stiff" to "stiff" and will not support the applied loads from the retaining wall. Therefore, PGI recommends performing ground improvements on the foundation soils in the vicinity of these test boring locations. Ground improvements should be performed by removing all fill cohesive soils below the bottom of the RW-1 footing and replacing it with compacted engineered non-expansive fill material. The engineered fill material should be well-graded granular material (ODOT 304 Limestone Aggregate) and compacted to 100% of the standard proctor dry density

2.0 INTRODUCTION

This report has been prepared for the CUY/SUM-271-00.00/14.87 and CUY/SUM-480-29.58/00.00 project which calls for design and construction of three (3) new retaining walls identified as RW-1 (WS1), RW-2 (SW1), and RW-3 (WS2) in Summit and Cuyahoga Counties, Ohio. These proposed retaining walls are to be constructed in association with the construction of two additional Lanes identified as S-W and W-S located along the outside shoulders of IR-271 SB and NB between the Summit County Line and Alexander Road. It represents the intent of DLZ Ohio, Inc. (DLZ), the design engineer, and the Ohio Department of Transportation (ODOT), the owner, to secure subsurface information at the selected locations in accordance with ODOT's *Specifications for Geotechnical Exploration*s, and to obtain recommendations regarding geotechnical factors pertaining to design and construction of this project.

2.1 Project Description

Present plans call for design and construction of three (3) new retaining walls along the outside shoulders of IR-271 SB and NB in the vicinity of the Summit and Cuyahoga County line. The design information provided by DLZ personnel indicates that the first proposed retaining wall identified as RW-1 (WS1) will be constructed between IR-271 SB mainline and proposed Lane W-S, starting at Station 3243+75 and ending at Station 3247+25 of the Lane W-S. The proposed RW-1 structure is expected to be Semi-Gravity Wall System Type and will be 350 feet in length and 16 feet in maximum height. The second proposed retaining wall identified as RW-2 (SW1) will be located between the proposed Lane S-W and the existing IR-480 EB flyover bridge rear abutment, starting at Station 3046+75 and ending at Station 3048+75 of the Lane S-W. The proposed RW-2 structure is expected to be Soil Nail Wall System Type and will be 200 feet in length and 10 feet in maximum height. The third proposed retaining wall identified as RW-3 (WS2) will be located between the proposed Lane W-S and the existing Oak Leaf Road, starting at Station 3251+25 and ending at Station 3262+00. The proposed RW-3 structure is expected to be Soldier Pile Wall System Type and will be 1075 feet in length and 8 feet in maximum height. The Site Location Map is indicated in Figure 2.1.

This report has been developed based on the field exploration program, laboratory testing, and information secured for site-specific studies. It must be noted that, as with any geotechnical exploration program, the site exploration identifies actual subsurface conditions only at those locations where samples were obtained. The data derived through sampling and subsequent laboratory testing was reduced by geotechnical engineers and geologists who then rendered an opinion regarding the overall subsurface

Specifications for Geotechnical Explorations. The groundwater conditions were monitored during and upon completion of the drilling operations.

Task III - Testing Program, which consisted of performing soil classification and engineering properties tests on selected soil samples, and classifying the soils in accordance with the ODOT Soil Classification System.

Task IV - Subsurface Exploration Report, which included the following:

- A brief description of the project and our exploration methods
- Geology of the site
- Typed drilling logs and laboratory test results
- A description of subsurface soil, rock, and groundwater conditions
- Recommendations for foundation design for each retaining wall including shallow and deep foundations, earthwork considerations, groundwater management, and construction monitoring
- Provide Soil Parameters for lateral load analysis is to be performed by others
- Preparation of Geotechnical Design Checklist
- Subsurface Exploration Plans

The scope of services did not include any environmental assessments for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater or air, on, below, or around this site. Any statement in this report or on the boring logs regarding odors, colors or unusual or suspicious items or conditions is strictly for the client's information.

5.2 Groundwater Conditions

The groundwater levels were measured in all of the test boring locations during drilling and upon completion of drilling operations. The results of these measurements are summarized in Table 5.2.1. It should be noted that groundwater elevations are subject to seasonal fluctuations. Groundwater monitoring wells are essential to accurately define the position of the groundwater table; however, installation of monitoring wells was not included in our scope of services. All test borings were backfilled upon completion for safety purposes. Note that test boring B-049-0-13 advanced for RW-3 was left open in order to take an extended groundwater level reading. However, after 72 hours the boring location was under water due to surface runoff from heavy rains and snow melt and an extended groundwater reading was not made.

Table 5.2.1 – Groundwater Information

Boring Number	Surface Elevation (ft.)	Groundwater Depth (ft.)		Groundwater Elevation (ft.)	
		D.D.	U.C.	D.D.	U.C.
Retaining Wall RW-1					
B-007-1-13	1,040.0	23.5	20.0	1,016.5	1,020.0
B-007-4-13	1,035.4	18.5	26.0	1,016.9	1,009.4
Retaining Wall RW-2					
B-007-2-13	1,040.8	23.5	16.0	1,017.3	1,024.8
B-007-3-13	1,040.7	18.5	24.5	1,022.2	1,016.2
B-007-5-14	1,067.3	37.0	Dry	1,030.3	Dry
Retaining Wall RW-3					
B-047-0-13	1039.7	18.5	16.0	1021.2	1023.7
B-048-0-13	1045.5	28.5	15.0	1017.0	1030.5
B-049-0-13	1046.1	28.5	NR	1017.6	NR
B-050-0-13	1045.1	28.5	30.0	1015.5	1015.1
B-051-0-13	1044.0	28.5	26.0	1025.5	1018.0
B-052-0-13	1044.0	18.5	Dry	1015.6	Dry
B-053-0-13	1047.0	Dry	Dry	Dry	Dry
B-054-0-13	1043.6	28.5	Dry	1015.1	Dry

Note: Stations, Offsets, and Elevations for RW-1 and RW-2 test borings were provided by DLZ. Survey Information for RW-3 test borings has not been provided. DD = During Drilling, UC = Upon Completion, NR = No Reading, NA = Not Available

6.0 ANALYSIS AND RECOMMENDATIONS

Based upon the findings of the field exploration program, laboratory testing, and subsequent engineering analysis, the following sections have been prepared to address the geotechnical aspects related to the design and construction of the proposed Retaining Walls RW-1, RW-2, and RW-3. The foundation recommendations are provided in accordance with the ODOT *Bridge Design Manual* issued in 2007 using *LRFD and ASD Design Specifications*.

6.1 Retaining Wall Foundation Systems

Retaining Wall RW-1: Soil information obtained from test borings B-007-1-13 and B-007-4-13 was used to design the foundation system at RW-1. Design information provided by DLZ personnel indicates that a Semi-Gravity Type Wall System will be used to retain the soils at RW-1 location. The maximum wall height will be approximately 16.0 feet. This wall system consists of a reinforced concrete cantilever wall which relies on self-weight and bending action of the wall stem in order to resist lateral earth pressures. The site plans provided by DLZ indicates that the width of the retaining wall footing will be 9.5 feet and bearing elevation of the footing bottom will range from 1036.5 feet to 1037.0 feet. As outlined in Section 5.1, "Subsurface Soil Conditions", subsurface soils encountered in both test borings were generally cohesive in nature and consisted of both fill materials and natural soils. Fill soils were encountered to the depth of 8.5 feet (elevation 1031.6 feet) in test boring B-007-1-13 and to the depth of 3.5 feet (elevation 1031.9 feet) in test boring B-007-4-13. The consistency of these fill cohesive soils encountered above the natural soils ranged from “medium stiff” to “stiff” and will not support the applied loads from the retaining wall. Therefore, PGI recommends performing ground improvement on the foundation soils at the RW-1 in the vicinity of these test boring locations. Ground improvements should be performed by removing all fill cohesive soils below the bottom of the RW-1 footing and replacing it with compacted engineered non-expansive fill material. The engineered fill material should be well-graded granular material (ODOT 304 limestone Aggregate) and compacted to 100% of the standard proctor dry density of that material. The granular pad should be extended at least 2 feet beyond the proposed retaining wall footing perimeter.

Bearing capacity analysis was performed by using effective stress parameters to estimate the nominal bearing resistance of the continuous footings supported on foundation soils. Results of the bearing capacity analysis are attached in the Appendix. Nominal bearing resistance corresponding to bearing elevation at each boring location is summarized in Table 6.1.1 for DLZ personnel to verify the applied design pressure at Strength and Extreme Limit States. Because the nominal bearing resistance was computed using a semi empirical method, a resistance factor (ϕ) of 0.45 must be applied to compute the factored bearing resistance at Strength Limit State.

Table 6.1.1–Estimated Design Parameters at Strength Limit State for RW-1

Boring No.	Location	Depth of Bottom of Footing Below Proposed Grade (feet)	Effective Width of Spread Footing (feet)	Proposed Bearing Elevation (feet)	Nominal Bearing Resistance (ksf)
B-007-1-13	RW-1	3.5	9.5	1037.0	19.1
B-007-4-13	RW-1	3.5	9.5	1036.5	19.1

Consolidation settlement analysis was performed at the test boring locations using estimated soil parameters derived from laboratory moisture content tests and our local experience. The site plans provided by DLZ indicates that the maximum design pressure at the Service Limit State will be 1.75 ksf. Results of the settlement analysis are attached in Appendix B. Table 6.1.2 summarizes the applied factored loads and effective footing sizes at the Service Limit State used to calculate the estimated settlement. Based on the settlement analysis, it is estimated that the maximum total settlement and differential settlement will not exceed one inch and one-half of an inch, respectively. Settlement in granular soil will occur immediately during construction.

Table 6.1.2 – Estimated Design Parameters at Service Limit State for RW-1

Boring No.	Location	Effective Footing Width (feet)	Effective Footing Length (feet)	Applied Designed Pressure (ksf)	Elastic Settlement (inches)
B-007-1-13	RW-1	9.5	350.0	2.0	0.36
B-007-4-13	RW-1	9.5	350.0	2.0	0.52

If continuous footings are used to support the horizontal or inclined loads, failure by sliding must also be analyzed at Strength and Extreme Limit States. In order to calculate factored nominal sliding resistance between the interface of the footing and the ODOT 304 aggregate, a friction angle value of 32 degrees is estimated. A resistance factor (ϕ) of 0.85 should be applied to compute factored sliding resistance when checking sliding at Strength Limit State. Since the continuous footings will be placed on relatively level ground, global stability of the footing is not a concern. Prior to placing ODOT 304

Soil nailing consists of solid steel reinforcing bars which are installed in closely spaced intervals in holes drilled through unsupported soils with structural grout placed around it. A temporary shotcrete layer is applied on the excavated face of the earth after placing the reinforcement and before the next lift of soil is to be excavated downward. The reinforcement typically consists of welded wire mesh (WWM), which is placed at approximately the middle of the facing thickness. A hex nut and washers are subsequently installed to secure the nail head against the bearing plate. The estimated soil parameters provided below can be used for preliminary design purposes of the soil nail walls. The design value for Ultimate Bond Stress is based on Table 3.10 "Estimated Bond Strength of Soil Nails in Soil and Rock" of the FHWAO-IF-03-017, "Geotechnical Engineering Circular No. 7, Soil Nail Walls" issued March 2003, and assumes gravity grouting of the nails.

Soil Type: Embankment Fill Soils	
Silt and Clay (A-6a)/Sandy Silt (A-4a)	
Bulk Unit Weight:	130 pcf
Saturated Unit Weight:	67.6 pcf
Undrained Shear Strength:	1000 psf
Average Friction Angle (Phi):	25 degrees
Cohesion:	300 psf
Ultimate Bond Stress (psi)	6 psi

Soil Type: Foundation Natural Soils	
Silt and Clay (A-6a)/Sandy Silt (A-4a)	
Bulk Unit Weight:	135 pcf
Saturated Unit Weight:	72.6 pcf
Undrained Shear Strength:	2000 psf
Average Friction Angle (Phi):	28 degrees
Cohesion:	350 psf

The Soil Nail Wall System should be designed, constructed, and tested in accordance with FHWAO-IF-03-017, "Geotechnical Engineering Circular No. 7, Soil Nail Walls" issued March 2003. The minimum Safety Factors based on Table 5.3 "Minimum Recommended Factors of Safety for the Design of Soil Nail Walls using the ASD Method" of the above manual should be used as a structural design guide for designing the Soil Nail Wall System. The Soil Nail Wall System should be designed and constructed by a speciality contactor who is prequalified with ODOT. The Soil Nail Wall and its components must be protected against corrosion by epoxy or an encapsulated paint coating in addition to structural grout and shotcrete protection. A geocomposite strip drainage media should be placed before applying the shotcrete in order to minimize infiltration of water into the soils and to reduce the deterioration and sloughing of soils behind the Soil Nail Wall. Construction of the first lift of the Soil be selected in such a way to minimize the induced vibrations. PGI recommends performing a structure survey before the pile driving and monitoring vibrations during pile driving.

6.2 Lateral Earth Pressures and Drainage

The Retaining Wall Systems must be designed to resist active lateral earth pressures exerted by backfill soils. Porous backfill must be placed behind the walls at a minimum of 2.0 feet in thickness normal to the retaining wall to prevent hydrostatic pressure build up in accordance with ODOT Item 518 - “Drainage of Structures”. It is suggested that filter fabric, ODOT Item 712.09, Type A, be placed between granular backfill material and retaining soils. This will ensure that fine particles from within the embankment do not migrate into the voids of the porous backfill. The porous backfill should meet the requirements of ODOT granular material Type B. The estimated soil parameters provided below can be used in calculations for the lateral earth pressure of the semi-gravity walls and Solider Pile Walls.

Semi-Gravity Wall
Silt and Clay (A-6a)/Sandy Silt (A-4a)

Bulk Unit Weight:	125 pcf
Saturated Unit Weight:	62.6 pcf
Average Friction Angle (Phi):	28 degrees
At Rest Coefficient (K_o):	0.531
Active Pressure Coefficient (K_a):	0.361
Passive Pressure Coefficient (K_p):	2.770

Granular Material Type B

Bulk Unit Weight:	130 pcf
Saturated Unit Weight:	67.6 pcf
Average Friction Angle (Phi):	34 degrees
At Rest Coefficient (K_o):	0.441
Active Pressure Coefficient (K_a):	0.283
Passive Pressure Coefficient (K_p):	3.537

Soldier Pile Wall
Embankment fill

Bulk Unit Weight:	125 pcf
Saturated Unit Weight:	62.6 pcf
Average Friction Angle (Phi):	28 degrees
At Rest Coefficient (K_o):	0.531
Active Pressure Coefficient (K_a):	0.361
Passive Pressure Coefficient (K_p):	2.770

Silt and Clay (A-6a)/Sandy Silt (A-4a)

Bulk Unit Weight:	132 pcf
Saturated Unit Weight:	69.6 pcf

Average Friction Angle (Phi):	30 degrees
At Rest Coefficient (K_o):	0.500
Active Pressure Coefficient (K_a):	0.333
Passive Pressure Coefficient (K_p):	3.000

Gravel and Stone Fragments with Sand (A-1-b)

Bulk Unit Weight:	130 pcf
Saturated Unit Weight:	67.6 pcf
Average Friction Angle (Phi):	34 degrees
At Rest Coefficient (K_o):	0.441
Active Pressure Coefficient (K_a):	0.283
Passive Pressure Coefficient (K_p):	3.537

6.3 Groundwater Management

Groundwater was encountered in all test borings at the RW-1 and RW-2 sites and in 7 of the eight test borings at the RW-3 site during and upon completion of drilling operations. If the bottom of the excavation for the retaining wall foundations extend below the groundwater level near the boring locations, water infiltration is anticipated. Moderate to high volume pumping or dewatering will be required. Pumping can be controlled through the use of sump pumps. It must be noted that the groundwater levels during construction may vary due to seasonal fluctuations, and groundwater may occur where not encountered previously.

6.4 Earthwork and Construction Supervision

All excavations should comply with all current and applicable local, state and federal safety codes, regulations and practices, including the Occupational Safety and Health Administration (OSHA). The contractor should take measures to ensure that the foundation of the east abutment of the IR-480 EB Ramp Bridge will not be undermined during the construction of RW-2. Soil excavations are expected during construction of the retaining walls. The proposed temporary cut soil slope for the RW-1 foundation excavations may be constructed using a one (1) horizontal to one (1) vertical slope on cohesive soils provided that the temporary cut slope must be protected against any wet weather and subsequent surface run off and construction duration must be maximum of two (2) months. Also, regular traffic must be at least 3 feet away from the crest of the 1:1 slope. The lift height of the proposed cut slope should be 4 feet at a time for the RW-2 Wall. All excavations should be conducted in accordance with ODOT's "Construction and Materials Specifications," Item 503 - "Excavation for Structures". Due to the stagnant water and cattails along the ditch at RW-1 and RW-2 retaining wall sites, pockets of soft soils may be encountered at the proposed footing grade. If encountered, they should be excavated and replaced with compacted engineered fill. All excavation and backfilling operations should be conducted in accordance with ODOT's *Construction and Materials Specifications*, Item 503 - "Excavation for Structures" issued in January 2013 and under the supervision of competent geotechnical personnel. The contractor will need to take precautions and use light weight equipment while performing compaction operations behind and within 3 feet of the retaining walls.

At the RW-1 site, the engineered fill material should be placed in lifts of eight (8) inches in thickness (loose measure) and compacted to an unyielding condition with a minimum of 100 percent of the maximum dry density of the material as determined by the Standard Proctor Test (ASTM D 698). The moisture content of the material should be within ±2.0 percent of the optimum moisture content as determined by the Standard Proctor Test. Before placing engineered fill material, any water in the excavation must be removed. All fill material must be approved by a qualified geotechnical engineer prior to placement. All in-place density tests should be performed as per Supplement 1015 “Compaction Testing of Unbound Materials” during earthwork construction. The tests should be performed by a qualified soil technician in accordance with the appropriate ASTM procedures.

PROJECT: CUY-271-0.00 - RW-1 (WS1)	DRILLING FIRM / OPERATOR: DLZ / A. MITCHELL	DRILL RIG: CME 850 TRACKED	STATION / OFFSET: 3248+28.2, 44.2' RT	EXPLORATION ID B-007-1-13
TYPE: NEW RETAINING WALL	SAMPLING FIRM / LOGGER: PGI / F.BUSHER	HAMMER: CME AUTOMATIC	ALIGNMENT: LANE W-S BASELINE	
PID: 80418 BR ID:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 11/13/12	ELEVATION: 1040.1 (MSL) EOB: 50.0 ft.	PAGE 1 OF 2
START: 5/20/13 END: 5/20/13	SAMPLING METHOD: SPT	ENERGY RATIO (%): 81.2	COORD: 41.205963400, -81.304170700	

MATERIAL DESCRIPTION AND NOTES	ELEV. 1040.1	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
TOPSOIL (6" THICK)	1039.6																	
STIFF, BROWN, **SILT AND CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, FILL, DAMP	1036.6	1–3	3 3 6	12	78	SS-1	4.00	-	-	-	-	-	-	-	-	14	A-6a (V)	
STIFF TO MEDIUM STIFF, DARK GRAY TO BROWN AND GRAY, **SILTY CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, FILL, DAMP TO MOIST		4–5	2 4 6	14	100	SS-2	3.00	-	-	-	-	-	-	-	-	19	A-6b (V)	
@6.0'; MEDIUM STIFF, BROWN AND DARK GRAY, MOIST	1031.6	6–8	2 2 3	7	78	SS-3	2.00	-	-	-	-	-	-	-	-	21	A-6b (V)	
STIFF TO VERY STIFF, BROWN, **SILT AND CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, DAMP		9–10	2 4 6	14	100	SS-4	3.00	6	5	11	38	40	30	18	12	15	A-6a (9)	
@11.0'; VERY STIFF		11–12	4 9 10	26	100	SS-5	4.5+	-	-	-	-	-	-	-	-	16	A-6a (V)	
@13.5'; PUSHED SHELBY TUBE	1026.6	13																
VERY STIFF TO STIFF, GRAY, **SANDY SILT**, SOME CLAY, TRACE STONE FRAGMENTS, DAMP @13.5'; UNDRAINED SHEAR STRENGTH = 10,607 PSF		14–15			83	ST-6	4.5+	4	6	11	44	35	24	16	8	14	A-4a (8)	
		16–18	5 6 9	20	100	SS-7	4.5+	-	-	-	-	-	-	-	-	13	A-4a (V)	
@18.5'; STIFF	1019.1	19–20 ▼	2 5 6	15	100	SS-8	4.00	-	-	-	-	-	-	-	-	14	A-4a (V)	
VERY DENSE, GRAY, **STONE FRAGMENTS**, WET @21'; POSSIBLE COBBLE OR BOULDER ZONE	1016.6	21–23 W	21 45 50	129	67	SS-9	-	-	-	-	-	-	-	-	-	7	A-1-a (V)	
VERY DENSE, GRAY, **COARSE AND FINE SAND**, LITTLE TO TRACE STONE FRAGMENTS, TRACE FINES, WET		24–25	12 18 19	50	78	SS-10	-	-	-	-	-	-	-	-	-	12	A-3a (V)	
@26.0'; TRACE STONE FRAGMENTS	1011.1	26–28	9 12 33	61	100	SS-11	-	-	-	-	-	-	-	-	-	18	A-3a (V)	
HARD TO STIFF, GRAY, **SANDY SILT**, SOME CLAY, LITTLE STONE FRAGMENTS, DAMP		29–33	6 14 13	37	67	SS-12	-	-	-	-	-	-	-	-	-	14	A-4a (V)	
@33.5'; STIFF		34–38	3 4 6	14	100	SS-13	2.50	12	9	14	35	30	22	15	7	13	A-4a (6)	
		39	7 13	50	78	SS-14	4.5+	-	-	-	-	-	-	-	-	11	A-4a (V)	

PID: 80418	BR ID:	PROJECT: CUY-271-0.00 - RW-1 (WS1)	STATION / OFFSET: 3248+28.2, 44.2' RT	START: 5/20/13	END: 5/20/13	PG 2 OF 2	B-007-1-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 1000.1	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
HARD TO STIFF, GRAY, **SANDY SILT**, SOME CLAY, LITTLE STONE FRAGMENTS, DAMP *(continued)*	996.6	41 42 43	24															
HARD TO VERY STIFF, GRAY, **SILT AND CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, DAMP		44 45	5 10 13	31	100	SS-15	4.5+	-	-	-	-	-	-	-	-	14	A-6a (V)	
		46 47 48																
@48.5'; VERY STIFF	990.1	49 EOB 50	4 8 12	27	100	SS-16	4.5+	-	-	-	-	-	-	-	-	13	A-6a (V)	

NOTES: GROUNDWATER WAS ENCOUNTERED AT 23.5' DURING DRILLING AND 20.0' UPON COMPLETION OF DRILLING OPERATIONS.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: HOLE WAS GROUTED USING 1/2 BAGS BENTONITE/CEMENT MIX

PROJECT: CUY-271-0.00 - RW-1 (WS1)	DRILLING FIRM / OPERATOR: DLZ / A. MITCHELL	DRILL RIG: CME 850 TRACKED	STATION / OFFSET: 3247+95.6, 22.0' RT
TYPE: NEW RETAINING WALL	SAMPLING FIRM / LOGGER: PGI / F.BUSHER	HAMMER: CME AUTOMATIC	ALIGNMENT: LANE W-S BASELINE
PID: 80418 BR ID:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 11/13/12	ELEVATION: 1035.4 (MSL) EOB: 63.9 ft.
START: 5/21/13 END: 5/21/13	SAMPLING METHOD: SPT	ENERGY RATIO (%): 81.2	COORD: 41.210120100, -81.304255200

EXPLORATION ID: B-007-4-13

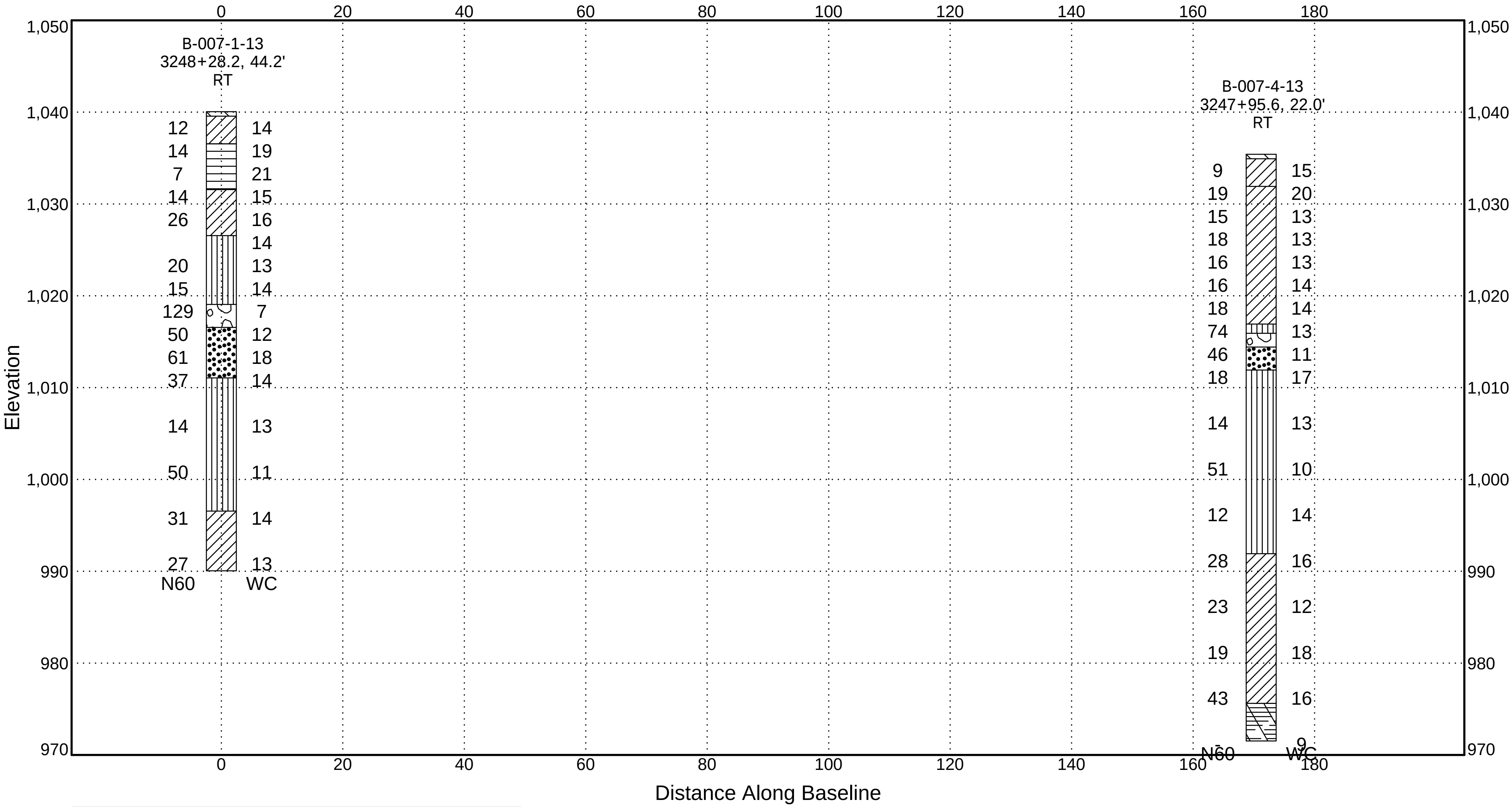
MATERIAL DESCRIPTION AND NOTES	ELEV. 1035.4	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
TOPSOIL (6" THICK)	1034.9																	
STIFF, BROWN, **SILT AND CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, TRACE ROOTS, FILL, DAMP	1031.9		3 3 4	9	100	SS-1	4.00	-	-	-	-	-	-	-	-	15	A-6a (V)	
VERY STIFF TO STIFF, BROWN TO GRAY, **SILT AND CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, MOIST TO DAMP			5 6 8	19	100	SS-2	4.5+	4	5	8	35	48	29	18	11	20	A-6a (8)	
@6.0'; STIFF, GRAY, DAMP			3 5 6	15	100	SS-3	4.5+	-	-	-	-	-	-	-	-	13	A-6a (V)	
@8.5'; GRAY, DAMP			3 6 7	18	100	SS-4	4.5+	-	-	-	-	-	-	-	-	13	A-6a (V)	
@11.0'; GRAY, DAMP			3 5 7	16	100	SS-5	4.5+	-	-	-	-	-	-	-	-	13	A-6a (V)	
@13.5'; GRAY, DAMP			2 5 7	16	100	SS-6	3.25	-	-	-	-	-	-	-	-	14	A-6a (V)	
@16.0'; GRAY	1016.9	W	3 6 7	18	78	SS-7	4.5+	-	-	-	-	-	-	-	-	14	A-6a (V)	
MEDIUM DENSE, GRAY, **NON-PLASTIC SANDY SILT**, LITTLE STONE FRAGMENTS, WET	1015.9		4 9 46	74	-	SS-8A&B	-	-	-	-	-	-	-	-	-	13	A-4a (V)	
							-	-	-	-	-	-	-	-	-	7	A-1-a (V)	
@19.5'; POSSIBLE BOULDER OR COBBLE ZONE	1014.4																	
DENSE, GRAY, **COARSE AND FINE SAND**, LITTLE FINES, TRACE STONE FRAGMENTS, WET @21.5'; SAND HEAVED 1.5' INTO AUGER	1011.9		2 8 26	46	78	SS-9	-	-	-	-	-	-	-	-	-	11	A-3a (V)	
STIFF TO HARD, GRAY, **SANDY SILT**, SOME CLAY, TRACE TO LITTLE STONE FRAGMENTS, MOIST TO DAMP @24.0'; VERY STIFF, DAMP		▼	4 5 8	18	100	SS-10	4.50	-	-	-	-	-	-	-	-	17	A-4a (V)	
@26.0'; PUSHED SHELBY TUBE					67	ST-11	-	-	-	-	-	-	-	-	-	-	A-4a (V)	
@28.5'; LITTLE STONE FRAGMENTS			2 4 6	14	67	SS-12	2.25	-	-	-	-	-	-	-	-	13	A-4a (V)	
@33.5'; HARD, LITTLE STONE FRAGMENTS			9 14 24	51	78	SS-13	4.5+	-	-	-	-	-	-	-	-	10	A-4a (V)	
@38.5'; LITTLE STONE FRAGMENTS, DAMP			4 4	12	67	SS-14	3.50	-	-	-	-	-	-	-	-	14	A-4a (V)	

PID: 80418	BR ID:	PROJECT: CUY-271-0.00 - RW-1 (WS1)	STATION / OFFSET: 3247+95.6, 22.0' RT	START: 5/21/13	END: 5/21/13	PG 2 OF 2	B-007-4-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 995.4	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
STIFF TO HARD, GRAY, **SANDY SILT**, SOME CLAY, TRACE TO LITTLE STONE FRAGMENTS, MOIST TO DAMP *(continued)*	991.9	41, 42, 43	5															
VERY STIFF TO STIFF, GRAY, **SILT AND CLAY**, TRACE SAND, TRACE STONE FRAGMENTS, DAMP TO MOIST		44, 45	6 9 12	28	83	SS-15	3.00	-	-	-	-	-	-	-	-	16	A-6a (V)	
		46, 47, 48, 49, 50	3 7 10	23	83	SS-16	3.75	-	-	-	-	-	-	-	-	12	A-6a (V)	
		51, 52, 53, 54, 55	6 6 8	19	72	SS-17	2.75	3	3	7	31	56	30	17	13	18	A-6a (9)	
@58.5'; STIFF	975.6	56, 57, 58, 59, 60	1 7 25	43	100	SS-18	2.50	-	-	-	-	-	-	-	-	16	A-6a (V)	
SHALE, GRAY, SEVERELY WEATHERED, VERY WEAK.	971.5	TR 61, 62, 63	50/5"	-	100	SS-19	-	-	-	-			-	-	-	9	Rock (V)	

NOTES: GROUNDWATER WAS ENCOUNTERED AT 18.5' DURING DRILLING AND 26.0' UPON COMPLETION OF DRILLING OPERATIONS.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: HOLE WAS GROUTED USING 1/2 BAGS BENTONITE/CEMENT MIX



Borehole	North	East	Elev.	Depth
B-007-1-13	614837	2240064	1040.1	50.0
B-007-4-13	614995	2239997	1035.4	63.9

DISTANCES:
Beginning 0
Ending 180
VIEWING ANGLES (degrees):
Horizontal 0.0
Vertical 0.0

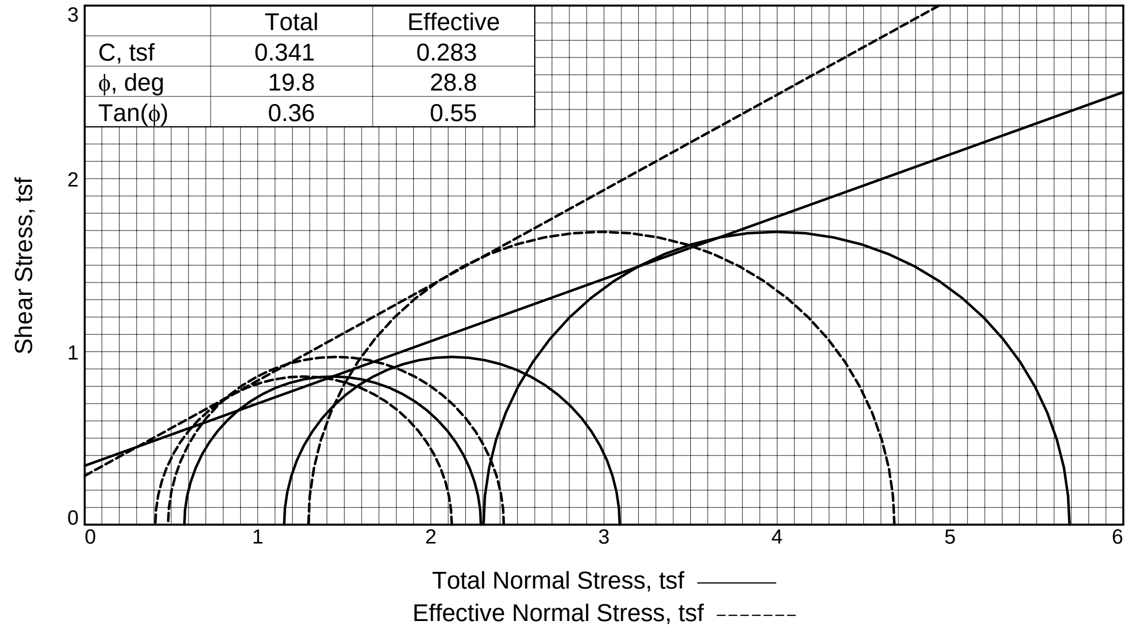
Position	North	East
Left, Front	614837	2240064
Right, Front	615003	2239994
Left, Back	614837	2240064
Right, Back	615003	2239994

SOIL BORINGS PROFILE
RETAINING WALL RW-1 (WS1)

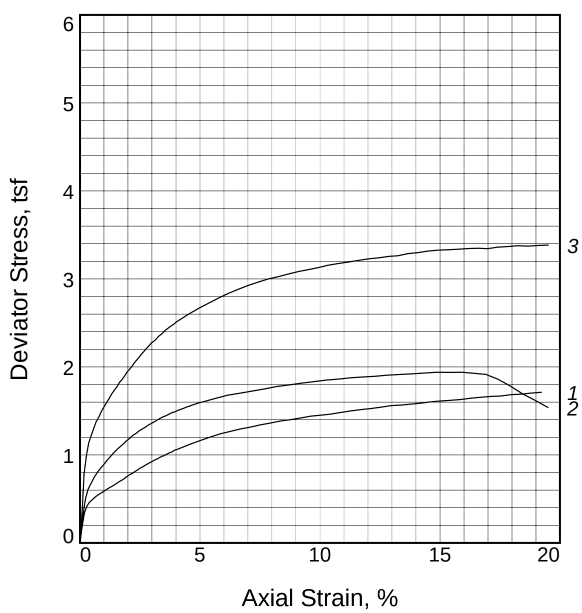
CUY-271-0.00

Summit/Cuyahoga Counties

PROJECT #	DATE	PLATE
80418	Aug 14	1



	Total	Effective
C, tsf	0.341	0.283
ϕ, deg	19.8	28.8
Tan(ϕ)	0.36	0.55



	Sample No.	1	2	3
Initial	Water Content, %	22.6	16.7	18.0
	Dry Density, pcf	106.2	108.9	111.0
	Saturation, %	103.6	82.5	93.8
	Void Ratio	0.5878	0.5472	0.5189
	Diameter, in.	2.83	2.82	2.83
	Height, in.	5.61	4.57	5.63
At Test	Water Content, %	21.8	20.3	19.2
	Dry Density, pcf	106.2	108.9	111.0
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.5878	0.5472	0.5189
	Diameter, in.	2.83	2.82	2.83
	Height, in.	5.61	4.57	5.63
	Strain rate, in./min.	0.01	0.01	0.01
	Back Pressure, tsf	4.03	4.03	4.03
	Cell Pressure, tsf	4.61	5.18	6.34
	Fail. Stress, tsf	1.71	1.94	3.39
	Total Pore Pr., tsf	4.20	4.70	5.04
	Ult. Stress, tsf	1.71	1.54	3.39
	Total Pore Pr., tsf	4.20	5.17	5.04
	$\bar{\sigma}_1$ Failure, tsf	2.12	2.42	4.68
	$\bar{\sigma}_3$ Failure, tsf	0.41	0.48	1.29

Type of Test:
CU with Pore Pressures

Sample Type: Shelby Tube

Description: lean clay with sand

LL= 39 **PL=** 20 **PI=** 19

Assumed Specific Gravity= 2.7

Remarks:

Figure ________

Client: ODOT

Project: CUY/SUM-271/480-00.00 - RW-2 (SW1)

Source of Sample: B-0075-14 **Depth:** 28.5'-30.5'

Sample Number: ST-3

Proj. No.: 1122-1001.00 **Date Sampled:** 6-19-14

DLZ

Tested By: Steve Robinson **Checked By:** Barry Wong

7/1/2014
3:21 PM

TRIAXIAL COMPRESSION TEST

CU with Pore Pressures

Date: 6-19-14
Client: ODOT
Project: CUY/SUM-271/480-00.00
Project No.: 1122-1001.00
Location: B-007
Depth: 28.5'-30.5' **Sample Number:** ST-3
Description: lean clay with sand
Remarks:
Type of Sample: Shelby Tube
Assumed Specific Gravity=2.7 **LL=**39 **PL=**20 **PI=**19
Test Method: COE uniform strain

Parameters for Specimen No. 1

Specimen Parameter	Initial	Saturated	Consolidated	Final
Moisture content: Moist soil+tare, gms.	79.970			1304.120
Moisture content: Dry soil+tare, gms.	75.500			1074.020
Moisture content: Tare, gms.	55.680			82.160
Moisture, %	22.6	21.8	21.8	23.2
Moist specimen weight, gms.	1201.9			
Diameter, in.	2.83	2.83	2.83	
Area, in.^{2}	6.28	6.28	6.28	
Height, in.	5.61	5.61	5.61	
Net decrease in height, in.		0.00	0.00	
Wet density, pcf	130.1	129.3	129.3	
Dry density, pcf	106.2	106.2	106.2	
Void ratio	0.5878	0.5878	0.5878	
Saturation, %	103.6	100.0	100.0	

Test Readings for Specimen No. 1

Consolidation cell pressure = 64.00 psi (4.608 tsf)
Consolidation back pressure = 56.00 psi (4.032 tsf)
Consolidation effective confining stress = 0.576 tsf
Strain rate, in./min. = 0.01
Fail. Stress = 1.713 tsf **at reading no.** 76
Ult. Stress = 1.713 tsf **at reading no.** 76

Parameters for Specimen No. 2

Specimen Parameter	Initial	Saturated	Consolidated	Final
Moisture content: Moist soil+tare, gms.	344.140			1082.370
Moisture content: Dry soil+tare, gms.	303.340			883.730
Moisture content: Tare, gms.	59.380			81.720
Moisture, %	16.7	20.3	20.3	24.8
Moist specimen weight, gms.	954.8			
Diameter, in.	2.82	2.82	2.82	
Area, in.²	6.26	6.26	6.26	
Height, in.	4.57	4.57	4.57	
Net decrease in height, in.		0.00	0.00	
Wet density, pcf	127.2	131.0	131.0	
Dry density, pcf	108.9	108.9	108.9	
Void ratio	0.5472	0.5472	0.5472	
Saturation, %	82.5	100.0	100.0	

Test Readings for Specimen No. 2

Consolidation cell pressure = 72.00 psi (5.184 tsf)

Consolidation back pressure = 56.00 psi (4.032 tsf)

Consolidation effective confining stress = 1.152 tsf

Strain rate, in./min. = 0.01

Fail. Stress = 1.939 tsf **at reading no.** 61

Ult. Stress = 1.539 tsf **at reading no.** 68

No.	Def. Dial in.	Load Dial	Load lbs.	Strain %	Deviator Stress tsf	Minor Eff. Stress tsf	Major Eff. Stress tsf	1:3 Ratio	Pore Press. psi	P tsf	Q tsf
0	0.0010	14.003	0.0	0.0	0.000	1.118	1.118	1.00	56.48	1.118	0.000
1	0.0057	37.843	23.8	0.1	0.274	0.968	1.242	1.28	58.56	1.105	0.137
2	0.0118	58.175	44.2	0.2	0.507	0.832	1.339	1.61	60.44	1.085	0.253
3	0.0169	67.536	53.5	0.3	0.614	0.757	1.371	1.81	61.48	1.064	0.307
4	0.0234	74.278	60.3	0.5	0.690	0.698	1.388	1.99	62.31	1.043	0.345
5	0.0285	79.519	65.5	0.6	0.749	0.647	1.396	2.16	63.01	1.022	0.375
6	0.0349	84.884	70.9	0.7	0.809	0.598	1.408	2.35	63.69	1.003	0.405
7	0.0413	89.178	75.2	0.9	0.857	0.557	1.414	2.54	64.26	0.986	0.429
8	0.0475	92.989	79.0	1.0	0.899	0.518	1.417	2.74	64.80	0.968	0.450
9	0.0532	97.091	83.1	1.1	0.945	0.482	1.427	2.96	65.31	0.954	0.472
10	0.0593	100.684	86.7	1.3	0.984	0.452	1.436	3.18	65.72	0.944	0.492
11	0.0652	104.351	90.3	1.4	1.025	0.421	1.446	3.43	66.15	0.933	0.512
12	0.0716	107.848	93.8	1.5	1.063	0.393	1.456	3.70	66.54	0.925	0.531
13	0.0781	110.994	97.0	1.7	1.097	0.369	1.466	3.97	66.87	0.918	0.548
14	0.0840	113.822	99.8	1.8	1.127	0.348	1.476	4.24	67.16	0.912	0.564
15	0.0894	116.733	102.7	1.9	1.159	0.327	1.486	4.54	67.45	0.907	0.579
16	0.0953	119.406	105.4	2.1	1.187	0.312	1.499	4.81	67.67	0.906	0.594
17	0.1015	122.597	108.6	2.2	1.222	0.294	1.516	5.15	67.91	0.905	0.611
18	0.1076	124.743	110.7	2.3	1.244	0.282	1.526	5.42	68.09	0.904	0.622
19	0.1138	127.432	113.4	2.5	1.273	0.268	1.541	5.75	68.28	0.904	0.636
20	0.1196	129.487	115.5	2.6	1.294	0.258	1.552	6.02	68.42	0.905	0.647
21	0.1256	131.519	117.5	2.7	1.315	0.248	1.563	6.29	68.55	0.906	0.657
22	0.1312	133.908	119.9	2.8	1.340	0.239	1.579	6.61	68.68	0.909	0.670
23	0.1368	135.657	121.7	3.0	1.358	0.233	1.591	6.82	68.76	0.912	0.679
24	0.1428	137.546	123.5	3.1	1.377	0.227	1.604	7.07	68.85	0.915	0.689
25	0.1481	139.522	125.5	3.2	1.397	0.221	1.618	7.32	68.93	0.920	0.699

Shear Strength by Direct Shear (Small Shear Box)

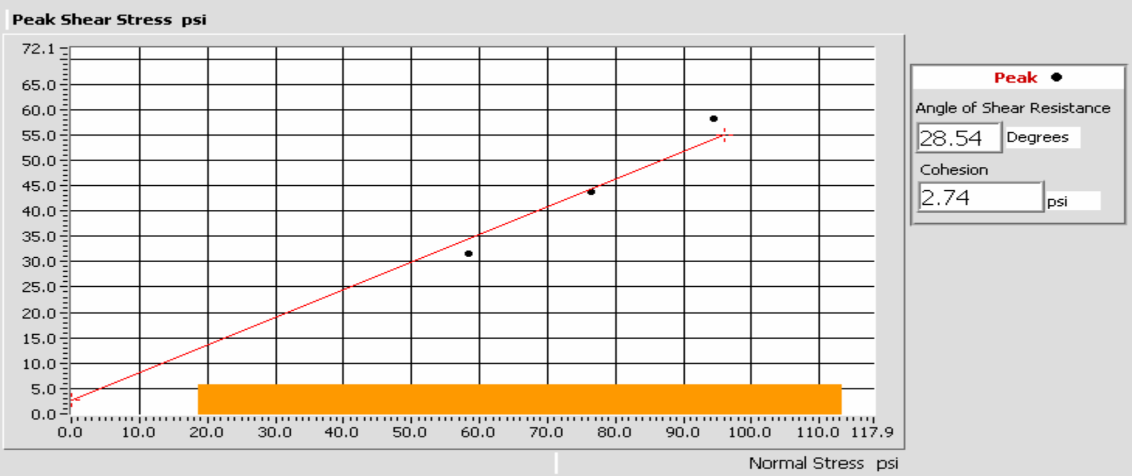
Pro Geotech, Inc.

Client	DLZ	**Lab Ref**	
Project	CUY-271-0.00 - RW-3 (WS2)	**Job**	G14004G
Borehole	B-054-0-13	**Sample**	ST-9

Test Details			
Standard	ASTM D3080-03 / AASHTO T236-92	**Particle Specific Gravity**	2.70
Sample Type	Thin walled push in sample	**Single or Multi Stage**	Single Stage
Lab. Temperature	72.0 deg.F	**Location**	Hancock County, OH
Sample Description	GRAY SANDY SILT, SOME CLAY, TRACE STONE FRAGMENTS		
Variations from procedure	None		

Test Summary				
Reference	A	B	C	
Normal Stress	76.38 psi	58.41 psi	94.35 psi	
Peak Strength	43.65 psi	31.46 psi	58.07 psi	
Corresponding Horizontal Displacement	0.2247 in	0.1997 in	0.2829 in	
Residual Stress	N/A	N/A	N/A	
Rate of Shear Displacement	Stage 1: 0.002845in/min	Stage 1: 0.002737in/min	Stage 1: 0.004097in/min	
Final Height	0.7367 in	0.7541 in	0.7384 in	
Sample Area	4.90870 in2	4.90870 in2	4.90870 in2	
Initial Wet Unit Weight	139.45 lbf/ft3	137.03 lbf/ft3	145.21 lbf/ft3	
Initial Dry Unit Weight	120.05 lbf/ft3	117.65 lbf/ft3	127.67 lbf/ft3	
Final Wet Unit Weight	149.50 lbf/ft3	146.15 lbf/ft3	156.09 lbf/ft3	
Final Dry Unit Weight	132.48 lbf/ft3	127.67 lbf/ft3	140.05 lbf/ft3	
Final Moisture Content	12.8 %	14.5 %	11.4 %	
Particle Specific Gravity	2.70	2.70	2.70	
Final Void Ratio	0.2727	0.3207	0.2039	
Final Saturation	127.18%	121.85%	151.58%	

Maximum Shear Stress vs Normal Stress



BEARING CAPACITY ANALYSIS

Project	**CUY-271-0.00**
Project#	**G13005G**
Bore#	**B-007-1-13 (RW-1)**
Method	**AASHTO 10.6.3.1.2**
Foundation Dimension	
Width of Footing (B_f) (feet)	9.5
Length of Footing (L_f) (feet)	350.0
Length (L_f)/Width (B_f) (>5 is continous footing)	36.8
Type of Footing	Continuous
Footing Bearing Elevation (feet)	1036.5
Depth of Footing (D_f) Feet below Proposed Grade	3.5
Depth of groundwater Table (D_w) below Footing (ft)	16.4
Height of Slope (Hs) (feet)	Flat Ground
Soil Parameters	
Undrained Shear Strength/Cohesion (psf)	0
Angle of internal friction (Phi) Degrees	32
Unit Weight of soil above base of footing (pcf)	125
Unit Weight of soil below base of footing (pcf)	125
Bearing Capacity Factors	
N_c	35.50
N_q	23.20
$N\gamma$	30.20
Shape Correction Factors	
s_c	1.00
s_q	1.00
s_γ	1.00
Load Inclination Factors	
ic	1.0
iq	1.0
i_γ	1.0
Correction for Water Table	
D_f+1.5B_f	17.8
C_{wq}	1.0
C_{wr}	0.5
Embedment Depth Correction Factor	
Df/Bf	0.4
d_q	1.0
Bearing Capacity Terms	
Cohesion Term	0
Surcharge Term	10150
Unit Weight Term	8966
Nominal Bearing Resistence (psf)	19116

AASHTO Eqn 10.6.3.1.2a

qn = c*Nc*Sc*ic + (Gamma)*Df*Nq*sq*dq*iq*Cwq+0.5*(Gamma)*Bf*Nr*sr*ir*Cw2

BEARING CAPACITY ANALYSIS

Project	**CUY-271-0.00**
Project#	**G13005G**
Bore#	**B-007-4-13 (RW-1)**
Method	**AASHTO 10.6.3.1.2**
Foundation Dimension	
Width of Footing (B_f) (feet)	9.5
Length of Footing (L_f) (feet)	350.0
Length (L_f)/Width (B_f) (>5 is continous footing)	36.8
Type of Footing	Continuous
Footing Bearing Elevation (feet)	1037.0
Depth of Footing (D_f) Feet below Proposed Grade	3.5
Depth of groundwater Table (D_w) below Footing (ft)	20.1
Height of Slope (Hs) (feet)	Flat Ground
Soil Parameters	
Undrained Shear Strength/Cohesion (psf)	0
Angle of internal friction (Phi) Degrees	32
Unit Weight of soil above base of footing (pcf)	125
Unit Weight of soil below base of footing (pcf)	125
Bearing Capacity Factors	
N_c	35.50
N_q	23.20
$N\gamma$	30.20
Shape Correction Factors	
s_c	1.00
s_q	1.00
s_γ	1.00
Load Inclination Factors	
ic	1.0
iq	1.0
i_γ	1.0
Correction for Water Table	
$D_f+1.5B_f$	17.8
C_{wq}	1.0
C_{wr}	0.5
Embedment Depth Correction Factor	
Df/Bf	0.4
d_q	1.0
Bearing Capacity Terms	
Cohesion Term	0
Surcharge Term	10150
Unit Weight Term	8966
Nominal Bearing Resistence (psf)	19116

AASHTO Eqn 10.6.3.1.2a

qn = c*Nc*Sc*ic + (Gamma)*Df*Nq*sq*dq*iq*Cwq+0.5*(Gamma)*Bf*Nr*sr*ir*Cw2

RETAINING WALL SETTLEMENT ANALYSES - RETAINING WALL RW-1

Project:	CUY-271-0.00	**Project #**	G13005G	**Test Boring #**	B-007-4-13
Type of Foundation	Compression Index (Cc) (From Lab Test)			Depth of Ground Water Level (feet)	18.5
Shallow Foundation (Continuous)	Recompression Index (Cr) (From Lab Test)			Unit Weight of Water (pcf)	62.4
Length = 350'	Depth of Footing (D_f) below ground (feet)	1.0		Specific Gravity of Soil Solids (G)	
Width = 9.5'	Applied Design Pressure (psf)	2,000		Unit Weight of Soil above the base of foundation (pcf)	125

Depth Below the Foundation (Z)	**AVERAGE PROPERTIES**		**CALCULATIONS**		**Total**
D_f= -1.1' & Z=0.0	Thickness of Layer (feet)	4.6	OB Pressure at the top Layer(psf)	0	**Setlement**
	Ave. Corrected SPT Value (N_{60})	20	OB Pressure at the center Layer (psf)	311	**(inches)**
	Specific Gravity of Soil Solids (G)	2.65	Excess Pressure At Center Due to appliedLoad	1610	
(Above Water Table)	Moisture content (%)	8	Bearing Capacity Index (C)	190	
Z=2.30' (At Centre of Layer)	Liquid Limit (%)	NP	Immediate Settlement in Foundation Soil (inches)	0.23	**0.23**
	Plastic Limit (%)	NP	Initial Void Ratio (e_0)	0.32	
	Plasticity Index (%)	NP			
	Unit Weight of soil (pcf)	135			
D_f=3.5' & Z=4.6'	Submerged Unit Weight of Soil (pcf)		OB Pressure at the bottom Layer (psf)	621	
Df=3.5' & Z=4.6'	Thickness of Layer (feet)	15	OB Pressure at the top Layer(psf)	621	**Setlement**
	Ave. Corrected SPT Value (N_{60})	17	OB Pressure at the center Layer (psf)	1559	**(inches)**
	Specific Gravity of Soil Solids (G)	2.7	Excess Pressure At Center Due to appliedLoad	880	
(above Water Table)	Moisture content (%)	15	Compression Index (C_c)	0.15	
Z=12.10' (At Centre of Layer)	Liquid Limit (%)	29	Recompression Index (C_r)	0.015	0.015
	Plastic Limit (%)	18	Initial Void Ratio (e_0)	0.55	
	Plasticity Index (%)	11	Settlement due to compression (inches)	3.39	
	Unit Weight of soil (pcf)	125	Settlement due to recompression (inches)	0.34	**0.34**
D_f=18.5' & Z=19.6'	Submerged Unit Weight of Soil (pcf)		OB Pressure at the bottom Layer (psf)	2496	
D_f=18.5' & Z=19.6'	Thickness of Layer (feet)	5	OB Pressure at the top Layer(psf)	2496	**Setlement**
	Ave. Corrected SPT Value (N_{60})	60	OB Pressure at the center Layer (psf)	2678	**(inches)**
	Specific Gravity of Soil Solids (G)	2.65	Excess Pressure At Center Due to appliedLoad	601	
(below Water Table)	Moisture content (%)	11	Bearing Capacity Index (C)	190	
Z=22.1' (At Centre of Layer)	Liquid Limit (%)	NP	Immediate Settlement in Foundation Soil (inches)	0.03	**0.03**
	Plastic Limit (%)	NP	Initial Void Ratio (e_0)	0.36	
	Plasticity Index (%)	NP			
	Unit Weight of soil (pcf)	135			
D_f=23.5' & Z=24.6'	Submerged Unit Weight of Soil (pcf)	72.6	OB Pressure at the bottom Layer (psf)	2859	

Project:	CUY-271-0.00		Project #	G13005G	Test Boring #	B-007-4-13
Df=23.5' & Z=24.6'	Thickness of Layer (feet)	20	OB Pressure at the top Layer(psf)	2859	**Setlement**	
	Ave. Corrected SPT Value (N_{60})	15	OB Pressure at the center Layer (psf)	3435	**(inches)**	
	Specific Gravity of Soil Solids (G)	2.7	Excess Pressure At Center Due to appliedLoad	482		
(Below the Water Table)	Moisture content (%)	14	Compression Index (C_c)	0.14		
Z=34.60' (At Centre of Layer)	Liquid Limit (%)	22	Recompression Index (C_r)	0.014	0.014	
	Plastic Limit (%)	15	Initial Void Ratio (e_0)	0.60		
	Plasticity Index (%)	7	Settlement due to compression (inches)	1.20		
	Unit Weight of soil (pcf)	120	Settlement due to recompression (inches)	0.12	**0.12**	
D_f=43.5' & Z=44.6'	Submerged Unit Weight of Soil (pcf)	57.6	OB Pressure at the bottom Layer (psf)	4011		
Df=43.5' & Z=44.6'	Thickness of Layer (feet)	16.3	OB Pressure at the top Layer(psf)	4011	**Setlement**	
	Ave. Corrected SPT Value (N_{60})	28	OB Pressure at the center Layer (psf)	4578	**(inches)**	
	Specific Gravity of Soil Solids (G)	2.7	Excess Pressure At Center Due to appliedLoad	305		
(Below the Water Table)	Moisture content (%)	16	Compression Index (C_c)	0.16		
Z=52.75' (At Centre of Layer)	Liquid Limit (%)	30	Recompression Index (C_r)	0.016	0.016	
	Plastic Limit (%)	17	Initial Void Ratio (e_0)	0.48		
	Plasticity Index (%)	13	Settlement due to compression (inches)	0.59		
	Unit Weight of soil (pcf)	132	Settlement due to recompression (inches)	0.06	**0.06**	
D_f=59.8' & Z=60.9'	Submerged Unit Weight of Soil (pcf)	69.6	OB Pressure at the bottom Layer (psf)	5145		

Total Settlement:	**0.78**
Consolidation Settlement:	**0.52**
Immediate Settlement:	**0.26**

RETAINING WALL SETTLEMENT ANALYSES - RETAINING WALL RW-1

Project:	CUY-271-0.00	**Project #**	G13005G	**Test Boring #**	B-007-1-13
Type of Foundation	Compression Index (Cc) (From Lab Test)		Depth of Ground Water Level (feet)		21
Shallow Foundation (Continuous)	Recompression Index (Cr) (From Lab Test)		Unit Weight of Water (pcf)		62.4
Length = 350'	Depth of Footing (D_f) below ground (feet)	3.1	Specific Gravity of Soil Solids (G)		
Width = 9.5'	Applied Design Pressure (psf)	2,000	Unit Weight of Soil above the base of foundation (pcf)		125

Depth Below the Foundation (Z)	**AVERAGE PROPERTIES**		**CALCULATIONS**		**Total**
D_f= 3.1' & Z=0.0	Thickness of Layer (feet)	5.4	OB Pressure at the top Layer(psf)	388	**Setlement**
	Ave. Corrected SPT Value (N_{60})	20	OB Pressure at the center Layer (psf)	752	**(inches)**
	Specific Gravity of Soil Solids (G)	2.65	Excess Pressure At Center Due to appliedLoad	1557	
(above Water Table)	Moisture content (%)	8	Bearing Capacity Index (C)	190	
Z=2.70' (At Centre of Layer)	Liquid Limit (%)	NP	Immediate Settlement in Foundation Soil (inches)	0.17	**0.17**
	Plastic Limit (%)	NP	Initial Void Ratio (e_0)	0.32	
	Plasticity Index (%)	NP			
	Unit Weight of soil (pcf)	135			
D_f=8.5' & Z=5.4'	Submerged Unit Weight of Soil (pcf)		OB Pressure at the bottom Layer (psf)	1117	
D_f=8.5' & Z=5.4'	Thickness of Layer (feet)	5	OB Pressure at the top Layer(psf)	1117	**Setlement**
	Ave. Corrected SPT Value (N_{60})	20	OB Pressure at the center Layer (psf)	1429	**(inches)**
	Specific Gravity of Soil Solids (G)	2.7	Excess Pressure At Center Due to appliedLoad	1092	
(above Water Table)	Moisture content (%)	16	Compression Index (C_c)	0.16	
Z=7.90' (At Centre of Layer)	Liquid Limit (%)	30	Recompression Index (C_r)	0.016	0.016
	Plastic Limit (%)	18	Initial Void Ratio (e_0)	0.56	
	Plasticity Index (%)	12	Settlement due to compression (inches)	1.51	
	Unit Weight of soil (pcf)	125	Settlement due to recompression (inches)	0.15	**0.15**
D_f=13.5' & Z=10.4'	Submerged Unit Weight of Soil (pcf)		OB Pressure at the bottom Layer (psf)	1742	
Df=13.5' & Z=10.4'	Thickness of Layer (feet)	7.5	OB Pressure at the top Layer(psf)	1742	**Setlement**
	Ave. Corrected SPT Value (N_{60})	19	OB Pressure at the center Layer (psf)	2237	**(inches)**
	Specific Gravity of Soil Solids (G)	2.7	Excess Pressure At Center Due to appliedLoad	803	
(above Water Table)	Moisture content (%)	14	Compression Index (C_c)	0.14	
Z=14.15' (At Centre of Layer)	Liquid Limit (%)	24	Recompression Index (C_r)	0.014	0.014
	Plastic Limit (%)	16	Initial Void Ratio (e_0)	0.46	
	Plasticity Index (%)	8	Settlement due to compression (inches)	1.15	
	Unit Weight of soil (pcf)	132	Settlement due to recompression (inches)	0.12	**0.12**
D_f=21.0' & Z=17.9'	Submerged Unit Weight of Soil (pcf)		OB Pressure at the bottom Layer (psf)	2732	

Project:	CUY-271-0.00		Project #	G13005G	Test Boring #	B-007-1-13
D_f=21.0' & Z=17.9'	Thickness of Layer (feet)	8	OB Pressure at the top Layer(psf)	2732	**Setlement**	
	Ave. Corrected SPT Value (N_{60})	60	OB Pressure at the center Layer (psf)	3022	**(inches)**	
	Specific Gravity of Soil Solids (G)	2.65	Excess Pressure At Center Due to appliedLoad	605		
(below Water Table)	Moisture content (%)	11	Bearing Capacity Index (C)	190		
Z=21.9' (At Centre of Layer)	Liquid Limit (%)	NP	Immediate Settlement in Foundation Soil (inches)	0.04	**0.04**	
	Plastic Limit (%)	NP	Initial Void Ratio (e_0)	0.36		
	Plasticity Index (%)	NP				
	Unit Weight of soil (pcf)	135				
D_f=29.0' & Z=25.9'	Submerged Unit Weight of Soil (pcf)	72.6	OB Pressure at the bottom Layer (psf)	3312		
Df=29.0' & Z=25.9'	Thickness of Layer (feet)	14.5	OB Pressure at the top Layer(psf)	3312	**Setlement**	
	Ave. Corrected SPT Value (N_{60})	33	OB Pressure at the center Layer (psf)	3839	**(inches)**	
	Specific Gravity of Soil Solids (G)	2.7	Excess Pressure At Center Due to appliedLoad	445		
(Below the Water Table)	Moisture content (%)	13	Compression Index (C_c)	0.13		
Z=33.15' (At Centre of Layer)	Liquid Limit (%)	22	Recompression Index (C_r)	0.013	0.013	
	Plastic Limit (%)	15	Initial Void Ratio (e_0)	0.41		
	Plasticity Index (%)	7	Settlement due to compression (inches)	0.76		
	Unit Weight of soil (pcf)	135	Settlement due to recompression (inches)	0.08	**0.08**	
D_f=43.5' & Z=40.4'	Submerged Unit Weight of Soil (pcf)	72.6	OB Pressure at the bottom Layer (psf)	4365		
Df=43.5' & Z=40.4'	Thickness of Layer (feet)	6.5	OB Pressure at the top Layer(psf)	4365	**Setlement**	
	Ave. Corrected SPT Value (N_{60})	29	OB Pressure at the center Layer (psf)	4591	**(inches)**	
	Specific Gravity of Soil Solids (G)	2.7	Excess Pressure At Center Due to appliedLoad	357		
(Below the Water Table)	Moisture content (%)	14	Compression Index (C_c)	0.14		
Z=43.65' (At Centre of Layer)	Liquid Limit (%)	30	Recompression Index (C_r)	0.014	0.014	
	Plastic Limit (%)	17	Initial Void Ratio (e_0)	0.46		
	Plasticity Index (%)	13	Settlement due to compression (inches)	0.24		
	Unit Weight of soil (pcf)	132	Settlement due to recompression (inches)	0.02	**0.02**	
D_f=50.0' & Z=46.9'	Submerged Unit Weight of Soil (pcf)	69.6	OB Pressure at the bottom Layer (psf)	4817		

Total Settlement: 0.57
Consolidation Settlement: 0.36
Immediate Settlement: 0.21

CUY-271-0.00 - Retaining Wall (RW-1)
Stress Distribution using 2 V : 1 H Slope Method for Strip Footing

Boring No.: B-007-4-13 **Retaining Wall - RW-1**

Width of the footing B (feet)	**9.5**	**Applied Design Pressure (psf)**				**2000**			
Depth (Z) below the footing (feet)	2.3	12.1	22.1	34.6	52.75				
Vertical Stress Intensity at Z q (psf)	1610	880	601	431	305				

Boring No.: B-007-1-13 **Retaining Wall - RW-1**

Width of the footing B (feet)	**9.5**	**Applied Design Pressure (psf)**				**2000**			
Depth (Z) below the footing (feet)	2.7	7.9	14.15	21.9	33.15	43.65			
Vertical Stress Intensity at Z q (psf)	1557	1092	803	605	445	357			

APPENDIX 3:

SETTLEMENT CALCULATIONS FOR ALL STRUCTURES

Pages 155-177

E.L. ROBINSON ENGINEERING
950 Goodale Boulevard, Suite 180
Grandview Heights, OH 43212
P: (614) 586-0642 **F:** (614) 586-0648

Project	**MAH-680-2.83**
Bridge No.	**MAH-680-0282**
Vestal Road Over I.R.680	

Structure Data

Four-Span continuous painted steel girders with reinforced concrete deck on new semi-integral abutments, new bearings and new cap and column piers

Soil Profile

Site stratigraphy consists of hard silt and clay (A-4a) near the ground surface to an elevation of about 932 ft. where a dense sand and gravel or sand layer (A-1-b, A-3a) 5 to 15 ft. in thickness was encountered. Below this was another hard layer of silt/ sand mixture (A-4a), and beneath it a hard silt layer. No free ground water was encountered to the depths drilled.

Abutments

It should be noted that the work consisted of removal of existing superstructure and the three existing piers and raise the abutment seats. Therefore, new footings are constructed for the piers only. No new footings for the abutments.

Piers

Allowable Bearing Pressure	3.5	tsf
Max. Bearing Pressure	2.5	tsf
Footing width(B) Pier-1	14.5	ft.
Footing width(B) Pier-2	9	ft.
Footing width(B) Pier-3	14.5	ft.
Pier-1 B.O.F Elevation	940.0	ft.
Pier-2 B.O.F Elevation	939.0	ft.
Pier-3 B.O.F Elevation	937.0	ft.

Settlements (No calculations are provided in the original Geotechnical Report)

Substructure	Borehole	Settlement (inches)		
		Calculated	End of Construction	Current
Pier-1	B-001-0-12	0.69	0.480	
Pier-2	B-002-0-12	0.50	0.480	
Pier-3	B-003-0-12	0.47	0.120	

E.L. ROBINSON ENGINEERING
950 Goodale Boulevard, Suite 180
Grandview Heights, OH 43212
P: (614) 586-0642 **F:** (614) 586-0648

JOB	MAH-680-0282
CALCULATED BY	BKA
CHECKED BY	JN
SUBJECT	Pier-1 Settlement

Pier-1 Settlement Calculations

The original Hough method was used to estimate immediate settlement. It was based on uncorrected SPT N-values and included recommendations for cohesionless as well as cohesive soils such as sandy clay and remolded clay, respectively.

$$\Delta H = H(1/C')\log[(\sigma'_{vo} + \Delta\sigma_v)/\sigma'_{vo}]$$

Where:

C = Bearing Capacity Index.

σ'_0 = Initial average effective stress of the subdivided soil layer.

$\Delta\sigma'_v$ = Vertical stress increase in the subdivided soil layer due to applied foundation load.

Pier-1 Information

Allowable Bearing Pressure	3.5	tsf
Max. Bearing Pressure	2.5	tsf
Footing width-B	14.5	ft.
Footing Length-L	50	ft.
B.O.F Elevation	940	ft.
Realted Borehole	B-001-0-12	
D_f	8	ft.
g_{soil}	130	pcf
g_{water}	62.4	pcf
Water Tablel Level	890	ft.

Table 1: Settlement Calculations Using Hough Method

Soil Layer	Elev. (ft.)		Layer Thick. (ft.)	N_{ave}	Mid Point Depth Z (ft.)	$\Delta\sigma$'v (psf)	$\Delta\sigma$'o (psf)	N'_{ave}	C	ΔH_i (in)
	From	To								
1	940	932.5	7.5	49	3.75	3695.4	1527.5	56	132	0.36
2	932.5	920	12.5	70	13.75	2012.8	2827.5	59	168	0.21
3	920	913.8	6.2	75	23.1	1318.9	4043	53	125	0.07
4	913.8	908	5.8	77	29.1	1051.1	4823	50	145	0.04
							Total Settlement		**ΔH**	**0.69**

Total Calculated Settlement= **0.69** **inches**

Measured Settlement at the end of construction=			
	Right Mon.	**0.00**	**inches**
	Left Mon.	**0.48**	**inches**

E.L. ROBINSON
ENGINEERING

950 Goodale Boulevard, Suite 180
Grandview Heights, OH 43212
P: (614) 586-0642 **F:** (614) 586-0648

JOB	MAH-680-0282
CALCULATED BY	BKA
CHECKED BY	JN
SUBJECT	Pier-2 Settlement

Pier-2 Settlement Calculations

The original Hough method was used to estimate immediate settlement. It was based on uncorrected SPT N-values and included recommendations for cohesionless as well as cohesive soils such as sandy clay and remolded clay, respectively.

$$\Delta H = H(1/C')\log[(\sigma'_{vo} + \Delta\sigma_v)/\sigma'_{vo}]$$

Where:

C = Bearing Capacity Index.

σ'_0 = Initial average effective stress of the subdivided soil layer.

$\Delta\sigma'_v$ = Vertical stress increase in the subdivided soil layer due to applied foundation load.

Pier-2 Information

Allowable Bearing Pressure	3.5	tsf
Max. Bearing Pressure	2.5	tsf
Footing width-B	9	ft.
Footing Length-L	50	ft.
B.O.F Elevation	939	ft.
Realted Borehole	B-002-0-12	
D_f	8	ft.
g_{soil}	130	pcf
g_{water}	62.4	pcf
Water Tablel Level	890	ft.

Table 1: Settlement Calculations Using Hough Method

Soil Layer	Elev. (ft.)		Layer Thick. (ft.)	N_{ave}	Mid Point Depth Z (ft.)	$\Delta\sigma'$v (psf)	$\Delta\sigma'$o (psf)	N'_{ave}	C	ΔH_i (in)
	From	To								
1	939	932.5	6.5	49	3.25	3449.3	1462.5	57	162	0.25
2	932.5	928	4.5	78	8.75	2157.6	2177.5	75	265	0.06
3	928	911.5	16.5	80	19.25	1150.1	3542.5	60	162	0.15
4	911.5	903	8.5	90	31.75	675.4	5167.5	56	162	0.03
							Total Settlement		**ΔH**	**0.50**

Total Calculated Settlement= 0.50 inches

Measured Settlement at the end of construction=

Right Mon.	**0.00**	**inches**
Left Mon.	**0.48**	**inches**

E.L. ROBINSON ENGINEERING

950 Goodale Boulevard, Suite 180
Grandview Heights, OH 43212
P: (614) 586-0642 **F:** (614) 586-0648

JOB	MAH-680-0282
CALCULATED BY	BKA
CHECKED BY	JN
SUBJECT	Pier-3 Settlement

Pier-3 Settlement Calculations

The original Hough method was used to estimate immediate settlement. It was based on uncorrected SPT N-values and included recommendations for cohesionless as well as cohesive soils such as sandy clay and remolded clay, respectively.

$$\Delta H = H(1/C')\log[(\sigma'_{vo} + \Delta\sigma_v)/\sigma'_{vo}]$$

Where:

C = Bearing Capacity Index.

σ'_0 = Initial average effective stress of the subdivided soil layer.

$\Delta\sigma'_v$ = Vertical stress increase in the subdivided soil layer due to applied foundation load.

Pier-3 Information

Allowable Bearing Pressure	3.5	tsf
Max. Bearing Pressure	2.5	tsf
Footing width-B	14.5	ft.
Footing Length-L	50	ft.
B.O.F Elevation	937	ft.
Realted Borehole	B-003-0-12	
D_f	8	ft.
g_{soil}	130	pcf
g_{water}	62.4	pcf
Water Tablel Level	890	ft.

Table 1: Settlement Calculations Using Hough Method

Soil Layer	Elev. (ft.)		Layer Thick. (ft.)	N_{ave}	Mid Point Depth Z (ft.)	$\Delta\sigma'v$ (psf)	$\Delta\sigma'o$ (psf)	N'_{ave}	C	ΔH_i (in)
	From	To								
1	937	933.5	3.5	60	1.75	4310.7	1267.5	75	177	0.15
2	933.5	931	2.5	100	4.75	3439.5	1657.5	110	300	0.05
3	931	926.5	4.5	100	8.25	2735.5	2112.5	97	300	0.06
4	926.5	917	9.5	92	15.25	1867.4	3022.5	75	228	0.10
5	917	908	9	77	24.5	1247.6	4225	53	125	0.10
							Total Settlement		**ΔH**	**0.47**

Total Calculated Settlement= **0.47** **inches**

Measured Settlement at the end of construction= **Right Mon.** **0.12** **inches**

Left Mon. **0.12** **inches**

E.L. ROBINSON ENGINEERING
950 Goodale Boulevard, Suite 180
Grandview Heights, OH 43212
P: (614) 586-0642 **F:** (614) 586-0648

Project	**Wall-CUY-77-14.35**
Wall No.	**Wall-1**

Along IR-77

Sta. 73+25.00, 56.33 RT to Sta. 74+18.36, 59.33 RT

Height Ranges from 13.41 to 17.33

Retaining Wall Type

A cast-in-place (CIP) concrete cantilever wall

Soil Profile

Material visually identified as fill was encountered below the pavement to a depth of 25.5 feet and consisted of medium-dense to hard SANDY SILT (A-4a) and dense COARSE AND FINE SAND (A-3a). Natural soils were encountered below the fill to the termination depth of 40.0 feet and consisted of mediumdense FINE SAND (A-3). Groundwater seepage was not encountered and the boring appeared to be dry at the completion of drilling.

Footing Information

Factored Bearing Resistance	6	ksf
Max. Service Load Pressure	2.5	ksf
Footing width-B	12.5	ft.
Footing Length-L	93.36	ft.
B.O.F Elevation	671.25	ft.

Settlements (calculations are not provided in the original Geotechnical Report)

Substructure	Borehole	Settlement (inches)		
		Calculated	End of Construction	Current
Spread footing	BB-104	0.69	0.120	

E.L. ROBINSON ENGINEERING
950 Goodale Boulevard, Suite 180
Grandview Heights, OH 43212
P: (614) 586-0642 **F:** (614) 586-0648

JOB	Wall-CUY-77-14.35
CALCULATED BY	BKA
CHECKED BY	JN
SUBJECT	Wall 1

Rear Abutment Settlement Calculations

The original Hough method was used to estimate immediate settlement. It was based on uncorrected SPT N-values and included recommendations for cohesionless as well as cohesive soils such as sandy clay and remolded clay, respectively.

$$\Delta H = H(1/C')\log[(\sigma'_{vo} + \Delta\sigma_v)/\sigma'_{vo}]$$

Where:

C = Bearing Capacity Index.
σ'_0 = Initial average effective stress of the subdivided soil layer.
$\Delta\sigma'_v$ = Vertical stress increase in the subdivided soil layer due to applied foundation load.

Rear Abutment Information

Factored Bearing Resistance	6	ksf
Max. Service Load Pressure	2.5	ksf
Footing width-B	12.5	ft.
Footing Length-L	93.36	ft.
B.O.F Elevation	671.25	ft.
Realted Borehole	BB-104	
D_f	5	ft.
g_{soil}	130	pcf
g_{water}	62.4	pcf
Water Tablel Level	615	ft.

Table 1: Settlement Calculations Using Hough Method

Soil Layer	Elev. (ft.)		Layer Thick. (ft.)	N_{ave}	Mid Point Depth Z (ft.)	Δσ'v (psf)	Δσ'o (psf)	N'_{ave}	C	ΔH_i (in)
	From	To								
1	671.25	660.3	10.95	43	5.475	1642.2	1361.75	52	120	0.376
2	660.3	657.3	3	36	12.45	1105.1	2268.5	34	83	0.075
3	657.3	655.3	2	46	14.95	981.3	2593.5	40	105	0.032
4	655.3	640.8	14.5	23	23.2	701.1	3666	17	63	0.210
							Total Settlement		**ΔH**	**0.69**

Total Calculated Settlement= **0.69** **inches**

Measured Settlement at the end of construction= **Mon.2** **0.00** **inches**
Mon.1 **0.12** **inches**

E.L. ROBINSON ENGINEERING
950 Goodale Boulevard, Suite 180
Grandview Heights, OH 43212
P: (614) 586-0642 **F:** (614) 586-0648

Project	**Wall-CUY-77-14.35**
Wall No.	**Wall-2 Left Rear Abutment WingWall**

Along IR-77

Sta. 74+24.52, 1.42 LT to Sta. 75+18.48, 1.42 LT

Height Ranges from 16.84 to 19.94

Retaining Wall Type

A cast-in-place (CIP) concrete cantilever wall

Soil Profile

Material visually identified as fill was encountered below the pavement to a depth of 32.0 feet and consisted of medium-dense to very-dense gravel with sand (A-1-b) and dense coarse and fine sand (A-3a). Natural soils were encountered below the fill to the termination depth of 90.0 feet and consisted of medium-dense fine sand (A-3), medium-dense to very-dense coarse and fine sand (A-3a), stiff to very-stiff silt (A-4b), and medium-stiff to stiff silty-clay (A-6b). Groundwater seepage was encountered at a depth of 53.5 feet and groundwater was encountered at a depth of 58.5 feet.

Footing Information

Factored Bearing Resistance	6	ksf
Max. Service Load Pressure	2.9	ksf
Footing width-B	14	ft.
Footing Length-L	93.96	ft.
B.O.F Elevation	672.25	ft.

Settlements (calculations are not provided in the original Geotechnical Report)

Substructure	Borehole	Settlement (inches)		
		Calculated	End of Construction	Current
Spread footing	BB-106	0.89	0.240	

E.L. ROBINSON ENGINEERING
950 Goodale Boulevard, Suite 180
Grandview Heights, OH 43212
P: (614) 586-0642 **F:** (614) 586-0648

JOB	CUY-77-1433
CALCULATED BY	BKA
CHECKED BY	JN
SUBJECT	Wall 2

Rear Abutment Settlement Calculations

The original Hough method was used to estimate immediate settlement. It was based on uncorrected SPT N-values and included recommendations for cohesionless as well as cohesive soils such as sandy clay and remolded clay, respectively.

$$\Delta H = H(1/C')\log[(\sigma'_{vo} + \Delta\sigma_v)/\sigma'_{vo}]$$

Where:

C = Bearing Capacity Index.
σ'_0 = Initial average effective stress of the subdivided soil layer.

$\Delta\sigma'_v$ = Vertical stress increase in the subdivided soil layer due to applied foundation load.

Rear Abutment Information

Factored Bearing Resistance	6	ksf
Max. Service Load Pressure	2.9	ksf
Footing width-B	14	ft.
Footing Length-L	93.96	ft.
B.O.F Elevation	672.25	ft.
Realted Borehole	BB-106	
D_f	5	ft.
g_{soil}	130	pcf
g_{water}	62.4	pcf
Water Tablel Level	615	ft.

Table 1: Settlement Calculations Using Hough Method

Soil Layer	Elev. (ft.)		Layer Thick. (ft.)	N_{ave}	Mid Point Depth Z (ft.)	Δσ'v (psf)	Δσ'o (psf)	N'_{ave}	C	ΔH_i (in)
	From	To								
1	672.25	661.4	10.85	44	5.425	1976.0	1355.25	53	123	0.413
2	661.4	652.4	9	38	15.35	1189.1	2645.5	33	92	0.189
3	652.4	632.4	20	35	29.85	702.7	4530.5	23	86	0.175
4	632.4	613.4	19	15	49.35	420.2	7065.5	8	51	0.112
							Total Settlement		**ΔH**	**0.89**

Total Calculated Settlement= **0.89** **inches**

Measured Settlement at the end of construction= **Mon.2** **0.24** **inches**
Mon.1 **0.24** **inches**

E.L. ROBINSON ENGINEERING
950 Goodale Boulevard, Suite 180
Grandview Heights, OH 43212
P: (614) 586-0642 **F:** (614) 586-0648

Project — **Wall-CUY-77-14.35**
Wall No. — **Wall-3 Right Forward Abutment WingWall**
Along IR-77
Sta. 79+36.52, 1.42 RT to Sta. 80+30.48, 1.42 RT
Height Ranges from 18.12 to 19.19

Retaining Wall Type

A cast-in-place (CIP) concrete cantilever wall

Soil Profile

Material visually identified as fill was encountered below the pavement to a depth of 32.0 feet and consisted of dense sandy-silt (A-4a), dense silt (A-4b), medium-dense fine sand (A-3) and dense to very-dense coarse and fine sand (A-3a). Natural soils were encountered below the fill to the termination depth of 90.0 feet and consisted of medium-dense to very-dense fine sand (A-3), medium-stiff to dense sandy silt (A-4a), and dense silt (A-4b). Groundwater seepage was encountered at a depth of 58.5 feet and water was measured at a depth of 68.5 feet at the

Footing Information

Factored Bearing Resistance	6	ksf
Max. Service Load Pressure	2.9	ksf
Footing width-B	14	ft.
Footing Length-L	93.96	ft.
B.O.F Elevation	677.5	ft.

Settlements (calculations are not provided in the original Geotechnical Report)

Substructure	Borehole	Settlement (inches)		
		Calculated	End of Construction	Current
Spread footing	BB-111	0.76	0.240	

E.L. ROBINSON
ENGINEERING

950 Goodale Boulevard, Suite 180
Grandview Heights, OH 43212
P: (614) 586-0642 **F:** (614) 586-0648

JOB	CUY-77-1433
CALCULATED BY	BKA
CHECKED BY	JN
SUBJECT	Wall 3

Rear Abutment Settlement Calculations

The original Hough method was used to estimate immediate settlement. It was based on uncorrected SPT N-values and included recommendations for cohesionless as well as cohesive soils such as sandy clay and remolded clay, respectively.

$$\Delta H = H(1/C')\log[(\sigma'_{vo} + \Delta\sigma_v)/\sigma'_{vo}]$$

Where:

C = Bearing Capacity Index.

σ'_0 = Initial average effective stress of the subdivided soil layer.

$\Delta\sigma'_v$ = Vertical stress increase in the subdivided soil layer due to applied foundation load.

Rear Abutment Information

Factored Bearing Resistance	6	ksf
Max. Service Load Pressure	2.9	ksf
Footing width-B	14	ft.
Footing Length-L	93.96	ft.
B.O.F Elevation	677.5	ft.
Realted Borehole	BB-111	
D_f	5	ft.
g_{soil}	130	pcf
g_{water}	62.4	pcf
Water Tablel Level	615	ft.

Table 1: Settlement Calculations Using Hough Method

Soil Layer	Elev. (ft.)		Layer Thick. (ft.)	N_{ave}	Mid Point Depth Z (ft.)	$\Delta\sigma$'v (psf)	$\Delta\sigma$'o (psf)	N'_{ave}	C	ΔH_i (in)
	From	To								
1	677.5	668.3	9.2	46	4.6	2080.9	1248	58	123	0.382
2	668.3	658.3	10	41	14.2	1250.7	2496	37	110	0.192
3	658.3	648.4	9.9	39	24.15	846.6	3789.5	28	105	0.099
4	648.4	638.4	10	41	34.1	619.3	5083	26	70	0.086
							Total Settlement		**ΔH**	**0.76**

Total Calculated Settlement= **0.76** **inches**

Measured Settlement at the end of construction=	**Mon.2**	**0.24**	**inches**
	Mon.1	**0.24**	**inches**

E.L. ROBINSON ENGINEERING
950 Goodale Boulevard, Suite 180
Grandview Heights, OH 43212
P: (614) 586-0642 **F:** (614) 586-0648

JOB	CUY-77-14.35
CALCULATED BY	BKA
CHECKED BY	JN
SUBJECT	Wall 4

Project Wall-CUY-77-14.35
Wall No. Wall-4
Along IR-77
Sta. 81+85.00, 59.33 LT to Sta. 80+36.64, 59.33 LT
Height Ranges from 15.13 to 18.74

Retaining Wall Type

A cast-in-place (CIP) concrete cantilever wall

Soil Profile

Fill material was encountered to a depth of 23.0 feet and consisted of medium-dense SANDYSILT (A-4a) and dense GRAVEL WITH SAND (A-1-b). Natural soils were encountered below the fill to the termination depth of 40.0 feet and consisted of soft to medium-stiff SILT AND CLAY (A-6a), medium-dense to dense COARSE AND FINE SAND (A-3a), and dense FINE SAND (A-3). No groundwater seepage was encountered and the boring appeared to be dry at the completion of drilling.

Footing Information

Factored Bearing Resistance	6	ksf	
Max. Service Load Pressure	2.9	ksf	
Footing width-B	10	ft.	
Footing Length-L	148.36	ft.	15.13
B.O.F Elevation	676.5	ft.	

Settlements (calculations are not provided in the original Geotechnical Report)

Substructure	Borehole	Settlement (inches)		
		Calculated	End of Construction	Current
Spread footing	BB-112	0.54	0.120	

E.L. ROBINSON
ENGINEERING

950 Goodale Boulevard, Suite 180
Grandview Heights, OH 43212
P: (614) 586-0642 **F:** (614) 586-0648

JOB	CUY-77-14.35
CALCULATED BY	BKA
CHECKED BY	JN
SUBJECT	Wall 4

Rear Abutment Settlement Calculations

The original Hough method was used to estimate immediate settlement. It was based on uncorrected SPT N-values and included recommendations for cohesionless as well as cohesive soils such as sandy clay and remolded clay, respectively.

$$\Delta H = H(1/C')\log[(\sigma'_{v_o} + \Delta\sigma_v)/\sigma'_{v_o}]$$

Where:

C = Bearing Capacity Index.

σ'_0 = Initial average effective stress of the subdivided soil layer.

$\Delta\sigma'_v$ = Vertical stress increase in the subdivided soil layer due to applied foundation load.

Rear Abutment Information

Factored Bearing Resistance	6	ksf
Max. Service Load Pressure	2.9	ksf
Footing width-B	10	ft.
Footing Length-L	148.36	ft.
B.O.F Elevation	676.5	ft.
Realted Borehole	BB-112	
D_f	5	ft.
g_{soil}	130	pcf
g_{water}	62.4	pcf
Water Tablel Level	615	ft.

Table 1: Settlement Calculations Using Hough Method

Soil Layer	Elev. (ft.)		Layer Thick. (ft.)	N_{ave}	Mid Point Depth Z (ft.)	$\Delta\sigma$'v (psf)	$\Delta\sigma$'o (psf)	N'_{ave}	C	ΔH_i (in)
	From	To								
1	676.5	665.9	10.6	45	5.3	1830.0	1339	55	190	0.250
2	665.9	662.4	3.5	36	12.35	1197.8	2255.5	34	71	0.109
3	662.4	651.9	10.5	46	19.35	874.1	3165.5	37	93	0.143
4	651.9	648.9	3	31	26.1	683.1	4043	22	70	0.035
							Total Settlement		**ΔH**	**0.54**

Total Calculated Settlement= **0.54** **inches**

Measured Settlement at the end of construction=	**Mon.2**	**0.00**	**inches**
	Mon.1	**0.12**	**inches**

950 Goodale Boulevard, Suite 180
Grandview Heights, OH 43212
P: (614) 586-0642 **F:** (614) 586-0648

Project	**FAI-33-7.31**
Bridge No.	**FAI-33-1309**
Delmont Over US33 byPass	

Structure Data

Four-Span continuous HPS 70W composite steel girder with semi-integral type abutment, cap and columns type piers & on spread footings

Soil Profile

Each of the five borings first encountered between 3 and 12 inches of topsoil. Underlying the topsoil, the five borings typically encountered cohesive soils consisting of stiff to hard silt and clay (A-6a) and silty clay (A-6b) to depths of between 10.5 and 20.5 feet. Some of these soils were organic in nature. Underlying these cohesive soils, each of the five borings generally encountered medium dense to very dense non-cohesive soils, including gravel with sand (A-1-b), gravel sand, and silt (A-2-4), fine sand (A-3), and coarse and fine sand (A-3a). These soils were encountered to the completion depths of the borings in boring B-33 and B-34 and were encountered to depths of between 49.5 and 62 feet in borings B-30, B-31, and B-32, where bedrock was encountered. It should be noted that material classified as silt (A-4b) was encountered in boring B-34. However, this material was encountered at depths of greater than 50 feet. Bedrock was encountered in borings B-30, B-31, and B-32 at depths of between 49.5 and 62 feet. The bedrock consisted of medium-hard broken sandstone with RQDs of between 30% and 50%. Water seepage was encountered between at depths of between 7.2 and 17 feet.

Abutments

Allowable Bearing Pressure	2	tsf
Max. Bearing Pressure	1.86	tsf
Footing width-B	8	ft.
Rear Abutment B.O.F Elevation	911.75	ft.
Forward Abutment B.O.F Elevation	912.3	ft.

Piers

Allowable Bearing Pressure	3	tsf
Max. Bearing Pressure	2.93	tsf
Footing width-B	14	ft.
Pier-1 B.O.F Elevation	896.0	ft.
Pier-2 B.O.F Elevation	898.0	ft.
Pier-3 B.O.F Elevation	894.0	ft.

Settlements (No calculations are provided in the original Geotechnical Report)

Substructure	**Borehole**	**Settlement (inches)**		
		Calculated	End of Construction	Current
Rear Abutment	B-30	0.91	1.200	
Forward Abutment	B-34	0.73	0.480	
Pier-1	B-31	1.03	0.600	
Pier-2	B-32	1.30	0.360	
Pier-3	B-33	1.03	0.840	

E.L. ROBINSON
ENGINEERING

950 Goodale Boulevard, Suite 180
Grandview Heights, OH 43212
P: (614) 586-0642 **F:** (614) 586-0648

JOB	FAI-33-1309
CALCULATED BY	BKA
CHECKED BY	JN
SUBJECT	Rear Abut. Settlement

Rear Abutment Settlement Calculations

The original Hough method was used to estimate immediate settlement. It was based on uncorrected SPT N-values and included recommendations for cohesionless as well as cohesive soils such as sandy clay and remolded clay, respectively.

$$\Delta H = H(1/C')\log[(\sigma'_{vo} + \Delta\sigma_v)/\sigma'_{vo}]$$

Where:

C = Bearing Capacity Index.

σ'_0 = Initial average effective stress of the subdivided soil layer.

$\Delta\sigma'_v$ = Vertical stress increase in the subdivided soil layer due to applied foundation load.

Rear Abutment Information

Allowable Bearing Pressure	2	tsf
Max. Bearing Pressure	1.86	tsf
Footing width-B	8	ft.
Footing Length-L	54.58333333	ft.
B.O.F Elevation	911.75	ft.
Realted Borehole	B-30	
D_f	5.5	ft.
g_{soil}	130	pcf
g_{water}	62.4	pcf
Water Tablel Level	887.8	ft.

Table 1: Settlement Calculations Using Hough Method

Soil Layer	Elev. (ft.)		Layer Thick. (ft.)	N_{ave}	Mid Point Depth Z (ft.)	Δσ'v (psf)	Δσ'o (psf)	N'_{ave}	C	ΔH_i (in)
	From	To								
1	911.75	898.3	13.45	30	6.725	1799.4	1589.25	34	115	0.462
2	898.3	895.8	2.5	20	14.7	1032.9	2626	17	69	0.063
3	895.8	893.3	2.5	28	17.2	898.0	2951	23	55	0.063
4	893.3	885.8	7.5	21	22.2	700.5	3601	16	45	0.154
5	885.8	883.3	2.5	46	27.2	564.3	4173	32	67	0.025
6	883.3	881.8	1.5	24	29.2	521.2	4308.2	16	69	0.013
7	881.8	878.8	3	34	31.45	478.6	4460.3	23	64	0.025
8	878.8	871.8	7	42	36.45	401.4	4330.3	29	101	0.032
9	871.8	853.8	18	37	48.95	275.5	5643.3	22	63	0.071
							Total Settlement		**ΔH**	**0.91**

Total Calculated Settlement= **0.91** **inches**

Measured Settlement at the end of construction= **Right Mon.** **0.96** **inches**
Left Mon. **1.20** **inches**

E.L. ROBINSON ENGINEERING
950 Goodale Boulevard, Suite 180
Grandview Heights, OH 43212
P: (614) 586-0642 **F:** (614) 586-0648

JOB	FAI-33-1309
CALCULATED BY	BKA
CHECKED BY	JN
SUBJECT	Forward Abut. Settlement

Forward Abutment Settlement Calculations

The original Hough method was used to estimate immediate settlement. It was based on uncorrected SPT N-values and included recommendations for cohesionless as well as cohesive soils such as sandy clay and remolded clay, respectively.

$$\Delta H = H(1/C')\log[(\sigma'_{vo} + \Delta\sigma_v)/\sigma'_{vo}]$$

Where:

C = Bearing Capacity Index.

σ'_0 = Initial average effective stress of the subdivided soil layer.

$\Delta\sigma'_v$ = Vertical stress increase in the subdivided soil layer due to applied foundation load.

Forward Abutment Information

Allowable Bearing Pressure	2	tsf
Max. Bearing Pressure	1.86	tsf
Footing width-B	8	ft.
Footing Length-L	54.6	ft.
B.O.F Elevation	912.3	ft.
Realted Borehole	B-34	
D_f	5.5	ft.
g_{soil}	130	pcf
g_{water}	62.4	pcf
Water Tablel Level	899.8	ft.

Table 1: Settlement Calculations Using Hough Method

Soil Layer	Elev. (ft.)		Layer Thick. (ft.)	N_{ave}	Mid Point Depth Z (ft.)	Δσ'v (psf)	Δσ'o (psf)	N'_{ave}	C	ΔH_i (in)
	From	To								
1	912.3	907.3	5	29	2.5	2710.2	1040	40	302	0.11
2	907.3	899.8	7.5	24	8.75	1531.2	1852.5	25	109	0.22
3	899.8	892.3	7.5	27	16.25	945.7	2593.5	24	77	0.16
4	892.3	890.3	2	25	21	741.1	2914.6	21	89	0.03
5	890.3	887.3	3	12	23.5	660.4	3083.6	10	56	0.05
6	887.3	883.3	4	51	27	568.9	3320.2	40	155	0.02
7	883.3	873.3	10	34	34	436.6	3793.4	25	98	0.06
8	873.3	865.3	8	31	43	326.4	4401.8	21	56	0.05
9	865.3	855.3	10	50	52	254.0	5010.2	32	86	0.03
							Total Settlement		ΔH	0.73

Total Calculated Settlement= 0.73 inches

Measured Settlement at the end of construction= Right Mon. 0.48 inches
Left Mon. 0.36 inches

E.L. ROBINSON ENGINEERING

950 Goodale Boulevard, Suite 180
Grandview Heights, OH 43212
P: (614) 586-0642 **F:** (614) 586-0648

JOB	FAI-33-1309
CALCULATED BY	BKA
CHECKED BY	JN
SUBJECT	Pier-1 Settlement

Pier-1 Settlement Calculations

The original Hough method was used to estimate immediate settlement. It was based on uncorrected SPT N-values and included recommendations for cohesionless as well as cohesive soils such as sandy clay and remolded clay, respectively.

$$\Delta H = H(1/C')\log[(\sigma'_{vo} + \Delta\sigma_v)/\sigma'_{vo}]$$

Where:

C = Bearing Capacity Index.

σ'_0 = Initial average effective stress of the subdivided soil layer.

$\Delta\sigma'_v$ = Vertical stress increase in the subdivided soil layer due to applied foundation load.

Pier-1 Information

Allowable Bearing Pressure	3	tsf
Max. Bearing Pressure	2.93	tsf
Footing width-B	14	ft.
Footing Length-L	43	ft.
B.O.F Elevation	896	ft.
Realted Borehole	B-31	
D_f	4.4	ft.
g_{soil}	130	pcf
g_{water}	62.4	pcf
Water Tablel Level	887.8	ft.

Table 1: Settlement Calculations Using Hough Method

Soil Layer	Elev. (ft.) From	Elev. (ft.) To	Layer Thick. (ft.)	N_{ave}	Mid Point Depth Z (ft.)	Δσ'v (psf)	Δσ'o (psf)	N'_{ave}	C	ΔH_i (in)
1	896	892.8	3.2	31	1.6	5070.3	780	50	186	0.18
2	892.8	890.3	2.5	35	4.45	4029.6	1150.5	46	120	0.16
3	890.3	887.8	2.5	22	6.95	3371.1	1475.5	26	112	0.14
4	887.8	885.3	2.5	39	9.45	2868.2	1722.5	42	162	0.08
5	885.3	875.3	10	30	15.7	2023.5	2145	29	157	0.22
6	875.3	871.3	4	39	22.7	1463.1	2618.2	34	137	0.07
7	871.3	866.3	5	39	27.2	1219.7	2922.4	32	126	0.07
8	866.3	853.3	13	48	36.2	887.3	3530.8	36	140	0.11
							Total Settlement		**ΔH**	**1.03**

Total Calculated Settlement= **1.03** **inches**

Measured Settlement at the end of construction=	**Right Mon.**	**0.60**	**inches**
	Left Mon.	**0.48**	**inches**

E.L. ROBINSON ENGINEERING

950 Goodale Boulevard, Suite 180
Grandview Heights, OH 43212
P: (614) 586-0642 **F:** (614) 586-0648

JOB	FAI-33-1309
CALCULATED BY	BKA
CHECKED BY	JN
SUBJECT	Pier-2 Settlement

Pier-2 Settlement Calculations

The original Hough method was used to estimate immediate settlement. It was based on uncorrected SPT N-values and included recommendations for cohesionless as well as cohesive soils such as sandy clay and remolded clay, respectively.

$$\Delta H = H(1/C')\log[(\sigma'_{vo} + \Delta\sigma_v)/\sigma'_{vo}]$$

Where:

C = Bearing Capacity Index.
σ'_0 = Initial average effective stress of the subdivided soil layer.

$\Delta\sigma'_v$ = Vertical stress increase in the subdivided soil layer due to applied foundation load.

Pier-2 Information

Allowable Bearing Pressure	3	tsf
Max. Bearing Pressure	2.93	tsf
Footing width-B	14	ft.
Footing Length-L	43	ft.
B.O.F Elevation	898	ft.
Realted Borehole	B-32	
D_f	5.5	ft.
g_{soil}	130	pcf
g_{water}	62.4	pcf
Water Tablel Level	890.6	ft.

Table 1: Settlement Calculations Using Hough Method

Soil Layer	Elev. (ft.)		Layer Thick. (ft.)	N_{ave}	Mid Point Depth Z (ft.)	Δσ'v (psf)	Δσ'o (psf)	N'_{ave}	C	ΔH_i (in)
	From	To								
1	898	894.6	3.4	28	1.7	5026.7	936	41	160	0.21
2	894.6	892.1	2.5	37	4.65	3969.7	1319.5	46	260	0.07
3	892.1	887.1	5	29	8.4	3064.0	1807	31	97	0.27
4	887.1	884.6	2.5	30	12.15	2446.1	2216.5	28	119	0.08
5	884.6	882.1	2.5	19	14.65	2135.8	2385.5	17	71	0.12
6	882.1	880.1	2	19	16.9	1905.9	2537.6	17	77	0.08
7	880.1	877.1	3	8	19.4	1692.6	2706.6	7	40	0.19
8	877.1	873.1	4	36	22.9	1450.7	2943.2	30	119	0.07
9	873.1	868.1	5	47	27.4	1210.4	3403.4	36	120	0.07
10	868.1	858.1	10	58	34.9	926.1	4066.4	41	155	0.07
11	858.1	853.1	5	34	42.4	732.4	4698.2	22	90	0.04
12	853.1	843.1	10	50	49.9	594.3	5392.4	30	116	0.05
							Total Settlement		**ΔH**	**1.30**

Total Calculated Settlement= 1.30 inches

Measured Settlement at the end of construction= Right Mon. 0.36 inches
Left Mon. 0.24 inches

E.L. ROBINSON ENGINEERING

950 Goodale Boulevard, Suite 180
Grandview Heights, OH 43212
P: (614) 586-0642 F: (614) 586-0648

JOB	FAI-33-1309
CALCULATED BY	BKA
CHECKED BY	JN
SUBJECT	Pier-3 Settlement

Pier-3 Settlement Calculations

The original Hough method was used to estimate immediate settlement. It was based on uncorrected SPT N-values and included recommendations for cohesionless as well as cohesive soils such as sandy clay and remolded clay, respectively.

$$\Delta H = H(1/C')\log[(\sigma'_{vo} + \Delta\sigma_v)/\sigma'_{vo}]$$

Where:

C = Bearing Capacity Index.

σ'_0 = Initial average effective stress of the subdivided soil layer.

$\Delta\sigma'_v$ = Vertical stress increase in the subdivided soil layer due to applied foundation load.

Pier-3 Information

Allowable Bearing Pressure	3	tsf
Max. Bearing Pressure	2.93	tsf
Footing width-B	14	ft.
Footing Length-L	43	ft.
B.O.F Elevation	894	ft.
Realted Borehole	B-33	
D_f	4.34	ft.
g_{soil}	125	pcf
g_{water}	62.4	pcf
Water Tablel Level	889.3	ft.

Table 1: Settlement Calculations Using Hough Method

Soil Layer	Elev. (ft.)		Layer Thick. (ft.)	N_{ave}	Mid Point Depth Z (ft.)	$\Delta\sigma'v$ (psf)	$\Delta\sigma'o$ (psf)	N'_{ave}	C	ΔH_i (in)
	From	To								
1	894	889.3	4.7	35	2.35	4757.7	836.25	54	185	0.25
2	889.3	886.3	3	39	6.2	3549.6	1223.9	50	92	0.23
3	886.3	872.3	14	45	14.7	2130.3	1943.2	46	129	0.42
4	872.3	867.8	4.5	35	23.95	1388.5	3395.85	27	61	0.13
							Total Settlement		**ΔH**	**1.03**

Total Calculated Settlement= **1.03** **inches**

Measured Settlement at the end of construction= **Right Mon.** **0.84** **inches**

Left Mon. **0.36** **inches**

E.L. ROBINSON ENGINEERING
950 Goodale Boulevard, Suite 180
Grandview Heights, OH 43212
P: (614) 586-0642 **F:** (614) 586-0648

JOB	CUY-271-0.00
CALCULATED BY	BKA
CHECKED BY	JN
SUBJECT	RW-1 (WS1) Settlement

Project CUY-271-0.00
Wall No. RW-1 (WS1)
Along lane W-S
From Sta. 3243+75 to Sta.3247+25
Height Range from 6.78 to 12.39

Retaining Wall Type

A cast-in-place (CIP) concrete cantilever wall

Soil Profile

The subsurface soils encountered were predominantly cohesive in nature and consisted of both fill materials and natural soils. The fill materials encountered above the natural soils consisted of silt and clay (A-6a) and silty clay (A-6b). The approximate thickness of the fill material was 8.5 feet in test boring B-007-1-13 and 3.5 feet in test boring B-007-4-13. Natural soils encountered above bedrock in test boring B-007-4-13 and to the termination depth in test boring B-007-1-13 consisted of sandy silt (A-4a), silt and clay (A-6a), non-plastic sandy silt (A-4a), and coarse and fine sand (A-3a). Bedrock consisting of gray, severely to highly weathered shale was encountered at an approximate depth of 59.8 feet in test boring B-007-4-13. The consistency of the cohesive soils ranged from "medium stiff" to "hard" but was generally "very stiff". The relative density of the non-cohesive soils ranged from “dense” to "very dense".

Footing Information

Factored Bearing Resistance	8.6	ksf
Max. Service Load Pressure	1.71	ksf
Footing width-B	9.5	ft.
Footing Length-L	350	ft.
B.O.F Elevation	1037	ft.

Settlements (calculations are provided in the original Geotechnical Report)

Substructure	Borehole	Settlement (inches)		
		Calculated	End of Construction	Current
Spread footing	B-007-4-13	0.78	0.0	N.A
Spread footing	B-007-1-13	0.57	0.0	N.A

RETAINING WALL SETTLEMENT ANALYSES - RETAINING WALL RW-1

Project:	CUY-271-0.00	**Project #**	G13005G	**Test Boring #**	B-007-4-13
Type of Foundation	Compression Index (Cc) (From Lab Test)			Depth of Ground Water Level (feet)	18.5
Shallow Foundation (Continuous)	Recompression Index (Cr) (From Lab Test)			Unit Weight of Water (pcf)	62.4
Length = 350'	Depth of Footing (D_f) below ground (feet)	1.0		Specific Gravity of Soil Solids (G)	
Width = 9.5'	Applied Design Pressure (psf)	2,000		Unit Weight of Soil above the base of foundation (pcf)	125
Depth Below the Foundation (Z)	**AVERAGE PROPERTIES**		**CALCULATIONS**		**Total**
D_f= -1.1' & Z=0.0	Thickness of Layer (feet)	4.6	OB Pressure at the top Layer(psf)	0	**Setlement**
	Ave. Corrected SPT Value (N_{60})	20	OB Pressure at the center Layer (psf)	311	**(inches)**
	Specific Gravity of Soil Solids (G)	2.65	Excess Pressure At Center Due to appliedLoad	1610	
(Above Water Table)	Moisture content (%)	8	Bearing Capacity Index (C)	190	
Z=2.30' (At Centre of Layer)	Liquid Limit (%)	NP	Immediate Settlement in Foundation Soil (inches)	0.23	**0.23**
	Plastic Limit (%)	NP	Initial Void Ratio (e_0)	0.32	
	Plasticity Index (%)	NP			
	Unit Weight of soil (pcf)	135			
D_f=3.5' & Z=4.6'	Submerged Unit Weight of Soil (pcf)		OB Pressure at the bottom Layer (psf)	621	
Df=3.5' & Z=4.6'	Thickness of Layer (feet)	15	OB Pressure at the top Layer(psf)	621	**Setlement**
	Ave. Corrected SPT Value (N_{60})	17	OB Pressure at the center Layer (psf)	1559	**(inches)**
	Specific Gravity of Soil Solids (G)	2.7	Excess Pressure At Center Due to appliedLoad	880	
(above Water Table)	Moisture content (%)	15	Compression Index (C_c)	0.15	
Z=12.10' (At Centre of Layer)	Liquid Limit (%)	29	Recompression Index (C_r)	0.015	0.015
	Plastic Limit (%)	18	Initial Void Ratio (e_0)	0.55	
	Plasticity Index (%)	11	Settlement due to compression (inches)	3.39	
	Unit Weight of soil (pcf)	125	Settlement due to recompression (inches)	0.34	**0.34**
D_f=18.5' & Z=19.6'	Submerged Unit Weight of Soil (pcf)		OB Pressure at the bottom Layer (psf)	2496	
D_f=18.5' & Z=19.6'	Thickness of Layer (feet)	5	OB Pressure at the top Layer(psf)	2496	**Setlement**
	Ave. Corrected SPT Value (N_{60})	60	OB Pressure at the center Layer (psf)	2678	**(inches)**
	Specific Gravity of Soil Solids (G)	2.65	Excess Pressure At Center Due to appliedLoad	601	
(below Water Table)	Moisture content (%)	11	Bearing Capacity Index (C)	190	
Z=22.1' (At Centre of Layer)	Liquid Limit (%)	NP	Immediate Settlement in Foundation Soil (inches)	0.03	**0.03**
	Plastic Limit (%)	NP	Initial Void Ratio (e_0)	0.36	
	Plasticity Index (%)	NP			
	Unit Weight of soil (pcf)	135			
D_f=23.5' & Z=24.6'	Submerged Unit Weight of Soil (pcf)	72.6	OB Pressure at the bottom Layer (psf)	2859	

Project:	CUY-271-0.00		Project #	G13005G	Test Boring #	B-007-4-13
Df=23.5' & Z=24.6'	Thickness of Layer (feet)	20	OB Pressure at the top Layer(psf)	2859		**Setlement**
	Ave. Corrected SPT Value (N_{60})	15	OB Pressure at the center Layer (psf)	3435		**(inches)**
	Specific Gravity of Soil Solids (G)	2.7	Excess Pressure At Center Due to appliedLoad	482		
(Below the Water Table)	Moisture content (%)	14	Compression Index (C_c)	0.14		
Z=34.60' (At Centre of Layer)	Liquid Limit (%)	22	Recompression Index (C_r)	0.014		0.014
	Plastic Limit (%)	15	Initial Void Ratio (e_0)	0.60		
	Plasticity Index (%)	7	Settlement due to compression (inches)	1.20		
	Unit Weight of soil (pcf)	120	Settlement due to recompression (inches)	0.12		**0.12**
D_f=43.5' & Z=44.6'	Submerged Unit Weight of Soil (pcf)	57.6	OB Pressure at the bottom Layer (psf)	4011		
Df=43.5' & Z=44.6'	Thickness of Layer (feet)	16.3	OB Pressure at the top Layer(psf)	4011		**Setlement**
	Ave. Corrected SPT Value (N_{60})	28	OB Pressure at the center Layer (psf)	4578		**(inches)**
	Specific Gravity of Soil Solids (G)	2.7	Excess Pressure At Center Due to appliedLoad	305		
(Below the Water Table)	Moisture content (%)	16	Compression Index (C_c)	0.16		
Z=52.75' (At Centre of Layer)	Liquid Limit (%)	30	Recompression Index (C_r)	0.016		0.016
	Plastic Limit (%)	17	Initial Void Ratio (e_0)	0.48		
	Plasticity Index (%)	13	Settlement due to compression (inches)	0.59		
	Unit Weight of soil (pcf)	132	Settlement due to recompression (inches)	0.06		**0.06**
D_f=59.8' & Z=60.9'	Submerged Unit Weight of Soil (pcf)	69.6	OB Pressure at the bottom Layer (psf)	5145		

Total Settlement: **0.78**
Consolidation Settlement: **0.52**
Immediate Settlement: **0.26**

RETAINING WALL SETTLEMENT ANALYSES - RETAINING WALL RW-1

Project:	CUY-271-0.00	**Project #**	G13005G	**Test Boring #**	B-007-1-13
Type of Foundation	Compression Index (Cc) (From Lab Test)			Depth of Ground Water Level (feet)	21
Shallow Foundation (Continuous)	Recompression Index (Cr) (From Lab Test)			Unit Weight of Water (pcf)	62.4
Length = 350'	Depth of Footing (D_f) below ground (feet)	3.1		Specific Gravity of Soil Solids (G)	
Width = 9.5'	Applied Design Pressure (psf)	2,000		Unit Weight of Soil above the base of foundation (pcf)	125

Depth Below the Foundation (Z)	**AVERAGE PROPERTIES**		**CALCULATIONS**		**Total**
D_f= 3.1' & Z=0.0	Thickness of Layer (feet)	5.4	OB Pressure at the top Layer(psf)	388	**Setlement**
	Ave. Corrected SPT Value (N_{60})	20	OB Pressure at the center Layer (psf)	752	**(inches)**
	Specific Gravity of Soil Solids (G)	2.65	Excess Pressure At Center Due to appliedLoad	1557	
(above Water Table)	Moisture content (%)	8	Bearing Capacity Index (C)	190	
Z=2.70' (At Centre of Layer)	Liquid Limit (%)	NP	Immediate Settlement in Foundation Soil (inches)	0.17	**0.17**
	Plastic Limit (%)	NP	Initial Void Ratio (e_0)	0.32	
	Plasticity Index (%)	NP			
	Unit Weight of soil (pcf)	135			
D_f=8.5' & Z=5.4'	Submerged Unit Weight of Soil (pcf)		OB Pressure at the bottom Layer (psf)	1117	
D_f=8.5' & Z=5.4'	Thickness of Layer (feet)	5	OB Pressure at the top Layer(psf)	1117	**Setlement**
	Ave. Corrected SPT Value (N_{60})	20	OB Pressure at the center Layer (psf)	1429	**(inches)**
	Specific Gravity of Soil Solids (G)	2.7	Excess Pressure At Center Due to appliedLoad	1092	
(above Water Table)	Moisture content (%)	16	Compression Index (C_c)	0.16	
Z=7.90' (At Centre of Layer)	Liquid Limit (%)	30	Recompression Index (C_r)	0.016	0.016
	Plastic Limit (%)	18	Initial Void Ratio (e_0)	0.56	
	Plasticity Index (%)	12	Settlement due to compression (inches)	1.51	
	Unit Weight of soil (pcf)	125	Settlement due to recompression (inches)	0.15	**0.15**
D_f=13.5' & Z=10.4'	Submerged Unit Weight of Soil (pcf)		OB Pressure at the bottom Layer (psf)	1742	
Df=13.5' & Z=10.4'	Thickness of Layer (feet)	7.5	OB Pressure at the top Layer(psf)	1742	**Setlement**
	Ave. Corrected SPT Value (N_{60})	19	OB Pressure at the center Layer (psf)	2237	**(inches)**
	Specific Gravity of Soil Solids (G)	2.7	Excess Pressure At Center Due to appliedLoad	803	
(above Water Table)	Moisture content (%)	14	Compression Index (C_c)	0.14	
Z=14.15' (At Centre of Layer)	Liquid Limit (%)	24	Recompression Index (C_r)	0.014	0.014
	Plastic Limit (%)	16	Initial Void Ratio (e_0)	0.46	
	Plasticity Index (%)	8	Settlement due to compression (inches)	1.15	
	Unit Weight of soil (pcf)	132	Settlement due to recompression (inches)	0.12	**0.12**
D_f=21.0' & Z=17.9'	Submerged Unit Weight of Soil (pcf)		OB Pressure at the bottom Layer (psf)	2732	

Project:	CUY-271-0.00		Project #	G13005G	Test Boring #	B-007-1-13
D_f=21.0' & Z=17.9'	Thickness of Layer (feet)	8	OB Pressure at the top Layer(psf)	2732		**Setlement**
	Ave. Corrected SPT Value (N_{60})	60	OB Pressure at the center Layer (psf)	3022		**(inches)**
	Specific Gravity of Soil Solids (G)	2.65	Excess Pressure At Center Due to appliedLoad	605		
(below Water Table)	Moisture content (%)	11	Bearing Capacity Index (C)	190		
Z=21.9' (At Centre of Layer)	Liquid Limit (%)	NP	Immediate Settlement in Foundation Soil (inches)	0.04		**0.04**
	Plastic Limit (%)	NP	Initial Void Ratio (e_0)	0.36		
	Plasticity Index (%)	NP				
	Unit Weight of soil (pcf)	135				
D_f=29.0' & Z=25.9'	Submerged Unit Weight of Soil (pcf)	72.6	OB Pressure at the bottom Layer (psf)	3312		
Df=29.0' & Z=25.9'	Thickness of Layer (feet)	14.5	OB Pressure at the top Layer(psf)	3312		**Setlement**
	Ave. Corrected SPT Value (N_{60})	33	OB Pressure at the center Layer (psf)	3839		**(inches)**
	Specific Gravity of Soil Solids (G)	2.7	Excess Pressure At Center Due to appliedLoad	445		
(Below the Water Table)	Moisture content (%)	13	Compression Index (C_c)	0.13		
Z=33.15' (At Centre of Layer)	Liquid Limit (%)	22	Recompression Index (C_r)	0.013		0.013
	Plastic Limit (%)	15	Initial Void Ratio (e_0)	0.41		
	Plasticity Index (%)	7	Settlement due to compression (inches)	0.76		
	Unit Weight of soil (pcf)	135	Settlement due to recompression (inches)	0.08		**0.08**
D_f=43.5' & Z=40.4'	Submerged Unit Weight of Soil (pcf)	72.6	OB Pressure at the bottom Layer (psf)	4365		
Df=43.5' & Z=40.4'	Thickness of Layer (feet)	6.5	OB Pressure at the top Layer(psf)	4365		**Setlement**
	Ave. Corrected SPT Value (N_{60})	29	OB Pressure at the center Layer (psf)	4591		**(inches)**
	Specific Gravity of Soil Solids (G)	2.7	Excess Pressure At Center Due to appliedLoad	357		
(Below the Water Table)	Moisture content (%)	14	Compression Index (C_c)	0.14		
Z=43.65' (At Centre of Layer)	Liquid Limit (%)	30	Recompression Index (C_r)	0.014		0.014
	Plastic Limit (%)	17	Initial Void Ratio (e_0)	0.46		
	Plasticity Index (%)	13	Settlement due to compression (inches)	0.24		
	Unit Weight of soil (pcf)	132	Settlement due to recompression (inches)	0.02		**0.02**
D_f=50.0' & Z=46.9'	Submerged Unit Weight of Soil (pcf)	69.6	OB Pressure at the bottom Layer (psf)	4817		

Total Settlement: **0.57**

Consolidation Settlement: **0.36**

Immediate Settlement: **0.21**