FINAL FOUNDATIONS REPORT – BRIDGES and ASSOCIATED WINGWALLS

SR1/I-95 Interchange

New Castle County, Delaware

State Project Contract No. 28-090-03
Federal Project No. IM-N056(35)

Prepared for:
Delaware Department of Transportation

RK&K Commission No. 103-059-27U

April 2009

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Final Foundation Report – Bridges and Associated Wingwalls
SR 1 / I-95 Interchange Improvements - Newark, Delaware
Contract No. 28-090-03
Commission No. 103-059-27U

April 24, 2009

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Rummel, Klepper & Kahl, LLP, in conjunction with URS Corporation, is pleased to submit the Final Foundation Report (FFR) for Bridges and Associated Wingwalls within the SR 1/I-95 Interchange Improvements project.

The FFR – Bridges and Associated Wingwalls describes the subsurface exploration program, the general site conditions, proposed construction, the subsurface conditions, and presents geotechnical engineering data for this project.

This FFR supersedes in its entirety the Preliminary Foundation Report (PFR) dated October 1, 2008 prepared by RK&K in conjunction with the URS Corporation. This report also incorporates comments provided by Pennoni Associates, Inc. dated December 4, 2008 and FHWA dated December 5, 2008. A copy of the comments and our responses are contained in Appendix G of this report.

We appreciate having had the opportunity to provide geotechnical consultation for this project.

Very truly yours,
RUMMEL, KLEPPER & KAHL, LLP and URS CORPORATION

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EXECUTIVE SUMMARY

In accordance with our proposal, Rummel, Klepper & Kahl, LLP (RK&K) in conjunction with the URS Corporation (URS) have completed the Final Foundation Report (FFR) for the SR1/I-95 Interchange – Bridges and Associated Wingwalls. A separate FFR was submitted by RK&K/URS which addresses the retaining walls within the project limits. Wingwalls in the FFR – Bridges and Associated Wingwalls is defined by a length of 30-ft from the center line of bearing. Any wall beyond this length is considered to be a retaining wall unless noted otherwise. The exception to this criteria is the wingwalls for Structure S7 - Ramp R1 over SR 7. The length of the Ramp R1 over SR 7, Abutment A wingwalls range from about 40 to 50-ft in length.

The purpose of this study was to determine the general subsurface conditions at the project site and to evaluate those conditions with respect to geotechnical engineering considerations for the proposed construction. The specific scope of our services on this project consisted of evaluating subsurface data acquired using soil borings, in situ testing, laboratory testing; developing geotechnical recommendations; and submitting our findings in a FFR. Based on this geotechnical study, recommendations are provided for bridge foundation design, bridge wingwall foundation design, sequence of construction, and other geotechnical concerns.

This FFR incorporates and supersedes the alternatives analyses for Ramps A, B, C, G1, over SR 7 and Ramp A over I-95 as presented by URS in December 2007. In addition, this FFR incorporates final foundation recommendations developed by RK&K for Ramp B over I-95 and Ramp R1 over SR 7.

This FFR supersedes in its entirety the Preliminary Foundation Report (PFR) – Bridges and Wingwalls dated October 1, 2008 prepared by RK&K in conjunction with the URS Corporation. This report also incorporates comments provided by Pennoni Associates, Inc. dated December 4, 2008 and FHWA dated December 5, 2008. A copy of the comments and our responses are contained in Appendix G of this report.

Bridge and wingwall construction within the limits of the proposed SR 1/I-95 Interchange will consist of the following structures:

Table E.1 – Proposed Bridge and Wingwall Structure Locations				
Bridge No.	**Description**	**Designer**	**No. of Spans**	**Report Section**
S1	Ramp A over I-95	URS	4	Section 2.5.1
S2	Ramp A over SR 7	URS	4	Section 2.5.2
S3	Ramp B over SR 7	URS	2	Section 2.5.3
S4	Ramp C over SR 7	URS	2	Section 2.5.4
S5	Ramp G1 over SR 7	URS	4	Section 2.5.5
S6	Ramp B over Northbound I-95	RK&K	4	Section 2.5.6
S7	Ramp R1 over SR 7	RK&K	3	Section 2.5.7
S8	SR 1 over Eagle Run Bridge	RK&K	Widening	Section 2.5.8

Bridge Foundation Alternative Analysis

The primary purpose of the bridge foundation alterative analysis was to assess suitable foundation types relative to the physical constraints of the site and the subsurface conditions that have been encountered during the initial and supplemental subsurface exploration.

The following bridge foundations were evaluated for construction. Additional discussion for each foundation type is further discussed in the sections indicated below.

- Prestressed- Precast Concrete Piles (Section 5.1.1)
- Driven Cast-in-Place Piles (Section 5.1.2)
- Drilled Shafts (Section 5.1.3)
- Steel H-Piles (Section 5.1.4)
- Steel Pipe Piles (Section 5.1.5)
- Shallow Foundation (Section 5.1.6)

Bridge Wingwall Foundation Alternative Analysis

The primary purpose of the bridge wingwall foundation alterative analysis was to assess suitable wall types relative to the physical constraints of the site and the subsurface conditions that were encountered.

The following wingwall foundations were evaluated. Additional discussion for each foundation type is further discussed in the sections indicated below.

- Cast-in-Place - CIP (Section 5.2.1)
- Cast-in-Place Pile Supported (Section 5.2.2)
- Mechanically Stabilized Earth Walls – MSE (Section 5.2.3)
- Other Wall Types (Section 5.2.4)

Summary of Bridge Foundation Analysis

For the SR 1/I-95 Interchange project, it is recommended that the majority of the bridge structures be supported on 24 to 36-inch diameter driven open-ended steel pipe pile foundations. Detailed recommendations for each structure are in Section 5.4 of this report. As stated previously, low displacement piles are recommended as they provide greater field flexibility to accommodate unanticipated changes in subsurface conditions, specifically at the pile tip where inclusions of dense, discontinuous layers of sand can dramatically influence the pile installation operations. Even though H-piles are feasible for use where they meet minimum unbraced length, comparison of the axial load resistance per lineal foot of installed pile also proves the pipe sections to be more cost efficient than H-piles for the typical applications of this project, see supporting calculations in Appendix F. Additionally, in several pier locations, there is not enough room available to place a large number of H-piles with a moderate capacity, therefore, higher capacity pipe piles are recommended. Some efficiency is gained by using a common foundation type.

The exception to this recommendation is Structure S7 – Ramp R1 over SR 7. Because of the relatively low foundation loads, we recommend the abutments and pier be supported on driven H piles.

Based on the results of the settlement plate data, the elastic and consolidation properties of the soils were determined. Using these results and the in situ testing and laboratory testing, the settlements of the wingwalls will be between 3 to 8-inches. This is more than enough settlement to cease downdrag on the piles as reported in the FFR for the Bridges and Wingwalls. However, the settlements will occur quickly; typically in two to four walls.

It is recommended that MSE retaining walls be used for construction of the bridge wingwalls. To meet design criteria for this project, several abutment wingwalls will require Lightweight Engineered Fill (LWEF) and No. 57 stone to be used within the reinforcement and retained zones. Also, to satisfy bearing and global stability, the minimum reinforcement length was increased for select structures from 0.7H to as much as 1.2H, where H is the height of the MSE wall from top of wall to the leveling pad. Some of the taller wingwalls will be located on relative soft soils; therefore some of the construction of these walls may need to be staged. These topics are further discussed in Section 5.6.1 of this report. Even with these special treatments

and quarantine periods, it will take less time and cost less to construct MSE walls then to use a pile supported CIP concrete wall.

The exception to this recommendation is the Structure S7 – Ramp R1 over SR 7 where the wingwalls should be a CIP concrete wall bearing on driven H piles because of the existing pond the wingwalls will support.

Typically, the drained condition governs the design of the retaining walls; however, at taller portions of the walls, undrained conditions seems to be the controlling failure mode. Therefore, the Contractor is required to verify the external stability using both the drained and undrained condition summarized in Section 5.6 of this report for each retaining wall.

Special considerations include the sequence of construction, anticipated settlement from construction, downdrag on piles, drivability, and constructability. These items are discussed in further detail in Section 5.5 of this report.

Supporting calculations and a preliminary cost comparison are contained in Appendix F of this report.

This executive summary is provided solely for the purpose of overview. Any party that relies on this report must read the full report, including the appendices. This executive summary omits several details, any one of which could be important to the proper application of the report.

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RK&K

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1 INTRODUCTION

In accordance with our proposal, Rummel, Klepper & Kahl, LLP (RK&K) in conjunction with the URS Corporation (URS) have completed the Final Foundation Report (FFR) for the SR1/I-95 Interchange – Bridges and Associated Wingwalls.

The purpose of this study was to determine the general subsurface conditions at the project site, to evaluate those conditions with respect to geotechnical engineering considerations for the proposed construction, and provide final geotechnical recommendations to guide design of bridge foundations and wingwalls. The specific scope of our services on this project consisted of evaluating data acquired using soil borings, in situ testing, laboratory testing, developing geotechnical recommendations, and submitting our findings in a FFR. Based on this geotechnical study, recommendations are provided for bridge foundation design, bridge wingwall foundation design, sequence of construction, and other geotechnical concerns.

RK&K has previously explored the subsurface conditions for the Delaware Turnpike Improvements Project in New Castle County, Delaware. These exploration efforts have been divided into four separate Geotechnical Data Reports (GDR's): Report No. 1 – Mainline, Report No. 2 – SR 1 Interchange, Report No. 3 – Toll Plaza, and Report No. 4 - Northbound Widening. The information contained in these GDR's and supplemental borings drilled for the Churchmans Road Bridge over I-95 and Churchmans Road/SR 7 Interchange, are contained in Appendix B of this report.

A supplemental subsurface exploration program was provided by DelDOT consisting of 33 additional Standard Penetration Test (SPT) Borings and associated laboratory testing in June 2008. The supplemental subsurface exploration program is discussed detail further in Section 3.5 of this report. The results of the supplemental subsurface exploration are contained in Appendix C of this report.

This FFR updates and supersedes the alternatives analyses for Ramps A, B, C, G1, over SR 7 and Ramp A over I-95 as presented by URS in December 2007 and the Preliminary Foundation Report (PFR) – Bridges and Wingwalls dated October 1, 2008 prepared by RK&K in conjunction with the URS Corporation. This report also incorporates comments provided by Pennoni Associates, Inc. dated December 4, 2008 and FHWA dated December 5, 2008. A copy of the comments and our responses are contained in Appendix G of this report.

2 SITE AND PROJECT DESCRIPTION

2.1 SITE DESCRIPTION

The SR 1/I-95 Interchange is a full cloverleaf interchange that connects Interstate 95 to SR 1 and SR 7 in New Castle County, Delaware. I-95 runs from southwest to northeast through the interchange. The north-south intersecting highway is designated as SR 7 to the north of the interchange and SR 1 to the south of the interchange. SR 7 is a principal arterial road, while SR 1 is a multi-lane expressway that connects I-95 to Dover and has become the major north-south spine road in Delaware. Figure A-1 in Appendix A of this report provides a project location plan. Details of the existing interchange and proposed improvements are shown in Figures A-2a through A-2q.

In the northbound direction, I-95 carries five through lanes. There is an auxiliary lane between the SR 273 interchange and the SR 1 interchange that drops at the ramp to southbound SR 1. A weave condition exists on the mainline between the loop ramps to and from SR 7. In the southeast quadrant of the interchange, a single-lane ramp connects northbound SR 1 to northbound I-95. This ramp also contains a slip ramp that provides a direct connection between the Christiana Mall Ring Road and northbound I-95. Five through lanes currently continue north on I-95 beyond the interchange through Churchmans Marsh.

In the southbound direction, I-95 carries four through lanes plus a collector-distributor (C-D) road through the SR 1/I-95 interchange to handle the movements between I-95 and SR 7/SR 1. Traffic heading south on SR 1 from southbound I-95 exits the mainline onto the C-D road, and a weave condition exists on the C-D road between the loop ramps to and from SR 1. Traffic from southbound SR 7 to southbound I-95 merges onto the C-D road before the C-D road rejoins the I-95 mainline. There is no ramp provided for the southbound I-95 to northbound SR 7 movement; this movement is carried by a ramp from southbound I-95 to Churchmans Road, approximately ½ mile upstream from the SR 1/I-95 interchange. Four mainline lanes are provided on southbound I-95, south of the interchange.

A large number of ramp movements are provided in the area, due to the proximity of the 1.6 million square foot Christiana Mall (located just south of I-95 in the southeast quadrant of the interchange) and the SR 7/Churchmans Road interchange, located just north of I-95 in the intensively developed Churchmans Crossing area. Traveling northbound on SR 1 towards the interchange, motorists encounter a diverge ramp to the mall, two merge ramps from the mall, the diverge to northbound I-95, and a weave under the I-95 overpass. Southbound motorists on SR 7 encounter a diverge to southbound I-95, a weave under the I-95 overpass, a merge from

northbound I-95, a diverge to the Mall Access Road, a diverge to Road A, and a merge from Road A within approximately 1.5 miles.

A portion of the embankment that will support the proposed Ramps A, B, C, and G1 was constructed between January and August 2008. The construction, settlement, and our review of this embankment is discussed in Section 2.3 of this report.

2.2 PREVIOUS LOCAL PROJECTS

In the last ten years, two significant DelDOT projects have been constructed in the local area. These include the SR 7 over Churchmans Road (SR-58) Bridge and approach ramps in 1998 and the Churchmans Road (SR-58) over I-95 Bridge in 2004. All of these structures are pile supported and the piles bear in the same geologic formation as is present at the SR 1/I-95 interchange.

2.2.1 SR 7 over Churchmans Road (SR-58) Bridge

This project was originally built using SI units. RK&K has converted these units into traditional units. This bridge is a 51.3m (168.29-ft) long two-span bridge to carry SR 7 over Churchmans Road to eliminate an at-grade intersection. The intersection of SR 7 and Churchmans Road is about 1,650-ft north of the I-95 over SR 1/SR 7 bridges. This bridge is supported on driven concrete-filled, steel shell piles with a butt diameter of 350-mm (14-inches) and a 7.62-m (25-ft) long section that tapers to a 200-mm (8-inch) tip diameter. The Bottom of Footing (BOF) for the full height CIP abutments and wingwalls was EL 15m (EL 49.21-ft) and the tip elevations of the piles were generally about datum or slightly lower. A High Strain Dynamic test and a static pile load test were performed for a test pile in Abutment B, Pile B81. This pile was driven on November 16, 1998 using an ICE I-60S hammer rated at 60,000 ft-lbs; re-strike occurred on November 20, 1998, and the static load test was conducted on November 23, 1998. The CAPWAP analysis is summarized in Table 2.1.

Table 2.1 – CAPWAP Results for Pile B81

Pile Butt Elevation	Test Type	Date	Blow Counts	Pile Tip Elev	Resistances (kips)			Avg Unit Resistance (ksf)		Pile Stress
					R_u	R_{skin}	R_{toe}	Skin	Toe	(ksi)
17.193m (56.40-ft)	EOD	11/16 /98	23/12-inches	+0.526m (+1.73-ft)	NR	NR	NR	NR	NR	NR
17.193m (56.40-ft)	BOR	11/20 /98	23/3-inches	+0.450m (+1.48-ft)	538.5	357.7	180.8	2.95	516	50.46

EOD: End of Driving
BOR: Beginning of Restrike
NR: Not Reported

The pile load test was performed using the quick load test method. The pile was loaded to 400-kips, unloaded and then reloaded to 470-kips. The deflections above a load of 370-kips, including the reload curve, appeared to fall on a straight line suggesting the shaft friction was exceeded above 370-kips, consistent with the CAPWAP results. The maximum pile head deflection was 0.863-inches at 470-kips.

The approach embankments for this bridge were built using MSE walls labeled A through D near the abutments, as summarized in Table 2.2, and 2(H):1(V) slopes away elsewhere. These walls were typically no more than 28-ft high near the abridge abutments and the approach embankments tapered to at-grade about 1,400-ft to the south and 1,200-ft to the north of the abutments. The top of the embankment is about 32-m (105-ft) wide. The contract documents indicated that the retaining walls listed in Table 2.2 were to be constructed as two stage walls, but the construction of these walls was revised in the field to be constructed as one stage walls.

Table 2.2 - MSE Wall Dimensions

Wall	Length (m / ft)	Leveling Pad Elevation (m / ft)	Minimum Height[1] (m / ft)	Maximum Height[1] (m / ft)
A	304 / 997	17.07 / 56.0	2 / 6.6	9.5 / 31.2
B	193 / 633	16.95 / 55.6	3 / 9.8	7.8 / 26.6
C	8.01 / 26.3	19.95 / 65.4	2.2 / 7.2	6 / 19.7
D	11.11 / 36.4	18.15 / 59.54	1.5 / 4.9	5.5 / 18.0

Note:
1. Heights are measured from the top of leveling pad and are not the net amount of fill used to construct the embankments.

The foundation subgrade was improved by installing wick drains to accelerate consolidation to reduce differential settlements and downdrag on the pile foundations. These wick drains were installed between June and December 1998 on a 3.0-m (9.8-ft) triangular pattern. There was difficulty installing the wicks and several needed to be pre-drilled, but in some areas vibratory installation was used. The need for the wick drains was based on the presence of a peat/organic layer between EL 12.2 and 13.6m (EL 40.0 and 44.6-ft) under Abutment A and between EL 6.6m and EL 11.0m (EL 21.6 and EL 36-ft) at Abutment B. This material was too deep to excavate, but was expected to caused settlements and hence downdrag on the piles.

Ten settlement plates were installed to verify the settlement of the MSE approach embankments: five under the North Approach and five under the South Approach. Tables 2.3 and 2.4 summarize the results of the settlement plate readings. These tables were taken from an internal RK&K memorandum dated December 6, 1999. Both RK&K, the design engineer and VSL, the Contractor's engineer, overestimated the amount of settlement. VSL used C_r ranging from 0.029 to 0.058 for the stiff underlying clay and 0.124 for the overlying, softer material in their settlement estimates. RK&K used larger values and also included secondary consolidation in their estimate. Our back calculations using these settlement plate data resulted in C_r = 0.07 for the organic layer and for the stiff clay C_r = 0.02.

Table 2.3 – North Approach: Measurements From Settlement Platforms Along SR 7

Settlement Plate No.	Baseline Station	Offset (m)	Total Measured Settlement mm (inch)	RK&K Est. Settlement mm (inch)	VSL Est. Settlement mm (inch)	Height of Embankment Feet
SP-1	21+765	19.6 LT	46 (1.8)	125 (5)*** Extrapolated	N/C	16
SP-3	21+700	19.5 RT	142 (5.6)	200 (8)	115-170 (4.5-6.8)	21
SP-4	21+700	0	105 (4.2)*	432 (17)	150-210 (6.0-8.4)	28
SP-5	21+750	0	130 (5.1)**	300 (11.8)	N/C	25
SP-6	21+800	0	72 (2.9)	127 (5)	N/C	23

N/C: Not Calculated
* SP-4 was damaged beyond the point of use in March 1999 before the final embankment height was achieved.
** SP-5 was disturbed several times during embankment construction, so the final measurement may not be correct.
*** Value based on extrapolated settlement values estimated at the centerline of the embankment.

Table 2.4 – South Approach: Measurements From Settlement Platforms Along SR 7

Settlement Plate No.	Baseline Station	Offset (m)	Total Measured Settlement mm (inch)	RK&K Est. Settlement mm(inch)	VSL Est. Settlement mm (inch)	Height of Embankment Feet
SP-2	21+517	13.2 LT	104 (4.1)	114 (4.5)** Extrapolated	N/C	12
SP-7	21+500	0	20 (0.8)*	162 (6.4)	N/C	11
SP-8	21+550	0	27 (1.1)*	211 (8.3)	N/C	17
SP-9	21+600	0	42 (1.6)	250 (10)	150-235 (6.0-9.4)	21
SP-10	21+600	14 LT	52 (2)	127 (5)	115-185 (4.6-7.4)	21

N/C: Not Calculated
* SP-7 and 8 were damaged beyond the point of use in March 1999 before the final embankment height was achieved.
** Value based on extrapolated settlement values estimated at the centerline of the embankment.

2.2.2 Churchmans Road (SR-58) over I-95 Bridge

Construction of the I-95 widening was completed in 2008. This bridge replaced the original bridge with a longer span to accommodate the widened I-95. This bridge is a 785.8-ft long, four-span bridge with a sharp skew angle. It crosses I-95 near I-95 baseline Station 465+91.

The piles for the Churchmans Road (SR-58) bridge over I-95 were driven in 2004 with a Delmag pile hammer with a rated energy of 35.278 ft-kips. This bridge is supported on driven concrete-filled, fluted steel shell piles with a butt diameter of 14-inches and a 15-ft long section that tapered to an 8-inch tip diameter. The shells were 3 gauge steel. The Bottom of Footing (BOF) for the full height abutments and piers and the range of pile tip elevations actually driven are summarized in Table 2.5.

Table 2.5 - Churchmans Road over I-95 Foundation Tip Elevations

Location	Abutment A	Pier 1	Pier 2	Pier 3b	Abutment B
BOF Elevation (ft)	68	56.13	54.52	50.91	68
Pile Tip Elevation (ft)	+0.25 to 12.83	-3.42 to +7.58	-0.75 to 15.91	+1.77 to -9.87	+0.08 to +17.0

A High Strain Dynamic Test was performed for a pile in Pier 1 and for two test piles in Pier 3 as summarized in Table 2.6.

Table 2.6 - CAPWAP Results for Churchmans Road Bridge over I-95

Pile Location & Pile No.	Pile Cut Off Elev	Test Type	Date	Blow Counts	Pile Tip Elev	CAPWAP Results					
						Resistances (kips)			Average Unit Resistance (ksf)		Pile Stress (ksi)
						R_u	R_{skin}	R_{toe}	Skin	Toe	
Pier 1 27	57.13	EOD	6-10-04	36/ft	-2.0	NR	NR	NR	NR	NR	NR
Pier 1 27	57.13	BOR	6-14-04	11/0.05-ft	-2.05	593	552	41	2.51	117	NR
Pier 3 TP-3 (5)	51.91	EOD	7-20-04	36/ft	-3	377	262	115	1.37	329	35
Pier 3 TP-3 (5)	51.91	BOR	7-22-04	11/0.11-ft	-3	587	477	110	2.50	315	65
Pier 3 TP-4 (2)	51.91	EOD	7-20-04	14/8-inches[1]	-1	377	262	115	1.12	329	47
Pier 3 TP-4 (2)	51.91	BOR	7-22-04	10/0.11-ft	-1	590	434	56	1.85	160	55

EOD: End of Driving
BOR: Beginning of Restrike
Note 1: Pile buckling damaged one of the transducers near the end of driving.

TP-4 is a TAPERTUBE consisting of a 0.25-inch smooth pile section with a tapered 15-ft tip. This pile was damaged during initial drive and then cut off and spliced for the re-strike. We estimated the unit resistances from available data for the PDA report.

2.3 RAMP A, B, C, AND G1 EMBANKMENT

An embankment will be constructed to support the proposed Ramps A, B, C1, and G1. The embankment will start near Ramp A Station 1237+00 and will continue to about Station 1244+50 for a length of about 750-ft. The height of the proposed final fill will typically be about 38-ft above the existing ground surface. The bottom width of the fill embankment ranges from about 200 to 275-ft, depending on the location of the proposed ramps. The top of the final proposed roadway elevation near Ramp A Station 1237+00 will be near EL 122 and the existing

ground surface is near EL 87. At Ramp A Station 1242+00 the proposed final roadway elevation will be near EL 110 and the existing ground surface is near EL 72. The proposed final roadway elevation at Station 1244+50 will be near EL 104. The proposed embankment will have 2(H):1(V) side slopes.

The settlement monitoring points were installed on a 2-in layer of sand prior to placement of any fill from the DelDOT Contract 25-090-01. The settlement base plate consisted of a 3 x 3-ft thick steel plate with a 1-in diameter threaded galvanized riser pipe located in the center of the plate. The threaded riser pipe was protected by a 3-in diameter independent casing. The riser and protective casing were extended in increments as much as 4-ft long, and the top of the riser and casing were maintained at a minimum height of 1-ft above the fill at all times. The elevations of the settlement monitoring points were surveyed by DelDOT survey crews on the following schedule:

- Upon installation
- 24 to 48-hours after installation
- Weekly during fill operations
- Weekly upon completion of the fill for a period of 30-days, and
- Monthly thereafter

Starting in January 2008, as part of Contract 25-090-01 (Delaware Turnpike Improvements, I-95 Mainline Widening) DelDOT began placing fill for the construction of the Ramp A embankment as shown in Figure E-1 in Appendix E of this report. Six settlement monitoring points were installed to monitor the settlement of the existing ground surface from the construction of the Ramp A, B, C, and G1 embankment. The existing ground surface prior to placement of embankment fill was near EL 86. A total of approximately 18,000-cyds of fill were placed between January and April 2008 to a maximum height of approximately 10 to 12-ft to EL 97. The dimensions of the fill from this contract were approximately 450 x 200-ft with 2(H):1(V) side slopes. Upon completion, the fill placed during this contract was stabilized with temporary seed and straw mulch.

Under DelDOT Contract 27-037-01 (Glenville Wetland Mitigation Bank) additional fill continued to be placed in the same area as DelDOT Contract 25-090-01 for the construction of the Ramp A, B, C, and G1 embankment. A total of approximately 34,500-cyds of additional fill was placed in July and was completed by August 26, 2008. The height of the embankment was raised to approximately 27 to 30-ft. An approximately 5-ft wide bench was constructed at EL 102 in the northern portion of the site and at EL 97 in the southern portion of the site. 2(H):1(V) slopes were extended above this point to the final fill elevation. The final embankment fill elevation

from this work was near EL 113 to EL 116. Upon completion, the fill placed during this contract was stabilized with temporary seed and straw mulch.

The final embankment height in this area will be raised to approximately EL 122, which will require between 6 to 9-ft of additional fill to be placed during construction of the SR 1 interchange.

The settlement monitoring data is summarized in Figures E-2a through E-2f in Appendix E of this report. The location of the settlement monitoring points is contained in Figure E-1 in Appendix E of this report. Figure E-1 also depicts the limits of the Ramp A, B, C, and G1 embankment fill from both contracts.

The settlement monitoring readings began on January 26, 2008 and continued regularly through January 2009. Unfortunately, during the second phase some settlement plates were damaged so that the total settlements are not accurate. The time rate of settlement after about August 26, 2008 is considered to be unreliable. Table 2.7 summarizes the settlement monitoring data as of January 14, 2009.

Supporting calculations are provided in Appendix F.

Table 2.7 – Settlement Plate Data Phase I

Settlement Plate No.	Total Fill (ft)	Total Settlement (in)
SP-1	8.9	0.8
SP-2	8.1	1.4
SP-3	10.9	0.4
SP-4	13.2	0.9
SP-5	11.7	1.1
SP-6	12.0	1.1

Compaction testing was conducted at various stages of construction of the Ramp A, B, C, and G1. Table 2.8 summarizes the material that was placed during the first phase of the embankment construction.

Table 2.8 - Compaction Testing for First Phase of Embankment Construction							
	Wet Unit Weight (pcf)	**Natural Moisture Content (%)**	**Percent Passing No. 200 Sieve**	**LL**	**PI**	**AASHTO Classification**	**Percent Compaction**
Maximum	135.7	10.6	22.5	NP	NP	A-1-b	100.4
Minimum	126.4	7.0	11.2	NP	NP	A-1-b	94.1
Average	130.3	8.4	14.3	NP	NP	A-1-b	96.8

Table 2.9 summarizes the material that was placed during the second phase of the embankment construction.

Table 2.9 – Compaction Testing for Second Phase of Embankment Construction							
	Wet Unit Weight (pcf)	**Natural Moisture Content (%)**	**Percent Passing No. 200 Sieve**	**LL**	**PI**	**AASHTO Classification**	**Percent Compaction**
Maximum	138.8	17.0	58.5	31.1	9.5	A-4(2)	100.9
Minimum	127.5	9.5	26.0	25.2	NP	A-2-4(0)	93.5
Average	131.1	14.3	42.5	27.7	-	-	97.4

The results of the test embankment evaluations were used to estimate soil parameters for settlement of other embankments and structures. These evaluations are in Section 4.3 of this report.

Based on our review of the compaction and material testing conducted during the construction of the Ramp A, B, C, and G1 embankment, to date, it is our opinion that the embankment:

- Was constructed with satisfactory borrow material for embankments in accordance with Section 209 – Borrow, Delaware Department of Transportation; Specifications for Road and Bridge Construction, dated August 2001 with supplements.
- Was placed and compacted in accordance with in accordance with Section 202 – Excavation and Embankment, Delaware Department of Transportation; Specifications for Road and Bridge Construction, dated August 2001 with supplements.

2.4 EXISTING STRUCTURES WITHIN THE PROJECT LIMITS

2.4.1 SR 7 Re-Alignment Fly-over Bridge No. 223

The existing Mall Access Road is a three-span bridge which carries traffic over the NB and SB lanes of SR 7. According to Delaware plans entitled "***Route 7 Re-Alignment and Fly-Over Bridge No. 223***" from 1977, the total length of the bridge is 217-ft, 9 $^{3}/_{8}$-in. The span lengths for this structure are summarized in Table 2.10.

Table 2.10 – Bridge No. 223 Span Lengths

Span Number	Location	Length (ft)
1	West Abutment to Pier 1	70.53
2	Pier 1 to Pier 2	80.0
3	Pier 2 to Abutment B	67.3

The total width of the bridge is 70-ft from out to out; Existing Ramp A carries three 12-ft lanes of traffic on the bridge and existing Ramp B carries one. A 6-ft median separates existing Ramps A and B and 6-ft, 8-in shoulders are on either side of the bridge. The top of the bridge's roadway ranges approximately from EL 77 at the East Abutment to EL 80 at the West Abutment. The bridge provides a clearance of 14.5-ft over the SB lanes of SR 7 and a 15-ft, 3-in clearance over the NB lanes. The abutments and piers are supported on cast-in-place concrete piles (12-in diameter, 7 Ga) driven to a 45-ton bearing capacity. This bridge will be demolished and replaced with a new bridge; Structure S7 – Ramp R1 over SR 7 as described in Section 2.5.7 of this report.

2.4.2 SR 1 over Eagle Run

According to the Route 7 over Eagle Run General Plans of unknown date, the existing SR 1 over Eagle Run Bridge is a three span structure with a length totaling 200-ft. The width of the existing bridge for the SR 1 northbound crossing is 42-ft and 54-ft along southbound SR 1. There is a 28-ft median between the bridges. The span lengths for this structure are summarized in Table 2.11.

Table 2.11 – SR 1 over Eagle Run Span Lengths		
Span Number	**Location**	**Length (ft)**
1	Abutment A to Bent 1	60
2	Bent 1 to Bent 2	70
3	Bent 2 to Abutment B	70

Abutment 1

Abutment 1 crosses SR 1 baseline Station at approximately 373+76.57. The foundation for Abutment 1 consists of 14-in diameter tapered cast-in-place (CIP) concrete piles foundation with a minimum capacity of 60-tons. The pile tip diameter is 8-inches. The length of the uniform taper is 15-ft from the tip. The pile wall thickness is 2-inches and the piles were filled with Class B concrete. The bottom of footing is near EL 2. The minimum pile tip elevation indicated on the plans is EL -49. The width of the foundation is 14-ft and is supported by one row of vertical piles and one row of 3(H):12(V) battered piles.

Bent 1

The centerline of Bent 1 crosses SR 1 baseline Station at 374+36.57. Bent 1 is supported on two rows of 14-in diameter tapered CIP piles with a minimum capacity of 60-tons. The pile tip diameter is 8-inches. The length of the uniform taper is 15-ft from the tip. The pile wall thickness is 2-inches and the piles were filled with Class B concrete. The piles are battered at 2(H):12(V) with a minimum tip elevation as indicated on the plans of EL -56.

Bent 2

The centerline of Bent 2 crosses SR 1 baseline Station at 375+06.57. Bent 2 is supported on two rows of 18-in diameter tapered CIP piles with a minimum capacity of 110-tons. The pile tip diameter is 8-inches. The length of the uniform taper is 15-ft from the tip. The pile wall thickness is 2-inches and the piles are filled with Class B concrete. The piles are battered at 2(H):12(V). The minimum tip elevation indicated on the plans for the piles driven for the southbound foundation is EL -51.0. The minimum tip elevation indicated on the plans for the piles driven for the northbound foundation is EL -66.

Abutment 2

Abutment 2 crosses SR 1 baseline Station at 375+76.57. The foundation for Abutment 2 consists of 14-in diameter tapered CIP concrete piles with a minimum capacity of 60-tons. The pile tip diameter is 8-inches. The length of the uniform taper is 15-ft from the tip. The pile wall thickness is 2-inches and the piles were filled with Class B concrete. The bottom of footing is near EL 2. The width of the foundation is 14-ft and is supported by one row of vertical piles and

one row of 3(H):12(V) battered piles. The minimum pile tip elevation indicated on the plans for the battered front row of piles is EL -51.0 for the southbound abutment and EL -45 for the northbound abutment. The minimum pile tip elevation as indicated on the plans for the vertical back row of piles is EL -55 for Wingwall C and EL -60 for Wingwall D.

2.4.3 Road A over SR 7

According to the Road A over SR 7 General Plans of unknown date, the existing Road A over SR 7 is a two span structure with a length totaling 236-ft. Bridge crosses the Ramp A baseline at Station 1275+65. The width of the existing bridge is 44-ft. The span lengths for this structure are summarized in Table 2.12.

Table 2.12 – Road A Bridge Span Lengths

Span Number	Location	Length (ft)
1	Abutment A to Pier	110
2	Pier to Abutment B	136

Abutment 1

The foundation for Abutment 1 consists of 14-in diameter tapered cast-in-place (CIP) concrete piles with a minimum capacity of 60-tons. The pile tip diameter is 8-inches. The length of the uniform taper is 15-ft from the tip. The pile wall thickness is 2-inches and the piles were filled with Class B concrete. The bottom of footing is near EL 45. The minimum pile tip elevation indicated on the plans is EL -7. The width of the foundation is 11-ft and is supported by one row of vertical piles and one row of 3.5(H):12(V) battered piles.

The length of Wingwall A is approximately 30.7-ft. The width and footing elevation varies for the entire length of the foundation. Adjacent to the abutment and for a length of 7.9-ft the bearing elevation of the foundation is at EL 45. The footing width in this section is 11-ft and is supported on two rows 14-inch diameter tapered CIP piles. The front row is battered at 3.5(H):1(V) and the back row is vertical. The foundation elevation for the middle portion of the wingwall, approximately 17.3-ft, is near EL 53. The footing width in this section is 8-ft and is supported on two rows of 14-inch diameter tapered CIP piles, one battered at 3.5(H):1(V) and one vertical.
The length of Wingwall B is approximately 30-ft. The width and footing elevation varies for the entire length of the foundation. Adjacent to the abutment and for a length of 10.2-ft the bearing elevation of the foundation is at EL 45. The footing width in this section is 11-ft and is supported on two rows 14-inch diameter tapered CIP piles. The front row is battered at 3.5(H):1(V) and the back row is vertical. The foundation elevation for the middle portion of the wingwall,

approximately 17.3-ft, is near EL 53. The footing width in this section is 8-ft and is supported on two rows of 14-inch diameter tapered CIP piles, one battered at 3.5(H):1(V) and one vertical.

Pier

The pier is supported on three 10 x 10-ft wide foundations. Each foundation is supported on three rows of 14-in diameter tapered CIP piles with a minimum capacity of 60-tons. The perimeter piles are battered at 1(H):6(V) and the center pile for each foundation is vertical pile. The minimum tip elevation indicated on the plans of EL -15.The pile tip diameter is 8-inches. The length of the uniform taper is 15-ft from the tip. The pile wall thickness is 2-inches and the piles were filled with Class B concrete. The foundation is supported by three rows of piles,

Abutment 2

The foundation for Abutment 2 consists of 14-in diameter tapered CIP piles with a minimum capacity of 60-tons. The pile tip diameter is 8-inches. The length of the uniform taper is 15-ft from the tip. The pile wall thickness is 2-inches and the piles were filled with Class B concrete. The bottom of footing is near EL 32. The width of the foundation is 15-ft and is supported by one row of vertical piles and two rows of 3.5(H):12(V) battered piles. The minimum pile tip elevation indicated on the plans for the battered front row of piles is EL -15.0.

The length of Wingwall C is approximately 49-ft. The width and footing elevation varies for the entire length of the foundation. Adjacent to the abutment and for a length of 9-ft the bearing elevation of the foundation is at EL 32. The footing width in this section is 15-ft and is supported on three rows of 14-inch diameter tapered CIP piles. The front two rows are battered at 3.5(H):1(V) and the back row is vertical. The foundation elevation for the middle portion of the wingwall, approximately 15-ft, is near EL 40. The footing width in this section is 13-ft and is supported on two rows of 14-inch diameter tapered CIP piles, one battered at 3.5(H):1(V) and one vertical. The remainder of the foundation bears at EL 47. The footing width in this section is 10-ft and is supported on two rows of 14-inch diameter tapered CIP piles, one battered at 3.5(H):1(V) and one vertical.

The length of Wingwall D is approximately 45-ft. The width and footing elevation varies for the entire length of the foundation. Adjacent to the abutment and for a length of 11.3-ft the bearing elevation of the foundation is at EL 32. The footing width in this section is 15-ft and is supported on three rows of 14-inch diameter tapered CIP piles. The front two rows are battered at 3.5(H):1(V) and the back row is vertical. The foundation elevation for the middle portion of the wingwall, approximately 13.35-ft, is near EL 41. The footing width in this section is 13-ft and is supported on two rows of 14-inch diameter tapered CIP piles, one battered at 3.5(H):1(V) and one vertical. The remainder of the foundation bears at EL 47.5. The footing width in this

section is 10-ft and is supported on two rows of 14-inch diameter tapered CIP piles, one battered at 3.5(H):1(V) and one vertical.

2.5 PROPOSED BRIDGE STRUCTURES

Bridge and wingwall construction within the limits of the proposed SR 1/I-95 Interchange will consist of the following structures:

Table 2.13 – Proposed Bridge and Wingwall Structure Locations

Bridge No.	Description	Designer	No. of Spans	Report Section
S1	Ramp A over I-95	URS	4	Section 2.5.1
S2	Ramp A over SR 7	URS	4	Section 2.5.2
S3	Ramp B over SR 7	URS	2	Section 2.5.3
S4	Ramp C over SR 7	URS	2	Section 2.5.4
S5	Ramp G1 over SR 7	URS	4	Section 2.5.5
S6	Ramp B over Northbound I-95	RK&K	4	Section 2.5.6
S7	Ramp R1 over SR 7	RK&K	3	Section 2.5.7
S8	SR 1 over Eagle Run Bridge	RK&K	Widening	Section 2.5.8

In general, wingwalls are measured as 30-ft from the center line of bearing of abutment for this project. Retaining wall structures beyond this length are considered stand alone retaining walls and are addressed in a separate report.

The exception to this is Structure S7: Ramp R1 over SR 7 wingwalls for Abutment A. Wingwall I will be approximately 50-ft long and wingwall II will be approximately 40-ft long. The wingwalls for Abutment B will be less than 30-ft long.

The sequence for construction for the entire interchange project is discussed in Section 5.5.1 of this report.

2.5.1 Structure S1: Ramp A over I-95

Bridge over I-95 will consist of a four span structure from I-95 southbound to SR 1 southbound. The total length of the bridge will be about 809-ft from the centerline of bearing to centerline of bearing. The bridge will have a total width of about 57-ft from out to out. The width of the bridge will accommodate three 12-ft travel lanes. The north shoulder will be 6-ft wide and the

south shoulder 12-ft wide. The radius of curvature from Abutment A to Ramp A Station 1231+07.05 is 1,850-ft. From this station to Abutment B the radius of curvature is 3,700-ft. The top of the bridge roadway surface will range from approximately EL 112 to 122. The span lengths for this structure are summarized in Table 2.14.

Table 2.14 – Structure S1 Span Lengths

Span Number	Location	Length (ft)
1	Abutment A to Pier 1	168
2	Pier 1 to Pier 2	248
3	Pier 2 to Pier 3	215
4	Pier 3 to Abutment B	178

Table 2.15 summarizes the approximate station and top of pier cap elevation for each structural element.

Table 2.15 – Structure S1 Location of Structural Elements

Structural Element	Ramp A Station	Top of Pile Cap EL
Abutment A	1227+80.25	101
Pier 1	1229+48.25	83
Pier 2	1231+96.25	87
Pier 3	1234+11.25	87
Abutment B	1235+89.25	111

The following tables summarize the service limit state and strength limit state reactions for each for the Structure S1 abutments and piers.

Table 2.16 – Structure S1 Service Limit State Reactions						
Structure S6	**Vertical F_z**	**Lateral F_x**	**Lateral F_y**	**M_x**	**Torsion M_z**	**M_y**
	(Kips)	**(Kips)**	**(Kips)**	**(kip-ft)**	**(kip-ft)**	**(kip-ft)**
Abutment A	1610	355	-	-	-	-
Pier 1, 2, & 3						
Case 1	4910	125	245	5855	-	12765
Case 2	4910	80	265	13080	-	5115
Abutment B	1610	355	-	-	-	-

Table 2.17 – Structure S1 Strength Limit State Reactions						
Structure S6	**Vertical F_z**	**Lateral F_x**	**Lateral F_y**	**M_x**	**Torsion M_z**	**M_y**
	(Kips)	**(Kips)**	**(Kips)**	**(kip-ft)**	**(kip-ft)**	**(kip-ft)**
Abutment A	2230	495	-		-	-
Pier 1, 2, & 3						
Case 1	5530	195	275	9790	-	7010
Case 2	5530	50	335	11915	-	1760
Case 3	6185	140	260	5245	-	16020
Case 4	6185	80	280	14900	-	7880
Abutment B	2230	495	-	-	-	-

Abutment A

Abutment A will have a centerline of bearing at Ramp A Station 1227+80.25. The finished deck elevation of the bridge will range from approximately EL 112 to 113 at Abutment A. The associated roadway will be supported by a Mechanically Stabilized Earth (MSE) wall with an exposed height of approximately 17-ft at the front of the abutment and 28-ft at the rear of the abutment. The top of the leveling pad will be near EL. 80. The wingwalls will be MSE's that will extend 30-ft behind the abutment.

Pier 1

Pier 1 will be an integral pier supported on a single column and will be constructed near Ramp A Station 1229+48.25. The ground surface elevation at the pier is near EL 87. The Ramp A road surface at the pier location will be near EL 118.

Pier 2

Pier 2 will be an integral pier supported on a single column and will be constructed near Ramp A Station 1231+96.25. The ground surface elevation at the pier is near EL 91. The Ramp A road surface at the pier location will be near EL 120. Pier 2 for Ramp A over I-95 will be founded within the median of I-95, with approximately 20 ft of available footing width between the NB and SB lanes. Limitations on the footing size, as well as maintenance of traffic during construction are primary considerations for this structure.

Pier 3

Pier 3 will be an integral pier supported on a single column and will be constructed near Ramp A Station 1234+11.25. The ground surface elevation at the pier is near EL 89. The Ramp A road surface at the pier location will be near EL 122.

Abutment B

Abutment B will have a centerline of bearing at Ramp A Station 1235+89.25. The finished deck elevation of the bridge will be approximately EL 122 at Abutment B. The associated roadway will be supported by an MSE wall with an exposed height of approximately 23-ft at the front of the abutment and 32-ft at the rear of the abutment. The top of the leveling pad will be near EL. 86. One wingwall will be an MSE that will extend 30-ft behind the abutment on the northwest side. The east side of the abutment will adjoin the embankment for Ramp B over I-95.

2.5.2 Structure S2: Ramp A over SR 7

Bridge over SR 7 will consist of a four span structure from I-95 southbound to SR 1 southbound. The total length of the bridge will be about 481-ft from the centerline of bearing to centerline of bearing. The bridge will have a total width of about 45-ft from out to out. The width of the bridge will accommodate two 12-ft travel lanes. The east shoulder will be 12-ft wide and the west shoulder 6-ft wide. The radius of curvature for this structure is 1,850-ft. The top of the bridge roadway surface will range from approximately EL 95 to 105. The span lengths for this structure are summarized in Table 2.18.

Table 2.18 – Structure S2 Span Lengths		
Span Number	**Location**	**Length (ft)**
1	Abutment A to Pier 1	106
2	Pier 1 to Pier 2	120
3	Pier 2 to Pier 3	140
4	Pier 3 to Abutment B	115

The MSE walls for the north approach to Ramp A over SR 7 will be on top of an existing slope. Depending on height, the base elevation of the MSE wall may need to be lowered to provide an appropriate bearing capacity for the tall wall, as well as to satisfy the AASHTO requirement for a minimum 4-ft bench.

Table 2.19 summarizes the approximate station and top of pier cap elevation for each structural element.

Table 2.19 – Structure S2 Location of Structural Elements		
Structural Element	**Ramp A Station**	**Top of Pile Cap EL**
Abutment A	1245+81	97
Pier 1	1246+87	62
Pier 2	1248+07	66
Pier 3	1249+47	67
Abutment B	1250+62	86

The following tables summarize the service limit state and strength limit state reactions for each for the Structure S2 abutments and piers.

Table 2.20 – Structure S2 Service Limit State Reactions						
Structure S6	**Vertical F_z**	**Lateral F_x**	**Lateral F_y**	**M_x**	**Torsion M_z**	**M_y**
	(Kips)	**(Kips)**	**(Kips)**	**(kip-ft)**	**(kip-ft)**	**(kip-ft)**
Abutment A	865	200	-	-	-	-
Pier 1 & 3	2525	15	240	7736	-	4500
Pier 2						
Case 1	1405	6	190	6095	-	230
Case 2	950	6	115	3590	-	230
Abutment B	865	200	-	-	-	-

Table 2.21 – Structure S2 Strength Limit State Reactions						
Structure S6	**Vertical F_z**	**Lateral F_x**	**Lateral F_y**	**M_x**	**Torsion M_z**	**M_y**
	(Kips)	**(Kips)**	**(Kips)**	**(kip-ft)**	**(kip-ft)**	**(kip-ft)**
Abutment A	1210	290	-	-	-	-
Pier 1 & 3						
Case 1	2665	35	265	8230	-	1185
Case 2	3200	17	250	8082	-	5970
Pier 2						
Case 1	1375	19	205	6390	-	640
Case 2	1785	8	200	6235	-	275
Case 3	965	19	125	3730	-	640
Case 4	1210	8	120	3725	-	275
Abutment B	1210	290	-	-	-	-

Abutment A

Abutment A will have a centerline of bearing at Ramp A Station 1245+81. The finished deck elevation of the bridge will range from approximately EL 105 to 106 at Abutment A. The associated roadway will be supported by an MSE wall with an exposed height of approximately 23-ft at the front of the abutment and 30-ft at the rear of the abutment. The top of the leveling pad will be near EL. 72. The east wingwall will be an approximately 20-ft long MSE wall and adjoin the Ramp B embankment. The west wingwall will be an approximately 25-ft long MSE wall and adjoin the Ramp G1 abutment.

Pier 1

Pier 1 will be an integral pier supported on a single column and will be constructed near Ramp A Station 1246+87. The ground surface elevation at the pier is near EL 65. The Ramp A road surface at the pier location will be near EL 104.

Pier 2

Pier 2 will be a cross girder supported by two columns and will be constructed near Ramp A Station 1248+07. The ground surface elevation at the pier is near EL 70. The Ramp A road surface at the pier location will be near EL 101.

Pier 3

Pier 3 will be an integral pier supported on a single column and will be constructed near Ramp A Station 1249+47. The ground surface elevation at the pier is near EL 69. The Ramp A road surface at the pier location will be near EL 99.

Abutment B

Abutment B will have a centerline of bearing at Ramp A Station 1250+62. The finished deck elevation of the bridge will range from approximately EL 95 to 96 at Abutment B. The associated roadway will be supported by an MSE wall with an exposed height of approximately 20 ft at the front of the abutment and 27-ft at the rear of the abutment. The top of the leveling pad will be near EL. 66. The wingwalls will be MSE's that will extend 30-ft behind the abutment.

2.5.3 Structure S3: Ramp B over SR 7

Construction of the Ramp B Bridge over SR 7 will consist of a two span structure from SR 1 northbound to I-95 northbound. The total length of the bridge will be about 365-ft from the centerline of bearing to centerline of bearing. The bridge will have a total width of about 45-ft from out to out. The width of the bridge will accommodate two 12-ft travel lanes. The east shoulder will be 12-ft wide and the west shoulder 6-ft wide. The radius of curvature for this structure is 1,825-ft. The top of the bridge roadway surface will range from approximately EL 92 to 95. The span lengths for this structure are summarized in Table 2.22.

Table 2.22 – Structure S3 Span Lengths

Span Number	Location	Length (ft)
1	Abutment A to Pier 1	182.5
2	Pier 1 to Abutment B	182.5

Table 2.23 summarizes the approximate station and top of pier cap elevation for each structural element.

Table 2.23 – Structure S3 Location of Structural Elements

Structural Element	Ramp B Station	Top of Pile Cap EL
Abutment A	1433+28	81
Pier 1	1435+10.50	63
Abutment B	1436+93	85

The following tables summarize the service limit state and strength limit state reactions for each for the Structure S3 abutments and piers.

Table 2.24 – Structure S3 Service Limit State Reactions

Structure S6	Vertical F_z	Lateral F_x	Lateral F_y	M_x	Torsion M_z	M_y
	(Kips)	(Kips)	(Kips)	(kip-ft)	(kip-ft)	(kip-ft)
Abutment A	990	275	-	-	-	-
Pier 1						
Case 1	2440	10	250	7310	-	335
Case 2	1600	37	165	4450	-	335
Abutment B	990	275	-	-	-	-

Table 2.25 – Structure S3 Strength Limit State Reactions

Structure S6	Vertical F_z	Lateral F_x	Lateral F_y	M_x	Torsion M_z	M_y
	(Kips)	(Kips)	(Kips)	(kip-ft)	(kip-ft)	(kip-ft)
Abutment A	1370	385	-	-	-	-
Pier 1						
Case 1	2535	30	275	7760	-	960
Case 2	3095	12	265	7950	-	405
Case 3	1690	30	180	4735	-	950
Case 4	2030	40	175	4820	-	405
Abutment B	1370	385	-	-	-	-

Abutment A

Abutment A will have a centerline of bearing at Ramp B Station 1433+28. The finished deck elevation of the bridge will be approximately EL 92 at Abutment A. The associated roadway will be supported by an MSE wall with an exposed height of approximately 19 ft at the front of the abutment and 28-ft at the rear of the abutment. The top of the leveling pad will be near EL. 61. The west wingwall will be an approximately 20-ft long MSE wall and adjoin the Ramp A embankment. The east wingwall will be approximately a 30-ft long MSE wall.

Pier 1

Pier 1 will be a cross girder supported by two columns and will be constructed near Ramp B Station 1435+10.50. The ground surface elevation at the pier is near EL 67. The Ramp B road surface at the pier location will be near EL 94.

Abutment B

Abutment B will have a centerline of bearing at Ramp B Station 1436+93. The finished deck elevation of the bridge will be approximately EL 95 at Abutment B. The associated roadway will be supported by an MSE wall with an exposed height of approximately 16 feet at the front of the abutment and 25-ft at the rear of the abutment. The top of the leveling pad will be near EL. 67. The wingwalls will be MSE's that will extend 30-ft behind the abutment.

2.5.4 Structure S4: Ramp C over SR 7

Construction of the Ramp C Bridge over SR 7 will consist of a two span structure from I-95 northbound to SR 7 northbound. The total length of the bridge will be about 258-ft from the centerline of bearing to centerline of bearing. The bridge will have a total width of about 30-ft from out to out. The width of the bridge will accommodate one 15-ft travel lane. The north shoulder will be 8-ft wide and the south shoulder 4-ft wide. The radius of curvature from Abutment A to Ramp C Station 1116+14.75 is 485-ft. From this Station to Abutment B the radius of curvature is 600-ft. The top of the bridge roadway surface will range from approximately EL 90 to 97. The span lengths for this structure are summarized in Table 2.26.

Table 2.26 – Structure S4 Span Lengths

Span Number	Location	Length (ft)
1	Abutment A to Pier 1	110
2	Pier 1 to Abutment B	148

Table 2.27 summarizes the approximate station and top of pier cap elevation for each structural element.

Table 2.27 – Structure S4 Location of Structural Elements

Structural Element	Ramp B Station	Top of Pile Cap EL
Abutment A	1114+66.25	89
Pier 1	1115+76.25	71
Abutment B	1117+24	82

The following tables summarize the service limit state and strength limit state reactions for each for the Structure S4 abutments and piers.

Table 2.28 – Structure S4 Service Limit State Reactions

Structure S6	Vertical F_z (Kips)	Lateral F_x (Kips)	Lateral F_y (Kips)	M_x (kip-ft)	Torsion M_z (kip-ft)	M_y (kip-ft)
Abutment A	560	135	-	-	-	-
Pier 1						
Case 1	2345	115	58	1555	-	5425
Case 2	2345	102	78	4195	-	3780
Abutment B	800	155	-	-	-	-

Table 2.29 – Structure S4 Strength Limit State Reactions

Structure S6	Vertical F_z (Kips)	Lateral F_x (Kips)	Lateral F_y (Kips)	M_x (kip-ft)	Torsion M_z (kip-ft)	M_y (kip-ft)
Abutment A	800	200	-	-	-	-
Pier 1						
Case 1	2395	161	56	1960	-	5685
Case 2	2395	121	120	4250	-	4254
Case 3	2973	143	72	1575	-	6344
Case 4	2973	127	97	5010	-	4195
Abutment B	1115	220	-	-	-	-

Abutment A

Abutment A will have a centerline of bearing at Ramp C Station 1114+66.25. The finished deck elevation of the bridge will be approximately EL 97 at Abutment A. The associated roadway will be supported by an MSE wall with an exposed height of approximately 19 ft at the front of the abutment and 25-ft at the rear of the abutment. The top of the leveling pad will be near EL. 69. The wingwalls will be MSE's that will extend 30-ft behind the abutment.

Pier 1

Pier 1 will be an integral pier supported on a single column and will be constructed near Ramp C Station 1115+76.25. The ground surface elevation at the pier is near EL 74. The Ramp G1 road surface at the pier location will be near EL 97.

Abutment B

Abutment B will have a centerline of bearing at Ramp C Station 1117+24. The finished deck elevation of the bridge will range from approximately EL 90 to EL 91 at Abutment B. The associated roadway will be supported by an MSE wall with an exposed height of approximately 16-ft at the front of the abutment and 22-ft at the rear of the abutment. The top of the leveling pad will be near EL. 64. The wingwalls will be MSE's that will extend 30-ft behind the abutment.

2.5.5 Structure S5: Ramp G1 over SR 7

Construction of the Ramp G1 Bridge over SR 7 will consist of a four span structure from I-95 southbound to SR 7 southbound. The total length of the bridge will be about 585-ft from the centerline of bearing to centerline of bearing. The bridge will have a total width of about 36-ft from out to out. The width of the bridge will accommodate one 15-ft travel lane. The east shoulder will be 12-ft wide and the west shoulder 6-ft wide. The radius of curvature from Abutment A to Ramp G1 Station 1313+72.51 will be 835-ft. From this Station to Ramp G1 Station 1311+76.51 the ramp is straight or tangent. From this station to Abutment B the radius of curvature is 2,500-ft. The top of the bridge roadway surface will range from approximately EL 94 to 107. The span lengths for this structure are summarized in Table 2.30.

Table 2.30 – Structure S5 Span Lengths

Span Number	Location	Length (ft)
1	Abutment A to Pier 1	150
2	Pier 1 to Pier 2	161
3	Pier 2 to Pier 3	144
4	Pier 3 to Abutment B	130

The MSE walls for the north approach to Ramp G1 over SR 7 will be located at the crest of an existing slope. Depending on height, the base elevation of the MSE wall may need to be lowered to provide an appropriate bearing capacity for the tall wall, as well as to satisfy the AASHTO requirement for a minimum 4-ft bench.

Table 2.31 summarizes the approximate station and top of pier cap elevation for each structural element.

Table 2.31 – Structure S5 Location of Structural Elements

Structural Element	Ramp B Station	Top of Pile Cap EL
Abutment A	1311+00	98
Pier 1	1313+03	66
Pier 2	1314+64	66
Pier 3	1316+08	63
Abutment B	1317+38	84

The following tables summarize the service limit state and strength limit state reactions for each for the Structure S5 abutments and piers.

Table 2.32 – Structure S5 Service Limit State Reactions

Structure S6	Vertical F_z	Lateral F_x	Lateral F_y	M_x	Torsion M_z	M_y
	(Kips)	(Kips)	(Kips)	(kip-ft)	(kip-ft)	(kip-ft)
Abutment A	900	185	-	-	-	-
Pier 1 to 3						
Case 1	2430	15	250	9230	-	3720
Abutment B	900	185	-	-	-	-

Table 2.33 – Structure S4 Strength Limit State Reactions						
Structure S6	**Vertical F_z**	**Lateral F_x**	**Lateral F_y**	**M_x**	**Torsion M_z**	**M_y**
	(Kips)	**(Kips)**	**(Kips)**	**(kip-ft)**	**(kip-ft)**	**(kip-ft)**
Abutment A	1250	260	-	-	-	-
Pier 1 to 3						
Case 1	2695	44	280	9972	-	1635
Case 2	3062	18	260	9707	-	4932
Abutment B	1250	260	-	-	-	-

Abutment A

Abutment A will have a centerline of bearing at Ramp G1 Station 1311+00. The finished deck elevation of the bridge will range from approximately EL 106 to EL 107 at Abutment A. The associated roadway will be supported by an MSE wall with an exposed height of approximately 31-ft at the front of the abutment and 38-ft at the rear of the abutment. The top of the leveling pad will be near EL. 63. The west wingwall will be an MSE wall that will extend 30-ft behind the abutment. There will be no east wingwall as the east side of the abutment adjoins the Ramp A embankment.

Pier 1

Pier 1 will be an integral pier supported on a single column and will be constructed near Ramp G1 Station 1313+03. The ground surface elevation at the pier is near EL 71. The Ramp G1 road surface at the pier location will be near EL 104.

Pier 2

Pier 2 will be an integral pier supported on a single column and will be constructed near Ramp G1 Station 1314+64. The ground surface elevation at the pier is near EL 70. The Ramp G1 road surface at the pier location will be near EL 101.

Pier 3

Pier 3 will be an integral pier supported on a single column and will be constructed near Ramp G1 Station 1316+08. The ground surface elevation at the pier is near EL 66. The Ramp G1 road surface at the pier location will be near EL 97.

Abutment B

Abutment B will have a centerline of bearing at Ramp G1 Station 1317+38. The finished deck elevation of the bridge will be approximately EL 94 at Abutment B. The associated roadway will

be supported by an MSE wall with an exposed height of approximately 19-ft at the front of the abutment and 27-ft at the rear of the abutment. The top of the leveling pad will be near EL. 63. The wingwalls will be MSE's that will extend 30-ft behind the abutment.

2.5.6 Structure S6: Ramp B over Northbound I-95

Construction of the Ramp B Bridge over the Northbound I-95 roadway will consist of a four span structure carrying a single lane of traffic from SR 7 northbound to I-95 northbound. The bridge superstructure will consist of a CIP concrete deck supported by steel plate girders. The total length of the bridge will be about 750-ft from centerline of bearing to centerline of bearing. The bridge will have a total width of about 33-ft 10.5-in from out to out. The width of the bridge will accommodate one 15-ft travel lane, and 4-ft and 12-ft wide shoulders. The radius of curvature for this structure is approximately 1,900-ft. The elevation of the bridge roadway surface will range from approximately EL 122.80 to EL 114.97 along the Ramp B baseline. The span lengths for this structure are summarized in Table 2.34. Table 2.35 summarizes the approximate station and top of pile cap elevation for each structural element.

Table 2.34 – Structure S6 Span Lengths

Span Number	Location	Length (ft)
1	Abutment A to Pier 1	195
2	Pier 1 to Pier 2	195
3	Pier 2 to Pier 3	195
4	Pier 3 to Abutment B	165

Table 2.35 – Structure S6 Location of Structural Elements

Structural Element	Ramp B Station	Top of Pile Cap EL
Abutment A	450+65	109
Pier 1	452+60	88
Pier 2	454+55	89
Pier 3	456+50	87
Abutment B	458+15	103

The following table summarizes the service limit state and strength limit state reactions for each for the Structure S6 abutments and piers.

Table 2.36 – Structure S6 Service Limit State Reactions						
Structure S6	Vertical F_y	Lateral F_x	Lateral F_z	M_x	Torsion M_y	M_z
	(Kips)	(Kips)	(Kips)	(kip-ft)	(kip-ft)	(kip-ft)
Abutment A	720	30	30	-	-	-
Pier 1	2,800	525	50	2,050	190	11,000
Pier 2	1,285	25	390	4,200	100	400
Pier 3	2,405	710	60	2,450	400	12,000
Abutment B	590	25	60	-	-	-

Table 2.37 – Structure S6 Strength Limit State Reactions						
Structure S6	Vertical F_y	Lateral F_x	Lateral F_z	M_x	Torsion M_y	M_z
	(Kips)	(Kips)	(Kips)	(kip-ft)	(kip-ft)	(kip-ft)
Abutment A	1,050	35	25	-	-	-
Pier 1	3,770	500	50	3,755	270	11,500
Pier 2	1,865	25	515	5,580	100	200
Pier 3	3,470	665	50	3,630	150	11,000
Abutment B	885	30	50	-	-	-

Abutment A

Abutment A will be constructed with centerline of bearing at Ramp B Station 450+65. The bottom of footing (top of pile cap) for Abutment A will be located at approximately EL 109. The top of the leveling pad of the MSE wall will be near EL 87.5. The exposed wall height for the MSE wall will be about 33.5-ft. The Ramp B road surface will be near EL 120.65 and the existing ground surface is near EL 88.

The wingwalls will be parallel MSE's that will extend 30-ft behind the abutment.

Pier 1

Pier 1 will be a concrete integral pier supported on a single column and will be constructed near Ramp B Station 452+60. The ground surface elevation at the pier location is near EL 88.75. The Ramp B road surface elevation at the pier location will be near EL 122.66.

Pier 2

Pier 2 will be a straddle bent, with a concrete-filled steel tub girder supported on two circular concrete columns, and will be constructed near Ramp B Station 454+55. The approximate

ground surface elevation at the pier location is EL 89. The length of the straddle bent will be approximately 89-ft between centers of columns. The Ramp B road surface at the pier location will be near EL 122.24.

Pier 3

Pier 3 will be a concrete integral pier supported on a single column and will be constructed near Ramp B Station 456+50. The ground surface elevation at the pier location is near EL 87. The Ramp B road surface at the pier location will be near EL 119.35.

Abutment B

Abutment B will be constructed with centerline of bearing at Ramp B Station 458+15 and will be located in the median of I-95. The bottom of footing (top of pile cap) for Abutment B will be located at approximately EL 103. The top of the leveling pad of the MSE wall will be near EL 83. The exposed wall height for the MSE wall will be about 32-ft. The Ramp B road surface will be near EL 114.97 and the ground surface is near approximately EL 85.

The wingwalls will be parallel MSE walls that will extend 30-ft behind the abutment.

2.5.7 Structure S7: Ramp R1 over SR 7

The Ramp R1 Bridge will be the new mall access road, and it will consist of a double span bridge over Ramps U, B, and A, and both the Northbound and Southbound lanes of SR 7. In total, the length of the bridge will be about 226-ft from center of bearing to center of bearing. The bridge will have a total width of about 73-ft from out to out. Three travel lanes, 12-ft each, will lead towards the mall and two lanes at 12-ft each will lead away from the mall. A 6-ft median will separate the travel directions. The bridge's roadway surface will range approximately from EL 47.5 to 50. The bridge will provide a minimum clearance of 16.5-ft over the aforementioned Ramps and SR 7. In addition, 30-ft approach slabs will be located at either abutment. The span lengths for this structure are summarized in Table 2.38.

Table 2.38 – Structure S7 Span Lengths

Span Number	Location	Length (ft)
1	Abutment A to Pier 1	114
2	Pier 1 to Abutment B	112

The Table 2.39 summarizes the service limit state and strength limit state reactions per pile for each of the Structure S7 foundation elements.

Table 2.39 – Structure S7 Service and Strength Limit State Reactions		
Structure S7	**Strength Vertical (Kips)**	**Service Vertical (Kips)**
Abutment A	173.5	119.3
Pier 1	155	-
Abutment B	173.5	119.3
Wingwalls	181	123

Abutment A

Abutment A will be constructed at Ramp R1 Station 101+14.11, connecting to the Mall Ring Road. The bottom of footing (top of pile cap) for Abutment A will be located at approximately EL 40, 3-ft below the proposed ground surface of Ramp U. The Ramp R1 road surface will be at approximate elevation of EL 49.4.

Wingwall I will be approximately 50-ft long and will be located west of Abutment A and will parallel Ramp U. The anticipated height of wingwall I, from the top of the pile cap to the Ramp R1 roadway surface will be about 25-ft.

Wingwall II will be approximately 40-ft long and will be located east of Abutment A and will parallel Ramp U. The anticipated height of wingwall II, from the top of the pile cap to the Ramp R1 roadway surface will be about 25-ft.

Center Pier

The center pier supporting this two-span bridge will be constructed at Ramp R1 Station 102+28.27. The pier pile cap is anticipated to be 72-ft long, 10-ft wide, and approximately 4-ft thick and will support five columns. The proposed bottom of footing is anticipated to be near EL 42, approximately 3-ft below the final ground surface elevation. Pre-cast concrete traffic barriers will separate the pier columns which are immediately bordered by Ramp B to the East and Ramp A to the West.

Abutment B

Abutment B will be constructed at Ramp R1 Station 103+40.36. The bottom of Abutment B (top of pile cap) will be located at approximately EL 39, 4-ft below the proposed ground surface elevation of Ramp R1. The Ramp R1 road surface will be at approximate elevation of EL 50.

Wingwall III will be approximately 26-ft long and will be located west of Abutment B. The anticipated height of wingwall III, from the top of the pile cap to the Ramp R1 roadway surface will be about 25-ft.

Wingwall IV will be approximately 26-ft long and will be located east of Abutment B. The anticipated height of wingwall IV, from the top of the pile cap to the Ramp R1 roadway surface will be about 25-ft.

2.5.8 Structure S8: SR 1 over Eagle Run

The proposed construction for SR 1 over Eagle Run will consist of widening the existing northbound SR 1 bridge approximately 12-ft to the west. No new foundations will be required for this bridge. The existing foundation for this structure is discussed in Section 2.2.2 of this report.

3 SUBSURFACE EXPLORATION AND LABORATORY TESTING

The subsurface exploration consisted of reviewing existing published geologic literature and maps, reviewing historic boring data, drilling Standard Penetration Test (SPT) borings, performing Cone Penetrometer Tests (CPT) probes, and Dilatometer Test (DMT) probes, and performing laboratory testing on representative samples.

The subsurface exploration was conducted in two phases. The initial subsurface exploration for the SR 1/I-95 interchange and fifth lane widening projects are discussed in Section 3.2, and the supplemental subsurface exploration is discussed in Section 3.5 of this report. In addition, the initial and supplemental laboratory testing programs are discussed in Sections 3.3 and 3.5.3, respectively.

The following tables, located in Appendix C of this report, have been created to summarize the subsurface exploration and laboratory testing programs from both phases of work for this project.

- Table C-1 – Summary of SPT/CPT/DMT Locations
- Table C-2a – Summary of Laboratory Soil Classification Testing
- Table C-2b – Summary of Thin-Walled Tube Samples
- Table C-2c – Summary of Consolidation Test Data
- Table C-3 – Summary of Groundwater Data
- Table C-4a – Summary of Groundwater Monitoring Data – Interchange
- Table C-4b – Summary of Groundwater Monitoring Data - Mainline

3.1 HISTORIC BORING DATA

In the vicinity of the project are the following projects which contained Subsurface Exploration Data useful to this project:

- Churchman's Road Bridge over I-95
- Churchman's Road & SR 7 Interchange
- Route 7 Re-Alignment and Fly-over Bridge No. 223
- Route 7 over Eagle Run
- Road A over SR 7

The historic boring data is contained in Appendix B of this report.

3.2 INITIAL SUBSURFACE EXPLORATION

The initial subsurface exploration program conducted by RK&K was for various Delaware Improvements Projects in New Castle County, Delaware. The initial subsurface exploration program began in August 2004 and was completed in April 2006. A total of approximately 615 SPT borings, in situ testing (DMT and CPT) probes, and test pits were conducted for the all of the Delaware Turnpike Improvements Projects. These exploration efforts have been summarized in four separate Geotechnical Data Reports (GDR's):

- Report No. 1: Mainline Improvements
- Report No. 2: I-95/SR 1 Interchange GDR with Supplemental Laboratory Test Data
- Report No. 3: Toll Plaza
- Report No. 4: Northbound Widening

An electronic PDF copy, on a CD, of the GDR's indicated above are contained in Appendix B of this report. The GDR's contain a summary of the subsurface exploration, a boring location plan, test boring logs, and results of the laboratory testing program.

3.2.1 Initial Standard Penetration Test Boring Exploration

The initial subsurface exploration for the SR 1/I-95 Interchange and the fifth lane widening projects (summarized in GDR Reports No. 2 and 4) consisted of drilling 206 SPT borings, 27 CPT, and 23 DMT probes with various drill rigs between October 11, 2004 and March 4, 2005. Eleven test pits in existing stock pile areas were also excavated as part of this assignment.

The initial soil borings were drilled by The Robert B. Balter Company (RBB) of Owings Mills, Maryland and their subconsultants under the full-time supervision of RK&K. Boring locations are shown in Figures A-2a through A-2q.

All fieldwork was performed in accordance with contract specifications entitled "General and Technical Specifications, Test Borings, In-situ Testing and Laboratory Testing for Rummel, Klepper & Kahl, March 2003". Test boring locations were staked in the field by Karins and Associates.

The test borings were drilled using both truck and ATV drilling equipment. The type of drill rig is indicated on the test boring log. Soil borings were advanced using hollow stem augers or casing as recorded on the test boring logs.

Soil samples were obtained at a maximum 5-foot interval in accordance with the SPT method. In general, the SPT consists of advancing a 2-in outside diameter sampling spoon 18-inches by driving it with a 140-pound hammer falling 30-inches. The values reported on the boring logs are the blows required to advance the sampler three successive increments. The first 6-in increment is considered as seating. The sum of the number of blows for the second and third increments is the "N" value, which is an index of soil strength. Both safety and automatic hammers were used during this subsurface exploration phase.

Relatively undisturbed soil samples were obtained using Shelby tubes, 3-in diameter thin-walled steel tube samplers. These tubes were hydraulically pressed into fine-grained soils to retrieve an undisturbed soil sample for soil strength and consolidation testing.

The split-barrel samples are stored in jars at the storage container at DelDOT's Northern District Maintenance Yard located off East Regal Boulevard in Newark, DE. The thin-wall tube soil samples were delivered to RBB for testing.

The soils were classified in general accordance with the Unified Soil Classification System (USCS) and the American Association of State Highway and Transportation Officials (AASHTO) classification system. RK&K field personnel recorded the classifications, observations, water and cave-in depths, and field sampling information on the Test Boring Logs. The results of this work are contained in Appendix B. Recovered soil samples were reviewed by geotechnical engineers and geologists. Descriptions of the soil classification system and sample procedures are also included in Appendix B.

3.2.2 Cone Penetration Test

The CPT probes were advanced with a truck or ATV-mounted CPT rig. The CPT consists of pushing a series of cylindrical rods with a cone at the base into the soil at a constant rate of 2.0-cm/sec. Continuous measurement of penetration resistance on the cone tip (Qc) and friction on a friction sleeve (Fs) were recorded during the penetration. Correlations have been developed by several authors to estimate the soil types, friction angle, undrained shear strength, stress history, modulus, and SPT N-value from the measured data. The parameters need to be correlated using laboratory testing to be most effective in determining soil parameters.

CPT tests were performed by In-situ Soil Testing, Inc. The results of the CPT probes are contained within Appendix B of this report.

3.2.3 Flat-Plate Dilatometer

The Flat-Plate DMT probes were advanced with a truck or ATV-mounted DMT rig. The DMT consists of pushing a flat blade located at the end of a series of drill rods to a desired test depth. Once the desired test depth was reached, gas pressure was used to expand a circular steel membrane horizontally into the soil. Three pressures are recorded. Pressure A is the pressure on the blade before expansion and Pressure B is the pressure required to produce an expansion of one millimeter of the membrane into the soil. The membrane is deflated and a third pressure is recorded, Pressure C. After the three pressures are recorded, the probes are pushed to the next desired test depth. The thrust required to push the blade was measured using a load cell.

The DMT test results can be used to estimate a wide range of soil properties. The properties of primary concern for our study are the soil classification, the stress history and the undrained shear strength, Su of granular soils. Other properties that the DMT can be used to estimate include: coefficient of lateral earth pressure at rest (K_o), drained plane strain friction angle (ϕ'_{ps}), preconsolidation pressure (σ_{pc}), dilatometer modulus (E_D), and tangent modulus (M). The parameters need to be correlated using laboratory testing to be most effective in determining soil parameters.

DMT tests were performed by In-situ Soil Testing, Inc. The results of the DMT probes are contained within Appendix B of this report.

3.3 INITIAL LABORATORY TESTING PROGRAM

The initial laboratory testing program consisted of determining the natural moisture content, the grain-size distribution, and the Atterberg limits of soil samples recovered from the split barrel samples and undisturbed tube samples. The shear strength parameters were determined on tube samples using the Direct Shear (DS), Unconfined Compressive (UCC), CIUC tests with pore pressure, and Unconsolidated-Undrained (UU) Triaxial compressive test methods. Consolidation testing was also performed on selected soil samples.

The results of the testing are presented on a consolidated graph in Figures C-2 and C-3. This presentation aided in determining shear strength parameters for each Stratum.

Laboratory testing for bulk bag samples included determining the standard moisture-density relationship and California Bearing Ratio (CBR).

The initial laboratory testing program was conducted by RBB, an AMRL accredited laboratory. Results of the laboratory testing are summarized in Appendix B. The natural moisture content and Atterberg limits are shown on the Test Boring Logs contained in Appendix B. Grain-size distribution graphs and the results of the laboratory testing are included in Appendix B.

3.4 GROUNDWATER MONITORING WELLS

To more accurately determine the hydrostatic water, piezometers were installed throughout the project area and were monitored from November 2004 to April 2005 and again in August and January 2009. The actual level of the hydrostatic water table and the amount and level of perched water should be anticipated to fluctuate throughout the year, depending upon variations in precipitation, surface run-off, infiltration, site topography, and drainage.

Piezometers were installed in or adjacent to 40 borings of this drilling assignment to obtain long-term water level readings. Monitoring Well Construction Logs are contained in Appendix B of the Geotechnical Data Reports.

A summary of the groundwater well readings to date are contained in Appendix C, Tables C-4a and C-4b for the Interchange and Mainline wells, respectively. A total of three groundwater monitoring wells are not accessible due to recent repaving activities in this area.

3.5 SUPPLEMENTAL SUBSURFACE EXPLORATION AND LABORATORY TESTING PROGRAM

To develop the supplemental subsurface exploration program, the existing SPT borings and in situ tests previously conducted by RK&K for the SR 1/I-95 Interchange and along Mainline I-95 fifth lane widening in 2005 and 2006 was compared with the latest interchange alignment. The locations of existing subsurface data with respect to proposed bridge foundations, retaining walls, stormwater management, and roadway was evaluated by both RK&K and URS. All borings or CPT/DMT probes within 25-ft of a structural element were considered to be acceptable for use with the new horizontal alignment. However, in some cases it was not safe to drill additional borings at the location of the structures; therefore, they were offset as described below.

The need for the supplemental borings was determined based on the anticipated locations of retaining walls and bridge foundations. Access to the additional borings was also evaluated, specifically with respect to maintenance of traffic (MOT) needs. Borings that would require anything other than a shoulder or one lane closure were offset to avoid disruptions to traffic.

The locations of the supplemental borings for this project are indicated in Figures A-2a through A-2q in Appendix A of this report.

The following is a summary of the supplemental subsurface exploration and associated laboratory testing that were conducted for the design of this project. The results of the supplemental subsurface exploration and associated laboratory testing are contained in Appendix C of this report.

- **Mall Access Bridge over SR 7 (Bridge R1).** Six borings were required for this structure because the location of the bridge has been shifted approximately 125-ft south of the previously drilled location.
- **Ramp B Bridge over I-95 and Associated Retaining Walls.** These structures were not part of the original plan and therefore no drilling was completed during the previous subsurface exploration. A total of eleven borings were drilled for these structures.
- **Additional Retaining Wall Borings.** Five additional borings were drilled for retaining walls 10, 13, and 16.
- **Two Stormwater Management sites.** A total of four borings with in situ infiltration testing were conducted for two SWM facilities.
- **Ramp A, B, C, and G1 Embankment.** Two new embankment borings were drilled in the vicinity of the Ramp A, B, C, and G1 embankment. These borings provided additional information for the construction of the proposed approximately 35-ft high embankment.
- **Bridges.** Five additional borings are required for the URS bridge and wingwall structures. The purpose of these borings was to further evaluate the subsurface data in the area of the bridge foundations and to obtain geotechnical data from a depth of 80 to 125-ft below the existing ground surface.

3.5.1 Supplemental Subsurface Exploration

The supplemental borings were drilled by the Walton Corporation of Newark, Delaware under contract to DelDOT. The supplemental subsurface exploration was completed in two phases. The first phase was started on July 23, 2008 and was completed on August 18, 2008. The work conducted during this time period was predominately within the SR 1 interchange. Additional supplemental borings were drilled along I-95 northbound from October 20, 2008 through October 29, 2008. RK&K and URS provided full-time supervision of the supplemental field work. The supplemental boring locations are shown in Figures A-2a through A-2q.

The test borings were drilled using both truck and ATV drilling equipment. The type of drill rig is indicated on the test boring log. Soil borings were advanced using hollow stem augers, mud rotary, or casing as recorded on the test boring logs. A safety hammer was used to drive the spilt spoon barrel for the supplemental borings.

Soil samples were obtained at a maximum 5-foot interval in accordance with the SPT method. In general, the SPT consists of advancing a 2-in outside diameter sampling spoon 18-inches by driving it with a 140-pound hammer falling 30-inches. The values reported on the boring logs are the blows required to advance three or four successive increments. The first 6-in increment is considered as seating. The sum of the number of blows for the second and third increments is the "N" value, which is a relative indicator of soil strength.

Relatively undisturbed samples of fine-grained soils were obtained using either a thin-walled tube sampler or a double/triple core barrel sampler known as a Denison sampler. These tubes were hydraulically pressed into fine-grained soils to retrieve an undisturbed soil sample for soil strength and consolidation testing.

Bulk samples from auger cuttings were also obtained from the supplemental URS borings at various depths.

The soils were classified in general accordance with the Unified Soil Classification System (USCS) and the American Association of State Highway and Transportation Officials (AASHTO) classification system. The DelDOT AASHTO graphical symbols are shown on the Summary of Boring Data, all Figures A-3 and A-4. RK&K and URS field personnel recorded the classifications, observations, water and cave-in depths, and field sampling information on the Test Boring Logs.

3.5.2 Supplemental Laboratory Testing

The supplemental laboratory testing program consisted of determining the natural moisture content, the grain-size distribution, and the Atterberg limits of soil samples recovered from the split barrel samples and undisturbed tube samples. The shear strength parameters were determined on tube samples using the Direct Shear (DS), Unconfined Compressive (UCC), and Unconsolidated-Undrained (UU) Triaxial compressive test methods. Consolidation testing was also performed on selected soil samples. In addition, some undisturbed samples were also tested to determine the in situ unit weight and specific gravity.

The supplemental laboratory testing program was conducted by DelDOT, Materials and Research Division. Results of the laboratory testing are summarized in Appendix C. The

Natural Moisture Content results are shown on the Test Boring Logs contained in Appendix C. Grain-size distribution graphs and the results of the laboratory testing are included in Appendix C.

Four bulk samples were sent to Sailors Engineering Associates, Inc. to estimate the coefficient of friction on uncoated steel plates and steel plates coated with Slickcoat. The coefficient of friction for uncoated plates ranged from 0.52 to 0.57, averaging 0.55. For coated plates, the coefficient of friction ranged from 0.32 to 0.40, averaging 0.37. The percent reduction for using Slickcoat compared to uncoated steel was approximately 33-percent. The results of the testing are provided in Appendix C of this report.

4 SUBSURFACE CONDITIONS

4.1 GEOLOGY

According to the *Geology of the Newark Area, Delaware, Geologic Map Series No. 3* and the *Geology of Wilmington Area, Delaware, Geologic Map Series No. 4,* the project site is located in the Atlantic Coastal Plain Physiographic Province. The SR 1/I-95 Interchange is located about two miles south of the fall line, where the overlying sediments of the Atlantic Coastal Plain to the south or southeast intersect the crystalline rocks and weathered in-place residuum of the Piedmont Province.

Based upon our review of the geologic literature, the base of sedimentation/top-of-bedrock at this location is expected to be over 200 feet deep. The formations mapped within the project site are the Potomac Formation unconformably overlain by the Columbia Formation, except where the Columbia Formation has eroded away. Each formation is described in further detail below.

Due to extensive road building and commercial development, significant amounts of fill materials are likely to exist on the project site. Although significant amounts of recent alluvial and marsh deposits exist to the east nearby, the higher topography within this project site limits the amount of these sediments within the project site.

4.1.1 Columbia Formation – Pleistocene Epoch

The locally discontinuous alluvial deposits of the Columbia Formation are believed to have been deposited during the Pleistocene. Where present, these sediments predominantly consist of variably loose to medium dense silty and poorly graded sands, but may also include clayey sands or firm to stiff low plasticity clays that would be expected to possess less desirable characteristics with respect to both the potential magnitude of settlement and the time duration for settlement to occur under the influence of embankment loads.

Within the project site, this formation is mapped within the existing SR 1 Interchange access ramps from I-95 and along I-95. The thickness of this formation is mapped as less than 5-ft near the contact of the eastern portion of the SR 1 interchange with I-95 to about 20-ft west of the Churchmans Road Bridge.

4.1.2 Potomac Formation - Cretaceous Period

The deposits locally referred to as the Potomac Formation are exposed on the south side of the interchange, but buried beneath the Columbia Formation on the northern portion of the project site.

The Potomac Formation sediments in northern Delaware are believed to have been deposited in a vast alluvial plain by an interconnected network of rivers during the Cretaceous. The formation is primarily composed of fine-grained materials in over-bank interfluvial facies, with laterally discontinuous fluvial sands forming a three-dimensional labyrinth in the flood plain muds.

The Potomac Formation has been subjected to high levels of preconsolidation imparted by the weight of younger deposits that have since been eroded away. Characterizing the physical properties of the formation is complicated by the interfluvial mode of deposition, the erratic presence of discontinuous channel and overbank sands, and degradation of the silt and clay properties by weathering processes, which could extend to variable depths.

4.2 GENERALIZED SUBSURFACE CONDITIONS

The Summary of Boring Data and Test Boring Logs contained in Appendices A, and B and C, respectively, provide details related to the subsurface conditions encountered in the various borings. The stratification lines shown on the Test Boring Logs and Summary of Boring Data represent approximate transitions between material types. In situ, strata changes could occur gradually or at slightly different levels. Also, the borings depict conditions at particular locations and at the particular times indicated. Some conditions, particularly groundwater conditions between borings could vary from the conditions encountered at the particular boring locations.

In general, the subsurface materials encountered within the project limits have been broken into four strata as defined below for this report:

- **FILL**
- **Stratum I:** Columbia Formation – Coarse Grained
- **Stratum IIa:** Potomac Formation – Fine Grained
- **Stratum IIb:** Potomac Formation – Coarse Grained and Non Plastic Fine Grained

Generalized soil parameters are summarized by stratum in Section 4.3 of this report. More detailed descriptions of the subsurface conditions at each structure location are in Section 4.4 of this report.

FILL: FILL depths extended as deep as 38.5-ft below the existing ground surface with an average depth below the existing ground surface of approximately 7-ft. The FILL material encountered within the project site consisted of stiff to very stiff Sandy and Silty CLAY (USCS: CL, CL-ML) [AASHTO: A-4, A-6] as well as very loose to dense Silty and Clayey SAND (USCS: SM, SC) [AASHTO: A-2-4]. Varying amounts of organic material and generally trace amounts of both asphalt fragments and angular gravel fragments were also encountered in this Stratum.

The SPT N-values generally ranged from approximately 4 blows per foot (bpf) to 56-bpf, with an average SPT N-value of 17-bpf. The presence of gravel and debris in the FILL Stratum may have exaggerated some SPT N-values. The natural moisture content for this Stratum ranged from 6.3 to 22.6-percent and averaged 14.1-percent. The liquid limit ranged in value from Non-Plastic (NP) to 32 with an average value of 26. The plasticity index ranged from NP to 13 and averaged 9. The average percentage fines (percent passing the No. 200 sieve) for the FILL material was 55-percent, with a maximum of 97-percent and a minimum of 22-percent.

Stratum I - Columbia Formation - Coarse Grained Soil: Stratum I encountered within the project limits generally consisted of very loose to medium dense SAND with varying percentages of silt and clay (USCS: SC, SM, SC-SM, SW-SM) [AASHTO: A-4, A-2-4, A-1-b, A-6]. Trace amounts of organic material, gravel, and gravel-sized rock fragments were also encountered in some borings. Discontinuous lenses of silt and clay (USCS: ML, CL) [AASHTO: A-4 and A-6] were found sporadically throughout the Stratum.

This Stratum was generally encountered below the FILL or the existing ground surface. The majority of this Stratum was encountered within the borings along I-95 and within the existing SR 1 interchange.

The SPT N-values for this Stratum typically ranged from 4 to 60-bpf, averaging approximately 16-bpf. The natural moisture content for this Stratum averaged 10.8-percent and ranged from 5.2 to 19.1-percent. The liquid limit ranged from NP to 28 and the plasticity index ranged from NP to 11. The average percentage of fines (percent passing the No. 200 sieve) for Stratum I was 23-percent, with a maximum of 44-percent and a minimum of 11-percent.

Based on the CPT and DMT probes, the angle of friction for this Stratum ranged from 39 to 50-degrees, with an average angle of friction near 45-degrees. The over consolidation ratio (OCR) for this Stratum from the CPT generally was between 1 (normally consolidated) to 1.5.

The average angle of friction based on the average SPT N-value of 16-bpf was about 32-degrees using the Meyerhof equation where:

$$\phi = 27 + \frac{10N}{35}$$

$\phi =$ Angle of Friction
N = SPT N-value
Reference: *Foundation Engineering*, Peck, Hanson, and Thornburn (1974), Figure 19.5

Stratum IIa – Cretaceous - Fine Grained Soil: Stratum IIa within the limits of the project site generally consisted of medium stiff to hard CLAY and SILT with varying percentages of sand and trace to little amounts of lignite and mica (USCS: CL, CL-ML, CH, MH, OL, ML) [AASHTO: A-4, A-6, A-7-5, A-7-6, A-8]. This stratum was interbedded with Stratum IIb and was generally encountered below the existing FILL or Stratum I.

The Stratum IIa SPT N-values typically ranged from 8 to 50-bpf, averaging 21-bpf. The natural moisture content averaged 43-percent and ranged from 34 to 58-percent. The liquid limit ranged from 18 to 67, averaging 36. The plasticity index ranged from 3 to 39, averaging 16. The average percentage of fines (percent passing the No. 200 sieve) ranged from 32 to 100-percent, averaging 84-percent.

Based on the CPT probes, the undrained shear strength ranged from 300-psf to 8,900-psf, averaging 3,100-psf. The OCR values from the CPT probes ranged from normally consolidated to 33, and were typically near 4.

The undrained shear strength results from the unconsolidated undrained (UU) Triaxial testing ranged from 700 to 8,230-psf, averaging 2,970-psf. The undrained shear strength tended to increase with depth as shown in Figure C-1. The undrained shear strength test results from the unconfined compression (UCC) testing averaged 2,485-psf and ranged from 465 to 5,620-psf.

The undrained shear strength estimated from average SPT N-value was about 2.7-ksf using the following equation:

$$s_u = \frac{N(1000)}{7.5}$$

s_u = Undrained Shear Strength (psf)
N = SPT N value
Reference: Terzaghi and Peck (1967), Carter and Bently (1991), and DM 7.1

The results of the CU Triaxial testing with pore pressure for an effective stress state were an angle of friction of 24-degrees and cohesion of 379-psf. For a 95-percent confidence effective stress the angle of friction ranges from 19.8 to 29.0-degrees. The effective stress CU tests with a 95-percent confidence interval are summarized in Figures C-3a and C-3b in Appendix C of this report.

The drained angle of friction from the CIUC and direct shear testing for stratum IIa is summarized vs the liquid limit and the plasticity index in Figures C-5a and C-5b, respectively. The upper and lower bounds of the Mitchell line are depicted in these figures.

The direct shear test results from the undisturbed Shelby tubes indicated an average drained angle of friction of 14-deg and a cohesion of 700-psf. For a 95-percent confidence the drained angle of friction ranges from 13 to 17-degrees and the cohesion ranges from 550 to 850-psf.

The pH of the soil ranged from 4.11 to 6.19. The average pH of this stratum is 5.0. The resistivity averaged 4,675 ohm-cm and ranged from 1,900 to 15,000 ohm-cm.

P_c and OCR for this stratum were determined from the laboratory testing using the traditional Casagrande method, Work Energy Method, and verified using the CPT and DMT results. The P_c ranged from 1 to 8.5-tsf and averaged 4-tsf. The average OCR was about 3 and ranged from 1 to 5. Figure C-2 shows the stress history of Stratum IIa. Supporting calculations for development of the generalized consolidation parameters are provided in Appendix F.

Stratum IIb – Cretaceous Coarse Grained Soil: Stratum IIb within the project limits generally consisted of medium dense to very dense, Silty and/or Clayey, poor to well graded SAND with trace lignite and mica (USCS: SP-SM, SC-SM, SM, SC, SW-SM) [AASHTO: A-3, A-4, A-2-4, A-2-6, A-1-a, A-1-b]. Interbedded thin lenses of Stratum IIb are also classified as a very stiff SILT with varying percentages of Sand (USCS: ML) [AASHTO: A-4].

Typical SPT N-values for this stratum is 32-bpf using the Meyerhof equation this results in an angle of friction of 37-degrees. The natural moisture content averaged 16.5-percent and ranged from 4.1 to 29-percent. The liquid limit ranged from NP to 33 and the plasticity index ranged from NP to 13. The average percentage of fines (percent passing the No. 200 sieve) for Stratum IIb ranged from 8 to 81-percent, averaging 31-percent.

Based on the CPT and DMT soundings, the angle of friction for this stratum ranged from 33 to 50-degrees, with an average angle of friction near 40-degrees. The over consolidation ratio

(OCR) for this stratum from the CPT generally was between 1 (normally consolidated) to 16, averaging 3.

4.3 GENERALIZED SOIL PARAMETERS FOR DESIGN

Table 4.1 summarizes the general soil parameters which were developed for preliminary evaluations of the proposed foundations. Structure specific soil parameters were also evaluated and are summarized, as needed, in Sections 4.4.1 through 4.4.8 of this report. The following figures summarize the laboratory test results and were used to develop the generalized soil parameters. The laboratory test results and the test boring logs from the supplemental program are contained in Appendix C of this report.

- Figure C-1 summarizes the undrained shear strength testing for Stratum IIa
- Figure C-2 summarizes the stress history of Stratum IIa
- Figures C-3a and C-3b summarize the Stratum IIa CU testing with pore pressure for the effective stress state.

Table 4.1 – Generalized Soil Parameters

Strata	Total Unit Weight – γ (pcf)	Drained		Undrained
		Angle of Friction – ϕ (deg)	Cohesion – c (psf)	Shear - S_u (psf)
FILL	125	28	-	-
I	125	30	-	-
IIa – Above EL 40	125	19 to 29	0 to 700	1,500
IIa - EL 40 to 20	125	19 to 29	0 to 700	2,500
IIa - EL 20 to 0	125	19 to 29	0 to 700	3,500
IIa - Below EL 0	125	19 to 29	0 to 700	5,000
IIb – Below EL -5	125	34	-	-

The shear strength parameters were developed from CPT and DMT probes, Consolidated Undrained (CU) with pore pressure, and UU Traxial Tests, and verified using SPT N-value correlations and laboratory tests as well as engineering judgment. A summary of the classification testing for select SPT soil samples is summarized in Table C-2a in Appendix C of this report. Table C-2b in Appendix C summarizes the undisturbed laboratory test results for this project. Table C-2c in Appendix C summarizes the consolidation test results.

Pc and OCR were determined from the laboratory testing, DMT, and CPT soundings. The OCR was observed to be relatively high for the soils in this area as the soils are highly overconsolidated. We assumed that the sands were similarly pre-loaded.

It appears as though several of the consolidation tests underestimate the OCR. Many of these test results have a relatively flat consolidation curve. According to AASHTO LRFD Figure C10.6.2.4.3-1, a flat laboratory curve is an indication of a poor quality sample. The in-situ preconsolidation stress is likely higher than the value measured through the laboratory testing. The Schmertmann 1955 method can be used to plot the field (in-situ) curve based on the laboratory data. Studies indicate that this method produces a higher preconsolidation stress than determined through laboratory testing.

The unit weights typically ranged from 120 to 130-pcf. The analyses conducted for this project were not very sensitive to unit weight so 125-pcf was used as an average. Where the groundwater level was one footing width (strap length) below the leveling pad, the wet unit weight of water was used in our analysis of external stability in MSEW. In cases were groundwater is within one footing width of the leveling pad elevation, the following equation was used to adjust the unit weight:

$$\bar{\gamma} = \gamma' + \frac{d}{B}(\gamma - \gamma')$$

Where:

$\bar{\gamma}$ = Modified Unit Weight due to location of groundwater to foundation elevation
d = Depth to Groundwater below foundation elevation
B = Footing width
γ = Unit Weight
γ' = Saturated unit weight minus the unit weight of water
Reference: Das. 2005, pg 386

The Elastic modulus (Es) was estimated from the CPT and DMT probes, SPT N-values correlations, laboratory test results, the results of the settlement plates from the Ramp A, B, C, and G1 embankment, and engineering judgment.

Back Calculation Using Settlement Plate Results – Method I (RK&K)

The settlement plate readings from the existing embankment were used to back calculate the soil parameters for settlement calculations for the proposed retaining walls and the bridge abutments. The settlement plate reading plots are included in Appendix E of this report.

Geotechnical software FoSSA and finite element software Plaxis 8.0 were used to calculate the settlement below the embankment.

The initial settlement calculation was performed for the first phase of the embankment construction built to a height of 10-ft. During the first phase of the construction an average of 1.2-inches of total settlement was observed below the embankment at the location of the settlement plates. Settlement calculation was performed assuming coarse grained soil layers experienced only elastic settlement and fine grained soil layers experienced only consolidation settlement. The soil parameters were optimized to match the calculated settlement under the embankment with the settlement plate readings. Once the optimized parameters were developed they were compared to the laboratory test results, correlations with index testing and published values.

The Elastic Modulus (Es) of the coarse grained soil layers were optimized maintaining the same ratios between strata as calculated using the Cone Penetration Test (CPT) sounding in the vicinity of the embankment. The optimized Elastic Moduli are within acceptable range of published values (Bowles 1982, *Foundation Analysis and Design*). Table 4.2a summarizes the Elastic Modulus of the soil layer developed from CPT sounding and the optimized Elastic Modulus.

Table 4.2a – Elastic Modulus from Phase 1 Embankment Construction

Soil Stratum	Elevation	CPT Result, Es (ksf)	Optimized, Es (ksf)
Stratum 1	El 70 and above	500	1500
Stratum 2a	EL 60 - EL 70	300	818
Stratum 2a	EL 50 - EL 60	200	545
Stratum 2a	EL 20 - EL 50	300	818
Stratum 2a	EL 20 and Below	500	1364
Stratum 2b	Interbedded	500	1364

The soil strata at the embankment site are over-consolidated with the Over Consolidation Ratio (OCR) ranging between 2.1 and 5.3 with a typical value near 3.0 but decreasing with depth. The increase in stress due to the embankment is less than the preconsolidation pressure of the soil

strata; therefore, the consolidation settlement below the embankment is highly influenced by the recompression index (C_r) of the soil. The C_r value was optimized to match the total settlement with the settlement plate results. Typical values of C_r range from 0.015 – 0.35 (Roscoe et al. 1958) and are often assumed to be 5 to 10-percent of C_c. Table 4.2b summarizes the consolidation soil parameters from laboratory test results and the optimized recompression index values. The optimized C_r values are slightly lower than the laboratory test results but are still reasonable.

Table 4.2b – Back Calculated Consolidation Soil Parameters

Soil Stratum	Elevation	OCR	e_o	Consolidation Test Results		Optimized
				C_c	C_r	C_r
Stratum 2a	EL 60 - EL 70	5.3	0.655	0.092	0.020	0.015
Stratum 2a	EL 50 - EL 60	5.3	0.655	0.092	0.020	0.015
Stratum 2a	EL 20 - EL 50	2.3	0.605	0.105	0.025	0.015
Stratum 2a	EL 20 and Below	2.3	0.605	0.105	0.025	0.015

Table 4.3 summarizes the total settlement below the embankment at the location of the settlement plates. The settlement estimates are slightly higher than the actual measured settlements, but using the optimized values should result in reasonable settlement estimates.

Table 4.3 – Calculated and Actual Settlement (Phase I)

Analysis Type	Analysis	Soil Parameters	Calculated Settlement (in)	Actual Settlement (in)
Infinite Embankment	FoSSA Analysis	Optimized Soil Parameters to Match Field Settlement Plate Data	1.3	1.2
Infinite Embankment	Plaxis Analysis	Optimized Soil Parameters to Match Field Settlement Plate Data	1.5	1.2

It was brought to our attention that the survey monitoring points were hit multiple times by the construction equipment during the placement of the embankment FILL. Because of this and other possible errors in reading the settlement plates, the total settlement for the second phase construction assumes no settlement occurred during the placement of new fill with each extension of the riser pipes.

The settlement plate data indicates that most of the settlement occurred immediately after the construction of the embankment. During approximately 2-months between the completion of first phase construction and beginning of second phase construction when no fill was added, the settlement plates did not show any significant settlement. Similarly, the settlement plate readings from about 2-weeks after the completion of the full height embankment to date show minimal or no settlement of the embankment.

Back Calculation Using Settlement Plate Results – Method II (URS)

The settlement plate readings from the test embankment and existing consolidation tests were used to back calculate the thickness of the fine-grained stratum consolidating under the test embankment load. The settlement plate reading plots are included in Appendix E of this report. A generalized soil profile was established based on the local borings, ISC-74, ISC-118, IEB-13, IEB13A, and ISC-103A, which included 18-feet of sand overlying fine-grained soil. Elastic properties of the sand were estimated using correlations with SPT values and consolidation parameters of the fine-grained soil were estimated using laboratory consolidation testing of material located directly beneath the test embankment or in close proximity. Elastic and consolidation settlement was estimated for coarse grained and fine grained soils, respectively. The thickness of the fine-grained stratum was increased incrementally until the estimated settlement approximated the experienced settlement of the test embankment.

Using this method total settlement of 2.5-inches was estimated assuming consolidation of 20-feet of the fine-grained layer. The estimated settlement of 2.5-inches compares favorably with the range of 1.5 to 3.5-inches established at the settlement plate locations. Additionally, SPT values generally increase at depths in excess of 20-feet.

Supporting calculations are provided in Appendix F.

4.4 SUBSURFACE CONDITIONS – LOCATION SPECIFIC

The subsurface conditions at each bridge location are summarized in the following Sections of this report.

- Structure S1: Ramp A over I-95 (Section 4.4.1) - URS
- Structure S2: Ramp A over SR 7 (Section 4.4.2) - URS
- Structure S3: Ramp B over SR 7 (Section 4.4.3) - URS
- Structure S4: Ramp C over SR 7 (Section 4.4.4) - URS
- Structure S5: Ramp G1 over SR 7 (Section 4.4.5) - URS
- Structure S6: Ramp B over Northbound I-95 (Section 4.4.6) – RK&K
- Structure S7: Ramp R1 over SR 7 (Section 4.4.7) – RK&K
- Structure S8: SR 1 over Eagle Run Widening (Section 4.4.8) – RK&K

4.4.1 Structure S1: Ramp A over I-95

Table 4.4 summarizes the borings that were used for the evaluation of subsurface conditions for the Ramp A over I-95 structure. The Summary of Subsurface Data for this structure is contained in Figure A-3a, located in Appendix A of this report.

Table 4.4 - Subsurface Exploration: Structure S1 - Ramp A over I-95

Structure	SPT Borings	CPT Probes	DMT Probes
Abutment A	URS-3	-	IDMT-21
Pier 1	IRW-68, IBR-24	-	-
Pier 2	IBR-28, IBR-28A	-	-
Pier 3	IBR-26A, IBR-27A	-	-
Abutment B	IBR-26, IBR-27	-	-

FILL: FILL extended to a depth of about 28-ft below the existing ground surface. The FILL material encountered within the limits of the proposed Ramp A over I-95 consisted of stiff to very stiff Sandy and Silty CLAY (USCS: CL, CL-ML) [AASHTO: A-4, A-6], Sandy and Clayey SILT (USCS: ML) [AASHTO: A-4], and very loose to dense SAND with varying percentages of silt and clay (USCS: SM, SP-SM) [AASHTO: A-2-4, A-2]. Varying amounts of organic material and generally trace amounts of gravel were also encountered in this stratum.

The SPT N-values generally ranged from approximately 4 to 30-bpf, with an average SPT N-value of 14-bpf.

Stratum I - Columbia Formation - Coarse Grained Soil: Stratum I generally consisted of very loose to dense, SAND with varying percentages of silt and clay (USCS: SC, SM, SC-SM, SP-

SM) [AASHTO: A-2-4, A-2-6]. Discontinuous lenses of silt and clay (USCS: ML, CL) [AASHTO: A-4, A-6] were found sporadically throughout the stratum.

The SPT N-values generally ranged from approximately Weight of Rods (W.O.R.) to 27-bpf, with an average SPT N-value of 11-bpf. The natural moisture content averaged 17.5-percent and ranged from 11.5 to 21.2-percent. The liquid limit ranged from non-plastic to 45, averaging 34 where existent. The plasticity index ranged from non-plastic to 25, averaging 14 where existent. The average percentage of fines (percent passing the No. 200 sieve) ranged from 11 to 92-percent, averaging 54-percent.

Stratum IIa – Cretaceous - Fine Grained Soil: Stratum IIa generally consisted of medium stiff to very hard CLAY and SILT with varying percentages of sand and trace to little amounts of lignite and mica (USCS: CL, CL-ML, CH, MH) [AASHTO: A-4, A-6, A-7-5]. Interbedded thin lenses of Stratum IIb are also classified as silty SAND (USCS: SM) [AASHTO: A-2-4].

The Stratum IIa SPT N-values typically ranged from 6-bpf to 100+ bpf, averaging 28-bpf. The natural moisture content averaged 19.6-percent and ranged from 13.5 to 35.7-percent. The liquid limit ranged from 22 to 61, averaging 35. The plasticity index ranged from 4 to 32, averaging 16. The average percentage of fines (percent passing the No. 200 sieve) ranged from 49 to 100-percent, averaging 85-percent.

The undrained shear strength results from two unconsolidated undrained (UU) Triaxial tests were 3,040-psf and 3,930-psf, averaging 3,485-psf. One direct shear test result indicated an average drained angle of friction of 11-deg and a cohesion of 920-psf.

Stratum IIb – Cretaceous Coarse Grained Soil: Stratum IIb generally consisted of medium dense to very dense, Silty and Clayey SAND (USCS: SM, SC, SP-SM, SC-SM) [AASHTO: A-2-4, A-1-b, A-4].

The SPT N-values for this stratum averaged 44-bpf and typically ranged from 17 to 100+ bpf. The natural moisture content averaged 19.6-percent and ranged from 13.5 to 35.7-percent. The average percentage of fines (percent passing the No. 200 sieve) ranged from 11 to 45-percent, averaging 27-percent.

For Structure S1, site specific soil parameters were developed and used for design of the foundation elements. The Structure S1 soil parameters are summarized in Table 4.5 below. Table 4.6 summarizes the soil parameters used for the settlement analysis of this structure.

Table 4.5 – Structure S1 Soil Parameters				
Strata	Total Unit Weight – γ (pcf)	Drained		Undrained Shear - S_u (psf)
		Angle of Friction ϕ (deg)	Cohesion – c (psf)	
I	100 to 125	30 to 33	-	-
IIa – EL 85 to 30	120 to 125	24	-	1,000 to 2,500
IIa - EL 30 to 0	125 to 130	27	-	2,500 to 3,500
IIa - EL 0 to -20	130	29 to 30	-	3,500 to 6,000

Table 4.6 – Structure S1 Soil Parameters for Settlement						
Strata	Total Unit Weight – γ (pcf)	E (tsf)	C_c	C_r	e_o	P_c (tsf)
I	125	125 to 400	-	-	-	-
IIa – EL 85 to 40	125	0.244	0.244	0.051	0.614	3.8

E – Soil Modulus
C_c – Compression Index
C_r – Recompression Index
e_o – Initial Void Ratio
P_c – Preconsolidation Pressure

Groundwater for this structure is anticipated to be near EL 75, approximately 11 to 14-ft below the existing ground surface.

4.4.2 Structure S2: Ramp A over SR 7

Table 4.7 summarizes the borings that were used for the evaluation of subsurface conditions for the Ramp A over SR 7 structure. The Summary of Subsurface Data for this structure is contained in Figure A-3b, located in Appendix A of this report.

Table 4.7 - Subsurface Exploration: Structure S2 - Ramp A over SR 7			
Structure	SPT Borings	CPT Probes	DMT Probes
Abutment A	URS-4, IBR-19	-	-
Pier 1	IBR-23	-	-
Pier 2	IBR-16	-	-
Pier 3	IBR-10, IBR-12	-	-
Abutment B	IRW-63, URS-5	-	-

FILL: FILL extended to a depth of about 8-ft below the existing ground surface. The FILL material encountered within the limits of the proposed Ramp A over SR 7 consisted of soft Sandy CLAY (USCS: CL) [AASHTO: A-6] and very loose to medium dense SAND with varying percentages of silt and clay (USCS: SM, SC-SM) [AASHTO: A-2-4].

The SPT N-values generally ranged from approximately 2 to 14-bpf, with an average SPT N-value of 9-bpf.

Stratum I - Columbia Formation - Coarse Grained Soil: Stratum I generally consisted of loose to dense SAND with varying percentages of silt and clay (USCS: SC, SM) [AASHTO: A-2-4, A-2-6].

The SPT N-values generally ranged from approximately 10 to 29-bpf, with an average SPT N-value of 16-bpf. The one natural moisture content tested in the stratum was 15.5-percent. The one liquid limit tested in the stratum was 24 and the plasticity index was 8. The percentage of fines (percent passing the No. 200 sieve) for the tested sample was 44-percent.

Stratum IIa – Cretaceous - Fine Grained Soil: Stratum IIa generally consisted of soft to very hard CLAY and SILT with varying percentages of sand and trace to little amounts of lignite and mica (USCS: CL, CL-ML, ML, CH, MH) [AASHTO: A-4, A-6, A-7-5, A-7-6]. Interbedded thin lenses of Stratum IIb are also classified as silty and clayey SAND (USCS: SM, SC-SM) [AASHTO: A-2-4].

The Stratum IIa SPT N-values typically ranged from 4 to 83-bpf, averaging 44-bpf. The natural moisture content averaged 20.3-percent and ranged from 14.9 to 31.4-percent. The liquid limit ranged from 18 to 55, averaging 38. The plasticity index ranged from 4 to 30, averaging 17. The average percentage of fines (percent passing the No. 200 sieve) ranged from 42 to 99-percent, averaging 87-percent.

The undrained shear strength result from one unconsolidated undrained (UU) Triaxial test was 1,250-psf.

Stratum IIb – Cretaceous Coarse Grained Soil: Stratum IIb generally consisted of loose to very dense, SAND with varying percentages of silt and clay (USCS: SM, SC-SM, SP, SP-SM) [AASHTO: A-2-4, A-1-b, A-4].

The SPT N-values for this stratum averaged 35-bpf and typically ranged from 6 to 100+ bpf. The natural moisture content averaged 20.6-percent and ranged from 12.8 to 30.1-percent. The

average percentage of fines (percent passing the No. 200 sieve) ranged from 17 to 46-percent, averaging 33-percent.

For Structure S2, site specific soil parameters were developed and used for design of the foundation elements. The Structure S2 soil parameters are summarized in Table 4.8 below. Table 4.9 summarizes the soil parameters used for the settlement analysis of this structure.

Table 4.8 – Structure S2 Soil Parameters

Strata	Total Unit Weight – γ (pcf)	Drained		Undrained Shear - S_u (psf)
		Angle of Friction ϕ (deg)	Cohesion – c (psf)	
IIa – EL 70 to 50	120 to 125	24	-	1,250 to 2,000
IIa - EL 50 to -10	125	27	-	2,000 to 4,000
IIa - EL -10 to -20	125 to 130	29 to 30	-	4,000 to 7,000
IIb	115 to 130	31 to 38	-	-

Table 4.9 – Structure S2 Soil Parameters for Settlement

Strata	Total Unit Weight – γ (pcf)	E (tsf)	C_c	C_r	e_o	P_c (tsf)
IIa – EL 70 to 40	125	-	0.29 to 0.33	0.065 to 0.068	0.786 to 0.851	3.48 to 4.5

E – Soil Modulus
C_c – Compression Index
C_r – Recompression Index
e_o – Initial Void Ratio
P_c – Preconsolidation Pressure

Groundwater for this structure is anticipated to be near EL 30, approximately 30 to 45-ft below the existing ground surface.

4.4.3 Structure S3: Ramp B over SR 7

Table 4.10 summarizes the borings that were used for the evaluation of subsurface conditions for the Ramp B over SR 7 structure. The Summary of Subsurface Data for this structure is contained in Figure A-3c, located in Appendix A of this report.

Table 4.10 - Subsurface Exploration: Structure S3 - Ramp B over SR 7			
Structure	**SPT Borings**	**CPT Probes**	**DMT Probes**
Abutment A	IBR-09A	-	IDMT-16
Pier 1	IBR-09, URS-5	-	-
Abutment B	IBR-10, IBR-23	-	-

FILL: FILL extended to a depth of about 8-ft below the existing ground surface. The FILL material encountered within the limits of the proposed Ramp B over SR 7 consisted of soft Sandy CLAY (USCS: CL) [AASHTO: A-6], stiff to hard SILT (USCS: ML) [AASHTO: A-4], and very loose to medium dense SAND with varying percentages of silt and clay (USCS: SM, SC-SM) [AASHTO: A-2-4].

The SPT N-values generally ranged from approximately 2 to 34-bpf, with an average SPT N-value of 13-bpf.

Stratum I - Columbia Formation - Coarse Grained Soil: Stratum I was not encountered within the limits of the proposed Ramp B over SR 7.

Stratum IIa – Cretaceous - Fine Grained Soil: Stratum IIa generally consisted of soft to very hard CLAY and SILT with varying percentages of sand and trace to little amounts of lignite and mica (USCS: CL, CL-ML, ML, CH) [AASHTO: A-4, A-6, A-7-5, A-7-6]. Interbedded thin lenses of Stratum IIb are also classified as silty SAND (USCS: SM, SP) [AASHTO: A-2-4, A-4].

The Stratum IIa SPT N-values typically ranged from 4 to 70-bpf, averaging 22-bpf. The natural moisture content averaged 19.1-percent and ranged from 11.3 to 31.4-percent. The liquid limit ranged from 21 to 56, averaging 37. The plasticity index ranged from 5 to 30, averaging 17. The average percentage of fines (percent passing the No. 200 sieve) ranged from 67 to 99-percent, averaging 88-percent.

The undrained shear strength result from one unconsolidated undrained (UU) Triaxial test was 1,250-psf. The undrained shear strength test result from one unconfined compression (UCC) test was 2,757-psf. One direct shear test result indicated a drained angle of friction of 16-deg and a cohesion of 785-psf.

Stratum IIb – Cretaceous Coarse Grained Soil: Stratum IIb generally consisted of loose to very dense, SAND with varying percentages of silt and clay (USCS: SM, SP, SP-SM) [AASHTO: A-2-4, A-1-b, A-4]. Interbedded thin lenses of Stratum IIa are also classified as CLAY (USCS: CL) [AASHTO: A-6].

The SPT N-values for this stratum averaged 35-bpf and typically ranged from 6 to 91 bpf. The natural moisture content averaged 20.6-percent and ranged from 14.6 to 30.1-percent. The average percentage of fines (percent passing the No. 200 sieve) ranged from 11 to 46-percent, averaging 23-percent.

For Structure S3, site specific soil parameters were developed and used for design of the foundation elements. The Structure S3 soil parameters are summarized in Table 4.11 below. Table 4.12 summarizes the soil parameters used for the settlement analysis of this structure.

Table 4.11 – Structure S3 Soil Parameters

Strata	Total Unit Weight – γ (pcf)	Drained		Undrained Shear - S_u (psf)
		Angle of Friction ϕ (deg)	Cohesion – c (psf)	
IIa – EL 70 to 45	120 to 125	24	-	1,250 to 2,000
IIa - EL 45 to -5	125	27	-	2,000 to 4,000
IIa - EL -5 to -20	125 to 130	29 to 30	-	4,000 to 8,000
IIb	115 to 135	28 to 40	-	-

Table 4.12 – Structure S3 Soil Parameters for Settlement

Strata	Total Unit Weight – γ (pcf)	E (tsf)	C_c	C_r	e_o	P_c (tsf)
IIa – EL 70 to 40	125	-	0.29 to 0.33	0.065 to 0.068	0.786 to 0.851	3.48 to 4.5

E – Soil Modulus
C_c – Compression Index
C_r – Recompression Index
e_o – Initial Void Ratio
P_c – Preconsolidation Pressure

Groundwater for this structure is anticipated to be near EL 30, approximately 30 to 45-ft below the existing ground surface.

4.4.4 Structure S4: Ramp C over SR 7

Table 4.13 summarizes the borings that were used for the evaluation of subsurface conditions for the Ramp C over SR 7 structure. The Summary of Subsurface Data for this structure is contained in Figure A-3d, located in Appendix A of this report.

Table 4.13 - Subsurface Exploration: Structure S4 - Ramp C over SR 7

Structure	SPT Borings	CPT Probes	DMT Probes
Abutment A	URS-1	-	-
Pier 1	IBR-43, IBR-20	-	-
Abutment B	IBR-31	-	-

FILL: FILL extended to a depth of about 10-ft below the existing ground surface. The FILL material encountered within the limits of the proposed Ramp C over SR 7 consisted of loose to medium dense silty SAND (USCS: SM) [AASHTO: A-2-4] and loose silty GRAVEL (USCS: GM) [AASHTO: A-1-a].

The SPT N-values generally ranged from approximately 9 to 20-bpf, with an average SPT N-value of 11-bpf.

Stratum I - Columbia Formation - Coarse Grained Soil: Stratum I generally consisted of medium dense to dense SAND with varying percentages of silt (USCS: SM) [AASHTO: A-2-4] and very stiff SILT(USCS: ML) [AASHTO: A-4].

The SPT N-values generally ranged from approximately 13 to 29-bpf, with an average SPT N-value of 20-bpf. The natural moisture content averaged 15.5-percent and ranged from 12.1 to 20.2-percent. The liquid limit ranged from 21 to 31, averaging 24.2. The plasticity index ranged from 2.5 to 13, averaging 5. The average percentage of fines (percent passing the No. 200 sieve) ranged from 44 to 75-percent, averaging 54-percent.

Stratum IIa – Cretaceous - Fine Grained Soil: Stratum IIa generally consisted of medium stiff to very hard CLAY and SILT with varying percentages of sand and trace to little amounts of lignite and mica (USCS: CL, CL-ML, ML, CH) [AASHTO: A-4, A-6, A-7-6]. Interbedded thin lenses of Stratum IIb are also classified as silty and clayey SAND (USCS: SM, SP-SM) [AASHTO: A-2-4].

The Stratum IIa SPT N-values typically ranged from 5-bpf to 100+ bpf, averaging 21-bpf. The natural moisture content averaged 21.9-percent and ranged from 15.5 to 28.6-percent. The liquid limit ranged from 20 to 61, averaging 37. The plasticity index ranged from 10 to 34, averaging 17. The average percentage of fines (percent passing the No. 200 sieve) ranged from 38 to 100-percent, averaging 87-percent.

Stratum IIb – Cretaceous Coarse Grained Soil: Stratum IIb generally consisted of loose to very dense, SAND with varying percentages of silt and clay (USCS: SM, SC-SM, SP, SP-SM) [AASHTO: A-2-4, A-2-6, A-4].

The SPT N-values for this stratum averaged 51-bpf and typically ranged from 9 to 100+ bpf. The natural moisture content averaged 17.5-percent and ranged from 10.6 to 25.1-percent. The liquid limit ranged from non-plastic to 19.4, averaging 18 where existent. The plasticity index ranged from non-plastic to 4, averaging 3 where existent. The average percentage of fines (percent passing the No. 200 sieve) ranged from 15 to 62-percent, averaging 33-percent.

For Structure S4, site specific soil parameters were developed and used for design of the foundation elements. The Structure S4 soil parameters are summarized in Table 4.14 below. Table 4.15 summarizes the soil parameters used for the settlement analysis of this structure.

Table 4.14 – Structure S4 Soil Parameters

Strata	Total Unit Weight – γ (pcf)	Drained		Undrained Shear - S_u (psf)
		Angle of Friction ϕ (deg)	Cohesion – c (psf)	
IIa – EL 70 to 35	120 to 125	24	-	1,250 to 2,000
IIa - EL 35 to -10	125	27	-	2,000 to 4,000
IIa - EL -10 to -20	125 to 130	29 to 30	-	4,000 to 8,000
IIb	115 to 135	30 to 42	-	-

Table 4.15 – Structure S4 Soil Parameters for Settlement						
Strata	**Total Unit Weight – γ (pcf)**	**E (tsf)**	**C_c**	**C_r**	**e_o**	**P_c (tsf)**
IIa – EL 70 to 40	125	-	0.29 to 0.33	0.065 to 0.068	0.786 to 0.851	3.48 to 4.5
E – Soil Modulus C_c – Compression Index C_r – Recompression Index			e_o – Initial Void Ratio P_c – Preconsolidation Pressure			

Groundwater for this structure is anticipated to be near EL 30, approximately 30 to 45-ft below the existing ground surface.

4.4.5 Structure S5: Ramp G1 over SR 7

Table 4.16 summarizes the borings that were used for the evaluation of subsurface conditions for the Ramp G1 over SR 7 structure. The Summary of Subsurface Data for this structure is contained in Figure A-3e, located in Appendix A of this report.

Table 4.16 - Subsurface Exploration: Structure S5 - Ramp G1 over SR 7			
Structure	**SPT Borings**	**CPT Probes**	**DMT Probes**
Abutment A	IBR-31, URS-4	-	-
Pier 1	IBR-20, IBR-22	-	-
Pier 2	IBR-15, IBR-16	-	-
Pier 3	IBR-06	-	-
Abutment B	IBR-04, IBR-05, IBR-07	-	-

FILL: FILL extended to a depth of about 2-ft below the existing ground surface. The FILL material encountered within the limits of the proposed Ramp G1 over SR 7 consisted of medium dense silty SAND (USCS: SM) [AASHTO: A-2-4].

The one SPT N-values taken in the FILL material had a value of 20-bpf.
Stratum I - Columbia Formation - Coarse Grained Soil: Stratum I generally consisted of medium dense to dense SAND with varying percentages of silt (USCS: SM) [AASHTO: A-2-4].

The SPT N-values generally ranged from approximately 12 to 29-bpf, with an average SPT N-value of 17-bpf.

Stratum IIa – Cretaceous - Fine Grained Soil: Stratum IIa generally consisted of soft to very hard CLAY and SILT with varying percentages of sand and trace to little amounts of lignite and mica (USCS: CL, CL-ML, ML, CH) [AASHTO: A-4, A-6, A-7-6]. Interbedded thin lenses of Stratum IIb are also classified as silty and clayey SAND (USCS: SM, SC-SM) [AASHTO: A-2-4].

The Stratum IIa SPT N-values typically ranged from 4-bpf to 100+ bpf, averaging 23-bpf. The natural moisture content averaged 20.0-percent and ranged from 12.6 to 29.8-percent. The liquid limit ranged from 16 to 66, averaging 38. The plasticity index ranged from 1 to 39, averaging 16. The average percentage of fines (percent passing the No. 200 sieve) ranged from 46 to 100-percent, averaging 88-percent.

The undrained shear strength results from the unconsolidated undrained (UU) Triaxial testing ranged from 2,232 to 4,479-psf, averaging 3,356-psf. The undrained shear strength test results from the unconfined compression (UCC) testing averaged 1,767-psf and ranged from 741 to 2,272-psf. The direct shear test results from the undisturbed Shelby tubes indicated an average drained angle of friction of 14-deg and an average cohesion of 868-psf.

Stratum IIb – Cretaceous Coarse Grained Soil: Stratum IIb generally consisted of loose to very dense, SAND with varying percentages of silt and clay (USCS: SM, SC-SM, SP, SP-SM) [AASHTO: A-2-4, A-1-b, A-4].

The SPT N-values for this stratum averaged 35-bpf and typically ranged from 7 to 100+ bpf. The natural moisture content averaged 18.3-percent and ranged from 9.3 to 25.1-percent. The liquid limit ranged from non-plastic to 39, averaging 25 where existent. The plasticity index ranged from non-plastic to 19, averaging 9 where existent. The average percentage of fines (percent passing the No. 200 sieve) ranged from 9 to 81-percent, averaging 37-percent.

For Structure S5, site specific soil parameters were developed and used for design of the foundation elements. The Structure S5 soil parameters are summarized in Table 4.17 below. Table 4.18 summarizes the soil parameters used for the settlement analysis of this structure.

Table 4.17 – Structure S5 Soil Parameters				
Strata	**Total Unit Weight – γ (pcf)**	**Drained**		**Undrained Shear - S_u (psf)**
		Angle of Friction ϕ (deg)	**Cohesion – c (psf)**	
IIa – EL 70 to 20	120 to 125	24	-	1,250 to 2,500
IIa - EL 20 to -15	125	27	-	2,500 to 4,000
IIa - EL -15 to -25	125 to 130	29 to 30	-	4,000 to 8,000
IIb	115 to 135	30 to 42	-	-

Table 4.18 – Structure S5 Soil Parameters for Settlement						
Strata	**Total Unit Weight – γ (pcf)**	**E (tsf)**	**C_c**	**C_r**	**e_o**	**P_c (tsf)**
IIa – EL 70 to 40	125	-	0.29 to 0.33	0.065 to 0.068	0.786 to 0.851	3.48 to 4.5
E – Soil Modulus C_c – Compression Index C_r – Recompression Index			e_o – Initial Void Ratio P_c – Preconsolidation Pressure			

Groundwater for this structure is anticipated to be near EL 30, approximately 30 to 45-ft below the existing ground surface.

4.4.6 Structure S6: Ramp B over Northbound I-95

Table 4.19 summarizes the borings that were used for the evaluation of the Ramp B over Northbound I-95 structure. The Summary of Subsurface Data for this structure is contained in Figures A-4a and A-4b, located in Appendix A of this report.

Table 4.19 - Subsurface Exploration: Structure S6 - Ramp B over Northbound I-95			
Structure	**SPT Borings**	**CPT Probes**	**DMT Probes**
Abutment A	IBR-26A, IBR-27A, IBR-55	ICPT-15A	-
Pier 1	IBR-28, IBR-56, URS-2	ICPT-17 , ICPT-18	-
Pier 2	IBR-24, IBR-32, IBR-57, IBR-58, IRW-64	-	-
Pier 3	IBR-59, IBR-60, IRW-68, URS-3	ICPT-21, ICPT-22, ICPT-23	IDMT-21
Abutment B	IBR-33, IBR-61, IRW-69, IRW-70, IRW-89	-	IDMT-22

FILL: FILL materials encountered within the limits of the proposed Ramp B over I-95 NB extended to depths from 2-ft to 15-ft below the existing ground surface. The FILL material encountered within the project site consisted of stiff Silty CLAY (USCS: CL-ML) [AASHTO: A-4] as well as loose to medium dense Silty SAND (USCS: SM) [AASHTO: A-2-4, A-4]. Varying amounts of organic material and angular gravel were also encountered in this stratum.

The SPT N-values generally ranged from approximately 7 to 26-bpf, with an average SPT N-value of 13.6-bpf. The natural moisture content was 12.5-percent in the one sample tested.

Stratum I – Columbia Formation – Fine-Grained to Coarse-Grained Soil: Stratum I generally consisted of loose to dense silty SAND (USCS: SP-SM, SM) [AASHTO: A-2-4, A-1-b] and medium stiff to stiff Silty or Sandy CLAY (USCS: CL-ML, CL) [AASHTO: A-4, A-6, A-7-5]. Trace amounts of organics, gravel, and mica were also encountered.

The SPT N-values ranged from 5 to 41-bpf, with an average SPT N-value of 15.7-bpf. The natural moisture content averaged 17-percent and ranged from 9.1 to 22-percent.

Stratum IIa – Cretaceous – Fine-Grained Soil: Stratum IIa generally consisted of soft to hard CLAY and SILT with varying percentages of sand, lignite, and mica (USCS: CL, CL-ML, CH, MH, OH) [AASHTO: A-4, A-6, A-7-5, A-7-6]. This stratum was interbedded with Stratum IIb.

The Stratum IIa SPT N-values ranged from 4 to 100-bpf, averaging 23-bpf. The natural moisture content averaged 20-percent and ranged from 14.9 to 34-percent. The liquid limit ranged from 22 to 56, averaging 36.7. The plasticity index ranged from 4 to 30, averaging 16.3. The percentage of fines (percent passing the No. 200 sieve) ranged from 57 to 100-percent, averaging 86-percent.

Stratum IIb – Cretaceous Coarse Grained Soil: Stratum IIb generally consisted of loose to very dense silty SAND with traces of lignite and/or mica (USCS: SP-SM, SM) [AASHTO: A-3, A-4, A-2-4]. Interbedded thin lenses of Stratum IIb are also classified as stiff to hard SILT or CLAY with varying percentages of Sand (USCS: ML, CL-ML, CL) [AASHTO: A-4, A-6].

The SPT N-values for this stratum averaged 33-bpf and typically ranged from 7 to 100+ bpf. The natural moisture content averaged 22.9-percent and ranged from 17.3 to 30.1-percent. The percentage of fines (percent passing the No. 200 sieve) ranged from 19 to 78-percent, averaging 45.6-percent. The range of percent fines includes the thin lenses of this stratum which are interbedded within Stratum IIa.

For Structure S6, site specific soil parameters were developed and used for design of the foundation elements. The Structure S6 soil parameters are summarized in Table 4.20 below. Tables 4.21 and 4.22 summarize the soil parameters used for the settlement analysis of this structure per abutment location. Supporting calculations for development of soil parameters for Structure S6 are provided in Appendix F.

Table 4.20 – Structure S6 Soil Parameters

Strata	Total Unit Weight – γ (pcf)	Modulus of Horizontal Subgrade Reaction (pci)	Drained		Undrained Shear - S_u (psf)
			Angle of Friction – ϕ (deg)	Cohesion – c (psf)	
FILL	125	55	28	-	-
I	125	55	32	-	-
IIa – Above EL 40	130	60	24	-	1,700
IIa - EL 40 to 20	130	100	24	-	2,500
IIa - EL 20 to 0	130	140	24	-	3,500
IIa – Below EL 0	130	200	24	-	5,000

Table 4.21 – Structure S6 Soil Parameters for Settlement Abutment A

Strata	Total Unit Weight – γ (pcf)	μ	E (ksf)	C_c	C_r	e_o	C_v (ft^2/day)	OCR
I – Above EL 65	125	0.3	1500	-	-	-	-	-
IIa – EL 65 to 40	130	0.4	818	0.092	0.015	0.655	-	5.3
IIa - EL 40 to 20	130	0.4	818	0.092	0.015	0.655	-	5.3
IIa- EL 20 to 0	130	0.4	818	0.105	0.015	0.605	-	2.3
IIa – Below EL 0	130	0.4	1364	0.105	0.015	0.605	-	2.3

μ – Poisson Ratio
C_c – Compression Index
C_r – Recompression Index
e_o – Initial Void Ratio
C_v – Coefficient of Consolidation
OCR – Overconsolidation Ratio

Table 4.22 – Structure S6 Soil Parameters for Settlement Abutment B

Strata	Total Unit Weight – γ (pcf)	μ	E (ksf)	C_c	C_r	e_o	C_v (ft^2/day)	OCR
I – Below EL 79	130	0.3	1500	-	-	-	-	-
IIa – EL 79 40	130	0.4	818	0.24	0.015	0.748	-	3.3
IIa - EL 40 to 20	130	0.4	818	0.22	0.015	0.839	-	3.3
IIa - EL 20 to 0	130	0.4	818	0.22	0.015	0.839	-	2.1
IIa – Below EL 0	130	0.4	1364	0.22	0.015	0.839	-	2.1

μ – Poisson Ratio
C_c – Compression Index
C_r – Recompression Index
e_o – Initial Void Ratio
C_v – Coefficient of Consolidation
OCR – Overconsolidation Ratio

Groundwater for this structure is anticipated to be near EL 75, approximately 11 to 14-ft below the existing ground surface.

4.4.7 Structure S7: Ramp R1 over SR 7

Table 4.23 summarizes the borings that were used for the evaluation of the subsurface conditions for the Ramp R1 over SR 7 structure. The Summary of Subsurface Data for this structure is contained in Figures A-4c through A-4f, located in Appendix A of this report.

Table 4.23 - Subsurface Exploration: Structure S7 - Ramp R1 over SR 7

Structure	SPT Borings	CPT Probes	DMT Probes
Abutment A Wingwalls I and II	IBR-03, IBR-51 IBR-52, IRW-11 IRW-10	ICPT-01, ICPT-02, ICPT-03	IDMT-04
Pier	IBR-50, IBR-53	ICPT-01, ICPT-02, ICPT-03	IDMT-04
Abutment B Wingwalls III and IV	IBR-01, IBR-49 IBR-54	ICPT-01, ICPT-02, ICPT-03	IDMT-04

FILL: FILL materials extended to a depth of about 2-ft below the existing ground surface. The FILL material encountered within the project site consisted of stiff to very stiff Silty CLAY (USCS: CL-ML) [AASHTO: A-4] as well as loose to dense Silty and Clayey SAND (USCS: SM, SC) [AASHTO: A-2-4, A-4]. Varying amounts of organic material and angular gravel were also encountered in this stratum.

The SPT N-values generally ranged from approximately 8 to 30-bpf, with an average SPT N-value of 18-bpf. The natural moisture content averaged 11.3-percent and ranged from 5 to 16.4-percent.

Stratum I – Columbia Formation - Fine to Coarse-Grained Soil: Stratum I generally consisted of loose to medium dense silty or clayey SAND (USCS: SC, SM) [AASHTO: A-2-4, A-2-6] and medium stiff to stiff SILT and CLAY (USCS: CL-ML, CL) [AASHTO: A-4, A-6]. Trace amounts of gravel were also encountered.

The SPT N-values ranged from 6 to 13-bpf, with an average SPT N-value of 10-bpf.

Stratum IIa – Cretaceous - Fine-Grained Soil: Stratum IIa generally consisted of soft to hard CLAY and SILT with varying percentages of sand, lignite, and mica (USCS: CL, CL-ML, CH, ML) [AASHTO: A-4, A-6, A-7-5, A-7-6]. This stratum was interbedded with Stratum IIb.

The Stratum IIa SPT N-values ranged from 4-bpf to 72-bpf, averaging 18-bpf. The natural moisture content averaged 20.4-percent and ranged from 11.2 to 30.7-percent. The liquid limit ranged from 16 to 72, averaging 20. The plasticity index ranged from 2 to 46, averaging 17. The percentage of fines (percent passing the No. 200 sieve) ranged from 52 to 100-percent, averaging 82-percent.

Based on the nearby CPT and DMT probes, the undrained shear strength ranged from 700-psf to 10,400-psf, averaging 2,700-psf.

Stratum IIb – Cretaceous Coarse Grained Soil: Stratum IIb generally consisted of loose to very dense silty SAND with traces of lignite and/or mica (USCS: SP, SM) [AASHTO: A-3, A-4, A-2-4]. Interbedded thin lenses of Stratum IIb are also classified as very stiff to hard SILT with varying percentages of Sand (USCS: ML, CL-ML) [AASHTO: A-4].

The SPT N-values for this stratum typically averaged 44-bpf and ranged from 7 to 100+ bpf. The natural moisture content averaged 22.4-percent and ranged from 7 to 41.3-percent. The percentage of fines (percent passing the No. 200 sieve) ranged from 8 to 92.7-percent, averaging 44.2-percent. The range of percent fines includes the thin lenses of this stratum which are interbedded within Stratum IIa.

The friction angle from the nearby CPT and DMT probes ranged from 33 to 50-degrees, averaging 42-degrees.

For Structure S7, site specific soil parameters were developed and used for design of the foundation elements. The Structure S7 soil parameters are summarized in Table 4.24 below. The stress history for this structure is contained in Figure C-4 in Appendix C of this report. Supporting calculations for development of soil parameters for Structure S7 are provided in Appendix F.

Table 4.24 – Structure S7 Soil Parameters					
		Drained			
Strata	Total Unit Weight – γ (pcf)	Angle of Friction – ϕ (deg)	Cohesion – c (psf)	Undrained Shear - S_u (psf)	Modulus of Horizontal Subgrade Reaction k (pci)
IIa – Above EL 30	130	24	-	800/1,000	250
IIa – EL 30 to 25	130	24	-	2,000	250
IIa – EL 25 to 20	130	24	-	2,500	250
IIb – EL 20 to 10	130	34	-	-	75
IIa – EL 10 to 0	130	24	-	3,500	500
IIb – EL 0 to -18	130	34	-	-	75
IIa – EL -18 to -29	130	24	-	5,000	500
IIb – Below EL -29	130	34	-	-	150

Groundwater for this structure is anticipated to be near EL 30 to 45. Groundwater was encountered approximately 10-ft below the existing ground surface near Abutment A and near existing ground surface for Abutment B. Groundwater is anticipated to be approximately 10 to 15-ft below the existing ground surface elevation of SR 7.

4.4.8 Structure S8: SR 7 over Eagle Run

The borings that were previously drilled by the Walton Corporation in 1989 for SR 1 over Eagle Run are summarized in Table 4.25. The Summary of Subsurface Data for this structure is contained in Appendix B of this report, drawing number 114 through 116.

Table 4.25 - Subsurface Exploration: Structure S8 - SR 1 over Eagle Run			
Structure	SPT Borings	CPT Probes	DMT Probes
Abutment 1	S-7, S-13	-	-
Bent 1	S-9	-	-
Bent 2	S-10, S-15	-	-
Abutment 2	S-11, S-16	-	-
Wingwall C	S-12	-	-
Wingwall D	S-17	-	-

4.5 GROUNDWATER

In addition to the Potomac Formation being the primary bearing stratum for deep foundations at this site, the sand facies of this formation are also a substantial aquifer, reportedly providing the largest source of groundwater in New Castle County. Groundwater occurs within the discontinuous sands of the formation, accounting for variable depths at which water was encountered during the drilling operation, and can be under artesian pressure. No artesian conditions were encountered during this study. It is important to distinguish groundwater conditions within the Potomac Formation from the perched groundwater conditions within the overlying Pleistocene sands.

Table C-3 in Appendix C summarizes the groundwater depth and elevation encountered in the boreholes. Tables C-4a and C-4b in Appendix C summarizes the groundwater depth and elevation from the groundwater monitoring wells constructed for both the Interchange and Mainline, respectively.

A discussion regarding the depth where groundwater was encountered below the existing ground surface for each structure is contained in Sections 4.4.1 through 4.4.8 of this report.

Groundwater was encountered at depths ranging from 0 to about 40-ft below the existing ground surface. In some boreholes, groundwater was not encountered. To achieve a more accurate determination of the hydrostatic water table, perforated pipes or piezometers were installed to monitor over a period of time. The actual level of the hydrostatic water table and the amount and level of perched water should be anticipated to fluctuate throughout the year, depending upon variations in precipitation, surface run-off, infiltration, site topography, and drainage.

It is generally desirable to allow test borings to remain open for at least twenty-four hours after the completion of drilling and the removal of the drill tools and casing from the borehole. The purpose of this procedure is to allow the groundwater level in each borehole to recover from the effects of the test drilling. In clay soils, the length of time may extend several days before the groundwater level recovers to the pre-drilling elevation.

In addition to groundwater levels, the depth to the bottom of each borehole was measured to determine the susceptibility of the borehole to collapse or cave. This information provides the Contractor with information regarding the "stand-up" time of the soil or the ability of the sides of an excavation to remain vertical or near vertical during trench excavation.

It was necessary to backfill certain borings immediately after the completion of drilling. In cases where the boring was immediately backfilled, the boring logs note the depth where groundwater was observed either within the recovered soil sample, on the split barrel sampler, on the drill rods, or in the soil brought to the surface by the hollow stem augers.

5 EVALUATIONS AND FINAL RECOMMENDATIONS

The following recommendations have been developed on the basis of the previously described project characteristics and subsurface conditions. If there are any significant changes to the project characteristics or if significantly different subsurface conditions are encountered during construction, RK&K/URS should be consulted so that the recommendations of this report can be reviewed.

The recommendations provided in this report are based on the TS&L submission, the results of the supplemental subsurface exploration and laboratory testing, and the results of the instrumented test embankment.

The clays in this area have OCR's that range from about 2 to over 5 and have undrained shear strengths that range from about 1-ksf to over 4-ksf. Because of this variability, for some walls the undrained condition governs the design. This is described in more detail in Section 5.6.

5.1 BRIDGE FOUNDATION ALTERNATIVE ANALYSIS

The primary purpose of the bridge foundation alterative analysis was to assess suitable foundation types relative to the applied loads, physical constraints of the site, and the subsurface conditions that have been encountered during the initial and supplemental subsurface exploration.

The following bridge foundations were evaluated for construction. Detailed recommendations for each bridge structure are contained in Sections 5.4.1 through 5.4.8 of this report.

- Prestressed- Precast Concrete Piles (Section 5.1.1)
- Driven Cast-in-Place Piles (Section 5.1.2)
- Drilled Shafts (Section 5.1.3)
- Steel H-Piles (Section 5.1.4)
- Steel Pipe Piles (Section 5.1.5)
- Shallow Foundations (Section 5.1.6)

5.1.1 Prestressed-Precast Concrete Piles (PPC)

Prestressed-Precast Concrete (PPC) piles have been used successfully in this general geologic setting but are not recommended for the particular conditions of this project for several reasons.

The main advantage to PPC piles is they tend to be less expensive per pile due to higher load carrying capabilities than most other piles.

The *disadvantages* of PPC piles are they tend to be more difficult to install and more susceptible to damage from striking unusually soft or hard layers or obstructions during driving. Also, it is not recommended that prestressed-precast concrete piles be spliced, but if splices are required they need to be part of the design prior to installation. This reduces the ability to adjust or respond to differing subsurface conditions.

PCC piles tend to be heavier per foot compared to steel pipe piles or H piles. This will require larger cranes and other equipment to fit into a rather cramped work site. Specifically, the access to install test piles will be limited by median pier locations within the I-95 and SR 7 right-of-way, thereby creating a higher preference for the flexibility to accommodate differing site conditions with steel pile options that are more readily spliced or cut off, as necessary. Second, the installation of test piles to establish order lengths prior to production generally requires a relatively large number of piles of the same size, so as to avoid the delays and production driving inefficiencies associated with concurrent precasting of production length piles. At this site, it is possible that each foundation element may require piles of different lengths so there will be significant delays between the test pile program and production pile installation. Third, 18 to 24-in diameter PPC piles of the 65 to 70-ft lengths necessary to develop estimated factored resistances on the order of 170 tons and 215 tons, respectively, would be difficult to handle within median locations. The maneuverability of a crane may be further inhibited by recommended limits on operating radii for the relatively heavy pile sections. Fourth, inclusions of dense, discontinuous layers of sand can dramatically influence the end bearing of displacement piles, which do not readily adapt for use in Potomac Formation unless significant cost savings can be realized after accounting for cost of test pile installations on relatively frequent substructure intervals.

5.1.2 Driven Cast-in-Place Piles

Mandrel driven steel shells, filled with concrete, such as the Monotube® pile, manufactured by the Monotube Pile Corporation of Canton, OH, can provide a high-capacity deep foundation at

locations where dense bank sands are of sufficient thickness and areal extent to consistently develop the product's primary advantage - friction on the tapered end section of the pile. Based on the anticipated structural loads for this project, monotube piles would not satisfy the slenderness ratio requirement at the MSE abutments. Sites such as this, where clayey soils predominate, or the occurrence and thickness of sand is possibly erratic, driven cast-in-place shell piles would not be appropriate for uniform use, and therefore are not expected to provide an economical foundation support option.

5.1.3 Drilled Shafts

Even though drilled shafts often provide an appropriate foundation support system within areas where plan footing dimensions must be limited, such as highway median locations, the potential disadvantages associated with this mode of construction are avoidable in favor of low-displacement driven steel pile alternatives. Specific drawbacks associated with drilled shafts include the need to place casing through the Pleistocene sands in the upper 20 to 25 feet adjacent to active traffic lanes. The potential for encountering groundwater within the Cretaceous deposits below that depth would likely require full depth casing or slurry. Also, in view of the very large shaft diameters on the order of 8 to 10 feet necessary to accommodate a factored 2,900 ton mono-shaft reaction for the I-95 Ramp A median pier, we anticipate removal of excavation spoils, installation of a reinforcing cage in excess of 100 feet long, and placement of concrete would all pose significant challenges for maintenance of traffic.

Finally, due to the non-redundancy of a mono-shaft foundation, crosshole sonic logging (CSL) and probably a load test would be needed before production work can begin, in addition to the delay time required to cure the concrete and possibly perform coring or remedial grouting if suspected concrete deficiencies are detected by the CSL testing. CSL will also need to be performed on some if not all production shafts.

5.1.4 Steel H-Piles

Low-displacement steel H-piles provide the significant advantage of flexibility for this particular site where the conditions of end bearing are likely to vary abruptly from dense sand to stiff or hard clay over short distances

Advantages to steel H-piles are that they can be easily spliced during installation, they are available in various sizes, and they provide high capacity with minimal displacement.

The *disadvantages* to steel H-piles are they may be damaged or deflected by obstructions, and the capacity of individual piles is smaller than that of pipe piles. Also, due to their different behavior when plugged or unplugged, pile capacity evaluations for H-piles tend to be more difficult when using dynamic methods.

For this project, H-piles sizes of 12 and 14-in are expected to provide nominal (ultimate) resistances (R_n) on the order of 180 to 250 tons, respectively, at estimated plan embedment depths of approximately 80 feet as illustrated in Figures D-1 and D-2 in Appendix D of this report. Factored resistances will be based upon a resistance factor (Ø) of 0.65, assuming driving criteria is established by high-strain dynamic tests with signal matching at the beginning of restrike (BOR) for at least one production pile per pier/abutment, and that quality control of the remaining piles is calibrated by wave equation or dynamic testing is illustrated in Figures D-3 and D-4 in Appendix D of this report. Figures D-1 through D-4 do not account for group effects that will vary according to pile spacing and may reduce the estimated resistances. These figures should be used for preliminary sizing purposes only, subsurface unit specific soil strata and parameters have been established to estimate pile lengths for this report and are presented in Section 5.4.1 through 5.4.8.

Unsupported pile lengths of as much as 25 to 35 feet for piles extended through MSE walls at abutments is a concern for such slender piles and may not satisfy the slenderness ratio requirements.

Further discussion regarding downdrag on piles and pile installation concerns are contained in Section 5.5 – Special Considerations.

5.1.5 Steel Pipe Piles

Similar to H-piles, pipe piles, driven open-ended, provide a low-displacement pile alternative with the operational flexibility to splice and cut off piles, as necessary in response to variable subsurface conditions. In addition, the section modulus for pipe pile sections is appropriate for the 25 to 35-foot unsupported lengths extending through wrap-around MSE walls. They can be cleaned out and driven further, and they provide high capacity with minimal displacement, thereby, reducing the risk of dislocating nearby piles.

For this project, pipe piles with 18 to 42-in diameters are expected to provide nominal (ultimate) resistances (R_n) on the order of 220 to 570 tons, respectively, at estimated plan embedment depths of approximately 80-ft as summarized in Figures D-5 and 6 in Appendix D of this report. These resistances do not account for group effects that will vary according to pile spacing and may reduce the estimated resistances. Figures D-7 and 8 in Appendix D of this report depict

factored resistances based upon comparable assumptions related to the selected resistance factor (Ø of 0.65) as discussed in the Steel H-pile section above. Figures D-5 through D-8 do not account for group effects that will vary according to pile spacing and may reduce the estimated resistances. These figures should be used for preliminary sizing purposes only, subsurface unit specific soil strata and parameters have been established to estimate pile lengths for this report and are presented in Sections 5.4.1 through 5.4.8.

Even though pipe piles possess a higher (and directionally uniform) moment of inertia beneficial to the resistance to bending, none of the pipe pile sections are deemed adequate to resist lateral forces at the Ramp A/G1 north abutment, which will need to be strapped into the MSE section. Supporting calculations are provided in Appendix F.

The plugging of an open pile section was evaluated. The FHWA Manual on Driven Piles reported plugging occurs at penetration-to-pile diameter ratios of 10 to 20. Studies have reported plugging of open pile sections in any soil material is probable under static loading conditions once the penetration to pile diameter ratio exceeds 20.

The abutment piles will be 24-inches in diameter, therefore, plugging is probable for a pile penetration in excess of 40-ft. Similarly the pier piles will be 36-inches in diameter, therefore, plugging is probable for a pile penetration in excess of 60-ft. For the given loads and the existing soil profile, with the toe resistance from plugging of piles taken into consideration, the difference in pile length for the abutments is about 5-ft and the change in pile length for piers is about 9-ft. Due to the high degree of uncertainty of plug formation and plug response under static and dynamic loading and the change in pile length not being very significant, toe resistance from plugging of piles was not included in our estimate of the total pile resistance. Supporting calculations for structure pile foundations are in Appendix F of this report.

5.1.6 Shallow Foundation – Spread Footing

The *advantages* of shallow spread footing foundations are in the cost savings in construction and material costs, a reduced construction footprint and reduced schedule.

The *disadvantages* of the shallow foundation are larger excavation and the increased risk of foundation movements. In some areas, shallow groundwater could complicate constructability.

This foundation type was only considered for the Ramp R1 pier as it was not structurally feasible at the other locations. Based on the anticipated structural loads, bearing resistance, and anticipated differential settlement, a shallow foundation is not recommended at the Ramp

R1 pier. Preliminary calculations for the Ramp R1 shallow foundation pier option are provided in Appendix F.

5.2 BRIDGE WINGWALL FOUNDATION ALTERNATIVE ANALYSIS

The primary purpose of the wingwall foundation alternative analysis was to assess suitable wall types relative to the physical constraints of the site and the subsurface conditions encountered. The following is a general discussion of the advantages and disadvantages of each wall type and the recommended wall type.

The following wingwall foundations were evaluated for construction. Detailed recommendations for each wingwall structure are contained in Sections 5.6.2 through 5.6.9 of this report.

- Cast-in-Place (Section 5.2.1)
- Pile Supported Cast In Place (Section 5.2.2)
- Mechanically Stabilized Earth Walls – MSE (Section 5.2.3)
- Other Wall Types (Section 5.2.4)

5.2.1 Cast-in-Place - CIP

Cast-in-place Concrete Walls (CIP) with a spread footing could be used to retain the soil mass. Wall heights can range from 5 to 60-ft, but usually, above 20-ft the wall heights become uneconomical.

Advantages of the CIP wall include a conventional wall system with well established design procedures and performance characteristics, durability, ability to easily be formed, textured, or colored to meet aesthetic requirements.

Disadvantages include a relatively long construction period due to undercutting, excavation, dewatering, form work, steel placement and curing. The rigid wall system is sensitive to total and differential settlements unless the CIP wingwalls are pile supported; the differential settlement with the bridge abutments will likely be significant.

5.2.2 Pile Supported Cast-In Place – Pile Supported CIP

The *advantages* to installing a pile supported CIP wall include those of CIP walls discussed above with the addition that differential settlements are reduced because the foundation loads can be transferred to deeper soils.

A pile supported CIP wall is typically more expensive and time consuming to construct compared to a CIP wall or an MSE wall. For structure S2, Ramp A over SR 7, it is anticipated that the cost for a pile supported CIP abutment will be approximately $1,640,000.

This wall option was only considered for the wingwalls for Structure S7 – Ramp R1 over SR 7. The construction of the Abutment A wingwalls of S7 will be adjacent to an existing and fully functioning SWM pond. Other wall types as discussed in Section 5.2.4 of this report identify constructability issues. Approximately 10 to 25-ft of cut is required to reach proposed grade for the S7 wingwalls.

Pile supported CIP wingwalls were not evaluated for other locations as they are located within embankment fill.

5.2.3 Mechanically Stabilized Earth Walls (MSE)

An MSE wall is typically associated with fill wall construction, and typically consists of facing, such as segmental precast units, dry block concrete or CIP concrete facing units connected to horizontal steel strips, bars, or geosynthetic that create a reinforced soil mass. The reinforcement is typically placed in horizontal layers between successive layers of granular backfill. A free draining, low plasticity backfill is required to provide adequate performance of the wall. MSE walls can be used in cut situations as well. The additional cost of the excavations is usually offset by the savings in construction costs and schedule as compared to a CIP wall.

The wingwalls may be constructed using MSE walls. The design of MSE for the retaining walls for internal stability will be the Contractor's responsibility and will need to be designed by a Professional Engineer licensed in the State of Delaware and reviewed by the Engineer. Minimum reinforcement length should be designed to satisfy external and global stability. Additional discussion is contained in Section 5.6 of this report.

Advantages of an MSE wall include a relatively rapid construction schedule that does not require specialized labor or equipment, provided excavation for the reinforcement is not extensive. This type of retaining wall can accommodate relatively large total and differential settlements without distress, and the reinforcement materials are light and easy to handle. Facing panels can be designed for various architectural finishes. If large settlements or bearing capacity difficulties are expected, the MSE's can be built with a temporary facing until the settlements have ceased, then the permanent facing can be installed or light weight material can be used such as No. 57 stone or Lightweight Engineered Fill (LWEF).

Based upon the above, we expect that the highest anticipated MSE abutment walls can be constructed to accommodate the total magnitude of estimated settlement, without the specific need for two-stage construction to induce settlement prior to erecting the facing. However, appropriately spaced vertical wall joints, controlled panel sizes, and some combination of lighter weight crushed stone backfill materials or LWEF and staged construction have been considered to control potential differential deflections with nearby structures or provide adequate global stability.

Disadvantages for the construction of an MSE for a cut structure include additional excavation for the reinforcement zone.

In the Preliminary Foundation Report (PFR), it was noted that abutment piles driven prior to MSE construction through the MSE section would most likely be subject to downdrag, not only from the granular MSE backfill, but also due to adhesion within the natural clayey soils in long-term consolidation below the base of wall. Based upon settlement measurements of the test embankment it is now evident that primary consolidation occurs rapidly enough that downdrag can effectively be eliminated with reasonable quarantine periods to affect consolidation and pipe sleeves through the MSE fill.

Accordingly, design alternatives based upon reducing the influence of downdrag are addressed in Section 5.5.2 of this report.

5.2.4 Other Wall Types

Other wall types considered for the construction of the wingwalls for this project were:

- Soldier Pile and Lagging
- Soldier Pile and Lagging with Tiebacks
- Sheet Pile Wall

Based on engineering judgment, these wall types were not further evaluated or developed due to constructability and anticipated construction costs. Predominately, a soldier pile and lagging wall was not considered as tiebacks would be needed during construction as the exposed wall heights are greater then 10-ft. In addition, the majority of the wingwalls are within embankment fill, as opposed to a cut situation where the above referenced walls are typically considered.

5.3 SUMMARY OF BRIDGE AND WINGWALL FOUNDATION ANALYSIS

For the SR 1/I-95 Interchange project, it is recommended that the majority of the bridge structures be supported on 24 to 36-inch diameter driven open-ended steel pipe pile foundations. Detailed recommendations for each structure are in Section 5.4 of this report. As stated previously, low displacement piles are recommended as they provide greater field flexibility to accommodate unanticipated changes in subsurface conditions, specifically at the pile tip where inclusions of dense, discontinuous layers of sand can dramatically influence the pile installation operations. Even though H-piles are feasible for use where they meet minimum unbraced length, comparison of the axial load resistance per lineal foot of installed pile also proves the pipe sections to be more cost efficient than H-piles for the typical applications of this project, see supporting calculations in Appendix F. Additionally, in several pier locations, there is not enough room available to place a large number of H-piles with a moderate capacity, therefore, higher capacity pipe piles are recommended. Some efficiency is gained by using a common foundation type.

The exception to this recommendation is Structure S7 – Ramp R1 over SR 7. Because of the relatively low foundation loads, we recommend the abutments and pier be supported on driven H piles.

Using staged construction will not greatly affect the stability of the MSE walls as a quarantine period is used mostly to allow settlements to occur. In staged construction significant gains in shear strength are only realized for under or normally consolidated clays where the increase in stress due to the weight of the embankment will eventually increase the shear strength of the foundation soils. This is not the case at this site as the soil is overconsolidated. However, a slight increase in total strength is realized on this site if the pore water pressures are allowed to dissipate as the walls are constructed and the soil begins to exhibit a friction angle in conjunction with shear strength. As such, where possible an incremental quarantine period is recommended at specific structures discussed in Sections 5.6.2 through 5.6.9.

It is recommended that MSE retaining walls be used for construction of the bridge wingwalls. To meet design criteria for this project, several abutment wingwalls will require Lightweight Engineered Fill (LWEF) and No. 57 stone to be used within the reinforcement and retained zones. Also, to satisfy bearing and global stability, the minimum reinforcement length was increased for select structures from 0.7H to as much as 1.2H, where H is the height of the MSE wall from top of wall to the leveling pad. Some of the taller wingwalls will be located on relative soft soils; therefore some of the construction of these walls may need to be staged. These topics are further discussed in Section 5.6.1 of this report. Even with these special treatments

and quarantine period, it will take less time and cost less to construct MSE walls then to use a pile supported CIP concrete wall.

To provide the most economical design, the cost for the following special treatment options for RW9 were evaluated. This analysis was used to determine the cost-effectiveness of various treatments for the entire project.

- Case 1: Increased reinforcement zone (L/H=1.0) with select fill, retained wedge behind the wall of common borrow (Type F).
- Case 2: L/H=0.7 with No. 57 Stone in the reinforcement zone and LWEF in the retained backfill wedge behind the retaining wall.
- Case 3: L/H=0.7 with combination of LWEF and No. 57 stone in the reinforcement zone and in the retained backfill wedge behind the retaining wall. The height of the LWEF for this case (H_2) is 17-ft and the height of No. 57 Stone (H_1) is 12-ft, see Figure A-1.
- Case 4: Stone columns, geopiers, or vibro-piers.

Table 5.1 summarizes the additional costs for the construction of each case listed above for RW9. The cost per foot of wall was estimated for a length of approximately 750-ft.

Table 5.1 - Summary of RW9 Special Treatment Options

Case No.	Additional Cost	Cost Per Foot of Wall
1	$171,900	$230
2	$236,800	$320
3	$470,940	$630
4	$438,570	$580

We recommend that the most cost effective option is to increase the reinforcement zone. In some walls, similarly increasing the reinforcement zone or using select fill in the retained backfill wedge behind the wall will still not provide adequate stability. In these areas, LWEF and staged construction should be used.

The exception to this recommendation is the Structure S7 – Ramp R1 over SR 7 where the wingwalls should be a CIP concrete wall bearing on driven H piles because of the existing pond the wingwalls will support.

Typically, the drained condition governs the design of the wingwalls; therefore, the Contractor is required to verify the external stability using both the drained and undrained condition summarized in Section 5.6 of this report for each retaining wall.

Special considerations include the sequence of construction, anticipated settlement from construction, downdrag on piles, driveability, and constructability. These items are discussed in further detail in Section 5.5 of this report.

Supporting calculations and a preliminary cost comparison are contained in Appendix F of this report.

5.4 BRIDGE FOUNDATION RECOMMENDATIONS

Bridge foundations are provided below for each structure. These recommendations are based on the TS&L structural drawings and the results of the supplemental subsurface exploration and laboratory testing.

5.4.1 Structure S1: Ramp A over I-95

We recommend open-ended steel pipe piles be used for the support of all substructure units for Ramp A over I-95. Specifically, we recommend 24-inch diameter piles for support of the abutments and 36-inch diameter piles for support of the piers. As discussed in Section 5.1.5, pipe piles are generally preferred as they provide the same moment of inertia in all directions, satisfy the slenderness ratio criteria at tall MSE abutments, and provide operational flexibility to splice and cut off piles where needed. Pipe piles will provide a viable means to contain the work area disruption within a limited width at the I-95 median. The low displacement of open-ended driving will also reduce heave potential and enable the piles to be driven to greater depths and capabilities than would otherwise be possible with a displacement pile. Based upon typical wall thicknesses suitable to the anticipated driven capability requirements, pipe sections are more cost efficient per ton of axial resistance than H-piles, supporting calculations are provided in Appendix F. The pile length, thickness, and nominal pile resistance required are summarized in Table 5.1, below. Supporting calculations for the axial capacities of the piles are included in Appendix F. It is anticipated that the piles will be spaced at a minimum of 2.5 times the diameter of the piles, therefore, group effects are not anticipated. All structural steel for this project should have a yield strength of 50-ksi.

It is anticipated that the Ramp A embankment, adjacent wingwalls and Retaining Walls 12, 13 and 5 will be constructed prior to driving the abutment piles. The proposed sequence of construction is further discussed in Section 5.5.1 of this report.

By constructing the embankment, wingwalls, and retaining walls first and imposing an estimated 30 to 60-day quarantine period prior to driving the abutment piles, downdrag forces on the piles

should not develop. Otherwise, downdrag forces will need to be incorporated into the pile design and piles may need to be driven deeper than indicated in Table 5.2.

Table 5.2 – Structure S1 Pile Foundation Recommendations

Structural Element	Pile Dia. (in)	Pipe Thickness (in)	Est. Pile Embedment (ft)[1] / Est. Tip Elevation	Service Loads Per Pile (kips)	Strength Loads Per Pile (kips)	Nominal Pile Driving Resistance Required (R_{ndr}) Per Pile (kips)
Abutment A	24	0.5	71 / 10	H: 59 V: 270	H: 83 V: 375	577
Pier 1	36	0.625	71 / 10	H: 36 V: 685	H: 38 V: 835	1285
Pier 2	36	0.625	84 / 1	H: 36 V: 685	H: 38 V: 835	1285
Pier 3	36	0.625	58 /24	H: 36 V: 685	H: 38 V: 835	1285
Abutment B	24	0.5	71 / 20	H: 59 V: 270	H: 83 V: 375	577

R_n = Nominal (ultimate) Bearing Resistance
R_n= Strength Load / (ϕ_{dyn})
Strength Load = $\Sigma\eta_i\gamma_i Q_i \leq \phi_{dyn}R_n = \phi_{static}R_{n\ static}$
ϕ_{dyn} = 0.65
γ_{dd} = 0.85
ϕ_{static} = 0.35
R_{ndr} = Nominal Pile Driving Resistance Required For High-Strain Dynamic Testing
$R_{ndr} = R_{sdd} + R_{n.}$
R_{sdd} = Skin Friction to be overcome during driving through downdrag zone. (If required)

η = Load Modifier
γ = Load Factor
Q = Service Load
H = Lateral Load
V = Axial Load

Note 1: Pile embedments and required driving resistances for the abutments assume that the embankments are built before the piles are driven so that no downdrag loads are imposed on the piles. Pile embedment is defined as extending down from the base of the MSE section.

Table 5.3 summarizes the maximum horizontal deflection at the bottom of the pile cap for the loads in Table 5.2 above. The minimum embedment is measured from the bottom of the pile cap and represents the minimum depth to which piles must be driven to resist lateral loads

created by the bridge superstructure. As discussed in Section 5.1.5 these abutments will be strapped and therefore lateral forces due to earth pressure will be resisted by the MSE.

Table 5.3 - Structure S1 Maximum Horizontal Pile Deflections

Structural Element	Maximum Deflection (in)	Minimum Embedment (ft)
Pier 1	0.12	24
Pier 2	0.12	26
Pier 3	0.25	27

The drivability analysis, discussed further in Section 5.5.3, and included in Appendix F, indicates the piles can be safely driven to the required nominal pile resistance without over stressing the pile.

5.4.2 Structure S2: Ramp A over SR 7

We recommend open-ended steel pipe piles be used for the support of all substructure units for Ramp A over SR 7. Specifically, we recommend 24-inch diameter piles for support of the abutments and 30-inch diameter piles for support of the piers. As discussed in Section 5.1.5, pipe piles are generally preferred as they satisfy the slenderness ratio criteria at tall MSE abutments and provide operational flexibility to splice and cut off piles where needed. The low displacement of open-ended driving will also reduce heave potential and enable the piles to be driven to greater depths and capabilities than would otherwise be possible with a displacement pile. Based upon typical wall thicknesses suitable to the anticipated driven capability requirements, pipe sections are more cost efficient per ton of axial resistance than H-piles, supporting calculations are provided in Appendix F. The pile length, thickness, and nominal pile resistance required are summarized in Table 5.3 below. Supporting calculations for the axial capacities of the piles are included in Appendix F. It is anticipated that the piles will be spaced at a minimum of 2.5 times the diameter of the piles, therefore, group effects are not anticipated. All structural steel for this project should have a yield strength of 50-ksi.

It is anticipated that the Ramp A, G1, B and C embankments, the associated wingwalls and Retaining Walls 1, 2, 3, 4 6, 9, and 16 will be constructed prior to driving the abutment piles. The proposed sequence of construction is further discussed in Section 5.5.1 of this report.

By constructing the embankment, wingwalls, and retaining walls first and imposing an estimated 30 to 60-day quarantine period prior to driving the abutment piles, downdrag forces on the piles

should not develop. Otherwise, downdrag forces will need to be incorporated into the pile design and piles may need to be driven deeper than indicated in Table 5.4.

Table 5.4 – Structure S2 Pile Foundation Recommendations

Structural Element	Pile Dia. (in)	Pipe Thickness (in)	Est. Pile Embedment (ft)[1] / Est. Tip Elevation	Service Loads Per Pile (kips)	Strength Loads Per Pile (kips)	Nominal Pile Driving Resistance Required (R_{ndr}) Per Pile (kips)
Abutment A	24	0.5	45 / 32	H: 40 V: 175	H: 58 V: 245	377
Pier 1	30	0.625	69 / -9	H: 43 V: 565	H: 48 V: 680	1046
Pier 2	30	0.625	51 / 14	H: 21 V: 430	H: 23 V: 500	769
Pier 3	30	0.625	83 /-18	H: 43 V: 565	H: 48 V: 680	1046
Abutment B	24	0.5	45 / 23	H: 40 V: 175	H: 58 V: 245	377

R_n = Nominal (ultimate) Bearing Resistance
R_n= Strength Load / (ϕ_{dyn})
Strength Load = $\Sigma\eta_i\gamma_i Q_i \leq \phi_{dyn}R_n = \phi_{static}R_{n\ static}$
ϕ_{dyn} = 0.65
γ_{dd} = 0.85
ϕ_{static} = 0.35
R_{ndr} = Nominal Pile Driving Resistance Required For High-Strain Dynamic Testing
$R_{ndr} = R_{sdd} + R_n$.
R_{sdd} = Skin Friction to be overcome during driving through downdrag zone. (If required)

η = Load Modifier
γ = Load Factor
Q = Service Load
H = Lateral Load
V = Axial Load

Note 1: Pile embedments and required driving resistances for the abutments assume that the embankments are built before the piles are driven so that no downdrag loads are imposed on the piles. Pile embedment is defined as extending down from the base of the MSE section.

Table 5.5 summarizes the maximum horizontal deflection at the bottom of the pile cap for the loads in Table 5.4 above. The minimum embedment is measured from the bottom of the pile cap and represents the minimum depth to which piles must be driven to resist lateral loads

created by the bridge superstructure. As discussed in Section 5.1.5 these abutments will be strapped and therefore lateral forces due to earth pressure will be resisted by the MSE.

Table 5.5 - Structure S2 Maximum Horizontal Pile Deflections

Structural Element	Maximum Deflection (in)	Minimum Embedment (ft)
Pier 1	0.40	25.5
Pier 2	0.05	17
Pier 3	0.23	22.5

The drivability analysis, discussed further in Section 5.5.3, and included in Appendix F, indicates the piles can be safely driven to the required nominal pile resistance without over stressing the pile.

5.4.3 Structure S3: Ramp B over SR 7

We recommend open-ended steel pipe piles be used for the support of all substructure units for Ramp B over SR 7. Specifically, we recommend 24-inch diameter piles for support of the abutments and 36-inch diameter piles for support of the pier. As discussed in Section 5.1.5, pipe piles are generally preferred as they satisfy the slenderness ratio criteria at tall MSE abutments and provide operational flexibility to splice and cut off piles where needed. The low displacement of open-ended driving will also reduce heave potential and enable the piles to be driven to greater depths and capabilities than would otherwise be possible with displacement piles. Based upon typical wall thicknesses suitable to the anticipated driven capability requirements, pipe sections are more cost efficient per ton of axial resistance than H-piles, supporting calculations are provided in Appendix F. The pile length, thickness, and nominal pile resistance required are summarized in Table 5.5 below. Supporting calculations for the axial capacities of the piles are included in Appendix F. It is anticipated that the piles will be spaced at a minimum of 2.5 times the diameter of the piles, therefore, group effects are not anticipated. All structural steel for this project should have a yield strength of 50-ksi.

It is anticipated that the Ramp A, G1, B and C embankments, the associated wingwalls and Retaining Walls 1, 2, 3, 4 6, 9, and 16 will be constructed prior to driving the abutment piles. The proposed sequence of construction is further discussed in Section 5.5.1 of this report.

By constructing the embankment, wingwalls, and retaining walls first and imposing an estimated 30 to 60-day quarantine period prior to driving the abutment piles, downdrag forces on the piles

should not develop. Otherwise, downdrag forces will need to be incorporated into the pile design and piles may need to be driven deeper than indicated in Table 5.6.

Table 5.6 – Structure S3 Pile Foundation Recommendations

Structural Element	Pile Dia. (in)	Pipe Thickness (in)	Est. Pile Embedment (ft)[1] / Est. Tip Elevation	Service Loads Per Pile (kips)	Strength Loads Per Pile (kips)	Nominal Pile Driving Resistance Required (R_{ndr}) Per Pile (kips)
Abutment A	24	0.5	54 / 16	H: 55 V: 200	H: 77 V: 275	423
Pier 1	36	0.625	73 / -13	H: 28 V: 645	H: 31 V: 775	1192
Abutment B	24	0.5	49 / 19	H: 55 V: 200	H: 77 V: 275	423

R_n = Nominal (ultimate) Bearing Resistance
R_n= Strength Load / (ϕ_{dyn})
Strength Load = $\Sigma\eta_i\gamma_i Q_i \leq \phi_{dyn}R_n = \phi_{static}R_{n\ static}$
ϕ_{dyn} = 0.65
γ_{dd} = 0.85
ϕ_{static} = 0.35
R_{ndr} = Nominal Pile Driving Resistance Required For High-Strain Dynamic Testing
$R_{ndr} = R_{sdd} + R_{n.}$
R_{sdd} = Skin Friction to be overcome during driving through downdrag zone. (If required)

η = Load Modifier
γ = Load Factor
Q = Service Load
H = Lateral Load
V = Axial Load

Note 1: Pile embedments and required driving resistances for the abutments assume that the embankments are built before the piles are driven so that no downdrag loads are imposed on the piles. Pile embedment is defined as extending down from the base of the MSE section.

Table 5.7 summarizes the maximum horizontal deflection at the bottom of the pile cap for the loads in Table 5.6 above. The minimum embedment is measured from the bottom of the pile cap and represents the minimum depth to which piles must be driven to resist lateral loads created by the bridge superstructure. As discussed in Section 5.1.5 these abutments will be strapped and therefore lateral forces due to earth pressure will be resisted by the MSE.

Table 5.7 - Structure S3 Maximum Horizontal Pile Deflections		
Structural Element	**Maximum Deflection (in)**	**Minimum Embedment (ft)**
Pier 1	0.02	15.5

The drivability analysis, discussed further in Section 5.5.3, and included in Appendix F, indicates the piles can be safely driven to the required nominal pile resistance without over stressing the pile.

5.4.4 Structure S4: Ramp C over SR 7

We recommend open-ended steel pipe piles be used for the support of all substructure units for Ramp C over SR 7. Specifically, we recommend 24-inch diameter piles for support of the abutments and 30-inch diameter piles for support of the pier. As discussed in Section 5.1.4 and 5.1.5, pipe piles or H piles are generally preferred as they provide operational flexibility to splice and cut off piles where needed and the low displacement of open-ended driving will also reduce heave potential and enable the piles to be driven to greater depths and capabilities than would otherwise be possible with displacement piles. Based upon typical wall thicknesses suitable to the anticipated driven capability requirements, pipe sections are more cost efficient per ton of axial resistance than H-piles, supporting calculations are provided in Appendix F. Furthermore, as pipe piles are the preferred alternative at all other bridge locations, their use for support at Ramp C provides efficiency and economy of scale.

The pile length, thickness, and nominal pile resistance required are summarized in Table 5.7 below. Supporting calculations for the axial capacities of the piles are included in Appendix F. It is anticipated that the piles will be spaced at a minimum of 2.5 times the diameter of the piles, therefore, group effects are not anticipated. All structural steel for this project should have a yield strength of 50-ksi.

It is anticipated that the Ramp A, G1, B and C embankments, the associated wingwalls and Retaining Walls 2, 6, 10, and 16 will be constructed prior to driving the abutment piles. The proposed sequence of construction is further discussed in Section 5.5.1 of this report.

By constructing the embankment, wingwalls, and retaining walls first and imposing an estimated 30 to 60-day quarantine period prior to driving the abutment piles, downdrag forces on the piles should not develop. Otherwise, downdrag forces will need to be incorporated into the pile design and piles may need to be driven deeper than indicated in Table 5.8.

Table 5.8 – Structure S4 Pile Foundation Recommendations

Structural Element	Pile Dia. (in)	Pipe Thickness (in)	Est. Pile Embedment (ft)[1] / Est. Tip Elevation	Service Loads Per Pile (kips)	Strength Loads Per Pile (kips)	Nominal Pile Driving Resistance Required (R_{ndr}) Per Pile (kips)
Abutment A	24	0.5	48 / 24	H: 34 V: 200	H: 50 V: 280	431
Pier 1	30	0.625	67 / 4	H: 21 V: 435	H: 29 V: 530	815
Abutment B	24	0.5	49 / 19	H: 39 V: 200	H: 55 V: 280	431

R_n = Nominal (ultimate) Bearing Resistance
R_n= Strength Load / (ϕ_{dyn})
Strength Load = $\Sigma\eta_i\gamma_i Q_i \leq \phi_{dyn}R_n = \phi_{static}R_{n\ static}$
ϕ_{dyn} = 0.65
γ_{dd} = 0.85
ϕ_{static} = 0.35
R_{ndr} = Nominal Pile Driving Resistance Required For High-Strain Dynamic Testing
$R_{ndr} = R_{sdd} + R_n$.
R_{sdd} = Skin Friction to be overcome during driving through downdrag zone. (If required)

η = Load Modifier
γ = Load Factor
Q = Service Load
H = Lateral Load
V = Axial Load

Note 1: Pile embedments and required driving resistances for the abutments assume that the embankments are built before the piles are driven so that no downdrag loads are imposed on the piles. Pile embedment is defined as extending down from the base of the MSE section.

Table 5.9 summarizes the maximum horizontal deflection at the bottom of the pile cap for the loads in Table 5.8 above. The minimum embedment is measured from the bottom of the pile cap and represents the minimum depth to which piles must be driven to resist lateral loads created by the bridge superstructure. As discussed in Section 5.1.5 these abutments will be strapped and therefore lateral forces due to earth pressure will be resisted by the MSE.

Table 5.9 - Structure S4 Maximum Horizontal Pile Deflections

Structural Element	Maximum Deflection (in)	Minimum Embedment (ft)
Pier 1	0.21	22.5

The drivability analysis, discussed further in Section 5.5.3, and included in Appendix F, indicates the piles can be safely driven to the required nominal pile resistance without over stressing the pile.

5.4.5 Structure S5: Ramp G1 over SR 7

We recommend open-ended steel pipe piles be used for the support of all substructure units for Ramp G1 over SR 7. Specifically, we recommend 24-inch diameter piles for support of the abutments and 36-inch diameter piles for support of the pier. As discussed in Section 5.1.5, pipe piles are generally preferred as they satisfy the slenderness ratio criteria at tall MSE abutments and provide operational flexibility to splice and cut off piles where needed. The low displacement of open-ended driving will also reduce heave potential and enable the piles to be driven to greater depths and capabilities than would otherwise be possible with displacement piles. Based upon typical wall thicknesses suitable to the anticipated driven capability requirements, pipe sections are more cost efficient per ton of axial resistance than H-piles, supporting calculations are provided in Appendix F. The pile length, thickness, and nominal pile resistance required are summarized in Table 5.9 below. Supporting calculations for the axial capacities of the piles are included in Appendix F. It is anticipated that the piles will be spaced at a minimum of 2.5 times the diameter of the piles, therefore, group effects are not anticipated. All structural steel for this project should have a yield strength of 50-ksi.

It is anticipated that the Ramp A, G1, B and C embankments, the associated wingwalls and Retaining Walls 2, 6, 7, 8, and 16 will be constructed prior to driving the abutment piles. The proposed sequence of construction is further discussed in Section 5.5.1 of this report.

By constructing the embankment, wingwalls, and retaining walls first and imposing an estimated 30 to 60-day quarantine period prior to driving the abutment piles, downdrag forces on the piles should not develop. Otherwise, downdrag forces will need to be incorporated into the pile design and piles may need to be driven deeper than indicated in Table 5.10.

Table 5.10 – Structure S5 Pile Foundation Recommendations

Structural Element	Pile Dia. (in)	Pipe Thickness (in)	Est. Pile Embedment (ft)[1] / Est. Tip Elevation	Service Loads Per Pile (kips)	Strength Loads Per Pile (kips)	Nominal Pile Driving Resistance Required (R_{ndr}) Per Pile (kips)
Abutment A	24	0.5	53 / 15	H: 31 V: 225	H: 65 V: 315	485
Pier 1	36	0.625	54 / 11	H: 45 V: 610	H: 51 V: 735	1131
Pier 2	36	0.625	41 / 24	H: 45 V: 610	H: 51 V: 735	1131
Pier 3	36	0.625	64 / -4	H: 45 V: 610	H: 51 V: 735	1131
Abutment B	24	0.5	51 / 15	H: 31 V: 225	H: 65 V: 315	485

R_n = Nominal (ultimate) Bearing Resistance
R_n= Strength Load / (ϕ_{dyn})
Strength Load = $\Sigma\eta_i\gamma_i Q_i \leq \phi_{dyn}R_n = \phi_{static}R_{n\ static}$
ϕ_{dyn} = 0.65
γ_{dd} = 0.85
ϕ_{static} = 0.35
R_{ndr} = Nominal Pile Driving Resistance Required For High-Strain Dynamic Testing
$R_{ndr} = R_{sdd} + R_{n.}$
R_{sdd} = Skin Friction to be overcome during driving through downdrag zone. (If required)

η = Load Modifier
γ = Load Factor
Q = Service Load
H = Lateral Load
V = Axial Load

Note 1: Pile embedments and required driving resistances for the abutments assume that the embankments are built before the piles are driven so that no downdrag loads are imposed on the piles. Pile embedment is defined as extending down from the base of the MSE section.

Table 5.11 summarizes the maximum horizontal deflection at the bottom of the pile cap for the loads in Table 5.10 above. The minimum embedment is measured from the bottom of the pile cap and represents the minimum depth to which piles must be driven to resist lateral loads created by the bridge superstructure. As discussed in Section 5.1.5 these abutments will be strapped and therefore lateral forces due to earth pressure will be resisted by the MSE.

Table 5.11 - Structure S5 Maximum Horizontal Pile Deflections		
Structural Element	**Maximum Deflection (in)**	**Minimum Embedment (ft)**
Pier 1	0.36	28.5
Pier 2	0.16	23.5
Pier 3	0.16	22.5

The drivability analysis, discussed further in Section 5.5.3, and included in Appendix F, indicates the piles can be safely driven to the required nominal pile resistance without over stressing the pile.

5.4.6 Structure S6: Ramp B over Northbound I-95

It is recommended that the abutments and pier for S6 – Ramp B over Northbound I-95 be supported on driven open-ended 24 and 36-in diameter pipe piles as detailed in Table 5.11. Service loads were used to estimate deflections and the strength loads were used to determine the required nominal pile driving resistance required and tip elevations. The piles will be spaced at a minimum of 2.5 times the diameter of the piles, therefore, group effects are not anticipated. All structural steel for this project should have a yield strength of 50-ksi.

It is anticipated that the Ramp A, B, C, and G1 embankment and the adjacent Retaining Wall RW 5 will be constructed prior to driving the Abutment A pile, and retaining walls RW 14 and RW 15 will be constructed prior to driving the Abutment B piles. The proposed sequence of construction is further discussed in Section 5.5.1 of this report.

By constructing the embankment and retaining walls first and imposing an estimated 30 to 60-day quarantine period prior to driving the abutment piles, downdrag forces on the piles can be minimized. Otherwise, downdrag forces will need to be incorporated into the pile design and piles may need to be driven deeper than indicated in Table 5.12.

Table 5.12 – Structure S6 Pile Foundation Recommendations

Structural Element	Pile Dia. (in)	Pipe Thickness (in)	Est. Pile Length (ft) / Est. Tip Elevation	Service Loads Per Pile (kips)	Strength Loads Per Pile (kips)	Nominal Pile Driving Resistance Required (R_{ndr}) Per Pile (kips)
Abutment A	24	0.5	86 / 35	H: 5 V: 90	V: 131	V: 202
Pier 1	36	0.625	80 / + 6	H: 46 V: 409	V: 516	V: 794
Pier 2	36	0.625	51 / 33	H: 33 V: 159	V: 220	V: 339
Pier 3	36	0.625	76 / + 8	H: 63 V: 394	V: 482	V: 742
Abutment B	24	0.5	73 / 41	H: 8 V: 74	V: 111	V: 171

R_n = Nominal (ultimate) Bearing Resistance
R_n= Strength Load / (ϕ_{dyn})
Strength Load = $\Sigma\eta_i\gamma_i Q_i \leq \phi_{dyn}R_n = \phi_{static}R_{n\ static}$
ϕ_{dyn} = 0.65
γ_{dd} = 0.85
ϕ_{static} = 0.35
R_{ndr} = Nominal Pile Driving Resistance Required For High-Strain Dynamic Testing
$R_{ndr} = R_{sdd} + R_n$.
R_{sdd} = Skin Friction to be overcome during driving through downdrag zone. (If required)

η = Load Modifier
γ = Load Factor
Q = Service Load
H = Lateral Load
V = Axial Load

Note 1: Pile lengths and required driving resistances for the abutments assume that the embankments are built before the piles are driven so that no downdrag loads are imposed on the piles; and the piles stick up 35-ft into the MSE.

Table 5.13 summarizes the maximum horizontal deflection at the bottom of the pile cap for the loads in Table 5.12 above. The distance to the point of fixity is measured from the bottom of the pile cap.

Table 5.13 - Structure S6 Maximum Horizontal Pile Deflections		
Structural Element	**Maximum Deflection (in)**	**Distance to Point of Fixity (ft)**
Abutment A	0.05	17
Pier 1	0.15	24
Pier 2	0.10	23
Pier 3	0.25	24
Abutment B	0.05	17

The pile length for the bridge abutments and piers were evaluated using two software programs: ALLPile version 7.4k and FHWA DRIVEN version 1.2 software, in addition to a hand calculation. The nominal resistance factor (ϕ_{stat}) of a single pile in axial compression (static analysis method) of 0.35 was used for our analysis since the vertical capacity of the piles was evaluated using the α-method.

- ALLPile is a windows based analysis program that provides analysis for vertical pile capacity, compression (with settlement), uplift, and lateral deflection. The vertical analysis is based on approaches and methods recommended in FHWA, AASHTO, and NAVFAC DM-7. The lateral analysis uses the P-Y method evaluated by a finite-difference technique to model the soil-structure interaction.
- FHWA DRIVEN is a windows based program developed by FHWA to analyze the axial capacity of driven piles.

The pile settlement for the abutments and pier was estimated using ALLPile. Based on the subsurface exploration and anticipated structural loads, settlements of the permanent pile supported abutments is anticipated to be less than 0.5-in at all locations.

To evaluate the drivability of these piles we performed a wave equation analysis using GRLWEAP. For 24-inch diameter pipe piles driven to a nominal pile driving resistance of 202-kips, the required termination criteria will be about 30-blows/ft using a 75.4-ft-kip pile driver with a stroke length of 11.4-ft. The maximum compressive stress in the pile will be 35-ksi and the maximum tensile stress will be 21-ksi.

For a 36-inch diameter pipe pile driven to a nominal pile driving resistance of 794 –kips the required termination criteria will be about 119-blows/ft using a 165-ft-kip pile driver with a stroke length of 12.1-ft. The maximum compressive stress in the pile will be 39-ksi and the maximum tensile stress will be 16-ksi.

Based on this analysis the recommended pile can be safely driven to the required nominal pile driving resistance without over stressing the pile.

Groundwater for this structure is anticipated to be near EL 75, approximately 8 to 11-ft below the proposed bottom of footing elevation. Groundwater is not anticipated to be within the excavation limits of the proposed foundations for this structure. Section 5.7 of this report discusses general dewatering and drainage recommendations for this project.

Supporting calculations for this structure are provided in Appendix F.

5.4.7 Structure S7: Ramp R1 over SR 7

It is recommended that the abutments and pier for S7 – Ramp R1 over SR 7 be supported on driven HP 14x73 piles. The pile type, length, and the nominal pile driving resistance required per foundation element are summarized in Table 5.14 below. A center to center pile spacing of 5-ft is recommended for the front two rows of battered piles at 4(H):12(V) and a center to center pile spacing of 10-ft is recommended for the vertical back row of piles. All pier piles will be vertical. All structural steel for this project should have a yield strength of 50-ksi.

The following table summarizes the recommended pile size, estimated length of the piles, the estimated pile tip elevation, the strength loads, the service loads and the nominal driving resistance required, per pile. Service loads were used to estimate deflections and the strength loads were used to determine the required nominal pile driving resistance required and tip elevations. Pile service and strength loads are summarized in Table 2.6 of this report.

Table 5.14 – Structure S7 Abutment Pile Foundation Recommendations

Structural Element	Pile Size	Est. Pile Length (ft) / Est. Tip Elevation	Nominal Pile Driving Resistance Required (R_{ndr}) Per Pile (kips)
Abutment A	HP 14x73	63 / -23	V: 290
Pier	HP 14x73	39 / 3	V: 240
Abutment B	HP 14x73	47 / -8	V: 290

R_n = Nominal (ultimate) Bearing Resistance
R_n= Strength Load / (ϕ_{dyn})
Strength Load = $\Sigma\eta_i\gamma_i Q_i \leq \phi_{dyn}R_n = \phi_{static}R_{n\ static}$
ϕ_{dyn} = 0.65
ϕ_{static} = 0.35
R_{ndr} = Nominal Pile Driving Resistance Required
$R_{ndr} = R_n$.

η = Load Modifier
γ = Load Factor
Q = Service Load
H = Lateral Load
V = Axial Load

Table 5.15 – Structure S7 Lateral Analysis

Structural Element	Point of Fixity (ft) Below Bottom of Footing	Maximum Lateral Deflection (in)
Abutment A	10	<0.25
Pier	3	<0.25
Abutment B	10	<0.25

Point of Fixity – Depth below the top of pile where pile deflection is 0-inches.

The pile lengths for the abutments and pier were evaluated using the same two software programs that were summarized in Section 5.4.6 of this report.

The pile settlement for the abutments and pier was estimated using ALLPile. Based on the subsurface exploration and anticipated structural loads, settlements of the permanent pile supported abutments is anticipated to be about 0.3-in. The anticipated settlement of the piles for the pier is estimated to be about 0.25-in.

A drivability analysis using a wave equation analysis was conducted to assess the feasibility of the recommended piles at Abutment A using GRLWEAP. For an HP 14x73 pile driven to a pile tip elevation of EL -23, the required termination criteria will be about 55-blows/ft at the minimum tip elevation when using a 75.44-ft-kip pile driver with a stroke length of 11.43-ft. It should be

noted that a very dense layer from about EL 0 to -18 must be fully penetrated. It is estimated that the blow counts will be as high as 72-blows/ft through this stratum. The maximum compressive stress in the pile will be 39.57-ksi and the maximum tensile stress will be 0.29-ksi.

To evaluate the drivability of the recommended piles at Abutment B, we performed a wave equation analysis using GRLWEAP. For an HP 14x73 pile driven to a pile tip elevation of EL -8, the required termination criteria will be about 114-blows/ft at the minimum tip elevation when using a 75.44-ft-kip pile driver with a stroke length of 11.43-ft. The maximum compressive stress in the pile will be 42.17-ksi and the maximum tensile stress will be 2.11-ksi.

At the pier location, a drivability analysis of these piles was performed using GRLWEAP. For an HP 14x73 pile driven to a pile tip elevation of EL 3, the required termination criteria will be about 50-blows/ft at the minimum tip elevation when using a 75.44-ft-kip pile driver with a stroke length of 11.43-ft. The maximum compressive stress in the pile will be 37.66-ksi and the maximum tensile stress will be 2.78-ksi.

Based on this analysis, the recommended pile can be safely driven to the required minimum pile tip elevation without overstressing the pile.

Groundwater for this structure is anticipated to be near EL 30 to 45. Groundwater is anticipated to be near the proposed bottom of footing elevation for both Abutments A and B. Groundwater is anticipated to be approximately 10-ft below the proposed pier footing elevation. Dewatering during construction of the pile caps is anticipated. Section 5.7 of this report discusses general dewatering and drainage recommendations for this project.

Supporting calculations for this structure are provided in Appendix F.

5.4.8 Structure S8: SR 1 over Eagle Run

No foundation recommendations are required for this structure since the foundation is already in place. The existing foundation is discussed in Section 2.2.2 of this report.

5.5 SPECIAL CONSIDERATIONS

5.5.1 Sequence of Construction

Based on our understanding of the proposed construction and traffic sequencing, we recommend the following generalized sequence of construction:

- Construct the entire Ramp A, B, C, and G1 embankment
- Construct the entire height of the wingwalls at the following locations
    - Structure S1 – Ramp A over I-95: Abutment A
    - Structure S1 – Ramp A over I-95: Abutment B
    - Structure S2 – Ramp A over SR 7: Abutment A
    - Structure S2 – Ramp A over SR 7: Abutment B
    - Structure S3 – Ramp B over SR 7: Abutment A
    - Structure S3 – Ramp B over SR 7: Abutment B
    - Structure S4 - Ramp C over SR 7: Abutment A
    - Structure S4 - Ramp C over SR 7: Abutment B
    - Structure S5 – Ramp G1 over SR 7: Abutment A
    - Structure S5 – Ramp G1 over SR 7: Abutment B
    - Structure S6 – Ramp B over I-95: Abutment A
- Pile windows or casing should be installed at the proposed pile locations during the wingwall and abutment face wall construction
- Upon completion of the full height of the wingwalls indicated above, impose an estimated 30-day quarantine period prior to driving the abutment piles, placement of parapets or pavements. For wingwalls with staged construction, this requirement can be reduced by the amount of quarantine proposed during construction.
- Monitor the settlement of the wingwalls as described in Section 5.5.4 and at the locations indicated on the plans using settlement plates
- Upon completion of the quarantine period, as judged by the Engineer or other qualified owner's representative, drive piles to the pile driving resistance indicated on the plans, construct parapets, and place pavements.

5.5.2 Downdrag on Piles

Due to the new fill being placed at the abutments and wingwalls the development of downdrag forces is a possibility along abutment piles. If the Contractor adheres to the recommended construction sequence, summarized in Section 5.5.1, it is anticipated that the long-term settlement at the abutments will be lower than the 0.4 to 0.5-inches of movement necessary to mobilize downdrag as specified in FHWA NHI-05-042 and thus downdrag forces will not develop. As exhibited by the test embankment, once the overburden load is placed most of the settlement of the underlying soil appears to occur within two to four weeks and the residual settlement is not anticipated to reach the values established by the FHWA.

As such, we recommend an estimated 30 to 60-day quarantine period for all bridge abutments and MSE retaining walls. We anticipate the majority of the settlement of the bridge abutments

and MSE retaining walls will take place during the quarantine period, which should be verified by the installation and monitoring of settlement plates as discussed in Section 5.5.4. Piles driven after the quarantine period has confirmed the essential completion of settlement will not experience downdrag loads. Additionally the use of lighter weight material such as No. 57 stone or LWEF will reduce total settlement, further reducing the potential for downdrag to develop.

If piles are driven prior to building the embankments, downdrag forces will develop and pile lengths will theoretically increase by as much as 75-percent at the abutments, affecting drivability and the economic comparisons.

It may also be possible to re-tap the piles prior to construction of the abutment pile cap to release the downdrag forces. The disadvantage of this alternative is that multiple mobilizations of the pile driving contractor would be required and therefore additional costs incurred. Another alternative that would reduce downdrag loads, but not fully eliminate them, is to coat the piles with silicone epoxy or other friction reducers. With this method pile lengths would be longer than recommendations in this report, but not to the degree of uncoated piles.

5.5.3 Pile Driveability

A wave equation analysis using GRLWEAP was conducted for representative conditions within the SR 1 interchange. Details for the Wave Equation analysis conducted per structure are summarized in Sections 5.4.1 through 5.4.8 of this report.

Based on our analysis, the recommended piles can be safely driven to the required ultimate load and tip elevation without overstressing the pile. Based on the subsurface conditions, it is anticipated that the installation of the piles will be difficult through the heavily overconsolidated clays encountered within the project limits.

The Contractor is required to evaluate and determine the actual pile driving criteria as summarized in Section 5.5.8 of this report.

5.5.4 Instrumentation Monitoring

A construction monitoring program consisting of settlement survey points, settlement plates, and piezometers should be implemented to monitor the horizontal and vertical displacement of the proposed foundations as construction progresses. Elevation monitoring to measure potential displacement and heave due to driving adjacent piles should be conducted. The survey monitoring points and settlement plates should be located by repeatable survey. Settlement monitoring points should be located at the top of each abutment wingwall. A minimum of two

settlement monitoring points (bench marks) should be located and operational at all times during the monitoring period outside the influence zone of the embankment construction. The location of the bench marks should be determined by the Contractor at the time of construction.

The settlement monitoring points and piezometers should be read, weekly prior to mobilizing construction equipment to the project site, at least weekly during construction and 30-days after completion of the filling operations, and monthly for a period of approximately 6-months. This schedule maybe modified once construction starts and may be relaxed significantly if little movement is noticed. The monitoring points should be established to an accuracy of at least 0.05-inch.

The selection of the monitoring points should be approved by the Engineer. Daily observations should be made and documented to determine if any surficial signs of distress are evident. During construction frequency of the monitoring program may be adjusted by the Contractor with the approval of the Engineer. The instrumentation data should be presented in graphical and tabular formats. The instrumentation data should be provided to the Engineer within 24-hours or one business day after each reading.

5.5.5 Seismic Design Considerations

Based on the results of the subsurface exploration, the laboratory test data, and our review of the AASHTO Site Class Definition from Table 3.10.3.1-1, we recommend a project site classification of D – Stiff Soil. Typically, SPT N-values within the upper 70 to 85-feet of the profile are between 15 and 50-bpf and have measured undrained shear strengths of at least 1 to 2-ksf.

5.5.6 Corrosion Protection

Corrosion potential for this project is based on the corrosion and deterioration criteria set forth in AASHTO, Section 10.7.5. For this project, the following applicable soil corrosion potential criteria from AASHTO, Section 10.7.5, is indicated below.

- pH less than 5.5, or
- Resistivity less than 2,000 ohm-cm, or
- Sulfate concentrations greater than 1,000-ppm.

It is anticipated that corrosion protection will be required for all piles since the average in situ pH is less than 5.5. Only three of the twenty-three samples tested had a pH above 5.5. The

average resistivity is greater than 2,000 ohm-cm. One of the twenty-three samples tested had a resistivity less than 2,000 ohm-cm.

The results of pH and resistivity testing are contained in the Appendix B - Geotechnical Data Reports: Delaware Turnpike Improvements, Report No. 2: I-95/SR 1 Interchanger GDR with Supplemental Laboratory Test Data.

Corrosion protection for the construction of the MSE reinforced zones is discussed in Section 5.6.1.

5.5.7 Temporary Cut Slope and Support of Excavation (SOE)

The design of Temporary Excavation Support Systems will be the Contractor's responsibility; the support of excavation system will need to be designed by a Professional Engineer licensed in the state of Delaware and reviewed by the Engineer.

Construction safety is the responsibility of the Contractor. The actual stability of the excavations should be evaluated by the Contractor in accordance with OSHA regulations. For planning purposes, temporary cut slopes less than 10-ft in height should be maintained at a 1(H):1(V) slopes, otherwise, 2(H):1(V) slopes should be used. It is likely that a support of excavation system will be required to avoid undermining the existing roadways in most areas.

For planning purposes it can be assumed that top-down construction, such as sheet pile or soldier pile and lagging wall, will most likely be needed to support proposed construction in areas where the slopes cannot be safely laid back.

Typically, granular soils with standard penetration N-values lower than 10 will not generally provide sufficient stand-up time and are sensitive to construction vibrations.

5.5.8 High-Strain Dynamic Pile Installation and Monitoring Requirements

High strain dynamic pile testing used with wave equation analysis is recommended to determine the driving criteria. Dynamic verification of capacity is associated with a resistance factor of 0.65, assuming that static load testing will not be performed.

Compatibility of the pile driving equipment with the soil conditions and pile type are essential to achieving the required pile penetration and a satisfactory foundation. To assess drivability, the Contractor should submit a list of proposed driving equipment along with a wave equation

analysis using GRLWEAP or another approved software application to assess the suitability of the driving system. The wave equation analysis should be performed by an engineer licensed in the State of Delaware and submitted to the Engineer for review prior to driving any test piles. Thereafter, the driving criteria should be developed using CAPWAP signal matching results within a refined WEAP run for the test pile to account for the actual conditions of driving.

All pile driving and monitoring requirements for the installation of piles should be in accordance with ***Delaware Department of Transportation: Specifications for Road and Bridge Construction*** – August 2001, Section 619.

We recommend that a minimum of one high strain dynamic test with signal matching be performed for each foundation element. In no case should fewer than two high-strain dynamic tests be performed for each structure. The piles should be driven to the nominal driving resistance required per pile (R_{ndr}) as indicated in this report. The test pile should be installed with the same equipment used for the wave equation analysis and the production piles. The ground surface within a distance of 15-ft of the test pile should be excavated to the depth of the proposed bottom of footing elevation to avoid excessive overburden pressures. Production piles may be used as test piles.

Continuous driving and installation records should be maintained for all driven piles. Production piles should be driven utilizing the same hammer and equipment as the test pile based on the driving criteria established as a result of the test pile program. If the Contractor changes any of the following: hammer, cushion, or driving methods, a revised wave equation should be submitted and a high strain dynamic pile test should be performed.

During pile driving, the depth of embedment, blow counts, driving rate, the driving records, and appropriateness of the bearing materials should be verified. The interior of pipe piles should be cleaned to the depth of the concrete and approved prior to placing concrete.

The sequence of driving piles in groups can affect the pile lengths and driving resistances due to ground densification. We recommend that the piles in the centers of a pile group be driven first. This procedure reduces pile drift and makes driving easier.

Pile penetration displaces the soil laterally and may cause surface heave during installation. The surface heave can cause adjacent piles to move upward. RK&K/URS recommends that level readings be taken periodically on the top of adjacent piles to verify heave is not occurring. Piles that have been heaved should be retapped. Additionally, care should be taken to keep construction equipment as far away as possible from driven piles. Heavy equipment traveling or operating too closely can also displace them laterally.

To the extent possible, the installation of piles should be a continuous operation without termination of driving until the point of acceptable resistance or embedment is achieved. If driving is interrupted, the pile should be driven at least 12-inches after driving is resumed, provided that this will not overstress the piles.

5.5.9 Static Load Test

Static load tests could be used to increase the resistance factors used in the design. One pile load test would be required for each pile size. This would allow the resistance factor (ϕ_{dyn}) to be increased from 0.65 (for high-strain testing) to 0.70 (for one load test) to 0.75 (for two load tests). The increased resistance factor allowed by the load testing program would effectively reduce the estimated length of the piles, but only for the diameter of the pile tested. The effects of plugging will likely vary between the 24-inch and 36-inch diameter piles, so the results of a load test on one pile size might not be applicable to other pile sizes although the static pile load test for one pile size would provide confidence in the pile driving installation and correlations with the High Strain Dynamic Testing on the other pile sizes.

It is anticipated the following number of piles, by pile diameter, will be installed for this project:

24-inch diameter: 64-piles
30-inch diameter: 39-piles
36-inch diameter: 123-piles

Accordingly, we expect that a load testing program conducted in advance of production pile driving would provide some cost savings for overall project requirements. However, it will take time in the schedule and a relatively large area to conduct the pile load tests. Using the criteria in ASTM D 1143, we anticipate an area of approximately 20 x 20-ft (for the load frame only) would be required to conduct a test pile for the 24-inch pile and 30 x 30-ft for the 36-inch diameter pile. This area does not include site access, equipment staging, or equipment for construction of the load test frame. Based on the existing and proposed features for this project, we anticipate only two static load tests could be conducted within the limits of the project, both of these are near proposed abutments where the foundations will consist of 24-inch piles. We anticipate static pile loads tests could be conducted at the following locations:

- Ramp A Station 1234+50 – 24-inch diameter pipe pile. This test would be in the vicinity of the Ramp A over I-95 (S1) Abutment B in addition to Ramp B over I-95 (S6) Abutment A. This test could be conducted in the area between I-95 northbound and the existing SR 7 access ramps.

- Ramp C over SR 7 (S4) Abutment B – 24-inch diameter pipe pile.

Load tests at these locating would be applicable to only the northern portion of the site and would not be representative of the south portion of the site where sand layers were not encountered. These tests would also only be representative of the 24-inch diameter pipe piles so the increased resistance factors can be used only for the 24-inch piles.

Due to lack of testing space, it is not possible to conduct a pile load test in the vicinity of the proposed piers. The proposed pier locations are within the I-95 median and SR 7.

It is anticipated that the savings in pile length for the 24-inch diameter pile will be about half the cost of the two static load tests or about the cost of one static load test. Therefore, due to the space constraints indicated above and the time to conduct a load test with such large loads, it is not recommended that a static load test be performed as it will not provide a significant economical savings for this project.

5.6 WINGWALL RECOMMENDATIONS

Wingwall recommendations are provided below for each structure. These recommendations are based on the TS&L structural drawings and the available subsurface information at the time of this report.

5.6.1 MSE Wingwall General Recommendations

The following sections are general recommendations for construction of the MSE wingwalls. Additional or modifications to the general recommendations for MSE wingwall construction are discussed in each specific wingwall section.

The detailed internal and external stability design of the MSE walls is the Contractor's responsibility and will need to be designed by a Professional Engineer licensed in the State of Delaware and reviewed by the Engineer. For our analysis, we evaluated the global and external stabilities (bearing capacity, sliding, and overturning) and settlements to determine the suitability of MSE construction for this project.

We have performed multiple iterations for each wingwall location to evaluate the global and external stability (bearing capacity, sliding and over turning) to optimize the special treatment techniques recommended for the construction of the MSE walls for this project. Based on these calculations "typical" MSE walls (L/H=0.7 to 0.8) will not provide adequate stability at the

locations listed in Table 5.16. The locations listed in Table 5.16 do not satisfy the minimum design criteria for external (bearing capacity, sliding, and overturning) & global stabilities.

Based on our cost study in Appendix F, it is our opinion that Case 1 is the lowest cost treatment option. Case 2 is slightly more costly and case 3 and 4 are significantly more costly. Based on our cost study in Appendix E, it is our opinion that Case 1 is the lowest cost treatment option. Case 2 is slightly more costly and Case 3 and 4 are significantly more costly. We do not recommend stone columns, geopiers, or vibro piers because a specialty contractor will be required to construct them, constrained working space, and this would be an additional step in the process thus increasing the construction schedule.

Stone columns, geopiers, or vibro piers are formed by excavating a 2 to 3-ft diameter hole about 20 to 40-ft deep and backfilling with compacted crushed stone. The crushed stone in each hole is placed and compacted in thin lifts. This treatment option is a proven technology that has been demonstrated to improve foundation soils and can use relatively small equipment for installation. Typically, a specialty contractor is required for installation. Also, the installation of stone columns, geopiers, or vibro piers below the groundwater elevation is difficult and requires the use of open graded aggregate such as No. 57 stone and perhaps temporary casing to keep the hole open which will increase the construction time and cost. The use of stone columns or other foundation treatments would also increase the production schedule since an additional treatment process must be incorporated into the construction sequence.

Table 5.16 - Wingwalls Requiring Special MSE Treatments		
Structure No.	**Location Description**	**Designer**
S1 Abutment A Face Only	Ramp A over I-95	URS
S1 Abutment B	Ramp A over I-95	URS
S2 Abutment A	Ramp A over SR 7	URS
S2 Abutment B	Ramp A over SR 7	URS
S3 Abutment A	Ramp B over SR 7	URS
S3 Abutment B	Ramp B over SR 7	URS
S4 Abutment A	Ramp C over SR 7	URS
S4 Abutment B	Ramp C over SR 7	URS
S5 Abutment A	Ramp G1 over SR 7	URS
S5 Abutment B	Ramp G1 over SR 7	URS
S6 Abutment B	Ramp B over I-95	RK&K

Special Treatments

To meet the project design criteria (AASHTO LRFD – 2007 with interims) for the locations listed above in Table 5.15, an alternative backfill within the reinforced zone is recommended and in some cases increasing the length of the reinforcement zone and the quality of the retained backfill wedge material is needed.

Table 5.17 summarizes the retaining wall locations where increasing the reinforcement zone strap length and in some cases increasing the quality of the retained backfill wedge material are needed to meet the project design criteria. Table 5.17 summarizes the increased L/H ratio and recommended retained backfill wedge material. These treatments are generally the lowest cost option to provide the required stability and are further discussed in Sections 5.6.2 through 5.6.9 of this report.

Table 5.17 – Wingwalls With Increased Strap Length			
Structure No.	**Location Description**	**L/H**	**Wedge Material**
S1 Abutment A Face Only	Ramp A over I-95	1.0	Select Fill
S1 Abutment B	Ramp A over I-95	1.0	Select Fill
S2 Abutment B	Ramp A over SR 7	0.85	See Figure A-6
S3 Abutment A	Ramp B over SR 7	0.90	See Figure A-6
S4 Abutment B	Ramp C over SR 7	1.0	See Figure A-6
S5 Abutment A	Ramp G1 over SR 7	0.80	See Figure A-6
S5 Abutment B Wingwalls	Ramp G1 over SR 7	1.0	See Figure A-6

Table 5.18 summarizes locations where a combination of LWEF and select fill or No. 57 is recommended to meet the project design criteria. This concept is illustrated in Figure A-6 in Appendix A of this report. In most areas using a combination of LWEF and select fill or No. 57 stone in both the reinforcement zone and retained backfill wedge is the lowest cost option that will provide the required stability and are further discussed in Sections 5.6.2 through 5.6.9 of this report. The amount of LWEF for each location has been minimized as much as possible. Figure A-5 in Appendix A of this report provides a typical plan view illustrating the overlap of the reinforcement zones for the abutment wingwalls.

Table 5.18 – Wingwalls Requiring LWEF and Select Fill				
Structure No.	**Location Description**	**L/H**	**Height LWEF : No. 57 Stone (ft)[1]**	**Wedge Material**
S2 Abutment A	Ramp A over SR 7	0.7	17:27	See Figure A-6
S3 Abutment B	Ramp B over SR 7	0.7	14:24	See Figure A-6
S4 Abutment B	Ramp C over SR 7	1.0	24.5:0	See Figure A-6
S5 Abutment A	Ramp G1 over SR 7	0.8	19:24	See Figure A-6
S6 Abutment B	Ramp B over I-95	1.0	14:21[2]	See Figure A-6
Note 1: See Figure A-5 Note 2: Select Fill instead of No. 57 Stone above LWEF				

Table 5.19 summarizes the locations where staged construction is required.

Table 5.19 – Retaining Walls Requiring Staged Construction

Structure No.	Location Description	Maximum Height (ft)	Height Stage 1 (ft)
S1 Abutment A	Ramp A over I-95	33	22
S1 Abutment B	Ramp A over I-95	37	32
S2 Abutment A	Ramp A over SR 7	44	38
S2 Abutment B	Ramp A over SR 7	37	30
S3 Abutment A	Ramp B over SR 7	31	27
S3 Abutment B	Ramp B over SR 7	38	35
S4 Abutment A	Ramp C over SR 7	30	25
S5 Abutment A	Ramp G1 over SR 7	43	39
S5 Abutment B	Ramp G1 over SR 7	32	29
Note: 5-ft intervals after minimum 7-day quarantine period			

Bearing Resistance

The nominal bearing resistance, Meyerhof stress, and eccentricity (e) were estimated using a software program entitled MSEW, a design and analysis software for mechanically stabilized earth walls, and with manual hand calculations. The factored bearing resistance was estimated using the following equation:

$$q_r = \phi_b q_n$$

Where: q_r – Factored Bearing Resistance

ϕ_b – Bearing Resistance Factor from AASHTO (Table 11.5.6-1)

ϕ_b – MSE Walls = 0.65

q_n – Nominal Bearing Resistance

Proper construction procedures should be used to maintain the bearing qualities of the footing excavations. Foundations and excavations should be protected from the detrimental effects of precipitation, seepage, surface run off, or frost. Before placing the leveling pad or new compacted fill, the subgrade should be reviewed and tested by a professional Geotechnical Engineer, licensed in the State of Delaware. In the field, if the material is judged unsuitable, it should be undercut to firm material. The undercut area should be backfilled and compacted with crusher run, dense graded aggregate or lean (2,000-psi) concrete in accordance with Section 207 – Excavation and Backfilling for Structures, Delaware Department of Transportation;

Specifications for Road and Bridge Construction, dated August 2001 with supplements. Prior to placing new fill, the exposed ground surface should be proof-rolled to locate any soft spots requiring additional undercutting in accordance with Section 207 – Excavation and Backfilling for Structures, Delaware Department of Transportation; ***Specifications for Road and Bridge Construction***, dated August 2001 with supplements. Undercut areas should be backfilled with a graded aggregate base such as CR-6.

Corrosion Protection

The reinforcement for the MSE wall will be embedded in select fill, LWEF, or No. 57 stone and not in situ materials. As indicated in FHWA NHI-00-044, the retaining wall backfill material should meet certain electrochemical properties. Table 5.20 provides details regarding the acceptable limits of electrochemical properties and the corresponding test method for the reinforcement backfill.

Table 5.20 – Acceptable Limits of Electrochemical Properties for Backfill

Property	Criteria	Test Method
Resistivity	Greater than 3,000 ohm-cm	AASHTO T-288-91
pH	5 to 10	AASHTO T-289-91
Chlorides	Less than 100 PPM	AASHTO T-291-91
Sulfates	Less than 200 PPM	AASHTO T-290-91
Organic Content	1% max	AASHTO T-267-86

Select Fill For Reinforcement and Retained Zones

Except in areas requiring No. 57 stone or LWEF in the reinforcement zones, the fill should consist of granular soil meeting the requirements described below and in Table 5.5. All backfill material used in the reinforced zone should be free of organics, and should conform to the following gradation limits as determined by AASHTO T-27.

U.S. Sieve Size	Percent Passing
100 mm (4-inches)	100
No. 40 mesh sieve	0 - 60
No. 200 mesh sieve	0 – 15

The backfill should conform to the following additional requirements.

- The Plasticity Index (PI) as determined by AASHTO T-90 should not exceed 6.
- The material should exhibit an angle of internal friction of not less than 34-deg, as determined by the standard direct shear test AASHTO T-236 on the portion finer that the

No. 10 sieve, using a sample of the material compacted to 92 percent of the AASHTO T-180. No testing is required for backfills with 80 percent of the sizes greater that 3/4 – inches.

- Soundness – The materials should be substantially free of shale or other soft, poor-durability particles. The material shall have a magnesium sulfate soundness loss of less than 30 percent after four cycles, measured in accordance with AASHTO T-104, or a sodium sulfate loss of less than 15 percent after five cycles determined in accordance with AASHTO T-104.

Light weight walk behind compaction equipment may be required near the wall face to attain the proper degree of compaction without overstressing connections or the facing panels. Extra care should be given to avoid damaging the wall due to heavier loads produced by larger construction equipment.

Onsite soil (Type F borrow) may be used to construct the remainder of the embankment behind the MSE except where noted that the wedge material behind the wingwall should be as recommended in Table 5.16. This should be placed and compacted in accordance with in accordance Section 202 – Excavation and Embankment, Delaware Department of Transportation; ***Specifications for Road and Bridge Construction***, dated August 2001 with supplements.

Lightweight Engineered Fill (LWEF) For Reinforcement and Retained Zones

For this project, LWEF is defined as a self-leveling and self-compacting, cementitious material with a maximum unit weight of no more than 40-pcf, and an unconfined compressive strength of 120-psi or less at 28-days. The LWEF should be placed in two to four foot lifts coincident with strap levels on the wall. A minimum unconfined compressive strength of at least 20-psi is required for placement of the subsequent lift of LWEF. The MSE facing panels may be used as a form for placement of the LWEF if they are properly sealed during placement of the LWEF. The sealing for placement of the LWEF should not compromise the hydrostatic potential of the wall.

No. 57 Stone For Reinforcement and Retained Zones

No. 57 stone, in accordance Section 813 – Grading Requirements Minimum and Maximum Percent Passing, Delaware Department of Transportation; ***Specifications for Road and Bridge Construction***, dated August 2001 with supplements, were specified, should be placed within the reinforcement and retained zones.

Surface and Subsurface Drainage Requirements

It is likely that during excavation trapped water within the existing non-plastic materials will be encountered. It is anticipated that minor dewatering during construction will be required using sumps and trenches.

The MSE reinforcement fill will consist of free draining materials and will not require a blanket drain or chimney drain. However, we do recommend a face drain at least 12-inches thick consisting of No. 57 stone. The reinforcement zone below the permanent groundwater elevation should be No. 57 stone. Based on the proposed gradation of the specified granular backfill, water will seep through the retaining wall face. A face drain is not necessary in areas were LWEF will be placed.

Erosion Control

Exposed slopes should be protected from erosion in accordance with local sediment and erosion control regulations and as described in the Erosion and Sediment Control Plans. Runoff onto new construction or other disturbed areas should be diverted until vegetation has been firmly established.

Reinforcement Length and Global Stability

A resistance factor of 0.65, approximately a minimum Factor of Safety (FS) of 1.5, was used to evaluate global stability. The reinforcement length for all wingwalls should be a minimum of 0.7H, where H is the height of the retaining wall from the top of the leveling pad to the ground surface above the wall, unless otherwise noted below. The minimum length of reinforcement regardless of the wall height should be 8-ft.

The global stability was evaluated using the following two software programs:

- GSTABL7 with STEDwin is a slope stability analysis program that evaluates the stability of slopes using limit equilibrium methods. All slope stability models with this program are deterministic. For this project, all slope stabilities were evaluated using the Modified Bishop Method.

- Slope/W is a slope stability analysis program that evaluates the stability of slopes using limit equilibrium methods. The stability of a slope can be evaluated using either deterministic or probabilistic input parameters. For this project, the Morgenstern-Price method was used. Slope/W was used in areas of symmetrical construction since GSTABL7 will only calculate the stability in one direction.

Back to Back Wall Construction

Back to back retaining wall construction is discussed specific to each wall in Sections 5.6.2 through 5.6.8. Back to back retaining walls should be designed as independent structures with the active thrust reduced using the procedures outlined in FHWA Publication No. FHWA-NHI-00-043: Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guideline.

Back to back retaining walls can be designed independently if the distance between the wall reinforcement zones is greater than 0.5H, where H is the exposed height of the retaining wall. For this case, there is no overlapping of the back to back retaining wall retained wedge zone.

For cases where the reinforcement zones overlap by more than 0.3H, no active earth thrust from the backfill needs to be considered for external stability.

For intermediate cases, the active earth thrust maybe linearly interpolated from the full active case to zero.

Back to back retaining wall construction with reinforcement zones or retained wedges that will overlap is anticipated for Structure S1 – Ramp A over I-95 Abutment A, Structure S2 – Ramp A over SR 7, Abutment B, Structure S4 – Ramp C over SR 7, Abutments A and B, Structure S5 – Ramp G1 over SR 7, Abutment B, and Structure S6 - Ramp B, Abutments A and B wingwalls.

The use of a single reinforcement (common) to connect both wall facings in this area is not recommended. By connecting the wall facings with a common reinforcement method changes the strain pattern of the structure and will result in higher reinforcement tensions, thus the recommendations provided in this report would not be applicable.

5.6.2 Structure S1: Ramp A over I-95 – Wingwalls

Based on our review of the existing subsurface conditions and laboratory testing received to date, we recommend that the MSE wingwalls be constructed prior to the installation of the piles for the Structure S1 – Ramp A over I-95 abutments in order to avoid development of downdrag loads on the abutment piles. After the completion of settlement, bridge foundation piles should be driven through sleeves or casing extended through the MSE section.

Table 5.21 summarizes the soil parameters to be used by the Contractor during design of the MSE wall for this structure. Where the groundwater level was one footing width (strap length) below the leveling pad, the wet unit weight of water was used in our analysis of external stability in MSEW. In cases were groundwater is within one footing width of the leveling pad elevation,

the adjusted the unit weight was used. The design unit weight indicated in Table 5.21 has been adjusted due to the presence of shallow groundwater.

Table 5.21 – Structure S1 Soil Parameters For Contractor's Design

Strata	Total Unit Weight – γ (pcf)	Drained Angle of Friction – ϕ (deg)	Undrained Shear - S_u (psf)
Select Fill	125	34	-
Foundation Soils Abutment A	70	27	1,000
Foundation Soil Abutment B	70	28	1,000

Groundwater for this structure is anticipated to be near EL 75. Groundwater is not anticipated to be within the excavation limits of the proposed foundations for this structure. Section 5.7 of this report discusses general dewatering and drainage recommendations for this project.

Abutment A

It is recommended that the walls at Abutment A be constructed with select fill in both the reinforced and retained zones. Based on the face to face distance of 54-feet for wingwalls adjacent to Abutment A, it is anticipated that the retained wedge zones will overlap. Our evaluation of the external stability of these walls assumes that active earth thrust from the backfill is reduced in accordance with the FHWA reference noted above. Our evaluation of the retaining wall at the abutment face, however, assumes a full active wedge zone at the rear of the retaining wall.

Due to the maximum 33-foot design height of these walls relative to the undrained shear strength of the underlying soil, the rate of construction will need to be staged to control the risk of general shear failure. As indicated by the calculations included in Appendix F, the MSE abutment and associated wingwalls can be constructed to a maximum height of 22-feet in a first stage of loading. Once this height is reached the underlying soil must have sufficient time to consolidate and allow any excess pore water pressures to dissipate. Following the initial stage of construction, it is our recommendation that the walls can be constructed in 5-foot intervals with a minimum 7-day quarantine period between intervals. Settlement plates should be monitored on a daily basis and after 7 days the engineer or a qualified owner's representative should observe the data to verify settlement has ceased and give permission for the Contractor to continue with filling operations.

The minimum reinforcement length for the wingwalls is 23.5-ft and for the abutment face is 33-feet. The length at the abutment wingwalls was selected based on the minimum 0.70 L/H ratio established by AASHTO. The length at the abutment face was increased to satisfy minimum AASHTO requirements against general shear or global stability failures.

Factored bearing resistances calculated for the wall at various stages of construction and after construction are included in Appendix F. Based on a resistance factor of 0.65 the factored bearing resistance for the initial stages of construction, prior to any fill placement, is 4.0-ksf, which should be verified prior to construction.

It is anticipated that the total settlement for the Abutment A wingwalls will be about 3.9-inches and for the Abutment A face retaining wall will be about 4.9-inches. Vertical control joints should be provided to accommodate differential settlement. Based on the results of settlement monitoring plates at the test embankment, the settlement should occur within 2 to 4-weeks after fill placement to final grade. Settlements should be monitored to verify movement has substantially ceased prior to driving piles and construction of parapets and pavements.

Global stability analyses, included in Appendix F, were also performed which satisfied the aforementioned criteria.

Abutment B

It is recommended that the walls at Abutment B be constructed with select fill in both the reinforced and retained zones. Only one wingwall is proposed at Abutment B, as the east side of Ramp A is adjoined to the Ramp B embankment. Due to the maximum 37-foot design height of these walls relative to the undrained shear strength of the underlying soil, the rate of construction will need to be staged to control the risk of general shear failure. As indicated by the calculations included in Appendix F, the MSE abutment and associated wingwalls can be constructed to a maximum height of 32-feet in a first stage of loading. Once this height is reached the underlying soil must have sufficient time to consolidate and allow any excess pore water pressures to dissipate. Following the initial stage of construction, it is our recommendation that the walls can be constructed in 5-foot intervals with a minimum 7-day quarantine period between intervals. Settlement plates should be monitored on a daily basis and after 7 days the engineer or a qualified owner's representative should observe the data to verify settlement has ceased and give permission for the Contractor to continue with filling operations.

The minimum reinforcement length for the wingwall and the abutment face is 38-ft. The length was increased to satisfy minimum AASHTO requirements against general shear or global stability failures. Factored bearing resistances calculated for the wall at various stages of

construction and after construction are included in Appendix F. Based on a resistance factor of 0.65 the factored bearing resistance for the initial stages of construction, prior to any fill placement, is 6.0-ksf, which should be verified prior to construction.

It is anticipated that the total settlement for the Abutment B wingwall and face retaining wall will be about 3.7-inches, see Appendix F. Vertical control joints should be provided to accommodate differential settlement. Based on the results of settlement monitoring plates at the test embankment, the settlement should occur within 2 to 4-weeks after fill placement to final grade. Settlements should be monitored to verify movement has substantially ceased prior to driving piles and construction of parapets and pavements.

Global stability analyses, included in Appendix F, were also performed which satisfied the aforementioned criteria.

5.6.3 Structure S2: Ramp A over SR 7 – Wingwalls

Based on our review of the existing subsurface conditions and laboratory testing received to date, we recommend that the MSE wingwalls be constructed prior to the installation of the piles for the Structure S2 – Ramp A over SR 7 abutments in order to avoid development of downdrag loads on the abutment piles. After the completion of settlement, bridge foundation piles should be driven through sleeves or casing extended through the MSE section.

Table 5.22 summarizes the soil parameters to be used by the Contractor during design of the MSE wall for this structure.

Table 5.22 – Structure S2 Soil Parameters For Contractor's Design

Strata	Total Unit Weight – γ (pcf)	Long-Term (Consolidated-Drained) Angle of Friction – ϕ (deg)	Short-Term (Unconsolidated-Undrained) Shear - S_u (psf)
Select Fill	125	34	-
LWEF	40	38	-
No. 57 Stone	105	38	-
Foundation Soils Abutment A	120	27	1,250
Foundation Soil Abutment B	120	28	1,400

Perched groundwater for this structure is anticipated to be near EL 70 at Abutment A which may fall within the excavation limits and EL 45 at Abutment B which is not expected to be within the excavation limits. Section 5.7 of this report discusses general dewatering and drainage recommendations for this project.

Abutment A

It is recommended that the walls at Abutment A be constructed with LWEF in both the reinforced and retained zones to a height of 17-feet above the leveling pad. Above the LWEF for the remaining height of the MSE wall should be constructed using No. 57 stone. A transition due to varying heights of light-weight materials will be required when transitioning from the Abutment A walls to RW 3 and Abutment A of S5-Ramp G1 over SR 7. RW 3 is discussed in a separate report and S5 is discussed in Section 5.6.6 of this report. Wall construction using select fill was investigated at these walls but general shear and global stability analyses could not attain acceptable levels of stability. During these analyses the length of reinforcement was increased but did not provide sufficient enough improvement.

Due to the maximum 44-foot design height of these walls relative to the undrained shear strength of the underlying soil, the rate of construction will need to be staged to control the risk of general shear failure. As indicated by the calculations included in Appendix F, the MSE abutment and associated wingwalls can be constructed to a maximum height of 38-feet in a first stage of loading. Once this height is reached the underlying soil must have sufficient time to consolidate and allow any excess pore water pressures to dissipate. Following the initial stage of construction, it is our recommendation that the walls can be constructed in 5-foot intervals with a minimum 7-day quarantine period between intervals. Settlement plates should be monitored on a daily basis and after 7 days the engineer or a qualified owner's representative should observe the data to verify settlement has ceased and give permission for the Contractor to continue with filling operations.

The minimum reinforcement length for the walls is 31-ft and was selected based on the minimum 0.70 L/H ratio established by AASHTO. Factored bearing resistances calculated for the wall at various stages of construction and after construction are included in Appendix F. Based on a resistance factor of 0.65 the factored bearing resistance for the initial stages of construction, prior to any fill placement, is 5.0-ksf, which should be verified prior to construction.

It is anticipated that the total settlement for the Abutment A walls will be about 3.6-inches. Vertical control joints should be provided to accommodate differential settlement. Based on the results of settlement monitoring plates at the test embankment, the settlement should occur within 2 to 4-weeks after fill placement to final grade. Settlements should be monitored to verify

movement has substantially ceased prior to driving piles and construction of parapets and pavements.

Global stability analyses, included in Appendix F, were also performed which satisfied the aforementioned criteria.

Abutment B

It is recommended that the walls at Abutment B be constructed with select fill in both the reinforced and retained zones. Based on the face to face distance of 42-feet for wingwalls adjacent to Abutment B, it is anticipated that the reinforced zones will overlap. Our evaluation of the external stability of these walls assumes that active earth thrust from the backfill is reduced in accordance with the FHWA reference noted above. Our evaluation of the retaining wall at the abutment face, however, assumes a full active wedge zone at the rear of the retaining wall.

Due to the maximum 37-foot design height of these walls relative to the undrained shear strength of the underlying soil, the rate of construction will need to be staged to control the risk of general shear failure. As indicated by the calculations included in Appendix F, the MSE abutment and associated wingwalls can be constructed to a maximum height of 30-feet in a first stage of loading. Once this height is reached the underlying soil must have sufficient time to consolidate and allow any excess pore water pressures to dissipate. Following the initial stage of construction, it is our recommendation that the walls can be constructed in 5-foot intervals with a minimum 7-day quarantine period between intervals. Settlement plates should be monitored on a daily basis and after 7 days the engineer or a qualified owner's representative should observe the data to verify settlement has ceased and give permission for the Contractor to continue with filling operations.

The minimum reinforcement length for the wingwalls is 26-ft and for the abutment face is 33-feet. The length at the abutment wingwalls was selected based on the minimum 0.70 L/H ratio established by AASHTO. The length at the abutment face was increased to satisfy minimum AASHTO requirements against general shear or global stability failures.

Factored bearing resistances calculated for the wall at various stages of construction and after construction are included in Appendix F. Based on a resistance factor of 0.65 the factored bearing resistance for the initial stages of construction, prior to any fill placement, is 5.1-ksf, which should be verified prior to construction.

It is anticipated that the total settlement for the Abutment B wingwalls will be about 3.1-inches and for the Abutment B face retaining wall will be about 4.0-inches, see Appendix F. Vertical control joints should be provided to accommodate differential settlement. Based on the results

of settlement monitoring plates at the test embankment, the settlement should occur within 2 to 4-weeks after fill placement to final grade. Settlements should be monitored to verify movement has substantially ceased prior to driving piles and construction of parapets and pavements.

Global stability analyses, included in Appendix F, were also performed which satisfied the aforementioned criteria.

5.6.4 Structure S3: Ramp B over SR 7 – Wingwalls

Based on our review of the existing subsurface conditions and laboratory testing received to date, we recommend that the MSE wingwalls be constructed prior to the installation of the piles for the Structure S3 – Ramp B over SR 7 abutments in order to avoid development of downdrag loads on the abutment piles. After the completion of settlement, bridge foundation piles should be driven through sleeves or casing extended through the MSE section.

Table 5.23 summarizes the soil parameters to be used by the Contractor during design of the MSE wall for this structure.

Table 5.23 – Structure S3 Soil Parameters For Contractor's Design

Strata	Total Unit Weight – γ (pcf)	Long-Term (Consolidated-Drained) Angle of Friction – ϕ (deg)	Short-Term (Unconsolidated-Undrained) Shear - S_u (psf)
Select Fill	125	34	-
LWEF	40	38	-
No. 57 Stone	105	38	-
Foundation Soils Abutment A	120	27	1,400
Foundation Soil Abutment B	120	28	1,250

Perched groundwater for this structure is anticipated to be near EL 70 at Abutment A which may fall within the excavation limits and EL 45 at Abutment B which is not expected to be within the excavation limits. Section 5.7 of this report discusses general dewatering and drainage recommendations for this project.

Abutment A

It is recommended that the walls at Abutment A be constructed with select fill in both the reinforced and retained zones. Due to the maximum 31-foot design height of these walls relative to the undrained shear strength of the underlying soil, the rate of construction will need to be staged to control the risk of general shear failure. As indicated by the calculations included in Appendix F, the MSE abutment and associated wingwalls can be constructed to a maximum height of 27-feet in a first stage of loading. Once this height is reached the underlying soil must have sufficient time to consolidate and allow any excess pore water pressures to dissipate. Following the initial stage of construction, it is our recommendation that the walls can be constructed in 5-foot intervals with a minimum 7-day quarantine period between intervals. Settlement plates should be monitored on a daily basis and after 7 days the engineer or a qualified owner's representative should observe the data to verify settlement has ceased and give permission for the Contractor to continue with filling operations.

The minimum reinforcement length for the walls is 26-ft. The length was increased to satisfy minimum AASHTO requirements against general shear or global stability failures. Factored bearing resistances calculated for the wall at various stages of construction and after construction are included in Appendix F. Based on a resistance factor of 0.65 the factored bearing resistance for the initial stages of construction, prior to any fill placement, is 5.5-ksf, which should be verified prior to construction.

It is anticipated that the total settlement for the Abutment A walls will be about 3.9-inches, see Appendix F. Vertical control joints should be provided to accommodate differential settlement. Based on the results of settlement monitoring plates at the test embankment, the settlement should occur within 2 to 4-weeks after fill placement to final grade. Settlements should be monitored to verify movement has substantially ceased prior to driving piles and construction of parapets and pavements.

Global stability analyses, included in Appendix F, were also performed which satisfied the aforementioned criteria.

Abutment B

It is recommended that the walls at Abutment B be constructed with LWEF in both the reinforced and retained zones to a height of 14-feet above the leveling pad. Above the LWEF for the remaining height of the MSE wall should be constructed using No. 57 stone. A transition due to varying heights of light-weight materials will be required when transitioning from RW 3, which is discussed in a separate report. Wall construction using select fill was investigated at these walls but general shear and global stability analyses could not attain acceptable levels of

stability. During these analyses the length of reinforcement was increased but did not provide sufficient enough improvement.

Due to the maximum 38-foot design height of these walls relative to the undrained shear strength of the underlying soil, the rate of construction will need to be staged to control the risk of general shear failure. As indicated by the calculations included in Appendix F, the MSE abutment and associated wingwalls can be constructed to a maximum height of 35-feet in a first stage of loading. Once this height is reached the underlying soil must have sufficient time to consolidate and allow any excess pore water pressures to dissipate. Following the initial stage of construction, it is our recommendation that the walls can be constructed in 5-foot intervals with a minimum 7-day quarantine period between intervals. Settlement plates should be monitored on a daily basis and after 7 days the engineer or a qualified owner's representative should observe the data to verify settlement has ceased and give permission for the Contractor to continue with filling operations.

The minimum reinforcement length for the walls is 27-ft and was selected based on the minimum 0.70 L/H ratio established by AASHTO. Factored bearing resistances calculated for the wall at various stages of construction and after construction are included in Appendix F. Based on a resistance factor of 0.65 the factored bearing resistance for the initial stages of construction, prior to any fill placement, is 5.5-ksf, which should be verified prior to construction.

It is anticipated that the total settlement for the Abutment B walls will be about 3.1-inches. Vertical control joints should be provided to accommodate differential settlement. Based on the results of settlement monitoring plates at the test embankment, the settlement should occur within 2 to 4-weeks after fill placement to final grade. Settlements should be monitored to verify movement has substantially ceased prior to driving piles and construction of parapets and pavements.

Global stability analyses, included in Appendix F, were also performed which satisfied the aforementioned criteria.

5.6.5 Structure S4: Ramp C over SR 7 – Wingwalls

Based on our review of the existing subsurface conditions and laboratory testing received to date, we recommend that the MSE wingwalls be constructed prior to the installation of the piles for the Structure S4 – Ramp C over SR 7 abutments in order to avoid development of downdrag loads on the abutment piles. After the completion of settlement, bridge foundation piles should be driven through sleeves or casing extended through the MSE section.

Table 5.24 summarizes the soil parameters to be used by the Contractor during design of the MSE wall for this structure.

Table 5.24 – Structure S4 Soil Parameters For Contractor's Design

Strata	Total Unit Weight – γ (pcf)	Long-Term (Consolidated-Drained) Angle of Friction – ϕ (deg)	Short-Term (Unconsolidated-Undrained) Shear - S_u (psf)
Select Fill	125	34	-
LWEF	40	38	-
No. 57 Stone	105	38	-
Foundation Soils Abutment A	120	27	1,250
Foundation Soil Abutment B	120	28	1,250

Perched groundwater for this structure is anticipated to be near EL 65 which may be within the excavation limits. Section 5.7 of this report discusses general dewatering and drainage recommendations for this project.

Abutment A

It is recommended that the walls at Abutment A be constructed with select fill in both the reinforced and retained zones. Based on the face to face distance of 29-feet for wingwalls adjacent to Abutment A, it is anticipated that the reinforced zones will overlap. Our evaluation of the external stability of these walls assumes that active earth thrust from the backfill is reduced in accordance with the FHWA reference noted above. Our evaluation of the retaining wall at the abutment face, however, assumes a full active wedge zone at the rear of the retaining wall.

Due to the maximum 30-foot design height of these walls relative to the undrained shear strength of the underlying soil, the rate of construction will need to be staged to control the risk of general shear failure. As indicated by the calculations included in Appendix F, the MSE abutment and associated wingwalls can be constructed to a maximum height of 25-feet in a first stage of loading. Once this height is reached the underlying soil must have sufficient time to consolidate and allow any excess pore water pressures to dissipate. Following the initial stage of construction, it is our recommendation that the walls can be constructed in 5-foot intervals with a minimum 7-day quarantine period between intervals. Settlement plates should be monitored on a daily basis and after 7 days the engineer or a qualified owner's representative should observe the data to verify settlement has ceased and give permission for the Contractor to continue with filling operations.

The minimum reinforcement length for the wingwalls is 21-ft and for the abutment face is 24-feet. The length at the abutment wingwalls was selected based on the minimum 0.70 L/H ratio established by AASHTO. The length at the abutment face was increased to satisfy minimum AASHTO requirements against general shear or global stability failures.

Factored bearing resistances calculated for the wall at various stages of construction and after construction are included in Appendix F. Based on a resistance factor of 0.65 the factored bearing resistance for the initial stages of construction, prior to any fill placement, is 5.0-ksf, which should be verified prior to construction.

It is anticipated that the total settlement for the Abutment A wingwalls will be about 3.3-inches and for the Abutment B face retaining wall will be about 4.1-inches, see Appendix F. Vertical control joints should be provided to accommodate differential settlement. Based on the results of settlement monitoring plates at the test embankment, the settlement should occur within 2 to 4-weeks after fill placement to final grade. Settlements should be monitored to verify movement has substantially ceased prior to driving piles and construction of parapets and pavements.

Global stability analyses, included in Appendix F, were also performed which satisfied the aforementioned criteria.

Abutment B

It is recommended that the walls at Abutment B be constructed with LWEF to their full height. The wingwall and face wall at Abutment B adjoin RW 16 and as such the same design cross section was selected. This will reduce the potential for differential settlement between the walls. The bearing elevation of the east wingwall for Abutment B, RW 2, and RW 6 should be lowered outside the zone of influence for the Abutment B west wingwall.

The minimum reinforcement length for the walls is 24.5-ft and was controlled by the design cross section of RW 16. Factored bearing resistances calculated for the wall at various stages of construction and after construction are included in Appendix F. Based on a resistance factor of 0.65 the factored bearing resistance for the initial stages of construction, prior to any fill placement, is 5.0-ksf, which should be verified prior to construction.

It is anticipated that the total settlement for the Abutment B walls will be about 1.3-inches. Vertical control joints should be provided to accommodate differential settlement. Based on the results of settlement monitoring plates at the test embankment, the settlement should occur within 2 to 4-weeks after fill placement to final grade. Settlements should be monitored to verify

movement has substantially ceased prior to driving piles and construction of parapets and pavements.

Global stability analyses, included in Appendix F, were also performed which satisfied the aforementioned criteria.

5.6.6 Structure S5: Ramp G1 over SR 7 – Wingwalls

The MSE walls for the north approach to Ramp G1 over SR 7 are above the existing slope. Depending on the height, the base elevation of the MSE wall may need to be lowered to provide an appropriate bearing capacity for the wall, as well as to satisfy the AASHTO requirements for a minimum 4-ft bench in front of the wall.

Based on our review of the existing subsurface conditions and laboratory testing received to date, we recommend that the MSE wingwalls be constructed prior to the installation of the piles for the Structure S5 – Ramp G1 over SR 7 abutments in order to avoid development of downdrag loads on the abutment piles. After the completion of settlement, bridge foundation piles should be driven through sleeves or casing extended through the MSE section.

Table 5.25 summarizes the soil parameters to be used by the Contractor during design of the MSE wall for this structure.

Table 5.25 – Structure S5 Soil Parameters For Contractor's Design

Strata	Total Unit Weight – γ (pcf)	Long-Term (Consolidated-Drained) Angle of Friction – ϕ (deg)	Short-Term (Unconsolidated-Undrained) Shear - S_u (psf)
Select Fill	125	34	-
LWEF	40	38	-
No. 57 Stone	105	38	-
Foundation Soils Abutment A	120	27	1,250
Foundation Soil Abutment B	120	28	1,400

Perched groundwater for this structure is anticipated to be near EL 65 which may fall within the excavation limits. Section 5.7 of this report discusses general dewatering and drainage recommendations for this project.

Abutment A

It is recommended that the walls at Abutment A be constructed with LWEF in both the reinforced and retained zones to a height of 19-feet above the leveling pad. Above the LWEF for the remaining height of the MSE wall should be constructed using No. 57 stone. A transition due to varying heights of light-weight materials will be required when transitioning from the Abutment A walls to RW 2 and Abutment A of S2-Ramp A over SR 7. RW 2 is discussed in a separate report and S2 is discussed in Section 5.6.3 of this report. Wall construction using select fill was investigated at these walls but general shear and global stability analyses could not attain acceptable levels of stability. During these analyses the length of reinforcement was increased but did not provide sufficient enough improvement.

Due to the maximum 43-foot design height of these walls relative to the undrained shear strength of the underlying soil, the rate of construction will need to be staged to control the risk of general shear failure. As indicated by the calculations included in Appendix F, the MSE abutment and associated wingwalls can be constructed to a maximum height of 39-feet in a first stage of loading. Once this height is reached the underlying soil must have sufficient time to consolidate and allow any excess pore water pressures to dissipate. Following the initial stage of construction, it is our recommendation that the walls can be constructed in 5-foot intervals with a minimum 7-day quarantine period between intervals. Settlement plates should be monitored on a daily basis and after 7 days the engineer or a qualified owner's representative should observe the data to verify settlement has ceased and give permission for the Contractor to continue with filling operations.

The minimum reinforcement length for the walls is 33-ft and was controlled by the design cross section of RW 2. Factored bearing resistances calculated for the wall at various stages of construction and after construction are included in Appendix F. Based on a resistance factor of 0.65 the factored bearing resistance for the initial stages of construction, prior to any fill placement, is 5.3-ksf, which should be verified prior to construction.

It is anticipated that the total settlement for the Abutment A walls will be about 3.5-inches. Vertical control joints should be provided to accommodate differential settlement. Based on the results of settlement monitoring plates at the test embankment, the settlement should occur within 2 to 4-weeks after fill placement to final grade. Settlements should be monitored to verify movement has substantially ceased prior to driving piles and construction of parapets and pavements.

Global stability analyses, included in Appendix F, were also performed which satisfied the aforementioned criteria.

Abutment B

It is recommended that the walls at Abutment B be constructed with select fill in both the reinforced and retained zones. Based on the face to face distance of 56-feet for wingwalls adjacent to Abutment B, it is anticipated that the retained wedge zones will overlap. Our evaluation of the external stability of these walls assumes that active earth thrust from the backfill is reduced in accordance with the FHWA reference noted above. Our evaluation of the retaining wall at the abutment face, however, assumes a full active wedge zone at the rear of the retaining wall.

Due to the maximum 32-foot design height of these walls relative to the undrained shear strength of the underlying soil, the rate of construction will need to be staged to control the risk of general shear failure. As indicated by the calculations included in Appendix F, the MSE abutment and associated wingwalls can be constructed to a maximum height of 29-feet in a first stage of loading. Once this height is reached the underlying soil must have sufficient time to consolidate and allow any excess pore water pressures to dissipate. Following the initial stage of construction, it is our recommendation that the walls can be constructed in 5-foot intervals with a minimum 7-day quarantine period between intervals. Settlement plates should be monitored on a daily basis and after 7 days the engineer or a qualified owner's representative should observe the data to verify settlement has ceased and give permission for the Contractor to continue with filling operations.

The minimum reinforcement length for the wingwalls is 32-ft and for the abutment face is 27-feet. The length at the abutment wingwalls was controlled by the design cross section of RW 7 and RW 8. The length at the abutment face was increased to satisfy minimum AASHTO requirements against general shear or global stability failures.

Factored bearing resistances calculated for the wall at various stages of construction and after construction are included in Appendix F. Based on a resistance factor of 0.65 the factored bearing resistance for the initial stages of construction, prior to any fill placement, is 6.0-ksf, which should be verified prior to construction.

It is anticipated that the total settlement for the Abutment B wingwalls will be about 4.2-inches and for the Abutment B face retaining wall will be about 5.0-inches, see Appendix F. Vertical control joints should be provided to accommodate differential settlement. Based on the results of settlement monitoring plates at the test embankment, the settlement should occur within 2 to 4-weeks after fill placement to final grade. Settlements should be monitored to verify movement has substantially ceased prior to driving piles and construction of parapets and pavements.

Global stability analyses, included in Appendix F, were also performed which satisfied the aforementioned criteria.

5.6.7 Structure S6: Ramp B over Northbound I-95 – Wingwalls

Based on our review the existing subsurface conditions and laboratory testing, we recommend that the MSE wingwalls be constructed prior to the installation of the piles for the Structure S6 – Ramp B over I-95 Northbound abutments to reduce downdrag loads on the abutment piles. The piles should be driven through sleeves or casing installed during construction of the MSE's.

Table 5.26 summarizes the soil parameters to be used by the Contractor during design of the MSE wall for this structure. Where the groundwater level was one footing width (strap length) below the leveling pad, the wet unit weight of water was used in our analysis of external stability in MSEW. In cases were groundwater is within one footing width of the leveling pad elevation, the adjusted the unit weight was used. The design unit weight indicated in Table 5.24 has been adjusted due to the presence of shallow groundwater.

Table 5.26 – Structure S6 Soil Parameters For Contractor's Design

Strata	Design Unit Weight – γ (pcf)	Angle of Friction – ϕ (deg)	Undrained Shear - S_u (psf)
Common Borrow	125	28	-
Select Fill	125	34	
LWEF	40	-	18,000-psi
Foundation Soils Abutment A	68	29	1,700
Foundation Soil Abutment B	68	28	-

Based on the proposed wingwall locations adjacent to Abutments A and B, it is anticipated that the reinforcement zones will overlap. Our evaluation of the external stability of these walls assumes that active earth thrust from the backfill is significantly reduced, since the overlap of the reinforcement is anticipated to be greater than 0.3H, where H is the height of the retaining wall.

The use of a single reinforcement (common) to connect both wall facings in this area is not recommended. By connecting the wall facings with a common reinforcement method changes

the strain pattern of the structure and will result in higher reinforcement tensions, thus the recommendations provided in this report would not be applicable.

Groundwater for this structure is anticipated to be near EL 75, approximately 8 to 11-ft below the proposed bottom of footing elevation. Groundwater is not anticipated to be within the excavation limits of the proposed foundations for this structure. Section 5.7 of this report discusses general dewatering and drainage recommendations for this project.

Supporting calculations for this structure are provided in Appendix F.

Abutment A

The Structure S6 wingwalls at Abutment A may be designed for a factored bearing resistance of 8.4 ksf and constructed with select fill for the full height of the abutment. This was calculated using a nominal bearing resistance of 19.9-kpsf and applying a bearing resistance factor of 0.65. The minimum reinforcement length for the wingwalls and abutment face is 25-ft. The factored bearing resistance indicated above should be verified prior to construction.

It is anticipated that the total settlement for Abutment A will be about 1.5-inches. Differential settlement of the wingwalls is not anticipated. Based on the results of settlement monitoring plates at Ramp A, B, C, and G1, the settlement will occur in 2 to 4-weeks of completion of the embankment. Settlements should be monitored to verify settlement will have substantially ceased prior to driving piles and construction of parapets and pavements.

Abutment B

The Structure S6 wingwalls at Abutment B may be designed for a factored bearing resistance of 5.8-ksf. The factored bearing resistance was calculated using a nominal bearing resistance of 8.9-ksf and applying a bearing resistance factor of 0.65. The minimum reinforcement length for the wingwalls is 25-ft. The minimum reinforcement length for the abutment face is 35-ft. The factored bearing resistance indicated above should be verified prior to construction.

Abutment B will require LWEF to be installed from the bottom of the excavation to a height of 14-ft above the leveling pad. Above the LWEF for the remaining height of the MSE wall should be constructed using select fill. A transition from LWEF to select backfill will be required when transiting from the Abutment B wingwalls to RW 14 and 15. RW 14 and 15 are discussed in a separate report.

It is anticipated that the total settlement for Abutment B will be about 1.5-inches. Differential settlement of the wingwalls is not anticipated. Based on the results of settlement monitoring plates at Ramp A, B, C, and G1, the settlement will occur in 2 to 4-weesk of completion of the

embankment. Settlements should be monitored to verify settlement will have substantially ceased prior to driving piles and construction of parapets and pavements.

5.6.8 Structure S7: Ramp R1 over SR 7 – Wingwalls

We recommend that the wingwalls for Structure S7 be a CIP concrete wall supported on driven H piles. Table 5.27 summarizes the recommended pile size, estimated length of the piles, the recommended pile tip elevation, the strength loads, the service loads and the nominal driving resistance required, per pile. Service loads were used to estimate settlements and the strength loads were used to determine the required nominal pile driving resistance required and tip elevations.

Table 5.27 – Structure S7 Wingwall Pile Foundation Recommendations

Structural Element	Pile Size	Est. Pile Length (ft)/ Est. Tip Elevation	Nominal Pile Driving Resistance Required (R_{ndr}) Per Pile (kips)
Wingwalls I and II	HP 14x73	64 / -24	V: 280
Wingwalls III and IV	HP 14x73	48 / -9	V: 280

R_n = Nominal (ultimate) Bearing Resistance
R_n= Strength Load / (ϕ_{dyn})
Strength Load = $\Sigma\eta_i\gamma_i Q_i \leq \phi_{dyn}R_n = \phi_{static}R_{n\ static}$
ϕ_{dyn} = 0.65
ϕ_{static} = 0.35
R_{ndr} = Nominal Pile Driving Resistance Required for High-Strain Dynamic Testing
$R_{ndr} = R_n$.

η = Load Modifier
γ = Load Factor
Q = Service Load
V = Axial Load

Groundwater for this structure is anticipated to be near EL 30 to 45. Groundwater is anticipated to be near the proposed bottom of footing elevation for both Abutments A and B. Dewatering during construction of the pile caps is anticipated. Section 5.7 of this report discusses general dewatering and drainage recommendations for this project.

Supporting calculations for this structure are provided in Appendix F.

5.6.9 Structure S8: SR 1 over Eagle Run – Wingwalls

No foundation recommendations are required for this structure since the foundation is already in place. The existing foundation is discussed in Section 2.2.2 of this report.

5.7 GENERAL DEWATERING AND DRAINAGE

In general, the pile caps for the bridge abutment foundations will be located at a safe distance above the elevation of the groundwater table. It is anticipated that groundwater will be encountered for construction of most wingwalls near the elevation of the leveling pad. A detailed discussion regarding the depth of groundwater with respect to the proposed construction is contained in Sections 5.4.1 through 5.4.8 and 5.6.2 through 5.6.9 of this report.

Water trapped in sandy seams, in FILL, near the contact between strata, or interbedded lenses of Stratum IIb could cause minor construction difficulties. The Contractor should be prepared to dewater any groundwater, surface runoff, or water collected after a rain event.

Problems associated with groundwater should be minor in nature. If groundwater is encountered during construction, appropriate dewatering should be carried out so that construction will be performed in a relatively dry condition. Dewatering should be able to be handled with conventional ditching, sumps, and pumping. The sump should be located at least 3-ft away from the excavation or other proposed structures to avoid softening of footing subgrade areas.

Adequate drainage should be provided at the site to minimize any increase in moisture content of the foundation soils and pavement. All runoff from adjacent areas should be diverted away from the bridge, retaining walls, roadways, and excavations to prevent ponding of water around the footings. The site drainage should be such that the runoff onto adjacent properties is controlled properly.

6 BASIS OF RECOMMENDATIONS

This report has been prepared to present the geotechnical conditions at the site and provide geotechnical recommendations to serve as a basis for design and preparation of plans and specifications. The opinions, conclusions and recommendations contained in this report are based upon our professional judgment and generally accepted principles of geotechnical engineering. Inherent to these are the assumptions that the earthwork and foundation construction should be monitored and tested by an engineering technician acting under the guidance of a geotechnical engineer licensed in the State of Delaware.

These analyses and recommendations are, of necessity, based on the information available at the time of the actual writing of the report and on the site and subsurface conditions that existed at the time the exploratory borings were drilled. Further, assumptions have been made regarding the lateral extent of conditions between exploratory borings.

The following is a summary of the geotechnical responsibilities for this report, by firm:

- The Geotechnical Data Reports for the Delaware Turnpike Improvements: Mainline, Toll Plaza, Northbound Widening, and SR 1 Interchange were prepared by RK&K.
- The Geotechnical Data Reports for the Delaware Turnpike Improvements: Mainline, Toll Plaza, Northbound Widening, and SR 1 Interchange were provided to URS for their review and use during design.
- A supplemental subsurface exploration and laboratory testing program for this project was developed by both RK&K and URS.
- The supplemental field work was inspected by the firm requesting the additional geotechnical information. In addition, supplemental laboratory test requests were developed by the firm that inspected the borings.
- Specific foundation recommendations; included supporting calculations, for the proposed construction of the bridge and wingwalls were provided by the firm indicated Table 6.1 below.
- General project recommendations and discussions applicable to the overall construction of the interchange were developed by both RK&K and URS.

Table 6.1 – Structures By Designing Firm	
Description	**Designing Firm**
Structure S1 - Ramp A over I-95	URS
Structure S2 - Ramp A over SR 7	URS
Structure S3 - Ramp B over SR 7	URS
Structure S4 - Ramp C over SR 7	URS
Structure S5 - Ramp G1 over SR 7	URS
Structure S6 - Ramp B over Northbound I-95	RK&K
Structure S7 - Ramp R1 over SR 7	RK&K
Structure S8 - SR 1 over Eagle Run Bridge	RK&K

The nature and extent of variations between borings may not become evident until construction. If variations from the anticipated conditions are encountered, it may be necessary to revise the recommendations in this report.

Our professional services have been performed in accordance with generally accepted engineering principles and practices; no other warranty, expressed or implied, is made. RK&K/URS assumes no responsibility for interpretations made by others on the work performed by RK&K/URS.

We recommend that this report be made available in its entirety to contractors for informational purposes only. The boring logs and laboratory test data contained in this report represent an integral part of this report and incorrect interpretation of the data may occur if the attachments are separated from the text. The project plans or specifications should include the following note:

> *A geotechnical report has been prepared for this project by Rummel, Klepper & Kahl, LLP in conjunction with URS Corporation. This report is for informational purposes only and shall not be considered as part of the contract documents. The opinions and conclusions of RK&K/URS represent our interpretation of the subsurface conditions and the planned construction at the time of the report preparation. The data in this report may not be adequate for contractors estimating purposes.*