FINAL ROADWAY GEOTECHNICAL ENGINEERING REPORT

FLATIRON | ZACHRY
a joint venture

INTERSTATE 85/385 INTERCHANGE IMPROVEMENTS
FEDERAL AID PROJECT NO. IM23(009)
FILE NO. 23.038111
PROJECT ID 0038111-R01
CECS PROJECT NO. 4177
GREENVILLE COUNTY, SOUTH CAROLINA
ECS PROJECT NO. 08-9283
SEPTEMBER 21, 2015
Rev December 8, 2015

ECS

CECS Civil Engineering Consulting Services, Inc.

SCDOT

U.S. Department of Transportation Federal Highway Administration

FINAL ROADWAY GEOTECHNICAL ENGINEERING REPORT

Interstate 85/385 Interchange Improvements
ROADWAY GEOTECHNICAL ENGINEERING REPORT
Federal Aid Project No. IM23(009)
File No. 23.038111
Greenville County, South Carolina

Prepared For:

Mr. Rocque L. Kneece, P.E.
Civil Engineering Consulting Services, Inc.
2000 Park Street – Suite 201
Columbia, SC 29201

Prepared By:

ECS CAROLINAS, LLP
1812 Center Park Drive
Suite D
Charlotte, NC 28217

ECS Project No:

08-9283

Report Date:

September 21, 2015
Rev December 8, 2015

September 21, 2015
Rev December 8, 2015

Mr. Rocque L. Kneece, P.E.
Civil Engineering Consulting Services, Inc.
2000 Park Street – Suite 201
Columbia, SC 29201

Reference: Final Roadway Geotechnical Engineering Report
Interstate 85/385 Interchange Improvements
Federal Aid Project No. IM23(009)
File No. 23.038111
Project ID 0038111-R01
CECS Project No. 4177
Greenville County, South Carolina
ECS Project No. 08-9283

Dear Mr. Kneece:

ECS Carolinas, LLP (ECS) has completed the revisions to Final Roadway Geotechnical Engineering Report (RGER) for the above referenced project. The purpose of this report is to provide final geotechnical analyses and information to the design team. This report provides updates and revisions as requested by SCDOT or based on modifications to the Roadway, Structure, or Drainage Plans. Modifications to the report and appendix include:

- Wall 31B results in Table 5.16 updated to 0.58 for ESA and 0.55 for TSA.

ECS Carolinas, LLP appreciates the opportunity to assist you during this phase of the project. If you have questions concerning this report, please contact our office.

Respectfully,

ECS CAROLINAS, LLP

Robert L. Hendrick, III, E.I.
Project Manager

Marc F. Plotkin, P.E.
Principal Engineer
SC Registration No. 30565

SOUTH CAROLINA LICENSED PROFESSIONAL ENGINEER No. 30565 12-8-15 MARC F. PLOTKIN

Richard L. Nance, P.E.
Senior Principal Engineer/VP
SC Registration No. 07332

SOUTH CAROLINA ECS CAROLINAS, LLP No. COA3239 CERTIFICATE OF AUTHORIZATION

Table of Contents

LIST OF APPENDICES

1. INTRODUCTION

Flatiron/Zachary JV has been selected by SCDOT to design and construct the Interstate 85/385 Interchange Improvements in Greenville County, South Carolina. ECS Carolinas, LLP (ECS) has been selected by the design team as the Geotechnical Engineer for the project.

ECS is pleased to present this Final Roadway Geotechnical Engineering Report (RGER) for the Interstate I-85/385 Interchange improvements. This report includes the analyses for roadway improvements/extension including embankments, MSE walls, and cut sections. Analyses and construction recommendations for the retaining walls and slopes along I-85/385 Interchange that are located within 150 feet the bridges are provided in their respective Bridge Final Geotechnical Engineering Report (BGER), which are presented under separate covers.

2. PROJECT INFORMATION

2.1. Project Location

The project site is centered at the intersection of Interstates 85 and 385 in Greenville, South Carolina. The interchange is located approximately 6 miles east-southeast of downtown Greenville. The bridge and associated retaining walls/slopes within 150 feet of the proposed bridge alignments are not included in this report and are addressed in their respective BGER. See Appendix A for the location map.

2.2. Project Description

The general project entails improvements to the I-85/385 Interchange including:

A. At the I-85/385 Interchange, remove all existing bridges and construct seven (7) new bridges with associated new collector distributor (CD) roadways and added travel lanes.
B. Replace the Roper Mountain Road Overpass over I-85 with a new bridge.
C. Relocate Chrome Drive and portions of Roper Mountain Road.
D. Widen the existing I-385 NB and SB Overpass Bridges over Garlington Road and GE Railroad.
E. Construct a new bridge over Garlington Road carrying the I-385 NB/I-85 NB CD traffic.
F. Provide clearance for new and existing north and south bound lanes of I-385 and new collector distributor roadways at the existing Woodruff Road underpass for I-385.
G. Resurface portions of I-85 and I-385 extending 2 to 5 miles north and south of the current I-85/385 Interchange.

2.2.1 <u>Report Description</u>

This RGER addresses geotechnical considerations associated with roadway, embankments, and retaining walls situated more than 150 feet from new bridges. Geotechnical considerations for embankment and retaining walls within 150 feet of new bridges are addressed in the individual Bridge Geotechnical Engineering Reports (BGER).

2.3. Roadway Geotechnical Considerations

The I-85/385 project is one of the largest projects undertaken by the SCDOT. The complexity of replacing an active interchange while maintaining traffic requires unique roadway geometries and elevation changes. These geometries and elevation changes present geotechnical challenges and considerations ranging from tall MSE walls adjacent to existing slopes and anchored retaining walls to allow relatively deep excavations adjacent to active roadways to sliver cuts and fills of less than 2 feet for minor re-grading. This RGER presents our geotechnical engineering analysis as it relates to the interchange improvements.

This RGER considers aspects of the project greater than 150 feet from proposed bridges. Refer to the individual BGER for embankment and MSE Wall consideration within 150 feet of the individual bridges. This RGER is based on available subsurface information and our experience in the regional geology.

2.4. Roadway Design Sections

In order to accommodate the roadway alignment, cuts of up to 27.5 feet and fills upwards of 54 feet are required. In some cases, new fills are required adjacent to proposed cuts to facilitate the interchange and various ramp alignments. Table 2.1 summarizes the various roadway portions of the project that are outside of the 150' limits of the bridge abutments. Table 2.1-A summarizes the various walls associated with the roadway that are outside the 150' limits of the bridge abutments.

The specific embankments, cuts, and MSE Walls within 150' of new bridges are addressed in the individual BGER for each bridge. The roadway design for I-85 from Station 375+00 to 435+00 and Pelham Road is not complete. Those portions of the interchange improvement project will be incorporated into the revised report or transmitted under a separate cover.

Table 2.1 – Roadway Design Sections (Descriptions and Cut/Fill Summary)

Alignment	Description	Bridge	Wall	ROC	Station Begin	Station End	Max. Cut Depth, ft	Max. Fill Thickness ft
Ramp 1	I-85 S to I-385 N	N/A	RSS,12	IV	49+65.68	120+31.47	23	27
Ramp 1A[6]	I-85 S to I-385 SB C/D	5	12,13[6]	IV[1]	48+15.42	71+29.42	24	42
					89+52.25	97+36.64	0	45
Ramp 1B	I-85 S to Woodruff RD	10	N/A	IV	49+42.69	67.61.35	10	30
					72+81.14	79+94.00	sliver	18
Ramp 2	I-385 S to I-85 S	N/A	N/A	IV	50+00	68+38.31	14	20
Ramp 2A[6]	I-385 S to I-85 N	9,11	16[6]	IV[1]	46+39.91	70+20.81	9	25
					76+89.65	133+70.35	21	25
Ramp 2B[4]	I-385 S to I-385 SB C/D	7	33	IV[1,2]	18+33.96	32+66.95	N/A	47
					40+41.96	49+41.34	N/A	54
Ramp 3	I-85 N to I-385 S	N/A	N/A	IV	30+00	48+17.34	24	42
Ramp 3A[4]	I-85 N to I-385 N	N/A	10	IV	287+34.88	320+96.08	13	23
Ramp 4[4]	I-385 NB C/D to I-85 N	N/A	14, 36	IV[1]	38+60.08	68+25.46	5	47
Ramp 4B[4]	I-385 NB C/D to I-85 S	6	N/A	IV[1]	378+66.47	388+61.50	N/A	42
					411+31.33	422+84.17	13	24
Ramp 5	Woodruff RD to I-85 S	N/A	RSS	IV[3]	111+89.37	145+61.43	8	25
Ramp 7	Woodruff RD to I-85 N	N/A	N/A	IV[3]	21+16.46	29+45.81	N/A	5
Ramp 8	Woodruff RD to I-385 N	4	N/A	IV[3]	51+06.57	56+35.33	N/A	10
					60+83.70	65+90.06	N/A	24
Ramp 8A	Woodruff RD to I-385 NB C/D	N/A	N/A	IV[3]	10+00	21+31.96	N/A	17.5
Ramp 9	I-385 SB C/D to Woodruff RD	N/A	N/A	IV[3]	53+72.26	58+25	sliver	sliver
Ramp 10	Woodruff RD to I-385 S	N/A	N/A	IV[3]	21+00	33+16.58	10	5
Ramp 11	I-385 N to Woodruff RD	N/A	N/A	IV[3]	50+00	63+98.04	8	N/A
I-85	I-85 Mainline	N/A	N/A	IV[1]	203+00	375+00	22	17
I-85 NB C/D	I-85 N to I-385	N/A	N/A	IV	240+00	298+04.38	20	5
I-385	I-385 Mainline	1/2A, 2B, 12	RSS, 38	IV[1,3]	303+89.49	372+15.54	7	23
					378+95.5	393+84.84	sliver	40
					401+82.01	447+60	10	sliver
I-385 NB C/D[4]	I-385 N to I-85	3	21, 26	IV[1,3]	327+79.9	374+36.28	20	25
I-385 SB C/D	I-85 S to I-385 S	N/A	27, 33	IV[3]	97+01.12	140+10.55	22	41

Note: 1. ROC=III where retaining walls are present.
2. ROC=I~~I~~ where MSE Wall height exceeds 50 feet.
3. ROC=I within 150 feet of Woodruff Road.
4. Ramp alignment includes an MSE Wall, see table 2.4 for additional MSE Wall details.
5. Ramp alignment includes a Reinforce Soil Slope, see table 2.4
6. Alignment includes a soldier pile and lagging wall or soil nail wall see Table 2.2
N/A – Not Applicable

Table 2.1A – Roadway Design Section (Summary of Retaining Walls)

Wall Number	Type	Alignment	Begin Station	End Station
1	MSE	Ramp 1B	75+46.40	72+83.00
		Ramp 1B	72+83.00	76+60.32
2A	MSE	I-85	278.91+65	282+30.89
2B	Barrier	I-85	282+30.89	335+57.56
10	MSE	Ramp 3A	292+63.17	296+94.27
11	Barrier	Ramp 2A	85+12.48	89+96.46
12	Pile and Lagging/MSE	Ramp 1	73+17.45	79+15.57
		Ramp 1A	48+15.42	71+29.42
13	MSE	Ramp 1A	63+00.00	71+29.42
14	Concrete	Ramp 4	60+00.00	65+89.96
15	Barrier	I-85	301+54.55	332+92.82
16B	Pile and Lagging/MSE	Ramp 2A	105+58.00	114+34.28
16A	Barrier	Ramp 2A	97+83.52	105+58.00
21	Pile and Lagging	I-385 NBCD	337+09.99	346+41.58
22	Barrier	I-385	332+17.07	436+66.72
23	Barrier	I-385	344+84.77	355+38.66
24	Barrier	I-385	350+11.00	388+10.60
25	Barrier	I-385 NBCD	357+63.27	363+03.10
26	Soil Nail Wall	I-385 NBCD	358+65.60	361+32.39
27	Pile and Lagging/Soil Nail Wall	I-385 SBCD	116+29.20	122+39.44
30	Barrier	I-385 NBCD	373+74.71	374+93.09
31A	Barrier	I-385	378+64.49	387+40.09
31B	MSE	Ramp 4B	382+12.55	384+67.14
32	MSE	I-385 NBCD	379+53.08	380+02.71
		Ramp 4	38+60.08	52+36.35
33	MSE	Ramp 2B	40+37.82	49+41.34
		I-385 SBCD	99+07.93	99+05.20
34	Barrier	Ramp 2B	39+14.67	44+26.84
36A	Barrier	Ramp 4B	382+96.78	386+45.11
36B	MSE	Ramp 4	50+68.09	54+18.80
38	pile and lagging	I-385	423+00.12	446+69.02
39	Barrier	I-85	276+97.93	290+92.04

2.4.1 Excavation (Cut) Sections

The Roadway plans indicate excavation (cut) sections will be less than forty (40) feet deep. The cut sections on this project range from approximately 0 to 25 feet in depth. Cut sections on this project are grouped into three categories:

- Cut slopes with slope inclinations flatter than 2H:1V.
- Cut slopes with slope inclinations between 1.5 to 2H:1V.
- Cut sections with retaining walls (Soil Nail, MSE or Soldier Pile and Lagging).

Table 2.2 summarizes the anticipated maximum cut sections and identifies if the cut section incorporates a slope inclination steeper than 2H:1V or a retaining wall. Soldier piles with concrete lagging panels, reinforced concrete walls, or soil nail walls are planned at each cut wall location. The planned wall type is also summarized in the table.

Table 2.2 - Excavation (Cut) Sections/Slopes

Alignment	Station		Approximate Maximum Cut Depth (ft)
	Excavation Begin	Excavation End	
Ramp 1	51+50	58+00	10 +/-
Ramp 1	71+00	79+09	10 +/-
Ramp 1 (Wall 12 – Pile and Lagging)	73+18.75	79+08.75	9 +/-
Ramp 1	79+08.75	82+00	25 +/-
Ramp 1	104+50	114+50	11 +/-
Ramp 1A[2]	48+15.42	53+00	25 +/-
Ramp 1A (Wall 12 – Pile and Lagging with Soil Nail Wall from Sta. 49+69.7 to 50+76.66)	48+15.42	63+00	25 +/-
Ramp 1B	54+00	60+00	28 +/-
Ramp 2	50+00	55+00	7 +/-
Ramp 2	65+00	68+38.31	13 +/-
Ramp 2A	46+39.91	53+50	5 +/-
Ramp 2A	61+50	70+50	12 +/-
Ramp 2A	77+50	84+50	18 +/-
Ramp 2A	92+00	105+58.00	7 +/-
Ramp 2A (Wall 16 – Pile and Lagging with Soil Nail Wall from Sta. 109+42 to 110+58)	105+58.00	114+33.28	21 +/-
Ramp 2A	120+00	126+50	19 +/-
Ramp 3	33+00	35+50	5 +/-

Table 2.2 - Excavation (Cut) Sections/Slopes (con't)			
Alignment	**Station**		**Approximate Maximum Cut Depth (ft)**
	Excavation Begin	**Excavation End**	
Ramp 3	35+50	38+50	21 +/-
Ramp 3	41+50	49+50	9 +/-
Ramp 3A	288+50	297+50	13 +/-
Ramp 4 (Wall 14 – Reinforced Concrete from Sta. 60+00 to 65+89.96)	60+000	68+25.46	5 +/-
Ramp 4B	415+50	422+84.17	14 +/-
Ramp 5	112+50	117+00	5 +/-
Ramp 5	124+00	127+00	8 +/-
Ramp 5	131+00	135+00	5 +/-
Ramp 5	142+50	145+61.13	5 +/-
Ramp 10	21+00	31+00	10 +/-
Ramp 11	50+00	55+00	8 +/-
I-85[1]	203+00	223+00	22 +/-
I-85	225+00	229+00	11 +/-
I-85	353+00	374+00	18 +/-
I-85 NB C/D	240+00	251+00	7.5 +/-
I-85	265+00	266+00	6 +/-
I-85 NB C/D	286+00	288+00	15 +/-
I-85 NB C/D	291+50	296+00	25 +/-
I-385	303+89.49	306+00	4 +/-
I-385	312+00	327+50	7 +/-
I-385 (Wall 38 – Pile and Lagging)	423+00.12	446+69.02	10 +/-
I-385 NB C/D	327+79.9	332+00	20+/-
I-385 NB C/D (Wall 21 – Pile and Lagging)	337+09.99	346+44.92	10.5 +/-
I-385 NB C/D	352+00	369+00	20 +/-
I-385 NB C/D (Wall 26 – Soil Nail Wall)	358+65.60	361+32.39	21 +/-
I-385 SB C/D (Wall 27)	116+29.20	122+39.44	22 +/-

Notes: 1. Portion of cut slope at an inclination steeper than 2H:1V
2. Portion of cut includes a retaining wall.

2.4.2 Embankments (Fill) Sections

Embankment fills of up to 39 feet are planned as part of this project, although typical fill heights will be less than 25 feet. In general, embankment fills will be constructed with 2H:1V slope inclinations. Where embankment fills are steeper than 2H:1V, the embankments will be constructed as reinforced soil slopes. The locations of the reinforced slopes are summarized in Table 2.4

Table 2.3 summarizes approximate maximum embankment fill heights and Table 2.4 summarizes the Reinforced Soil Slopes.

Table 2.3 - Embankment (Fill) Sections/Slopes

Alignment	Station		Approximate Maximum Fill Depth (ft)
	Embankment Begin	Embankment End	
Ramp 1	49+65.68	51+00	7.5 +/-
Ramp 1	59+00	76+00	27 +/-
Ramp 1	106+00	109+00	5 +/-
Ramp 1	114+50	120+31.47	8 +/-
Ramp 1A	51+50	55+00	5 +/-
Ramp 1A	58+00	71+29.42	42 +/-
Ramp 1A	89+52.25	97+36.64	45 +/-
Ramp 1B	63+00	67+61.35	30 +/-
Ramp 1B	72+81.40	79+94.00	17 +/-
Ramp 2	56+50	65+00	20 +/-
Ramp 2A	68+00	70+20.81	25 +/-
Ramp 2A	76+89.65	81+00	26 +/-
Ramp 2A	83+00	90+50	5 +/-
Ramp2A	115+00	119+00	10 +/-
Ramp 2B	18+33.96	32+66.95	47 +/-
Ramp 2B	40+41.95	49+41.34	54 +/-
Ramp 3	30+00	32+00	5 +/-
Ramp 3	34+00	36+00	6 +/-
Ramp 3	38+00	48+17.34	42 +/-
Ramp 3A	288+00	289+00	23 +/-
Ramp 3A	295+50	305+00	20 +/-
Ramp 4	38+60.08	59+50	47 +/-
Ramp 4B	378+66.47	388+68.50	42 +/-
Ramp 4B	411+31.33	417+00	24 +/-
Ramp 5	129+00	141+50	28 +/-
Ramp 7	21+16.64	29+45.81	5 +/-
Ramp 8	54+50	56+35.33	10 +/-

Table 2.3 - Embankment (Fill) Sections/Slopes (con't)

Alignment	Station		Approximate Maximum Fill Depth (ft)
	Embankment Begin	Embankment End	
Ramp 8	60+83.70	65+90.06	24 +/-
Ramp 8A	12+50	17+50	5 +/-
Ramp 8A	19+00	21+31.96	17.5 +/-
Ramp 10	31+50	33+16.58	5 +/-
I-85	222+00	224+00	16 +/-
I-85	230+00	235+00	17 +/-
I-85	344+50	357+00	10 +/-
I-85	352+00	375+00	10 +/-
I-85 NB C/D	290+00	293+00	5 +/-
I-385	330+00	343+00	23 +/-
I-385	378+95.5	393+84.84	40+/-
I-385	401+82.01	407+50	7 +/-
I-385 NB C/D	348+50	358+00	10 +/-
I-385 NB C/D	369+00	374+36.28	25 +/-
I-385 SB C/D	97+01.72	104+68.73	41 +/-
I-385 SB C/D	133+00	140+10.55	17 +/-

Table 2.4 - Reinforced Soil Slopes

Alignment	Station Begin	Station End	Approximate Maximum Wall Height (ft)
Ramp 1	62+50	64+00	27 +/-
Ramp 8A	12+00	13+00	7.5 +/-
Ramp 5	137+50	140+50	25 +/-
I-85	230+50	233+40	17 +/-
I-385	337+50	342+53.07	
I-385SB C/D	139+00	140+10.55	

2.4.3 MSE Walls

MSE Walls are being considered for areas where 2:1 slopes are not feasible. The MSE Walls are planned for Cut sections and Fill sections, and in some cases there will be back-to-back MSE Walls. Back-to-back MSE Walls are two parallel MSE Walls that share a common reinforcement and/or retained zone. This report considers external and global MSE Wall stability. Internal wall stability will be evaluated by the wall designer. Back-to-back wall internal stability should be evaluated in accordance with FHWA

Geotechnical Engineering Circular No. 11 – Design and Construction of Mechanically Stablized Earth Walls and Reinforced Soil Slopes (Publication No. FHWA-NHI-10-024). Table 2.5 summarizes the maximum MSE Walls for walls situated more than 150 feet from a bridge end bent.

Table 2.5 - MSE Walls

Wall No.	Alignment	Station Begin	Station End	Cut Wall/ Fill Wall/ Back-to-Back Walls	Approximate Maximum Wall Height (ft)
12	Ramp 1A (RT)	62+99.99	71+29.42	Fill	42 +/-
13	Ramp 1A (LT)	63+00.00	71+29.42	Back-to-Back Wall	42 +/-
1	Ramp 1B	75+46.40	72+83	Fill	21.5 +/-
		72+83.00	76+60.32	Fill	16 +/-
33	Ramp 2B – I-385 SB C/D	40+37.82	49+41.34	Back-to-Back Wall	67.6 +/-
		99+07.93	99+05.20		
10	Ramp 3A	292+63.17	296+94.27	Fill	27 +/-
32	I-385 NB C/D –Ramp 4	379+53.08	380+02.71	Fill	56 +/-
		52+36.35	52+36.35		
36	Ramp 4	50+68.09	54+18.80	Fill	23.5 +/-
2A	I-85	278+91.65	282+31.89	Fill	17 +/-
31B	Ramp 4B	382+12.55	384+67.14	Fill	11 +/-

2.4.4 Pipe Culverts

There are several cross-line pipe culverts on the project as shown on the drainage plans. Borings were performed at each end of the cross-line pipe culverts in accordance with the GDM.

2.5 Field Testing Summary

The SCDOT provided a Geotechnical Data Report prepared by Florence & Hutcheson, an ICA Company (F&H) prepared for the project dated January 25, 2013. In order to satisfy the requirements of the SCDOT GDM Chapter 4, Subsurface Investigation Guidelines and to obtain additional information to evaluate geotechnical aspects of the project, the design-build team contracted with Thompson Engineering (TE) and ECS to obtain additional geotechnical subsurface investigations and laboratory testing based on the final layout of the interchange improvements. The three subsurface explorations are described in the following sections.

2.5.1. Florence & Hutcheson Geotechnical Data Report

Florence & Hutcheson completed a subsurface exploration and laboratory testing program, the results of which were transmitted in a report titled "Geotechnical Data Report", dated January 25, 2013. A total of seventy two (72) preliminary borings were completed (B-1 through B-47, B-49 through B-68, and B-70 through B-74). Borings B-48 and B-69 were omitted. The preliminary borings were made on the original SCDOT Alignments. Some of the alignments were changed during the design-build process. Therefore, some of the borings are located more than 150' from the new alignments.

The F&H boring locations are shown on the figures included in Appendix A. The locations presented are per the latitude/longitude coordinates noted on the F&H boring logs. The boring locations relative to the proposed roadway alignment are also presented in Table B-1 in Appendix B. Those alignments, stations and offsets do not correspond to the positioning presented in the F&H report as the locations presented below have been adjusted to the new alignment not the original alignment from the RFP.

2.5.2. Thompson Engineering Geotechnical Data Report

TE completed a total of one hundred sixty (160) soil borings along the I-85/385 interchange improvement alignments. Those borings were provided in the Geotechnical Subsurface Data Report (GDSR) titled "Interstate 85/385 Interchange Improvements, Roadways and Retaining Walls" and dated August 18, 2015. In addition to those borings, ECS considered borings B01-SPT-01, B01-SPT-06 and B12-SPT-03 in this analysis. The Individual boring logs for borings along the road alignments and (outside 150 feet of the bridges) are provided in Appendix C along with the three (3) bridge borings referenced in this report. The TE Roadway GDSR is provided in Appendix P. Please refer to the individual Bridge BGER for the GDSR associated with the bridge borings.

The borings were drilled utilizing two CME 550X, Diedrich D50, D120, and a tripod drill rig using mud rotary and hollow stem auger drilling techniques. Standard Penetration Tests (SPTs) were conducted at 2-ft. intervals within the top 10 ft. and 5-ft. intervals thereafter until achieving the boring termination depths. The SPT is used to provide an index for estimating soil strength and density. In conjunction with the penetration testing, split barrel soil samples were recovered for soil classification and laboratory testing at various intervals. The N-values presented in the boring logs prepared by TE are uncorrected, field N-values.

A summary of boring locations associated with the I-85/385 interchange improvements are shown in Table B-2 in Appendix B. The water table depths in Table B-2 are reported as the stabilized (24-hr.) water readings, where applicable. When 24-hr water readings were not reported, the water table depth was reported at the 0-hr (time of drilling) elevations.

In addition to the GDSR, TE completed a series of laboratory tests to evaluate materials in cut areas of the project for reuse on as fill including California Bearing Ratio, Triaxial Shear and proctor testing. The results of that laboratory testing are presented in Addendum No. 1 to the GDSR, and are included in Appendix P. At the time this report was prepared the laboratory testing was incomplete. The complete report will be submitted with the revised report.

2.5.3. ECS Subsurface Exploration

ECS completed a total of twenty-two (22) soil borings along the I-85/385 interchange improvement alignments. Individual boring logs for borings along the road alignments and (outside 150 feet of the bridges) are provided in Appendix C.

The borings were drilled utilizing a two CME 550 drill rigs with hollow stem augers. Standard Penetration Tests (SPTs) were conducted at 2-ft. intervals within the top 10 ft. and 5-ft. intervals thereafter until achieving the boring termination depths. The SPT is used to provide an index for estimating soil strength and density. In conjunction with the penetration testing, split barrel soil samples were recovered for soil classification and laboratory testing at various intervals. The N-values presented in the boring logs prepared by ECS are uncorrected, field N-values.

A summary of boring locations associated with the I-85/385 interchange improvements are shown in Table B-3 in Appendix B. The water table depths in Table B-3 are reported as the stabilized (24-hr.) water readings, where applicable. When 24-hr water readings were not reported, the water table depth was reported at the 0-hr (time of drilling) elevations. Note that several borings are planned and have not yet been performed. Those borings are shown in Table B-3.

2.6 Laboratory Testing Summary

TE, ECS and F&H completed laboratory testing programs. The laboratory results performed by TE and F&H are summarized in Appendix P and Appendix Q respectively. The laboratory results performed by ECS are provided in Appendix N. Table 2.6 is a summary of the laboratory tests performed by TE, ECS, and F&H for the borings in the vicinity of the I-85/385 Interchange roadways.

Table 2.6 Summary of Laboratory Test Quantities

Test Type	Quantity		
	F&H	TE	ECS
Atterberg Limits	268	687	25
Full Sieve Analysis	348	686	--
Hydrometer Analysis	--	105	--
% Passing the #200 Sieve	--	2	25
Moisture Content	347	688	26

In addition to the I-85/385 interchange specific laboratory test data, TE and F&H performed advanced laboratory testing including shear strength testing and consolidation testing at various locations across the general project site. Because the entire project is situated within a region of similar geologic origin, the laboratory test results were considered in our analysis. Table 2.7 is a summary of the TE and F&H advanced laboratory tests on undisturbed (Shelby Tube) samples.

Table 2.7 Summary of Advanced Laboratory Tests

Consultant	Boring Number	Sample Number	Depth (ft)	USCS Classification	Laboratory Test
F&H	B-13	ST-1	20.3-20.8	SM	Triaxial Compression
F&H	B-39	ST-2	8-9.2	CL	Consolidation
F&H	B-39	ST-1	4-5.3	SC	Triaxial Compression
F&H	B-39	ST-2	8-9.2	CL	Triaxial Compression
F&H	B-40	ST-1	6-7.5	SM	Consolidation
F&H	B-40	ST-3	10-11.3	SM	Triaxial Compression
F&H	B-40	ST-2	8-9.2	SM	Triaxial Compression
F&H	B-40	ST-1	6-7.5	SM	Triaxial Compression
F&H	B-43	ST-1	2-2.9	SC	Triaxial Compression
F&H	B-44	ST-1	4-5	SM	Triaxial Compression
F&H	B-44	ST-2	8-8.9	SM	Triaxial Compression
F&H	B-46	ST-1	4-4.8	SM	Triaxial Compression
F&H	B-46	ST-2	8-9.5	SM	Triaxial Compression

Table 2.7 Summary of Advanced Laboratory Tests (con't)

Consultant	Boring Number	Sample Number	Depth (ft)	USCS Classification	Laboratory Test
F&H	B-49	ST-2	8-9.3	SM	Triaxial Compression
F&H	B-49	ST-1	4-4.8	SM	Triaxial Compression
F&H	B-51	ST-2	6-7.2	SM	Triaxial Compression
F&H	B-53	ST-2	8-9.3	SM	Triaxial Compression
F&H	B-54	ST-1	4-5.7	SM	Triaxial Compression
F&H	B-54	ST-2	8-9.5	ML	Triaxial Compression
F&H	B-61	ST-1	2-3.3	SM	Consolidation
F&H	B-61	ST-1	2-3.3	SM	Triaxial Compression
F&H	B-64	ST-1	4-5.2	SM	Triaxial Compression
F&H	B-64	ST-2	8-9.5	SM	Triaxial Compression
F&H	B-65	ST-2	10-11.4	SM	Consolidation
F&H	B-65	ST-2	10-11.4	SM	Triaxial Compression
F&H	B-67	ST-1	4-4.7	SM	Consolidation
F&H	B-67	ST-1	4-4.7	SM	Triaxial Compression
F&H	B-68	ST-1	2-3.5	SP-SM	Triaxial Compression
F&H	B-68	ST-2	6-7.3	SM	Triaxial Compression
F&H	B-70	ST-1	6-6.6	SM	Triaxial Compression
F&H	B-74	ST-1	4-5.3	ML	Consolidation
F&H	B-74	ST-1	4-5.3	ML	Triaxial Compression
TE	B01-SPT-09	T-1	19-21	SC	Triaxial CU
TE	B01-SPT-14	T-1	25-27	CL	Triaxial CU
TE	B06-SPT-12	T-3	35-37	ML	Triaxial CU
TE	BR11-SPT-02	T-1	9.5-11.5	ML	Triaxial CU
TE	R2-43	T-1	10-12	SM	Direct Shear
TE	RRM-47	T-1	25-27	ML	Triaxial CU
TE	W1B-2R-02	T-1	8-10	SM	Triaxial CU
TE	W1B-2R-03	T-1	4-6	CL	Triaxial CU
TE	W1B-2R-03	T-2	15-17	SM	Triaxial CU
TE	W2A-MB2-01	T-1	10-12	CL	Triaxial CU
TE	W3A-1R-01	T-1	12-14	CL	Triaxial CU
TE	W4B-1L-02	T-2	8-10	ML	Direct Shear
TE	WCR-1L-02	T-1	15-17	ML	Triaxial CU

3 SUBSURFACE CONDITIONS

3.1 Geology

The United States Geologic Survey (USGS) presents the I-85/I-385 Interchange improvement project site within the limits of the Mauldin 7.5 minute topographic quadrangle map. The Geologic Map of the Greenville 1°x2° Quadrangle, South Carolina, Georgia, and North Carolina (Arthur E. Nelson, J. Wright Horton, Jr., and James W. Clarke dated April 12, 1990), identifies the project within the Inner Piedmont Physiographic Province of South Carolina. The Piedmont Province consists mainly of residual soils underlain by parent bedrock. The Generalized Geologic Map of South Carolina (revised by Willoughby, Howard, and Nystrom in 2005) identifies parent bedrock within this this region in the Sixmile thrust sheet limits. The Sixmile Thrust sheet contains muscovite-biotite schist, biotite schist, sillimanite-mica schist and gneiss, amphibolite, biotite gneisses including some that are porphyroblastic, felsic gneiss, and some manganiferous schist and metamorphosed manganese silicate.

The native soils in the Piedmont Province consist mainly of residuum with underlying saprolites weathered from the parent bedrock (Sixmile thrust sheet), which can be found in both weathered and unweathered states. Although the surficial materials (residual soils) normally retain the structure of the original parent bedrock, they typically have a much lower density and exhibit strengths and other engineering properties typical of soil. In a mature weathering profile of the Piedmont Province, the soils are generally found to be finer grained at the surface where more extensive weathering has occurred. The particle size of the soils generally becomes more granular with increasing depth and gradually changes first to weathered and finally to unweathered parent bedrock. The mineral composition of the parent rock and the environment in which weathering occurs largely control the residual soil engineering characteristics.

The boundary between soil and rock is not sharply defined. This transitional zone termed “partially weathered rock” (PWR) is normally found overlying the parent bedrock. Partially weathered rock is defined, for engineering purposes and by Section 11.4 of the GDM, as residual material with Standard Penetration Test resistances greater than 100 blows per foot (bpf). The partially weathered rock is considered in geotechnical engineering as an Intermediate Geomaterial (IGM). The degree of weathering is facilitated by fractures, joints, and the presence of less resistant rock types. Consequently, the profile of the PWR and hard rock is generally irregular and erratic, even over short horizontal distances.

Alluvial soils in the piedmont occur in river and stream flood plains. The engineering characteristics of the alluvium are dependent on the depositional environmental.

The natural geology across the project extent has been modified by past grading that included cut excavation and embankment fill, in most cases associated with the existing I-85/385 interchange.

3.2 Subsurface Information

A total of two-hundred-fifty-four (254) borings were drilled along or adjacent to the I-85/385 interchange alignments and were considered in this report.

A Boring Location Plan is attached in Appendix A. The boring locations are represented on the drawing based on station and offset provided on the F&H, TE and ECS boring logs. The referenced boring logs and associated test data are presented in Appendix C and included in the individual GSDR in Appendix P and Q of this report.

For the purpose of this report, we have identified a “PWR Lens” as a layer of partially weathered rock (located within the residual soil zone) having a thickness of 5 feet or less. A “PWR Layer” is referred to as a layer of partially weathered rock (also within the residual soil zone) with a thickness greater than 5 feet, overlying a deeper residual soil layer “Continuous PWR” refers to the layer of PWR that is encountered directly above the bedrock layer.

3.3 Groundwater

Groundwater measurements were not reported on the F&H boring logs. Groundwater measurements were attempted by ECS and TE at the termination of drilling and at least 24 hours after completion of drilling (when possible without impacting the health and safety of the traveling public) as summarized in Appendix B of this RGER. Groundwater was encountered at several boring locations at depths ranging from approximately 0.8 to 42.2 feet below the ground surface which corresponds to elevations ranging from 847.6 to 1062.9 feet. Measured groundwater depths and elevations are provided in the tables in Appendix B.

Fluctuations in the groundwater elevation should be expected depending on precipitation, run-off, utility leaks, and other factors not evident at the time of our evaluation. During prolonged rainy or cold seasons shallower perched water conditions can develop where surface water becomes trapped above less permeable fine grained soils. Normally, the highest groundwater levels occur in late winter and spring and the lowest levels occur in late summer and fall. Depending on time of construction, groundwater may be encountered at more elevated or shallower depths and at locations not explored during this study.

4 GEOTECHNICAL SEISMIC CONSIDERATIONS

4.1 Seismic Design

Based on the GDM Section 8.9.1 and Bridge Design Memorandum DM0211, the Operational Category for the bridges on this project is OC=I.

The Roadway Structure Operational Classification (ROC) will be the same as the bridges for embankments within one hundred fifty (150') feet of the end of the bridges, which is ROC=I for all bridges with the exception of the Roper Mountain Road and Woodruff Road bridges. The Roper Mountain Road and Woodruff Road Roadway Operational Category is ROC=II. Roper Mountain Road is addressed under a separate covers for Bridge and Roadway. Woodruff Road is discussed in this report with respect to the construction of new retaining walls adjacent to the existing bridge.

For the roadway embankments beyond these limits, the Roadway Structure Operational Classification is ROC=IV with the exception of portions of the roadway with structures (e.g. retaining walls) which are classified as ROC=III. ROC=I is required for embankments with a flexible walls heights greater than fifty (50') feet. A MSE Wall height greater than 50 feet is planned along Ramp 2B (Wall 33) between stations 40+32.75 to about station 42+75; therefore, this portion of the roadway embankment has a ROC=I. The ROC for various portions of the interchange are provided in Table 2.1. The ROC for structures and embankments considered in this RGER are summarized as follows:

- ROC=I: Flexible Walls with heights greater than 50 ft.
- ROC=II: Flexible retaining walls adjacent to Woodruff Road overpass.
- ROC=III: All other retaining walls and reinforced earth structures (i.e. Reinforced Soil Slopes)
- ROC=IV: All other embankments not listed above.

4.2 Seismic Response

The SCDOT provided a three point Acceleration Design Response Spectrum (ADRS) curve dated March 27, 2014 based on a Site Class D. The site classification is based on four (4) Multichannel Analysis of Surface Wave (MASW) tests performed by F&H as part of their geotechnical report date January, 2013. The MASW results are presented in Appendix E of this report, and the results are summarized in Table 4.1 below. The testing indicates a weighted average shear wave velocity of 1,145 feet per second (fps), which correlates to a Site Class D based on the procedures outlined in the SCDOT GDM. Table 4.2 summarizes the ADRS parameters provided by SCDOT.

Table 4.1 MASW Test Results

MASW Analysis No.	Alignment	Station	Offset (feet)	Average Shear Wave Velocity in Top 100 feet (fps)
MASW-1	I-385 NB C/D	359+39	17' RT	1,405.6
MASW-2	I-385	393+66	115' RT	1,034.8
MASW-3	Ramp 4B	408+70	102' RT	1,081.5
MASW-4	Roper Mt. Rd.	36+15	25' LT	1,060.2

Table 4.2 Summary of ADRS Seismic Design Values

Design EQ	PGA	S_{DS}	S_{D1}	M_W	R (km)	Geologic Condition	Site Class
FEE	0.07	0.11	0.06	7.37	267.2	Hard Rock Basement Outcrop	D
SEE	0.20	0.29	0.14	7.37	266.4	Hard Rock Basement Outcrop	D

Based on the shear wave velocity measurements, the seismic Site Class for the ~~bridge~~ roadway has been determined to be a “D”.

4.3 Seismic Soil Shear Strength Loss and Liquefaction Triggering

A geotechnical seismic hazard evaluation was performed to determine if the soils located within the roadway project limits are susceptible to Soil Shear Strength Loss (SSL) and or Liquefaction during the design seismic events. Soil Shear Strength Loss (SSL) and seismic settlements were evaluated using the procedures outlined by Idriss and Boulanger (2008) and Chapter 13 – “Geotechnical Seismic Hazards” of the 2010 GDM to determine soil SSL.

The SPT field results, N_{Meas}, have been corrected to account for energy losses, normalized to a reference overburden pressure of 1 tsf (1 atm), and corrected for fines content to an equivalent clean sand. The corrected SPT penetration results were used to estimate static soil shear strengths, evaluate soil shear strength loss (SSL), estimate seismic soil shear strengths, and estimate seismic settlement.

The SSL and Liquefaction triggering analysis considers an age factor to account for the reduction in SSL and Liquefaction potential as geologic formations age. For SSL and Liquefaction triggering analysis, the soils across the general project extent are grouped into three (3) geologic origins including man-made fills, Alluvial Soils and Piedmont Residual Soils. Age Factors of 1.2, 1.5 and up to 2.5 are used for man-made fills, Alluvial Soils, and Piedmont Residual Soils respectively. The Age Factor for Alluvial soils considers an age of at least 10,000 years (based on Table 13.4 of the GDM) and was estimated with GDM Equation 13-47 where $K_{DR} = 0.17^*\text{Log}_{10}(t)+0.83$ and t is in years. For existing fills associated with existing approach embankments an Age Factor of 1.2 was used in the analysis. The age factor for existing fills accounts for the age of the deposit (40 to 50 years based on original bridge construction), as well as the compactive effort, or artificial aging due to compaction.

The analysis was performed using a Moment Magnitude of 7.37 and a Peak Ground Acceleration of 0.20g and 0.07g for the SEE and FEE design events, respectively. The potential for seismic soil shear strength loss (SSL) and liquefaction of the subsurface soils was evaluated by first screening the SPT soil borings to determine if the soils encountered are susceptible to soil shear strength loss. Soils identified as susceptible to soil SSL, were then evaluated to determine if the seismic demand (FEE and SEE) was capable of triggering soil SSL.

The SSL and liquefaction triggering evaluation analysis was performed on all borings considered in the global stability or settlement analysis. That analysis indicated that triggering and shear strength loss will not occur at the FEE or SEE event, and seismic induced soil settlement will generally be less than ¼”, except for the analyses preformed on borings R2A-104, BX-385-01, W4-1R-09, and R8A-31 where seismic settlement is estimated at 0.28”, 0.62”, 0.28” and 0.27” respectively. The estimated settlement will not exceed performance tolerances of embankments or roadway structures. Example and representative borings and evaluations are presented in Appendix F of this report. The seismic soil settlement and triggering analysis is limited to non-man-made fill materials or material above the groundwater level.

5 GEOTECHNICAL RECOMMENDATIONS

Based on the borings, laboratory test results, existing borings and our experience, the following recommendations are presented. The design is based on the field investigation available at the time of this report and comments from SCDOT.

5.1 Roadway Operational Category

The Roadway Structure Operational Classification (ROC) varies depending on proximity to roadway structure. Table 5.1 summarizes the ROC definition as reproduced from Table 8-11 in Bridge Design Memorandum DM0211.

Table 5.1 – Roadway Structure Operational Classification (SCDOT GDM Table 8-11)

Roadway Structure Operational Classification (ROC)	Description
I	Roadway embankments located within 150 feeet of a bridge with OC=I Roadway structures located within 150 feet of a bridge with OC=I Rigid walls with heights greater than 15 feet. Flexible walls with heights greater than 50 feet.
II	Roadway embankments located within 150 feeet of a bridge with OC=II Structures (not classified as ROC=I) located within 150 feet of a bridge with OC=II
III	Roadway embankments located within 150 feeet of a bridge with OC=III Structures (not classified as ROC=I) located within 150 feet of a bridge with OC=III Roadway embankments located within 150 feeet of a bridge with OC=III Structures (not classified as ROC=I) located more than 150 feet from a bridge
IV	Roadway embankments located more than 150 feet from a bridge.

In general a ROC=IV is considered for the interchange improvements for roadways and embankments situated more than 150 feet from a bridge. Roadway embankments with structures (i.e. retaining walls) are classified as ROC=III. ROC=I is required for embankments with a flexible walls heights greater than fifty (50’) feet. A MSE Wall height greater than 50 feet is planned along Ramp 2B (Wall 33) between stations 40+32.75 to about station 42+75; therefore, this portion of the roadway embankment has a ROC=I. The ROC for various portions of the interchange are provided in Table 2.1. The ROC for structures and embankments considered in this RGER are summarized as follows:

- ROC=I: Flexible Walls with heights greater than 50 ft.
- ROC=II: Flexible retaining walls adjacent to Woodruff Road overpass.
- ROC=III: All other retaining walls and reinforced earth structures (i.e. Reinforced Soil Slopes)
- ROC=IV: All other embankments not listed above.

5.2 Geotechnical Resistance Factors

The following tables are the geotechnical resistance factors utilized in our analyses of embankments, cut sections, MSE walls, and reinforced soil slopes, and can be found in the Bridge Design Memorandum – DM0310, dated July 22, 2010. Bridge Design Memorandum DM0211 requires all embankments classified as ROC=IV be evaluated for strength and service limit states only. Furthermore, DM0211 states that the resistance factors and performance limits for embankments classified as ROC=III shall be the same as the requirements for embankments classified as ROC=III.

Table 5.2 SCDOT Resistance Factors for Flexible Retaining Walls (SCDOT GDM Table 9-7)

Performance Limit		Limit States		
		Strength	Service	Extreme Event
Soil Bearing Resistance		0.65	N/A	1.00
Sliding Frictional Resistance		1.00	N/A	1.00
Lateral Displacement		N/A	1.00	1.00
Vertical Settlement		N/A	1.00	1.00
Global Stability Fill Walls	ROC- I, II	N/A	0.65	0.90
	ROC= III		0.75	1.00
Global Stability Cut Walls	ROC- I, II	N/A	0.60	0.90
	ROC= III		0.70	1.00

Table 5.3 SCDOT Resistance Factors for Embankments (Fill/Cut Section) (SCDOT GDM Table 9-9)

Performance Limit		Limit States		
		Strength	Service	Extreme Event
Lateral Displacement		N/A	1.00	1.00
Vertical Settlement		N/A	1.00	1.00
Global Stability Embankment (Fill)	ROC= I, II	N/A	0.65	0.90
	ROC= III		0.75	1.00
Global Stability Cut Section	ROC= I, II	N/A	0.60	0.90
	ROC= III		0.70	1.00

Table 5.4 SCDOT Resistance Factors for Reinforced Soils (SCDOT GDM Table 9-10)

Performance Limit		Limit States		
		Strength	**Service**	**Extreme Event**
Tensile Resistance of Metallic Reinforcement and Connectors[1]	Strip Reinforcement	0.75	N/A	1.00
	Grid Reinforcement[2]	0.65		0.85
Tensile Resistance of Geosynthetic Reinforcement and Connectors		0.90	N/A	1.20
Pullout Resistance of Tensile Reinforcement		0.90	N/A	1.20

1. Apply to gross cross-section less sacrificial area. For sections with holes, reduce the gross area and apply to net section less sacrificial area.
2. Applies to grid reinforcements connected to a rigid facing element (concrete panel or block). For grid reinforcements connected to a flexible facing mat or which are continuous with the facing mat, use the resistance factor for strip reinforcements.

5.3 Excavation (Cut) Sections

This report addresses the cut sections along the I-85/385 interchange more than 150 feet from the bridge abutments. Cut sections within 150 feet of the bridge end bents will be discussed in the BGER for each bridge or the project RGER. The design approach for Cut Walls was previously submitted to SCDOT in a design memorandum. That memorandum is included in Appendix S of this report.

The cut sections on this project range in depth from approximately 0 to 25 feet. Cut sections on this project are grouped into three categories:

- Cut slopes with slope inclinations shallower than 2H:1V.
- Cut slopes with slope inclinations between 1.7 to 2H:1V.
- Cut sections with retaining walls (Soil Nail, MSE or Soldier Pile and Lagging).

Cut slopes steeper than 2H:1V and cut walls are identified in Table 2.2. The Resistance Factors for Embankments (Fill/Cut Section) and Resistance Factors for Flexible Retaining Walls tables in section 5.2 of this report summarize the minimum resistance factors for cut sections (embankment and walls) based on Tables provided in Bridge Design Memorandum – DM0310. Cut slopes were evaluated for Strength and Service load cases per Bridge Design Memorandum DM0211. Cut walls were evaluated for Strength and Extreme Event states.

Stability analyses were performed at selected locations, generally at locations where the higher cut segments are planned. The stability analysis is further discussed in section 5.8 of this report. The results of the cut slope static stability analyses are presented in Table 5.15. The global stability analyses are presented in Appendix K. The strength values presented in Appendix H have been used in the analyses.

Each of the cut walls, not including soil nail walls, will be designed by a structural engineer and are planned as soldier pile walls with concrete lagging panels. The SCDOT GDM does not provide specific guidance or requirements for establishing lateral earth pressure diagrams for cantilevered soldier pile and lagging walls, but rather refers to the AASHTO and FHWA guidance. As such, we recommend using the lateral earth pressure diagrams provided in Chapter 3.11.5.6 of the AASHTO LRFD Bridge Design Specifications (Figures 3.11.5.6-1 to 3.11.5.6-3) for permanent cantilever soldier pile and lagging walls.

We recommend soldier pile and lagging walls be designed with the following parameters:

- Rankine Active earth pressure coefficients are recommended for all cantilever soldier and pile lagging walls supporting roadways, embankments, or landscape areas.
- Rankine At-Rest pressure coefficients are recommended for all soldier pile and lagging walls situated near existing structures or in movement sensitive areas (i.e. where wall deflections of less than ½" are required).
- Rankine Passive earth pressure coefficients are recommended for all soldier pile and lagging walls with an embedment of less than 12 feet.
- A passive earth pressure coefficient based on a log-spiral failure surface is recommended for soldier pile and lagging walls with embedment of greater than 12 feet. In our opinion, the log-spiral failure surface, such as presented on Figures 16 and 17 in FHWA Geotechnical Engineering Circular No. 4 - Ground Anchors and Anchored Systems, provides a more realistic passive pressure distribution than Rankine parameters and based on our experience and this method is generally accepted by the FHWA and.
- For walls with sloping backfill we recommend Rankine Active earth pressure coefficients. The At-Rest coefficient provides a conservative estimate of the increased earth pressure due to sloping ground.

Appendix J summarizes the recommended design parameters (soil unit weights and earth pressure coefficients) for each of the soldier pile and lagging wall envelopes.

There are several locations along the roadway alignment where proposed drainage structures are situated in front of (i.e. parallel) to the retaining wall, or where new and existing draining structures pass beneath the retaining walls. Where new pipes are parallel to the proposed wall, the pipe should be installed prior to the proposed wall or the wall design should account for the temporary reduction in passive resistance. Where pipes pass beneath walls, the pipes should be designed to account for the increased loading associated with the wall backfill. We recommend the top of each pipe be situated a minimum of 1 foot below the bottom of retaining.

Partially Weathered Rock (PWR) is anticipated above the pile tip elevation at several pile and panel wall locations. The contractor should be prepared to predrill should pile refusal occur during pile driving operations. Pre-drilling is anticipated at Wall 12 from Station 51+00 to 55+00, Wall 27, and Wall 38 from Station 438+30 to 446+69. Where predrilling is used, the pre-drilled hole shall be backfilled with lean concrete or flowable fill.

5.4 Embankments (Fill) Sections

This report addresses embankment sections along the I-85/385 interchange more than 150 feet from the bridge abutments. Embankments within 150 feet of the bridge end bents and for Roper Mountain Road and Chrome Drive are discussed in individual BGER for each bridge or the main Roper Mountain Road and Chrome Drive RGER.

The fill section heights on this portion of the project range from approximately 0 to 38.5 feet. We anticipate that all fill slopes will be at an inclination of 2H:1V or less. Critical sections of the 2H:1V fill slopes were analyzed for stability analysis per SCDOT requirements. Fill slopes steeper than 2V:1H are designed as reinforced soil slopes and are discussed in section 5.7 of this report. The Resistance Factors for Embankments (Fill/Cut Section) table in section 5.2 summarizes the minimum resistance factors for fill sections based on Table 9-9 from the Bridge Design Memorandum – DM0310. Fill sections were evaluated for Strength and Service Event load cases per Bridge Design Memorandum DM0211.

Should seeps or thick lenses of highly plastic soils be observed in the planned fill and cut slopes that are steeper than 2H:1V, ECS should be consulted to determine if the steeper slopes may be constructed as planned or if slope flattening or reinforcing is required. Similarly, if soft or wet ground conditions are observed at the base of planned fill embankments, the QA representative should determine the limits of undercutting required or required in-situ treatment.

Stability analyses were performed at selected locations, generally at locations where the higher fill embankments are planned or where softer ground conditions were encountered. The stability analysis is further discussed in section 5.8 of this report. The results of the static stability analyses are presented in Table 5.14. Calculations of the slope stability analyses are presented in Appendix K. The strength values presented in Appendix H were used in the analyses.

Several of the new embankment fills consist of shoulder or widening fills of existing embankment. Where the new fill meets the existing slope, the existing slope should be benched to limit the potential for a preferential failure surface and to allow compaction at the interface. Benches should have a minimum horizontal length of 8 feet and a vertical rise of no more than 3 feet. Fill slopes of 2H:1V or steeper should be overbuilt (i.e. fill should temporarily extend beyond the final slope face) to allow compaction at the slope face. After compaction is complete, the slope may be re-graded to the final inclination.

Settlement analyses have also been performed for the critical fill segments. The settlement analysis is further discussed in section 5.9 of this report, and the results of those analyses are presented in Tables 5.21 to 5.22. The settlement analysis was completed using the computer program FoSSa by Adama Engineering, Inc.

5.4.1 Barrier Walls

This report addresses barrier walls along the I-85/385 interchange. Barrier walls should be designed as rigid reinforced concrete gravity walls. The Resistance Factors for Rigid Gravity Retaining walls table in Section 5.2 summarizes the minimum resistance factors based on Table 9-6 from the SCDOT Geotechnical Design Manual.

Barrier wall heights on this portion of the project range from approximately 3 to 14.5 feet. Barrier walls greater than 5 feet were evaluated for overall stability per the SCDOT requirements, and results are presented in subsequent sections of this report. In general, we do not anticipate global stability issues for the barrier walls. Retaining walls should be designed to withstand the lateral earth pressures exerted upon them, and to resist additional lateral pressures generated by surcharge loads such as traffic loads, adjacent slab loads or from foundations bearing behind the walls.

For wall conditions where wall movement cannot be tolerated or where the wall is restrained at the top, such as the loading dock walls, the “At Rest” earth pressure should be used. For wall conditions where outward wall movement on the order of 0.5 percent of the wall height can be tolerated, the “Active” earth pressure should be used. In the design of barrier walls to restrain compacted backfill, engineered fill or in-situ residual soils, the coefficient of lateral earth pressure can be used to determine lateral earth pressure loads. We recommend the following earth pressure coefficients for walls backfilled with Borrow Material:

- “At Rest” Earth Pressure (K_o), 0.50
- “Active” Earth Pressure (K_a), 0.33
- “Passive” Earth Pressure (K_p), 3.0

The lateral earth pressure values presented above assume level backfill fill behind the wall, and do not account for hydrostatic pressures against the walls or surcharge loads from overlying or nearby construction.

Resistance to sliding can be provided by friction between the bottom of the wall foundation and the underlying soils and by passive resistance of soil adjacent to the wall foundation. The passive resistance should only be used in situations where the soil adjacent to the toe of the wall will not be eroded or otherwise removed in the future. A coefficient of friction of 0.40 for concrete bearing on approved soils is recommended.

Drainage behind freestanding retaining walls is considered essential towards relieving hydrostatic pressures. Drainage can be established by providing a perimeter drainage system located just above the below grade/retaining wall footings which discharges by gravity flow to a suitable outlet.

There are several locations along the roadway alignment where proposed drainage structures pass beneath the barrier walls. Where pipes pass beneath walls, the pipes should be designed to account for the increased loading associated with the wall backfill. We recommend the top of each pipe be situated a minimum of 1 foot below the bottom of retaining.

SCDOT Table 805-811A provided design criteria for standard concrete barrier retaining walls. In general the soils encountered near the barrier walls meet the required design criteria with a minimum friction angle equal to or greater than 28 degrees, a static bearing pressure in excess of 1100 psf, and a seismic bearing pressure in excess of 1250 psf, an ultimate bearing pressure in excess of 3300 psf, and a coefficient of sliding friction in excess of 0.40.

5.5 MSE Wall Geotechnical Recommendations

This report addresses the MSE walls along the I-85/385 interchange. MSE walls along within 150 feet of the bridge end bents will be discussed in their respective BGER.

The MSE walls were evaluated for overall stability, soil bearing, sliding, overturning, and settlement per SCDOT requirements using boring and laboratory test data at or near the vicinity of the MSE wall locations. The Resistance Factors for Flexible Retaining Walls table in section 5.2 summarizes the minimum resistance factors for embankment sections based on Table 9-7 from the Bridge Design Memorandum – DM0310.

The MSE wall heights on this portion of the project range from approximately 0 to 58 feet. In general, we do not anticipate global stability issues for MSE walls. Based on the subsurface conditions fill slopes may generally be designed as 2H:1V slopes or flatter. Slope inclinations steeper than 2H:1V will require internal reinforcement and should be designed as reinforced soil slopes as discussed in section 5.7 of this report.

The MSE Wall reinforced zone was modeled based on soil properties outlined Appendix H of this report. This analysis assumed a reinforced zone approximately 0.7 to 1.7 times the wall height at all sections, and assumes that the MSE wall will be designed by an MSE wall design/builder licensed in South Carolina, and that internal stability of the wall will meet or exceed AASHTO and SCDOT requirements. Additional information regarding the soil parameters and the results of the global stability analyses are located in Section 6.2 of this report.

Based on the results of the analysis, the available subsurface information, and our experience in the Piedmont formation, the proposed MSE Walls will exhibit geotechnical resistance factors for bearing capacity in accordance with SCDOT requirements.

Table 5.5 - MSE Wall External Stability Analysis Results

Wall Number/Location	**Design Height, H,,** ft[1]	**Minimum Reinforcement Length, B_{req}, ft (%H)**	**Calculated Resistance Factor**		**Max. Factored Bearing Resistance,** psf
			Bearing Capacity[2]	**Sliding**[3]	
Walls 12 and 13	39 < H ≤ 41.5	29.5 ft (0.71H)	0.55	0.76	12,604
	29 < H ≤ 39	27.5 ft (0.71H)	0.56	0.77	11,643
	21 < H ≤ 29	20.5 ft (0.71H)	0.53	0.82	9,757
	15 < H ≤ 21	16 ft (0.76H)	0.42	0.82	8,900
	9 < H ≤ 15	12 ft (0.80H)	0.38	0.86	7,513
	H ≤ 9	9 ft (1.0H)	0.28	0.85	6,255
Wall 33	58 < H ≤ 68	58 ft (0.85H)	0.63	0.56	14,974
	49 < H ≤ 58	49.5 ft (0.85H)	0.64	0.57	12,725
	39 < H ≤ 49	42.5 ft (0.87H)	0.64	0.57	10,966
	29 < H ≤ 39	35 ft (0.90H)	0.62	0.57	9,118
	21 < H ≤ 29	25.5 ft (0.88H)	0.58	0.61	7,594
	15 < H ≤ 21	18.5 ft (0.88H)	0.63	0.66	5,420
	9 < H ≤ 15	13 ft (0.87H)	0.63	0.75	4,226
	H ≤ 9	9 ft (1.0H)	0.51	0.79	3,455
Wall 10	21 < H ≤ 27	21 ft (0.78H)	0.63	0.79	6,833
	15 < H ≤ 21	17 ft (0.81H)	0.62	0.80	5,592
	9 < H ≤ 15	13 ft (0.87H)	0.59	0.84	4,347
	H ≤ 9	9.5 ft (1.06H)	0.49	0.85	3,383
Wall 32	49 < H ≤ 56	49 ft (0.88H)	0.45	0.59	17,306
	39 < H ≤ 49	49 ft (1.0H)	0.36	0.53	18,223
	29 < H ≤ 39	30 (0.76H)	0.65	0.72	9,523
	21 < H ≤ 29	23 ft (0.79H)	0.63	0.73	7,448
	15 < H ≤ 21	17 ft (0.81H)	0.65	0.77	5,517
	9 < H ≤ 15	13 ft (0.86H)	0.63	0.80	4,254
	H ≤ 9	9 ft (1.0H)	0.56	0.85	3,128
Wall 36B	15 < H ≤ 23.5	18 ft (0.77H)	0.64	0.80	6,002
	9 < H ≤ 15	13 ft (0.87H)	0.51	0.80	5,036
	H ≤ 9	9 ft (1.0H)	0.46	0.85	3,627
Wall 1	15 < H ≤ 21.5	16 ft (0.74H)	0.36	0.78	10,785
	9 < H ≤ 15	12 ft (0.80H)	0.31	0.81	9,220
	H ≤ 9	9 ft (1.0H)	0.23	0.81	7,665
Wall 2A	9 < H ≤ 17	13 ft (0.76H)	0.38	0.80	8,185
	H ≤ 9	8 ft (0.89H)	0.34	0.88	5,320
Wall 31B	H ≤ 11	11 ft (1.0H)	0.35	0.94	5,589

Table 5.6 - MSE Wall External Stability Analysis Results (Extreme Event I)

Location	**Design Height, H,** ft[1]	**Minimum Reinforcement Length, B_{req}, ft (%H)**	**Calculated Resistance Factor**		**Max. Factored Bearing Resistance,** psf
			Bearing Capacity[2]	**Sliding**[3]	
Walls 12 and 13	39 < H ≤ 41.5	29.5 ft (0.71H)	0.84	0.86	19,390
	29 < H ≤ 39	27.5 ft (0.71H)	0.91	0.88	17,913
	21 < H ≤ 29	20.5 ft (0.71H)	0.90	0.93	15,010
	15 < H ≤ 21	16 ft (0.74H)	0.60	0.93	13,693
	9 < H ≤ 15	12 ft (0.80H)	0.51	0.99	11,558
	H ≤ 9	9 ft (1.0H)	0.27	0.92	9,623
Wall 33	58 < H ≤ 68	58 ft (0.85H)	0.65	0.61	23,037
	49 < H ≤ 58	49.5 ft (0.85H)	0.67	0.63	19,577
	39 < H ≤ 49	42.5 ft (0.87H)	0.66	0.64	16,870
	29 < H ≤ 39	35 ft (0.90H)	0.63	0.65	14,028
	21 < H ≤ 29	25.5 ft (0.88H)	0.61	0.70	11,683
	15 < H ≤ 21	18.5 ft (0.88H)	0.67	0.75	8,339
	9 < H ≤ 15	13 ft (0.87H)	0.73	0.86	6,501
	H ≤ 9	9 ft (1.0H)	0.49	0.86	5,316
Wall 10	21 < H ≤ 27	21 ft (0.78H)	0.75	0.92	10,513
	15 < H ≤ 21	17 ft (0.81H)	0.70	0.94	8,603
	9 < H ≤ 15	13 ft (0.87H)	0.64	0.99	6,687
	H ≤ 9	9.5 ft (1.06H)	0.44	0.95	5,204
Wall 32	49 < H ≤ 56	49 ft (0.88H)	0.46	0.66	26,625
	39 < H ≤ 49	49 ft (1.0H)	0.33	0.59	28,036
	29 < H ≤ 39	30 ft (0.76H)	0.85	0.82	14,650
	21 < H ≤ 29	23 ft (0.79H)	0.79	0.83	11,458
	15 < H ≤ 21	17 ft (0.81H)	0.79	0.87	8,487
	9 < H ≤ 15	13 ft (0.86H)	0.74	0.92	6,545
	H ≤ 9	9 ft (1.0H)	0.54	0.92	4,812
Wall 36B	15 < H ≤ 23.5	18 ft (0.77H)	0.80	0.97	9,234
	9 < H ≤ 15	13 ft (0.87H)	0.56	0.99	7,748
	H ≤ 9	9 ft (1.0H)	0.44	1.0	5,580
Wall 1	15 < H ≤ 21.5	16 ft (0.74H)	0.50	0.85	16,593
	9 < H ≤ 15	12 ft (0.80H)	0.39	0.89	14,185
	H ≤ 9	9 ft (1.0H)	0.22	0.84	11,793
Wall 2A	9 < H ≤ 17	13 ft (0.76H)	0.55	0.96	12,593
	H ≤ 9	8 ft (0.89H)	0.38	0.99	8,185
Wall 31B	H ≤ 11	11 ft (1.0H)	0.30	0.30	8,599

Notes:
1. Height analyzed is measured from PGL to embedment depth.
2. Minimum Resistance factor is 0.65 for Static Bearing Capacity and 1.0 for Seismic Bearing Capacity.
3. Minimum resistance factor is 1.0 for sliding.

The maximum factored bearing stress is about 1.5 times the value obtained from ASD analysis. Therefore, for field inspection purposes, it should be recognized in plan sets and performance documents that this is a factored value.

The required embedment depths for MSE walls should be noted on the plans. These depths are presented in section C.4 of SCDOT's GDM for Mechanically Stabilized Walls. Additional embedment depths may be required based on table C-6 from the GDM as presented below. A horizontal bench with a minimum width of 4.0 feet shall extend from the toe of the wall before sloping to protect against local instability at the base of the wall.

Table 5.7 Minimum MSE Wall Embedment Depth Based on Local Bearing Capacity

Slope in Front of Wall	Minimum Embedment Depth
Horizontal (walls)	H/20
Horizontal (abutments)	H/10
3H:1V	H/10
2H:1V	H/7
1.5H:1V	H/5

Given the gradation of the proposed MSE reinforced backfill, which has a nominal size of about 3/4 inches, it is recommended that the vertical sides of MSE fill exposed to natural soil or soil backfill be covered with a non-woven geotextile that meets the SCDOT Specifications.

To enhance the life of metal reinforcement when roadways are located above an MSE wall, an impervious membrane that meets SCDOT specifications should be placed beneath the pavement aggregate base. A geotextile located at the top of the MSE stone is not required if an impervious membrane is used.

There are several locations along the roadway alignment where proposed drainage structures are situated in front of (i.e. parallel) MSE walls, or where new and existing draining structures pass beneath the MSE walls. Where new pipes are parallel to the proposed wall, the pipe should be installed prior to the proposed wall or the wall design should account for the temporary reduction in passive resistance. Where pipes pass beneath walls, the pipes should be designed to account for the increased loading associated with the wall backfill. We recommend the top of each pipe be situated a minimum of 1 foot below the bottom of retaining.

Settlement analyses have also been performed for the critical segments of the MSE walls. The settlement analysis is further discussed in section 5.9 of this report, and the results of those analyses are presented in Table 5.20. The settlement analysis was completed using the computer program FoSSa by Adama Engineering, Inc.

5.6 Soil Nail Wall Geotechnical Recommendations

This report addresses soil nail walls along the I-85/385 interchange. Soil Nail walls along within 150 feet of the bridge end bents will be discussed in their respective BGER. The exception is the soil nail wall adjacent to and beneath Woodruff Road. As the entire scope of that bridge modification incorporates the construction of a soil nail wall beneath and adjacent to the existing Woodruff Road bridge over I-385, the geotechnical recommendations for that soil nail wall are incorporated into this RGER.

Soil nail walls will be designed and installed by specialty geotechnical design/build contractors. The evaluation presented below is intended to demonstrate the minimum SCDOT global stability requirements can be achieved at the noted portion of the subject wall alignment. The specialty geotechnical design/build contractor must design the wall in accordance with SCDOT requirements including a final global stability analysis based on the contractor’s final nail layout and facing.

ECS selected a preliminary soil nail spacing of 5 foot horizontally and 4.25 foot vertically in this analysis. Our analysis considered an initial soil nail length up to 1.2 times the wall height for the preliminary global stability analysis with a 1.1 kip per foot pull out resistance per foot of nail penetration.

Soil Nail Wall design and construction must be in accordance with FHWA “Soil Nail Walls Reference Manual” Publication No. FHAW-NHI-147 (dated February 2015). A special provision for Soil Nail Wall construction is provided in Section 7 of this report. The SCDOT GDM does not provide specific resistance factors for soil nail wall design, but refers to FHWA guidance on these matters We recommend establishing resistance factors for soil nail wall design based on FHWA guidelines presented in Table 6.3 of the FHWA “Soil Nail Walls Reference Manual” Publication No. FHAW-NHI-14-7, and summarized below. We have modified Table 6.3 to provide similar formatting to the GDM Presentation.

Table 5.8 -- Resistance Factors for Soil Nail Walls

Performance Limit		Limit States		
		Strength	**Service**	**Extreme Event**
Overall Stability		0.65	N/A	0.90
Basal Heave	Short Term	0.65	N/A	N/A
	Long Term	0.50	N/A	N/A
Anchor Pull Out		N/A	0.65	0.65
Tensile Anchor Resistance(1)	Mild Steel (ASTM 615)	N/A	0.75	0.75
	High Strength Steel (ASTM A722)		0.65	0.65
Facing Resistance	Flexural Resistance	N/A	0.90	0.90
	Punching Shear		0.90	0.90
	Headed Stud – A307		0.70	0.65
	Headed Stud – A325		0.80	0.75
Lateral Sliding		N/A	0.90	1.00

5.7 Reinforced Soil Slope Recommendations

ECS completed an internal strength and slope stability analysis for the 1H:1V and 1.5H:1V slopes based on available subsurface information and geotextile reinforcement strengths. Based on our analysis, the slopes will require the use of uniaxial geogrids to maintain long-term stability. Our design is based on the use of uniaxial P1 (Long Term Design Strength (LTDS) = 405 lb/ft, P2 (LTDS = 720 lb/ft) and P3 (LTDS = 1305 lb/ft) geogrids as defined in the project special provisions..

The primary Geogrid reinforcement maintains the global and deep seated stability of the RSS. Additional measures are required to prevent surface sloughing at the slope face. For the slopes, we recommend the use of welded wire baskets to limit the potential for face instability.

Geogrids should be placed within the outer portion of the embankment. The primary grid reinforcement should maintain a vertical spacing of 3 feet (i.e. every third basket assuming 12-inch tall wire baskets). The lowest layer of Geogrid should be situation a minimum of 1.5 feet below the toe of slope. Tables 5.9 through 5.13 provide the recommended primary geogrid reinforcement elevations and reinforcement lengths.

Table 5.9 – Geogrid Reinforcement Recommendations I-85 Station 230+50 to 233+50 - P2 Geogrid	
Reinforcing Layer Elevation, ft	**Primary Reinforcing Layer Length, ft**
947.3	20
950.3	20
953.3	20
956.3	20
959.3	20
962.3	20
965.3	20

Table 5.10 – Geogrid Reinforcement Recommendations I-385 Station 337+50 to Station 342+53 and I-385 SBCD Station 139+00 to Station 140+10.55 P2 Geogrid	
Reinforcing Layer Elevation, ft	**Primary Reinforcing Layer Length, ft**
920.5	20
923.5	20
926.5	20
929.5	20
932.5	20
935.5	20

Table 5.11 – Geogrid Reinforcement Recommendations Ramp 1 Station 62+50 to 64+00 P1 Geogrid	
Reinforcing Layer Elevation ft	**Primary Reinforcing Layer Length, ft**
899	20
902	20
905	20
908	20
911	20
914	20
917	20
920	20
923	20
926	20

Table 5.12 – Geogrid Reinforcement Recommendations Ramp 5 Station 137+50 to 140+50 P3 Geogrid	
Reinforcing Layer Elevation Ft	**Primary Reinforcing Layer Length, ft**
938.7	25
941.7	25
944.7	25
947.7	25
950.7	25
953.7	25
956.7	25
959.7	25
962.7	25

Table 5.13 – Geogrid Reinforcement Recommendations Ramp 8A Station 12+00 to 13+00 P1 Geogrid	
Reinforcing Layer Elevation ft	**Primary Reinforcing Layer Length, ft**
959.3	10
962.3	10
965.3	10
968.3	10

Geogrids should be placed at the locations and elevations noted on the drawings. The lengths of the geogrids should match the lengths of the geogrids at specific elevations as noted on the drawings. The strong axis of uniaxial geogrids should be placed perpendicular to the slope face.

Geogrids should be installed in accordance with the manufacturer's instructions. All geogrids should be rolled out over relatively level compacted ground surfaces. The geogrids should be placed by unrolling perpendicular to the slope face. After unrolling, the geogrids should be tensioned by hand and secured in-place with staples, pins or stakes to maintain tension during backfilling to minimize wrinkles.

Backfill should be placed, spread and compacted in accordance with SCDOT specifications, the recommendations given in this report, and in such a manner that minimizes the development of wrinkles in and/or movement of the geogrid. To minimize geogrid wrinkles caused by the shoving action, materials should be pushed forward and spread gradually while lifting the blade. A minimum of 4 inches of material should exist between the geogrid and the tread of tracked equipment.

If design grades change from those noted in this report, ECS should be contacted for review and possible revision to the final construction documents. The use of wire baskets requires careful coordination to ensure the crest of the slope is at an elevation consistent with the basket height. If necessary, wire baskets can be "nested" to allow a variation in the slope crest elevation.
A global stability analysis of the critical geogrid reinforced soil was performed and the factor of safety exceeds the required minimum 1.33 or a Resistance Factor less than 0.75 (assuming no rigid face connection). The MSE Wall reinforced zone was modeled based on soil properties outlined in Appendix H of this report. This analysis assumed a reinforced soil plan as noted above. Additional information regarding the soil parameters and the results of the global stability analyses are located in section 5.8 of this report

5.8 Global Stability

The global stability of the proposed embankments, MSE walls, and cut slopes, and cut walls were evaluated with the computer program Slide 5.0 by Rocscience Inc, 439 University Ave Ste 780, Toronto, Ontario M5G 1Y88, e-mail: software@rocscience.com, website: www.rocscience.com.

The global stability analyses were conducted in the transverse direction for the service and extreme event I limit states. Slide considers numerous potential failure surfaces extending in front of, behind and through, MSE fill, RSS retained soil and foundation soils.

ECS completed the global stability analysis for critical areas of the MSE Walls, reinforced soil slopes, and the cut and fill embankment sections based on subsurface information obtained at the borings drilled in the vicinity of the respective locations. The MSE stability analysis was modeled with a vertical wall facing.

The slope stability computer program uses soil shear strength parameters under the Service limit state and the Extreme Event I limit state to estimate the factor of safety against slope instability accounting for force and moment equilibrium. The following sections summarize the soil profiles, shear strength parameters (Effective and Total) and geometry considered in the global stability analysis.

5.8.1 <u>Slope Stability Model Geometry and Critical Sections</u>

A total of forty-eight (48) cross sections were evaluated for global stability. The sections associated with Fill Embankments include:

- I-85 Sta. 373+00 Left, 2:1 Fill, approximate height of 9.5 feet
- I-85 Sta. 389+00 Left, 2:1 Fill, approximate height of 28.5 feet (for informational purposes only)
- I-385 NBCD Sta. 374+00 Right, 2:1 Fill, approximate height of 35 feet
- I-385 SBCD Sta. 102+50 Right, 2:1 Fill, approximate height of 43.5 feet
- I-385 Sta. 334+50 Left, 2:1 Fill, approximate height of 24.5 feet
- Ramp 1 Sta. 72+00 Right, 2:1 Fill, approximate height of 18.5 feet
- Ramp 1 Sta. 115+50, 4:1 Fill, approximate height of 6.5 feet
- Ramp 1B Sta. 77+00, 2:1 Fill, approximate height of 8 feet
- Ramp 2 Sta. 62+50, 4:1 Fill, approximate height of 9.5 feet
- Ramp 2A Sta. 71+00, 2:1 Fill, approximate height of 21.5 feet
- Ramp 3 Sta. 40+50, 2:1 Fill, approximate height of 9 feet
- Ramp 8/8A Sta. 21+00, 2:1 Fill, approximate height of 17.5 feet
- Ramp 9 Sta. 52+50 Right, 2:1 Fill, approximate height of 19 feet
- I-85 Sta. 231+00 Right, 1:1 Fill, approximate height of 17.5 feet
- I-385 Sta. 342+00 Left, 1:1 Fill, approximate height of 12.5 feet
- Ramp 1 Sta. 63+00 Right, 1.5:1 Fill, approximate height of 27 feet
- Ramp 5 Sta.139+00 Right, 1:1 Fill, approximate height of 27.5 feet
- Ramp 8A Sta. 12+50 Right, 1.5:1 Fill, approximate height of 8.5 feet

The sections associated with retaining walls at cut sections include:

- I-385 Sta. 439+00 Pile and Lagging, approximate height of 8.5 feet
- I-385 NBCD Sta. 340+50 Pile and Lagging, approximate height of 5.5 feet
- I-385 SBCD Sta. 118+00 Pile and Lagging, approximate height of 5 feet
- Ramp 1A Sta. 53+00 Pile and Lagging, approximate height of 4 feet
- Ramp 2A Sta. 107+50 Pile and Lagging, approximate height of 4.5 feet
- Ramp 2A Sta. 113+00 Pile and Lagging, approximate height of 6 feet
- I-385 NBCD Sta. 360+00 Soil Nail Wall, approximate height of 24.5 feet
- I-385 SBCD Sta. 121+00 Soil Nail Wall, approximate height of 22.5 feet
- I-85 Sta. 280+00 Concrete Retaining Wall, approximate height of 7.5 feet

The sections associated with slopes in cut sections include:

- I-85 Sta. 205+00, 1.8:1 slope, approximate depth of 36.5 feet

- I-85 Sta. 222+00, 1.7:1 slope, approximate depth of 17 feet
- I-385 NBCD Sta. 330+00, 2:1 slope, approximate depth of 22.5 feet
- Ramp 2, Sta. 67+50, 2:1 slope, approximate depth of 15 feet

The sections associated with MSE Walls include:

- I-85 Sta. 376+00, approximate height of 6 feet
- Walls 12 and 13 – Ramp 1A Sta. 70+50, Back-to-Back MSE walls, approximate height of 41.5 feet
- Wall 1 – Ramp 1B Sta. 76+00, approximate height of 21.5 feet
- Wall 33 – Ramp 2B Sta. 40+50, approximate height of 58 feet
- Wall 33 – Ramp 2B Sta. 44+00, approximate height of 36.5 feet
- Wall 10 – Ramp 3A Sta. 296+00, approximate height of 27 feet
- Wall 32 – Ramp 4 Sta. 40+00, approximate height of 39 feet
- Wall 32 – Ramp 4 Sta. 43+50, approximate height of 54 feet
- Wall 36 – Ramp 4B Sta. 388+00, approximate height of 23.5 feet
- Wall 2 – I-85 Sta. 281+00, approximate height of 17 feet
- Wall 31B – Ramp 4B Sta. 383+50, approximate height of 11 feet

The sections associated with barrier walls include:

- Wall 14 – Ramp 4 Station 64+50, approximate height of 7.5 feet.
- Wall 22 – I-385 Station 420+00, approximate height of 5.7 feet
- Wall 23 – I-385 Sta. 353+00, approximate height of 7.1 feet
- Wall 31 – I-385 NBCD Station 387+00, approximate height of 9.3 feet
- Wall 36A – Ramp 4 Station 386+00, approximate height of 14.5 feet

The global stability analysis was conducted in the transverse direction of the above critical sections. These locations were selected as the critical section because it represents the maximum heights for each MSE segment or slope inclination.

5.8.2 Soil Strength Parameters

Soil shear strength parameters were selected based on correlations provided in the SCDOT GDM, advanced laboratory testing, and our experience in the Piedmont geologic formation. Section 6.2 of this report further outlines the methods used to estimate shear strength parameters. Appendix H summarizes the soil strength parameters selected for this analysis of the roadway embankments, reinforced soil slopes, and the MSE walls.

Note that the Contractor has elected to construct the MSE walls with No. 57 stone which meets the backfill requirements indicated in the SCDOT Supplemental Technical Specification for Mechanically Stabilized Earth (MSE) Walls, which replaces Section 713 of the 2007 Standard Specifications for Highway Construction.

Reinforced soil slopes were modeled with embankment fill, actual fill materials shall meet the requirements Appendix K of the GDM including a minimum friction angle of 34 degrees and minimum total unit weight of 120 pcf.

5.8.3 Static (Service Limit) Slope Stability

The Service limit state was used to evaluate the static slope stability at the critical sections using the Bishop Simplified, Morgenstern-Price, and Spencer methods of analyzing slope stability. In accordance with Table 8-8 in the GDM and Sections 17.3.1 and 17.3.2, a uniform surcharge of 250 psf (Load Factor, γ=1.0) was used to simulate the live load surcharge (LS) for End-of-Construction and Long Term Loading conditions. In addition, for the Long Term loading condition a dead load surcharge of 140 psf was considered in accordance with Section 17.3.2 of the GDM to represent a 12 inch thick asphalt overlay.

A summary of the static (Service limit state) global slope stability analyses and the governing Demand/Capacity ratios (D/C) is provided in Tables 5.14 through 5.16. The Service limit state slope stability results indicate that the slope stability analysis using the provided soil shear strengths meets the design criteria and that ground modification and/or additional grid length will not be required.

Table 5.14 – Static Slope Stability Analysis (Fill Sections)

Bent Location (Ramp ID & Station No.)	Direction	Loading Condition	Demand/Capacity, D/C			Performance Criteria Met
			Morganstern - Price	Bishop	Spencer	
I-85 Station 373+00	Transverse	ESA	0.53	0.53	0.53	Yes
		TSA	0.53	0.53	0.53	Yes
I-85 Station 389+00	Transverse	ESA	0.67	0.67	0.67	Yes
		TSA	0.67	0.67	0.67	Yes
I-385 NBCD Station 374+00	Transverse	ESA	0.65	0.65	0.65	Yes
		TSA	0.65	0.65	0.65	Yes
I-385 SBCD Station 102+50	Transverse	ESA	0.70	0.70	0.70	Yes
		TSA	0.70	0.70	0.70	Yes
I-385 Station 334+50	Transverse	ESA	0.68	0.68	0.68	Yes
		TSA	0.66	0.65	0.66	Yes
Ramp 1 Station 72+00	Transverse	ESA	0.67	0.66	0.67	Yes
		TSA	0.64	0.64	0.64	Yes
Ramp 1 Station 115+50	Transverse	ESA	0.48	0.48	0.48	Yes
		TSA	0.38	0.38	0.38	Yes
Ramp 1B Station 77+00	Transverse	ESA	0.67	0.66	0.67	Yes
		TSA	0.63	0.63	0.63	Yes
Ramp 2 Station 62+50	Transverse	ESA	0.56	0.56	0.56	Yes
		TSA	0.44	0.44	0.44	Yes
Ramp 2A Station 71+00	Transverse	ESA	0.68	0.68	0.68	Yes
		TSA	0.68	0.68	0.68	Yes

Table 5.14 – Static Slope Stability Analysis (Fill Sections), con't

Bent Location (Ramp ID & Station No.)	Direction	Loading Condition	Demand/Capacity, D/C			Performance Criteria Met
			Morganstern - Price	Bishop	Spencer	
Ramp 3 Station 40+50	Transverse	ESA	0.65	0.65	0.65	Yes
		TSA	0.61	0.61	0.61	Yes
Ramp 8/8A Station 21+00	Transverse	ESA	0.68	0.68	0.68	Yes
		TSA	0.67	0.67	0.67	Yes
Ramp 9 Station 52+50	Transverse	ESA	0.69	0.68	0.68	Yes
		TSA	0.67	0.67	0.67	Yes
I-85 Station 231+00	Transverse	ESA	0.69	0.69	0.69	Yes
		TSA	0.68	0.68	0.68	Yes
I-385 Station 342+00	Transverse	ESA	0.71	0.71	0.71	Yes
		TSA	0.65	0.65	0.65	Yes
Ramp 1 Station 63+00	Transverse	ESA	0.72	0.72	0.72	Yes
		TSA	0.71	0.71	0.71	Yes
Ramp 5 Station 139+00	Transverse	ESA	0.73	0.72	0.74	Yes
		TSA	0.71	0.71	0.71	Yes
Ramp 8A Station 12+50	Transverse	ESA	0.66	0.66	0.66	Yes
		TSA	0.63	0.63	0.63	Yes

Table 5.15 – Static Slope Stability Analysis (Cut Sections)

Bent Location (Ramp ID & Station No.)	Direction	Loading Condition	Demand/Capacity, D/C			Performance Criteria Met
			Morganstern - Price	Bishop	Spencer	
I-385 Station 439+00	Transverse	ESA	0.60	0.60	0.60	Yes
		TSA	0.60	0.60	0.60	Yes
I-385 NBCD Station 340+50	Transverse	ESA	0.47	0.47	0.47	Yes
		TSA	0.17	0.17	0.17	Yes
I-385 SBCD Station 118+00	Transverse	ESA	0.66	0.66	0.66	Yes
		TSA	0.66	0.66	0.66	Yes
Ramp 1A Station 53+00	Transverse	ESA	0.47	0.47	0.47	Yes
		TSA	0.50	0.50	0.48	Yes

Table 5.15 – Static Slope Stability Analysis (Cut Sections), con't

Bent Location (Ramp ID & Station No.)	Direction	Loading Condition	Demand/Capacity, D/C			Performance Criteria Met
			Morganstern - Price	Bishop	Spencer	
Ramp 2A Station 107+50	Transverse	ESA	0.69	0.69	0.69	Yes
		TSA	0.10	0.10	0.10	Yes
Ramp 2A Station 113+00	Transverse	ESA	0.69	0.69	0.69	Yes
		TSA	0.69	0.69	0.69	Yes
I-385 NBCD Station 360+00	Transverse	ESA	0.64	0.65	0.63	Yes
		TSA	0.58	0.58	0.58	Yes
I-385 SBCD Station 121+00	Transverse	ESA	0.65	0.65	0.65	Yes
		TSA	0.63	0.63	0.63	Yes
I-85 Station 205+00	Transverse	ESA	0.40	0.40	0.40	Yes
		TSA	0.39	0.39	0.39	Yes
I-85 Station 222+00	Transverse	ESA	0.70	0.70	0.70	Yes
		TSA	0.70	0.70	0.70	Yes
I-385 Station 330+00	Transverse	ESA	0.68	0.68	0.68	Yes
		TSA	0.68	0.68	0.68	Yes
Ramp 2 Station 62+50	Transverse	ESA	0.63	0.63	0.63	Yes
		TSA	0.29	0.30	0.29	Yes
Wall 14 - Ramp 4 64+50	Transverse	ESA	0.65	0.65	0.65	Yes
		TSA	0.60	0.60	0.60	Yes

Table 5.16 – Static Slope Stability Analysis (MSE Walls)

Bent Location (Ramp ID & Station No.)	Direction	Loading Condition	Demand/Capacity, D/C			Performance Criteria Met
			Morganstern - Price	Bishop	Spencer	
I-85 Station 376+00	Transverse	ESA	0.72	0.73	0.72	Yes
		TSA	0.56	0.58	0.57	Yes
Wall 12 Ramp 1A Station 70+50	Transverse	ESA	0.60	0.60	0.60	Yes
		TSA	0.59	0.59	0.59	Yes

Table 5.16 – Static Slope Stability Analysis (MSE Walls), con't

Bent Location (Ramp ID & Station No.)	Direction	Loading Condition	Demand/Capacity, D/C			Performance Criteria Met
			Morganstern - Price	Bishop	Spencer	
Wall 13 Ramp 1A Station 70+50	Transverse	ESA	0.65	0.65	0.65	Yes
		TSA	0.63	0.64	0.63	Yes
Wall 1 Ramp 1B Station 76+00	Transverse	ESA	0.71	0.72	0.72	Yes
		TSA	0.68	0.69	0.69	Yes
Wall 33 Ramp 2B Station 40+50	Transverse	ESA	0.64	0.64	0.65	Yes
		TSA	0.63	0.63	0.63	Yes
Wall 33 Ramp 2B Station 44+00	Transverse	ESA	0.71	0.71	0.71	Yes
		TSA	0.71	0.71	0.71	Yes
Wall 10 Ramp 3A Station 296+00	Transverse	ESA	0.75	0.74	0.75	Yes
		TSA	0.58	0.54	0.57	Yes
Wall 32 Ramp 4 Station 40+00	Transverse	ESA	0.65	0.65	0.65	Yes
		TSA	0.64	0.64	0.64	Yes
Wall 32 Ramp 4 Station 43+50	Transverse	ESA	0.64	0.64	0.64	Yes
		TSA	0.64	0.63	0.63	Yes
Wall 32 and 36 Ramp 4B Station 388+00	Transverse	ESA	0.61	0.60	0.61	Yes
		TSA	0.58	0.58	0.59	Yes
Wall 36 Ramp 4B Station 388+00	Transverse	ESA	0.57	0.57	0.57	Yes
		TSA	0.59	0.58	0.59	Yes
Wall 32 Ramp 4B Station 388+00	Transverse	ESA	0.65	0.64	0.65	Yes
		TSA	0.45	0.42	0.44	Yes
Wall 2 I-85 Station 281+00	Transverse	ESA	0.59	0.59	0.59	Yes
		TSA	0.56	0.57	0.57	Yes
Wall 31B Ramp 4B Station 383+50	Transverse	ESA	0.58	0.58	0.58	Yes
		TSA	0.55	0.55	0.55	Yes

*Note: Stability analysis run on section not including headwall of culvert.
1. Effective Stress Analysis and Total Stress Analysis contained the same soil parameters.

Table 5.17 – Static Slope Stability – Barrier Walls

Bent Location (Ramp ID and Station No.)	Direction	Loading Condition	Demand/Capacity, D/C			Performance Criteria Met
			Morgenstern -Price	Bishop	Spencer	
Wall 14 Ramp 4 Station 64+50	Transverse	ESA	0.65	0.65	0.65	Yes
		TSA	0.60	0.60	0.60	Yes
Wall 22 I-385 Station 420+00	Transverse	ESA	0.66	0.66	0.67	Yes
		TSA	0.29	0.27	0.28	Yes
Wall 23 I-385 Station 353+00	Transverse	ESA	0.70	0.70	0.70	Yes
		TSA	0.65	0.65	0.66	Yes
Wall 31A I-385 Station 387+00	Transverse	ESA	0.75	0.74	0.75	Yes
		TSA	0.71	0.70	0.71	Yes
Wall 36A Ramp 4B Station 386+00	Transverse	ESA	0.74	0.73	0.74	Yes
		TSA	0.70	0.70	0.70	Yes

5.8.4 Extreme Limit State (Seismic) Global Stability

Section 13.15 of the GDM recommends accounting for Wave Scattering in accordance with Section 13.16 of the GDM when evaluating seismic slope stability of embankments greater than 20 feet in height. We considered wave scattering for a PGA=0.20, Sds=0.14, wall height of 6 feet to 54 feet resulting in pseudo-static horizontal accelerations (kh) ranging from 0.194 to 0.130.

As discussed in section 4 of this report, ECS evaluated the potential for liquefaction and SSL triggering events in accordance with Section 13.6 of the GDM for both the FEE and SEE seismic events. The analysis indicated that SSL and Liquefaction will not occur and the minimum Seismic D/C of 1.00 is achieved in all cases. Since the analysis demonstrated SSL and Liquefaction will not occur, the global stability analysis considered fully mobilized shear strengths (i.e. no shear strength loss).

The Bishop, Morganstern-Price, and Spencer slope stability method was used to evaluate the Demand/Capacity ratio (D/C), reinforced slope and retaining wall performance for structures with ROC=I, II or III. The analysis considered the surcharge loads presented in Section 5.8.3 of this report with a load factor load factor, γ=0.5 in accordance with Section 8.7 of the GDM.

Seismic loading was evaluated first for the Safety Evaluation Earthquake (SEE). Since minimum stability requirements were achieved in the SEE event, the walls and slopes are considered stable by inspection during FEE event. A summary of the seismic global slope stability analyses and the governing Demand/Capacity ratios (D/C) is provided in Tables 5.18 through 5.20. The seismic analysis was limited to sections evaluated for static global stability and with an ROC-I or ROC=II.

Table 5.18 – Extreme Limit State Global Stability Analysis (Fill Sections)

Bent Location (Ramp ID & Station No.)	Direction	k_h	Demand/Capacity, D/C			Performance Criteria Met
			Morganstern - Price	Bishop	Spencer	
I-85 Station 231+00	Transverse	0.20	0.88	0.88	0.88	Yes
I-385 Station 342+00	Transverse	0.20	0.83	0.83	0.83	Yes
Ramp 1 Station 63+00	Transverse	0.165	0.95	0.95	0.95	Yes
Ramp 5 Station 139+00	Transverse	0.165	0.93	0.93	0.93	Yes
Ramp 8A Station 12+50	Transverse	0.20	0.86	0.86	0.86	Yes

Table 5.19 – Extreme Limit State Global Stability Analysis (Cut Sections)

Bent Location (Ramp ID & Station No.)	Direction	k_h	Demand/Capacity, D/C			Performance Criteria Met
			Morganstern - Price	Bishop	Spencer	
I-385 Station 439+00	Transverse	0.20	0.77	0.78	0.76	Yes
I-385 NBCD Station 340+50	Transverse	0.20	0.41	0.41	0.41	Yes
I-385 SBCD Station 118+00	Transverse	0.20	0.81	0.81	0.80	Yes
Ramp 1A Station 53+00	Transverse	0.20	0.67	0.67	0.64	Yes

Table 5.19 – Extreme Limit State Global Stability Analysis (Cut Sections), con't

Bent Location (Ramp ID & Station No.)	**Direction**	**k_h**	**Demand/Capacity, D/C**			**Performance Criteria Met**
			Morganstern - Price	**Bishop**	**Spencer**	
Ramp 2A Station 107+50	Transverse	0.20	0.27	0.27	0.27	Yes
Ramp 2A Station 113+00	Transverse	0.20	0.83	0.84	0.83	Yes
I-385 NBCD Station 360+00	Transverse	0.168	0.66	0.67	0.65	Yes
I-385 SBCD Station 121+00	Transverse	0.171	0.73	0.72	0.72	Yes

Table 5.20 – Extreme Limit State Global Stability Analysis (MSE Walls)

Bent Location (Ramp ID & Station No.)	**Direction**	**k_h**	**Demand/Capacity, D/C**			**Performance Criteria Met**
			Morganstern - Price	**Bishop**	**Spencer**	
I-85 Station 376+00*	Transverse	0.20	0.74	0.76	0.76	Yes
Wall 12 Ramp 1A Station 70+50	Transverse	0.155	0.75	0.74	0.74	Yes
Wall 13 Ramp 1A Station 70+50	Transverse	0.155	0.81	0.82	0.81	Yes
Wall 1 Ramp 1B Station 76+00	Transverse	0.182	0.88	0.92	0.89	Yes
Wall 33 Ramp 2B Station 40+50	Transverse	0.130	0.79	0.76	0.79	Yes
Wall 33 Ramp 2B Station 44+00	Transverse	0.153	0.93	0.93	0.93	Yes
Wall 10 Ramp 3A Station 296+00	Transverse	0.167	0.81	0.69	0.78	Yes
Wall 32 Ramp 4 Station 40+00	Transverse	0.149	0.84	0.85	0.84	Yes

Table 5.20 – Extreme Limit State Global Stability Analysis (MSE Walls), con't

Bent Location (Ramp ID & Station No.)	Direction	k_h	Demand/Capacity, D/C			Performance Criteria Met
			Morganstern – Price	Bishop	Spencer	
Wall 32 Ramp 4 Station 43+50	Transverse	0.143	0.82	0.82	0.82	Yes
Wall 32 and 36 Ramp 4B Station 388+00	Transverse	0.147	0.83	0.82	0.83	Yes
Wall 36 Ramp 4B Station 388+00	Transverse	0.171	0.78	0.79	0.78	Yes
Wall 32 Ramp 4B Station 388+00	Transverse	0.176	0.65	0.58	0.63	Yes
Wall 2 I-85 Station 281+00	Transverse	0.20	0.76	0.77	0.76	Yes
Wall 31B Ramp 4B Station 383+50	Transverse	0.20	0.72	0.73	0.72	Yes

*Note: Stability analysis run on section not including headwall of culvert.

5.9 Settlement Considerations

5.9.1 *Elastic Settlement*

Elastic settlements resulting from the embankment and MSE wall construction being placed over the unsaturated cohesive soils and cohesionless soils were estimated using the computer program FoSSA by Adama Engineering, Inc. The performance criteria outlined in Chapter 10 of the GDM was referenced to establish acceptable limits for static settlement under the Service Limit State. We evaluated the embankment and MSE Wall settlement at each embankment fill or fill MSE wall cross section evaluated as part of the global stability evaluation discussed in Section 5.8 of this report. In addition, we evaluated settlement at each cross line pipe location where more than 5 feet of fill is required. Note that the cross line pipe on Ramp 1 Sta. 72+50 is similar to the Embankment cross section at 72+00 as such the cross line pipe settlement evaluation is assumed to represent the settlement of the embankment cross section at Sta 72+00.

Refer to Table 5.21 for a summary of the MSE Wall settlement analysis, Table 5.22 for a summary of embankment settlement analysis, and Table 5.23 for summary of cross pipe analysis.

Based on local experience and the available boring and laboratory data, ECS anticipates that surcharging will not be necessary during construction. Experience in the Piedmont indicates that the total settlement will occur within 2 to 6 weeks following completion of all fill placement at a given position. Appendix M contains our settlement analysis calculations.

Table 5.21 – Summary of Settlement Analysis for MSE Walls

MSE Wall			**Table 10-37 (Service Limit State)**				
Number	**Alignment**	**Station**					
Performance Limit			**RV-01**	**RV-02***	**RV-03**	**RV-04**	**0.150 L_{Reinf}**
Tolerance			18.00 in.	0.20 in./yr.	4.00 in./50 ft.	0.150 L_{Reinf}	
Bridge 4	I-385 NBCD	368+00	<0.1 in.	<0.20 in./yr.	<0.1 in**	<0.1	1.4
12 an 13	Ramp 1A	70+50	3.0 in.	<0.20 in./yr.	<3 in**	1.1	3.9
1	Ramp 1B	76+00	<0.1 in.	<0.20 in./yr.	<0.1 in**	<0.1	1.2
33	Ramp 2B	40+50	3.6 in.	<0.20 in./yr.	<3.6 in**	5.6	6.8
33	Ramp 2B	44+00	1.6 in.	<0.20 in./yr.	<1.6 in**	1.8	5.5
10	Ramp 3A	296+00	3.1 in.	<0.20 in./yr.	<3.1 in**	0.8	2.7
32	Ramp 4	40+00	2.2 in.	<0.20 in./yr.	<2.2 in**	1.6	6.5
32	Ramp 4	43+50	2.9 in.	<0.20 in./yr.	<2.9 in**	1.7	5.1
36	Ramp 4B	388+00	11.0 in.	<0.20 in./yr.	2.5 in./50 ft.	1.8	1.9

RV-01: Maximum Vertical Settlement at the top of wall profile grade over the design life of the embankment or wall.
RV-02: Maximum settlement rate per year after the wall has been constructed.
RV-03: Maximum vertical differential settlement observed longitudinally along the top of wall profile grade after the wall has been constructed.
RV-04: Maximum vertical differential settlement observed perpendicular to the top of wall profile after the wall has been constructed.
*Rate to be confirmed through settlement monitoring after fill placement.
**By inspection differential settlement is less than total settlement.

Table 5.22 Summary of Settlement Analysis for Embankments

Embankment		**Embankments Table 10-24 (Service Limit State)**		
Alignment	**Station**			
Performance Limit		**EV-01**	**EV-02**	**EV-03**
Tolerance		16 in.	0.20 in./yr.	2 in./50 ft.
I-85	373+00	0.2	<0.2 in/yr*	<2"**
I-85	389+00	0.0	<0.2 in/yr*	<2"**
I-385	334+50	2.4	<0.2 in/yr*	<2"**
I-385 NBCD	374+00	3.2	<0.2 in/yr*	<2"**
I-385 SBCD	102+50	2.0	<0.2 in/yr*	<2"**
Ramp 1	63+00	0.1	<0.2 in/yr*	<2"**
Ramp 1	115+50	1.3	<0.2 in/yr*	<2"**
Ramp 1B	77+00	0.1	<0.2 in/yr*	<2"**
Ramp 2	62+50	5.8	<0.2 in/yr*	<2"**
Ramp 2A	71+00	9.8	<0.2 in/yr*	<2"**
Ramp 3	40+50	2.2	<0.2 in/yr*	<2"**
Ramp 8	21+00	0.2	<0.2 in/yr*	<2"**
Ramp 9	52+50	0.5	<0.2 in/yr*	<2"**

EV-01: Maximum vertical settlement along the profile grade over the design life of the embankment.

EV-02: Maximum settlement rate per year after the roadway has been paved.
EV-03: Maximum vertical differential settlement occurring longitudinally along the profile grade after the roadway has been paved.
*Rate to be confirmed through settlement monitoring after fill placement.
**Secondary consolidation is not anticipated based on soil type, primary settlement will be complete prior to paving. As such long term settlement and differential settlement estimates are not anticipated.

Table 5.23 – Summary of Settlement Analysis for Cross Pipes

Structure	Alignment	Station	Max Fill Depth (ft.)	Settlement		
				End 1 (in.)	Middle (in.)	End 2 (in.)
Existing Pipe	I-85	200+00	0 +/-	N/A	N/A	N/A
NP-117	I-85	203+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-85	207+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-85	210+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-85	216+00	0 +/-	N/A	N/A	N/A
NP-305/ NP-306	I-85	223+00	6 +/-	<0.1	<0.1	<0.1
Existing Pipe	I-85	229+00	0 +/-	N/A	N/A	N/A
Existing Pipe*	I-85	231+00	11 +/-	1.3	0.0	0.0
NP-403/ NP-404	I-85	234+00	6 +/-	0.0	1.0	0.4
NP-504/ NP-505	I-85	241+00	0 +/-	N/A	N/A	N/A
NP-605	I-85	249+00	0 +/-	N/A	N/A	N/A
NP-702/ NP-703	I-85	250+00	0 +/-	N/A	N/A	N/A
NP-801	I-85	255+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-85	263+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-85	271+00	0 +/-	N/A	N/A	N/A
NP-7203	I-85	278+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-85/ Ramp 3	281+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-85/ Ramp 1B	284+00	0 +/-	N/A	N/A	N/A
Existing Pipe*	I-85/ Ramp3	284+00	5 +/-	1.2	2.5	1.0
NP-7225	I-85/ Ramp 1B	286+00	0 +/-	N/A	N/A	N/A
NP-7237	I-85	291+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-85	296+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-85/ Ramp 1/ Ramp 1B	297+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-85/ Ramp 1	303+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-85	311+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-85	318+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-85	320+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-85	322+00	0 +/-	N/A	N/A	N/A
Existing Pipe*	I-85	327+00	3 +/-	0.2	0.5	0.2
Existing Pipe	I-85	329+00	15 +/-	1.3	0.0	1.2
Existing Pipe*	I-85	332+00	4 +/-	1.2	2.5	1.0
Existing Pipe	I-85	337+00	0 +/-	N/A	N/A	N/A
Existing Pipe*	I-85/ Ramp 1/ Ramp 2A	342+00	6 +/-	0.2	0.8	0.2
Existing Pipe	I-85	350+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-85	357+00	5 +/-	0.4	0.1	0.0

Table 5.23 – Summary of Settlement Analysis for Cross Pipes (con't)

Structure	Alignment	Station	Max Fill Depth (ft.)	Settlement		
				End 1 (in.)	Middle (in.)	End 2 (in.)
Existing Pipe	I-85	362+00	5 +/-	<0.1	<0.1	<0.1
NP-W20101/ NP-W200100/ NP-W20105	I-85	367+00	5 +/-	0.1	0.5	0.0
NP-W20200	I-85	371+00	2 +/-	<0.1	<0.1	<0.1
Existing Pipe*	I-85	376+00	4 +/-	1.2	2.5	1.0
Existing Pipe*	I-85	381+50	6 +/-	0.2	0.8	0.2
Existing Pipe	I-85	386+00	6 +/-	NC	NC	NC
Existing Pipe	I-85	387+00	7 +/-	NC	NC	NC
Existing Pipe*	I-85	388+00	8 +/-	0.5	2.4	0.6
Existing Pipe*	I-85	397+00	5 +/-	1.2	2.5	1.0
Existing Pipe*	I-85	401+00	2 +/-	0.4	0.6	0.4
Existing Pipe	I-85	406+00	0 +/-	N/A	N/A	N/A
Existing Pipe*	I-85	413+00	5 +/-	1.2	2.5	1.0
Existing Pipe*	I-85	418+00	3 +/-	0.2	0.5	0.2
Existing Pipe	I-85	422+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-85	427+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-85	428+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-85	429+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-85	431+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-85	433+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-385	308+00	0 +/-	N/A	N/A	N/A
Existing Pipe*	I-385	332+00	5 +/-	1.2	2.5	1.0
NP-1215	I-385	334+00	Jack and Bore			
Existing Pipe	I-385	335+00	8 +/-	0.5	2.4	0.6
NP-126S*	I-385	339+00	5 +/-	1.2	2.5	1.0
NP-133S*	I-385	341+00	5 +/-	1.2	2.5	1.0
Existing Pipe*	I-385/ Ramp 11	344+00	3 +/-	0.2	0.5	0.2
NP-38S*	I-385	345+00	3 +/-	0.2	0.5	0.2
NP-36S/NP-35S*	I-385	346+00	3 +/-	0.2	0.5	0.2
Existing Pipe*	I-385/ Ramp 10	346+00	2 +/-	0.4	0.6	0.0
NP-24S*	I-385	351+00	3 +/-	0.2	0.5	0.2
Existing Pipe*	I-385	362+00	3 +/-	0.2	0.5	0.2
Existing Pipe*	I-385	375+00	8 +/-	0.5	2.4	0.6
Existing Pipe	I-385/ Ramp 4	381+00	36 +/-	0.4	1.5	3.4
NP-87S/NP-138S/NP-137S*	I-385/ Ramp 4	382+00	46 +/-	0.5	9.5	4.2
NP-94S/NP-95S*	I-385	386+00	5 +/-	1.2	2.5	1.0
Existing Pipe*	I-385	406+00	7 +/-	0.5	2.4	0.6
Existing Pipe*	I-385	410+00	4 +/-	1.2	2.5	1.0
NP-5615N/ NP-CECS-5704N/ NP-5613N*	I-385/ Ramp 2B/ Ramp 3A	411+00	2 +/-	0.4	0.6	0.4

Table 5.23 – Summary of Settlement Analysis for Cross Pipes (con't)

Structure	Alignment	Station	Max Fill Depth (ft.)	Settlement		
				End 1 (in.)	Middle (in.)	End 2 (in.)
Existing Pipe	I-385/ Ramp 2A/ Ramp 3	424+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-385	431+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-385	435+00	0 +/-	N/A	N/A	N/A
NP-97S*	I-385 SBCD	98+00	3 +/-	0.2	0.5	0.2
NP-65S/ NP-72S*	I-385 SBCD/ Ramp 9	116+50	3 +/-	0.2	0.5	0.2
Existing Pipe	I-385 SBCD	121+00	4 +/-			
NP-70S	I-385 SBCD	122+00	0 +/-	N/A	N/A	N/A
NP-58S	I-385 SBCD	128+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-385 SBCD/ Ramp 10	131+00	0 +/-	N/A	N/A	N/A
NP-500*	I-385 NBCD/ Ramp 11	348+00	2 +/-	0.4	0.6	0.0
NP-20S	I-385 NBCD	358+00	0 +/-	N/A	N/A	N/A
NP-17S	I-385 NBCD	360+00	0 +/-	N/A	N/A	N/A
NP-50S	I-385 NBCD	361+00	0 +/-	N/A	N/A	N/A
Existing Pipe	I-385 NBCD	363+00	0 +/-	N/A	N/A	N/A
NP-43S	I-385 NBCD	366+00	0 +/-	N/A	N/A	N/A
NP-41S	I-385 NBCD	368+00	0 +/-	N/A	N/A	N/A
NP-40S*	I-385 NBCD/ Ramp 8A	370+25	10 +/-	1.3	1.5	1.2
NP-STN-125*	Ramp 1	74+00	5 +/-	1.2	2.5	1.0
NP-5813	Ramp 1	82+00	0 +/-	N/A	N/A	N/A
Existing Pipe	Ramp 1	100+00	0 +/-	N/A	N/A	N/A
NP-STN-118	Ramp 1/ Ramp 1B	105+00	0 +/-	N/A	N/A	N/A
NP-STN-14	Ramp 1	108+00	0 +/-	N/A	N/A	N/A
NP-STN-5812	Ramp 1A	64+00	5 +/-	1.2	2.5	1.0
Existing Pipe	Ramp 1A/ Ramp 2B	89+00	54 +/-	0.0	4.7	2.0
NP-106S	Ramp 1A/ Ramp 2B	90+00	50 +/-	0.5	10.5	4.2
NP-STN-12*	Ramp 1B	55+75	4 +/-	1.2	2.5	1.0
NP-STN-7250*	Ramp 1B	73+00	16 +/-	4.7	12.6	10.3
Existing Pipe	Ramp 1B	76+00	11 +/-	<0.1	<0.1	<0.1
NP-6101/NP-6102	Ramp 2/ Ramp 2A	59+00	5 +/-	0.3	1.2	0.3
NP-6001N1*	Ramp 2/ Ramp 2A	61+00	8 +/-	0.5	2.4	0.6
NP-6201*	Ramp 2/ Ramp 4B	65+00	6 +/-	0.2	0.8	0.2
NP-STN-111	Ramp 2A	81+25	0 +/-	0.0	0.0	0.0
NP-STN-115*	Ramp 2A	84+00	3 +/-	1.2	2.5	1.0
Existing Pipe	Ramp 2A/ Ramp 4	94+00	0 +/-	N/A	N/A	N/A
NP-6309	Ramp 2A	115+00	5 +/-	1.2	2.5	1.0
Existing Pipe*	Ramp 2A	126+00	6 +/-	0.2	0.8	0.2
NP-7236	Ramp 2B	29+25	18 +/-	0.2	8.2	0.0
Existing Pipe	Ramp 2B	32+00	40 +/-	0.4	18	0
NP-723	Ramp 2B	33+00	19 +/-	0.3	18.5	0
NP-103S	Ramp 2B	44+25	37 +/-	2.6	2.9	1.6

Table 5.23 – Summary of Settlement Analysis for Cross Pipes (con't)

Structure	Alignment	Station	Max Fill Depth (ft.)	Settlement		
				End 1 (in.)	Middle (in.)	End 2 (in.)
NP-108S/NP-108S*	Ramp 3	39+00	9 +/-	1.3	1.5	1.2
NP-99S*	Ramp 3	44+00	8 +/-	0.5	2.4	0.6
Existing Pipe*	Ramp 3A	316+75	2 +/-	0.4	0.1	0.0
NP-90S*	Ramp 4	47+25	30 +/-	0.3	8.2	0.3
NP-150S*	Ramp 4	51+00	18 +/-	0.3	8.2	0.3
Existing Pipe*	Ramp 4/ Ramp 4B	55+00	38 +/-	0.4	18	0
NP-STN-109	Ramp 4	61+00	0 +/-	N/A	N/A	N/A
NP-7246	Ramp 4B	420+00	18 +/-	0.0	0.2	1.1
Existing Pipe	Ramp 5	105+00	0 +/-	0.0	0.0	0.0
Existing Pipe	Ramp 5	110+00	0 +/-	0.0	0.0	0.0
NP-804*	Ramp 5	116+00	3 +/-	0.2	0.5	0.2
NP-1008*	Ramp 7	26+00	3 +/-	0.2	0.5	0.2
NP-52S	Ramp 8A	50+00	0 +/-	N/A	N/A	N/A
Existing Pipe*	Pelham Ramp	53+00	3 +/-	0.1	0.5	0.0

N/A – Not applicable. Cross line pipe situated in an area of minimal fill resulting in negligible settlement.
NC – Not calculated, pending borings
*Settlement estimated based on cross section with similar subsurface conditions and embankment geometry.

5.9.2 *Consolidation Settlement*

The foundation soils at the along the embankments and walls generally consisted of residual silts and sands. Based on the granular nature of the soils, the elastic settlement estimates are assumed to represent the total at each analysis. Assuming fill is compacted to the densities required by the SCDOT Standard Specifications.

5.9.3 *Settlement Monitoring*

We recommend incorporating a settlement monitoring program to determine when settlement is substantially complete along MSE Walls prior to casting the concrete coping. We recommend establishing a settlement monument every 150 feet along the top of MSE walls. Settlement monitoring is ~~not~~ recommended every 500 feet for roadway embankments beyond the MSE Walls. Monitoring of the MSE wall may consist of establishing a control point on the top of the pre-cast concrete panel.

5.10 Pipe Culverts

Culverts should be constructed in accordance with SCDOT Standard Specifications for Highway Construction and with SCDOT Standard Drawings for Pipe Culverts. The contractor should have appropriate equipment and backups on hand at all times to effectively dewater excavations, should it be needed.

SCDOT Standard Drawings 714-020-00 and 714-120-00 requires various levels of undercut are required based on the SPT N-values within 5 feet below the pipe invert, and whether or not reinforcing geogrid will be used as part of the pipe bedding foundation. In general, undercuts will range from no undercut needed to 20 inches or more. If no undercut is required (for pipe foundation soils having SPT N-values of 15 bpf or greater), then bedding placement and pipe instulation should follow SCDOT Standard Drawings 714-005-00 and 714-020-00. For areas having SPT N-values less than 15 bpf within 5 feet below the pipe invert, then varying thicknesses of undercut will be required, depending on SPT N-values, pipe diameter, and whether or not geogrid is used as part of the pipe culvert foundation.

Unsuitable soils, including loose, soft, yielding, highly plastic or excessively wet soils should be removed from the foundation area and replaced with materials meeting the bedding requirements. The width and depth of undercutting should meet the minimum requirements as noted in Standard Drawing 714-005-00.

The replacement materials and pipe bedding should meet the requirements of the SCDOT Standard Specifications. When there is standing or running water in the trench, part of the bedding may consist of No. 57 stone provided the stone is surrounded on all sides by a non-woven geotextile fabric meeting the requirements of the SCDOT Standard Specifications.

For pipes bearing foundation soils having an SPT N-value greater than 15 bpf, the minimum bedding thickness should be placed in accordance with Standard Drawing 714-005-00. For pipes constructed in trenches, the trench width should be the greatest of 1½ times the pipe O.D. plus 12 inches, 1.0 times the pipe O.D. plus 24 inches, or as required to safely fit personnel and compaction equipment. The trench width should generally not exceed 3 times the pipe O.D. unless personnel safety and proper equipment operation are compromised. The limits of the required trench, bedding, and backfill should meet the requirements of the Standard Drawings for Pipe Culverts.

Table 5.24 below summarizes conditions along the main interchange, where noted soft foundation soil conditions exist, and which will require undercutting per at each pipe culvert location. Note that this analysis is not completed and will be provided with the revised report. The table is provided for information purposes only.

Table 5.24 Pipe Culvert Undercut Requirements

Pipe Location			Pipe Diameter (inch)	Closest Boring	Elevation 5 feet Below Lowest Invert (ft)	Lowest SPT N-Value within 5 feet Below Invert Elevation (bpf)	Required Depth of Undercut Below Bedding (2) (inch)	
Structure	Alignment	STA					Option 1 (no Geogrid)	Option 2 (with Geogrid)
NP-54S	I385SBCD	135+17.81	48	BX-385-01	928.30	2	27	12
NP-1004	I85NBCD	264+00.00	18	R85-02	1002.20	7	16	3
NP-1005	I85NBCD	265+00.00	24	R85-02	1000.68	7	16	3
NP-1006	I85NBCD	266+00.00	24	R85-63	1000.19	5	16	3
NP-1007	I85NBCD	267+73.00	24	R85-64	995.67	6	16	3
NP-1008	RAMP_7	25+34.00	24	R7-03A	991.67	8	16	3
NP-1009	I85NBCD	266+35.00	24	R85-63	996.67	5	16	3
NP-100S	RAMP_4	52+50.09	18	W4-1R-04	976.52	11	8	0
NP-1010	I85NBCD	270+00.00	24	W85-1L-02	994.08	100+	0	0

Table 5.24 Pipe Culvert Undercut Requirements (con't)

Pipe Location			Pipe Diameter (inch)	Closest Boring	Elevation 5 feet Below Lowest Invert (ft)	Lowest SPT N-Value within 5 feet Below Invert Elevation (bpf)	Required Depth of Undercut Below Bedding (2) (inch)	
Structure	Alignment	STA					Option 1 (no Geogrid)	Option 2 (with Geogrid)
NP-1012	I85	263+90.27	18	R85-02	1003.52	7	16	3
NP-1013	RAMP_7	26+30.00	24	R7-03A	991.21	8	16	3
NP-1014	RAMP_7	28+76.00	30	R7-03A	989.94	7	16	3
NP-102S	RAMP_1A	93+05.63	18	W2B-1R-03	1011.42	8	16	3
NP-103S	RAMP_2B	44+31.30	18	B-32	1009.80	18	0	0
NP-104S	RAMP_2B	46+00.00	18	B-33	1003.50	14	8	0
NP-105S	RAMP_2B	47+50.00	18	W2B-1R-04	995.54	12	8	0
NP-106S	RAMP_2B	41+93.00	24	W2B-1R-03	972.36	15	8	0
NP-107S	RAMP_1A	90+00.00	24	W2B-1R-01	988.77	4	20	6
NP-108S	RAMP_1A	89+47.18	18	BX-3-01	995.53	14	8	0
NP-109S	RAMP_3	38+50.00	18	B-21	996.62	7	16	3
NP-110S	RAMP_3	39+43.98	18	BX-3-01	996.01	14	8	0
NP-117	I85	203+41.41	18	I85-100	933.34	100+	0	0
NP-119S	I385NBCD	372+84.76	18	B-67	980.96	13	8	0
NP-11S	I385	347+50.00	30	BX-385-01	930.76	2	27	12
NP-120S	I385NBCD	333+80.73	30	R385-26	921.20	8	16	3
NP-121S	I385	333+80.18	30	R385-82	913.78	4	20	6
NP-122S	I385	335+00.44	36	R385-82	912.31	4	20	6
NP-123S	I385	334+95.96	42	R385-82	911.65	4	20	6
NP-126S	I385NBCD	338+83.00	54	R385NBCD-83	922.12	10	8	0
NP-131SS	I385NBCD	344+00.00	54	W385-1R-01	925.70	13	8	0
NP-132SS	I385NBCD	341+55.40	54	R385-27A	924.64	2	27	12
NP-133SS	I385	341+59.87	54	R385-27A	924.33	2	27	12
NP-134S	I385	346+57.27	30	W385-1R-02	928.37	12	8	0
NP-136S	RAMP_11	51+46.83	18	BX-385-01	929.75	2	27	12
NP-137S	I385	382+60.00	24	W4-1R-11	996.64	8	16	3
NP-138S	I385	382+60.00	24	W4-1R-11	996.34	8	16	3
NP-139S	I385	383+67.00	30	W4-1R-11	995.43	8	16	3
NP-140S	I385SB	384+00.00	30	W4-1R-11	995.23	8	16	3
NP-141S	I385	384+67.26	30	W4-1R-11	994.94	8	16	3
NP-142S	I385NB	386+01.51	36	B-7	994.04	8	16	3

Table 5.24 Pipe Culvert Undercut Requirements (con't)								
Pipe Location			**Pipe Diameter (inch)**	**Closest Boring**	**Elevation 5 feet Below Lowest Invert (ft)**	**Lowest SPT N-Value within 5 feet Below Invert Elevation (bpf)**	**Required Depth of Undercut Below Bedding (2) (inch)**	
Structure	**Alignment**	**STA**					**Option 1 (no Geogrid)**	**Option 2 (with Geogrid)**
NP143S	I385SB	384+78.00	18	W2B-1R-05	1000.20	3	20	6
NP-144S	I385SB	386+06.39	18	W2B-1R-04	995.34	9	8	0
NP-145S	RAMP_2B	48+51.11	18	W2B-1R-04	994.96	9	8	0
NP-146S	I385SBCD	122+38.00	18	W385-RS-03	939.75	3	20	6
NP-14S	I385	349+00.00	30	B-62	931.84	28	0	0
NP-1506	RAMP_4	51+16.00	18	W4-1R-08	989.70	6	16	3
NP-151S	RAMP_4	52+33.63	18	W4-1R-05	978.60	21	0	0
NP-152S	I385NBCD	351+66.98	36	BX-I385NBCD-01	932.92	22	0	0
NP-153S	RAMP_2B	41+95.95	24	W2B-1R-03	972.00	15	8	0
NP-154S	RAMP_1A	89+47.18	18	W2B-1R-01	995.53	11	8	0
NP-155S	RAMP_4	41+56.14	18	B-7	997.44	8	16	3
NP-156	I385NBCD	370+17.02	18	R8A-35	964.60	7	16	3
NP-16S	I385	352+99.48	24	W385-RN-02	933.84	28	0	0
NP-17S	I385NBCD	360+00.00	18	W385-RN-05	939.40	15	8	0
NP-18S	I385NBCD	358+00.00	36	W385-RN-04	937.02	100+	0	0
NP-1S	I385NBCD	338+90.27	54	R385NBCD-83	922.42	10	8	0
NP-20S	I385NBCD	358+00.00	18	W385-RN-04	938.52	100+	0	0
NP-21S	I385	356+00.00	36	W385-RN-03	935.67	100+	0	0
NP-22S	I385	354+08.00	36	B-63	933.82	13	8	0
NP-24S	I385NBCD	351+66.98	18	BX-I385NBCD-01	934.42	22	0	0
NP-25S	I385NBCD	347+92.96	36	W385-RN-01	928.68	15	8	0
NP-29SS	I385NBCD	346+18.93	54	W385-1R-02	926.61	12	8	0
NP-305	I85	223+38.37	18	R85-55/54	949.27	8	16	3
NP-306	I85	223+29.39	18	R85-55/54	947.03	8	16	3
NP-34S	RAMP_11	51+46.83	42	W385-1R-02	927.75	12	8	0
NP-35S	RAMP_11	51+46.83	24	W385-1R-02	929.25	12	8	0
NP-36S	I385	346+57.27	42	BX-385-01	927.37	2	27	12
NP-37S	I385	345+25.00	48	BX-385-01	926.49	2	27	12
NP-38S	I385	345+25.00	48	W385-1R-01	926.00	13	8	0

Table 5.24 Pipe Culvert Undercut Requirements (con't)

Pipe Location			Pipe Diameter (inch)	Closest Boring	Elevation 5 feet Below Lowest Invert (ft)	Lowest SPT N-Value within 5 feet Below Invert Elevation (bpf)	Required Depth of Undercut Below Bedding (2) (inch)	
Structure	Alignment	STA					Option 1 (no Geogrid)	Option 2 (with Geogrid)
NP-403	I85	234+07.00	15	R85-57	962.66	7	16	3
NP-404	I85	234+03.00	18	R85-57	958.86	13	8	0
NP-40S	RAMP_8A	19+99.99	24	R8A-35	963.58	7	16	3
NP-41S	RAMP_8A	18+00.00	18	BX-I385NBCD-02	952.00	20	0	0
NP-42S	I385NBCD	366+50.00	18	W385-2L-02	944.09	100+	0	0
NP-43S	I385NBCD	366+50.55	24	W385-2L-02	943.33	100+	0	0
NP-44S	I385NBCD	366+50.00	18	W385-2L-02	944.08	100+	0	0
NP-46S	I385NBCD	365+55.18	24	W385-2R-02	943.06	100+	0	0
NP-47S	I385NBCD	364+00.00	24	W385-2R-01	942.51	8	16	3
NP-48S	I385	361+60.00	24	W385-2R-01	939.74	8	16	3
NP-49S	I385NBCD	360+00.00	36	W385-RN-05	937.90	15	8	0
NP-500	I385NBCD	347+92.96	24	R11-28	929.68	40	0	0
NP-502	I385NBCD	339+39.40	54	R385NBCD-83	922.70	10	8	0
NP-503	I385	357+00.00	118	W385-RS-03	936.34	6	16	3
NP-504	I85	241+71.52	18	R85-58	980.02	6	16	3
NP-505	RAMP_5	128+17.93	18	R85-59	973.51	9	8	0
NP-50S	I385	361+60.00	18	W385-RN-06	940.14	51	0	0
NP-51S	I385NBCD	361+50.00	18	W385-RN-07	940.44	24	0	0
NP-52S	RAMP_8	54+27.61	18	BX-8-01	956.75	15	8	0
NP-53S	RAMP_8A	11+50.00	18	R8A-31	960.00	9	8	0
NP-5501N	I385	435+00.00	18	W385-4R-03	1046.29	9	8	0
NP-5502N	I385	431+60.00	18	W385-4R-03	1039.40	5	16	3
NP-5504N	I385	428+07.00	36	W385-4R-02	1031.18	5	16	3
NP-5505N	I385	431+60.00	36	W385-4R-02	1037.90	8	16	3
NP-55S	RAMP_10	33+16.58	48	BX-10-01	928.60	100+	0	0
NP-5601N	I385	422+00.00	18	R3A-46	1019.32	9	8	0
NP-5602N	I385	420+50.00	18	R2A-45	1016.14	7	16	3
NP-5603N	I385	428+07.00	24	W385-4R-01	1032.10	14	8	0
NP-5604N	I385	424+42.00	36	R3A-46	1021.52	8	16	3
NP-5605N	I385	423+03.00	36	R3A-46	1017.07	9	8	0

Table 5.24 Pipe Culvert Undercut Requirements (con't)

Pipe Location			Pipe Diameter (inch)	Closest Boring	Elevation 5 feet Below Lowest Invert (ft)	Lowest SPT N-Value within 5 feet Below Invert Elevation (bpf)	Required Depth of Undercut Below Bedding (2) (inch)	
Structure	Alignment	STA					Option 1 (no Geogrid)	Option 2 (with Geogrid)
NP-5606N	I385	422+00.00	36	R3A-46	1014.80	14	8	0
NP-5612N	I385	411+66.00	18	R385-74	1001.60	9	8	0
NP-5613N	I385	411+66.00	48	R3-75	998.60	8	16	3
NP-5614N	RAMP_3A	317+00.00	48	R3-75	999.40	8	16	3
NP-5615N	I385	411+50.00	54	R385-73	996.00	100+	0	0
NP-5616N	I385	417+00.00	24	R2A-45	1011.70	7	16	3
NP-5617N	I385	414+00.00	24	R2B-44	1008.70	9	8	0
NP-56S	RAMP_10	31+76.50	42	BX-10-01	930.10	100+	0	0
NP-57S	I385	351+14.00	42	BX-10-01	932.01	100+	0	0
NP-5801	RAMP_1A	69+18.00	30	W1A-1R-08	978.28	10	8	0
NP-5802	RAMP_1A	66+60.00	30	W1A-1R-07	973.89	8	16	3
NP-5803	RAMP_1A	64+00.00	36	R1A-76	965.90	13	8	0
NP-5804	RAMP_1A	61+75.00	42	W1A-1R-06	961.99	10	8	0
NP-5805	RAMP_1A	61+75.00	24	B-37	963.49	17	0	0
NP-5806	RAMP_1A	51+00.00	36	W1A-1R-04	917.07	2	27	12
NP-5807	RAMP_1A	57+75.00	42	R1A-51	944.48	16	0	0
NP-5813	I85	319+16.00	42	R1A-51	915.28	2	27	12
NP-58S	I385	354+61.00	18	R10-29	935.38	36	0	0
NP-59S	I385	354+61.00	36	R10-29	933.88	10	8	0
NP-6001N1	RAMP_2A	65+61.16	60	R2-39	994.47	3	18	4
NP-6002	RAMP_1B	61+35.00	36	W1B-2R-02	984.40	2	27	12
NP-6002N1	RAMP_2	61+71.22	18	R2-39	999.54	5	16	3
NP-605	RAMP_5	120+52.53	15	R85-61	991.08	11	8	0
NP-6101	RAMP_2A	64+00.00	24	R2-70	994.04	4	20	6
NP-6102	RAMP_2A	64+00.00	24	R2A-71	993.44	4	20	6
NP-6201	RAMP_2	65+41.00	24	R4B-85	992.26	12	8	0
NP-6203	RAMP_1B	65+06.00	36	W85-2L-03A	991.54	11	8	0
NP-6205	I85	281+53.00	36	R3-05	988.91	8	16	3
NP-6206	I85	279+83.00	30	W85-2L-02	991.03	5	16	3
NP-6207	RAMP_3	32+03.43	36	R3-05	984.26	3	20	6

Table 5.24 Pipe Culvert Undercut Requirements (con't)

Pipe Location			Pipe Diameter (inch)	Closest Boring	Elevation 5 feet Below Lowest Invert (ft)	Lowest SPT N-Value within 5 feet Below Invert Elevation (bpf)	Required Depth of Undercut Below Bedding (2) (inch)	
Structure	Alignment	STA					Option 1 (no Geogrid)	Option 2 (with Geogrid)
NP-6301	RAMP_4	61+19.08	30	W4-1R-01	967.23	5	16	3
NP-6302	RAMP_4	61+83.00	30	W4-1R-01	966.50	7	16	3
NP-6303	RAMP_4	65+85.00	30	W2A-2L-01	962.87	41	0	0
NP-6304	RAMP_4	68+00.00	30	B-54	960.67	6	16	3
NP-6305	I85	315+28.00	18	R1A-51	938.50	7	16	3
NP-6306	RAMP_2A	103+00.00	30	R85-09	942.87	17	0	0
NP-6307	RAMP_2A	108+00.00	30	W2A-1R-01	920.00	51	0	0
NP-6308	RAMP_2A	111+75.00	36	B11-STP-01	907.39	11	8	0
NP-6309	RAMP_2A	114+60.00	36	W2A-1R-06	896.70	16	0	0
NP-6310	RAMP_2A	114+60.00	36	W2A-1R-06	896.92	16	0	0
NP-6312	I85	306+00.00	36	R1A-76	967.54	15	8	0
NP-64S	I385SBCD	118+80.00	24	BX-I385SBCD-02	951.60	17	0	0
NP-65S	I385SBCD	116+50.00	24	BX-I385SBCD-02	962.82	100+	0	0
NP-67S	I385SBCD	118+79.43	24	B-41	950.49	100+	0	0
NP-68S	I385SBCD	119+80.00	24	W385-RS-06	948.39	100+	0	0
NP-69S	I385SBCD	122+38.00	18	W385-RS-03	939.62	3	20	6
NP-702	I85	251+53.35	18	R85-60	1000.37	8	16	3
NP-703	I85NBCD	250+65.09	18	R85-60	999.15	8	16	3
NP-70S	I385	360+26.67	36	W385-RS-03	937.90	3	20	6
NP-7101	RAMP_1B	76+40.45	18	R1B-04	1003.58	100+	0	0
NP-7104	I85	276+56.13	24	R85-66	994.56	7	16	3
NP-7112	RAMP_3	30+00.37	36	R85-66	986.35	7	16	3
NP-71S	I385	358+82.05	36	W385-RS-03	936.80	6	16	3
NP-7203	I85	278+50.70	18	W85-2L-01	991.97	19	0	0
NP-7204	I85	281+53.00	30	R3-05	989.07	8	16	3
NP-7205	RAMP_3	32+03.43	36	R3-05	984.26	3	20	6
NP-7206	RAMP_3	32+50.00	48	R3-05	983.07	3	20	6
NP-7207	I85	319+16.00	30	W1A-1R-04	916.28	2	27	12
NP-7209	I85	322+08.00	42	W1A-1R-03	910.10	12	8	0
NP-7210	I85	324+00.00	42	W1A-1R-02	901.11	3	20	6

Table 5.24 Pipe Culvert Undercut Requirements (con't)

Pipe Location			Pipe Diameter (inch)	Closest Boring	Elevation 5 feet Below Lowest Invert (ft)	Lowest SPT N-Value within 5 feet Below Invert Elevation (bpf)	Required Depth of Undercut Below Bedding (2) (inch)	
Structure	Alignment	STA					Option 1 (no Geogrid)	Option 2 (with Geogrid)
NP-7211	I85	328+31.98	24	R85-14A	895.68	4	20	6
NP-7213	RAMP_2A	98+00.00	30	B-54	959.47	6	16	3
NP-7214	RAMP_2A	100+00.00	30	B-54	954.46	6	16	3
NP-7217	RAMP_4	61+83.00	18	W2A-2L-01	966.50	25	0	0
NP-7218	RAMP_4	63+44.00	30	W2A-2L-01	965.33	25	0	0
NP-7220	I85	325+00.00	42	W1A-1R-01	899.77	4	20	6
NP-7221	I85	334+47.00	18	R85-15	916.39	100+	0	0
NP-7223	I85	319+50.00	42	W1A-1R-04	914.60	2	27	12
NP-7224	RAMP_1B	60+56.87	36	W1B-2R-02	982.70	6	16	3
NP-7225	I85	286+00.81	36	W1B-2R-02	979.70	100+	0	0
NP-7226	RAMP_1B	60+53.74	36	W1B-2R-02	979.56	100+	0	0
NP-7229	RAMP_2A	117+35.96	18	R85-13	899.73	4	20	6
NP-7230	I85	293+36.00	18	B-27	982.93	56	0	0
NP-7231	I85	290+74.93	18	B05-SPT-06	982.06	8	16	3
NP-7233	RAMP_2A	114+25.00	24	W2A-1R-06	900.50	9	8	0
NP-7234	I85	330+00.00	18	R85-13A	902.00	4	20	6
NP-7235	I85	311+00.00	36	R85-09	951.00	14	8	0
NP-7236	RAMP_2B	29+50.00	18	B-11	996.00	10	8	0
NP-7237	I85	291+04.75	18	B12-SPT-06	985.35	18	0	0
NP-7238	I85NBCD	291+50.00	18	B12-SPT-06	984.95	18	0	0
NP-7239	I85	309+00.00	36	B-54	958.10	6	16	3
NP-7240	I85	326+25.00	42	BX-1-02	898.16	20	0	0
NP-7241	I85	327+82.00	48	BX-1-02	895.22	8	16	3
NP-7242	I85	330+00.00	24	R85-14	901.49	4	20	6
NP-7243	I85	284+62.00	18	W1B-2R-01	983.74	9	8	0
NP-7244	I85	286+00.81	18	W1B-2R-02	981.20	6	16	3
NP-7245	I85	327+32.00	24	R85-13	898.65	4	20	6
NP-7246	RAMP_4B	420+34.00	24	W85-1L-04	995.20	13	8	0
NP-7247	RAMP_1A	51+00.00	24	W1A-1R-04	918.37	2	27	12
NP-7249	RAMP_1A	53+75.00	42	R1A-50	921.47	100+	0	0

Table 5.24 Pipe Culvert Undercut Requirements (con't)

Pipe Location			Pipe Diameter (inch)	Closest Boring	Elevation 5 feet Below Lowest Invert (ft)	Lowest SPT N-Value within 5 feet Below Invert Elevation (bpf)	Required Depth of Undercut Below Bedding (2) (inch)	
Structure	Alignment	STA					Option 1 (no Geogrid)	Option 2 (with Geogrid)
NP-7250	RAMP_1A	51+00.00	42	W1A-1R-05	916.57	8	16	3
NP-7251	I85	317+95.00	36	W2A-1R-01	923.85	62	0	0
NP-7252	RAMP_2A	109+00.00	36	W2A-1R-02	912.85	10	8	0
NP-7253	RAMP_2A	105+00.00	30	RCH-10	935.56	17	0	0
NP-7254	RAMP_2A	114+25.00	36	W2A-1R-04/05	897.37	7	16	3
NP-7255	RAMP_2A	114+60.00	48	W2A-1R-06	895.70	16	0	0
NP-7256	RAMP_2A	114+73.04	54	W2A-1R-06	894.00	16	0	0
NP-7257	RAMP_1	72+71.00	54	R85-14A	891.24	2	27	12
NP-7258	I85	327+10.00	48	BX-1-02	896.07	8	16	3
NP-7259	RAMP_1	103+18.00	36	W1.1-R2-01	974.06	9	8	0
NP-7260	RAMP_1	73+56.00	18	BX-1-02	899.56	20	0	0
NP-7261	RAMP_3A	297+25.00	18	W2A-MB1-01	976.50	2	27	12
NP-7264	RAMP_2A	110+60.00	18	W2A-1R-03	911.55	11	8	0
NP-7265	RAMP_2A	111+75.00	18	W2A-1R-04	908.89	3	20	6
NP-7266	RAMP_2A	114+25.00	36	W2A-1R-05	899.00	8	16	3
NP-7267	RAMP_1A	71+00.00	18	W1A-1R-08	979.80	17	0	0
NP-7268	RAMP_1A	51+00.00	18	R1A-77	918.87	24	0	0
NP-7270	RAMP_3A	289+16.00	18	W3A-1R-01	987.42	6	16	3
NP-7271	RAMP_3A	289+12.31	18	W3A-1R-01	991.94	6	16	3
NP-7272	I85	281+52.00	30	W85-2L-02	990.63	100+	0	0
NP-7273	I85	279+83.00	18	W85-2L-02	993.14	100+	0	0
NP-7276	RAMP_1B	74+16.00	18	W85-1L-03	996.32	100+	0	0
NP-7277	RAMP_2A	111+20.00	36	W2A-1R-04	908.23	3	20	6
NP-7278	I85	320+94.00	36	B-30	910.69	3	20	6
NP-7279	I85	320+05.00	36	W2A-1R-02	912.49	12	8	0
NP-7280	I85	318+57.00	18	R1A-50	917.88	100+	0	0
NP-7281	I85	331+99.00	18	R85-14	908.90	4	20	6
NP-7282	I85	293+75.00	18	B06-SPT-04	983.14	19	0	0
NP-7283	I85	294+35.00	18	B06-SPT-04	983.41	19	0	0
NP-7284	I85	294+55.00	18	B06-SPT-04	983.58	19	0	0

Table 5.24 Pipe Culvert Undercut Requirements (con't)

Pipe Location			Pipe Diameter (inch)	Closest Boring	Elevation 5 feet Below Lowest Invert (ft)	Lowest SPT N-Value within 5 feet Below Invert Elevation (bpf)	Required Depth of Undercut Below Bedding (2) (inch)	
Structure	Alignment	STA					Option 1 (no Geogrid)	Option 2 (with Geogrid)
NP-7285	I85NBCD	285+52.96	24	B07-SPT-02	976.00	6	16	3
NP-7286	I85	320+61.00	42	B11-SPT-06	912.90	56	0	0
NP-7287	I85	320+66.00	42	B11-SPT-06	912.30	56	0	0
NP-7288	RAMP_2A	89+00.00	30	W85-3R-02	974.00	9	8	0
NP-7289	I85	285+44.00	18	B09-SPT-01	983.31	5	16	3
NP-7290	I85	285+59.00	18	W1B-2R-02	983.16	6	16	3
NP-7291	RAMP_3A	287+83.00	24	W3A-1R-01	986.52	6	16	3
NP-7294	RAMP_1	104+64.00	36	R1-06	975.96	14	8	0
NP-7295	RAMP_2A	83+50.00	24	W2A-MB1-02	976.00	6	16	3
NP-7296	RAMP_1	111+50.00	18	R1-40	991.65	8	16	3
NP-7297	RAMP_1	111+50.00	18	R1-40	991.65	8	16	3
NP-7298	I85	290+74.93	18	BX-1B-01	982.06	14	8	0
NP-7299	RAMP_1	108+07.00	30	BX-1-01	980.24	8	16	3
NP-7299N	RAMP_2A	68+00.00	18	B-10	999.00	9	8	0
NP-72S	I385SBCD	116+50.00	24	BX-I385SBCD-02	962.82	100+	0	0
NP-7300	RAMP_1	115+00.00	18	B06-SPT-08	1003.81	22	0	0
NP-7301	RAMP_1	108+07.00	18	BX-1-01	981.24	8	16	3
NP-73S	I385	361+54.00	18	W385-RS-07	943.40	100+	0	0
NP-76S	I385NBCD	335+15.41	30	R385-26	921.36	8	16	3
NP-77S	I385NBCD	335+15.41	18	R385-26	922.36	8	16	3
NP-8000	RAMP_3	39+43.98	18	B12-SPT-03	996.00	8	16	3
NP-8001	I85	276+00.00	24	W85-1L-05	993.66	33	0	0
NP-801	RAMP_5	114+48.89	15	R85-62	1003.66	6	16	3
NP-802	RAMP_5	116+60.95	18	R85-62	1001.23	6	16	3
NP-804	RAMP_5	116+79.72	18	R85-62	1000.23	6	16	3
NP-81S	I385NBCD	329+50.07	18	R385-25A	916.92	31	0	0
NP-82S	I385NBCD	327+79.90	18	R385-25A	914.95	31	0	0
NP-83S	I385	327+15.00	18	R385-24A	913.90	31	0	0
NP-87S	I385	382+60.00	18	B-34	996.84	11	8	0
NP-906S	RAMP_4	47+24.29	18	W4-1R-07	1001.37	13	8	0

Table 5.24 Pipe Culvert Undercut Requirements (con't)

Pipe Location			Pipe Diameter (inch)	Closest Boring	Elevation 5 feet Below Lowest Invert (ft)	Lowest SPT N-Value within 5 feet Below Invert Elevation (bpf)	Required Depth of Undercut Below Bedding (2) (inch)	
Structure	Alignment	STA					Option 1 (no Geogrid)	Option 2 (with Geogrid)
NP-91000	RAMP_4	68+00.00	18	B-54	961.67	7	16	3
NP-91001	RAMP_2A	114+60.00	48	W2A-1R-06	894.55	16	0	0
NP-91002	RAMP_2A	115+36.00	24	R85-13	898.14	4	20	6
NP-91003	RAMP_2A	114+60.00	30	R85-13	897.42	4	20	6
NP-9200	RAMP_1	72+84.11	54	R85-14A	891.44	2	27	12
NP-92000	RAMP_8A	11+50.00	18	BX-8-01	960.00	25	0	0
NP-92001	I385	360+00.00	18	W385-RS-03	939.86	3	20	6
NP-92002	I385	360+00.00	18	W385-RS-03	939.86	3	20	6
NP-9201	RAMP_1	72+84.11	48	R85-14A	892.02	2	27	12
NP-9202	RAMP_1	107+50.00	18	R1-40	979.51	7	16	3
NP-9203	RAMP_1	105+50.00	30	BX-1-01	976.31	8	16	3
NP-9204	RAMP_1	107+50.00	18	BX-1-01	978.41	8	16	3
NP-92S	RAMP_4	44+42.00	18	W4-1R-07	999.38	13	8	0
NP-9300	I85	285+50.00	36	B07-SPT-02	975.71	6	16	3
NP-93000	I385	419+50.00	36	R2A-45/R3A-46	1009.10	9	8	0
NP-9301	I85NBCD	285+52.96	36	B07-SPT-02	975.00	6	16	3
NP-9302	I85NBCD	285+53.00	36	B07-SPT-02	974.49	6	16	3
NP-9303	I85NBCD	286+59.39	36	B07-SPT-02	973.27	21	0	0
NP-94000	I385NBCD	327+79.90	18	R385-24A	913.67	100+	0	0
NP-94S	I385SB	386+06.39	42	W2B-1R-05	993.34	9	8	0
NP-95000	RAMP_3	34+97.41	18	B-19	987.07	6	16	3
NP-95S	RAMP_2B	48+51.11	42	W2B-1R-05	992.96	9	8	0
NP-96S	RAMP_2B	48+51.11	42	W2B-1R-05	974.00	13	8	0
NP-97S	RAMP_2B	49+41.34	18	W2B-1R-05	999.84	3	20	3
NP-98000	I385SBCD	122+38.00	24	W385-RS-03	939.22	3	20	3
NP-98S	RAMP_2B	48+51.11	18	W2B-1R-05	994.96	9	8	0
NP-99S	RAMP_3	44+03.38	18	R385-37	1004.26	6	16	3
NP-CECS_5706N	I385	413+50.00	24	R2B-44	1008.10	8	16	3
NP-CECS-5701N	I385	424+00.00	18	R3A-46	1024.08	8	16	3
NP-CECS-5702N	RAMP_3A	320+67.00	36	R2A-45	1004.49	10	8	0

Table 5.24 Pipe Culvert Undercut Requirements (con't)								
Pipe Location			**Pipe Diameter (inch)**	**Closest Boring**	**Elevation 5 feet Below Lowest Invert (ft)**	**Lowest SPT N-Value within 5 feet Below Invert Elevation (bpf)**	**Required Depth of Undercut Below Bedding (2) (inch)**	
Structure	**Alignment**	**STA**					**Option 1 (no Geogrid)**	**Option 2 (with Geogrid)**
NP-CECS-5703N	RAMP_3A	317+00.00	30	R3-75	1001.00	8	16	3
NP-CECS-5704N	I385	411+50.00	48	R385-74	997.30	9	8	0
NP-CECS-5705N	I385	411+50.00	24	R385-74	1001.50	35	0	0
NP-CECS-5707N	I385	408+50.00	24	B06-SPT-09	1003.43	4	20	6
NP-CECS-5708N	I385	409+50.00	24	R2B-42	1002.33	9	8	0
NP-CECS-5709N	I385	411+50.00	30	R385-74	1001.00	35	0	0
NP-CECS-5710N	I385	405+50.00	18	B-12	1005.95	5	16	3
NP-CECS-5711N	I385	407+50.00	24	B06-SPT-09	1003.93	4	20	6
NP-STN-1	I85	278+50.70	24	W85-2L-02	992.07	5	16	3
NP-STN-100	RAMP_3A	289+16.00	24	W3A-1R-01	987.02	6	16	3
NP-STN-101	RAMP_3A	287+57.00	24	B07-SPT-02	986.42	4	20	6
NP-STN-102	I85NBCD	291+50.00	18	B12-SPT-06	984.95	18	0	0
NP-STN-103	I85NBCD	295+00.00	24	B12-SPT-06	980.15	19	0	0
NP-STN-104	I85NBCD	295+40.00	24	B06-SPT-03	979.80	54	0	0
NP-STN-106	I85NBCD	297+50.00	30	W85-3R-01	976.51	14	8	0
NP-STN-107	I85NBCD	298+00.00	18	B08-SPT-03	976.00	7	16	3
NP-STN-108	I85NBCD	298+00.00	30	B08-SPT-03	975.00	7	16	3
NP-STN-109	RAMP_4	61+19.08	18	BX-4-01	968.23	8	16	3
NP-STN-11	RAMP_2A	81+14.48	24	B-17	980.60	23	0	0
NP-STN-112	RAMP_2A	84+00.00	24	W2A-MB1-02	977.48	6	16	3
NP-STN-114	RAMP_2A	83+50.00	18	W2A-MB1-02	976.50	4	20	6
NP-STN-115	RAMP_4	55+35.00	24	W2A-MB2-01	977.10	14	8	0
NP-STN-116	RAMP_4	55+00.00	24	W2A-MB1-02	970.83	4	20	6
NP-STN-117	RAMP_4	57+01.64	30	W4-1R-03	969.33	5	16	3
NP-STN-12	RAMP_1B	55+80.00	18	BX-1B-01	981.80	3	20	6
NP-STN-120	RAMP_1	103+18.00	36	W1.1-R2-01	973.86	9	8	0
NP-STN-122	RAMP_1	99+00.02	36	W1A-1R-10	971.82	6	16	3
NP-STN-123	RAMP_2A	92+92.00	30	W2A-2L-01	971.24	13	8	0
NP-STN-124	RAMP_1	74+06.00	18	BX-1-02	899.56	1	27	12

Table 5.24 Pipe Culvert Undercut Requirements (con't)

Pipe Location			Pipe Diameter (inch)	Closest Boring	Elevation 5 feet Below Lowest Invert (ft)	Lowest SPT N-Value within 5 feet Below Invert Elevation (bpf)	Required Depth of Undercut Below Bedding (2) (inch)	
Structure	Alignment	STA					Option 1 (no Geogrid)	Option 2 (with Geogrid)
NP-STN-125	RAMP_1	72+84.11	18	R85-14A	898.57	4	20	6
NP-STN-126	RAMP_1	101+00.00	36	W1.1-R2-01	972.90	8	16	3
NP-STN-13	RAMP_1B	54+29.37	24	BX-1B-01	981.00	3	20	6
NP-STN-14	RAMP_1	108+00.00	30	BX-1-01	979.63	8	16	3
NP-STN-15	RAMP_1	107+50.00	30	BX-1-01	977.41	8	16	3
NP-STN-151	I85NBCD	291+03.99	18	B12-SPT-06	985.70	18	0	0
NP-STN-2	RAMP_1B	65+06.00	18	W85-2L-03A	991.76	11	8	0
NP-STN-3015	RAMP_2A	110+60.00	30	W2A-1R-01	910.55	10	8	0
NP-STN-3016	RAMP_2A	108+00.00	18	W2A-1R-01	920.50	62	0	0
NP-STN-50	I85NBCD	291+03.99	18	B12-SPT-12	985.70	9	8	0
NP-STN-53	I85	303+85.88	18	W1A-1R-10	976.70	12	8	0
NP-STN-54	RAMP_1	95+00.00	24	R1A-76	968.64	10	8	0
NP-STN-5810	RAMP_1	97+00.00	36	B-36	970.74	7	16	3
NP-STN-5811	RAMP_1	95+00.00	36	R1A-76	967.64	15	8	0
NP-STN-5812	RAMP_1A	64+00.00	36	R1A-76	965.70	13	8	0
NP-STN-5814	RAMP_1A	64+00.00	36	R1A-76	965.70	13	8	0
NP-STN-6	RAMP_1B	60+56.87	18	B-20	984.08	4	20	6
NP-STN-7	RAMP_1B	59+20.94	18	W1B-2R-03	987.41	10	8	0
NP-STN-7212	I85	328+31.98	48	R85-14A	893.68	4	20	6
NP-STN-7224	I85	327+32.00	48	BX-1-02	895.65	4	20	6
NP-STN-7225	I85	326+96.15	42	BX-1-02	896.85	4	20	6
NP-STN-7228	I85	277+81.00	18	W85-2L-01	993.10	12	8	0
NP-STN-7230	I85	313+90.00	36	R85-09	936.10	18	0	0
NP-STN-7231	I85	327+00.00	24	R85-13	898.43	4	20	6
NP-STN-7232	RAMP_2A	116+86.41	18	R85-13	899.29	4	20	6
NP-STN-7246	RAMP_1B	74+16.00	18	R1B-04	996.32	100+	0	0
NP-STN-7247	RAMP_4B	421+00.00	18	W85-1L-03	996.02	100+	0	0
NP-STN-7249	I85	277+81.00	30	W85-2L-01	992.10	12	8	0
NP-STN-7250	RAMP_1B	72+61.93	18	W1B-3R-02	1015.83	22	0	0

Table 5.24 Pipe Culvert Undercut Requirements (con't)

Pipe Location			Pipe Diameter (inch)	Closest Boring	Elevation 5 feet Below Lowest Invert (ft)	Lowest SPT N-Value within 5 feet Below Invert Elevation (bpf)	Required Depth of Undercut Below Bedding (2) (inch)	
Structure	Alignment	STA					Option 1 (no Geogrid)	Option 2 (with Geogrid)
NP-W20001	I85	362+25.68	18	R85-79	913.50	19	0	0
NP-W200100	I85	367+18.38	24	R85-17	900.75	3	20	6
NP-W20101	I85	367+23.33	24	R85-17	898.30	4	20	6
NP-W20105	I85	367+07.05	24	R85-80	906.63	9	8	0
NP-W20200	I85	371+31.24	18	R85-81	895.94	59	0	0
NP-W20201	I85	372+00.00	18	R85-81	895.46	59	0	0
NP-W20300	I85	356+97.19	18	R85-78	917.68	10	8	0
NP-W20301	I85	354+76.00	24	R85-78	916.17	10	8	0
NP-W20302	I85	354+75.84	36	R85-78	915.05	10	8	0
NP-W20303	I85	354+76.00	18	R1A-77	916.67	24	0	0
NP-W20304	RAMP_1	51+27.82	18	R1A-77	934.46	13	8	0
NP-W20305	I85	350+31.84	18	R1A-77	927.60	24	0	0
NP-W20401	I85	374+31.61	19"x30"	R85-18	890.44	8	16	3
NP-4444	ROPER	40+11		B-31	936.26	5	16	3
NP-6000	ROPER	40+11		W1A-1R-03	916.21	13	8	0
NP-7208	RAMP 1	80+79.02		W1A-1R-02	905.22	38	0	0
NP-7222	I85NBCD	270+00.00		W1A-1R-01	902.24	3	20	6
NP-7262	RAMP_2A	109+42.35		W2A-1R-02	919.73	12	8	0
NP-7263	RAMP_2A	109+72.35		W2A-1R-03	912.18	11	8	0
NP-8000	RAMP_4B	420+34.00		W85-1L-05	994.2	16	0	0
NP-98001	RAMP_2A	117+79.02		R85-13	891.39	8	16	3
Box Culvert	Ramp 2A	115+00 to 117+65	80" x 80"	R85-13 and W2A-1R-06	887	8	14	0

Notes:
1. Per SCDOT Std. Drawing 714-020-00, Table 714-020A

5.11 Corrosion and Deterioration

The effects of corrosion and deterioration from environmental conditions were considered in the selection of pile type and size. In accordance with Section 16-3 of the GDM, analysis for the long-term durability of the pile in service (i.e. corrosion and deterioration) was based on the 2012 AASHTO LRFD Bridge Design Specifications, Section 10.7.5. Site-specific laboratory soil pH testing of the subsurface soils was performed by TestAmerica Laboratories, Inc and incorporated into Thompson Engineering GSDR reports. The test results for corrosion testing are summarized in Table 5.24.

In accordance with AASHTO LRFD Section 10.7.5, the following soil or site conditions that should be considered indicative of a potential pile deterioration or corrosion situation are as follows:

- Resistivity less than 2,000 ohm-cm,
- pH less than 5.5,
- pH between 5.5 and 8.5 in soils with high organic content,
- Sulfate concentrations greater than 1,000 ppm,
- Landfills and cinder fills,
- Soils subject to mine or industrial drainage,
- Areas with a mixture of high resistivity soils and low resistivity high alkaline soils, and
- Insects (for wood piles only).

Table 5.25 – Summary of Corrosion Testing

Sample	Sample Depth (feet)	Resistivity (ohm-m)	pH	Sulfate (mg/kg)	Chloride (mg/kg)
B04-SPT-01	10.0 – 19.0	54,000	4.95	<23	<23
B05-SPT-01	6.0 – 15.0	36,000	5.43	<24	<24
B06-SPT-01	10.0 – 18.5	17,000	5.38	60	<26
B06-SPT-12	43.5 – 55.0	96,000	5.10	<26	<26
B09-SPT-01	18.5 – 30.0	62,000	5.28	<24	<24
B09-SPT-05	33.5 – 45.0	54,000	5.51	<29	<29
B10-SPT-01	4.0 – 9.0	23,000	5.13	<24	38
B10-SPT-02	6.0 – 12.0	38,000	5.15	<25	<25
B11-SPT-06	13.5 – 23.5	96,000	5.42	<23	<23

The corrosion test results were provided to the structural engineer to determine if additional measures are required to offset the low pH results for soldier pile and lagging walls MSE Walls and Reinforced Concrete walls (i.e. barrier walls), should be backfilled with materials meeting the electro chemical properties outlined in FHWA Manual FHWA-NHI-00-043.

6 CONSTRUCTION CONSIDERATIONS

6.1 Borrow Materials

Borrow materials for this project will meet the SCDOT 2007 Standard Specifications for Highway Construction, Section 203.2.1.8. SCDOT specifications indicate that in Greenville County borrow materials are classified as Group A. Borrow materials in Group A require AASHTO soils A-1, A-2, A-3, A-4, A-5, and A-6 to be within the top 5 feet of the embankment. Below a depth of 5 feet, any soil that does not meet the description of muck may be used to form embankments as long as it is stable when compacted to the required density.

Based on the soil borings, it is anticipated that potential project borrow source materials near Bridge 11 will meet the required SCDOT specifications indicated in Section 203.2.1.8 of the SCDOT 2007 Standard Specification for Highway Construction. When borrow sources are identified, bulk samples will be collected and the appropriate soil properties will be tested in accordance with Chapter 7 of the GDM.

6.2 In-Situ Soil Shear Strength

Static short-term soil shear strengths were computed using the SPT soil borings for the evaluation of the Strength and Service limit states. The corrected SPT blow counts were used to obtain total soil shear strength (cohesion, c) for cohesive soils and effective shear strength (internal friction angle, φ') for cohesionless soils based on correlations included in the 2010 SCDOT GDM, Sections 7.10 and 7.11, respectively. For sand-like soils (cohesionless soils, FC $\leq$ 20% and PI $\leq$ 7) an internal angle of friction was assigned based on correlations for sands. For clay-like soils (cohesive soils, FC > 20% and PI > 7) a cohesion value was assigned based on correlations for clays. The computed SPT soil shear strength parameters (internal angle of friction and cohesion) parameters were combined and evaluated to use either a lower bound shear strength approach or limit to the maximum allowable total and effective soil shear strengths in general accordance with Tables 7-15 and 7-16 of the SCDOT GDM, unless advanced laboratory testing warranted deviating from the SCDOT GDM.

Seismic soil shear strengths were computed using SPT soil borings for the evaluation of the Extreme Event I limit state. Soils that are not susceptible to soil shear strength loss were assigned the static short-term soil shear strengths. Static long-term soil shear strengths only affect clay-like soils after excess pore water pressure has dissipated. The static long-term soil shear strength of clay-like soils is typically modeled using drained effective shear strength parameters (i.e. internal friction angle, φ'). The effective shear strengths of normally consolidated clay soils were computed using the correlations included in SCDOT GDM Section 7.11.2.

6.3 Temporary Excavation Support

Several of the new roadway alignments are in close proximity to existing roadway alignments. The interaction and proximity of new alignments to old alignments has not been fully reviewed at this time and will be incorporated into future report revisions.

6.4 Embankment Modifications

Several of the new roadway alignments require modifying existing embankments. Several of these modifications include the widening of existing earthen embankments. When fill is placed on the side of an existing embankment, proper benching must be obtained to limit the potential for a preferential failure surface. The benching must be wide enough to allow for appropriate compaction equipment. Special care must be taken during construction to ensure that temporary benching does not destabilize existing slopes. We recommend a minimum bench width of 8 feet, and a maximum height of 3 feet. Benching applies to all embankments widening, including sliver fills of a few feet.

Where the project plans call for sliver cuts (i.e. removal of a few feet of soil from the slope face) the cut should start at the crest of the slope and proceed downward to limit the potential for destabilizing the slope.

7 NOTES ON PLANS

The following notes apply to borrow materials:

Provide borrow materials meeting the following minimum requirements:

- **A sandy material (35% or less passing 0.075 mm) with a minimum total soil unit weight, γ_{total} of 110 pcf, with a maximum dry density exceeding 100 pcf.**
- **Minimum friction angle, φ, of 30° and cohesion, c, of 50 psf for embankment fill.**
- **No. 57 Stone backfill for Mechanically Stabilized Earth Walls**

Walls 32 and 36 will require an embankment fill with a minimum friction angle, φ, of 32° and cohesion, c, of 50 psf. This requirement is between alignment Ramp 4, Stations 50+50 and 52+50.

In addition, determine the moisture-density relationship and classification of the material. Test and submit the classification, moisture-density relationship, and soil strength parameters of the material to the Engineer for acceptance. An AASHTO certified laboratory is required to perform the testing. Contact the RPG Geotechnical Engineer for a list of locally available AASHTO certified laboratories. The Department may perform independent testing to assure quality.

Determine the friction angle and cohesion using either direct shear testing or consolidated-undrained triaxial shear testing with pore pressure measurements. Direct Shear testing shall only be performed on soils with a fines content of less than 25 percent. Classification testing includes grain-size distribution with wash #200 sieve, moisture plasticity testing and natural moisture content. Use the Standard Proctor test to determine the moisture-density relationship. Remold all samples used in shear strength testing to 95 percent of the Standard Proctor density. Conduct shear strength testing at the initial selection of the borrow pit, any subsequent changes in borrow pits, and for every 10,000 cy of materials placed. Perform classification testing for every 50,000 cy of materials placed, including the material used for the shear strength testing. Additional shear testing may be required if, in the opinion of the RCE, the materials being placed are different from those originally tested.

If these minimum criteria cannot be met, provide the soil parameters for the intended borrow excavation material for the project site to the Engineer for review and acceptance. After acceptable borrow material is obtained, compact the fill to the required finish grade line using the compactive effort indicated in the Standard Specifications for Highway Construction, Section 205 (Embankment Construction).

The following notes apply to pre-drilling:

Partially Weathered Rock (PWR) is anticipated above the pile tip elevation as several pile and panel wall locations. The contractor should be prepared to predrill should pile refusal occur during pile driving operations. Pre-drilling is anticipated at the following locations:

-Wall 12 from Station 51+00 to 55+00
-Wall 27
-Wall 38 from Station 438+30 to 446+69

The following notes apply to muck excavation:

Any areas identified on the plans and any additional areas that are discovered to deflect or settle may require corrective action as directed by the RCE. This may include undercutting, placing No. 57 stone aggregate that is separated from other borrow materials by a geotextile for separation of sub-grade and sub-base, and/or additional compactive effort to the approval of the RCE.

In areas that require mucking or undercutting, borrow material soil may be placed as a bridge lift as long as the grade on which the material is being placed is at least 2 feet above ground water level. In the event that groundwater does not allow backfilling with a borrow material soil, use a No. 57 stone as the bridge lift material. Borrow material bridge lifts may not exceed a 2-foot thickness. The depth at which mucking or undercutting is required is dependent upon encountering a suitable bearing material within the excavation or if a predetermined elevation or depth is required. In most cases, do not undercut more than 3 to 5 feet. The RCE will determine the final mucking or undercutting thickness, unless otherwise specified in the project plans and/or specifications. If a suitable bearing soil is not encountered within this depth range, place a P1 biaxial geogrid with an aperture size of less than or equal to 1 inch and in accordance with the project special provisions beneath a 2-foot thick bridge lift of No. 57 stone. If additional compacted borrow material soil is needed to reach grade, place a geotextile for separation of sub-grade and sub-base between the No. 57 stone and the overlying compacted soil. A bridge lift consisting of borrow material soil may not be placed within 3 feet of the base of the pavement section. Place only compacted borrow material soil or No. 57 stone within this zone. Reference the Standard Specifications for Highway Construction, Earthwork Section, Division 200.

The following notes apply for MSE Wall Subgrades:

Prior to construction of the leveling pad and MSE fill, the RCE shall verify that the retaining wall is founded on subgrade materials possessing the minimum allowable bearing capacity noted on wall plan and elevation sheets. If the RCE determines that the subgrade is unacceptable for placement of MSE fill, the contractor shall undercut the subgrade to the limits directed by the RCE. Unacceptable subgrade materials include, but are not limited to, all high plasticity clays and elastic silts (CH, MH), low plasticity clays and silts (CL, ML) with an unconfined compressive strength less than 2,000 psf, and deleterious debris. Replacement of undercut material will be with Backfill Material, meeting requirements outlined in the SCDOT Standard Specifications for Highway Construction.

The foundation area for the MSE walls might have scattered pockets of soft soils that might be present at the surface or just below the surface for the base of the MSE fill. These soft pockets are only expected to extend a few feet below the base of the MSE fill. The quality assurance representative shall proofroll the subgrade in this area and/or conduct dynamic cone tests at regular intervals to determine that the subgrade meets the requirements of the paragraph above.

There are several locations along the roadway alignment where proposed drainage structures are situated in front of (i.e. parallel) MSE walls, or where new and existing draining structures pass beneath the MSE walls. Where new pipes are parallel to the

proposed wall, the pipe should be installed prior to the proposed wall or the wall design should account for the temporary reduction in passive resistance. Where pipes pass beneath walls, the pipes should be designed to account for the increased loading associated with the wall backfill. We recommend the top of each pipe be situated a minimum of 1 foot below the bottom of retaining.

The following notes apply for settlement and displacement monitoring:

The contractor shall establish a monitoring program consisting of settlement instruments. The settlement monitoring program must include establishing settlement monitoring instruments on the subgrade soils prior to fill placement, and at design pavement subgrade elevation. Settlement monitoring instruments are required at a spacing of every 100 feet along MSE Walls and every 500 feet along embankments with new fill thicknesses exceeding 20 feet. Instruments shall be established at the centerline of road and edge of pavement. Settlement monitoring shall continue until three consecutive measurements demonstrate the rate of settlement is less than 0.1 inches per year. No more than one measurement shall be obtained on a single day.

A minimum of 2 measurements shall be obtained on monuments prior to fill placement, and instruments shall be measured weekly during fill placement. Instrumentation measurements shall be provided to the Geotechnical Engineer within 24 hours of measurements for interpretation. Interpreted results shall be provided to the RCE.

The following notes apply to slope construction:

Where the new fill meets the existing slope, the existing slope shall be benched to limit the potential for a preferential failure surface and to allow compaction at the interface. Benches shall have a minimum horizontal length of 8 feet and a vertical rise of no more than 3 feet. Fill slopes of 2H:1V or steeper shall be overbuilt (i.e. fill should temporarily extend beyond the final slope face) to allow compaction at the slope face. After compaction is complete, the slope may be regraded to the final inclination.

Should seeps or thick lenses of highly plastic soils be observed in the planned fill and cut slopes that are steeper than 2H:1V, ECS must be contacted to determine if the steeper slopes may be constructed as planned or if slope flattening or reinforcing is required. Similarly, if soft or wet ground conditions are observed at the base of planned fill embankments, the QA representative must determine the limits of undercutting required or required in-situ treatment.

The following Plan Notes apply to Mechanically Stabilized Earth walls:

Reinforced Backfill (No. 57 stone for Walls Greater than 30 ft, Granular Fill for All other Walls)

No. 57 stone and granular fill may be used on walls with heights both greater than and less than 30 feet. A layer of non-woven geotextile separation fabric shall be placed at the interface between granular fill and No. 57 stone where both materials are used in a single wall.Where granular fill is placed adjacent to No 57 stone.

Several MSE Walls are Back-To-Back walls. For back-to-back walls where the reinforcement overlaps more than 0.3 times the height of the shorter wall, reinforcement

length may be reduced such that there is a minimum of 5 feet of strap overlap and a minimum of 0.6 times the height of the respective wall.

Internal Friction Angle (deg) = 36
Total Unit Weight = 105 pcf
Surcharge Dead Load for Pavement Overlay = 140 psf
Active Earth Pressure Coefficient = 0.26

Wall 12 and 13 – Ramp 1A 63+00 to 71+29.42
Foundation Soils
Total – Internal Friction Angle (deg) = 30
Total – Cohesion = 0 psf
Effective – Internal Friction Angle (deg) = 30
Effective – Cohesion = 0 psf

Wall Height	Min. Breq	Factored Bearing (Static)	Factored Bearing (Seismic)
39 < H ≤ 41.5	29.5 ft	12,600	19,300
29 < H ≤ 39	27.5 ft	11,600	17,900
21 < H ≤ 29	20.5 ft	9,700	15,000
15 < H ≤ 21	16 ft	8,900	13,600
9 < H ≤ 15	12 ft	7,500	11,500
H ≤ 9	9 ft	6,200	9,600

Wall 1 – Ramp 1B 72+83 to 76+60.32
Foundation Soils
Total – Internal Friction Angle (deg) = 32
Total – Cohesion = 0 psf
Effective – Internal Friction Angle (deg) = 32
Effective – Cohesion = 0 psf

Wall Height	Min. Breq	Factored Bearing (Static)	Factored Bearing (Seismic)
15 < H ≤ 21.5	16 ft	10,700	16,500
9 < H ≤ 15	12 ft	9,200	14,100
H ≤ 9	9 ft	7,600	11,700

Wall 33 – Ramp 2B-I-385 SB C/D 40+37.82 to 99+05.20
Foundation Soils
Total – Internal Friction Angle (deg) = 30
Total – Cohesion = 0 psf
Effective – Internal Friction Angle (deg) = 30
Effective – Cohesion = 0 psf

Wall Height	Min. Breq	Factored Bearing (Static)	Factored Bearing (Seismic)
58 < H ≤ 68	58 ft	14,900	23,000
49 < H ≤ 58	49.5 ft	12,700	19,500
39 < H ≤ 49	42.5 ft	10,900	16,800
29 < H ≤ 39	35 ft	9,100	14,000
21 < H ≤ 29	25.5 ft	7,500	11,600
15 < H ≤ 21	18.5 ft	5,400	8,300
9 < H ≤ 15	13 ft	4,200	6,500
H ≤ 9	9 ft	3,400	5,300

Wall 10 – Ramp 3A Sta. 292+63.17 to 296+94.27
Foundation Soils
Total – Internal Friction Angle (deg) = 26
Total – Cohesion = 0 psf
Effective – Internal Friction Angle (deg) = 26
Effective – Cohesion = 0 psf

Wall Height	Min. Breq	Factored Bearing (Static)	Factored Bearing (Seismic)
21 < H ≤ 27	21 ft	6,800	10,500
15 < H ≤ 21	17 ft	5,500	8,600
9 < H ≤ 15	13 ft	4,300	6,600
H ≤ 9	9.5 ft	3,300	5,200

Wall 32 – I-385 NB C/D-Ramp 4 Sta. 379+53.08 to 52+36.35
Foundation Soils
Total – Internal Friction Angle (deg) = 30
Total – Cohesion = 0 psf
Effective – Internal Friction Angle (deg) = 30
Effective – Cohesion = 0 psf

Wall Height	Min. Breq	Factored Bearing (Static)	Factored Bearing (Seismic)
49 < H ≤ 56	49 ft	17,300	26,600
39 < H ≤ 49	49 ft	18,200	28,000
29 < H ≤ 39	30 ft	9,500	14,600
21 < H ≤ 29	23 ft	7,400	11,400
15 < H ≤ 21	17 ft	5,500	8,400
9 < H ≤ 15	12.5 ft	4,200	6,500
H ≤ 9	9 ft	3,100	4,800

Wall 36B – Ramp 4 50+68.09 to 54+18.80
Foundation Soils
Total – Internal Friction Angle (deg) = 26
Total – Cohesion = 0 psf
Effective – Internal Friction Angle (deg) = 26
Effective – Cohesion = 0 psf

Wall Height	Min. Breq	Factored Bearing (Static)	Factored Bearing (Seismic)
15 < H ≤ 23.5	16 ft	10,700	16,500
9 < H ≤ 15	12 ft	9,200	14,100
H ≤ 9	9 ft	7,600	11,700

Wall 2A – I-85 278+91.65 to 282+30.89
Foundation Soils
Total – Internal Friction Angle (deg) = 26
Total – Cohesion = 0 psf
Effective – Internal Friction Angle (deg) = 26
Effective – Cohesion = 0 psf

Wall Height	Min. Breq	Factored Bearing (Static)	Factored Bearing (Seismic)
9 < H ≤ 17	13 ft	8,100	12,500
H ≤ 9	8 ft	5,300	8,100

Wall 31B – Ramp 4B 382+12.55 to 384+67.14
Foundation Soils
Total – Internal Friction Angle (deg) = 30
Total – Cohesion = 0 psf
Effective – Internal Friction Angle (deg) = 30
Effective – Cohesion = 0 psf

Wall Height	Min. Breq	Factored Bearing (Static)	Factored Bearing (Seismic)
H ≤ 11	11 ft	5,500	8,500

The following notes apply to Reinforced Soil Slopes:

Reinforced Backfill:
Internal Friction Angle (deg) = 34
Total Unit Weight = 120 pcf

Standing water may be present in some areas of the proposed reinforced soil slope. If standing water is present, a bridge lift shall be placed consisting of No. 57 stone.

Any guardrail post driven into the reinforced zone of the reinforced soil slope shall be installed with a wedge-shaped shoe.

All embankment fill material used shall meet the requirement of reinforced backfill soil as provided in the Special Provisions.

Reinforced backfill soil shall have an internal angle of friction of no less than 34 degrees and a total unit weight of no less than 120 pounds per cubic foot. See the Special Provisions for additional soil requirements and measurement and method of payment.

The following geotechnical notes are recommended for the reinforced concrete wall:

Reinforced Concrete Wall Notes:

Specifications:
AASHTO 2012 LRFD Bridge Design Specifications, 6th Edition,
With Interim Revisions through 2013

Design Data:
Load and Resistance Factor Design (LRFD) Method

Live Load: AASHTO HL-93
Live Load Surcharge: 240 psf
Vehicular Collision: TL-4

Active Earth Pressure Coefficient, K: 0.33
Seismic Earth Pressure Coefficient, K_{ae}: 0.47

Factored Net Bearing: Static = 6400 psf; Seismic = 8600 psf

Backfill material:
Stone or Granular Backfill:
Internal Friction Angle (deg) = 30

Total Unit Weight = 120 pcf

Foundation Soils:
Total – Internal Friction Angle (deg) = 28
Total – Cohesion = 0 psf
Effective – Internal Friction Angle (deg) = 28
Effective – Cohesion = 0 psf

Extreme Event I Limit States:

Design EQ	PGA	S_{DS}	S_{D1}
FEE	0.07	0.11	0.06
SEE	0.20	0.29	0.14

1. **Project Location and Site Class**
   - **Latitude: 34.8239**
   - **Longitude: -82.2964**
   - **Site Class: D**
2. **Design Earthquake**
   - **Functional Evaluation Earthquake (FEE) 15% Probability of Exceedance in 75 years**
   - **Safety Evaluation Earthquake (SEE) 3% Probability of Exceedance in 75 years**

8 PAVEMENT DESIGN

The RFP dated March 28, 2014, required that the pavement design be based on Exhibit 4E. Based on Exhibit 4E, the typical pavement section was incorporated in the project drawings.

Temporary pavements are anticipated along various portions of the project. Temporary pavement design was coordinated with the roadway designer.

APPENDICES

Appendix A – Site Vicinity Map and Boring Location Diagram

Appendix B – Interchange Boring Summary

Appendix C – I-85/385 Interchange Boring Logs

Appendix D -- Advanced Laboratory Test Data

Appendix E – Seismic Data (From SCDOT)

Appendix F – Liquefaction and Shear Strength Loss Triggering Evaluation

Appendix G – Summary of Measured and Corrected N-Values

Appendix H – Summary of Shear Strength Parameters

Appendix I – Summary of Settlement Parameters

Appendix J – Summary of Cut Wall Design Parameters

Appendix K – Global Stability Analysis

Appendix L – Mechanically Stabilized Earth Wall External Stability Calculations

Appendix M – Settlement Calculations

Appendix N – ECS Laboratory Test Results

Appendix O – ECS Hammer Calibrations

Appendix P – Thompson Engineering Geotechnical Data Report and Addenda

Appendix Q – F&H Geotechnical Data Report

Appendix R – Soil Nail Wall Special Provision

Appendix S – Cut Wall Design Memorandum and Comments

Appendix T – Cut Walls – Ramp 1A from Station 51+00 to 52+00