Chapter 9

GEOTECHNICAL RESISTANCE FACTORS

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

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CHAPTER 9

GEOTECHNICAL RESISTANCE FACTORS

9.1 INTRODUCTION

As described in Chapter 8, Resistance Factors (φ) are used in LRFD design to account for the variability associated with the resistance side of the basic LRFD Equation.

$$\boldsymbol{Q} \leq \varphi\, \boldsymbol{R_n} = \boldsymbol{R_r} \qquad \text{Equation 9-1}$$

Where,

Q	=	Factored Load
R_r	=	Factored Resistance
R_n	=	Nominal Resistance (i.e. ultimate capacity)
φ	=	Resistance Factor

AASHTO and FHWA have conducted studies to develop geotechnical Resistance Factors (φ) based on reliability theory that account for the uncertainties presented below:

- Accuracy of Prediction Models (Design Methodology)
- Site Characterization
- Reliability of material property measurements
- Material properties relative to location, direction, and time
- Material Resistance
- Sufficiency and applicability of sampling
- Soil Behavior
- Construction Effects on Designs

When insufficient statistical data was available, the studies performed a back-analysis of the geotechnical designs to obtain a resistance factor that maintains the current level of reliability that is inferred by the ASD design methodology using the appropriate Factors of Safety.

The LRFD geotechnical design philosophy and load factors for geotechnical engineering are provided in Chapter 8. The Performance Limits for the Service and Extreme Event limit states are provided in Chapter 10. The design methodology used in the application of the design criteria (load factors, resistance factors, and performance limits) is based on AASHTO design methodology with modifications/deviations as indicated in the following Chapters of this Manual:

- Chapter 12 – Earthquake Engineering
- Chapter 13 – Geotechnical Seismic Hazards
- Chapter 14 – Geotechnical Seismic Design
- Chapter 15 – Shallow Foundations
- Chapter 16 – Deep Foundations
- Chapter 17 – Stability and Settlement Analysis and Design
- Chapter 18 – Earth Retaining Structures
- Chapter 19 – Ground Improvement
- Chapter 20 – Geosynthetic Design
- Appendix C – MSE Walls

9.2 SOIL PROPERTIES

The geotechnical Resistance Factors (φ) provided in this Chapter are only appropriate when soil material properties are based on sampling/testing frequency, and testing methods as defined in this Manual. Geotechnical designs and/or analyses should be performed after establishing a “site” based on the site variability with respect to the soil properties that most affect the design or geotechnical analysis. A site variability of “Medium” or lower should be selected based on the requirements of Chapter 7.

Engineering judgment is important in the selection of soil properties but must be used judiciously in a manner that is consistent with the method used to develop the resistance factors and should not be used as a method to account for insufficient geotechnical information due to an inadequate subsurface investigation. As indicated above, the AASHTO resistance factors were developed by either reliability theory or by ASD back-calculation. LRFD resistance factors that were based on reliability theory were developed based on using “average” soil shear properties for each identified geologic unit. LRFD resistance factors that were developed based on a back-analysis of ASD design methodology should use the same method of selecting soil properties (lower bound, average, etc.) as previously used in ASD design. For further information into how the resistance factors were developed the AASHTO LRFD specifications and supporting reference documents should be consulted.

When sufficient subsurface information is available, soil properties should be rationally selected and substantiated by the use of statistical analyses of the geotechnical data. To arbitrarily select conservative soil properties may invalidate the assumptions made in the development of LRFD resistance factors by accounting for uncertainties multiple times; therefore, producing geotechnical designs that are more conservative and consequently have higher costs than the ASD design methodology previously used. When limited amount of subsurface information is available or the subsurface information is highly variable, it may not be possible to select an average soil property for design and a conservative selection of soil properties may be required so as to reduce the risk of poor performance of the structure being designed. Satisfactory performance of the structure outweighs any cost savings that may result from the use of less conservative soil properties.

9.3 RESISTANCE FACTORS FOR LRFD GEOTECHNICAL DESIGN

The geotechnical Resistance Factors (φ) that are provided are distinguished by type of geotechnical structure being designed as listed below.

- Deep Foundations
- Shallow Foundations
- Earth Retaining Structures
- Embankments
- Reinforced Earth Internal Stability

Resistance factors for the determination of liquefaction induced geotechnical earthquake hazards are also provided.

As indicated in Chapter 8, the Fatigue limit state is the only limit state that is not used in geotechnical analyses or designs. Geotechnical resistance factors are provided for the following limit state load combinations:

- Strength – This includes Strength I, II, III, IV, and V.
- Service – This includes Service I
- Extreme Event – This includes Extreme Event I (Seismic loadings) and Extreme Event II (Collision loadings)

Resistance factors are provided based on the type of analysis being performed and the method of determination. When resistance factors are not applicable to the limit state the term “N/A” has been used in the resistance factor tables included in this Chapter. The method of determination shall either be based on the method of construction control or the analytical method used in the design. For details of the analytical methods used in the design see the appropriate chapters in this Manual.

Some analytical methods have not been calibrated for LRFD design methodology. Geotechnical analyses that have not been calibrated include, global stability analyses (static and seismic), and liquefaction induced geotechnical earthquake hazards. For these analyses a load factor (γ) of unity (1.0) should be used. The resistance factors (φ) provided for these analyses is the inverse of the Factor of Safety (1/FS) and consequently have the same margin of safety as previously used in ASD designs. For global stability, Equation 9-1 can be written as indicated below.

$$\frac{R_n}{Q} = \frac{\text{Resisting Force}}{\text{Driving Force}} = FS \geq \frac{1}{\varphi} \qquad \text{Equation 9-2}$$

Where,

R_n	=	Nominal Resistance (i.e. ultimate capacity)
Q	=	Factored Load (With load factor, γ = 1.0)
FS	=	Factor of Safety
φ	=	Resistance Factor

The geotechnical Resistance Factors (φ) provided in this Chapter have been selected by the SCDOT based on the standard-of-practice that is presented in this Manual, South Carolina geology, and local experience. Although statistical data combined with calibration have not been used to select regionally specific geotechnical resistance factors, the resistance factors presented in AASHTO and FHWA publications have been adjusted based on substantial successful experience to justify these values. The AASHTO LRFD specifications should be consulted for any geotechnical resistance factors not provided in this Chapter. The PCS/GDS shall review the AASHTO LRFD geotechnical resistance factors that are not included in this Manual prior to approval.

9.4 SHALLOW FOUNDATIONS

Geotechnical Resistance Factors (φ) for shallow foundations have been modified slightly from those specified in the AASHTO LRFD specifications by varying resistance factors based on the structure operational classification (OC or ROC). Resistance factors for shallow foundations are shown in Table 9-1. Resistance factors for bearing resistance are specified for soil and rock. Resistance factors for sliding are based on the materials at the sliding interface.

Table 9-1, Resistance Factors for Shallow Foundations

Performance Limit		Limit States		
		Strength	**Service**	**Extreme Event**
Soil Bearing Resistance (Soil)	OC= I, II, III; ROC = I	0.40	N/A	0.60
	ROC = II or III	0.45		0.65
Soil Bearing Resistance (Rock)	OC= I, II, III; ROC = I	0.40	N/A	0.60
	ROC = II or III	0.45		0.65
Sliding Frictional Resistance (Cast-in-place Concrete on Sand)	OC= I, II, III; ROC = I	0.70	N/A	0.90
	ROC = II or III	0.80		0.95
Sliding Frictional Resistance (Cast-in-place Concrete on Clay)	OC= I, II, III; ROC = I	0.75	N/A	0.90
	ROC = II or III	0.85		0.95
Sliding Frictional Resistance (Precast Concrete on Sand)	OC= I, II, III; ROC = I	0.80	N/A	0.95
	ROC = II or III	0.90		1.00
Sliding Soil on Soil	OC= I, II, III; ROC = I	0.80	N/A	0.70
	ROC = II or III	0.90		0.80
Sliding Passive Resistance (Soil)	OC= I, II, III; ROC = I	0.40	N/A	0.55
	ROC = II or III	0.50		0.65
Lateral Displacement		N/A	1.00	1.00
Vertical Settlement		N/A	1.00	1.00

9.5 DEEP FOUNDATIONS

The design of deep foundations requires that foundations supporting bridge piers or abutments consider all limit state loading conditions applicable to the structure being designed. SCDOT has deviated in its application of LRFD design of deep foundations as presented in the AASHTO LRFD specifications. The deviations are a result of current design and construction practice, design policies, and experience obtained evaluating field load tests of driven piles and drilled shafts. The resistance factors used to determine the nominal resistance for single piles or drilled shafts in axial compression or uplift shall be based on the method of deep foundation load capacity verification during construction. The foundation capacity verification will typically be conducted at Test Pile (non-production piles) locations or at Index Pile (production pile) locations. Foundation capacity verification may be required at any foundation that does not meet foundation installation criteria or whose load carrying capacity is in question. A description of deep foundation load capacity verification methods (wave equation, static load testing, Osterberg cell, dynamic testing, and Statnamic testing) are presented in Chapters 16 and 24. All other resistance factors are based on the design methodology used for deep foundations presented in Chapter 16. The frequency of deep foundation load capacity verification is dependent on the Site Variability as defined in Chapter 7.

The Statnamic load testing method has been included as a method of verifying pile capacity due to its regional popularity and its economic advantages. Statnamic is a relatively new load testing method compared to static load testing or dynamic testing and has yet to be included in the AASHTO specifications. Statnamic load testing is regarded as a load testing method that purportedly falls between a static load test and a dynamic load test. The load applied to the top of the foundation is applied dynamically although at a much slower rate as compared to dynamic testing (PDA). The analysis of the Statnamic load test data requires that the dynamic resistance from the soil be subtracted from the total load applied to obtain the static resistance. Regional experience using Statnamic load testing has shown that dynamic resistance is greater for friction piles/drilled shafts in cohesive soils and consequently the reliability of this method is less for this type of foundation. For friction piles/drilled shafts in cohesionless soils or end-bearing piles/drilled shafts on rock, Intermediate Geomaterial (IGM) or dense sands the dynamic resistance is less and therefore the reliability of the Statnamic load testing method is better when compared to Statnamic load testing of friction piles/drilled shafts in cohesive soils. The method used to separate the dynamic resistance from the static resistance has not been nationally accepted (AASHTO) and the method's reliability has not been independently verified.

SCDOT has conservatively assigned resistance factors for Statnamic load testing based on the limited regional practice. Since cohesive soils tend to produce higher dynamic resistances as compared to cohesionless soils, a lower reliability has been assumed for friction piles/drilled shafts installed in cohesive soils. No increases in resistance factors will be allowed when performing multiple Statnamic tests within a "Site" as indicated in Table 9-4. In order to increase the resistance factors indicated in this Section, a full-scale static load test per "Site" will be required to calibrate the Statnamic load test method of analysis, with the approval of the PCS/GDS. The term "Site" is defined as indicated in Chapter 7.

Another very widely accepted method to verify the axial load capacity of deep foundations is the use of the Osterberg Cell. Since the Osterberg Cell is a type of static load test, the resistance factor for Osterberg Cell load testing method shall be the same as for conventional static load tests indicated in Tables 9-2 and 9-5.

9.5.1 Driven Piles

AASHTO specifications for driven piles differentiate between the predicted nominal axial capacities ($R_{nstatic}$) based on static analyses and the field verified pile capacities (R_n) by applying different geotechnical Resistance Factors (φ) for each of these axial capacities. Upon review of the AASHTO recommended geotechnical Resistance Factors (φ_{stat}) for the static capacity prediction, it was observed that the AASHTO geotechnical Resistance Factors (φ_{stat}) inherently presume a substantial amount of uncertainty in the predicted nominal axial capacity with respect to the field verified pile capacity using either dynamic formula, dynamic analysis, or static load tests. This presumption of greater uncertainty of predicted values vs. field verified values is logical and has merit for a national specification but it does not take into account the regional experience of predicting pile capacities. SCDOT has observed that when using the nominal axial compression pile capacity design methods presented in this Manual that there is rarely a need to extend the pile lengths in the field because the required pile capacity is achieved during pile driving. Driven piles are typically installed in cohesionless soils where pile resistance is most likely underpredicted. The predictive pile capacity method for driven piles installed in the

Cooper Marl has been developed based on pile load tests. It has been observed that the pile capacity methods predict fairly accurately when pile capacity verification is made using pile re-strikes with the Pile Driving Analyzer (PDA). Typically, pile lengths provided in the plans have sufficient length to achieve the required ultimate pile capacity at the end-of-driving or re-strikes when verified by wave equation, dynamic load testing (PDA), or static load tests.

SCDOT has elected to use resistance factors (φ) based on the construction pile capacity verification method required in the plans to predict the nominal axial capacities (static determination of ultimate pile capacity) during design, which is used to select number of piles and pile plan lengths.

Additional considerations that have gone into the selection of SCDOT geotechnical resistance factors are as follows:

- The definition of a “Site” is the same as presented in the AASHTO LRFD specifications with the exception that a “Site” can not have a variability greater than “Medium”. If a “Site” classifies as a “High” variability, the “Site” shall be reduced in size to maintain a variability of “Low” or “Medium.” The Site Variability shall be determined as indicated in Chapter 7.
- Resistance factors are based on a Site Variability of “Low” or “Medium”
- When field load testing is used, a minimum of one test pile is required per “Site” and it is typically placed at the weakest location based on the subsurface soil investigation and design methodology.
- The Contractor’s pile installation plan is reviewed by SCDOT and the pile driving installation equipment is evaluated using the Wave Equation
- Wave Equation Analysis is used to verify the field pile capacity during pile driving. The Wave Equation is calibrated using signal matching (CAPWAP) with the dynamic testing results.
- When load tests are performed, the test pile installation is monitored with the Pile Driving Analyzer (PDA).
- All bridges, regardless of their bridge Operational Classifications (OC), will be designed using the same geotechnical Resistance Factors to maintain the same level of variability.

Load modifiers presented in Chapter 8 are not used to account for the influence of redundancy in geotechnical foundation design. Redundancy in deep foundation design is taken into account by the selection of the geotechnical resistance factor. Non-redundant pile foundations are those pile footings with less than five piles supporting a single column, or less than five piles in a pile bent. Pile footings or pile bents with more than four piles are classified as redundant driven pile foundations.

A resistance factor of 1.0 should be used for soils encountered in scour zones or zones neglected in design when performing pile driveability evaluations or when determining the nominal axial compression capacity to be verified during driving. A resistance factor 10 percent greater than that shown in Table 9-2 can be used for the pile tested, but shall not exceed a resistance factor of 0.80.

Table 9-2, Geotechnical Resistance Factors for Driven Piles

Analysis and Method of Determination	Limit States			
	Strength		Service	Extreme Event
	Redundant	Non-Redundant		
Nominal Resistance Single Pile in Axial Compression with Wave Equation [(1)]	0.40	0.30	N/A	1.00
Nominal Resistance Single Pile in Axial Compression with Dynamic Testing (PDA) and calibrated Wave Equation [(2)]	0.65	0.55	N/A	1.00
Nominal Resistance Single Pile in Axial Compression with Static Load Testing. Dynamic Monitoring (PDA) of test pile installation and calibrated Wave Equation [(2,3)].	See Table 9-4		N/A	1.00
Nominal Resistance Single Pile in Axial Compression with Statnamic Load Testing For Friction Piles. Dynamic Monitoring (PDA) of test pile installation and calibrated Wave Equation [(2)]	0.65	0.55	N/A	1.00
Nominal Resistance Single Pile in Axial Compression with Statnamic Load Testing For End Bearing Piles in Rock, IGM, or Very Dense Sand. Dynamic Monitoring (PDA) of test pile installation and calibrated Wave Equation [(2)].	0.70	0.55	N/A	1.00
Pile Group Block Failure (Clay)	0.60	N/A	N/A	1.00
Nominal Resistance Single Pile in Axial Uplift Load with No Verification	0.35	0.25	N/A	0.80
Nominal Resistance Single Pile in Axial Uplift Load with Static Load Testing	0.60	0.50	N/A	0.80
Group Uplift Resistance	0.50	N/A	N/A	N/A
Single or Group Pile Lateral Load Geotechnical Analysis (Lateral Displacements)	N/A	N/A	1.00	1.00
Single or Group Pile Vertical Settlement	N/A	N/A	1.00	1.00
Pile Drivability – Geotechnical Analysis	1.00	1.00	N/A	N/A

[(1)] Applies only to factored loads less than or equal to 600 kips.
[(2)] See Table 9-3 for frequency of dynamic testing required.
[(3)] See Table 9-4 for number of static load testing required.

Dynamic testing is used to control the construction of pile foundations by verifying pile capacity (signal matching required - CAPWAP), calibrating wave equation inspector charts based on signal matching, and monitoring the pile driving hammer performance throughout the project.

In order to use the resistance factors indicated in Table 9-2, a minimum number of Index/Test piles with dynamic testing and signal matching as indicated in Table 9-3 will be required per "Site". The dynamic testing should be evenly distributed within a "Site". The test pile locations or bent locations where index piles will be monitored with dynamic testing should be indicated in the plans.

Table 9-3, Test/Index Piles with Dynamic Testing

Number of Driven Piles Located Within a Site	Number of Test/Index Piles Requiring Dynamic Testing and Signal Matching Analysis	
	Site Variability	
	Low	Medium
≤ 15	3	4
16 – 25	3	5
26 – 50	4	6
51 – 200	4	7
> 200	5	8

All test piles and index piles will require dynamic testing to monitor pile installation. Include additional dynamic testing if restrikes are required for test piles or index piles.

For bridges with 200 or less piles a minimum of 5 additional dynamic tests should be included in the contract to allow for evaluation of poor or highly variable hammer performance or pile restrikes to verify pile capacity throughout the "Site". For bridges with more than 200 piles a minimum 3.0% for "Sites" with "Low" variability or 6.0% for "Sites" with "Medium" variability should be included in the contract to allow for evaluation of poor or highly variable hammer performance or pile restrikes to verify pile capacity throughout the project. The additional dynamic testing of production piles shall be used uniformly throughout the "Site" for QC of the Contractor's pile driving operations.

Table 9-4, Number of Static Load Tests per Site

Number of Static Load Tests per Site	Resistance Factor (φ)			
	Low Site Variability		Medium Site Variability	
	Redundant	Non-Redundant	Redundant	Non-Redundant
1	0.80	0.65	0.70	0.55
2	0.90	0.70	0.75	0.60
3 or more	0.90	0.70	0.85	0.70

9.5.2 Drilled Shafts

Drilled shaft geotechnical resistance factors (φ) have been provided in Table 9-5. Load resistance factors are provided for Clay, Sand, Rock, and IGM. Statnamic load testing has also been included as indicated in Section 9.5.

Additional considerations that have gone into the selection of SCDOT geotechnical resistance factors are as follows:

- The definition of a "Site" is the same as presented in the AASHTO LRFD specifications with the exception that a "Site" can not have a variability greater than "Medium". If a "Site" classifies as a "High" variability, the "Site" shall be reduced in size to maintain a variability of "Low" or "Medium."
- Resistance factors are based on a site variability of "Low" or "Medium."

- When field load testing is used, a minimum of one test shaft is required per “Site” and it is typically placed at the weakest location based on the subsurface soil investigation and design methodology.

As discussed in Chapter 8, load modifiers will not be used to account for the influence of redundancy in geotechnical foundation design. Redundancy in deep foundations is taken into account by the selection of the geotechnical resistance factor. Non-redundant foundations are those drilled shaft footings with four or less drilled shafts supporting a single column or individual drilled shafts supporting individual columns in a bent. Drilled shaft footings with five or more drilled shafts are classified as redundant drilled shaft foundations. If foundation is a hammerhead (one shaft and one column) reduce the non-redundant resistance factor by 20 percent.

Because drilled shaft capacities can not be verified individually during construction (only drilled shaft installation monitoring), a single resistance factor will be provided for both redundant and non-redundant drilled shafts and no increases in resistance factors will be allowed when performing multiple load tests within a “Site” as indicated in Table 9-4. A resistance factor 10 percent greater than that shown in Table 9-5 can be used for the drilled shaft tested, but shall not exceed a resistance factor of 0.80.

Table 9-5, Resistance Factor for Drilled Shafts

Performance Limit		Limit States			
		Strength		Service	Extreme Event
		Redundant	Non-Redundant (1)		
Nominal Resistance Single Drilled Shaft in Axial Compression in Clay	Side	0.55	0.45	N/A	1.00
	Tip	0.50	0.40	N/A	1.00
Nominal Resistance Single Drilled Shaft in Axial Compression in Sand	Side	0.65	0.55	N/A	1.00
	Tip	0.60	0.50	N/A	1.00
Nominal Resistance Single Drilled Shaft in Axial Compression in IGM	Side	0.70	0.60	N/A	1.00
	Tip	0.65	0.55	N/A	1.00
Nominal Resistance Single Drilled Shaft in Axial Compression in Rock	Side	0.60	0.50	N/A	1.00
	Tip	0.60	0.50	N/A	1.00
Nominal Resistance Single Drilled Shaft in Axial Compression with Static Load Testing		0.70	0.70	N/A	1.00
Nominal Resistance Single Drilled Shaft in Axial Compression with Statnamic Load Testing.		0.65	0.65	N/A	1.00
Nominal Resistance Single Drilled Shaft in Axial Uplift Load (Side Resistance)	Clay	0.45	0.35	N/A	1.00
	Sand	0.55	0.45	N/A	1.00
	IGM	0.55	0.45	N/A	1.00
	Rock	0.50	0.40	N/A	1.00
Nominal Resistance Single Drilled Shaft in Axial Uplift with Static Load Testing		0.60	0.60	N/A	1.00
Drilled Shaft Group Block Failure (Clay)		0.55	N/A	N/A	1.00
Drilled Shaft Group Uplift Resistance		0.45	N/A	N/A	1.00
Single or Group Drilled Shaft Lateral Load Geotechnical Analysis (Structural Capacity)		N/A	N/A	1.00	1.00
Single or Group Drilled Shaft Lateral Load Geotechnical Analysis (Lateral Displacements)		N/A	N/A	1.00	1.00
Single or Group Drilled Shaft Vertical Settlement		N/A	N/A	1.00	1.00

(1) If foundation is a hammerhead (one shaft and one column) reduce the non-redundant resistance factor by 20 percent.

9.6 EARTH RETAINING STRUCTURES

Geotechnical Resistance Factors (φ) for earth retaining structures have been modified slightly from those specified in the AASHTO LRFD specifications by varying resistance factors based on retaining wall system type and the Roadway Structure Operational Classification (ROC). Resistance factors are provided for external stability of the structure with respect to bearing, sliding, and passive resistance. Resistance factors for bearing resistance are specified for soil and rock. Resistance factors for sliding are based on the materials at the sliding interface. For resistance factors due to internal stability of Mechanically Stabilized Earth (MSE) walls see Section 9.8. Resistance factors for Rigid Gravity Retaining Walls are provided in Table 9-6, Flexible Gravity Retaining Walls are provided in Table 9-7, and Cantilever Retaining Walls with or without anchors are provided in Table 9-8.

Rigid Gravity Retaining Walls include cast-in-place concrete walls and brick wall standards typically used in roadway projects. Flexible gravity retaining wall systems include bin walls, panel and block face MSE walls. Cantilever walls include sheet pile walls and soldier pile walls.

Table 9-6, Resistance Factors for Rigid Gravity Retaining Walls

Performance Limit		**Limit States**		
		Strength	**Service**	**Extreme Event**
Soil Bearing Resistance (Soil)	ROC = I, II	0.45	N/A	0.60
	ROC = III	0.45	N/A	0.60
Soil Bearing Resistance (Rock)		0.45	N/A	0.60
Sliding Frictional Resistance (Cast-in-place Concrete on Sand)	ROC = I, II	0.70	N/A	0.90
	ROC = III	0.80		0.95
Sliding Frictional Resistance (Cast-in-place Concrete on Clay)	ROC = I, II	0.75	N/A	0.90
	ROC = III	0.85		0.95
Sliding Frictional Resistance (Precast Concrete on Sand)	ROC = I, II	0.80	N/A	0.95
	ROC = III	0.90		1.00
Sliding Soil on Soil	ROC = I, II	0.80	N/A	0.70
	ROC = III	0.90		0.80
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 (1)
	ROC = III		0.75	1.00 (1)
Global Stability Cut Walls	ROC= I, II	N/A	0.60	0.90 (1)
	ROC = III		0.70	1.00 (1)

(1) Global stability analyses for Extreme Event I limit state that have resistance factors greater than specified require a displacement analysis to determine if it meets the performance limits presented in Chapter 10.

Table 9-7, Resistance Factors for Flexible Retaining Walls

Performance Limit		Limit States		
		Strength	Service	Extreme Event
Soil Bearing Resistance (Soil)		0.55	N/A	0.70
Soil Bearing Resistance (Rock)		0.55	N/A	0.70
Sliding Frictional Resistance (Soil on Soil)		0.90	N/A	0.95
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 (1)
	ROC = III		0.75	1.00 (1)
Global Stability Cut Walls	ROC= I, II	N/A	0.60	0.90 (1)
	ROC = III		0.70	1.00 (1)

(1) Global stability analyses for Extreme Event I limit state that have resistance factors greater than specified require a displacement analysis to determine if it meets the performance limits presented in Chapter 10.

Table 9-8, Resistance Factors for Cantilever Retaining Walls

Performance Limit		Limit States		
		Strength	Service	Extreme Event
Axial Compressive Resistance of Vertical Elements		Section 9.4 Applies		
Passive Resistance of Vertical Element		0.75	N/A	0.85
Flexural Capacity of Vertical Element		0.90	N/A	0.90
Tensile Resistance of Anchor (1)	Mild Steel (ASTM 615)	N/A	0.90 (1)	0.90 (1)
	High Strength Steel (ASTM A 722)		0.80 (1)	0.80 (1)
Pullout Resistance of Anchors (2)	Sand and Silts	N/A	0.65 (2)	0.90 (2)
	Clay		0.70 (2)	1.00 (2)
	Rock		0.50 (2)	1.00 (2)
Anchor Pullout Resistance Test (3) (With proof test of every production anchor)		N/A	1.00 (3)	1.00 (3)
Lateral Displacement		N/A	1.00	1.00
Vertical Settlement		N/A	1.00	1.00

(1) Apply to maximum proof test load for the anchor. For mild steel apply resistance factor to F_y. For high-strength steel apply the resistance factor to guaranteed ultimate tensile strength.

(2) Apply to presumptive ultimate unit bond stresses for preliminary design only. See AASHTO LRFD (C11.9.4.2) specifications for additional information.

(3) Apply where proof tests are conducted on every production anchor to load of 1.0 or greater times the factored load on the anchor.

9.7 EMBANKMENTS

Geotechnical Resistance Factors (φ) for embankments have been modified slightly from those specified in the AASHTO LRFD specifications by varying resistance factors based on the Roadway Structure Operational Classification (ROC). Resistance factors for embankments (fill) sections and cut-sections are shown in Table 9-9.

Table 9-9, Resistance Factors for Embankments (Fill / Cut Section)

Performance Limit		Limit States: Strength	Service	Extreme Event
Embankment Soil Bearing Resistance (Soil)		0.55	N/A	0.65
Embankment Soil Bearing Resistance (Rock)		0.55	N/A	0.65
Embankment Sliding Frictional Resistance		0.90	N/A	0.95
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 (1)
	ROC = III		0.75	1.00 (1)
Global Stability Cut Section	ROC= I, II	N/A	0.60	0.90 (1)
	ROC = III		0.70	1.00 (1)

(1) Global stability analyses for Extreme Event I limit state that have resistance factors greater than specified require a displacement analysis to determine if it meets the performance limits presented in Chapter 10.

9.8 REINFORCED SOIL (INTERNAL STABILITY)

Geotechnical Resistance Factors (φ) for analysis of internal stability of reinforced soils are based on AASHTO LRFD specifications. Resistance factors for internal stability of reinforced soils are shown in Table 9-10. Resistance factors may be used in reinforced soil slopes or MSE walls. The external stability of MSE walls shall be governed by the resistance factors provided for flexible walls in Table 9-7. The external stability of Reinforced Steepend Slopes (RSS) shall be governed by the resistance factors provided for embankments in Table 9-9.

Table 9-10, Resistance Factors for Reinforced Soils

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	1.00
Sliding at Soil Reinforcement Interface		0.80	N/A	1.00

(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.

9.9 LIQUEFACTION INDUCED GEOTECHNICAL EARTHQUAKE HAZARDS

Geotechnical Resistance Factors (φ) for shear strength loss (SSL) and SSL-induced geotechnical seismic hazards are provided in Table 9-11. Resistance factors for other earthquake hazards that are not liquefaction induced (i.e. seismic slope stability, lateral foundation displacements, downdrag on deep foundations, etc.) are addressed under the Extreme Event limit state for each specific structure. These resistance factors apply only to the Extreme Event I limit state.

Table 9-11, Resistance Factors for Soil Shear Strength Loss Induced Seismic Hazards

Earthquake Hazard Description	Resistance Factor Symbol φ	Design Earthquake	
		FEE	SEE
Sand-Like Soil Shear Strength Loss (Liquefaction) (Triggering)	$\varphi_{SL\text{-}Sand}$	0.85	0.90
Sand-Like Soil No Shear Strength Loss (No Liquefaction)	$\varphi_{NSL\text{-}Sand}$	0.70	0.75
Clay-Like Soil Shear Strength Loss (Triggering)	$\varphi_{SL\text{-}Clay}$	0.85	0.90
Flow Failure (Triggering)	φ_{Flow}	0.90	0.95
Lateral Spread (Triggering)	φ_{Spread}	0.90	0.95
Site R/W Seismic Instability (Triggering)	$\varphi_{EQ\text{-}Stability}$	0.90	0.95

9.10 REFERENCES

The geotechnical information contained in this Manual must be used in conjunction with the SCDOT Seismic Design Specifications for Highway Bridges, SCDOT Bridge Design Manual, and AASHTO LRFD Bridge Design Specifications. The geotechnical manual will take precedence over all references with respect to geotechnical engineering design.

AASHTO LRFD Bridge Design Specifications, U.S. Customary Units, 4th Edition, (2007), American Association of State Highway and Transportation Officials.

SCDOT Bridge Design Manual (2006), South Carolina Department of Transportation, http://www.scdot.org/doing/bridge/06design_manual.shtml

SCDOT Seismic Design Specifications for Highway Bridges (2008), South Carolina Department of Transportation, http://www.scdot.org/doing/bridge/bridgeseismic.shtml