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APPENDIX D

REINFORCED SOIL SLOPE

DESIGN GUIDELINES

D.1 INTRODUCTION

This document outlines SCDOT’s design methodology for Reinforced Soil Slopes (RSS). RSS structures 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 (unreinforced) slope in that the slope has an inclination ranging from 2H:1V to 1H:1V. This Appendix governs the design of permanent and temporary RSS structures. The design life of both permanent and temporary RSS is provided in Chapter 18.

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

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 the Operational Classification of the Bridge or Roadway (see 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.
D.3 SITE CONDITIONS

The second step in the design of 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 the RSS shall meet the criteria provided in the following tables. The actual soil strength parameters (φ, c and γ_t) shall be determined in accordance with Chapter 6.

Table D-1, Maximum Reinforced Backfill Properties

<table>
<thead>
<tr>
<th>Material Property</th>
<th>Granular Backfill</th>
</tr>
</thead>
<tbody>
<tr>
<td>Internal Friction Angle</td>
<td>34°</td>
</tr>
<tr>
<td>Total Unit Weight (lbs./cubic foot)</td>
<td>120</td>
</tr>
</tbody>
</table>

Table D-2, Granular Backfill Gradation Requirements

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>¾ in</td>
<td>100</td>
</tr>
<tr>
<td>No. 4</td>
<td>20-100</td>
</tr>
<tr>
<td>No. 40</td>
<td>0-60</td>
</tr>
<tr>
<td>No. 100</td>
<td>0-30</td>
</tr>
<tr>
<td>No. 200</td>
<td>0-15</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>≤ 6</td>
</tr>
<tr>
<td>Liquid Limit</td>
<td>≤ 30</td>
</tr>
<tr>
<td>Cu</td>
<td>≥ 4²</td>
</tr>
<tr>
<td>Organic Content</td>
<td>&lt; 1%</td>
</tr>
</tbody>
</table>

1 Cu = D_{soil}/D_{10}
2 Pullout testing required for Cu less than 4

Table D-3, Electrochemical Properties of Reinforced Backfill

<table>
<thead>
<tr>
<th>Reinforcement Material</th>
<th>Property</th>
<th>Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Metallic</td>
<td>Resistivity¹</td>
<td>&gt;3,000 ohm-cm</td>
</tr>
<tr>
<td>Metallic</td>
<td>Chlorides</td>
<td>&lt;100 ppm</td>
</tr>
<tr>
<td>Metallic</td>
<td>Sulfates</td>
<td>&lt;200 ppm</td>
</tr>
<tr>
<td>Metallic/Geosynthetic</td>
<td>pH</td>
<td>4.5 &lt; pH &lt; 9</td>
</tr>
</tbody>
</table>

¹ Chloride and sulfate testing are not required if the resistivity is greater than 5,000 ohm-cm

Table D-4, Temporary RSS Backfill Properties

<table>
<thead>
<tr>
<th>Gradation</th>
<th>Plasticity Index</th>
<th>Liquid Limit</th>
<th>Cu</th>
<th>pH</th>
<th>Internal Friction Angle</th>
<th>Total Unit Weight (pcf)</th>
<th>Organic Content</th>
</tr>
</thead>
<tbody>
<tr>
<td>¾ in</td>
<td>≤ 15</td>
<td>≤ 30</td>
<td>≥ 4</td>
<td>3 &lt; pH &lt; 10</td>
<td>&gt; 28°</td>
<td>&gt; 115</td>
<td>&lt;1%</td>
</tr>
<tr>
<td>100%</td>
<td>≤ 30</td>
<td>≤ 30</td>
<td>≥ 4</td>
<td>3 &lt; pH &lt; 10</td>
<td>&gt; 28°</td>
<td>&gt; 115</td>
<td>&lt;1%</td>
</tr>
</tbody>
</table>

D.5 DESIGN PARAMETERS FOR REINFORCEMENT

Portions of the following sections of this Appendix are adopted directly from Earth Retaining Structures, FHWA-NHI-07-071, June 2008 and Mechanically Stabilized Earth Walls and
Reinforced Soil Slopes Design & Construction (MSEW and RSS D&C), FHWA-NHI-00-043, March 2001 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 Earth Retaining Structures:

In this design step, the maximum factored tensile stress in each reinforcement layer (T_max) is compared to the nominal long-term reinforcement design strength as presented below:

\[
T_{\text{max}} \leq \phi R_c T_{al}
\]

Equation D-1

Where,

\( \phi = \) Resistance factor for tensile rupture (see Chapter 9)
\( R_c = \) Reinforcement coverage ratio as defined in Equation D-17 and Figure D-7
\( T_{al} = \) Nominal long-term reinforcement design strength

The nominal long-term reinforcement design strength (T_{al}) for LRFD is computed for inextensible and extensible reinforcements as presented in the following sections.

D.5.1 Reinforcement Pullout Resistance

"The purpose of this design step is to check the pullout resistance of the reinforcements. The resistance develops after the stress transfer between the soil and reinforcement takes place, which occurs through two mechanisms;

1. Friction along the soil-reinforcement interface (see Figure D-2a)
2. Passive soil resistance or lateral bearing capacity developed along the transverse sections of the reinforcement (see Figure D-2b)"
Stresses are transferred between soil and reinforcement by friction and/or passive resistance depending on reinforcement geometry.

**Friction** develops at locations where there is a relative shear displacement and corresponding shear stress between soil and reinforcement surface. Reinforcing elements where friction is important 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 rigid geogrids, bar mat, and wire mesh reinforcements. 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 carried by the reinforcement in two ways: in tension or in shear and bending.

The unfactored pullout resistance \( (P_r) \) of the reinforcement per unit width of reinforcement is estimated as:

\[
P_r = F^* \alpha \sigma_v
\]

Equation D-2

Parameters in Equation D-2 are defined in Equation D-16.

In this design step, the reinforcement pullout resistance is evaluated at each reinforcement level. The required total length for reinforcement to generate appropriate pullout resistance for each level is calculated and then, compared against the total reinforcement length initially estimated previously. The initially estimated total reinforcement length may have to be adjusted based on the required length calculated in this step.

### D.5.1.1 \( T_{al} \) for Inextensible Reinforcements

The nominal long-term design strength of inextensible reinforcement is provided below:

\[
T_{al} = \frac{A_c F_y}{b}
\]

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

The lower resistance factor of 0.65 (see Chapter 9) for grid reinforcement (as compared to a resistance factor of 0.75 (see Chapter 9) for strip reinforcement) accounts for the greater potential for local overstress due to load nonuniformities for steel grids than for steel strips or bars.

\( A_c \) for strips is determined as:

\[
A_c = b t_c = b(t_n - t_s)
\]

Equation D-4

Where,

- \( b \) = Unit width of sheet, grid, bar or mat
- \( t_c \) = Thickness at end of design life (see Figure D-3)
- \( t_n \) = Thickness at end of construction
- \( t_s \) = Sacrificial thickness of metal expected to be lost by uniform corrosion during the service life of the structure
When estimating $t_s$, 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.

D.5.1.2 Corrosion Rates

The corrosion rates presented below are suitable for conservative design. These rates assume a mildly corrosive backfill material having the controlled electrochemical property limits that are discussed previously.
Corrosion Rates – mildly corrosive backfill

For corrosion of galvanization on each side
- 0.58 mil./year/side (first 2 years)
- 0.16 mil./year/side (thereafter)

For corrosion of residual carbon steel on each side
- 0.47 mil./year/side (after zinc depletion)

Based on these rates, complete corrosion of galvanization with the minimum required thickness of 3.4 millimeters (mil.) (AASHTO M 111) 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. 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 environment 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 indicate that the upper 8 feet of the reinforced backfill (as measured from the roadway surface) 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.

D.5.1.3 \( T_{al} \) for Extensible Reinforcements

The nominal long-term design strength of extensible reinforcement is provided below:

\[
T_{al} = \frac{T_{ult}}{RF}
\]

Equation D-6

Where,
- \( T_{ult} \) = Minimum average roll value ultimate tensile strength
- \( RF \) = Combined strength reduction factor to account for potential long-term degradation due to installation damage, creep and chemical aging

\[
RF = RF_{ID}RF_{CR}RF_{D}
\]

Equation D-7

Where,
- \( RF_{ID} \) = Strength reduction factor to account for installation damage to reinforcement
- \( RF_{CR} \) = Strength reduction factor to prevent long-term creep rupture of reinforcement
- \( RF_{D} \) = Strength reduction factor to prevent rupture of reinforcement due to chemical and biological degradation
According to AASHTO (2007) values of $RF_{ID}$, $RF_{CR}$, $RF_{D}$ shall be determined from product specific test results.

D.5.1.4 Reduction Factor $RF_{ID}$

The following sections of this Appendix are adopted directly from *Earth Retaining Structures*, FHWA-NHI-07-071, June 2008 and *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction* (MSEW and RSS D&C), FHWA-NHI-00-043, March 2001 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.

$RF_{ID}$ can range from 1.05 to 3.0 depending on backfill gradation and product mass per unit weight. Even with product specific test results, the minimum reduction factor shall be 1.1 to account for testing uncertainties. The placement and compaction of the backfill material against the geosynthetic reinforcement may reduce its tensile strength. The level of damage for each geosynthetic reinforcement is variable and is a function of the weight and type of the construction equipment and the type of geosynthetic material. The installation damage is also influenced by the lift thickness and type of soil present on either side of the reinforcement. Where granular and angular soils are used for backfill, the damage is more severe than where softer, finer, soils are used. For a more detailed explanation of the $RF_{ID}$ factor, see *Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes*, FHWA-NHI-00-044.

D.5.1.5 Reduction Factor $RF_{CR}$

$RF_{CR}$ is obtained from long-term laboratory creep testing as detailed in Elias et al. (2001). This reduction factor is required to limit the load in the reinforcement to a level known as the creep limit, that will preclude creep rupture over the life of the structure. Creep in itself does not degrade the strength of the polymer. 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. The creep reduction factor is the ratio of the ultimate load to the extrapolated maximum sustainable load (i.e., creep limit) within the design life of the structure (e.g., several years for temporary structures, 75 to 100 years for permanent structures). Typical reduction factors as a function of polymer type are indicated in Table D-5.

<table>
<thead>
<tr>
<th>Polymer Type</th>
<th>$RF_{CR}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Polyester</td>
<td>1.6 to 2.5</td>
</tr>
<tr>
<td>Polypropylene</td>
<td>4.0 to 5.0</td>
</tr>
<tr>
<td>Polyethylene</td>
<td>2.6 to 5.0</td>
</tr>
</tbody>
</table>

Table D-5, Creep Reduction Factors (Earth Retaining Structures – June 2008)

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.5.1.6 Reduction Factor $RFD$

$RFD$ is dependent on the susceptibility of the geosynthetic to be attacked by microorganisms, chemicals, thermal oxidation, hydrolysis and stress cracking and can vary typically from 1.1 to 2.0. Even with product specific tests results, the minimum reduction factor shall be 1.1. A protocol for testing to obtain this reduction factor has been described in Elias et al. (1997).

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. (Note: The resistance factor ($\phi$) is the inverse of the Factor of Safety ($\phi = 1/FS$).) If the resistance factor is greater than indicated in Chapter 9, reinforcement of the slope is required. Further this stability check also identifies potential deep-seated failures. Deep-seated failure surfaces extend into the foundation soil and may require some form of ground improvement (see Chapters 19 and 20). Finally, this stability analysis is used to determine the first estimate of the length of reinforcement required to stabilize the slope.

This stability can be determined using classical slope stability analyses (see Chapter 17). The failure surfaces may be circular or non-circular (sliding block) and both should be checked. Overall stability analyses are performed for the Service limit state. The results of the overall stability analysis can, and does, effect the reinforcement length used in the design. It should be noted that it is assumed that all RSS are free draining and that pore water pressures are not allowed to build up behind the face of the slope.

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 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; however, 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 MSEW and RSS D&C:

Calculate the total reinforcement tension per unit width of slope $T_S$ required to obtain the required resistance factor $1/\phi$, 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:
\[ T_s = \left( \frac{1}{\varphi_r} - \frac{1}{\varphi_u} \right) \frac{M_D}{D} \]  

Equation D-8

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
- \( 1/\varphi_r \) = Target minimum slope resistance factor which is applied to both the soil and reinforcement
- \( 1/\varphi_u \) = Unreinforced slope resistance factor

The largest \( T_s \) calculated establishes the maximum total design tension (\( T_{S\text{-max}} \)) required. Determine the total design tension per unit width of slope (\( T_{S\text{-MAX}} \)) using Figure D-5. Compare \( T_{S\text{-MAX}} \) from the chart to \( T_{S\text{-max}} \) calculated from Equation D-8, if the results are significantly different, check the assumptions listed on the figure and check the calculations in the previous step (Unreinforced Stability) to determine if there is a larger potential failure surface not originally included in the analysis.
Note: $FS_R = 1/\phi_R$, where $\phi_R$ is the resistance factor (see Chapter 9)

**Figure D-5, Reinforcement Strength Requirements Chart Solution**
*(MSEW and RSS D&C – March 2001)*
According to MSEW and RSS D&C”

Figure D-5 is provided for a quick check of computer-generated results. The figure presents a simplified method based on a two-part wedge type failure surface and is limited by the assumptions noted on the figure.

Note that Figure D-5 is not intended to be a single design tool. Other design charts that are available from the literature, could also be used (e.g. Werner and Resl, 1986; Ruegger, 1986; Leshchinsky and Boedeker, 1989; and Jewell, 1990). Several computer programs are also available (see Section D.12) 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 geotextile, the determination of the distribution of the reinforcement comes next. According to MSE and RSS D&C:

For low slopes \((H \leq 20 \text{ feet})\) assume a uniform reinforcement distribution and use \(T_{S\text{-max}}\) to determine the spacing or the required tension, \(T_{\text{max}}\), requirements for each reinforcement layer.

For high slope \((H > 20 \text{ feet})\), divide the slope into two (top and bottom) or three (top, middle and bottom) reinforcement zones of equal height and use a factored \(T_{S\text{-max}}\) in each zone for spacing or design tension requirements. The total required tension in each zone is found from:

For two zones:

\[
T_{Bottom} = \frac{3}{4} T_{S\text{-max}} \quad \text{Equation D-9}
\]

\[
T_{Top} = \frac{1}{4} T_{S\text{-max}} \quad \text{Equation D-10}
\]

For three zones:

\[
T_{Bottom} = \frac{1}{2} T_{S\text{-max}} \quad \text{Equation D-11}
\]

\[
T_{Middle} = \frac{1}{3} T_{S\text{-max}} \quad \text{Equation D-12}
\]

\[
T_{Top} = \frac{1}{6} T_{S\text{-max}} \quad \text{Equation D-13}
\]

The force \((T_{S\text{-max}})\) is assumed to be uniformly distributed over the entire zone.

Determine reinforcement vertical spacing \((S_v)\) or the maximum design tension \((T_{\text{max}})\) requirements for each reinforcement layer.
For each zone, calculate $T_{\text{max}}$ 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:

$$T_{\text{max}} = \frac{T_{\text{zone}} S_V}{H_{\text{zone}}} = \frac{T_{\text{zone}}}{N} \leq T_s R_c$$  

Equation D-14

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
- $T_{\text{zone}}$ = Maximum reinforcement tension required for each zone; $T_{S\text{-max}}$ for low slopes ($H \leq 20$ feet)
- $T_a = T_{al}$
- $H_{\text{zone}}$ = Height of zone; $T_{\text{Top}}$, $T_{\text{Middle}}$, and $T_{\text{Bottom}}$ for high slopes ($H > 20$ feet)
- $N$ = Number of reinforcement layers

Use short (4 to 6 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 ($45^\circ$), 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 spacings 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:

$$\frac{1}{\phi} = FS = \frac{c' H + (\gamma_g - \gamma_w) H z \cos^2 \beta \tan \phi' + F_g \left( \cos \beta \sin \beta + \sin^2 \tan \phi' \right)}{\gamma_g H z \cos \beta \sin \beta}$$  

Equation D-15

Where,

- $c'$ = Effective cohesion
- $\phi'$ = Effective friction angle
- $\gamma_g$ = Saturated unit weight
- $\gamma_w$ = Unit weight of water
- $z$ = Vertical depth to failure plane defined by the depth to saturation
- $H$ = Vertical slope height
- $\beta$ = 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 M-288) and provide localized tensile reinforcement to the surficial soils.

If the interface friction angle of the intermediate reinforcements, \( \rho_{sr} \), is less than that of the primary reinforcement \( \rho_r \), then \( \rho_{sr} \) 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-8 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 two or three 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-5(b). Note: \( L_e \) is already included in the total length, \( L_T \) and \( L_B \) from Figure D-5(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.7.1.1 Estimating \( L_e \)

The length of reinforcement in the resisting zone (\( L_e \)) is determined using the following equation:

\[
L_e \geq \frac{T_{\text{max}}}{\phi F* \alpha \sigma_v^* 2CR_c}
\]  
Equation D-16

Where,

- \( T_{\text{max}} \) = Maximum factored tensile load in the reinforcement (calculated in Equation D-8)
- \( \phi \) = Resistance factor for reinforcement pullout (see Chapter 9)
- \( F* \) = Pullout friction factor (discussed below)
- \( \alpha \) = Scale effect correction factor (discussed below)
- \( \sigma_v^* \) = Unfactored 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 (see Figure D-7)
\[ R_e = \frac{b}{S_h} \] \hspace{1cm} \text{Equation D-17}

Where,
\[ b = \text{Gross width of the reinforcing element} \]
\[ S_h = \text{Center-to-center horizontal spacing between reinforcements} \]

D.7.1.2 Correction Factor (\( \alpha \))

The correction factor (\( \alpha \)) depends primarily upon the strain softening of the compacted granular backfill material, the extensibility, and the length of the reinforcement. Typical values of \( \alpha \) based on reinforcement type are presented in Table D-6. For inextensible reinforcement, \( \alpha \) is approximately 1, but it can be substantially smaller than 1 for extensible reinforcements. The \( \alpha \) 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-6 should be used for geogrids and geotextiles.

<table>
<thead>
<tr>
<th>Reinforcement Type</th>
<th>( \alpha )</th>
</tr>
</thead>
<tbody>
<tr>
<td>All steel reinforcements</td>
<td>1.0</td>
</tr>
<tr>
<td>Geogrids</td>
<td>0.8</td>
</tr>
<tr>
<td>Geotextiles</td>
<td>0.6</td>
</tr>
</tbody>
</table>

D.7.1.3 Pullout Friction Factor (\( F^* \))

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:

\[ F^* = F_q \alpha_\beta + \tan \rho \] \hspace{1cm} \text{Equation D-18}

Where,
\[ F_q = \text{The embedment (or surcharge) bearing capacity factor} \]
\[ \alpha_\beta = \text{A bearing factor for passive resistance which is based on the thickness per unit width of the bearing member} \]
\[ \rho = \text{The soil-reinforcement interaction friction angle} \]

Equation D-18 represents systems that have both the frictional and passive resistance components of the pullout resistance. In certain systems, however, one component is
much smaller than the other and can be neglected for practical purposes. In absence of site-specific pullout test 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:

$$F^* = \tan \rho = 1.2 + \log C_u$$

at the top of the structure = 2.0 maximum

$$F^* = \tan \varphi_r$$

at a depth of 20 feet and below

Where,

- $\rho$ = Interface friction angle mobilized along the reinforcement
- $\varphi_r$ = 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 ($S_t$) $\geq$ 6 inches, $F^*$ is a function of a bearing or embedment factor ($F_q$), applied over the contributing bearing factor ($\alpha\beta$), as follows:

$$F^* = F_q \alpha\beta = 40 \frac{t}{2S_t} = 20 \left(\frac{t}{S_t}\right)$$

at the top of the structure

$$F^* = F_q \alpha\beta = 20 \frac{t}{2S_t} = 10 \left(\frac{t}{S_t}\right)$$

at a depth of 20 feet and below

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, with the reduction factor often referred to as an interaction factor ($C_i$). In the absence of test data, the $F^*$ value for geosynthetic reinforcement should conservatively be estimated as:

$$F^* = 0.67 \tan \varphi_r$$
The relationship between $F^*$ and depth below the top of the wall for different reinforcement types is summarized in Figure D-6.

![Diagram showing typical values of $F^*$ for different reinforcement types.]

Figure D-6, Typical Values of $F^*$
(Earth Retaining Structures – June 2008)
D.8 SELECTION OF REINFORCEMENT

In this step, the type of reinforcement shall be determined. The two 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). 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.

D.9 EXTERNAL STABILITY

D.9.1 Sliding Resistance

According to MSE and RSS D&C:

Evaluate the width of the reinforced soil mass at any level to resist sliding along the reinforcement. A 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. The back of the wedge should be angled at 45° +φ/2 (see Figure D-8) or parallel to the back of the reinforced zone, which ever is flatter (i.e., the wedge should not pass through layers of reinforcement to avoid an overly conservative design).

![Figure D-8, Sliding Stability Analysis (MSEW and RSS D&C – March 2001)](image)

A simple analysis using a sliding block method can be performed as a check. In this method, an active wedge is assumed at the back of the reinforced soil mass with the back of the wedge extending up at an angle of 45° +φ/2. Using this assumption, the driving force is equal to the active earth pressure and the resisting force is the frictional resistance provided by the weakest layer, either the reinforced soil, the foundation soil or the soil-reinforcement interface. The following relationships are then used:

\[
\varphi \left( W + P_a \sin \phi_b \right) \tan \phi_{\text{min}} = P_a \cos \phi_b
\]

Equation D-25

Resistance Factor(Resisting Force) = Sliding Force

Equation D-24
\[ W = \frac{1}{2} L^2 \gamma_r \tan \theta \]  
\text{Equation D-26}

for \( L < H \)

\[ W = \left[ LH - \frac{H^2}{(2 \tan \theta)} \right] \gamma_r \]  
\text{Equation D-27}

for \( L > H \)

\[ P_a = \frac{1}{2} \gamma_b H^2 K_a \]  
\text{Equation D-28}

Where,
- \( L \) = Length of bottom reinforcing layer in each level where there is a reinforcement length change
- \( H \) = Height of Slope
- \( \varphi \) = Resistance Factor (see Chapter 9)
- \( \varphi_{\text{min.}} \) = Minimum angle of shearing friction either between reinforced soil and reinforcement or the friction angle of the foundation soil
- \( \theta \) = Slope angle
- \( \gamma_r \) & \( \gamma_b \) = Unit weight of the reinforced backfill and retained backfill, respectively
- \( \varphi_b \) = Friction angle of retained fill (Note: If drains/filters are placed on the backslope, then \( \varphi_b \) equals the interface friction angle between the geosynthetic and retained fill)

D.9.2 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.9.3 Local Bearing Failure at Toe

According to MSE and RSS D&C:

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-9), the resistance factor against failure by squeezing may be calculated from:

\[ \varphi_{\text{squeezing}} = \frac{\gamma D_s \tan \theta}{2c_u} + \frac{H \gamma}{4.14c_u} \leq 0.75 \]  
\text{Equation D-29}

Where,
- \( \theta \) = Angle of slope
\( \gamma \) = Unit weight of soil in slope

\( D_s \) = Depth of soft soil beneath slope base of the embankment

\( H \) = Height of slope

\( c_u \) = Undrained shear strength of soft soil beneath slope

D.9.4 Foundation Settlement

Caution is advised and rigorous analysis (i.e., numerical modeling) should be performed when the resistance factor \( (\phi) \) 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.10 WALL DRAINAGE SYSTEM DESIGN

The following section of this Appendix is adopted directly from Earth Retaining Structures, FHWA-NHI-07-071, June 2008 and Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction (MSEW and RSS D&C), FHWA-NHI-00-043, March 2001 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.10.1 Subsurface Water Control

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-10. Geocomposite drainage systems or conventional granular blanket and trench drains could be used. Granular drainage systems are not addressed in this Appendix.
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:

1. Geotextile filtration/clogging
2. Long-term compressive strength of polymeric core
3. Reduction of flow capacity due to intrusion of geotextile into the core
4. Long-term inflow/outflow capacity

Procedures for checking geotextile permeability and filtration/clogging criteria are presented in Geosynthetic Design and Construction Guidelines, August 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. 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.10.2 Surface Water Runoff

Surface water runoff should be collected above the reinforced slope and channeled or piped below the base of the slope. 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:

$$\lambda = d \gamma_w s$$

Equation D-30

Where,

- $\lambda = \text{Tractive shear stress}$
- $d = \text{Depth of water flow}$
- $\gamma_w = \text{Unit weight of water}$
- $s = \text{The vertical to horizontal angle of slope face}$

For $\lambda < 0.0145 \text{ pound per square inch (psi)}$, consider vegetation with temporary or permanent erosion control mat. For $\lambda > 0.0145 \text{ psi}$, 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 ultra-violet 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.11 SEISMIC DESIGN

The seismic stability design shall conform to the requirements of Chapters 13 and 14. All load and resistance factors shall conform to Chapters 8 and 9. In addition, all displacements shall conform to Chapter 10.

D.12 COMPUTER SOFTWARE

The following section of this Appendix is adopted directly from Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction (MSEW and RSS D&C), FHWA-NHI-00-043, March 2001 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.

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-5 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 three 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 a diskette 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.  
   33 The Horseshoe, Covered Bridge Farms  
   Newark, Delaware 19711, USA  
   Tel. (302) 368-3197, Fax (302) 731-1001

D.13 REFERENCES


