APPENDIX D

REINFORCED SOIL SLOPES

GEOTECHNICAL DESIGN MANUAL

January 2022

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APPENDIX D

REINFORCED SOIL SLOPE DESIGN GUIDELINES

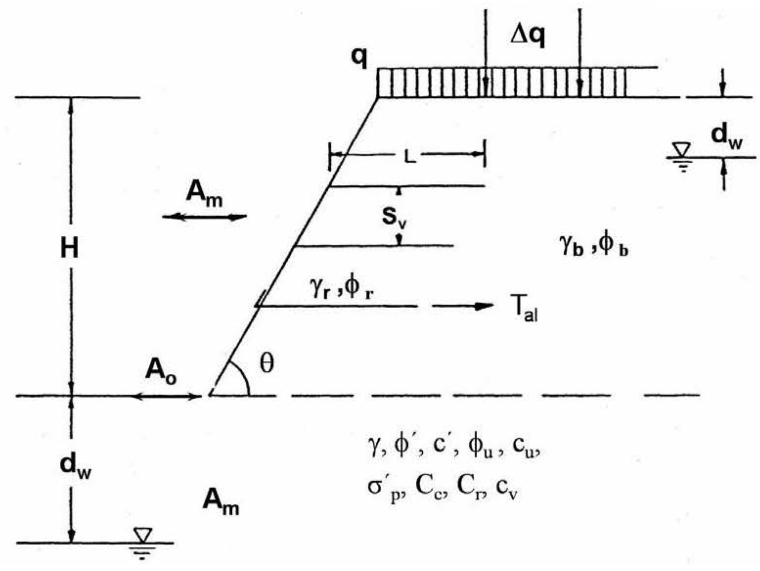
D.1 INTRODUCTION

This Appendix outlines SCDOT's design methodology for Reinforced Soil Slopes (RSS). RSSs are internally stabilized fill slopes, constructed of alternating layers of compacted soil and reinforcement. An RSS is different from an MSE wall or a conventional slope in that the slope has an inclination ranging from 1H:1V to 70° and will require a facing element. This Appendix governs the design of permanent and temporary RSSs. The design life of both permanent and temporary RSS is provided in Chapter 17. The procedures contained in this Appendix may also be used to design a reinforced embankment (2H:1V to 1H:1V).

This design process assumes that the existing subgrade soils provide a stable foundation for the founding of the RSS. If improvement is required, see Chapters 19 and 20. This procedure assumes that classical limit equilibrium slope stability methods are applicable (see Chapter 17).

D.2 DESIGN CONSIDERATIONS AND REQUIREMENTS

The first part of the design is determining the geometry, the external loading conditions, the performance criteria, and any construction constraints. The geometry should include the location relative to the remainder of the project (i.e., to the centerline and specific station). The geometry should also indicate the anticipated toe and crest of the slope (see Figure D-1). During this step of the design, external loads should be identified. These loads include, but are not limited to transient (traffic), permanent (weight of pavement surface) and/or seismically induced loads. The performance criteria are based on whether the RSS is a bridge or road embankment. Bridge embankment RSSs are further subdivided by OC (see Chapter 8) to meet the Performance Objectives for the bridge contained in the Seismic Specs. RSSs shall be designed for the appropriate limit state indicated in Chapter 8. The load and resistance factors are determined from Chapters 8 and 9, respectively. The Performance Limits are provided in Chapter 10. Any constraints on construction (i.e., soft ground, standing water, limited ROW, etc.) should also be identified during this step. These construction constraints should be carefully considered before deciding to use an RSS.



Notations:

H = slope height
θ = slope angle
T_{al} = allowable strength of reinforcement
L = length of reinforcement
S_v = vertical spacing of reinforcement
q = surcharge load
Δq = temporary live load
A_o = ground acceleration coefficient
A_m = design seismic acceleration
d_w = depth to ground water table in slope
d_{wf} = depth to ground water table in foundation
c_u and ϕ_u or c' and ϕ' = strength parameters for each soil layer
γ_{wet} and γ_{dry} = unit weights for each soil layer
C_c, C_r, c_v and σ'_p = consolidation parameters for each soil layer

Figure D-1, RSS Design Requirements and Geometry (Berg, Christopher and Samtani – Vol. II (2009))

D.3 SITE CONDITIONS

The second step in the design of an RSS is the evaluation of the topography, subsurface conditions, and in-situ soil/rock parameters. The topography evaluation should include reviewing the height requirements of the slope, the amount of space between the toe of the slope and the anticipated extent of the reinforcement, and the condition of the existing ground surface. This evaluation should identify the need for any temporary shoring that may be required to install the RSS (i.e., the grading of the site requires cutting). The subsurface conditions and in-situ soil/rock parameters shall be evaluated using the procedures presented in Chapters 4 through 7.

D.4 REINFORCED FILL MATERIAL PROPERTIES

The fill materials to be used to construct a permanent RSS shall meet the criteria provided in STS SC-M-206-1 (latest version) for *Reinforced Soil Slopes (RSS)*. The GEOR shall provide, in the plans, the fill material requirements for temporary RSSs. The soil strength parameters [φ, c (both total and effective) and γ_t] shall be determined in accordance with Chapter 5.

D.5 DESIGN PARAMETERS FOR REINFORCEMENT

Portions of the following sections of this Appendix are adopted directly from Tanyu, Sabatini and Berg (2008), and Berg, Christopher and Samtani – Volumes I and II (2009) and are used with the permission of the US Department of Transportation, Federal Highway Administration. Italics have been added to reflect additions or modifications to the selected text and to supply references to this Manual. According to Berg, et al. – Vol. I (2009):

D.5.1 <u>Reinforcement Pullout Resistance</u>

A reinforced soil mass is somewhat analogous to reinforced concrete in that the mechanical properties of the mass are improved by reinforcement placed parallel to the principal strain direction to compensate for soil's lack of tensile resistance. The improved tensile properties are a result of the interaction between the reinforcement and the soil. The composite material has the following characteristics:

- Stress transfer between the soil and reinforcement takes place continuously along the reinforcement
- Reinforcements are distributed throughout the soil zone with a degree of regularity

Stresses are transferred between soil and reinforcement by friction (*Figure D-2A*) and/or passive resistance (*Figure D-2B*) depending on the reinforcement geometry.

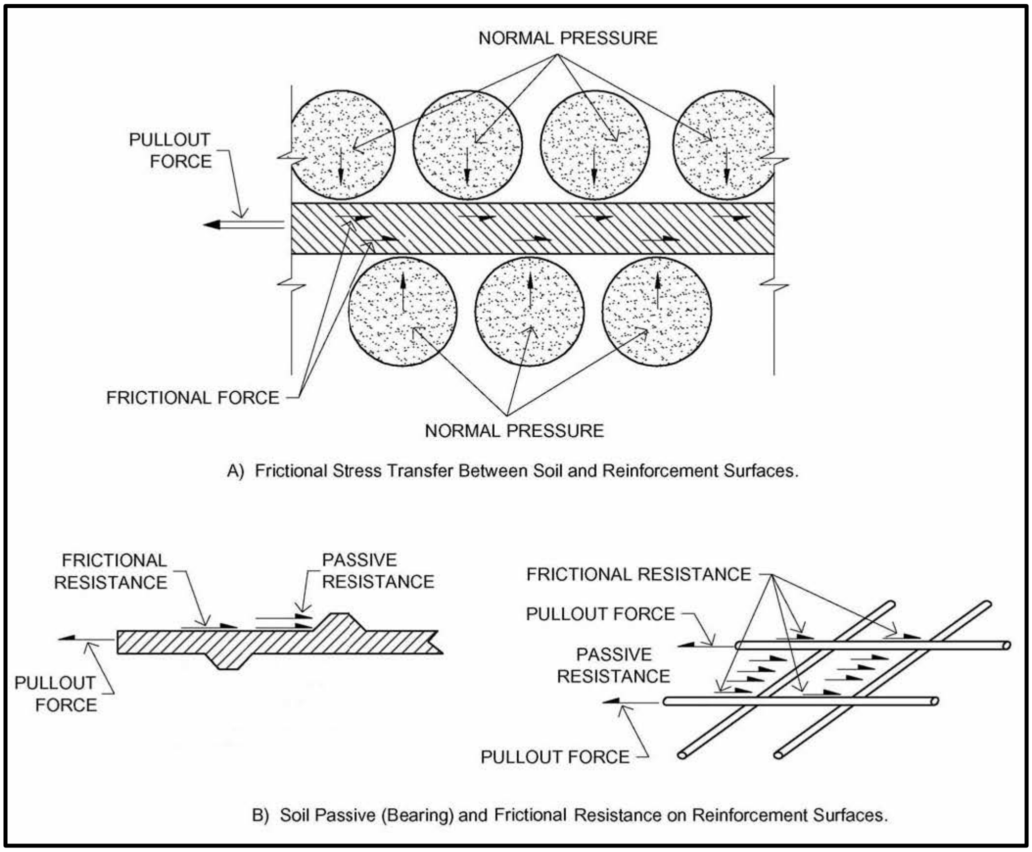


Figure D-2, Mechanisms of Pullout Resistance
(Berg, et al. – Vol. I (2009))

Friction develops at locations where there is a relative shear displacement and corresponding shear stress between soil and reinforcement surface. Reinforcing elements dependent on friction should be aligned with the direction of soil reinforcement relative movement. Examples of such reinforcing elements are steel strips, longitudinal bars in grids, geotextile, and some geogrid layers.

Passive Resistance occurs through the development of bearing type stresses on “transverse” reinforcement surfaces normal to the direction of soil reinforcement relative movement. Passive resistance is generally considered to be the primary interaction for bar mat, wire mesh reinforcements, and geogrids with relatively stiff cross machine direction ribs. The transverse ridges on “ribbed” strip reinforcements also provide some passive resistance.

The contribution of each transfer mechanism for a particular reinforcement will depend on the roughness of the surface (skin friction), normal effective stress, grid opening dimensions, thickness of the transverse members, and elongation characteristics of the reinforcement. Equally important for interaction development are the soil characteristics, including grain-size and grain-size distribution, particle shape, density, water content, cohesion, and stiffness.

The primary function of reinforcement is to restrain soil deformations. In doing so, stresses are transferred from the soil to the reinforcement. These stresses are resisted by the reinforcement tension and/or shear and bending.

Two types of reinforcement material can be considered:

- **Strips, bars, and steel grids** – A layer of steel strips, bars or grids is characterized by the cross-sectional area, the thickness and perimeter of the reinforcement element, and the center-to-center horizontal distance between elements (for steel grids, an element is considered to be a longitudinal member of the grid that extends into the wall).
- **Geotextiles and geogrids** – A layer of geosynthetic strips is characterized by the width of the strips and the center-to-center horizontal distance between them. The cross-sectional area is not needed, since the strength of the geosynthetic is expressed by a tensile force per unit width, rather than by stress. Difficulties in measuring the thickness of these thin and relatively compressible materials preclude reliable estimates of stress.

The structural design properties of reinforcement materials are a function of geometric characteristics, strength and stiffness, durability, and material type. The *2* most commonly used reinforcement materials, steel and geosynthetics, must be considered separately as *discussed in the following Sections*:

D.5.1.1 Inextensible Reinforcements

For steel reinforcements, the design life is achieved by reducing the cross-sectional area of the reinforcement used in the design calculations by the anticipated corrosion *(see next Section)* losses over the design life period as follows:

$$E_c = E_n - E_R$$ **Equation D-1**

Where,

E_c = Thickness of the reinforcement at the end of the design life
E_n = Nominal thickness at construction
E_R = Sacrificial thickness of metal expected to be lost by uniform corrosion during the service life of the structure

The nominal long-term design strength of inextensible reinforcement is obtained for steel strips and grids as shown in the following equation. T_{al} in units of force per unit width is used to provide a unified strength approach, which can be applied to any reinforcement. Tensile strength of a known steel or grid reinforcement can also be expressed in terms of the tensile load carried by the reinforcement, P_{tal}. Thus, nominal tensile strength may be calculated and expressed in the following terms:

$$T_{al} = \frac{F_y * A_c}{b}$$ **Equation D-2**

$$P_{tal} = F_y * A_c$$ **Equation D-3**

Where,

F_y = Minimum yield strength of steel
b = Unit width of sheet, grid, bar or mat
A_c = Design cross sectional area corrected for corrosion loss
T_{al} = Strength per unit reinforcement width
P_{tal} = Strength per reinforcement element

The LRFD resistance factors for steel reinforcements in RSSs are listed in *Chapter 9*. The resistance factor for strip reinforcement under static conditions is 0.75 (*see Chapter 9*). The resistance factor for steel grid reinforcement, for static loading, is 0.65 (*see Chapter 9*) when reinforcement is connected to a rigid facing element and is 0.75 when connected to a flexible facing. The lower resistance factor for grid reinforcing members connected to a rigid facing element (e.g., a concrete panel or block) is used to account for the greater potential for local overstress due to load unconformities for steel grids than for steel strips or bars. Transverse and longitudinal grid members are sized in accordance with ASTM *A1064 – Standard Specification for Carbon-Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete*.

A_c for strips is determined as:

$$\boldsymbol{A_c = b * E_c = b * (E_n - E_R)} \qquad \textbf{Equation D-4}$$

Where,

b = Unit width of sheet, grid, bar or mat
E_c = Thickness at end of design life (see Figure D-3)
E_n = Thickness at end of construction
E_R = Sacrificial thickness of metal expected to be lost by uniform corrosion during the service life of the structure

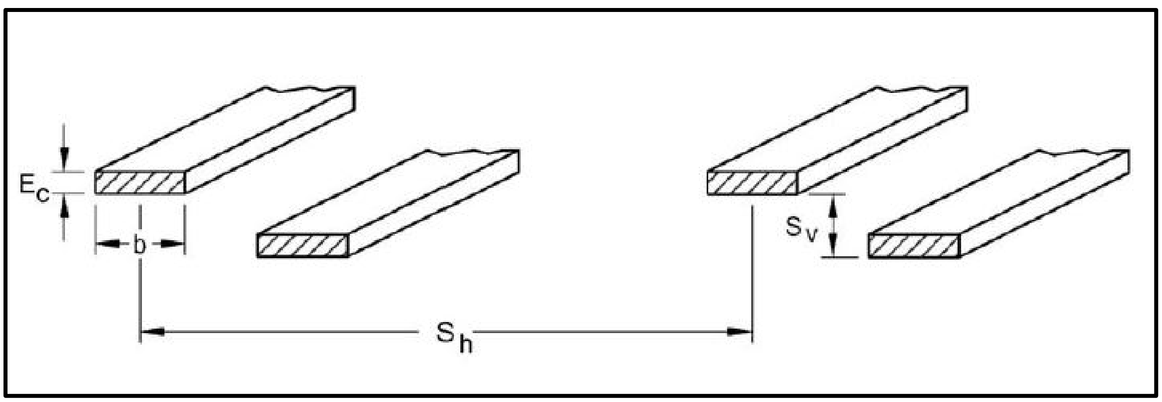


Figure D-3, Cross Section Area for Strips
(Berg, et al. – Vol. I (2009))

When estimating E_R, it may be assumed that equal loss occurs from the top and bottom of the strip.

A_c for bars is determined as:

$$A_c = N_b * \left(\frac{\pi * (D^*)^2}{4}\right)$$ **Equation D-5**

Where,

N_b = Number of bars per unit width b
D^* = Bar diameter after corrosion loss (Figure D-4)

When estimating D*, it may be assumed that corrosion losses occur uniformly over the area of the bar.

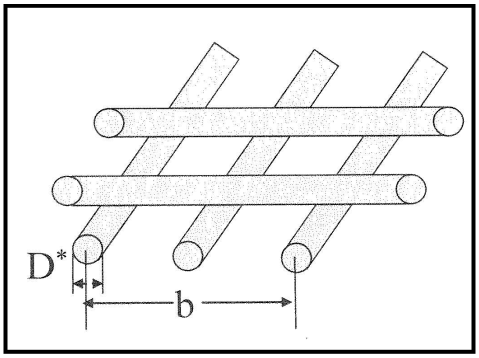


Figure D-4, Cross Section Area for Bars
(Berg, et al. – Vol. I (2009))

D.5.1.2 Corrosion Rates Inextensible Reinforcements

According to Berg, et al. – Vol. I (2009):

> The corrosion rates presented *below* are suitable for conservative design. These rates assume a mildly corrosive backfill material having the controlled electrochemical property limits as discussed in the STS SC-M-206-1.

Table D-1, Steel Corrosion Rates for Moderately Corrosive Reinforced Fill
(Berg, et al. – Vol. I (2009))

For zinc/side	0.58 mils/yr (first 2 years)
	0.16 mils/yr (thereafter)
For residual carbon steel/side[1]	0.47 mils/yr (thereafter)

[1]after zinc depletion

> Based on these rates, complete corrosion of galvanization with the minimum required thickness of *0.0034 inches (3.4 mils)* (AASHTO M111) is estimated to occur during the first 16 years and a carbon steel thickness or diameter loss of 0.055 inches to 0.08 inches would be anticipated over the remaining 75- to 100-year design life, respectively. Galvanization can be damaged during handling and construction by abrasion, scratching, notching, and cracking. Construction equipment should not travel directly on reinforcing elements and elements should not be dragged, excessively bent, or field cut. Galvanized reinforcement should be well supported during lifting and handling to prevent excessive bending. Any damaged section should be field repaired by coating the damaged area with a field grade zinc-rich paint.

The designer of an *RSS* structure should also consider the potential for changes in the reinforced backfill environment during the structure's service life. In certain parts of *South Carolina*, it can be expected that deicing salts might cause such an environmental change. For this problem, the depth of chloride infiltration and concentration are of concern.

For permanent structures directly supporting roadways exposed to deicing salts, limited data indicates that the upper 8 feet of the reinforced backfill (as measured from the roadway surface) or greater depths, depending on the gradation and compaction of the fill, are affected by higher corrosion rates not presently defined. Under these conditions, it is recommended that a 30 mil (minimum) geomembrane be placed below the road base and tied into a drainage system to mitigate the penetration of the deicing salts in lieu of higher corrosion rates. Alternatively free draining reinforced fil (e.g., No. 57 stone) has been found to allow salts to "flush out" and limit corrosion as discussed in *Elias, Fishman, Christopher and Berg (2009)*. Note that value of "higher" corrosion rate for deicing salt exposure is not defined.

The following project situations lie outside the scope of the previously presented values:

- Structures exposed to a marine or other chloride-rich environment. (Excluding locations where de-icing salts are used). For marine saltwater structures, carbon steel losses on the order of 3.2 mils per side or radius should be anticipated in the first few years, reducing to 0.67 to 0.70 mils thereafter. Zinc losses are likely to be quite rapid as compared to losses in reinforced fills meeting the *RSS* electro-chemical criteria. Total loss of zinc (3.4 mils) should be anticipated in the first year.
- Structures exposed to stray currents, such as from nearby underground power lines, and structures supporting or located adjacent to electrical railroads.
- Structures exposed to acidic water emanating from mine waste, abandoned coal mines, or pyrite-rich soil and rock strata.

Each of these situations creates a special set of conditions that should be specifically analyzed by a corrosion specialist.

D.5.1.3 Extensible Reinforcements

According to Berg, et al. – Vol. I (2009):

Selection of long-term nominal tensile strength, T_{al}, for geosynthetic reinforcement is determined by thorough consideration of all possible … time dependent strength losses over the design life period. The tensile properties of geosynthetics are affected by factors such as creep, installation damage, aging, temperature, and confining stress. Furthermore, characteristics of geosynthetic products manufactured with the same base polymer can vary widely requiring a T_{al} determination for each individual product with consideration of all these factors.

The GEOR for the RSS should refer to STSs SC-M-203-2 for Geogrid Soil Reinforcement and SC-M-203-3 for Geotextile Soil Reinforcement for the T_{al} that are assigned to specific geogrid and geotextile designations.

Polymeric reinforcement, although not susceptible to corrosion, may degrade due to physiochemical activity in the soil such as hydrolysis, oxidation, and environmental stress cracking depending on the polymer type. In addition, these materials are susceptible to installation damage and the effects of high temperature at the facing and connections. Temperature acts to accelerate creep and aging processes and temperature effects are accounted through their determination. While the normal range of in-ground temperature vary from 55° F in cold and temperate climates to 85° F in arid desert climates, temperatures at the facing and reinforcement connections can be as high as 120° F. Confining stress is not directly taken into account other than indirectly when installation damage is evaluated. For creep and durability, confining stress generally will tend to improve the long-term strength of the reinforcement.

The available long-term strength, T_{al}, is calculated as follows:

$$\boldsymbol{T_{al} = \frac{T_{ult}}{RF} = \frac{T_{ult}}{RF_{ID} * RF_{CR} * RF_D}}$$ **Equation D-6**

Where,

T_{ult} = Ultimate tensile strength (strength per unit width).

RF = Reduction factor. The product of all applicable reduction factors

RF_{ID} = Installation damage reduction factor accounts for the damaging effects of placement and compaction of soil or aggregate over the geosynthetic material during installation. A minimum reduction factor 1.1 should be used to account for testing uncertainties.

RF_{CR} = Creep reduction factor accounts for the effect of creep resulting from long-term sustained tensile load applied to the geosynthetic.

RF_D = Durability reduction factor accounts for the strength loss caused by chemical degradation (aging) of the polymer used in the geosynthetic reinforcement (e.g., oxidation of polyolefins, hydrolysis of polyesters, etc.). A minimum reduction factor 1.1 should be used to account for testing uncertainties.

RF_{ID}, RF_{CR} and RF_D reflect actual long-term strength losses, analogous to loss of steel strength due to corrosion. This long-term geosynthetic reinforcement strength loss concept is illustrated in Figure D-*5*. As shown in the figure, some strength losses occur immediately upon installation, and others occur throughout the design life of the reinforcement. Much of the long-term strength loss does not begin to occur until near the end of the reinforcement design life.

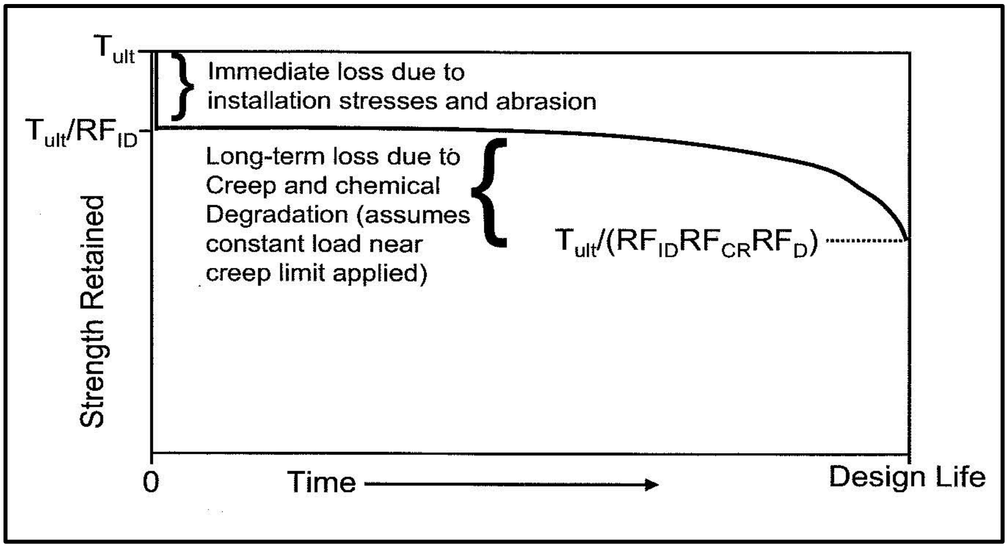


Figure D-5, Long-Term Geosynthetic Reinforcement Strength Concept (Berg, et al. – Vol. I (2009))

Because of varying polymer types, quality, additives and product geometry, each geosynthetic is different in its resistance to aging and attack by different chemical agents. Therefore, each product must be investigated individually, or in the context of product line where the same polymer source and additives are used, and the manufacturing process is the same for all products in the product line. This product line approach makes it possible to interpolate reduction factors for products in the product line not specifically tested using the reduction factors determined for the products in the product line that are specifically tested for each degradation mechanism.

The AASHTO LRFD Specifications provide minimum requirements for the assessment of T_{al} for use in the design of geosynthetic reinforced soil structures. Protocols for evaluating T_{al} are included in Berg, et al. – Vol. I (2009) with supporting information on testing procedures provided in Elias, et al. (2009).

The determination of reduction factors for each geosynthetic product and product line requires extensive field and/or laboratory testing which can take a year or more to complete.

D.5.1.4 Ultimate Tensile Strength, T_{ult}

The value selected for T_{ult}, for design purposes, is the minimum average roll value (MARV) for the product. The tensile strength of the reinforcement is determined from wide strip tests for geotextiles per ASTM D4595 – *Standard Test Method for Tensile Properties of Geotextiles by the Wide-Width Strip Method* or for geogrids per D6637 – *Standard Test Method for Determining Tensile Properties of Geogrids by the Single or Multi-Rib Tensile Method* based on the MARV for the product. This MARV accounts for statistical variance in the material strength. Other sources of uncertainty and variability in the long-term strength result from installation damage, creep extrapolation, and the chemical degradation process. It is assumed that the observed variability in the creep rupture envelope is 100 percent correlated with the short-term tensile strength, as the creep strength is typically directly proportional to the short-term tensile strength within a product line. Therefore, the MARV of T_{ult} adequately takes into account variability in the creep strength. Note that the MARV of T_{ult} is the minimum certifiable wide-width tensile strength provided by the product manufacturer.

D.5.1.5 Reduction Factors

The following Sections of this Appendix are adopted directly from Berg, et al. – Vol. I (2009) and are used with the permission of the US Department of Transportation, Federal Highway Administration. Italics have been added to reflect additions or modifications to the selected text and to supply references to this Manual.

D.5.1.5.1 Installation Damage Reduction Factor, RF_{ID}

According to Berg, et al. – Vol. I (2009):

> Damage during handling and construction, from abrasion and wear, punching and tear or scratching, notching, and cracking may occur in geosynthetics. These types of damage can only be avoided by using care during handling and construction. Construction equipment should not travel directly on geosynthetic materials.
>
> Damage during reinforced fill placement and compaction operations is a function of the severity of the loading imposed on the geosynthetic during construction operations and the size and angularity of the reinforced fill. For RSS construction, lightweight, low strength geotextiles and geogrids should be avoided to minimize damage with ensuing loss of strength.
>
> Protocols for field testing for this reduction factor are detailed in *Elias, et al. (2009)* and in ASTM D5818 – *Standard Practice for Exposure and Retrieval of Samples to Evaluate Installation Damage of Geosynthetics*. These protocols require that the geosynthetic material be subjected to a reinforced fill placement and compaction cycle, consistent with field practice. The ratio of the initial strength, to the strength of retrieved samples defines this reduction factor. For reinforcement applications, a minimum weight of 8.0 oz/yd^2 for geotextiles is recommended to minimize installation damage. In general, the combination of geosynthetic reinforcement, and backfill placement and gradation characteristics, should not result in a value of RF_{ID} greater than 1.7. If testing indicates that RF_{ID} will be greater than 1.7 (approximately a 40 percent strength loss); then that combination of geosynthetic and backfill conditions should not be used, as this or greater levels of damage will cause the remaining strength to be highly variable and therefore not adequately reliable for design.
>
> In general, RF_{ID} is strongly dependent on the backfill soil gradation characteristics and its angularity, especially for lighter weight geosynthetics. Provided a minimum of 6 inches of backfill material is placed between the reinforcement surface and the compaction and spreading equipment wheels/tracks, the backfill placement and compaction technique will have a lesser effect on RF_{ID}. Regarding geosynthetic characteristics, the geosynthetic weight/thickness or tensile strength may have a significant effect on RF_{ID}. However, for coated polyester geogrids, the coating thickness may overwhelm the effect of the product unit weight or thickness

on RF_{ID}. *Even with product specific testing results a minimum RF_{ID} 1.1 shall be used to account for testing uncertainties.*

D.5.1.5.2 Creep Reduction Factor, RF_{CR}

The creep reduction factor is required to limit the load in the reinforcement to a level known as the creep limit that will preclude excessive elongation and creep rupture over the life of the structure. The creep limit strength is thus analogous to yield strength in steel. Creep is essentially a long-term deformation process. As load is applied, molecular chains move relative to each other through straightening out of folded or curved/kinked chains or through breaking of inter-molecular bonds, resulting in no strength loss, but increased elongation.

Eventually, if the load levels are sufficiently high (i.e., constant load near the creep limit), the molecular chains can straighten/elongate no more without breaking the molecular chains. Significant strength loss occurs only when the straightening/slipping process is exhausted. If the load is high enough, molecular chains break, and both elongation and strength loss occur at an accelerating rate, eventually resulting in rupture. Generally this strength loss occurs only near the end of the design life of the geosynthetic under a given load level.

The creep reduction factor is obtained from long-term laboratory creep testing as detailed in Appendix D *of Berg, et al. – Vol. II (2009)*. Creep testing is essentially a constant load test on multiple product samples, loaded to various percentages of the ultimate product load, for periods of up to 10,000 hours. For creep testing one of two approaches may be used: 1) "conventional" creep testing per ASTM *D5262 – Standard Test Method for Evaluating the Unconfined Tension Creep and Creep Rupture Behavior for Geosynthetics*, or 2) a combination of Stepped Isothermal Method (SIM) per ASTM D6992 – *Standard Test Method for Accelerated Tensile Creep and Creep-Rupture of Geosynthetic Materials Based on Time-Temperature Superposition Using the Stepped Isothermal Method*, which is an accelerated method using stepped increases in temperatures to allow tests to be performed in a matter of days, and "conventional" creep testing. The creep reduction factor is the ratio of the ultimate load to the extrapolated maximum sustainable load (i.e., creep rupture limit) within the design life of the structure (e.g., several years for temporary structures *(less than 5 years)*, 75 to 100 years for permanent structures).

Typical ranges of RF_{CR} as a function of polymer type are *indicated in Table D-2*.

Table D-2, Creep Reduction Factors
(Berg, et al. – Vol. I (2009))

Polymer Type	RF_{CR}
Polyester (PET)	1.6 to 2.5
Polypropylene (PP)	4.0 to 5.0
High Density Polyethylene (HDPE)	2.6 to 5.0

If no product specific creep reduction factors are provided, then the maximum creep reduction factor for a specific polymer shall be used. If the polymer is unknown, then an RF_{CR} of 5.0 shall be used.

D.5.1.5.3 Durability Reduction Factor, RF_D

According to Berg, et al. – Vol. I (2009):

> This reduction factor is dependent on the susceptibility of the geosynthetic to be attacked by chemicals, thermal oxidation, hydrolysis, environment stress cracking, and microorganisms, and can vary typically from 1.1 to 2.0. *Even with product specific tests results, the minimum reduction factor shall be 1.1. Protocols for testing to obtain this reduction factor has been described in Elias, et al. (1999) and Elias, et al. (2009).*
>
> Due to the long-term nature of these durability evaluation protocols (2 to 3 years could be required to complete such tests), it is generally not practical to conduct such tests for typical geosynthetic reinforcement design, but are generally more suited for research activities. However, short-term index type tests can be conducted as indicators of good long-term durability performance, based on correlation to the long-term research results obtained and reported by Elias, et al. (1999). Such index test results, combined with a criteria applied to the test results that can be considered to indicate good long-term performance, can be used to justify a default value for RF_D that can be used for the determination of T_{al}.

Table D-3 provides the minimum testing requirements for the use of the default RF_D for geosynthetic reinforcement.

Table D-3, Minimum Testing Requirements for use RF_D (modified Berg, et al. – Vol. I (2009))

Geosynthetic Type	Property	Test Method	Criteria to allow use of Default RF_D
Polypropylene (PP) and Polyethylene (HDPE)	UV Oxidation Resistance	ASTM D4355	Min. 70% strength retained after 500 hrs. in weatherometer
Polyester (PET)[1]	Hydrolysis Resistance	Inherent Viscosity Method (ASTM D4603 and GRI Test Method GG8) or Determine Directly Using GEL Permeation Chromatography	Minimum Number (M_n) Average Molecular Weight of 25,000
		ASTM D7409	Maximum Carboxyl End Group (CEG) Content of 30
All Polymers	Survivability	Weight per Unit Area, ASTM D5261	Min. 8 oz/yd^2
All Polymers	Percent Post-consumer Recycled Material by Weight	Certification of Material Used	Maximum 0%

[1]Alternatively, a default RF_D = 1.3 may be used if product specific installation damage testing is performed and it is determined that RF_{ID} = 1.7 or less, and if the other requirements of this table are met.

ASTM D4355 – *Standard Test Method for Deterioration of Geotextiles by Exposure to Light, Moisture and Heat in a Xenon Arc Type Apparatus*
ASTM D4603 – *Standard Test Method for Determining Inherent Viscosity of Poly(Ethylene Terephthalate) (PET) by Glass Capillary Viscometer*
GRI GG8 – *Determination of the Number Average Molecular Weight of PET Yarns Based on Relative Viscosity Value*
ASTM D7409 – *Standard Test Method for Carboxyl End Group Content of Polyethylene Terephthalate (PET) Yarns*
ASTM D5261 – *Standard Test Method for Measuring Mass per Unit Area of Geotextiles*

D.6 UNREINFORCED STABILITY

The overall (global) stability of the unreinforced slope is checked first to determine if reinforcement is required, if the potential for deep-seated failure surfaces is possible, and to determine the approximate limit of reinforcement. If the resistance factor is less than required in Chapter 9, then, the unreinforced slope is stable and no reinforcement is required. It is noted that the resistance factor (φ) is the inverse of the Factor of Safety (i.e., φ = 1/FS). If φ is greater than indicated in Chapter 9, then the slope is considered unstable and reinforcement of the slope is required. According to Berg, et al. – Vol. II (2009):

Determine the size of the critical zone to be reinforced.

- Examine the full range of potential failure surfaces found to have:
  Unreinforced safety factor, FS_u (φ_u) ≤ Required safety factor, FS_r (φ_r)
- Plot all of these surfaces on the cross-section of the slope.
- The surfaces that just meet the required resistance factor (φ) roughly envelope the limits of the critical zone to be reinforced as shown in *Figure D-6*.

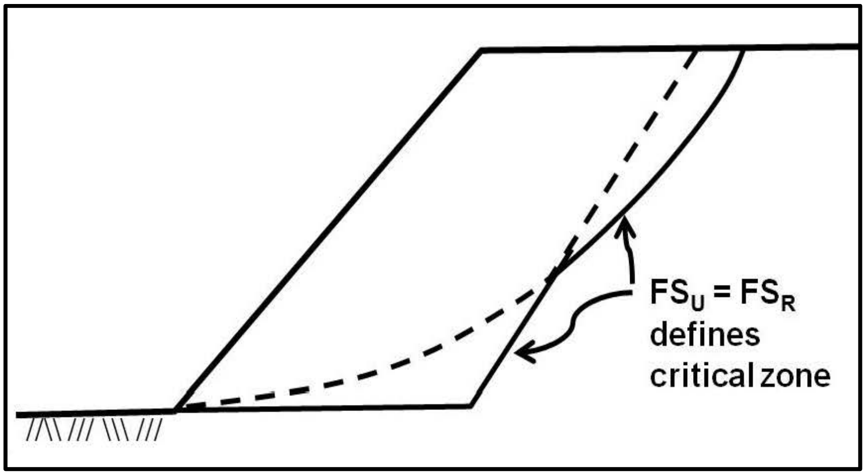


Figure D-6, Critical Zone
(Berg, et al. – Vol. II (2009))

Further, this stability check also identifies potential deep-seated failures. Deep-seated failure surfaces extend into the foundation soil and may be used to determine the first estimate of the length of reinforcement required to stabilize the slope. Depending on the length of reinforcement required some form of ground improvement (see Chapters 19 and 20) may be necessary.

This stability can be determined using classical slope stability analyses. The failure surfaces may be circular or non-circular (sliding block) and both should be checked. Overall stability analyses are performed for the Strength and Service limit states on roadway embankments only. RSSs located within bridge embankments requireStrength, Service and EE I limit state checks. It should be noted that it is assumed that all RSSs are free draining and that pore water pressures are not allowed to build up behind the face of the slope. In addition to checking deep seated failure potential, the potential for lateral squeeze at the toe should also be checked (see Chapter 17). If the potential for lateral squeeze is indicated, ground improvement at the toe may be required. Ground improvement may consist of the following options; please note that this is not an all-inclusive list, but is meant as an example of ground improvement options,

- Undercut and replace soft soils
- Toe berm construction
- Vertically stage construction the embankment to allow for strength gain with time
- Construction of a shear key beneath the toe of the embankment
- Use vertical reinforcing elements (i.e., stone columns, driven piling, deep mixing method columns, etc.)
- Improve subsurface drainage (i.e., use wick drains)

After the development of the final design, a compound global stability analysis shall be performed. As defined in Chapter 18, a compound stability analysis examines failure surfaces that pass through either the retained fill and reinforced soil mass to exit through the RSS face, or that pass through the retained fill, reinforced soil mass, and the foundation soil to exit either at or beyond the toe of the RSS. The actual strength parameters for the reinforced soil mass shall be used in the analysis. These analyses can only be performed once a specific reinforcement strength and type is selected.

D.7 REINFORCEMENT DESIGN

The reinforcement used in RSS may consist of either extensible (geosynthetics) or inextensible (metallic) reinforcement. While the use of inextensible (metallic) reinforcement is permitted, it is noted that the current STS for RSS is written based on the use of extensible (geogrid) reinforcement being used. The GEOR is required to write a Special Provision to SC-M-206-1 to allow the use of geotextiles in addition to geogrids, if the GEOR wants to allow the use of geotextiles as well as geogrids. If inextensible reinforcement is be used, the GEOR shall write a Special Provision indicating the soil and inextensible properties required. It is noted that the GEOR may review and use the latest version of STS SC-M-714 for *Mechanically Stabilized Earth (MSE) Walls* for information regarding soils and inextensible material properties. Inextensible reinforcement may only be used with wire baskets and must be connected to the baskets. In this step, the reinforcement is designed to provide a stable slope that meets the requirements of the project. According to Berg, et al. – Vol. II (2009):

> Calculate the total reinforcement tension per unit width of slope T_S required to obtain the required *resistance factor 1/φ_r* for each potential failure surface inside the critical zone in *the previous step* that extends through or below the toe of the slope using the following equation:

$$\boldsymbol{T_S = \left(\frac{1}{\varphi_r} - \frac{1}{\varphi_u}\right) * \left(\frac{M_D}{D}\right)} \quad \textbf{Equation D-7}$$

Where,

T_S = The sum of the required tensile force per unit width of reinforcement (considering rupture and pullout) in all reinforcement layers intersecting the failure surface

M_D = Driving moment about the center of the failure surface

D = The moment arm of T_S about the center of the failure circle, where,

- = Radius of circle R for continuous, sheet type extensible reinforcement (i.e., assumed to act tangentially to the circle) *(see Figure D-7)*
- = Radius of circle R for continuous, sheet type inextensible reinforcement (e.g., wire mesh reinforcement) to account for normal stress increase on adjacent soil *(see Figure D-7)*
- = Vertical distance, Y, to the centroid of T_S for discrete element, strip type reinforcement. Assume H/3 above slope base for preliminary calculations (i.e., assumed to act in a horizontal plane intersecting the failure surface at H/3 above the slope base) *(see Figure D-7)*

$1/\varphi_r$ = Target minimum slope *resistance factor* which is applied to both the soil and reinforcement

$1/\varphi_u$ = Unreinforced slope *resistance factor*

$T_{S\text{-}MAX}$ = The largest T_S calculated establishes the total design tension

Note: The maximum unreinforced resistance factor usually does not control the location of $T_{S\text{-}MAX}$; the most critical surface is the surface requiring the greatest amount of reinforcement strength.

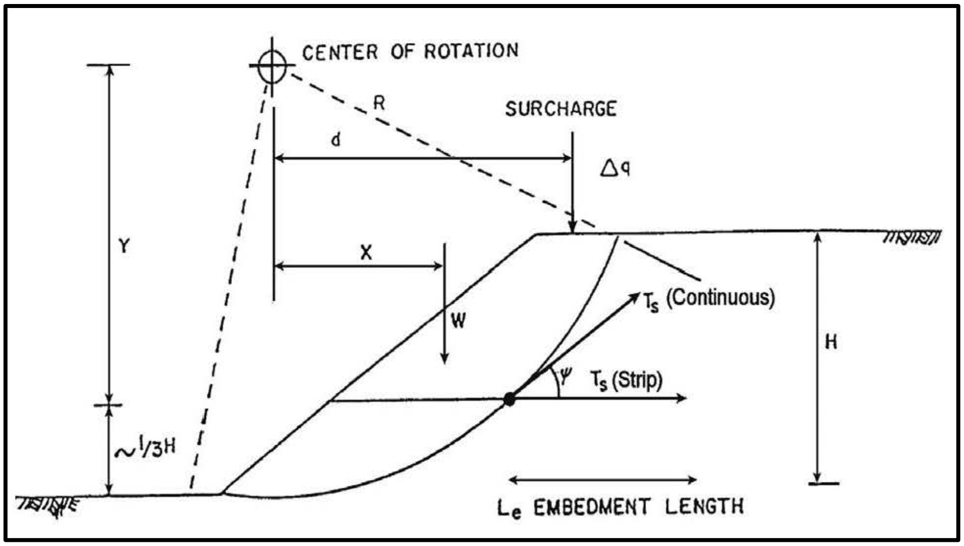


Figure D-7, Geometry of Rotational Shear Failure Surface (Berg, et al. – Vol. II (2009))

Determine the total design tension per unit width of slope ($T_{S\text{-}MAX}$) using Figures D-8 and D-9 and compare $T_{S\text{-}MAX}$ from the chart to $T_{S\text{-}MAX}$ calculated from Equation D-7. If significantly different, check the validity of the charts based on the limiting assumptions listed in the figure and recheck the calculations in the previous step (Unreinforced Stability) and Equation D-7.

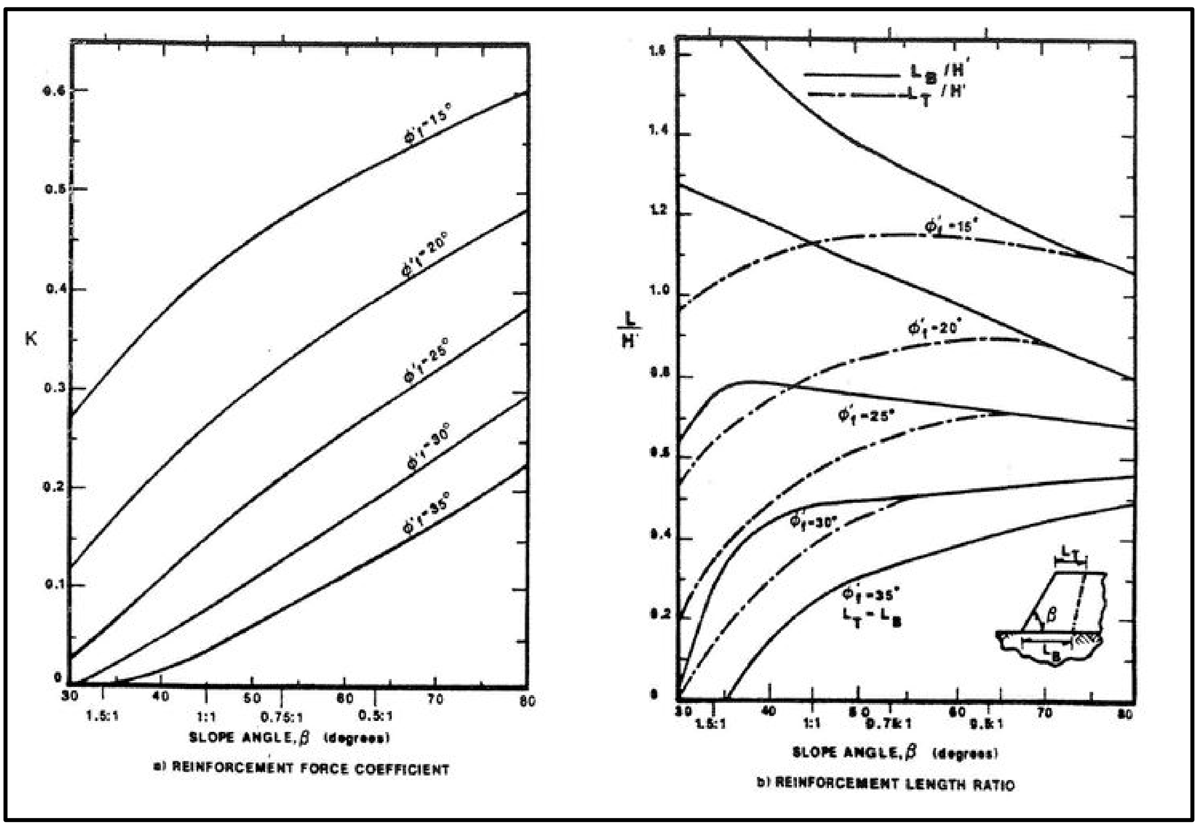


Figure D-8, Reinforcement Strength Requirements Chart Solution - A (Berg, et al. – Vol. II (2009))

CHART PROCEDURE:

1) Determine force coefficient K from figure above, where ϕ_r = friction angle of reinforced fill:

$$\phi_f = \tan^{-1}\left(\frac{\tan\phi_r}{FS_R}\right)$$

2) Determine:

$$T_{S\text{-}MAX} = 0.5\ K\ \gamma_r\ (H')^2$$

where: $H' = H + q/\gamma_r$
q = a uniform load

3) Determine the required reinforcement length at the top L_T and bottom L_B of the slope from the figure above.

LIMITING ASSUMPTIONS

- Extensible reinforcement.
- Slopes constructed with uniform, cohesionless soil, c = 0).
- No pore pressures within slope.
- Competent, level foundation soils.
- No seismic forces.
- Uniform surcharge not greater than $0.2\ \gamma_r$ H.
- Relatively high soil/reinforcement interface friction angle, $\phi_{sg} = 0.9\ \phi_r$ (may not be appropriate for some geotextiles).

Note: $FS_R = 1/\varphi_R$, where φ_R is the resistance factor (see Chapter 9)

Figure D-9, Reinforcement Strength Requirements Chart Solution - B (Berg, et al. – Vol. II (2009))

According to Berg, et al. – Vol. II (2009):

> *Figures D-8 and D-9* is provided for a quick check of computer-generated results. The figures present a simplified method based on a 2-part wedge type failure surface and is limited by the assumptions noted on the figure.
>
> Note that *Figures D-8 and D-9* is not intended to be a single design tool. Other design charts that are available from the literature could also be used (e.g., Ruegger, 1986; Leshchinsky and Boedeker, 1989; and Jewell, 1990). Several computer programs are also available (*see Section D.10*) for analyzing a slope with a given reinforcement and can be used as a check. Judgment in selection of other appropriate design methods (i.e., most conservative or experience) is required.

After determining the maximum required tensile strength of the reinforcement, the determination of the distribution of the reinforcement comes next. According to Berg, et al. – Vol. II (2009):

> For low slopes ($H \leq 20$ feet) assume a uniform reinforcement distribution and use $T_{S\text{-}MAX}$ to determine the spacing or the required tension, T_{MAX}, requirements for each reinforcement layer.
>
> For high slope ($H > 20$ feet), either a uniform reinforcement distribution may be used (preferable) or the slope may be divided *into 2* (top and bottom) or *3* (top,

middle and bottom) reinforcement zones of equal height, and use a factored T_{S-MAX} in each zone for spacing or design tension requirements (see *Figure D-9*). The total required tension in each zone is found from:

For 1 zone:

Use T_{S-MAX}

For 2 zones:

$$T_{Bottom} = {}^{3}/_{4} * (T_{S-MAX}) \qquad \textbf{Equation D-8}$$

$$T_{Top} = {}^{1}/_{4} * (T_{S-MAX}) \qquad \textbf{Equation D-9}$$

For 3 zones:

$$T_{Bottom} = {}^{1}/_{2} * (T_{S-MAX}) \qquad \textbf{Equation D-10}$$

$$T_{Middle} = {}^{1}/_{3} * (T_{S-MAX}) \qquad \textbf{Equation D-11}$$

$$T_{Top} = {}^{1}/_{6} * (T_{S-MAX}) \qquad \textbf{Equation D-12}$$

The force is assumed to be uniformly distributed over the entire zone.

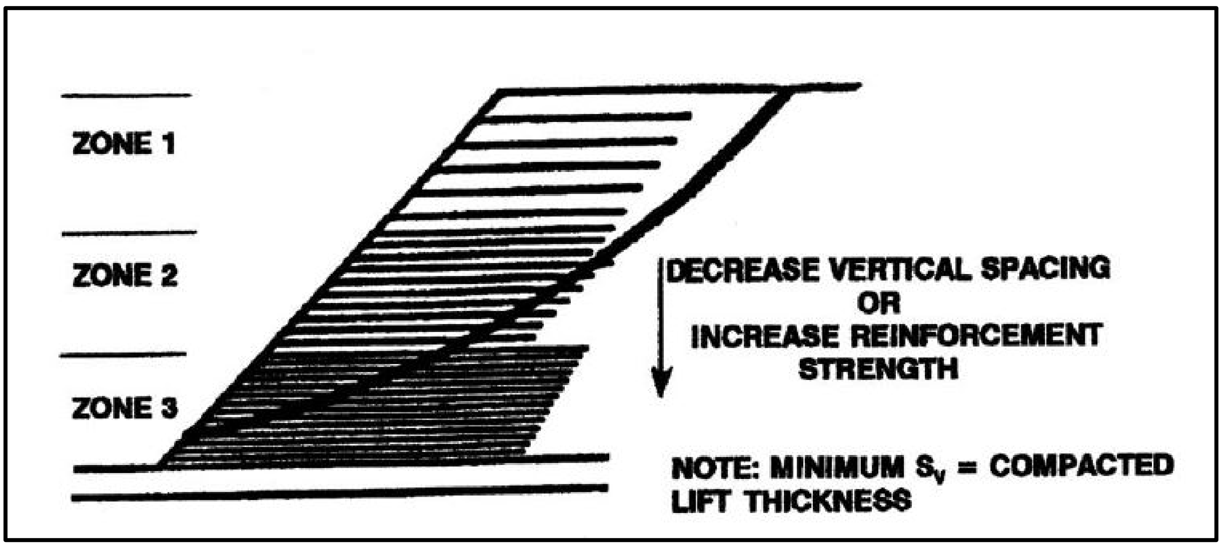


Figure D-10, Reinforcing Zone Vertical Layout
(Berg, et al. – Vol. II (2009))

Determine reinforcement vertical spacing (S_V) or the maximum design tension (T_{MAX}) requirements for each reinforcement layer.

For each zone, calculate T_{MAX} for each reinforcing layer in that zone based on an assumed S_V, or, if the allowable reinforcement strength is known, calculate the minimum vertical spacing and number of reinforcing layers N required for each zone based on:

$$T_{MAX} = \frac{T_{zone} * S_V}{H_{zone}} = \frac{T_{zone}}{N} \leq T_{al} * R_c \qquad \textbf{Equation D-13}$$

Where,

R_c = Coverage ratio of the reinforcement which equals the width of the reinforcement b divided by the horizontal spacing S_h

S_V = Vertical Spacing of reinforcement; multiples of compacted layer thickness of ease of construction *(see Figure D-10)*

T_{zone} = Maximum reinforcement tension required for each zone; $T_{S\text{-}MAX}$ for low slopes (H ≤ 20 feet)

T_{al} = T_{ult}/RF *(see Equation D-6)*

H_{zone} = Height of zone; T_{Top}, T_{Middle}, and T_{Bottom} for high slopes (H > 20 feet)

N = Number of reinforcement layers

Use short 4 to 6.5 feet lengths of intermediate reinforcement layers to maintain a maximum vertical spacing of 16 inches or less for face stability and compaction quality. For slopes flatter than 1H:1V (*45°*), closer spaced reinforcements (i.e., every lift or every other lift, but no greater than 16 inches) preclude having to wrap the face in well graded soils (e.g., sandy gravel and silty and clayey sands). Wrapped faces are required for steeper slopes and uniformly graded soils to prevent face sloughing. Alternative vertical spacing could be used to prevent face sloughing, but in these cases a face stability analysis should be performed either using the method presented in this chapter or by evaluating the face as an infinite slope using:

Equation D-14

$$\boldsymbol{\varphi = \frac{\gamma_g Hz \cos\beta \sin\beta}{c'H + (\gamma_g - \gamma_w)Hz\cos^2\beta\tan\phi' + F_g(\cos\beta\sin\beta + \sin^2\beta\tan\phi')}}$$

Where,

c' = Effective cohesion

ϕ' = Effective friction angle

γ_g = Saturated unit weight

γ_w = Unit weight of water

z = Vertical depth to failure plane defined by the depth to saturation

H = Vertical slope height

β = Slope angle

F_g = Summation of geosynthetic resisting force

Intermediate reinforcement should be placed in continuous layers and does not need to be as strong as the primary reinforcement, but it must be strong enough to survive construction (e.g., minimum survivability requirements for geotextiles in road stabilization applications in AASHTO M288) and provide localized tensile reinforcement to the surficial soils.

If the interface friction angle of the intermediate reinforcements, ρ_{sr}, is less than that of the primary reinforcement ρ_r, then ρ_{sr} should be used in the analysis for the portion of the failure surface intersecting the reinforced soil zone.

To ensure that the rule-of-thumb reinforcement distribution is adequate for critical or complex structures, recalculate T_S using *Equation D-7* to determine potential failure above each layer of primary reinforcement.

Check that the sum of the reinforcement forces passing through each failure surface is greater than T_S required for that surface. Only count reinforcement that extends *more than 3 feet* beyond the surface to account for pullout resistance. If the available reinforcement force is not sufficient, increase the length of reinforcement not passing through the surface or increase the strength of lower-level reinforcement. Simplify the layout by lengthening some reinforcement layers to create 2 or 3 sections of equal reinforcement length for ease of construction and inspection. Reinforcement layers do not generally need to extend to the limits of the critical zone, except for the lowest levels of each reinforcement section. Check the length using Figure *D-8(b)*. Note: L_e is already included in the total length, L_T and L_B from *Figure D-8(b).*

When checking a design that has zones of different reinforcement lengths, lower zones may be over reinforced to provide reduced lengths of upper reinforcement levels. In evaluating the length of requirements for such cases, the pullout stability for the reinforcement must be carefully checked in each zone for the critical surfaces exiting at the base of each length zone.

D.7.1 Estimating L_e

According to Berg, et al. – Vol. II (2009):

The embedment length L_e of each reinforcement layer beyond the critical sliding surface (i.e., circle found for $T_{S\text{-}MAX}$) must be sufficient to provide adequate pullout resistance based on:

$$L_e = \frac{T_{S-MAX}}{\varphi * (F^*) * \alpha * \sigma'_v * R_c * C} \qquad \textbf{Equation D-15}$$

Where,

$T_{S\text{-}MAX}$ = Maximum factored tensile load in the reinforcement (calculated in *Equation D-7)*

φ = Resistance factor for reinforcement pullout (*see Chapter 9*)

α = Scale effect correction factor (discussed in D.7.2)

F^* = Pullout friction factor (discussed in D.7.3)

σ'_v = Unfactored effective vertical stress at the reinforcement level in the resistance zone

C = Overall reinforcement surface area geometry factor (2 for strip, grid and sheet-type reinforcement)

R_c = Reinforcement coverage ratio
For continuous geosynthetic reinforcement R_c = 1

$$R_c = \frac{b}{S_h} \qquad \text{Equation D-16}$$

Where,
b = Gross width of the reinforcing element
S_h = Center-to-center horizontal spacing between reinforcements *(see Figure D-10)*

Minimum value of L_e is 3 feet. For cohesive soils, check L_e for both short- and long-term pullout conditions, when using the semi-empirical equations to obtain F*. For long-term design use *ϕ' of the reinforced fill with c' = 0.* For short-term evaluation, conservatively use *ϕ of the reinforced fill with c = 0* from consolidated undrained triaxial or direct shear tests or *perform* pullout tests.

When checking a design that has zones of different reinforcement length, lower zones may be over reinforced to provide reduced lengths of upper reinforcement levels. In evaluating the length requirements for such cases, the pullout stability for the reinforcement must be carefully checked in each zone for the critical surfaces existing at the base of each length zone.

D.7.2 Correction Factor (α)

According to Berg, et al. – Vol. I (2009):

The correction factor (α) depends primarily upon the strain softening of the compacted granular backfill material, the extensibility, and the length of the reinforcement. For inextensible *(metallic)* reinforcement, α is approximately 1, but it can be substantially smaller than 1 for extensible *(geosynthetic)* reinforcements. The α factor can be obtained from pullout tests on reinforcements with different lengths or derived using analytical or numerical load transfer models, which have been "calibrated" through numerical test simulations. In the absence of test data, *the values included in Table D-4 should be used for geogrids and geotextiles.*

Table D-4, Typical Values of α
(According to Berg, et al. – Vol. I (2009))

Reinforcement Type	α
All metallic reinforcements	1.0
Geogrids	0.8
Geotextiles	0.6

D.7.3 Pullout Friction Factor (F*)

According to Berg, et al. – Vol. I (2009):

The pullout friction factor can be obtained most accurately from laboratory or field pullout tests performed with the specific material to be used on the project *(i.e., select backfill and reinforcement)*. Alternatively, F* can be derived from empirical or theoretical relationships developed for each soil-reinforcement interaction mechanism and provided by the reinforcement supplier. For any reinforcement, F* can be estimated using the general equation:

$$F^* = F_q * \alpha_\beta + \tan\rho \qquad \textbf{Equation D-17}$$

Where,

F_q = The embedment (or surcharge) bearing capacity factor

α_β = A bearing factor for passive resistance which is based on the thickness per unit width of the bearing member

ρ = The soil-reinforcement interaction friction angle

In absence of site-specific pullout testing data, it is reasonable to use these semi-empirical relationships in conjunction with the standard specifications for backfill to provide a conservative evaluation of pullout resistance.

For steel ribbed reinforcement, F* is commonly estimated as:

$$F^* = \tan\rho = 1.2 + \log C_u \qquad \textbf{Equation D-18}$$

It is noted that at the top of the RSS, F* is at a maximum of 2.0.

For reinforcement located at a depth of 20 feet or more below the top of the RSS F* may be estimated using:

$$F^* = \tan\phi_r \qquad \textbf{Equation D-19}$$

Where,

ρ = Interface friction angle mobilized along the reinforcement

ϕ_r = Reinforced backfill peak friction angle

C_u = Uniformity coefficient of the backfill (*see Chapter 6*)

If the specific C_u for the wall backfill is unknown during design, a C_u of 4 should be assumed (i.e., F* = 1.8 at the top of the wall), for backfill meeting the requirements *previously provided.*

For steel grid reinforcements with transverse spacing (S_t) ≥ 6 inches, F* is a function of a bearing or embedment factor (F_q), applied over the contributing bearing factor (α_β), as follows at the top of the structure:

$$F^* = F_q * \alpha_\beta = 40 * \alpha_\beta = 40 * \left(\frac{t}{2S_t}\right) = 20 * \left(\frac{t}{S_t}\right) \qquad \textbf{Equation D-20}$$

While, for reinforcement located at a depth of 20 feet or more below the top of the RSS F* may be estimated using:

$$F^* = F_q * \alpha_\beta = 20 * \alpha_\beta = 20 * \left(\frac{t}{2S_t}\right) = 10 * \left(\frac{t}{S_t}\right) \quad \textbf{Equation D-21}$$

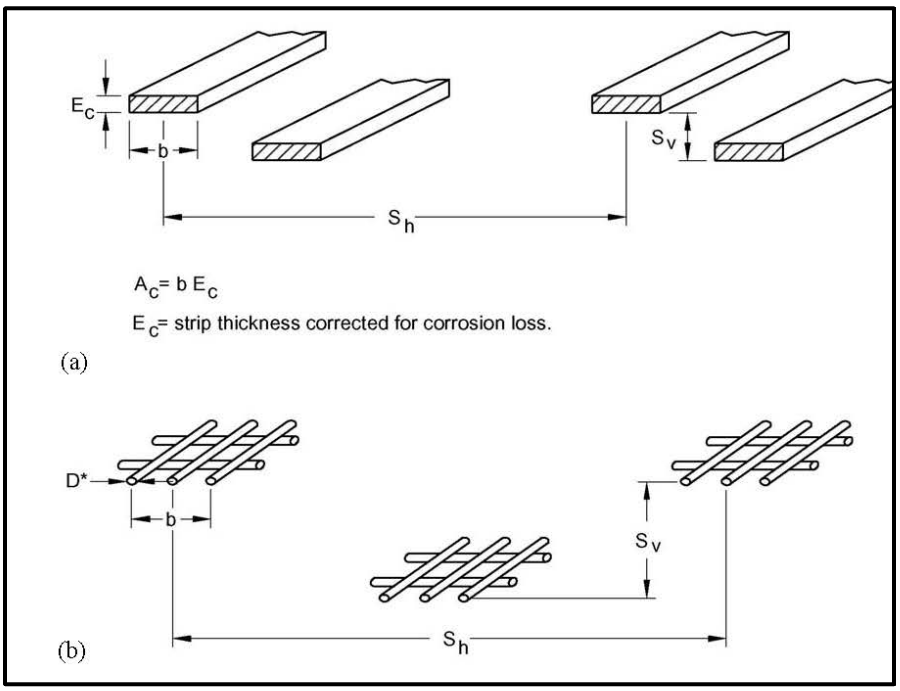
Where,

t = The thickness of the transverse bar

S_t = *The distance between individual bars in steel grid reinforcement and* shall be uniform throughout the length of the reinforcement, rather than having transverse grid members concentrated only in the resistance zone

For geosynthetic (i.e., geogrid and geotextile) sheet reinforcement, the pullout resistance is based on a reduction in the available soil friction. In the absence of test data, the F* value for geosynthetic reinforcement should conservatively be estimated as:

$$F^* = 0.67 * \tan \phi_r \quad \textbf{Equation D-22}$$



[1]For geosynthetic strips use (a)

[2]Please note that geosynthetic strips have no measureable thickness.

Figure D-11, Definitions of b, S_h and S_V
(Berg, et al. – Vol. I (2009))

D.7.4 Selection of Reinforcement

The type of reinforcement to be used in the RSS shall be determined. The 2 types of reinforcement are extensible and inextensible. Extensible reinforcements consist of geosynthetic materials, typically geogrids (biaxial or uniaxial) and geotextiles. These reinforcements are a wrapped face consisting of a layer of geogrid that wraps around the face and a layer of geotextile to prevent erosion of the reinforced soil materials. Inextensible reinforcements consist of bars or bar mats (metallic grids) and shall meet the requirements in STS SC-M-713 (latest version) for

Mechanically Stabilized Earth (MSE) Walls for the inextensible material properties. These reinforcements are typically connected to wire baskets at the front face to provide anchorage at the face of the slope. The selection of the type of reinforcement is influenced by the strength required to maintain stability and the aesthetic appearance required at the completion of the project.

When extensible reinforcements are used, the continuity of the reinforcement shall be assured. For geogrid used as the extensible reinforcement, the geogrid shall be placed so that the strong axis is perpendicular to the face of the RSS. The geogrid reinforcement materials to be used to construct an RSS shall meet the criteria provided in STS SC-M-203-2 (latest version) for *Geogrid Soil Reinforcement*. Indicate on the plans the required T_{al} for the geogrid soil reinforcement. Overlapping of geogrids in the strong axis direction is not permitted. The use of a mechanical connection (i.e., a bodkin connector) will be permitted, provided the strength of the connection is equal to the required geogrid strength or if reduced geogrid strength equal to the connection is used. Prior to using a mechanical connection obtain written permission from the OES/GDS. Geogrids may be overlapped in the transverse (i.e., perpendicular to the RSS face) direction. The minimum overlap in RSS shall be 12 inches. If a mechanical connection is allowed in the strong axis direction, the GEOR is reminded that the location, type and material for the connection, should be shown on the plans. In addition, the plans should also include a requirement for the Contractor to provide the results of testing of the mechanical connection.

When geotextiles are used as the extensible reinforcement, the geotextile shall be placed so that the strong axis is perpendicular to the face of the RSS. The geotextile reinforcement materials to be used to construct an RSS shall meet the criteria provided in STS SC-M-203-3 (latest version) for *Geotextile Soil Reinforcement*. Indicate on the plans the required T_{al} for the geotextile soil reinforcement. Overlapping of the geotextiles in the strong axis direction is not permitted. The use of sew seams may be permitted in the strong axis direction; however, the strength of the geotextile will be reduced to the strength of the sewn seam. The sewn seam strength (whether field or factory sewn) shall be at least 25 percent of T_{ult} (ASTM D4884 – *Standard Test Method for Strength of Sewn or Bonded Seams of Geotextiles*) in machine direction. Prior to using a sewn seams obtain written permission from the OES/GDS. For sewn seams use thread that consists of either polypropylene or polyester polymers and which has a strength matching the strength of the geotextile being seamed. Do not use nylon thread. Use thread that is of contrasting color to that of the geotextile itself. Use a double row of double-thread chain stitch, Type 401 (see ASTM D6193 – *Standard Practice for Stitches and Seams*). Use 150 to 400 stitches per yard depending on the weight of the geotextile. The GEOR should consultant with a geotextile manufacturer or supplier to determine the appropriate stitch density. Use either a “butterfly” seam (Type SSd) or “J” seam (Type SSn) (see ASTM D6193). Geotextiles may be overlapped in the transverse direction. The minimum overlap shall be 12 inches. If a sewn seam in the cross machine (i.e., transverse) direction is to be used as opposed to overlapping, the sewn seam strength (whether field or factory sewn) shall be at least 25 percent of T_{ult} (ASTM D4884) in the cross machine direction. If sewn seams are allowed, the GEOR is reminded that the ultimate strength of the seam in the machine and cross machine directions, the location of the sewn seam, the type of thread, the color contrast of the thread, the type and density of stitching, and the seam type shall be shown on the plans. In addition, the plans should also include a requirement for the Contractor to provide the results of sewn seam testing.

D.8 EXTERNAL STABILITY

D.8.1 Sliding Resistance

According to Berg, et al. – Vol. II (2009):

> Evaluate the width of the reinforced soil mass at any level to resist sliding along the reinforcement. Use a 2-part wedge type failure surface defined by the limits of the reinforcement (the length of reinforcement at the depth of evaluation defined *previously*). The analysis can best be performed using a computerized method which takes into account all soil strata and interface friction values. If the computer program does not account for the presence of reinforcement, the back of the wedge should be angled at 45° + φ/2 (*see Figure D-11*) or parallel to the back of the reinforced zone, whichever is flatter (i.e., the wedge should not pass through layers of reinforcement to avoid an overly conservative design). The frictional resistance provided by the weakest layer, either the reinforced soil, the foundation soil or the soil-reinforcement interface, should be used in the analysis.

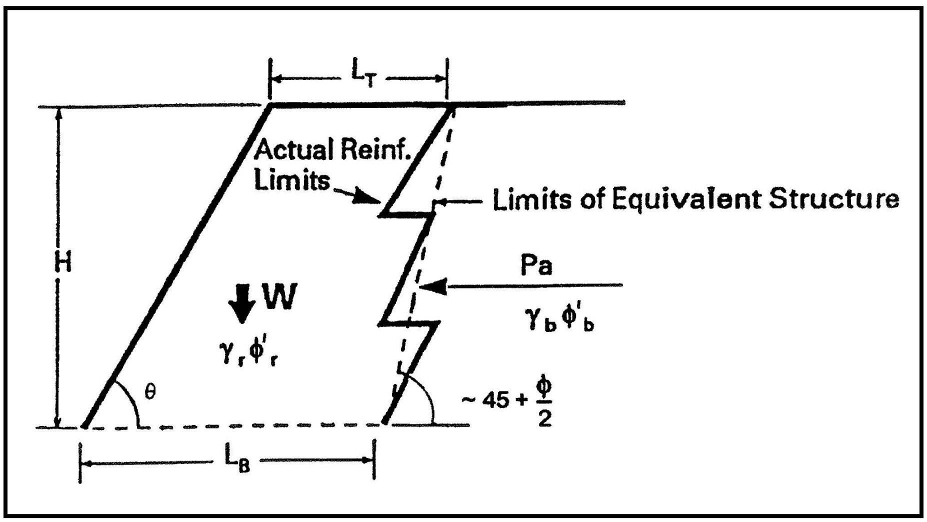


Figure D-12, Sliding Stability Analysis (Elias, Christopher and Berg (2001))

> *A simple analysis using a sliding block method can be performed as a check*. The method also assumes that the reinforcement layers are truncated along a plane parallel to the slope face, which may or may not be the case. The analysis is based on a 2-part wedge model to predict L_B assuming that the reinforcement interface is the weakest plane. The frictional resistance provided by the weakest layer in contact with either, the geosynthetics and reinforced soil *(i.e., the interface friction) or between the reinforced soil and* the foundation soil.

The frictional resistance between the reinforced soil and the foundation soil will depend on whether the foundation soil is Sand-Like or Clay-Like. Regardless of which soil comprises the foundation soil the following equation is required to be balanced:

Horizontal Driving Forces ≤ φ * Horizontal Resisting Forces **Equation D-23**

For Sand-Like soils use the following equations:

$$P_a * \cos\phi_b \leq \varphi * (W + P_a * \sin\phi_b) * \tan\phi_{min} \quad \textbf{Equation D-24}$$

For L < H

$$W = \frac{1}{2}L^2 * \gamma_r * \tan\theta \quad \textbf{Equation D-25}$$

Or for L > H

$$W = \left[L * H - \frac{H^2}{(2\tan\theta)}\right] * \gamma_r \quad \textbf{Equation D-26}$$

$$P_a = \frac{1}{2}\gamma_b * H^2 * K_a \quad \textbf{Equation D-27}$$

Where,

L = Length of bottom reinforcing layer in each level where there is a reinforcement length change
H = Height of Slope
φ = Resistance Factor (see Chapter 9)
$\phi_{min,}$ = Minimum angle of shearing friction either between reinforced soil and reinforcement or the friction angle of the foundation soil
θ = Slope angle
γ_r & γ_b = Unit weight of the reinforced backfill and retained backfill, respectively
ϕ_b = Friction angle of retained fill (Note: If drains/filters are placed on the backslope, then ϕ_b equals the interface friction angle between the geosynthetic and retained fill)

For Clay-Like soils use the following equation:

$$P_a \leq \varphi * c * L_B \quad \textbf{Equation D-28}$$

Where,

φ = Resistance Factor (see Chapter 9)
c = Cohesion
L_B = Length of base of RSS
P_a = Active earth pressure (see Equation D-27)

D.8.2 Global (Deep-Seated) Stability

This sub-step is to evaluate the potential for deep-seated failure surfaces beyond or below the reinforced soil mass to provide resistance factors that meet the requirements of Chapter 9. This check is similar to and may use the results of the Unreinforced Stability analysis discussed previously.

D.8.3 Local Bearing Failure at Toe

According to Berg, et al. – Vol. II (2009):

> If a weak layer exists beneath the embankment to limited depth D_s, which is less than the width of the slope b' (*see Figure D-12*), the *resistance factor* against failure by squeezing may be calculated *using the procedures contained in Chapter 17*.

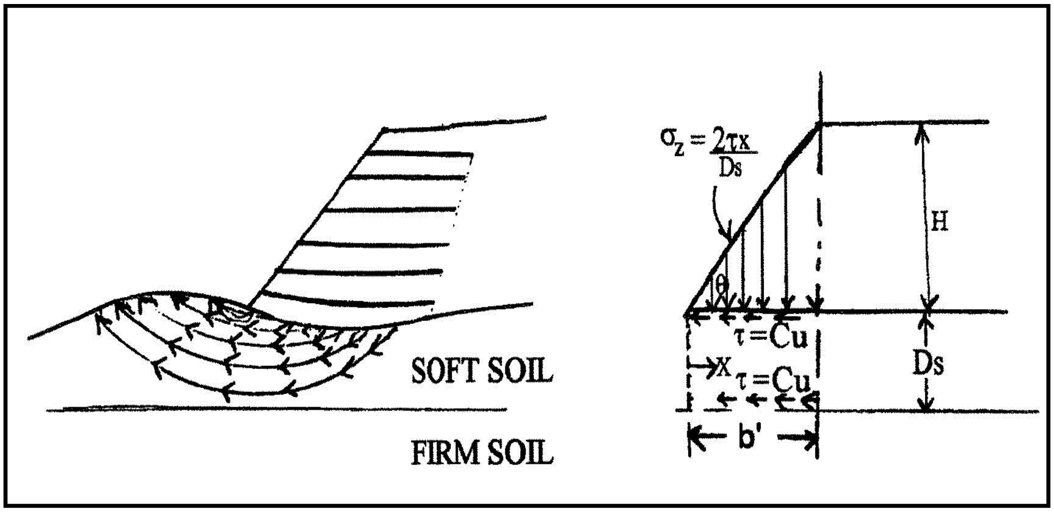


Figure D-13, Local Bearing Failure (Lateral Squeeze)
(Berg, et al. – Vol. II (2009))

> Caution is advised and rigorous analysis (i.e., numerical modeling) should be performed when the *resistance factor (φ) is greater than 0.5*. This approach is somewhat conservative as it does not provide any influence from the reinforcement. When the depth of the soft layer, D_s, is greater than the base width of the slope, b', general slope stability will govern design.

D.8.4 Foundation Settlement

The settlement (total, differential and time for settlement to occur) of the RSS shall be determined using the procedures provided in Chapter 17.

D.8.5 Seismic Stability

RSSs located within bridge embankments shall be designed seismically according to the procedures contained in Chapter 13. In addition, the RSS shall meet the requirements of Chapters 9 and 10 for resistance factors and displacements, respectively.

D.9 DRAINAGE SYSTEM DESIGN

The following Section of this Appendix is adopted directly from Berg, et al. – Vol. II (2009) and is used with the permission of the US Department of Transportation, Federal Highway Administration. Italics have been added to reflect additions or modifications to the selected text and to supply references to this Manual.

D.9.1 Subsurface Water Control

According to Berg, et al. – Vol. II (2009):

> Design of subsurface water drainage features should address flow rate, filtration, placement, and other details. Drains are typically placed at the rear of the reinforced soil mass in Figure *D-13*. Geocomposite drainage systems or conventional granular blanket and trench drains could be used. *Granular drainage systems are not addressed in this Appendix*.

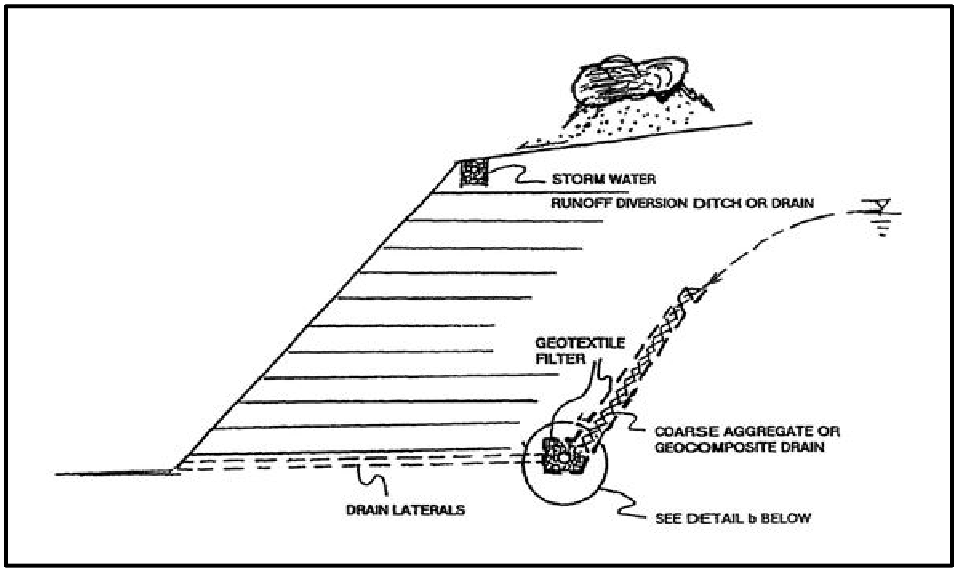


Figure D-14, Groundwater and Surface Drainage
(Berg, et al. – Vol. II (2009))

> Lateral spacing of outlets is dictated by site geometry and estimated flow. Outlet design should address long-term performance and maintenance requirements. Geosynthetic drainage composites can be used in subsurface water drainage design. Drainage composites should be designed with consideration for:
>
> - Geotextile filtration/clogging
> - Long-term compressive strength of polymeric core
> - Reduction of flow capacity due to intrusion of geotextile into the core
> - Long-term inflow/outflow capacity
>
> Procedures for checking geotextile permeability and filtration/clogging criteria are presented in Geosynthetic Design and Construction Guidelines, *Holtz, Christopher and Berg (2008), FHWA NHI-07-092*. Long-term compressive stress and eccentric loadings on the core of a geocomposite should be considered during design and selection. Though not yet addressed in standardized test methods or standards of practice, the following criteria are suggested for addressing core compression. The design pressure on a geocomposite core should be limited to either:
>
> - The maximum pressure sustained on the core in a test of 10,000 hours minimum duration

- The crushing pressure of a core, as defined with a quick loading test, *multiplied by a resistance factor of 0.2*

Note that crushing pressure can only be defined for some core types. For cases where a crushing pressure cannot be defined, suitability should be based on the maximum load resulting in a residual thickness of the core adequate to provide the required flow after 10,000 hours, or the maximum loading resulting in a residual thickness of the core adequate to provide the required flow as defined with the quick loading test *multiplied by a resistance factor of 0.2*.

Intrusion of the geotextiles into the core and long-term outflow capacity should be measured with a sustained transmissivity test. Slope stability analyses should account for interface shear strength along a geocomposite drain. The geocomposite/soil interface will most likely have a friction value that is lower than that of the soil. Thus, a potential failure surface may be induced along the interface. Geotextile reinforcements (primary and intermediate layers) must be more permeable than the reinforced fill material to prevent a hydraulic build up above the geotextile layers during precipitation. Special emphasis on the design and construction of subsurface drainage features is recommended for structures where drainage is critical for maintaining slope stability. Redundancy in the drainage system is also recommended for these cases.

D.9.2 Surface Water Runoff

Surface water runoff should be collected above the reinforced slope and channeled or piped below the base of the slope. This applies to be both permanent as well as temporary RSSs. Wrapped faces and/or intermediate layers of secondary reinforcement may be required at the face of reinforced slopes to prevent local sloughing. Intermediate layers of reinforcement help achieve compaction at the face, thus increasing soil shear strength and erosion resistance. These layers also act as reinforcement against shallow or sloughing types of slope failures. Intermediate reinforcement is typically placed on each or every other soil lift, except at lifts where primary structural reinforcement is placed. Intermediate reinforcement also is placed horizontally, adjacent to primary reinforcement and at the same elevation as the primary reinforcement when primary reinforcement is placed at less than 100 percent coverage in plan view. The intermediate reinforcement should extend 4 to 7 feet into the fill from the face. Select a long-term facing system to prevent or minimize erosion due to rainfall and runoff on the face.

Calculated flow-induced tractive shear stress on the face of the reinforced slope by:

$$\boldsymbol{\lambda = d * \gamma_w * s}$$ **Equation D-29**

Where,

λ = Tractive shear stress, psf
d = Depth of water flow, ft
γ_w = Unit weight of water, pcf

s = The vertical to horizontal angle of slope face, ft/ft

For λ < 2 psf, consider vegetation with temporary or permanent erosion control mat. For λ > 2 psf, consider vegetation with permanent erosion control mat or other armor type systems (e.g., riprap, gunite, prefabricated modular units, fabric-formed concrete, etc.). Select vegetation based on local horticultural and agronomic considerations and maintenance. Select synthetic (permanent) erosion control mat that is stabilized against ultraviolet light and is inert to naturally occurring soil-born chemicals and bacteria. Erosion control mats and blankets vary widely in type, cost, and more importantly, applicability to project conditions. Slope protection should not be left to the construction contractor or vendor's discretion.

D.10 COMPUTER SOFTWARE

The following Section of this Appendix is adopted directly from Berg, et al. – Vol. II (2009) and is used with the permission of the US Department of Transportation, Federal Highway Administration. Italics have been added to reflect additions or modifications to the selected text and to supply references to this Manual.

An alternative to reinforcement design is to develop a trial layout of reinforcement and analyze the reinforced slope with a computer program. Layout includes number, length, design strength, and vertical distribution of the geosynthetic reinforcement. The charts presented in Figure *D-8* provide a method for generating a preliminary layout. Note that these charts were developed with the specific assumptions noted in this figure.

Analyze the reinforced soil slope with the trial geosynthetic reinforcement layouts. The most economical reinforcement layout must provide the *maximum stability resistance factors* for internal, external and compound failure planes. A contour plot of the *highest resistance factor* values about the trial failure circle centroids is recommended to map and locate the *maximum resistance factor* values for the 3 modes of failure.

Computer generated designs made by software other than FHWA's ReSSA computer program shall meet the requirements of Chapter 26 and shall require verification that the computer program's design methodology meets the requirements provided herein. This shall be accomplished by either:

1. Provide complete, legible, calculations that show the design procedure step-by-step for the most critical geometry and loading condition that will govern each design section of the RSS structure. Calculations may be computer generated provided that all input, equations, and assumptions used are shown clearly.
2. Provide an electronic file with the input files and the full computer output of the FHWA sponsored computer program ReSSA (latest version) for the governing loading condition for each design section of the RSS structure. This software may be obtained at:

ADAMA Engineering, Inc.
12042 SE Sunnyside Road, Suite 711
Clackamas, OR 97015 USA
Tel. (971) 224-4187
adama@geoprograms.com

D.11 REFERENCES

Berg, R. R., Christopher, B. R., and Samtani, N. C. (2009), Design of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes – Volume I, Geotechnical Engineering Circular No. 11 – Volume I, (Publication No. FHWA NHI-10-024), National Highway Institute, Federal Highway Administration, U.S. Department of Transportation, Washington D.C.

Berg, R. R., Christopher, B. R., and Samtani, N. C. (2009), Design of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes – Volume II, Geotechnical Engineering Circular No. 11 – Volume II, (Publication No. FHWA NHI-10-025), National Highway Institute, Federal Highway Administration, U.S. Department of Transportation, Washington D.C.

Elias, V., Christopher, B. R., and Berg, R. R. (2001), Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction, (Publication No. FHWA NHI-00-043), National Highway Institute, Federal Highway Administration, U.S. Department of Transportation, Washington D.C.

Elias, V., Fishman, K. L., Christopher, B. R., and Berg, R. R. (2009), Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, (Publication No. FHWA-NHI-09-087), National Highway Institute, Federal Highway Administration, U.S. Department of Transportation, Washington D.C.

Elias, V., Salman, I., Juran, E., Pearce, E., and Lu, S. (1999), Testing Protocols for Oxidation and Hydrolysis of Geosynthetics, (Publication No. FHWA-RD-97-144, Office of Engineering R & D, Federal Highway Administration, U. S. Department of Transportation, Washington D.C.

Holtz, R. D., Christopher, B. R., and Berg, R. R. (2008), Geosynthetic Design and Construction Guidelines, (Publication No. FHWA NHI-97-092), National Highway Institute, Federal Highway Administration, U.S. Department of Transportation, Washington D.C.

Jewell, R. A., (1990), "Revised Design Charts for Steep Reinforced Slopes, Reinforced Embankments: Theory and Practice in the British Isles", *Thomas Telford*, London, United Kingdom.

Leshchinsky, E. and Boedeker, R. H., (1989) "Geosynthetic Reinforced Soil Structures", *Journal of Geotechnical Engineering,* ASCE, Volume 115, Issue 10, p. 1459-1478.

Ruegger, R., (1986), "Geotextile Reinforced Soil Structures on which Vegetation can be Established", *Proceedings of the 3rd International Conference on Geotextiles*, Vienna, Austria, Volume II.

SCDOT Standard Specifications for Highway Construction (2007), South Carolina Department of Transportation, https://www.scdot.org/business/pdf/2007_full_specbook.pdf.

Tanyu, B. F., Sabatini, P. J., and Berg, R. R. (2008), Earth Retaining Structures, (Publication No. FHWA NHI-07-071), National Highway Institute, Federal Highway Administration, U.S. Department of Transportation, Washington D.C.