FINAL FOUNDATIONS REPORT – RETAINING WALLS

SR1/I-95 Interchange

New Castle County, Delaware

State Contract No. 28-090-03

Federal Project No. IM-N065(35)

Prepared for:

Delaware Department of Transportation

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

April 2009

Prepared by:

RUMMEL, KLEPPER & KAHL, LLP
81 Mosher Street
Baltimore, Maryland 21217

URS Corporation
4 North Park Drive
Hunt Valley, Maryland 21030

Rummel, Klepper & Kahl, LLP
81 Mosher Street
Baltimore, Maryland 21217

URS Corporation
4 North Park Drive
Hunt Valley, Maryland 21030

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

April 24, 2009

Prepared for:
Delaware Department of Transportation
800 Bay Road
Dover, Delaware 19903

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

The FFR – Retaining Walls 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 F 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

Eric M. Klein, P.E.
Associate – Geotechnical
Engineering Department

Jennifer L. Trimble, P.E.
Senior Geotechnical Engineer

Robert W. O'Connor, P.E.
Chief Geotechnical Engineer

Jason P. Kotova, P.E.
Geotechnical Engineer

EXECUTIVE SUMMARY

In accordance with our proposal, Rummel, Klepper & Kahl, LLP (RK&K) in conjunction with URS Corporation (URS) have completed the Final Foundation Report (FFR) for the Retaining Walls for the SR1/I-95 Interchange. A separate FFR was submitted by RK&K/URS which addresses the bridges and associated wingwalls within the project limits. Wingwalls in the FFR – Bridges and Associated Wingwalls are 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.

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 retaining wall foundation design, including sequence of construction, and other geotechnical concerns.

This FFR supersedes in its entirety the Preliminary Foundation Report (PFR) – Retaining Walls 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 F of this report.

Retaining Wall construction within the limits of the proposed SR 1/I-95 Interchange will consist of the following structures:

Table E.1 – Proposed Retaining Walls Locations				
Retaining Wall No.	**Location**	**Designer**	**Length (ft)**	**Report Section**
RW1	Ramp A	URS	155	2.5.1
RW2	Ramp G1	URS	91	2.5.2
RW3	Ramp B	URS	263.5	2.5.3
RW4	SR 7 Northbound	RK&K	277	2.5.4
RW5	Ramp B	RK&K	128	2.5.5
RW6	Ramp C	RK&K	177	2.5.6
RW7	Ramp G1	RK&K	925	2.5.7
RW8	SR 7 Southbound/ Ramp C1	RK&K	399	2.5.8
RW9	Ramp A	RK&K	872	2.5.9
RW10	Ramp C	RK&K	355	2.5.10
RW11	***Eliminated***	RK&K	-	2.5.11
RW12	Ramp A	RK&K	358	2.5.12
RW13	Ramp A	RK&K	348	2.5.13
RW14	Ramp B	RK&K	735	2.5.14
RW15	Ramp B	RK&K	905	2.5.15
RW16	Northbound SR 7	RK&K	99	2.5.16

Retaining Wall Foundation Alternative Analysis

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

The following retaining wall types 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 (CIP) Pile Supported (Section 5.2.2)
- Mechanically Stabilized Earth Walls – MSE (Section 5.2.3)
- Other Wall Types (Section 5.2.4)

Summary of Foundation Analysis

It is recommended that MSE retaining walls be used for construction of the retaining walls. To meet design criteria for this project, portions of several retaining walls will require using Lightweight Engineered Fill (LWEF) and No. 57 stone within the reinforcement and retained soil zones. Also, to satisfy bearing and global stability, the minimum reinforcement length was increased for select structures from the typical value of 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 retaining walls near bridge abutments 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 5.3.1 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.

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 retaining walls 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.

Special considerations include the sequence of construction, dewatering and drainage, anticipated settlement from construction, and constructability. These items are discussed in further detail in Section 5.4 of this report.

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 5.3 of this report for each retaining wall.

Supporting calculations and a preliminary cost comparison are contained in Appendix E 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.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF APPENDICIES

APPENDIX A

APPENDIX B
(***All of Appendix B is on the CD included with this report***)

APPENDIX C

APPENDIX D

APPENDIX E

URS

APPENDIX F

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 SR 1/I-95 Interchange - Retaining Walls.

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 preliminary geotechnical recommendations to guide design of retaining walls. 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, preliminary recommendations are provided for retaining wall 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 – SR1 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 and supersede in its entirety the Preliminary Foundation Report (PFR) – Retaining Walls 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 F 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. Four through lanes currently continue north on I-95 beyond the interchange through Churchmans Marsh.

In the southbound direction, I-95 carries five 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 are 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) R_u	CAPWAP Results: Resistances (kips) R_{skin}	CAPWAP Results: Resistances (kips) R_{toe}	CAPWAP Results: Average Unit Resistance (ksf) Skin	CAPWAP Results: Average Unit Resistance (ksf) Toe	CAPWAP Results: Pile Stress (ksi)
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 D-1 in Appendix D 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 D-2a through D-2f in Appendix D of this report. The location of the settlement monitoring points is contained in Figure E-1 in Appendix D of this report. Figure D-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 E.

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

More detailed descriptions of the existing bridges are included in the FFR for Bridges and Wingwalls dated April 2009.

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 about 218-ft. 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 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 pier 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.

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. Span 1, from Abutment A to Bent 1 is 60-ft in length. Span 2, from Bent 1 to Bent 2, is 70-ft in length. Span 3, from Bent 2 to the center line of Abutment B, is 70-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.

2.4.3 Road A Bridge

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. Span 1, from the centerline of Abutment 1 to pier is 110-ft in length. Span 2, from pier to the center line of Abutment B, is 136-ft. The width of the existing bridge is 44-ft.

2.5 PROPOSED RETAINING WALLS

Retaining wall construction within the limits of the proposed SR 1/I-95 Interchange will consist of the following structures:

Table 2.10 – Proposed Retaining Walls

Retaining Wall No.	Location	Designer	Length (ft)	Report Section
RW1	Ramp A	URS	155	2.5.1
RW2	Ramp G1	URS	91	2.5.2
RW3	Ramp B	URS	263.5	2.5.3
RW4	SR 7 Northbound	RK&K	277	2.5.4
RW5	Ramp B	RK&K	128	2.5.5
RW6	Ramp C	RK&K	177	2.5.6
RW7	Ramp G1	RK&K	925	2.5.7
RW8	SR 7 Southbound/ Ramp C1	RK&K	399	2.5.8
RW9	Ramp A	RK&K	872	2.5.9
RW10	Ramp C	RK&K	355	2.5.10
RW11	***Eliminated***	RK&K	-	2.5.11
RW12	Ramp A	RK&K	358	2.5.12
RW13	Ramp A	RK&K	348	2.5.13
RW14	Ramp B	RK&K	735	2.5.14
RW15	Ramp B	RK&K	905	2.5.15
RW16	Northbound SR 7	RK&K	99	2.5.16

In general, bridge wingwalls are measured as 30-ft from the center line of bearing of abutment for this project and are contained in a separate Final Foundation Report - Bridge and Wingwalls. Retaining wall structures beyond this length are considered stand alone retaining walls and are addressed in this report. The wall lengths provided in this report are consistent with the TS&L drawings and do not include the length of wall to be constructed as a bridge wingwall.

2.5.1 Retaining Wall 1: Ramp A

The construction of Ramp A will require the construction of a retaining wall which will be approximately 155-ft in length and will be located along the left side of Ramp A, between Ramp A and Ramp B. The retaining wall will be a Mechanically Stabilized Earth (MSE) wall extending from Ramp A STA 1250+58 to Ramp A STA 1252+14. The location of Retaining Wall 1 is shown in Figure A-2g located in Appendix A of this report. The existing ground surface in this area ranges from approximately EL 66.1 to EL 67.8. Above the wall will be Ramp A and below the wall will be NB SR 7.

It should be noted that 30-ft of Retaining Wall 1 was considered in the design of the abutment for Structure S2, Ramp A over SR 7 NB.

Table 2.11 summarizes the dimensions of the proposed Retaining Wall 1. The design wall height is measured from the top of the leveling pad to the top elevation of the proposed wall. The anticipated front slope is also included in Table 2.11. It should be noted that Retaining Wall 1 and Retaining Wall 9 will be constructed as back to back walls for the entire length of Retaining Wall 1, and the least distance between the front face of the walls will be approximately 44-ft.

Table 2.11 - Dimensions of Retaining Wall RW1: Ramp A

RAMP A STA	Design Wall Height (ft)	Top of Leveling Pad EL.	Top of Wall EL.	Front Slope / Slope Height (ft)
1250+58	36.5	59.5	96.0	Horizontal
1251+10	34.7	59.5	94.2	Horizontal
1251+60	32.9	59.5	92.4	Horizontal
1252+14	32.0	59.5	91.5	Horizontal

As indicated above, the maximum wall height of Retaining Wall 1 is near 36.5-ft.

2.5.2 Retaining Wall 2: Ramp G1

The construction of Ramp G1 will require the construction of a retaining wall which will be approximately 91-ft in length and will be located along the right side of Ramp G1. The retaining wall will be a MSE wall extending from Ramp G1 STA 1310+65 to Ramp G1 STA 1311+57. The location of Retaining Wall 2 is shown in Figure A-2j located in Appendix A of this report. The existing ground surface elevation in this area is approximately EL 78.0. Ramp G1 will be located above the wall and SR 7 SB will be located below the wall.

It should be noted that 30-ft of Retaining Wall 2 was considered in the design of the abutment for Structure S5, Ramp G1 over SR 7.

Table 2.12 summarizes the dimensions of the proposed Retaining Wall 2. The design wall height is measured from the top of the leveling pad to the top elevation of the proposed wall. The anticipated front slope is also included in Table 2.12.

Table 2.12 - Dimensions of Retaining Wall RW2: Ramp G1

RAMP G1 STA	Design Wall Height (ft)	Top of Leveling Pad EL.	Top of Wall EL.	Front Slope / Slope Height (ft)
1310+65	36.7	71.0	107.7	Horizontal
1311+15	44.2	63.0	107.2	Horizontal
1311+56	43.5	63.0	106.5	Horizontal

As indicated above, the maximum wall height of Retaining Wall 2 is near 44.2-ft.

2.5.3 Retaining Wall 3: Ramp B

The construction of Ramp B will require the construction of a retaining wall which will be approximately 263.5-ft in length and will be located along the left side of Ramp B, between Ramp A and Ramp B. The retaining wall will be a MSE wall extending from Ramp B STA 436+90 to Ramp B STA 439+52. The location of Retaining Wall 3 is shown in Figure A-2g and A-2j located in Appendix A of this report. The existing ground surface elevation in this area ranges from approximately EL 66.3 to EL 75.2. Ramp B will be located above the wall and NB SR 7 will be located below the wall.

It should be noted that 30-ft of Retaining Wall 3 was considered in the design of the abutment for Structure S2 Ramp B over SR 7 NB.

Table 2.13 summarizes the dimensions of the proposed Retaining Wall 3. The design wall height is measured from the top of the leveling pad to the top elevation of the proposed wall. The anticipated front slope is also included in Table 2.13.

Table 2.13 - Dimensions of Retaining Wall RW3: Ramp B				
RAMP B STA	**Design Wall Height (ft)**	**Top of Leveling Pad EL.**	**Top of Wall EL.**	**Front Slope / Slope Height (ft)**
436+90	38.5	62.0	100.5	Horizontal
437+40	39.0	62.0	101.0	Horizontal
437+90	40.0	64.0	102.0	Horizontal
438+40	40.7	62.0	102.7	Horizontal
438+90	41.5	62.0	103.5	Horizontal
439+40	36.0	68.0	104.0	Horizontal
439+50	34.3	70.0	104.3	Horizontal

As indicated above, the maximum wall height of Retaining Wall 3 is near 41.5-ft.

2.5.4 Retaining Wall 4: SR 7 Northbound

The reconstruction of SR 7 North Bound (NB) will require the construction of a retaining wall which will be approximately 277-ft in length and will be located along the left side of SR 7 NB, between SR 7 NB and Ramp B. The retaining wall will be a MSE wall extending from SR 7 NB STA 630+30 to SR 7 NB STA 633+04.66. The location of Retaining Wall 4 is shown in Figure A-2g located in Appendix A of this report. The existing ground surface elevations in this area range from approximately EL 68.2 to EL 71.5. The wall will be built by lowering the grades about 6.1 to 13.2-ft to construct SR 7 NB.

Table 2.14 summarizes the dimensions of the proposed Retaining Wall 4. The design wall height is measured from the bottom of the leveling pad to the top elevation of the proposed wall. The anticipated front and back slopes are also included in Table 2.14. It should be noted that from approximately Ramp A STA 1252+00 to Ramp A STA 1255+00 the least distance between the faces of Retaining Wall 4 and Retaining Wall 9 is 100-ft.

Table 2.14 - Dimensions of Retaining Wall RW4: SR 7 Northbound					
Ramp A STA	**Design Wall Height (ft)**	**Bottom of Leveling Pad EL.**	**Top of Wall EL.**	**Front Slope**	**Back Slope / Slope Height (ft)**
1252+00	21.9	58.0	79.9	SR 7 NB	2(H):1(V)/ 1.7
1252+50	27.3	57.0	84.3	SR 7 NB	2(H):1(V)/ 4.5
1253+00	22.7	56.0	78.7	SR 7 NB	2(H):1(V)/ 8.2
1253+50	18.7	55.0	73.7	SR 7 NB	2(H):1(V)/ 11.2
1254+00	15.4	54.0	69.4	SR 7 NB	2(H):1(V)/ 13.5
1254+50	11.8	54.0	65.8	SR 7 NB	2(H):1(V)/15.2
1255+00	9.7	53.0	62.7	SR 7 NB	2(H):1(V)/16.2

As indicated above, the maximum wall height of Retaining Wall 4 is about 27.3-ft. Figure A-5a in Appendix A of this report shows a typical cross section of Retaining Wall 4.

2.5.5 Retaining Wall 5: Ramp B

The construction of Ramp B will require the construction of a retaining wall which will be approximately 128-ft in length and will be located along the left side of Ramp B, between the Ramp A bridge and Ramp B. The retaining wall will be a MSE wall extending from Ramp B STA 449+21.51 to Ramp B STA 450+35. The location of Retaining Wall 5 is shown in Figure A-2j and A-2l located in Appendix A of this report. The existing ground surface elevation in this area ranges from approximately EL 84.6 to EL 89.4. This wall will be located atop the Ramp A, B, C, G1 Embankment. Ramp A will be located above the wall and Ramp B will be located below the wall.

It should be noted that 30-ft of Retaining Wall 5 was considered in the design of the abutment for Structure S6, Ramp B over NB I-95.

Table 2.15 summarizes the dimensions of the proposed Retaining Wall 5. The design wall height is measured from the bottom of the leveling pad to the top elevation of the proposed wall. The anticipated front slope is also summarized in Table 2.15.

Table 2.15 - Dimensions of Retaining Wall RW5: Ramp B

RAMP A STA	Design Wall Height (ft)	Bottom of Leveling Pad EL.	Top of Wall EL.	Front Slope / Slope Height (ft)
1235+00	34.2	85.5	119.7	Horizontal
1235+50	33.5	85.5	119.0	Horizontal

As indicated above, the maximum wall height of Retaining Wall 5 is near 34.2-ft. Figure A-5b in Appendix A of this report represents a typical cross section of Retaining Wall 5.

2.5.6 Retaining Wall 6: Ramp C

The construction of Ramp C will require the construction of a retaining wall which will be approximately 177-ft in length and will be located along the right side of Ramp C, between Ramp C and Ramp G1. The retaining wall will be a MSE wall extending from Ramp C STA 1117+54.25 to Ramp C STA 1119+25. The location of Retaining Wall 6 is shown in Figure A-2j located in Appendix A of this report. The existing ground surface elevations in this area range from approximately EL 83.7 to EL 89.5. Ramp C will be located below Retaining Wall 6 and Ramp G1 will be located above the wall.

It should be noted that 30-ft of Retaining Wall 6 was considered in the design of the abutment for Structure S4, Ramp C over SR 7.

Table 2.16 summarizes the dimensions of the proposed Retaining Wall 6. The design wall height is measured from the bottom of the leveling pad to the bottom elevation of the proposed wall. The anticipated front and back slopes are also summarized in Table 2.16. Retaining Wall 6 and Retaining Wall 16 will be constructed as superimposed MSE walls; the least distance between the front face of the walls will be approximately 41-ft.

Table 2.16 - Dimensions of Retaining Wall RW6: Ramp C

RAMP C STA	Design Wall Height (ft)	Bottom of Leveling Pad EL.	Top of Wall EL.	Front Slope	Back Slope / Slope Height (ft)
1118+00	20.1	82.5	102.6	Ramp C	2(H):1(V)/ 5.8
1118+50	19.7	78.0	97.7	Ramp C	2(H):1(V)/ 11.5
1119+00	14.5	76.0	90.5	Ramp C	2(H):1(V)/ 19.3

As indicated above, the maximum wall height of Retaining Wall 6 is near 20.1-ft. Figure A-5c in Appendix A of this report shows a typical cross section of Retaining Wall 6.

2.5.7 Retaining Wall 7: Ramp G1

The construction of Ramp G1 will require the construction of a retaining wall which will be approximately 925-ft in length and will be located along the right side of Ramp G1. Retaining Wall 8 will be located on the left side of Ramp G1. Retaining Wall 7 will be a MSE wall extending from Ramp G1 STA 1317+68 to Ramp G1 STA 1326+88. The location of Retaining Wall 7 is shown in Figure A-2f and A-2g located in Appendix A of this report. The existing ground surface elevations in this area will range from EL 56.7 and EL 68.0. Ramp G1 will be located above the wall.

It should be noted that 30-ft of Retaining Wall 7 was considered in the design of the abutment for Structure S5, Ramp G1 over SR7.

Table 2.17 summarizes the dimensions of the proposed Retaining Wall 7. The design wall height is measured from the bottom of the leveling pad to the top elevation of the proposed wall. The existing Right-Of-Way Line runs parallel to Retaining Wall 7, and the least distance is 26-ft at approximately Ramp G1 STA 1318+00. The anticipated front slope is also included in Table 2.17. It should be noted that Retaining Wall 7 and Retaining Wall 8 will be constructed as back to back walls from Ramp G1 STA 1317+68 to approximately Ramp G1 STA 1320+10. The least distance between the front face of the walls will be approximately 41.2-ft.

Table 2.17 - Dimensions of Retaining Wall RW7: Ramp G1				
Ramp A STA	**Design Wall Height (ft)**	**Bottom of Leveling Pad EL.**	**Top of Wall EL.**	**Front Slope / Slope Height (ft)**
1251+00	32.0	62.5	94.5	Horizontal
1251+50	30.2	62.5	92.7	Horizontal
1252+00	28.3	62.5	90.8	Horizontal
1252+50	27.1	62.0	89.1	Horizontal
1253+00	26.2	61.5	87.7	Horizontal
1253+50	24.7	61.5	86.2	Horizontal
1254+00	24.8	60.0	84.8	Horizontal
1254+50	25.4	58.0	83.4	Horizontal
1255+00	24.9	57.0	81.9	Horizontal
1255+50	25.9	54.5	80.4	Horizontal
1256+00	23.5	55.5	79.0	Horizontal
1256+50	20.5	57.0	77.5	Horizontal
1257+00	19.0	57.0	76.0	Horizontal
1257+50	17.5	57.0	74.5	Horizontal
1258+00	16.3	56.5	72.8	Horizontal
1258+50	14.4	56.5	70.9	Horizontal
1259+00	12.7	56.5	69.2	Horizontal
1259+50	10.1	57.5	67.6	Horizontal

As indicated above, the maximum wall height of Retaining Wall 7 is near 32.0-ft. Figure A-5d in Appendix A of this report represents a typical cross section of Retaining Wall 7.

2.5.8 Retaining Wall 8: SR 7 Southbound/Ramp C1

The reconstruction of SR 7 South Bound (SB) will require the construction of a retaining wall which will be approximately 399-ft in length and will be located along the right side of SB SR 7, extending along the right side of Ramp C1. The retaining wall will be a MSE wall extending from Ramp C1 STA 1815+58.62 to Ramp C1 STA 1818+09, which corresponds to SR 7 SB STA 1614+14. The wall will then extend from SR 7 SB STA 1614+14 to SR 7 SB STA 1615+60. The location of Retaining Wall 8 is shown in Figure A-2g located in Appendix A of this report. The existing ground surface elevation in this area ranges from EL 56.7 and EL 68.0. Ramp G1 will be located above the wall and Ramp C1 will be located below the wall between Ramp C1

STA 1815+58.62 to Ramp C1 STA 1818+09. SR7 SB will be located below the wall between SR 7 SB STA 1614+14 to SR 7 SB STA 1615+60.

It should be noted that 30-ft of Retaining Wall 7 was considered in the design of the abutment for Structure S5, Ramp G1 over SR7.

Table 2.18 summarizes the dimensions of the proposed Retaining Wall 8. The design wall height is measured from the bottom of the leveling pad to the top elevation of the proposed wall. The anticipated front and back slopes are also included in Table 2.18. Retaining Wall 7 and Retaining Wall 8 will be constructed as back to back walls for the entire length of Retaining Wall 8. The least distance between the front face of the walls will be approximately 51.2-ft.

Table 2.18 - Dimensions of Retaining Wall RW8: SR 7 Southbound/Ramp C1

Ramp C1 STA	Design Wall Height (ft)	Bottom of Leveling Pad EL.	Top of Wall EL.	Front Slope	Back Slope / Slope Height (ft)
1816+00	22.0	62.0	84.0	Ramp C1	2(H):1(V)/ 7.8
1816+50	20.1	62.0	82.1	Ramp C1	2(H):1(V)/ 8.6
1817+00	17.1	63.0	80.1	Ramp C1	2(H):1(V)/ 9.3
1817+50	15.4	63.0	78.4	Ramp C1	2(H):1(V)/ 9.6
1818+00	14.1	63.0	77.1	Ramp C1	2(H):1(V)/ 9.6
1253+50(1)	13.7	63.0	76.7	SR 7 SB	2(H):1(V)/ 9.6
1254+00(1)	14.3	61.5	75.8	SR 7 SB	2(H):1(V)/ 9.0
1254+50(1)	12.9	61.5	74.4	SR 7 SB	2(H):1(V)/ 9.0
Note (1): Ramp A Station					

As indicated above, the maximum wall height of Retaining Wall 8 is near 22.0-ft. Figure A-5e in Appendix A of this report shows a typical cross section of Retaining Wall 8.

2.5.9 Retaining Wall 9: Ramp A

The construction of Ramp A will require the construction of a retaining wall which will be approximately 872-ft in length and will be located along the right side of Ramp A, between Ramp A and SB SR 7. The retaining wall will be a MSE wall extending from Ramp A STA 1250+92 to Ramp A STA 1259+50. The location of Retaining Wall 9 is shown in Figure A-2e and A-2g located in Appendix A of this report. The existing ground surface elevation in this area ranges from EL 57.6 to EL 68.3. Ramp A will be located above the wall and SB SR 7 will be located below the wall.

It should be noted that 30-ft of Retaining Wall 9 was considered in the design of the abutment for Structure S2, Ramp A over SR 7.

Table 2.19 summarizes the dimensions of the proposed Retaining Wall 9. The design wall height is measured from the bottom of the leveling pad to the top elevation of the proposed wall. The anticipated front slope is also included in Table 2.19. It should be noted that Retaining Wall 9 and Retaining Wall 1 will be constructed as back to back walls from Ramp A STA 1250+92 to Ramp A STA 1252+14. The least distance between the front face of the walls will be approximately 44-ft, and from approximately Ramp A STA 1252+14 to Ramp A STA 1255+00, the least distance between the front face of Retaining Wall 9 and Retaining Wall 4 will be approximately 100-ft.

Table 2.19 - Dimensions of Retaining Wall RW9: Ramp A

RAMP A STA	Design Wall Height (ft)	Bottom of Leveling Pad EL.	Top of Wall EL.	Front Slope / Slope Height (ft)
1251+00	32.8	63.0	95.8	Horizontal
1251+50	31.3	63.0	94.3	Horizontal
1252+00	30.8	62.0	92.8	Horizontal
1252+50	29.1	62.0	91.1	Horizontal
1253+00	28.4	61.0	89.4	Horizontal
1253+50	26.7	61.0	87.7	Horizontal
1254+00	25.8	60.0	85.8	Horizontal
1254+50	24.9	59.0	83.9	Horizontal
1255+00	23.0	59.0	82.0	Horizontal
1255+50	21.9	58.0	79.9	Horizontal
1256+00	20.0	58.0	78.0	Horizontal
1256+50	19.0	57.0	76.0	Horizontal
1257+00	17.0	57.0	74.0	Horizontal
1257+50	16.0	56.0	72.0	Horizontal
1258+00	13.5	56.0	69.5	Horizontal
1258+50	13.0	55.0	68.0	Horizontal
1259+00	11.9	54.0	65.9	Horizontal
1259+50	10.0	54.0	64.0	Horizontal

As indicated above, the maximum wall height of Retaining Wall 9 is near 32.8-ft. Figure A-5f in Appendix A of this report shows a typical cross section of Retaining Wall 9.

2.5.10 Retaining Wall 10: Ramp C

The construction of Ramp C will require the construction of a retaining wall that will be approximately 355-ft in length and will be located along the left side of Ramp C, between the SW Loop and Ramp C. Retaining Wall 10 will be a MSE wall extending from Ramp C STA 1110+75 to Ramp C STA 1114+36.25. The location of Retaining Wall 10 is shown in Figure A-2g and A-2h located in Appendix A of this report. The existing ground surface elevation in the area ranges from approximately EL 70.8 to EL 74.3. Ramp C will be located on top of Retaining Wall 10 and the SW Loop will be located below.

It should be noted that 30-ft of Retaining Wall 10 was considered in the design of the abutment for Structure S4, Ramp C over SR 7.

Table 2.20 summarizes the dimensions of the proposed Retaining Wall 10. The design wall height is measured from the bottom of the leveling pad to the top elevation of the proposed wall. The anticipated front slope is also included in Table 2.20.

Table 2.20 - Dimensions of Retaining Wall RW10: Ramp C

RAMP C STA	Design Wall Height (ft)	Bottom of Leveling Pad EL.	Top of Wall EL.	Front Slope
1111+00	13.8	69.0	82.8	SW Loop
1111+50	16.7	69.0	85.7	SW Loop
1112+00	19.8	69.0	88.8	SW Loop
1112+50	22.8	69.0	91.8	SW Loop
1113+00	25.8	69.0	94.8	SW Loop
1113+50	28.4	69.0	97.4	SW Loop
1114+00	30.2	69.0	99.2	SW Loop
1114+36.25	31.0	69.0	100.0	SW Loop

As indicated above, the maximum wall height of Retaining Wall 10 is near 31.0-ft. Figure A-5g in Appendix A of this report shows a typical cross section of Retaining Wall 10.

2.5.11 Retaining Wall 11: Eliminated

As of October 2008, TS&L plans included the construction of an approximately 2,060-ft long retaining wall adjacent to Ramp B1. An MSE retaining wall extending from Ramp B1 STA

541+52 to Ramp B1 STA 562+00 was anticipated. The maximum wall height was anticipated to be near 25-ft.

Based upon further review, this wall has been eliminated and replaced with the construction of a cut slope. Additional details regarding this slope are provided in a separate memorandum developed by RK&K dated February 6, 2009.

2.5.12 Retaining Wall 12: Ramp A

The construction of Ramp A will require the construction of a retaining wall which will be approximately 358-ft in length and will be located along the right side of Ramp A. Retaining Wall 12 will be located on the northwest side of Ramp A. The retaining wall will be a MSE wall extending from Ramp A STA 1224+00 to Ramp A STA 1227+50.25. The location of Retaining Wall 12 is shown in Figure A-2l located in Appendix A of this report. The existing ground surface elevation in this area ranges from approximately EL 80.2 to EL 91.2. Ramp A will be located above Retaining Wall 12.

It should be noted that 30-ft of Retaining Wall 12 was considered in the design of the abutment for Structure S1, Ramp A over I-95.

Table 2.21 summarizes the dimensions of the proposed Retaining Wall 12. The design wall height is measured from the bottom of the leveling pad to the top elevation of the proposed wall. The anticipated front slope is also included in Table 2.21. The existing Right-of-Way line runs parallel to Retaining Wall 12, and the least distance is approximately 39.7-ft at Ramp A STA 1225+00. Retaining Wall 12 and Retaining Wall 13 will be constructed as back to back walls. The least distance between the front face of the walls will be approximately 56.5-ft.

Table 2.21 - Dimensions of Retaining Wall RW12: Ramp A				
RAMP A STA	**Design Wall Height (ft)**	**Bottom of Leveling Pad EL.**	**Top of Wall EL.**	**Front Slope / Slope Height (ft)**
1224+00	12.1	85.0	97.1	2(H):1(V) / 1.9
1224+50	14.6	85.0	99.6	Horizontal
1225+00	20.1	82.0	102.1	Horizontal
1225+50	22.6	82.0	104.6	Horizontal
1226+00	28.0	79.0	107.0	Horizontal
1226+50	32.3	77.0	109.3	Horizontal
1227+00	34.4	77.0	111.4	Horizontal
1227+50	36.5	77.0	113.5	Horizontal

As indicated above, the maximum wall height of Retaining Wall 12 is near 36.5-ft. Figure A-5i in Appendix A of this report shows a typical cross section of Retaining Wall 12.

2.5.13 Retaining Wall 13: Ramp A

The construction of Ramp A will require the construction of a retaining wall which will be approximately 348-ft in length and will be located along the left side of Ramp A. Retaining Wall 13 will be located on the southeast side of Ramp A. The retaining wall will be a MSE wall extending from Ramp A STA 1224+00 to Ramp A STA 1227+50.25. The location of Retaining Wall 13 is shown in Figure A-2l located in Appendix A of this report. The existing ground surface elevation in this area ranges from EL 79.9 to EL 91.1. Ramp A will be located above Retaining Wall 13, and I-95 will be located below the wall.

It should be noted that 30-ft of Retaining Wall 13 was considered in the design of the abutment for Structure S1, Ramp A over I-95.

Table 2.22 summarizes the dimensions of the proposed Retaining Wall 13. The design wall height is measured from the bottom of the leveling pad to the top elevation of the proposed wall, measured to the nearest 10^{th} of a foot. The anticipated front slope is also included in Table 2.22. Retaining Wall 13 and Retaining Wall 12 will be constructed as back to back walls. The least distance between the front face of the walls will be approximately 56.5-ft.

Table 2.22 - Dimensions of Retaining Wall RW13: Ramp A				
RAMP A STA	**Design Wall Height (ft)**	**Bottom of Leveling Pad EL.**	**Top of Wall EL.**	**Front Slope / Slope Height (ft)**
1224+00	10.0	84.5	94.5	2(H):1(V) / 10.6
1224+50	12.5	84.5	97.0	2(H):1(V) / 9.1
1225+00	18.0	81.5	99.5	2(H):1(V) / 6.2
1225+50	20.5	81.5	102.0	2(H):1(V) / 4.7
1226+00	26.4	78.0	104.4	2(H):1(V) / 2.8
1226+50	28.7	78.0	106.7	Horizontal
1227+00	30.8	78.0	108.8	Horizontal
1227+50	32.9	78.0	110.9	Horizontal

As indicated above, the maximum wall height of Retaining Wall 13 is near 32.9-ft. Figure A-5i in Appendix A of this report represents a typical cross section of Retaining Wall 13.

2.5.14 Retaining Wall 14: Ramp B

The construction of Ramp B will require the construction of a retaining wall which will be approximately 735-ft in length and will be located along the southeast side of Ramp B, which will be located in the median between NB and SB I-95. Retaining Wall 15 will be located on the northwest side of Ramp B. Retaining Wall 14 will be a MSE wall extending from Ramp B STA 458+45 to Ramp B STA 465+80. The location of Retaining Wall 14 is shown in Figure A-2l and A-2m located in Appendix A of this report. The existing ground surface elevation in this area ranges from approximately EL 74.8 to EL 83.3. Ramp B will be located above Retaining Wall 14 and I-95 SB will be located below.

It should be noted that 30-ft of Retaining Wall 14 was considered in the design of the abutment for Structure S6, Ramp B over NB I-95.

Table 2.23 summarizes the dimensions of the proposed Retaining Wall 14. The design wall height is measured from the bottom of the leveling pad to the top elevation of the proposed wall. The anticipated front slope and back slope are also included in Table 2.23. It should be noted that Retaining Wall 14 and Retaining Wall 15 will be constructed as back to back walls for the entire length of Retaining Wall 14. The least distance between the front face of the walls will be approximately 38-ft.

Table 2.23 - Dimensions of Retaining Wall RW14: Ramp B					
RAMP A STA	**Design Wall Height (ft)**	**Bottom of Leveling Pad EL.**	**Top of Wall EL.**	**Front Slope**	**Back Slope / Slope Height (ft)**
1218+00	11.7	71.5	83.2	NB I-95	Horizontal
1218+50	13.6	71.5	85.1	NB I-95	Horizontal
1219+00	14.5	72.5	87.0	NB I-95	Horizontal
1219+50	16.4	72.5	88.9	NB I-95	Horizontal
1220+00	17.3	73.5	90.8	NB I-95	Horizontal
1220+50	18.7	74.0	92.7	NB I-95	Horizontal
1221+00	20.1	74.5	94.6	NB I-95	Horizontal
1221+50	22.0	74.5	96.5	NB I-95	Horizontal
1222+00	22.8	75.5	98.3	NB I-95	Horizontal
1222+50	23.6	76.5	100.1	NB I-95	Horizontal
1223+00	25.3	76.5	101.8	NB I-95	Horizontal
1223+50	25.5	78.0	103.5	NB I-95	Horizontal
1224+00	27.2	78.0	105.2	NB I-95	Horizontal
1224+50	27.9	79.0	106.9	NB I-95	Horizontal
1225+00	29.5	79.0	108.5	NB I-95	Horizontal
1225+50	30.2	80.0	110.2	NB I-95	Horizontal
1226+00	31.9	80.0	111.9	NB I-95	Horizontal

As indicated above, the maximum wall height of Retaining Wall 14 is near 31.9-ft. Figure A-5j in Appendix A of this report shows a typical cross section of Retaining Wall 14.

2.5.15 Retaining Wall 15: Ramp B

The construction of Ramp B will require the construction of a retaining wall which will be approximately 905-ft in length and will be located along the northwest side of Ramp B. Retaining Wall 14 will be located on the southeast side of Ramp B. Retaining Wall 15 will be a MSE wall extending from Ramp B STA 458+45 to Ramp B STA 467+50. The location of Retaining Wall 15 is shown in Figure A-2l and A-2m located in Appendix A of this report. The existing ground surface elevation for this area ranges from approximately EL 73.3 to EL 84.4. Ramp B will be located above Retaining Wall 15.

It should be noted that 30-ft of Retaining Wall 15 was considered in the design of the abutment for Structure S6, Ramp B over NB I-95.

Table 2.24 summarizes the dimensions of the proposed Retaining Wall 15. The design wall height is measured from the bottom of the leveling pad to the top elevation of the proposed wall. The anticipated front slope is also included in Table 2.24. It should be noted that Retaining Wall 14 and Retaining Wall 15 will be constructed as back to back walls from Ramp B STA 458+45 to Ramp B STA 466+60. The least distance between the front face of the walls will be approximately 38-ft.

Table 2.24 - Dimensions of Retaining Wall RW15: Ramp B

RAMP A STA	Design Wall Height (ft)	Bottom of Leveling Pad EL.	Top of Wall EL.	Front Slope / Slope Height (ft)
1217+50	12.7	69.0	81.7	Horizontal
1218+00	13.6	70.0	83.6	Horizontal
1218+50	15.5	70.0	85.5	Horizontal
1219+00	16.5	71.0	87.5	Horizontal
1219+50	16.8	72.5	89.3	Horizontal
1220+00	18.2	72.5	90.7	Horizontal
1220+50	19.6	73.5	93.1	Horizontal
1221+00	21.5	73.5	95.0	Horizontal
1221+50	22.4	74.5	96.9	Horizontal
1222+00	24.3	74.5	98.8	Horizontal
1222+50	25.1	75.5	100.6	Horizontal
1223+00	26.8	75.5	102.3	Horizontal
1223+50	27.5	76.5	104.0	Horizontal
1224+00	29.2	76.5	105.7	Horizontal
1224+50	30.0	77.5	107.5	Horizontal
1225+00	31.2	78.0	109.2	Horizontal
1225+50	32.9	78.0	110.9	Horizontal
1226+00	33.2	79.5	112.7	Horizontal

As indicated above, the maximum wall height of Retaining Wall 15 is near 33.2-ft. Figure A-5j in Appendix A of this report shows a typical cross section of Retaining Wall 15.

2.5.16 Retaining Wall 16: Northbound SR 7

The construction of Ramp C and the reconstruction of NB SR 7 will require the construction of a retaining wall which will be approximately 99-ft in length located along the right side of NB SR 7.

The retaining wall will be a MSE wall extending from NB SR 7 STA 640+41.76 to NB SR 7 STA 641+40. The location of Retaining Wall 16 is shown in Figure A-2j located in Appendix A of this report. The existing ground surface elevations in this area range from EL 66.2 to EL 71.9. Ramp C will be located above Retaining Wall 16 and SR 7 NB will be located below the wall.

Table 2.25 summarizes the dimensions of the proposed Retaining Wall 16. The design wall height is measured from the bottom of the leveling pad to the top elevation of the proposed wall. The anticipated back slope is also included in Table 2.25. Retaining Wall 6 and Retaining Wall 16 will be constructed as superimposed MSE walls. The least distance between the front face of the walls will be approximately 41-ft.

Table 2.25 - Dimensions of Retaining Wall RW16: Northbound SR 7

NB SR 7 STA	Design Wall Height (ft)	Bottom of Leveling Pad EL.	Top of Wall EL.	Back Slope / Slope Height (ft)
640+50	29.6	55.0	84.6	Note 1
641+00	18.5	61.0	79.5	2.2(H):1(V)/ 7.4
Note 1: At the time of this report, the final design of this area was not complete.				

The maximum anticipated wall height of Retaining Wall 16 is near 37.1-ft. Figure A-5k in Appendix A of this report represents a typical cross section of Retaining Wall 16.

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:

- Churchmans Road Bridge over I-95
- Churchmans 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 Turnpike 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: SR 1/I-95 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), CUIC 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 through 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:

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

Where:

$\phi =$ 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:

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

Where:

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-4a and C-4b, 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].

The typical SPT N-value for this stratum is 32-bpf. Using the Meyerhof equation 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 generalized 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 for 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

		Drained		Undrained
Strata	**Total Unit Weight – γ (pcf)**	**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	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:

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

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 D of this report.

Geotechnical software FoSSA 2.0 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 2 (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 E.

4.4 SUBSURFACE CONDITIONS – LOCATION SPECIFIC

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

- Retaining Wall RW1: Ramp A (Section 4.4.1) - URS
- Retaining Wall RW2: Ramp G1 (Section 4.4.2) - URS
- Retaining Wall RW3: Ramp B (Section 4.4.3) - URS
- Retaining Wall RW4: SR 7 Northbound (Section 4.4.4) – RK&K
- Retaining Wall RW5: Ramp B (Section 4.4.5) – RK&K
- Retaining Wall RW6: Ramp C (Section 4.4.6) – RK&K
- Retaining Wall RW7: Ramp G1 (Section 4.3.7) – RK&K
- Retaining Wall RW8: SR 7 Southbound/Ramp C1 (Section 4.4.8) – RK&K
- Retaining Wall RW9: Ramp A (Section 4.4.9) – RK&K
- Retaining Wall RW10: Ramp C (Section 4.4.10) – RK&K
- Retaining Wall RW11: Eliminated (Section 4.4.11) – RK&K
- Retaining Wall RW12: Ramp A (Section 4.4.12) – RK&K
- Retaining Wall RW13: Ramp A (Section 4.4.13) – RK&K
- Retaining Wall RW14: Ramp B (Section 4.4.14) – RK&K
- Retaining Wall RW15: Ramp B (Section 4.4.15) – RK&K
- Retaining Wall RW16: Northbound SR 7 (Section 4.4.16) – RK&K

4.4.1 Retaining Wall RW1: Ramp A

Table 4.4 summarizes the borings that were used for the evaluation of subsurface conditions for Retaining Wall 1. 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: Retaining Wall RW1 - Ramp A		
SPT Borings	**CPT Probes**	**DMT Probes**
IRW-63, URS-5	-	IDMT-16

FILL: FILL extended to a depth of about 4-ft below the existing ground surface and was only encountered at URS-5. The FILL material encountered consisted of loose to medium dense Silty SAND (USCS: SM) [AASHTO: A-2-4].

The SPT N-values generally ranged from 9-bpf to 14-bpf, with an average SPT N-value of 12-bpf.

Stratum I - Columbia Formation - Coarse Grained Soil: Stratum I was not encountered within the limits of the proposed Retaining Wall 1.

Stratum IIa – Cretaceous - Fine Grained Soil: Stratum IIa encountered within the limits of the proposed Retaining Wall 1 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, ML) [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, A-2-6].

The Stratum IIa SPT N-values typically ranged from 7-bpf to 33-bpf, averaging 18-bpf. The natural moisture content averaged 17.9-percent and ranged from 16.2 to 19.1-percent. The liquid limit ranged from 28 to 44, averaging 35. The plasticity index ranged from 9 to 20, averaging 13. The average percentage of fines (percent passing the No. 200 sieve) ranged from 52 to 98-percent, averaging 79-percent.

Stratum IIb – Cretaceous Coarse Grained Soil: Stratum IIb encountered within the limits of the proposed Retaining Wall 1 generally consisted of loose to very dense Silty and Clayey SAND (USCS: SM, SC) [AASHTO: A-2-4, A-2-6] with trace amounts of gravel and interbedded thin lenses of Stratum IIa.

The SPT N-values for this stratum typically averaged 30-bpf and ranged from 6 to 55-bpf.

For Retaining Wall 1, site specific soil parameters were developed and used for design of the foundation elements. The Retaining Wall 1 soil parameters are summarized in Table 4.5 below. Table 4.6 summarizes the soil parameters used for the settlement analysis of this structure. Supporting calculations for development of soil parameters for Structure RW1 are provided in Appendix E.

Table 4.5 – Summary or Retaining Wall RW1 Soil Parameters				
Strata	Total Unit Weight – γ (pcf)	Drained		Undrained
		Angle of Friction ϕ (deg)	Cohesion – c (psf)	Shear - S_u (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 4.6 Summary of Retaining Wall RW1 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.

4.4.2 Retaining Wall RW2: Ramp G1

Table 4.7 summarizes the borings that were used for the evaluation of subsurface conditions for the Retaining Wall 2. 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: Retaining Wall RW2 - Ramp G1		
SPT Borings	CPT Probes	DMT Probes
IBR-31, IRW-43	-	-

FILL: FILL was not encountered within the limits of the proposed Retaining Wall 2.

Stratum I - Columbia Formation - Coarse Grained Soil: Stratum I was not encountered within the limits of the proposed Retaining Wall 2.

Stratum IIa – Cretaceous - Fine Grained Soil: Stratum IIa encountered within the limits of the proposed Retaining Wall 2 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, ML, CH)

[AASHTO: A-4, A-6, A-7-5]. Interbedded thin lenses of Stratum IIb are also classified as silty SAND (USCS: SM, SP-SM, SP) [AASHTO: A-2-4, A-3].

The Stratum IIa SPT N-values typically ranged from 6 to 100+ blows per foot, averaging 22-bpf. The natural moisture content averaged 21.3-percent and ranged from 16.2 to 26.2-percent. The liquid limit ranged from 25 to 62, averaging 42. The plasticity index ranged from 10 to 33, averaging 20. The average percentage of fines (percent passing the No. 200 sieve) ranged from 50 to 99-percent, averaging 86-percent.

Stratum IIb – Cretaceous Coarse Grained Soil: Stratum IIb encountered within the limits of the proposed Retaining Wall 2 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-3].

The SPT N-values for this stratum typically averaged 27-bpf and ranged from 5 to 94-bpf. The natural moisture content averaged 15.9-percent and ranged from 9.7 to 22.0-percent. The average percentage of fines (percent passing the No. 200 sieve) ranged from 18 to 32-percent, averaging 25-percent.
For Retaining Wall 2, site specific soil parameters were developed and used for design of the foundation elements. The Retaining Wall 2 soil parameters are summarized in Table 4.8 below. Table 4.9 summarizes the soil parameters used for the settlement analysis of this structure. Supporting calculations for development of soil parameters for Structure RW2 are provided in Appendix E.

Table 4.8 – Summary of Retaining Wall RW2 Soil Parameters				
Strata	**Total Unit Weight – γ (pcf)**	**Drained**		**Undrained**
		Angle of Friction ϕ (deg)	**Cohesion – c (psf)**	**Shear - S_u (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 4.9 – Summary of Retaining Wall RW2 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 Retaining Wall RW3: Ramp B

Table 4.10 summarizes the borings that were used for the evaluation of subsurface conditions for the Retaining Wall 3. 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: Retaining Wall RW3 - Ramp B		
SPT Borings	**CPT Probes**	**DMT Probes**
IBR-19, IBR-23, URS-4	ICPT-12	-

FILL: FILL was not encountered within the limits of the proposed Retaining Wall 3.

Stratum I - Columbia Formation - Coarse Grained Soil: Stratum I encountered within the limits of the proposed Retaining Wall 3 generally consisted of loose to dense SAND with varying percentages of silt and clay (USCS: SM, SC, SP-SM) [AASHTO: A-2-4, A-3, A-4].

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

Stratum IIa – Cretaceous - Fine Grained Soil: Stratum IIa encountered within the limits of the proposed Retaining Wall 3 generally consisted of soft to 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]. Interbedded thin lenses of Stratum IIb are also classified as silty SAND (USCS: SM, SP, SC) [AASHTO: A-2-4, A-1-b].

The Stratum IIa SPT N-values typically ranged from 5 to 59-bpf, averaging 22-bpf. The natural moisture content averaged 20.5-percent and ranged from 14.9 to 26.8-percent. The liquid limit ranged from 19 to 55, averaging 40. The plasticity index ranged from 5 to 30, averaging 19. The average percentage of fines (percent passing the No. 200 sieve) ranged from 42 to 99-percent, averaging 89-percent.

Stratum IIb – Cretaceous Coarse Grained Soil: Stratum IIb encountered within the limits of the proposed Retaining Wall 3 generally consisted of loose to very dense, SAND with varying percentages of silt and clay (USCS: SM, SP, SC, SP-SM, SC-SM) [AASHTO: A-2-4, A-1-b, A-4, A-3, A-2-6].

The SPT N-values for this stratum typically averaged 48-bpf and ranged from 7 to 100+ blows per foot. The natural moisture content averaged 23.1-percent and ranged from 18.7 to 30.1-percent. The average percentage of fines (percent passing the No. 200 sieve) ranged from 19 to 46-percent, averaging 30-percent.

For Retaining Wall 3, site specific soil parameters were developed and used for design of the foundation elements. The Retaining Wall 3 soil parameters are summarized in Table 4.11 below. Table 4.12 summarizes the soil parameters used for the settlement analysis of this structure. Supporting calculations for development of soil parameters for Structure RW3 are provided in Appendix E.

Table 4.11 – Summary of Retaining Wall RW3 Soil Parameters				
Strata	Total Unit Weight – γ (pcf)	Drained: Angle of Friction ϕ (deg)	Drained: Cohesion – c (psf)	Undrained: Shear - S_u (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 4.12 – Summary of Retaining Wall RW3 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 Retaining Wall RW4: SR 7 Northbound

Table 4.13 summarizes the borings that were used for the evaluation of Retaining Wall 4. The Summary of Subsurface Data for this structure is contained in Figure A-4a, located in Appendix A of this report.

Table 4.13 - Subsurface Exploration: Retaining Wall RW4 - SR 7 Northbound		
SPT Borings	CPT Probes	DMT Probes
IBR-08, IBR-09A, IBR-13, IBR-13A, IRW-29	ICPT-06, ICPT-07, ICPT-08	-

FILL: FILL materials extended to depths of 0-ft to 12-ft below the existing ground surface. The FILL material encountered within the project site consisted of medium stiff to very stiff CLAY and

SILT (USCS: ML, CL) [AASHTO: A-4]. Trace amounts of organic material were also encountered in this stratum.

The SPT N-values generally ranged from 7 to 25-bpf, with an average SPT N-value of 14.6-bpf. The natural moisture content averaged 15.9-percent and ranged from 14.4 to 17.3-percent. The percentage of fines (percent passing the No. 200 sieve) ranged from 76 to 91-percent, averaging 83.5-percent.

Stratum IIa – Cretaceous - Fine-Grained Soil: Stratum IIa encountered near the proposed Retaining Wall 4 generally consisted of medium stiff 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]. This stratum was interbedded with Stratum IIb.

The Stratum IIa SPT N-values ranged from 7 to 85-bpf, averaging 23.9-bpf. The natural moisture content averaged 18.7-percent and ranged from 11.3 to 29.1-percent. The liquid limit ranged from 19 to 56, averaging 32.3. The plasticity index ranged from 5 to 29, averaging 13. The percentage of fines (percent passing the No. 200 sieve) ranged from 53 to 100-percent, averaging 84.8-percent.

Stratum IIb – Cretaceous Coarse Grained Soil: Stratum IIb encountered near the proposed Retaining Wall 4 generally consisted of loose to very dense silty or clayey SAND with traces of lignite, clay, and/or mica (USCS: SP-SM, SM, SW-SM) [AASHTO: A-1-b, A-2-4, A-4].

The SPT N-values for this stratum typically averaged 51.3-bpf and ranged from 8 to 100 bpf. The natural moisture content averaged 18.5-percent and ranged from 14.6 to 21.8-percent. The percentage of fines (percent passing the No. 200 sieve) ranged from 11 to 53-percent, averaging 23.4-percent.

For Retaining Wall 4, site specific soil parameters were developed and used for design of the foundation elements. The Retaining Wall 4 soil parameters are summarized in Table 4.14 below. Table 4.15 summarizes the soil parameters used for the settlement analysis of this structure. Supporting calculations for development of soil parameters for Retaining Wall 4 are provided in Appendix E.

Table 4.14 – Retaining Wall RW4 Soil Parameters

Strata	Total Unit Weight – γ (pcf)	Drained		Undrained
		Angle of Friction – ϕ (deg)	Cohesion – c (psf)	Shear - S_u (psf)
IIa – EL 58 to 38	130	24	-	2,500
IIa – EL 38 to 18	130	24	-	2,500
IIa - EL 18 to 0	130	24	-	3,500
IIa- EL 0 to -15	130	24	-	5,000
IIb – Below EL -15	130	34	-	-

Table 4.15 – Retaining Wall RW4 Soil Parameters for Settlement

Strata	Total Unit Weight – γ (pcf)	μ	E (ksf)	C_c	C_r	e_o	C_v (ft^2/day)	OCR
IIa – EL 58 to 38	130	0.4	545	0.016	0.016	0.633	-	3.4
IIa – EL 38 to 18	130	0.4	818	0.016	0.016	0.654	-	2.2
IIa - EL 18 to 0	130	0.4	1,364	0.016	0.016	0.654	-	2.0
IIa- EL 0 to -15	130	0.4	1,364	0.016	0.016	0.654	-	2.0
IIb – Below EL -15	130	0.3	1,364	-	-	-	-	-

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 62, approximately 8 to 10-ft below the existing ground surface.

4.4.5 Retaining Wall RW5: Ramp B

Table 4.16 summarizes the borings that were used for the evaluation of Retaining Wall 5. The Summary of Subsurface Data for this structure is contained in Figure A-4b, located in Appendix A of this report.

Table 4.16 - Summary of Subsurface Exploration: Retaining Wall RW5 - Ramp B		
SPT Borings	**CPT Probes**	**DMT Probes**
IBR-26, IBR-26A, IBR-27A, IBR-42A, IBR-55	ICPT-16, ICPT-17	-

FILL: FILL materials extended to depths of 0-ft to 11-ft below the existing ground surface. The FILL material encountered within the project site consisted of loose SAND (USCS: SM) [AASHTO: A-2-4] stiff to very stiff sandy SILT (USCS: ML) [AASHTO: A-4]. Trace amounts of organic material were also encountered in this stratum.

The SPT N-values generally ranged from 10 to 23-bpf, with an average SPT N-value of 13.8-bpf.

Stratum I – Columbia Formation - Fine to Coarse-Grained Soil: Stratum I encountered within the limits of the proposed Retaining Wall 5 consisted of loose to dense silty SAND (USCS: SM, SP-SM) [AASHTO: A-1-b, A-2-4] and medium stiff to stiff CLAY (USCS: CL) [AASHTO: A-4]. Variable amounts of gravel, mica, and/or organics were also encountered.

The SPT N-values ranged from 4 to 22-bpf, with an average SPT N-value of 13.3-bpf. The natural moisture content averaged 16-percent and ranged from 11.5 to 20.4-percent. The percentage of fines (percent passing the No. 200 sieve) ranged from 11 to 76-percent, averaging 37.7-percent.

Stratum IIa – Cretaceous - Fine-Grained Soil: Stratum IIa encountered near the proposed Retaining Wall 5 generally consisted of medium stiff 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]. This stratum was interbedded with Stratum IIb.

The Stratum IIa SPT N-values ranged from 7 to 100-bpf, averaging 30.2-bpf. The natural moisture content averaged 19.2-percent and ranged from 5.9 to 35.7-percent. The liquid limit ranged from 22 to 61, averaging 35.8. The plasticity index ranged from 7 to 32, averaging 15.6. The percentage of fines (percent passing the No. 200 sieve) ranged from 54 to 99-percent, averaging 87.3-percent.

Stratum IIb – Cretaceous Coarse Grained Soil: Stratum IIb encountered near the proposed Retaining Wall 5 generally consisted of loose to very dense silty or clayey SAND with traces of lignite, gravel, and mica (USCS: SP-SM, SM, SW-SM) [AASHTO: A-1-b, A-2-4, A-3, A-4].

The SPT N-values for this stratum typically averaged 49.9-bpf and ranged from 10 to 100 bpf. The natural moisture content averaged 18.5-percent and ranged from 16.7 to 20.4-percent. The percentage of fines (percent passing the No. 200 sieve) ranged from 8 to 37-percent, averaging 18-percent.

For Retaining Wall 5, site specific soil parameters were developed and used for design of the foundation elements. The Retaining Wall 5 soil parameters are summarized in Table 4.17 below. Table 4.18 summarizes the soil parameters used for the settlement analysis of this structure. Supporting calculations for development of soil parameters for Retaining Wall 5 are provided in Appendix E.

Table 4.17 – Retaining Wall RW5 Soil Parameters

Strata	Total Unit Weight – γ (pcf)	Drained		Undrained
		Angle of Friction – ϕ (deg)	Cohesion – c (psf)	Shear - S_u (psf)
I – Above EL 65	125	32	-	
IIa – EL 65 to 40	130	24	-	1,500
IIa - EL 40 to 20	130	24	-	2,500
IIa- EL 20 to 0	130	24	-	3,500
IIa – Below EL 0	130	24	-	5,000

Table 4.18 – Retaining Wall RW5 Soil Parameters for Settlement

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

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

4.4.6 Retaining Wall RW6: Ramp C

Table 4.19 summarizes the borings that were used for the evaluation of Retaining Wall 6. The Summary of Subsurface Data for this structure is contained in Figure A-4c, located in Appendix A of this report.

Table 4.19 - Subsurface Exploration: Retaining Wall RW6 - Ramp C

SPT Borings	CPT Probes	DMT Probes
IBR-18A, IBR-18B, IBR-18C, IBR-31, IRW-43, IRW-97	-	IDMT-20

Stratum I – Columbia Formation - Fine to Coarse-Grained Soil: Stratum I encountered within the limits of the proposed Retaining Wall 6 consisted of loose to dense silty SAND (USCS: SC-SM, SM, SP-SC) [AASHTO: A-1-b, A-2-4] and medium stiff to stiff CLAY (USCS: CL) [AASHTO: A-6]. Trace amounts of gravel and organics were also encountered.

The SPT N-values ranged from 5 to 34-bpf, with an average SPT N-value of 12-bpf. The natural moisture content averaged 14.8-percent and ranged from 9.7 to 17.8-percent. The percentage of fines (percent passing the No. 200 sieve) ranged from 14 to 65-percent, averaging 32.3-percent.

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

The Stratum IIa SPT N-values ranged from 6 to 53-bpf, averaging 20.2-bpf. The natural moisture content averaged 22.1-percent and ranged from 13.8 to 31.9-percent. The liquid limit ranged from 23 to 67, averaging 45.5. The plasticity index ranged from 8 to 37, averaging 21.5. The percentage of fines (percent passing the No. 200 sieve) ranged from 76 to 100-percent, averaging 91.2-percent.

Stratum IIb – Cretaceous Coarse Grained Soil: Stratum IIb encountered near the proposed Retaining Wall 6 generally consisted of medium dense to very dense silty or clayey SAND with traces of lignite and mica (USCS: SC-SM, SM, SW-SM) [AASHTO: A-2-4, A-4]. Interbedded thin lenses of Stratum IIb are also classified as a very stiff SILT with varying percentages of Sand (USCS: ML) [AASHTO: A-4].

The SPT N-values ranged from 23 to 114-bpf, with an average SPT N-value of 67.6-bpf. The natural moisture content averaged 17.2-percent and ranged from 13.2 to 22-percent. The Liquid limit of select samples from this area ranged from Non-Plastic to 25 and the Plasticity Index ranged from Non-Plastic to 10. The percentage of fines (percent passing the No. 200 sieve) ranged from 12 to 59-percent, averaging 29.5-percent.

For Retaining Wall 6, site specific soil parameters were developed and used for design of the foundation elements. The Retaining Wall 6 soil parameters are summarized in Table 4.20 below. Table 4.21 summarizes the soil parameters used for the settlement analysis of this structure. Supporting calculations for development of soil parameters for Retaining Wall 6 are provided in Appendix E.

Table 4.20 – Retaining Wall RW6 Soil Parameters

		Drained		Undrained
Strata	Total Unit Weight – γ (pcf)	Angle of Friction – ϕ (deg)	Cohesion – c (psf)	Shear - S_u (psf)
Select Fill	125	32	-	-
I – EL 79 to 63	125	32	-	-
IIa – EL 63 to 40	130	24	-	1,200
IIa – Below EL 40	130	24	-	1,700

Table 4.21 – Retaining Wall RW6 Soil Parameters for Settlement								
Strata	**Total Unit Weight – γ (pcf)**	**μ**	**E (ksf)**	**C_c**	**C_r**	**e_o**	**C_v (ft^2/day)**	**OCR**
IIa – Above EL 50	130	0.4	545	0.16	0.016	0.633	-	3.7
IIa – EL 50 to 20	130	0.4	818	0.16	0.016	0.654	-	2.7
IIa – EL 20 to 0	130	0.4	1,364	0.16	0.016	0.654	-	2.0
IIa – EL 0 to -20	130	0.4	1,364	0.16	0.016	0.654	-	-
IIb – Below EL -20	130	0.3	1,364	-	-	-	-	-

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 62, approximately 5 to 15-ft below the existing ground surface.

4.4.7 Retaining Wall RW7: Ramp G1

Table 4.22 summarizes the borings that were used for the evaluation of Retaining Wall 7. The Summary of Subsurface Data for this structure is contained in Figures A-4d and A-4f, located in Appendix A of this report.

Table 4.22 - Subsurface Exploration: Retaining Wall RW7 - Ramp G1		
SPT Borings	**CPT Probes**	**DMT Probes**
IBR-04, IBR-05, IRW-15, IRW-16, IRW-17, IRW-18, IRW-20, IRW-21, IRW-47, IRW-47A, IRW-48, ISC-35	-	IDMT-06, IMDT-07

Stratum I – Columbia Formation - Fine to Coarse-Grained Soil: Stratum I encountered within the limits of the proposed Retaining Wall 7 consisted of loose to dense silty or clayey SAND (USCS: SM) [AASHTO: A-1-b, A-2-4, A-4] and medium stiff to very stiff SILT and silty CLAY (USCS: CL-ML, ML) [AASHTO: A-4, A-6]. Variable amounts of gravel and organics were also encountered.

The SPT N-values ranged from 6 to 48-bpf, with an average SPT N-value of 15.5-bpf. Laboratory testing was not performed on Stratum I samples collected from the borings in the vicinity of Retaining Wall 7.

Stratum IIa – Cretaceous - Fine-Grained Soil: Stratum IIa encountered near the proposed Retaining Wall 7 generally consisted of medium stiff to hard CLAY and SILT with varying percentages of sand, lignite, and mica (USCS: CL, CL-ML, CH, ML, OL) [AASHTO: A-4, A-6, A-7-5, A-8]. This stratum was interbedded with Stratum IIb.

The Stratum IIa SPT N-values ranged from 5 to 73-bpf, averaging 24.5-bpf. The natural moisture content averaged 19.9-percent and ranged from 12.3 to 28-percent. The liquid limit ranged from 21 to 58, averaging 35.7. The plasticity index ranged from 3 to 29, averaging 14.7. The percentage of fines (percent passing the No. 200 sieve) ranged from 59 to 99-percent, averaging 86.9-percent.

Stratum IIb – Cretaceous Coarse Grained Soil: Stratum IIb encountered near the proposed Retaining Wall 7 generally consisted of loose to very dense clean, silty, or clayey SAND with traces of lignite and mica (USCS: SC-SM, SP-SM, SM, SW-SM) [AASHTO: A-1-a, A-1-b, A-2-4] and very stiff to hard non-plastic Sandy SILT (USCS: ML) [AASHTO: A-4].

The SPT N-values for this stratum typically averaged 43.8-bpf and ranged from 10 to 138-bpf. The natural moisture content averaged 15.8-percent and ranged from 4.1 to 24-percent. The percentage of fines (percent passing the No. 200 sieve) ranged from 8 to 81-percent, averaging 35.3-percent.

For Retaining Wall 7, site specific soil parameters were developed and used for design of the foundation elements. The Retaining Wall 7 soil parameters are summarized in Table 4.23 below. Table 4.24 summarizes the soil parameters used for the settlement analysis of this structure, and Figure A-4f in Appendix A shows the borings used. Supporting calculations for development of soil parameters for Retaining Wall RW7 are provided in Appendix E.

Table 4.23 – Retaining Wall RW7 Soil Parameters				
		Drained		Undrained
Strata	Total Unit Weight – γ (pcf)	Angle of Friction – ϕ (deg)	Cohesion – c (psf)	Shear - S_u (psf)
IIa – EL 62.5 to 39	130	24	-	2,000
IIb – EL 39 to 29	130	34	-	-
IIa - EL 29 to 19	130	18	-	2,500
IIb – Below EL 19	130	34	-	-

Table 4.24 – Retaining Wall RW7 Soil Parameters for Settlement								
Strata	Total Unit Weight – γ (pcf)	μ	E (ksf)	C_c	C_r	e_o	C_v (ft^2/day)	OCR
IIa – EL 60 to 50	130	0.4	818	0.015	0.015	0.49	-	3.5
IIa – EL 50 to 40	130	0.4	545	0.015	0.015	0.49	-	3.5
IIb - EL 40 to 30	130	0.3	1,364	-	-	-	-	-
IIa - EL 30 to 17	130	0.4	818	0.015	0.015	0.66	-	1.0
IIb – Below EL 17	130	0.3	1,364	-	-	-	-	-
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 62.5, approximately 3 to 5-ft below the existing ground surface.

4.4.8 Retaining Wall RW8: SR 7 Southbound/Ramp C1

Table 4.25 summarizes the borings that were used for the evaluation of Retaining Wall 8. The Summary of Subsurface Data for this structure is contained in Figures A-4e and A-4f, located in Appendix A of this report.

Table 4.25 - Subsurface Exploration: Retaining Wall RW8 - SR 7 Southbound/Ramp C1		
SPT Borings	**CPT Probes**	**DMT Probes**
IBR-04, IBR-05, IRW-17, IRW-18, IRW-19, IRW-20, IRW-21	-	IDMT-06, IDMT-07

Stratum I – Columbia Formation - Fine to Coarse-Grained Soil: Stratum I encountered within the limits of the proposed Retaining Wall 8 consisted of loose to dense silty or clayey SAND (USCS: SM) [AASHTO: A-1-b, A-2-4, A-4] and medium stiff to very stiff SILT and silty CLAY (USCS: CL-ML, ML) [AASHTO: A-4, A-6]. Variable amounts of gravel and organics were also encountered.

The SPT N-values ranged from 6 to 48-bpf, with an average SPT N-value of 15.5-bpf.

Stratum IIa – Cretaceous - Fine-Grained Soil: Stratum IIa encountered near the proposed Retaining Wall 8 generally consisted of medium stiff to hard CLAY and SILT with varying percentages of sand, lignite, and mica (USCS: CL, CL-ML, CH, ML, OL) [AASHTO: A-4, A-6, A-7-5, A-8]. This stratum was interbedded with Stratum IIb.

The Stratum IIa SPT N-values ranged from 5 to 73-bpf, averaging 24.5-bpf. The natural moisture content averaged 19.9-percent and ranged from 12.3 to 28-percent. The liquid limit ranged from 21 to 58, averaging 35.7. The plasticity index ranged from 3 to 29, averaging 14.7. The percentage of fines (percent passing the No. 200 sieve) ranged from 59 to 99-percent, averaging 86.9-percent.

Stratum IIb – Cretaceous Coarse Grained Soil: Stratum IIb encountered near the proposed Retaining Wall 8 generally consisted of loose to very dense clean, silty, or clayey SAND with traces of lignite and mica (USCS: SC-SM, SP-SM, SM, SW-SM) [AASHTO: A-1-a, A-1-b, A-2-4] and very stiff to hard Sandy SILT (USCS: ML) [AASHTO: A-4].

The SPT N-values for this stratum typically averaged 43.8-bpf and ranged from 10 to 138-bpf. The natural moisture content averaged 15.8-percent and ranged from 4.1 to 24-percent. The percentage of fines (percent passing the No. 200 sieve) ranged from 8 to 81-percent, averaging 35.3-percent.

For Retaining Wall 8, site specific soil parameters were developed and used for design of the foundation elements. The Retaining Wall 8 soil parameters are summarized in Table 4.26 below. Table 4.27 summarizes the soil parameters used for the settlement analysis of this structure, and Figure A-4f in Appendix A shows the borings used. Supporting calculations for development of soil parameters for Retaining Wall RW8 are provided in Appendix E.

Table 4.26 – Retaining Wall RW8 Soil Parameters

Strata	Total Unit Weight – γ (pcf)	Drained		Undrained
		Angle of Friction – ϕ (deg)	Cohesion – c (psf)	Shear - S_u (psf)
IIa – EL 62.0 to 12	130	24	-	2,000
IIb – Below EL 12	130	34	-	-

Table 4.27 – Retaining Wall RW8 Soil Parameters for Settlement

Strata	Total Unit Weight – γ (pcf)	μ	E (ksf)	C_c	C_r	e_o	C_v (ft^2/day)	OCR
IIa – EL 60 to 50	130	0.4	818	0.015	0.015	0.49	-	3.5
IIa – EL 50 to 40	130	0.4	545	0.015	0.015	0.49	-	3.5
IIb - EL 40 to 30	130	0.3	1,364	-	-	-	-	-
IIa - EL 30 to 17	130	0.4	818	0.015	0.015	0.66	-	1.0
IIb – Below EL 17	130	0.3	1,364	-	-	-	-	-

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 62.5, approximately 3 to 5-ft below the existing ground surface.

4.4.9 Retaining Wall RW9: Ramp A

Table 4.28 summarizes the borings that were used for the evaluation of Retaining Wall 9. The Summary of Subsurface Data for this structure is contained in Figure A-4g, located in Appendix A of this report.

Table 4.28 - Subsurface Exploration: Retaining Wall RW9 - Ramp A		
SPT Borings	**CPT Probes**	**DMT Probes**
IRW-22, IRW-24, IRW-46, IRW-60, IRW-61, IRW-62, IRW-63, ISC-44	-	IDMT-08, IDMT-16

FILL: FILL materials were encountered in Borings IRW-46, IRW-60, IRW-61, and IRW-62. The Fill material extended to depths of 2.5 to 7.5-ft below the existing ground surface, which corresponds to elevations that are above the proposed bottom of leveling pad, to a depth of approximately 3.0-ft below the proposed bottom of leveling pad.

The FILL material encountered within the project site consisted of medium stiff to very stiff SILT and Silty CLAY (USCS: CL-ML, ML) [AASHTO: A-4] as well as loose to medium dense Silty SAND (USCS: SM) [AASHTO: A-2-4]. Varying amounts of organic material and gravel were also encountered in this stratum.

The SPT N-values generally ranged from 8 to 22-bpf, with an average SPT N-value of 12.8-bpf.

Stratum IIa – Cretaceous - Fine-Grained Soil: Stratum IIa encountered near the proposed Retaining Wall 9 generally consisted of medium stiff 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-8]. This stratum was interbedded with Stratum IIb.

The Stratum IIa SPT N-values ranged from 7 to 33-bpf, averaging 16.2-bpf. The natural moisture content averaged 18.4-percent and ranged from 13.9 to 25.5-percent. The liquid limit ranged from 23 to 57, averaging 37.4. The plasticity index ranged from 4 to 30, averaging 16.3. The percentage of fines (percent passing the No. 200 sieve) ranged from 51 to 100-percent, averaging 87.7-percent.

Stratum IIb – Cretaceous Coarse Grained Soil: Stratum IIb encountered near the proposed Retaining Wall 9 generally consisted of loose to very dense silty SAND with traces of lignite, clay, and/or mica (USCS: SP-SM, SM) [AASHTO: A-2-4].

The SPT N-values for this stratum typically averaged 21.2-bpf and ranged from 8 to 100-bpf. The natural moisture content averaged 19-percent and ranged from 14.1 to 25.6-percent. The percentage of fines (percent passing the No. 200 sieve) ranged from 11 to 12-percent, averaging 11.5-percent.

For Retaining Wall 9, site specific soil parameters were developed and used for design of the foundation elements. The Retaining Wall 9 soil parameters are summarized in Table 4.29 below. Table 4.30 summarizes the soil parameters used for the settlement analysis of this structure. Supporting calculations for development of soil parameters for Retaining Wall 9 are provided in Appendix E.

Table 4.29 – Retaining Wall RW9 Soil Parameters

		Drained		Undrained
Strata	**Total Unit Weight – γ (pcf)**	**Angle of Friction – ϕ (deg)**	**Cohesion – c (psf)**	**Shear - S_u (psf)**
IIa- EL 63 to 45	130	24	-	2,500
IIa- EL 45 to 30	130	34	-	-
IIa- EL 30 to -27	130	24	-	2,500

Table 4.30 – Retaining Wall RW9 Soil Parameters for Settlement

Strata	**Total Unit Weight – γ (pcf)**	**μ**	**E (ksf)**	**C_c**	**C_r**	**e_o**	**C_v (ft^2/day)**	**OCR**
IIa- EL 63 to 45	130	0.4	545	0.118	0.016	0.633	-	3.7
IIa- EL 45 to 30	130	0.3	1,364	0.116	-	-	-	2.7
IIa- EL 30 to -27	130	0.4	1,364	0.116	0.016	0.654	-	2.0

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 54, approximately 5 to 15-ft below the existing ground surface.

4.4.10 Retaining Wall RW10: Ramp C

Table 4.31 summarizes the borings that were used for the evaluation of Retaining Wall 10. The Summary of Subsurface Data for this structure is contained in Figure A-4h, located in Appendix A of this report.

Table 4.31 - Subsurface Exploration: Retaining Wall RW10 - Ramp C		
SPT Borings	**CPT Probes**	**DMT Probes**
IBR-15, IRW-87, IRW-88, URS-1	ICPT-04	-

FILL: FILL materials were encountered in all the borings drilled within the vicinity of Retaining Wall 10. The Fill materials extended from depths of 4.5-ft to 7.1-ft below the existing ground surface, which corresponds to elevations that are above the proposed bottom of leveling pad to a depth of approximately 2.0-ft below the proposed bottom of leveling pad.

The FILL material encountered within the project site consisted of medium dense to dense SAND (USCS: SP, SM) [AASHTO: SM, A-2-4]. Trace amounts of silt and gravel were also encountered in this stratum.

The SPT N-values generally ranged from 26 to 49-bpf, with an average SPT N-value of 33.4-bpf.

Stratum I – Columbia Formation - Fine to Coarse-Grained Soil: Stratum I encountered within the limits of the proposed Retaining Wall 10 consisted of medium dense silty or clayey SAND (USCS: SC, SC-SM, SM) [AASHTO: A-2-4, A-4, A-6] and medium stiff to stiff SILT (USCS: ML) [AASHTO: A-4]. Traces of gravel and ironstone were also encountered.

The SPT N-values ranged from 13 to 29-bpf, with an average SPT N-value of 19.8-bpf.

Stratum IIa – Cretaceous - Fine-Grained Soil: Stratum IIa encountered near the proposed Retaining Wall 10 generally consisted of very stiff to hard CLAY and SILT with varying percentages of sand, lignite, and mica (USCS: CL, ML) [AASHTO: A-4, A-6]. This stratum was interbedded with Stratum IIb.

The Stratum IIa SPT N-values ranged from 19 to 51-bpf, averaging 33.4-bpf.

Stratum IIb – Cretaceous Coarse Grained Soil: Stratum IIb encountered near the proposed Retaining Wall 10 generally consisted of medium dense to very dense silty or clayey SAND with variable amounts of gravel and mica (USCS: SC, SM) [AASHTO: A-2-4, A-4].

The SPT N-values for this stratum typically averaged 27.2-bpf and ranged from 11 to 50 bpf.

For Retaining Wall 10, site specific soil parameters were developed and used for design of the foundation elements. The Retaining Wall 10 soil parameters are summarized in Table 4.32

below. Table 4.33 summarizes the soil parameters used for the settlement analysis of this structure. Supporting calculations for development of soil parameters for Retaining Wall 10 are provided in Appendix E.

Table 4.32 – Summary of Retaining Wall RW10 Soil Parameters

Strata	Total Unit Weight – γ (pcf)	Drained: Angle of Friction – ϕ (deg)	Drained: Cohesion – c (psf)	Undrained: Shear - S_u (psf)
IIa - EL 69 to 40	130	24	-	2,000
IIa - EL 40 to 20	130	24	-	3,000
IIa - EL 20 to 11	130	24	-	4,000
IIb – Below EL 11	130	34	-	-

Table 4.33 – Retaining Wall RW10 Soil Parameters for Settlement

Strata	Total Unit Weight – γ (pcf)	μ	E (ksf)	C_c	C_r	e_o	C_v (ft^2/day)	OCR
IIa – Above EL 40	130	0.4	818	0.17	0.01	0.77	0.15	3.3
IIa - EL 40 to 20	130	0.4	818	0.15	0.01	0.77	0.15	2.1
IIa - EL 20 to 11	130	0.4	1364	0.08	0.01	0.77	0.15	2.1
IIb – Below EL 11	130	0.3	1364	-	-	-	0.15	-

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 69, approximately 5-ft below the existing ground surface.

4.4.11 Retaining Wall RW11

This retaining wall has been eliminated.

4.4.12 Retaining Wall RW12: Ramp A

Table 4.34 summarizes the borings that were used for the evaluation of Retaining Wall 12. The Summary of Subsurface Data for this structure is contained in Figure A-4i and Figure A-4j, located in Appendix A of this report.

Table 4.34 - Subsurface Exploration: Retaining Wall RW12 - Ramp A		
SPT Borings	**CPT Probes**	**DMT Probes**
IRW-69, IRW-70, IRW-71, IRW-72, URS-3	ICPT-21, ICPT-21A	IDMT-21, IDMT-22

Stratum I – Columbia Formation - Fine to Coarse-Grained Soil: Stratum I encountered within the limits of the proposed Retaining Wall 12 consisted of loose to dense silty SAND

(USCS: SM) [AASHTO: A-1-b, A-2-4] and medium stiff to hard SILT and CLAY (USCS: CL, ML) [AASHTO: A-4, A-6]. Trace amounts of gravel, mica, and/or organics were also encountered.

The SPT N-values ranged from 5 to 32-bpf, with an average SPT N-value of 15-bpf. The natural moisture content averaged 15.7-percent and ranged from 9.1 to 22-percent. The percentage of fines (percent passing the No. 200 sieve) ranged from 14 to 97-percent, averaging 39.3-percent.

Stratum IIa – Cretaceous - Fine-Grained Soil: Stratum IIa encountered near the proposed Retaining Wall 12 generally consisted of stiff to hard CLAY and SILT with varying percentages of sand, lignite, and mica (USCS: CL, CL-ML, CH, ML, OL) [AASHTO: A-4, A-6, A-7-5, A-8]. This stratum was interbedded with Stratum IIb.

The Stratum IIa SPT N-values ranged from 12 to 86-bpf, averaging 28.1-bpf. The natural moisture content averaged 18.6-percent and ranged from 17.5 to 21.4-percent. The liquid limit ranged from 22 to 52, averaging 29. The plasticity index ranged from 5 to 31, averaging 11.8. The percentage of fines (percent passing the No. 200 sieve) ranged from 60 to 99-percent, averaging 73.2-percent.

Stratum IIb – Cretaceous Coarse Grained Soil: Stratum IIb encountered near the proposed Retaining Wall 12 generally consisted of loose to very dense silty SAND with traces of lignite, clay, and/or mica (USCS: SP-SM, SM) [AASHTO: A-1-b, A-2-4].

The SPT N-values for this stratum typically averaged 33.8-bpf and ranged from 10 to 100 bpf. The natural moisture content averaged 26.6-percent and ranged from 25.7 to 27.5-percent. The percentage of fines (percent passing the No. 200 sieve) ranged from 17 to 26-percent, averaging 21.5-percent.

For Retaining Wall 12, site specific soil parameters were developed and used for design of the foundation elements. The Retaining Wall 12 soil parameters are summarized in Table 4.35 below. Table 4.36 summarizes the soil parameters used for the settlement analysis of this structure. Supporting calculations for development of soil parameters for Retaining Wall RW12 are provided in Appendix E.

Table 4.35 – Summary of Retaining Wall RW12 Soil Parameters

		Drained		Undrained
Strata	Total Unit Weight – γ (pcf)	Angle of Friction – ϕ (deg)	Cohesion – c (psf)	Shear - S_u (psf)
I - EL 78 to 73	125	32	0	-
IIa - EL 73 to 50	130	34	0	-
IIa - EL 50 to 40	130	17	0	1,500
IIa - EL 40 to 20	130	17	0	2,500
IIa - EL 20 to 0	130	17	0	3,500

Table 4.36 – Retaining Wall RW12 Soil Parameters for Settlement

Strata	Total Unit Weight – γ (pcf)	μ	E (ksf)	C_c	C_r	e_o	C_v (ft^2/day)	OCR
I - EL 78 to 73	125	0.3	1,500	-	-	-	-	-
IIa - EL 73 to 50	130	0.3	1,364	-	-	-	-	-
IIa - EL 50 to -11	130	0.4	818	0.02	0.02	0.614	0.2086	3.0
IIb - EL -11 to -36	130	0.3	1,364	-	-	-	-	-
IIa - EL -36 to -146	130	0.4	1,364	0.016	0.016	0.654	0.2086	1.0

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 76, approximately 5 to 10-ft below the existing ground surface

4.4.13 Retaining Wall RW13

Table 4.37 summarizes the borings that were used for the evaluation of Retaining Wall 13. The Summary of Subsurface Data for this structure is contained in Figure A-4i and Figure A-4j, located in Appendix A of this report.

Table 4.37 - Subsurface Exploration: Retaining Wall RW13 - Ramp A

SPT Borings	CPT Probes	DMT Probes
IRW-69, IRW-70, IRW-71, IRW-72, URS-3	ICPT-21, ICPT-21A	IDMT-21, IDMT-22

Stratum I – Columbia Formation - Fine to Coarse-Grained Soil: Stratum I encountered within the limits of the proposed Retaining Wall 13 consisted of loose to dense silty SAND (USCS: SM) [AASHTO: A-1-b, A-2-4] and medium stiff to hard SILT and CLAY (USCS: CL, ML) [AASHTO: A-4, A-6]. Trace amounts of gravel, mica, and/or organics were also encountered.

The SPT N-values ranged from 5 to 32-bpf, with an average SPT N-value of 15-bpf. The natural moisture content averaged 15.7-percent and ranged from 9.1 to 22-percent. The percentage of fines (percent passing the No. 200 sieve) ranged from 14 to 97-percent, averaging 39.3-percent.

Stratum IIa – Cretaceous - Fine-Grained Soil: Stratum IIa encountered near the proposed Retaining Wall 13 generally consisted of stiff to hard CLAY and SILT with varying percentages of sand, lignite, and mica (USCS: CL, CL-ML, CH, ML, OL) [AASHTO: A-4, A-6, A-7-5, A-8]. This stratum was interbedded with Stratum IIb.

The Stratum IIa SPT N-values ranged from 11 to 47-bpf, averaging 24-bpf. The natural moisture content averaged 18.5-percent and ranged from 16 to 21.4-percent. The liquid limit ranged from 22 to 52, averaging 29.7. The plasticity index ranged from 5 to 31, averaging 12. The percentage of fines (percent passing the No. 200 sieve) ranged from 60 to 99-percent, averaging 77.2-percent.

Stratum IIb – Cretaceous Coarse Grained Soil: Stratum IIb encountered near the proposed Retaining Wall 13 generally consisted of loose to very dense silty SAND with traces of lignite, clay, and/or mica (USCS: SP-SM, SM) [AASHTO: A-1-b, A-2-4].

The SPT N-values for this stratum typically averaged 20.3-bpf and ranged from 6 to 54 bpf. The natural moisture content averaged 20.2-percent and ranged from 7.4 to 27.5-percent. The percentage of fines (percent passing the No. 200 sieve) ranged from 11 to 26-percent, averaging 18-percent.

For Retaining Wall 13, site specific soil parameters were developed and used for design of the foundation elements. The Retaining Wall 13 soil parameters are summarized in Table 4.38 below. Table 4.39 summarizes the soil parameters used for the settlement analysis of this structure. Supporting calculations for development of soil parameters for Retaining Wall RW13 are provided in Appendix E.

Table 4.38 – Summary of Retaining Wall RW13 Soil Parameters

Strata	Total Unit Weight – γ (pcf)	Drained		Undrained
		Angle of Friction – ϕ (deg)	Cohesion – c (psf)	Shear - S_u (psf)
I - EL 78 to 73	125	32	0	-
IIa - EL 73 to 50	130	34	0	-
IIa - EL 50 to 40	130	17	0	1,500
IIa - EL 40 to 20	130	17	0	2,500
IIa - EL 20 to 0	130	17	0	3,500

Table 4.39 – Retaining Wall RW13 Soil Parameters for Settlement

Strata	Total Unit Weight – γ (pcf)	μ	E (ksf)	C_c	C_r	e_o	C_v (ft^2/day)	OCR
I - EL 78 to 73	125	0.3	1,500	-	-	-	-	-
IIa - EL 73 to 50	130	0.3	1,364	-	-	-	-	-
IIa - EL 50 to -11	130	0.4	818	0.02	0.02	0.614	0.2086	3.0
IIb - EL -11 to -36	130	0.3	1,364	-	-	-	-	-
IIa - EL -36 to -146	130	0.4	1,364	0.016	0.016	0.654	0.2086	1.0

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 76, approximately 5 to 10-ft below the existing ground surface

4.4.14 Retaining Wall RW14

Table 4.40 summarizes the borings that were used for the evaluation of Retaining Wall 14. The Summary of Subsurface Data for this structure is contained in Figure A-4k, located in Appendix A of this report.

Table 4.40 - Subsurface Exploration: Retaining Wall RW14 - Ramp B		
SPT Borings	**CPT Probes**	**DMT Probes**
IBR-61, IRW-89, IRW-90, IRW-91	-	-

Stratum I – Columbia Formation - Fine to Coarse-Grained Soil: Stratum I encountered within the limits of the proposed Retaining Wall 14 consisted of very loose to very dense silty SAND (USCS: SC-SM, SM, SP-SM, SW-SM) [AASHTO: A-1-b, A-2-4, A-3, A-4] and medium stiff to very stiff SILT and CLAY (USCS: CL, CL-ML) [AASHTO: A-4, A-6]. Trace amounts of gravel and/or organics were also encountered.

The SPT N-values ranged from 3 to 100-bpf, with an average SPT N-value of 14.7-bpf. The natural moisture content averaged 17-percent and ranged from 7.4 to 29.3-percent. The percentage of fines (percent passing the No. 200 sieve) ranged from 7 to 97-percent, averaging 39.5-percent.

Stratum IIa – Cretaceous - Fine-Grained Soil: Stratum IIa encountered near the proposed Retaining Wall 14 generally consisted of medium stiff 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 5 to 44-bpf, averaging 20.1-bpf. The natural moisture content averaged 20.9-percent and ranged from 14.6 to 32.2-percent. The liquid limit ranged from 21 to 69, averaging 34.3. The plasticity index ranged from 5 to 35, averaging 14.9. The percentage of fines (percent passing the No. 200 sieve) ranged from 55 to 99-percent, averaging 81.9-percent.

Stratum IIb – Cretaceous Coarse Grained Soil: Stratum IIb encountered near the proposed Retaining Wall 14 generally consisted of medium dense to very dense silty SAND with traces of lignite, clay, and/or mica (USCS: SM) [AASHTO: A-2-4].

The SPT N-values for this stratum typically averaged 26.4-bpf and ranged from 13 to 54 bpf.

For Retaining Wall 14, site specific soil parameters were developed and used for design of the foundation elements. The Retaining Wall 14 soil parameters are summarized in Table 4.41 below. Table 4.42 summarizes the soil parameters used for the settlement analysis of this structure. Supporting calculations for development of soil parameters for Retaining Wall RW14 are provided in Appendix E.

Table 4.41 – Summary of Retaining Wall RW14 Soil Parameters

Strata	Total Unit Weight – γ (pcf)	Drained		Undrained
		Angle of Friction – ϕ (deg)	Cohesion – c (psf)	Shear - S_u (psf)
Ia – Below EL 79	130	32	-	-
IIa - EL 79 to 40	130	24	-	1,700
IIa - EL 40 to 20	130	24	-	2,500
IIa - EL 20 to 0	130	24	-	3,500
IIa – Below EL 0	130	24	-	5,000

Table 4.42 – Retaining Wall RW14 Soil Parameters for Settlement

Strata	Total Unit Weight – γ (pcf)	μ	E (ksf)	C_c	C_r	e_o	C_v (ft^2/day)	OCR
Ia – 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 5 to 10-ft below the existing ground surface.

4.4.15 Retaining Wall RW15

Table 4.43 summarizes the borings that were used for the evaluation of Retaining Wall 15. The Summary of Subsurface Data for this structure is contained in Figure A-4k, located in Appendix A of this report.

Table 4.43 - Summary of Subsurface Exploration: Retaining Wall RW15 - Ramp B		
SPT Borings	**CPT Probes**	**DMT Probes**
IBR-61, IRW-89, IRW-90, IRW-91	-	-

Stratum I – Columbia Formation - Fine to Coarse-Grained Soil: Stratum I encountered within the limits of the proposed Retaining Wall 15 consisted of very loose to very dense silty SAND (USCS: SC-SM, SM, SP-SM, SW-SM) [AASHTO: A-1-b, A-2-4, A-3, A-4] and medium stiff to very stiff SILT and CLAY (USCS: CL, CL-ML, ML) [AASHTO: A-4, A-6]. Trace amounts of gravel, mica, and/or organics were also encountered.

The SPT N-values ranged from 3 to 100-bpf, with an average SPT N-value of 15-bpf. The natural moisture content averaged 16.4-percent and ranged from 7.4 to 29.3-percent. The percentage of fines (percent passing the No. 200 sieve) ranged from 7 to 97-percent, averaging 39.5-percent.

Stratum IIa – Cretaceous - Fine-Grained Soil: Stratum IIa encountered near the proposed Retaining Wall 14 generally consisted of medium stiff 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 5 to 44-bpf, averaging 18.8-bpf. The natural moisture content averaged 20.5-percent and ranged from 14.6 to 32.2-percent. The liquid limit ranged from 21 to 69, averaging 34.2. The plasticity index ranged from 5 to 36, averaging 15. The percentage of fines (percent passing the No. 200 sieve) ranged from 55 to 99-percent, averaging 81.9-percent.

Stratum IIb – Cretaceous Coarse Grained Soil: Stratum IIb encountered near the proposed Retaining Wall 14 generally consisted of medium dense to very dense silty SAND with traces of lignite, clay, and/or mica (USCS: SM) [AASHTO: A-2-4].

The SPT N-values for this stratum typically averaged 27.4-bpf and ranged from 13 to 54 bpf.

For Retaining Wall 15, site specific soil parameters were developed and used for design of the foundation elements. The Retaining Wall 15 soil parameters are summarized in Table 4.44 below. Table 4.45 summarizes the soil parameters used for the settlement analysis of this structure. Supporting calculations for development of soil parameters for Retaining Wall RW15 are provided in Appendix E.

Table 4.44 – Retaining Wall RW15 Soil Parameters

Strata	Total Unit Weight – γ (pcf)	Drained		Undrained
		Angle of Friction – ϕ (deg)	Cohesion – c (psf)	Shear - S_u (psf)
Ia – Below EL 79	130	32	-	-
IIa - EL 79 to 40	130	24	-	1,700
IIa - EL 40 to 20	130	24	-	2,500
IIa - EL 20 to 0	130	24	-	3,500
IIa – Below EL 0	130	24	-	5,000

Table 4.45 – Retaining Wall RW15 Soil Parameters for Settlement

Strata	Total Unit Weight – γ (pcf)	μ	E (ksf)	C_c	C_r	e_o	C_v (ft^2/day)	OCR
Ia – 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 5 to 10-ft below the existing ground surface.

4.4.16 Retaining Wall RW16

Table 4.46 summarizes the borings that were used for the evaluation of Retaining Wall 16. The Summary of Subsurface Data for this structure is contained in Figure A-4c, located in Appendix A of this report.

Table 4.46 - Summary of Subsurface Exploration: Retaining Wall RW16 - NB SR 7		
SPT Borings	**CPT Probes**	**DMT Probes**
IBR-18A, IBR-18B, IBR-18C, IBR-31, IRW-43, IRW-97	-	IDMT-20

Stratum I – Columbia Formation - Fine to Coarse-Grained Soil: Stratum I encountered within the limits of the proposed Retaining Wall 16 consisted of loose to dense silty SAND (USCS: SC-SM, SM, SP-SC) [AASHTO: A-1-b, A-2-4] and medium stiff to stiff CLAY (USCS: CL) [AASHTO: A-6]. Trace amounts of gravel and organics were also encountered.

The SPT N-values ranged from 5 to 34-bpf, with an average SPT N-value of 12-bpf. The natural moisture content averaged 14.8-percent and ranged from 9.7 to 17.8-percent. The percentage of fines (percent passing the No. 200 sieve) ranged from 14 to 65-percent, averaging 32.3-percent.

Stratum IIa – Cretaceous - Fine-Grained Soil: Stratum IIa encountered near the proposed Retaining Wall 16 generally consisted of medium stiff to hard CLAY and SILT with varying percentages of sand and lignite (USCS: CL, CH, ML) [AASHTO: A-4, A-6, A-7-5]. This stratum was interbedded with Stratum IIb.

The Stratum IIa SPT N-values ranged from 6 to 53-bpf, averaging 20.2-bpf. The natural moisture content averaged 22.1-percent and ranged from 13.8 to 31.9-percent. The liquid limit ranged from 23 to 67, averaging 45.5. The plasticity index ranged from 8 to 37, averaging 21.5. The percentage of fines (percent passing the No. 200 sieve) ranged from 76 to 100-percent, averaging 91.2-percent.

Stratum IIb – Cretaceous Coarse Grained Soil: Stratum IIb encountered near the proposed Retaining Wall 6 generally consisted of medium dense to very dense silty or clayey SAND with traces of lignite and mica (USCS: SC-SM, SM, SW-SM) [AASHTO: A-2-4, A-4]. Interbedded thin lenses of Stratum IIb are also classified as a very stiff SILT with varying percentages of Sand (USCS: ML) [AASHTO: A-4].

The SPT N-values ranged from 23 to 114-bpf, with an average SPT N-value of 67.6-bpf. The natural moisture content averaged 17.2-percent and ranged from 13.2 to 22-percent. The Liquid limit of select samples from this area ranged from Non-Plastic to 25 and the Plasticity Index ranged from Non-Plastic to 10. The percentage of fines (percent passing the No. 200 sieve) ranged from 12 to 59-percent, averaging 29.5-percent.

For Retaining Wall 16, site specific soil parameters were developed and used for design of the foundation elements. The Retaining Wall 16 soil parameters are summarized in Table 4.47 below. Table 4.48 summarizes the soil parameters used for the settlement analysis of this structure. Supporting calculations for development of soil parameters for Retaining Wall RW16 are provided in Appendix E.

Table 4.47 – Retaining Wall RW16 Soil Parameters

		Drained		Undrained
Strata	Total Unit Weight – γ (pcf)	Angle of Friction – ϕ (deg)	Cohesion – c (psf)	Shear - S_u (psf)
IIa – EL 63 to 53	130	24	-	1,350
IIa – EL 53 to 15	130	24	-	2,500
IIb – Below EL 15	130	34	-	-

Table 4.48 – Retaining Wall RW16 Soil Parameters for Settlement

Strata	Total Unit Weight – γ (pcf)	μ	E (ksf)	C_c	C_r	e_o	C_v (ft^2/day)	OCR
IIa – Above EL 50	130	0.4	545	0.16	0.016	0.633	-	3.7
IIa - EL 50 to 20	130	0.4	818	0.16	0.016	0.654	-	2.7
IIa - EL 20 to 0	130	0.4	1,364	0.16	0.016	0.654	-	2.0
IIa – EL 0 to -20	130	0.4	1,364	0.16	0.016	0.654	-	-
IIb – Below EL -20	130	0.3	1,364	-	-	-	-	-

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 62, approximately 5 to 15-ft below the existing ground surface.

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 4.4.16 of this report.

Groundwater was encountered at depths ranging from 0 to about 40-ft below the existing ground surface. In some holes, groundwater was not encountered. To achieve a more accurate determination of the hydrostatic water table, perforated pipes or piezometers were installed to monitor 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 PRELIMINARY 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 5.3.

5.1 RETAINING WALL FOUNDATION ALTERNATIVE ANALYSIS

The primary purpose of the retaining wall foundation alterative 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 retaining wall foundations were evaluated for construction. Detailed recommendations for each Retaining Wall are contained in Section 5.3.2 through 5.3.14 of this report.

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

5.1.1 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 and sequence of construction.

5.1.2 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 the time consuming to construct compared to a conventional CIP wall or an MSE wall.

Pile supported CIP retaining walls were not evaluated at proposed retaining walls within the project limits as they are located within embankment fill.

5.1.3 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 for a MSE wall is usually offset by the savings in construction costs and schedule as compared to a CIP wall on spread footings.

The retaining walls 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 5.3 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.

5.1.4 Other Wall Types

Other wall types considered for the construction of the retaining walls 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 great then 10-ft. In addition, the majority of the retaining walls are within embankment fill, as opposed to a cut situation where the above referenced walls are typically considered.

5.2 SUMMARY OF FOUNDATION ANALYSIS

It is recommended that MSE retaining walls be used for construction of the retaining walls. To meet design criteria for this project, several retaining walls 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 retaining walls 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 5.3.1 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.

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 retaining walls 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.

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. At 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.

Special considerations include the sequence of construction, anticipated settlement from construction, and constructability. These items are discussed in further detail in Section 5.4 of this report.

Typically, the drained condition governs the design of the retaining walls; therefore, the Contractor is required to verify the external stability using both the drained and undrained condition summarized in Section 5.3 of this report for each retaining wall.

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

5.3 RETAINING WALL FOUNDATION RECOMMENDATIONS

Retaining Wall 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.

5.3.1 MSE Wall General Foundation Recommendations

The following sections are general recommendations for construction of the MSE retaining walls. Additional or modifications to the general recommendations for MSE retaining wall construction are discussed in each specific retaining wall 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 retaining wall 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 wall strap to height ratios (L/H=0.7 to 0.8) will not provide adequate stability at the locations summarized in Table 5.2. The locations listed in Table 5.2 do not satisfy the minimum design criteria for external (bearing capacity, sliding, and overturning) & global stabilities.

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 sine an additional treatment process must be incorporated into the construction sequence.

Table 5.2 – Retaining Walls Requiring Special MSE Treatments		
Structure No.	**Location Description**	**Designer**
RW1	Ramp A	URS
RW2	Ramp G1	URS
RW3	Ramp B	URS
RW4	SR 7 Northbound	RK&K
RW5	Ramp B	RK&K
RW6	Ramp C	RK&K
RW7	Ramp G1	RK&K
RW8	SR 7 Southbound/Ramp C1	RK&K
RW9	Ramp A	RK&K
RW10	Ramp C	RK&K
RW16	Northbound SR 7	RK&K

Special Treatment

To meet the project design criteria (AASHTO LRFD – 2007 with interims) for the locations listed above in Table 5.2, 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 5.3 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 5.3 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 5.3.2 through 5.3.14 of this report.

Table 5.3 – Retaining Walls With Increased Strap Length

Structure No.	Location Description	L/H	Wedge Material
RW4	SR 7 Northbound	1.2	No. 57 Stone
RW5	Ramp B	0.8	Common Borrow
RW6	Northbound SR 7	0.8	Select Fill
RW7	Ramp G1	1.1	Common Borrow
RW8	SR 7 Southbound/Ramp C1	1.2	No. 57 Stone
RW9	Ramp A	1.0	Common Borrow
RW10	Ramp C	1.0[1]	Select Fill

Notes:
1: Reinforcement Zone No. 57 Stone

Table 5.4 summarizes locations where a combination of LWEF and select fill or No. 57 stone is recommended to meet the project design criteria. This concept is illustrated in Figure A-7 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 5.3.2 through 5.2.14 of this report. The amount of LWEF for each location has been minimized as much as possible. Figure A-6 in Appendix A of this report provides a typical plan view illustrating the overlap of the reinforcement zones for the abutment wingwalls.

Table 5.4 – Retaining Walls Requiring LWEF and Select Fill

Structure No.	Location Description	L/H	Height LWEF : No. 57 Stone (ft)[1]	Wedge Material
RW2	Ramp G1	0.75	20:24	Combination LWEF and Select Fill
RW3	Ramp B	0.7	18:24	Combination LWEF and Select Fill
RW16	Ramp C	1.0	Full Height LWEF	Full Height LWEF

Note 1: See Figure A-6
Note 2: Select Fill instead of No. 57 Stone above LWEF

Table 5.5 summarizes the locations where staged construction is required.

Table 5.5 – Retaining Walls Requiring Staged Construction			
Structure No.	**Location Description**	**Maximum Design Height (ft)**	**Maximum Height Stage 1 (ft)**
RW1	Ramp A	37	30
RW2	Ramp G1	44	39
RW3	Ramp B	42	36
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 reinforcing straps 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 5.6 below details the limits of electrochemical properties and the corresponding test method.

Table 5.6 - 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 retained wedge behind the MSE wall should be as recommended in Tables 5.3 and 5.4. 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 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 retaining walls 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 retaining wall in Section 5.3.2 through 5.3.14. Where possible, back to back retaining walls should be designed as independent structures with the active thrust is 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 that will overlap is anticipated for Retaining Walls RW1 and RW9 and Retaining Walls RW14 and RW15.

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.

5.3.2 Retaining Wall RW1 – Ramp A

Table 5.7 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 5.7 has been adjusted due to the presence of shallow groundwater.

Table 5.7 – Retaining Wall RW1 Soil Parameters

Strata	**Design Unit Weight – γ (pcf)**	**Drained Angle of Friction – ϕ (deg)**	**Undrained Shear - S_u (psf)**
Select Fill	125	34	-
Foundation Soil	105	28	1,400

Perched groundwater for this structure is anticipated to be near EL 45 which is not expected to be within the excavation limits. Section 5.5 of this report discusses general dewatering and drainage recommendations for this project.

It is recommended that the wall be constructed with select fill in both the reinforced and retained zones. Based on the face to face distance of 42-feet between Retaining Walls 1 and 9 it is anticipated that the reinforced zones will overlap. Our evaluation of the external stability for this wall assumes that active earth thrust from the backfill is reduced in accordance with the FHWA.

Due to the maximum 37-foot design height of this wall 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 E, the wall 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 wall 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 wall is 26-ft which was selected based on the minimum 0.7 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 E. 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. The suitability of the wall subgrade should be verified prior to construction.

It is anticipated that the total settlement for the wall will be about 3.1-inches, see Appendix E. 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 construction of pavements.

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

5.3.3 Retaining Wall RW2 – Ramp G1

Retaining Wall RW2 is above an existing cut 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 horizontal bench in front of the wall.

Table 5.8 summarizes the soil parameters to be used by the Contractor during design of the MSE wall for this structure.

Table 5.8 – Retaining Wall RW2 Soil Parameters

Strata	Design Unit Weight – γ (pcf)	Drained Angle of Friction – ϕ (deg)	Undrained Shear - S_u (psf)
Select Fill	125	34	-
LWEF	40	38	-
No. 57 Stone	105	38	-
Foundation Soil	120	27	1,250

Perched groundwater for this structure is anticipated to be near EL 65 which may fall within the excavation limits. Section 5.5 of this report discusses general dewatering and drainage recommendations for this project.

It is recommended that the wall be constructed with LWEF in both the reinforced and retained zones to a height of 20-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 S5-Ramp G1 over SR 7 Abutment A wingwalls, which are discussed in a separate report. Wall construction using select fill was investigated at this wall 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 this wall 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 E, the wall 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 wall can be constructed in 5-foot intervals with a minimum 7-day quarantine period between intervals. Settlement plates shall be monitored on a daily basis and after 7 days the engineer or a qualified owner's representative shall 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 increased from the AASHTO minimum to satisfy minimum 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 E. 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 wall 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 construction of pavements.

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

5.3.4 Retaining Wall RW3 – Ramp B

Table 5.9 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 5.9 has been adjusted due to the presence of shallow groundwater.

Table 5.9 – Retaining Wall RW3 Soil Parameters

Strata	**Design Unit Weight – γ (pcf)**	**Drained Angle of Friction – ϕ (deg)**	**Undrained Shear - S_u (psf)**
Select Fill	125	34	-
LWEF	40	38	-
No. 57 Stone	105	38	-
Foundation Soil	105	28	1,250

Perched groundwater for this structure is anticipated to be near EL 70 which may fall within the excavation limits. Section 5.5 of this report discusses general dewatering and drainage recommendations for this project.

It is recommended that the wall be constructed with LWEF in both the reinforced and retained zones to a height of 18-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 Retaining Wall 3 to the S3-Ramp B over SR 7 wingwalls, which are discussed in a separate report. Wall construction using select fill was investigated at this wall 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 42-foot design height of this wall 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 E, the wall can be constructed to a maximum height of 36-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 wall can be constructed in 5-foot intervals with a minimum 7-day quarantine period between intervals. Settlement plates shall be monitored on a daily basis and after 7 days the engineer or a qualified owner’s representative shall 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 wall is 30-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 E. Based on a resistance factor of 0.65 the factored bearing resistance for the initial stages of construction, prior to any fill placement, is 4.6-ksf, which should be verified prior to construction.

It is anticipated that the total settlement for the wall 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 construction of pavements.

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

5.3.5 Retaining Wall RW4 – SR 7 Northbound

Based on our review of the existing subsurface conditions and laboratory testing, we recommend that the retaining wall be constructed as an MSE.

Retaining Wall RW4 may be designed for a factored bearing resistance of 5.9-ksf and constructed with select fill for the full height of the retaining wall. The factored bearing resistance was calculated using a nominal bearing resistance of 9.0-ksf and applying a bearing resistance factor of 0.65. The minimum reinforcement length to height ratio for the retaining wall should be L/H=1.2. The failure wedge behind the MSE wall should be constructed of No. 57 stone. The minimum reinforcement length was increased from 0.7H and select fill is recommended to provide an acceptable capacity to demand ration (CDR) with respect to bearing capacity. The factored bearing resistance indicated above should be verified in the field prior to construction. Isolated pockets of fill across the length of the wall will be encountered and should be undercut prior to construction of the wall.

Table 5.10 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 5.10 has been adjusted due to the presence of shallow groundwater.

Table 5.10 – Retaining Wall RW4 Soil Parameters For Contractor's Design

Strata	Design Unit Weight – γ (pcf)	Angle of Friction – ϕ (deg)	Undrained Shear - S_u (psf)
Select Fill	125	34	-
No. 57 Stone	105	38	-
Foundation Soil	68	24	2,500

Groundwater for this structure is anticipated to be near EL 62, approximately 4-ft above the proposed leveling pad elevation, and, therefore, groundwater is anticipated to be within the excavation limits of the proposed foundations for this structure. Section 5.4 of this report discusses general dewatering and drainage recommendations for this project.

The anticipated distance between the front face of Retaining Walls 4 and 9 is approximately 100-ft. Based on our analysis, the reinforcement zones for these walls will not overlap, and, therefore, the walls can be designed as independent walls. 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.

It is anticipated that the total settlement for retaining wall RW4 will be about 5 to 7-inches and the maximum total settlement will occur at approximately Ramp A Station 1252+00, at the midpoint between Retaining Walls 4 and 9.

Differential settlements from one wall section to the next along the length of the retaining wall and from front to back of the retaining wall are within tolerable limits. For an MSE wall, the allowable ratio for differential settlement over a given length is 1/100 if the panel areas are less than 30-sf., that is, one foot of settlement for each 100-foot of wall. The maximum ratio calculated is 1/3,125, between Ramp A Station 1252+50 and Station 1253+00.

Based on the results of settlement monitoring plates at Ramp A, B, C, and G1, the settlement will occur within 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.

Supporting calculations for this structure are provided in Appendix E.

5.3.6 Retaining Wall RW5 – Ramp B

Based on our review of the existing subsurface conditions and laboratory testing, we recommend that the retaining wall be constructed as an MSE.

Retaining Wall RW5 may be designed for a factored bearing resistance of 9.2-ksf and constructed with select fill for the full height of the retaining wall. The factored bearing resistance was calculated using a nominal bearing resistance of 14.2-ksf and applying a bearing resistance factor of 0.65. The minimum reinforcement length to height ratio for the retaining wall should be L/H=0.8. The minimum reinforcement length was increased from 0.7H and select fill is recommended to provide an acceptable capacity to demand ration (CDR) with respect to bearing capacity. The factored bearing resistance indicated above should be verified in the field prior to construction. Isolated pockets of existing fill across the length of the wall will be encountered and should be undercut prior to construction of the wall. It should be anticipated that existing fill will be encountered to a depth of approximately 5-ft across the length of the wall. The existing fill should be undercut and replaced with a graded aggregate base.

Table 5.11 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 5.10 has been adjusted due to the presence of shallow groundwater.

Table 5.11 – Retaining Wall RW5 Soil Parameters For Contractor's Design

Strata	Design Unit Weight – γ (pcf)	Angle of Friction – ϕ (deg)	Undrained Shear - S_u (psf)
Select Fill	125	34	-
Foundation Soil	63	29	1,500

Groundwater for this structure is anticipated to be near EL 62, approximately 4-ft above the proposed leveling pad elevation, and, therefore, groundwater is anticipated to be within the excavation limits of the proposed foundations for this structure. Section 5.4 of this report discusses general dewatering and drainage recommendations for this project.

It is anticipated that the total settlement for retaining wall RW5 will be about 2 to 3-inches and the maximum total settlement will occur in the middle of the reinforcement zone, near Ramp A Station 1235+68.

Differential settlements from one wall section to the next along the length of the retaining wall and from front to back of the retaining wall are within tolerable limits. For an MSE wall, the allowable ratio for differential settlement over a given length is 1/100 if the panel areas are less than 30-sf., that is, one foot of settlement for each 100-foot of wall. The maximum ratio calculated is 1/250, near Ramp A Station 1235+68 in the middle of the reinforcement zone of RW5.

Based on the results of settlement monitoring plates at Ramp A, B, C, and G1, the settlement will occur within 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.

Supporting calculations for this structure are provided in Appendix E.

5.3.7 Retaining Wall RW6 – Ramp C

Based on our review of the existing subsurface conditions and laboratory testing, we recommend that the retaining wall be constructed as an MSE. Retaining Walls 6 and 16 can be designed as separate walls and not a tiered system if Type B fill is used as the foundation material for Retaining Wall RW6. The foundation material for retaining wall RW16 is further discussed in Section 5.3.14 of this report.

Retaining wall RW6 should be founded on a zone of Type B borrow. This zone should extend from firm material subgrade to the base of the reinforcement zone and should extend horizontally from the reinforcement zone a distance equal to the depth below the wall.

Retaining Wall RW6 from Station 1117+50 to 1117+75 may be designed for a factored bearing resistance of 5.8-ksf and constructed with select fill for the full height of the retaining wall. 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 to height ratio for the retaining wall is L/H=0.8. The failure wedge behind the MSE wall may be constructed of common borrow (Type F). The minimum reinforcement length was increased from 0.7H and select fill is recommended to provide an acceptable capacity to demand ratio (CDR) with respect to bearing capacity.

Retaining Wall RW6 from Station 1117+75 to the end of the wall may be designed for a factored bearing resistance of 5.8-ksf and constructed with select fill for the full height of the retaining wall. 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 to height ratio for the retaining wall is L/H=0.8. The failure wedge behind the MSE wall should be constructed of select fill. The minimum reinforcement length was increased from 0.7H and select fill is recommended to provide an acceptable capacity to demand ratio (CDR) with respect to bearing capacity.

The factored bearing resistance indicated above should be verified in the field prior to construction.

Table 5.12 summarizes the soil parameters to be used by the Contractor during design of the MSE wall for this structure.

Table 5.12 – Retaining Wall RW6 Soil Parameters For Contractor's Design

Strata	Design Unit Weight – γ (pcf)	Angle of Friction – ϕ (deg)	Undrained Shear - S_u (psf)
Common Borrow	130	28	-
Select Fill	125	34	-
Foundation Soil	125	26	1,200

Groundwater for this structure is anticipated to be near EL 62, approximately 20-ft below the proposed leveling pad elevation. Groundwater is not anticipated to be within the excavation limits of the proposed foundations for this structure, therefore, water trapped in sandy seams, in FILL or interbedded lenses of Stratum IIb could cause minor construction difficulties. Section 5.4 of this report discusses general dewatering and drainage recommendations for this project.

It is anticipated that the total settlement for retaining wall RW6 will be about 4.5 to 5.5-inches and the maximum total settlement will occur approximately where the RW6 reinforcement zone meets the retained fill.

Differential settlements from one wall section to the next along the length of the retaining wall and from front to back of the retaining wall are within tolerable limits. For an MSE wall, the allowable ratio for differential settlement over a given length is 1/100 if the panel areas are less than 30-sf., that is, one foot of settlement for each 100-foot of wall. The maximum ratio calculated is 13/6,701.

Based on the results of settlement monitoring plates at Ramp A, B, C, and G1, the settlement will occur within 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.

Supporting calculations for this structure are provided in Appendix E.

5.3.8 Retaining Walls RW7 and RW8 – Ramp G1 and SR 7 Southbound/Ramp C1

Based on our review of the existing subsurface conditions and laboratory testing, we recommend that the retaining walls RW7 and RW8 be constructed as an MSE. From Ramp G1 Station 1317+68 to 1318+00 these retaining walls should be designed as overlapping MSE walls requiring that they be designed using ASD. Station 1318+00 to the end of the wall at Station 1356+88 they are independent wall and therefore should be designed using LRFD.

Retaining Walls RW7 and RW8 from Ramp G1 Station 1317+68 to 1318+00 may be designed for an allowable bearing capacity of 3.9-ksf and constructed with select fill for the full height of the retaining walls. There will be no backfill between the walls. The allowable bearing capacity was calculated using a factor of safety of 2.0. The minimum reinforcement length to height ratio for the retaining wall RW7 is L/H=0.9 and for RW8 is L/H=1.2. The minimum reinforcement length for retaining walls RW7 and RW8 were increased from 0.7H to provide an acceptable bearing capacity. The allowable bearing capacity indicated above should be verified in the field prior to construction.

The anticipated distance between the front face of Retaining Walls 7 and 8 is approximately 58-ft. Based on our analysis, the reinforcement zones for these walls will overlap, and, therefore, the walls can be designed as one structure. 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.

It should be anticipated that existing fill will be encountered in isolated areas of Retaining Wall 8. The existing fill should be undercut and replaced with a graded aggregate base.

Retaining Wall RW7 from Ramp G1 Station 1318+00 to the end of the wall at Station 1356+88 may be designed for a factored bearing resistance of 6.0-ksf and constructed with select fill for the full height of the retaining wall. Retaining Wall RW8 from Ramp C1 Station 1816+35 to SR 7 SB Station 1615+60 may be designed for a factored bearing resistance of 3.2-ksf with select fill in the MSE reinforcement zone. The failure wedge behind the retaining wall RW8 should be constructed of No. 57 stone. The factored bearing resistance was calculated using a nominal bearing resistance of 9.3-ksf for RW7 and 4.9-ksf for RW8 and applying a bearing resistance factor of 0.65. The minimum reinforcement length to height ratio for the retaining wall RW7 is L/H=1.1 and for RW8 is L/H=1.2. The minimum reinforcement length for retaining walls RW7 and RW8 was increased from 0.7H to provide an acceptable capacity to demand ratio (CDR) with respect to bearing capacity. The factored bearing resistance indicated above should be verified in the field prior to construction.

Table 5.13 summarizes the soil parameters to be used by the Contractor during design of the MSE walls 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 5.12 has been adjusted due to the presence of shallow groundwater.

Table 5.13 – Retaining Wall RW7 and RW8 Soil Parameters For Contractor's Design			
Strata	**Design Unit Weight – γ (pcf)**	**Angle of Friction – ϕ (deg)**	**Undrained Shear - S_u (psf)**
Select Fill	125	34	-
No. 57 Stone	105	38	-
Foundation Soil	68	24	2,000

Groundwater for this structure is anticipated to be near EL 62, approximately 2-ft below the proposed leveling pad elevation and therefore, groundwater is anticipated to be within the excavation limits of the proposed foundations for this structure. Section 5.4 of this report discusses general dewatering and drainage recommendations for this project.

It is anticipated that the total settlement for retaining walls RW 7 and 8 will be about 3.5 to 5-inches and the maximum total settlement will occur at approximately Ramp A Station 1252+00, at the midpoint between Retaining Walls RW4 and RW9.

Differential settlements from one wall section to the next along the length of the retaining walls and from front to back of the retaining walls are within tolerable limits. For an MSE wall, the allowable ratio for differential settlement over a given length is 1/100 if the panel areas are less than 30-sf., that is, one foot of settlement for each 100-foot of wall. The maximum ratio calculated is 1/249, near Ramp A Station 1251+50 from the contact of the RW7 reinforcement zone and the front face of RW8.

Based on the results of settlement monitoring plates at Ramp A, B, C, and G1, the settlement will occur within 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.

Supporting calculations for this structure are provided in Appendix E.

5.3.9 Retaining Wall RW9 – Ramp A

Based on our review of the existing subsurface conditions and laboratory testing, we recommend that the retaining wall be constructed as an MSE.

Retaining Wall RW9 from Ramp A Station 1250+92 to 1252+14 may be designed for a factored bearing resistance of 9.75-ksf and constructed with select fill for the full height of the retaining wall. The factored bearing resistance was calculated using a nominal bearing resistance of 15.0-ksf and applying a bearing resistance factor of 0.65. The minimum reinforcement length to height ratio for the retaining wall is L/H=0.7. The factored bearing resistance indicated above should be verified in the field prior to construction. It should be anticipated that existing fill will be encountered to a depth of approximately 5-ft across the length of the wall. The existing fill should be undercut and replaced with a graded aggregate base.

Retaining Wall RW9 from Ramp A Station 1252+14 to 1259+50 may be designed for a factored bearing resistance of 6.4-ksf and constructed with select fill for the full height of the retaining wall. The factored bearing resistance was calculated using a nominal bearing resistance of 9.8-ksf and applying a bearing resistance factor of 0.65. The minimum reinforcement length to height ratio for the retaining wall is L/H=1.0. The minimum reinforcement length was increased from 0.7H to provide an acceptable capacity to demand ratio (CDR) with respect to bearing capacity. The factored bearing resistance indicated above should be verified in the field prior to construction.

Table 5.14 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 5.13 has been adjusted due to the presence of shallow groundwater.

Table 5.14 – Retaining Wall RW9 Soil Parameters For Contractor's Design

Strata	Design Unit Weight – γ (pcf)	Angle of Friction – ϕ (deg)	Undrained Shear - S_u (psf)
Select Fill	125	34	-
Common Borrow	130	28	-
Foundation Soil STA 1251+00	92	24	2,500
Foundation Soil STA 1252+00	85	24	2,500

Groundwater for this structure is anticipated to be near EL 54, approximately 9-ft below the proposed bottom of the leveling pad elevation. Groundwater is not anticipated to be within the excavation limits of the proposed foundations for this structure, therefore, water trapped in sandy seams, in FILL or interbedded lenses of Stratum IIb could cause minor construction difficulties. Section 5.4 of this report discusses general dewatering and drainage recommendations for this project.

From Ramp A Station 1252+14 to 1259+50 retaining wall RW9 can be designed as a standalone structure as the anticipated distance between the front face of Retaining Walls RW4 and RW9 is approximately 100-ft and the reinforcement zones for these walls will not overlap.

It is anticipated that the total settlement for retaining wall RW9 will be about 2 to 3.5-inches and the maximum total settlement will occur at approximately Ramp A Station 1251+53, at the midpoint between Retaining Walls RW1 and RW9.

Differential settlements from one wall section to the next along the length of the retaining wall and from front to back of the retaining wall are within tolerable limits. For an MSE wall, the allowable ratio for differential settlement over a given length is 1/100 if the panel areas are less than 30-sf., that is, one foot of settlement for each 100-foot of wall. The maximum ratio calculated is 1/662, at Ramp A Station 1251+53 from the face of RW9 to the back of the reinforcement zone.

Based on the results of settlement monitoring plates at Ramp A, B, C, and G1, the settlement will occur within 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.

Supporting calculations for this structure are provided in Appendix E.

5.3.10 Retaining Wall RW10 – Ramp C

Based on our review of the existing subsurface conditions and laboratory testing, we recommend that the retaining wall be constructed as an MSE.

Retaining Wall RW10 from may be designed for a factored bearing resistance of 5.6-ksf and constructed with No. 57 stone for the full height of the retaining wall. The factored bearing resistance was calculated using a nominal bearing resistance of 8.6-ksf and applying a bearing resistance factor of 0.65. The minimum reinforcement length to height ratio for the retaining wall is L/H=1.0. The failure wedge behind the MSE wall should be constructed of select fill. The minimum reinforcement length was increased from 0.7H and select fill is recommended to provide an acceptable capacity to demand ratio (CDR) with respect to bearing capacity. The factored bearing resistance indicated above should be verified in the field prior to construction.

Table 5.15 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 5.14 has been adjusted due to the presence of shallow groundwater.

Table 5.15 – Retaining Wall RW10 Soil Parameters For Contractor's Design

Strata	Design Unit Weight – γ (pcf)	Angle of Friction – ϕ (deg)	Undrained Shear - S_u (psf)
Select Fill	125	34	-
No. 57 Stone	105	38	-
Foundation Soil	68	24	2,000

Groundwater for this structure is anticipated to be near EL 69, approximately the elevation of the proposed bottom of leveling pad. Groundwater is anticipated to be within the excavation limits of the proposed foundations for this structure, therefore, water trapped in sandy seams, in FILL or interbedded lenses of Stratum IIb could cause minor construction difficulties. Section 5.4 of this report discusses general dewatering and drainage recommendations for this project.

It is anticipated that the total settlement for retaining wall RW10 will be about 2.5 to 4.0-inches and the maximum total settlement will occur at approximately the midpoint of the retaining wall RW10 reinforced zone.

Differential settlements from one wall section to the next along the length of the retaining wall and from front to back of the retaining wall are within tolerable limits. For an MSE wall, the allowable ratio for differential settlement over a given length is 1/100 if the panel areas are less than 30-sf., that is, one foot of settlement for each 100-foot of wall. The maximum ratio calculated is 1/1,938.

Based on the results of settlement monitoring plates at Ramp A, B, C, and G1, the settlement will occur within 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.

Supporting calculations for this structure are provided in Appendix E.

5.3.11 Retaining Wall RW11 - Eliminated

This retaining wall has been eliminated from the project.

5.3.12 Retaining Walls RW12 and RW13 – Ramp A

Based on our review of the existing subsurface conditions and laboratory testing, we recommend that the retaining wall be constructed as an MSE.

Retaining Walls 12 and 13 may be designed for a factored bearing resistance of 6.7-ksf. This was calculated using a nominal bearing resistance of 10.3-ksf and applying a bearing resistance factor of 0.65. The minimum reinforcement length to height ratio for the retaining wall is L/H=0.7. The factored bearing resistance indicated above should be verified in the field prior to construction.

The anticipated distance between the front face of Retaining Walls 12 and 13 is approximately 55.6-ft. Based on our analysis it is anticipated that the reinforcement zone will not overlap. Based on our analysis the reinforcement zones for these walls will not overlap, and, therefore, the walls can be designed as independent walls; however, the active thrust is reduced. 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.

Table 5.16 summarizes the soil parameters to be used by the Contractor during design of the MSE walls 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 5.15 has been adjusted due to the presence of shallow groundwater.

Table 5.16 – Retaining Walls RW12 & RW13 Soil Parameters For Contractor's Design			
Strata	**Design Unit Weight – γ (pcf)**	**Angle of Friction – ϕ (deg)**	**Undrained Shear - S_u (psf)**
Select Fill	125	34	-
Common Borrow	125	28	-
Foundation Soil	68	32	2,000

Groundwater for this structure is anticipated to be near EL 76, approximately 1-ft below the proposed leveling pad elevation and therefore, groundwater is anticipated to be within the excavation limits of the proposed foundations for this structure. Section 5.4 of this report discusses general dewatering and drainage recommendations for this project.

It is anticipated that the total settlement for retaining walls RW 12 and RW 13 will be about 3 to 4.5-inches and the maximum total settlement will occur at approximately Ramp A Station 1227+00, at the midpoint between Retaining Walls 12 and 13.

Differential settlements from one wall section to the next along the length of the retaining walls and from front to back of the retaining walls are well within tolerable limits. For an MSE wall, the allowable ratio for differential settlement over a given length is 1/100 if the panel areas are less than 30-sf., that is, one foot of settlement for each 100-foot of wall. The maximum ratio calculated is 1/686, near Ramp A Station 1227+00 at the contact of retaining walls RW 12 and RW 13 reinforcement zones and the common borrow.

Based on the results of settlement monitoring plates at Ramp A, B, C, and G1, the settlement will occur within 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.

Supporting calculations for this structure are provided in Appendix E.

5.3.13 Retaining Walls RW14 and RW15 – Ramp B

Based on our review of the existing subsurface conditions and laboratory testing, we recommend that the retaining walls RW14 and RW15 be constructed as an MSE. The entire length of these retaining walls should be designed as overlapping MSE walls, requiring that they be designed using ASD.

Retaining Walls RW14 and RW15 may be designed for an allowable bearing capacity of 4.8-ksf and constructed with select fill for the full height of the retaining wall. There will be no backfill between the walls. The allowable bearing capacity was calculated using a factor of safety of 2.0. The minimum reinforcement length to height ratio for the retaining walls RW14 and RW15 is L/H=0.7. The allowable bearing capacity indicated above should be verified in the field prior to construction.

The anticipated distance between the front face of Retaining Walls 14 and 15 is approximately 38-ft. Based on our analysis, the reinforcement zones for these walls will overlap, and, therefore, the walls can be designed as one structure. 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.

Table 5.17 summarizes the soil parameters to be used by the Contractor during design of the MSE walls 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 5.16 has been adjusted due to the presence of shallow groundwater.

Table 5.17 – Retaining Walls RW14 & RW15 Soil Parameters For Contractor's Design

Strata	Design Unit Weight – γ (pcf)	Angle of Friction – ϕ (deg)	Undrained Shear - S_u (psf)
Select Fill	125	34	-
Foundation Soil	68	24	1,700

Groundwater for this structure is anticipated to be near EL 79, approximately 5-ft below the proposed bottom of leveling pad elevation. Groundwater is anticipated to be within the excavation limits of the proposed foundations for this structure, and water trapped in sandy seams, in FILL or interbedded lenses of Stratum IIb could cause minor construction difficulties. Section 5.4 of this report discusses general dewatering and drainage recommendations for this project.

It is anticipated that the total settlement for retaining walls RW14 and RW15 will be about 2 to 3-inches and the maximum total settlement will occur at the midpoint between retaining walls RW14 and RW15.

Differential settlements from one wall section to the next along the length of the retaining walls and from front to back of the retaining walls are well within tolerable limits. For an MSE wall, the allowable ratio for differential settlement over a given length is 1/100 if the panel areas are less than 30-sf., that is, one foot of settlement for each 100-foot of wall. The maximum ratio calculated is 1/320, at the midpoint between retaining walls RW14 and RW15.

Based on the results of settlement monitoring plates at Ramp A, B, C, and G1, the settlement will occur within 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.

Supporting calculations for this structure are provided in Appendix E.

5.3.14 Retaining Wall RW16 – Northbound SR 7

Based on our review of the existing subsurface conditions and laboratory testing, we recommend that the retaining wall be constructed as an MSE. Retaining Walls 6 and 16 can be designed as separate walls and not a tiered system if Type B fill is used below Retaining Wall 6. The foundation material for retaining wall RW6 is further discussed in Section 5.3.7 of this report.

Retaining Wall RW16 from may be designed for a factored bearing resistance of 2.4-ksf and constructed with LWEF for the full height of the retaining wall. The factored bearing resistance was calculated using a nominal bearing resistance of 3.7-ksf and applying a bearing resistance factor of 0.65. The minimum reinforcement length to height ratio for the retaining wall is L/H=1.0. The failure wedge behind the MSE wall should be constructed of LWEF. The minimum reinforcement length was increased from 0.7H and select fill is recommended to provide an acceptable capacity to demand ratio (CDR) with respect to bearing capacity and sliding resistance. The factored bearing resistance indicated above should be verified in the field prior to construction.

Isolated pockets of fill should be anticipated across the length of the wall and should be undercut prior to construction.

Table 5.18 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 5.17 has been adjusted due to the presence of shallow groundwater.

Table 5.18 – Retaining Wall RW16 Soil Parameters For Contractor's Design

Strata	Design Unit Weight – γ (pcf)	Angle of Friction – ϕ (deg)	Undrained Shear - S_u (psf)
LWEF	40	-	17,280
Foundation Soil	68	20	1,350

Groundwater for this structure is anticipated to be near EL 62, approximately the elevation of the proposed bottom of leveling pad. Groundwater is anticipated to be within the excavation limits of the proposed foundations for this structure, therefore, water trapped in sandy seams, in FILL or interbedded lenses of Stratum IIa could cause minor construction difficulties. Section 5.4 of this report discusses general dewatering and drainage recommendations for this project.

It is anticipated that the total settlement for retaining wall RW16 will be about 2.0 to 5.5-inches and the maximum total settlement will occur at approximately the midpoint of the retaining wall RW16 reinforced zone between Station 1117+50 and 1117+75.

Differential settlements from one wall section to the next along the length of the retaining wall and from front to back of the retaining wall are within tolerable limits. For an MSE wall, the allowable ratio for differential settlement over a given length is 1/100 if the panel areas are less than 30-sf., that is, one foot of settlement for each 100-foot of wall. The maximum ratio calculated is 1/371.

Based on the results of settlement monitoring plates at Ramp A, B, C, and G1, the settlement will occur within 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.

Supporting calculations for this structure are provided in Appendix E.

5.4 SPECIAL CONSIDERATIONS

5.4.1 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 height of the wingwalls and retaining walls.
- 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 construction, impose an estimated 30-day quarantine period prior to driving the abutment piles, placement of parapets or pavements. For retaining walls with staged construction, this requirement can be reduced by the amount of quarantine proposed during construction.
- Monitor the settlement of the retaining walls as described in Section 5.4.2 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.

5.4.2 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 may be appropriate for tightly spaced pile groups to measure potential displacement and heave due to driving adjacent piles. The survey monitoring points and settlement plates should be located by repeatable survey. Settlement monitoring points should be located at the top of each retaining wall. 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.

5.4.3 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 ranging from 1 to 2-ksf.

5.4.4 Corrosion Protection

Corrosion potential for this project is based on the corrosion and deterioration criteria set forth in AASHTO, Section 10.7.5. If soils meet the following criteria, they are to be considered corrosive.

- pH less than 5.5, or
- Resistivity less than 2,000 ohm-cm

It is anticipated that corrosion protection will be required 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: SR 1/I-95 Interchanger GDR with Supplemental Laboratory Test Data.

Corrosion protection for the construction of the MSE reinforced zones is discussed in Section 5.3.1.

5.4.5 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.

Design parameters per wall location are summarized in Sections 5.3.2 through 5.3.14 of this report.

5.5 DEWATERING AND DRAINAGE

It is anticipated the groundwater will be encountered for construction of most retaining walls near the elevation of the leveling pad. Water trapped in sandy seams, in FILL or interbedded lenses of Stratum IIb could cause minor construction difficulties. A detailed discussion regarding the depth of groundwater with respect to the proposed construction is contained in Sections 5.3.2 through 5.3.14 of this report. 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, except at the locations identified in this report. 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 or bearing 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 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 retaining walls were provided by the firm indicated 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**
Retaining Wall RW1	URS
Retaining Wall RW2	URS
Retaining Wall RW3	URS
Retaining Wall RW4	RK&K
Retaining Wall RW5	RK&K
Retaining Wall RW6	RK&K
Retaining Wall RW7	RK&K
Retaining Wall RW8	RK&K
Retaining Wall RW9	RK&K
Retaining Wall RW10	RK&K
Retaining Wall RW11	RK&K
Retaining Wall RW12	RK&K
Retaining Wall RW13	RK&K
Retaining Wall RW14	RK&K
Retaining Wall RW15	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.