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APPENDIX C
MECHANICALLY STABILIZED EARTH WALL
DESIGN GUIDELINES

C.1 INTRODUCTION

This document outlines SCDOT’s design methodology for Mechanically Stabilized Earth (MSE) Walls. MSE wall structures are internally stabilized fill walls constructed of alternating layers of compacted soil and reinforcement. The design of MSE walls follows the design steps provided in Chapter 18. This Appendix governs the design of permanent and temporary MSE wall structures. The design life of both permanent and temporary MSE walls is provided in Chapter 18. The design responsibilities of the SCDOT (or its representative) and the MSE wall supplier are outlined with respect to external and internal stability of the MSE wall structure.

C.2 DESIGN CONSIDERATIONS AND REQUIREMENTS

The first part of the design is determining if an MSE wall is appropriate for the application being planned. If an MSE wall is appropriate, determine 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) and should establish wall stationing as needed. The geometry should also indicate the anticipated top and base of the wall, as well as slopes that tie into the wall. During this step of the design process, 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 performance limits are provided in Chapter 10. Any constraints on construction should also be identified during this step (for example, soft ground, standing water, limited ROW, utilities, etc). These construction constraints should be carefully considered before deciding to use an MSE wall.

C.3 SITE CONDITIONS

The second step in the design of MSE walls is the evaluation of the topography, subsurface conditions, in-situ soil/rock parameters and the parameters for the backfill. The evaluation of the topography should include reviewing the height requirements of the wall, the amount of space between the front of the MSE wall 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 MSE wall (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. The reinforced backfill to be used to construct the MSE wall shall meet the criteria provided below.
### Table C-1, MSE Wall Reinforced Backfill Properties

<table>
<thead>
<tr>
<th>Material Property</th>
<th>Granular Backfill</th>
<th>Stone Backfill</th>
</tr>
</thead>
<tbody>
<tr>
<td>Internal Friction Angle(^1)</td>
<td>32° - 34°</td>
<td>36° - 38°</td>
</tr>
<tr>
<td>Total Unit Weight (lbs./cubic foot)</td>
<td>120</td>
<td>110</td>
</tr>
</tbody>
</table>

\(^1\)Based on Triaxial testing of samples recompacted to 95 percent of the Standard Proctor

### Table C-2, Granular Backfill Gradation Requirements

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1/2 in(^1)</td>
<td>100</td>
</tr>
<tr>
<td>3/4 in(^2)</td>
<td></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>(\leq 6)</td>
</tr>
<tr>
<td>Liquid Limit</td>
<td>(\leq 30)</td>
</tr>
<tr>
<td>CU(^3)</td>
<td>(\geq 4)</td>
</tr>
<tr>
<td>Organic Content</td>
<td>(&lt; 1%)</td>
</tr>
</tbody>
</table>

\(^1\)Inextensible reinforcement

\(^2\)Extensible reinforcement

\(^3\)CU = D\(_{60}\)/D\(_{10}\)

\(^4\)Pullout or additional internal friction testing required for CU less than 4

### Table C-3, Stone Backfill Gradation Requirements

<table>
<thead>
<tr>
<th>Reinforcement Material</th>
<th>Coarse Aggregate No.(^1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geosynthetic(^2)</td>
<td>67, 6M</td>
</tr>
<tr>
<td>Metallic(^3)</td>
<td>5, 57, 67, 6M</td>
</tr>
</tbody>
</table>

\(^1\)Meets the requirements of the SCDOT Standard Construction Specifications (latest edition)

\(^2\)Extensible reinforcement (polyester and/or polyolefin)

\(^3\)Inextensible reinforcement

### Table C-4, 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>Resistivity(^1)</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(^2)</td>
<td>pH</td>
<td>3.5 &lt; pH &lt; 9</td>
</tr>
<tr>
<td>Metallic/Geosynthetic(^3)</td>
<td>pH</td>
<td>4.5 &lt; pH &lt; 10</td>
</tr>
</tbody>
</table>

\(^1\)Chloride and Sulfate testing are not required if the resistivity is greater than 5,000 ohm-cm

\(^2\)Granular Backfill

\(^3\)Stone Backfill
Table C-5, Temporary MSE Granular Backfill Properties

<table>
<thead>
<tr>
<th>Gradation</th>
<th>Plasticity Index</th>
<th>Liquid Limit</th>
<th>$C_u$</th>
<th>Internal Friction Angle</th>
<th>Total Unit Weight (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>¾ in No. 200</td>
<td>≤ 15</td>
<td>≤ 30</td>
<td>≥ 4</td>
<td>28° - 30°</td>
<td>115</td>
</tr>
<tr>
<td>100% ≤ 30%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

C.4 INITIAL WALL GEOMETRY

The third step in the design of MSE walls is establishing the initial geometry of the MSE wall. Figure C-1 provides the terminology for MSE wall geometry. The height ($H$) of an MSE wall is measured vertically from the top of the MSE wall to the top of the leveling pad. MSE wall structures, with panel type facings, should not exceed heights of 40 feet, and with modular block type facings, should not exceed heights of 30 feet. Wall heights in excess of these limits will require approval from the GDS and the PCS/GDS. The length of reinforcement ($L$) is measured from the back of MSE wall panels. Alternately, the length of reinforcement ($B$) is measured from the front face for modular block type MSE walls. The minimum reinforcement length is 0.7$H$ or 8 feet whichever is greater. MSE wall structures with sloping surcharge fills or other concentrated loads will generally require longer reinforcement lengths of 0.8$H$ to 1.1$H$. MSE walls may be built to heights mentioned above; however, the external stability requirements may limit MSE wall height due to bearing capacity, settlement, or stability problems.

Figure C-1, MSE Wall Schematic
(Mechanically Stabilized Earth walls and Reinforced Soil Slopes Design & Construction Guidelines – March 2001)
The top of the leveling pad will require a minimum embedment below finished grade of 2 feet. Greater embedment depths may be required due to bearing capacity, settlement, stability, erosion, or scour requirements and if utilities, ditches, or other structures are located adjacent to the wall. The minimum embedment depths based on local bearing capacity considerations taking into account the geometry in front of the wall are presented in Table C-6.

<table>
<thead>
<tr>
<th>Slope in Front of Wall</th>
<th>Minimum Embedment Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal (walls)</td>
<td>H/20</td>
</tr>
<tr>
<td>Horizontal (abutments)</td>
<td>H/10</td>
</tr>
<tr>
<td>3H:1V</td>
<td>H/10</td>
</tr>
<tr>
<td>2H:1V</td>
<td>H/7</td>
</tr>
<tr>
<td>1.5H:1V</td>
<td>H/5</td>
</tr>
</tbody>
</table>

A minimum horizontal bench of 4 feet is required in front of the MSE wall structure, for MSE walls built on slopes. This minimum bench is required to protect against local instability at the base of the wall.

C.5  EXTERNAL STABILITY

The external stability analysis develops the unfactored and factored loads on an MSE wall and checks the eccentricity, sliding and bearing resistances. The external stability analyses cover the fourth step to the eighth step of the design process provided in Chapter 18. These analyses are normally performed by the geotechnical engineer-of-record, whether SCDOT personnel or consultant.

C.5.1 Unfactored Load Estimate

In this step, the geotechnical engineer-of-record is responsible for developing the unfactored loads that are used in the design of the MSE wall. These loads are the result of earth pressures induced by the retained fill materials and any surcharge loadings. The earth pressure loadings include the horizontal and vertical earth pressures and any soil surcharge loadings. There are three cases for the development of earth pressures; these are 1) horizontal backslope with traffic surcharge; 2) sloping backslope; and, 3) broken backslope. The surcharge loadings can include vehicle live loads, the loads imposed by a bridge, etc. These loading conditions are discussed in Chapter 8. In addition, Chapter 8 also provides some unit weights for materials that are used as surcharges. If a bridge is to be supported by shallow foundations, no portion of the spread footings shall be located above or within the reinforced soil mass of MSE walls.

C.5.1.1 Horizontal Backslope with Traffic Surcharge

The procedure for estimating the earth pressures acting on the back of the reinforced soil mass for the horizontal backslope with traffic surcharge is depicted in Figure C-2. The active earth pressure coefficient \( K_a \) for vertical walls (i.e., walls with less than 8° batter) with horizontal backfill is calculated according to the procedures provided in Chapter 18. When considering live load on MSE walls for this condition, the factored surcharge load is generally included over the reinforced soil mass during the evaluation of foundation bearing resistance, overall (global)
stability and tensile resistance of the reinforcement (see Figure C-2). The live load surcharge is not included over the reinforced soil mass in the evaluation of eccentricity, sliding, reinforcement pullout or other failure mechanisms for which the surcharge load increases the resistance to failure (i.e. increases stability). If the surcharge consists of point loads, Earth Retaining Structures, FHWA NHI-07-071, June, 2008.

\[
K_a = \frac{\sin^2(\theta + \phi')}{\Gamma \sin^2 \theta \sin(\theta - \delta)}
\]

Equation C-1

C.5.1.2 Sloping Backslope

The active earth pressure coefficient \(K_a\) changes when there is a slope behind the MSE wall. \(K_a\) is determined using the following equation:

\[
K_a = \frac{\sin^2(\theta + \phi')}{\Gamma \sin^2 \theta \sin(\theta - \delta)}
\]

Equation C-1
\[ \Gamma = \left[ 1 + \frac{\sin(\phi' + \delta)\sin(\phi' - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)} \right]^2 \]  
Equation C-2

Where,
- \( \beta \) = Nominal slope angle of backfill behind wall (see Figure C-3)
- \( \delta \) = Angle of wall friction
- \( \phi' \) = Friction angle of retained fill
- \( \theta \) = 90° for vertical wall

The force on the rear of the reinforced soil mass (\( F_T \)) and the resulting horizontal (\( F_H \)) and vertical (\( F_V \)) forces are determined from the following equations:

\[ F_T = \frac{1}{2} \gamma h^2 K_a \]  
Equation C-3

\[ F_H = F_T \cos \beta \]  
Equation C-4

\[ F_V = F_T \sin \beta \]  
Equation C-5

Where,
- \( \gamma \) = Unit weight of retained fill material
- \( h \) = See Figure C-3
- \( K_a \) = Determined in Equation C-1

Figure C-3, MSE Wall Earth Pressure for Sloping Backfill (Mechanically Stabilized Earth walls and Reinforced Soil Slopes Design & Construction Guidelines – March 2001)
C.5.1.3 Broken Backslope

For broken backslopes (see Figure C-4), the active earth pressure coefficient ($K_a$) is determined using Equation C-1. The force acting on the rear of the MSE wall, $F_T$, is determined using Equation C-3.


diagram of MSE wall earth pressure for broken backslope

C.5.2 Factored Loads

Portions of the following sections of this Appendix are adopted directly from Earth Retaining Structures – June 2008 and are used with the permission of the US Department of Transportation, Federal Highway Administration. The italics are added to reflect additions or modifications to the selected text and to supply references to this Manual. According to Earth Retaining Structures:

...the unfactored loads from the previous step are multiplied by load factors to obtain the factored loads for each limit state. The load factors for the limit state are provided in Chapter 8.

Load factors for permanent loads are selected to produce the maximum destabilizing effect for the design check being considered. For example, to produce the maximum destabilizing effect, when checking sliding resistance, $\gamma_{EV}$ is selected as the minimum value from Table 8-6 (i.e., $\gamma_{EV} = 1.00$) and when checking bearing resistance, $\gamma_{EV}$ is selected as the maximum value from Table 8-6 (i.e., $\gamma_{EV} = 1.35$)."
C.5.3 Eccentricity

According to Earth Retaining Structures:

The eccentricity of the wall \( e_B \) can be calculated for each load group as:

\[
e_B = \frac{B}{2} - X_o
\]

Equation C-6

Where,

- \( B \) = Base width (length of reinforcement elements)
- \( X_o \) = Location of the resultant from the toe of the wall (see Equation C-7)

The parameter \( X_o \) is calculated as:

\[
X_o = \left( \frac{M_{EV} - M_{HTOT}}{P_{EV}} \right)
\]

Equation C-7

Where,

- \( M_{EV} \) = Resisting moment due to factored vertical earth pressure calculated about the toe of the wall
- \( M_{HTOT} \) = Driving moment due to factored horizontal earth pressure from ground and factored live load surcharge calculated about the toe of the wall
- \( P_{EV} \) = Factored resultant force from vertical earth pressure due to the weight of reinforced soil

\( M_{EV} \) and \( M_{HTOT} \) are calculated using the factored loads. Example calculations for two different backslopes are given below:

For horizontal backslope with traffic surcharge (Figure C-5) condition:

\[
M_{EV} = P_{EV} \frac{L}{2}
\]

Equation C-8

\[
M_{HTOT} = \left( P_{EH} \frac{H}{3} \right) + \left( P_{LSH} \frac{H}{2} \right)
\]

Equation C-9

Where,

- \( P_{EH} \) = Factored resultant force from horizontal earth pressure
- \( P_{LSH} \) = Factored resultant force from horizontal component of surcharge load
It should be noted that the effect of external loadings on the MSE mass which increases sliding resistance, should only be included if the loadings are permanent. For example, live load traffic surcharges should be excluded.

For sloping backslope (Figure C-6) condition:

\[
M_{EV} = P_{EV1} \frac{L}{2} + P_{EV2} \frac{2L}{3} + P_{EH} \sin \beta L \quad \text{Equation C-10}
\]

\[
M_{HTOT} = P_{EH} \cos \beta \frac{h}{3} \quad \text{Equation C-11}
\]

Where,

- \( P_{EV2} \) = Factored resultant force from earth pressure due to the weight of soil of sloping backslope
- \( P_{EH} \sin \beta \) = Factored resultant force from vertical component of the earth pressure
- \( P_{EH} \cos \beta \) = Factored resultant force from horizontal component of the earth pressure
For Eccentricity to be considered acceptable, the calculated location of the resultant vertical force (based on factored loads) should be within the middle one-half of the base width for soil foundations (i.e., $e_{\text{max}} = B/4$) and middle three-fourths of the base width for rock foundations (i.e., $e_{\text{max}} = 3B/8$). Therefore, for each load group, $e_B$ must be less than $e_{\text{max}}$. If $e_B$ is greater than $e_{\text{max}}$, a longer length of reinforcement is required.

### C.5.4 Sliding Resistance

Step 7 of the MSE wall design consists of checking for sliding resistance. According to Earth Retaining Structures:

Sliding and overall stability usually govern the design of structures greater than about 30 feet high, structures constructed on weak foundation soils, or structures with a sloping surcharge.

The live load surcharge is not considered as a stabilizing force when checking sliding. The driving forces in a sliding evaluation will generally include factored horizontal loads due to earth, water, seismic, and surcharge pressures and the resisting force is provided by the minimum shear resistance between the base of the MSE wall and foundation soil.

Sliding along the base of the wall is evaluated using the procedures in Chapter 15 for spread footings.

It should be noted that any passive resistance provided by soil at the toe of the wall by embedment is ignored due to the potential for the soil to be removed through natural or manmade processes during the service life of the structure. The shear strength of the facing system is also conservatively neglected in most cases.
If the soil beneath the wall is cohesionless, the nominal sliding resistance \( R_t \) between the soil and foundation is:

\[
R_t = P_{EV} \tan \delta
\]

Equation C-12

Where,

\( P_{EV} = \) Minimum factored vertical load for the Strength limit state being considered

\( \delta = \) Coefficient of sliding friction at the base of the reinforced soil mass

For continuous sheet reinforcement, \( \delta \) is selected as the minimum of:

1. Friction angle of reinforced fill;
2. Friction angle of foundation soil; or
3. Interface friction angle between the reinforcement and soil.

For discontinuous reinforcement (e.g., geogrid), \( \delta \) is selected as the minimum value of (1) or (2) above.

Sliding resistance \( R_t \) of the MSE wall is considered adequate if \( R_t \) is equal to or greater than the maximum factored horizontal earth pressure force from the ground plus the factored live load surcharge calculated previously.

C.5.5 Bearing Resistance

Step eight of the MSE Wall design consists of checking the bearing resistance beneath the wall. According to Earth Retaining Structures:

Because of the flexibility of MSE walls and the inability of the flexible reinforcement to transmit moment, a uniform base pressure distribution is assumed over an equivalent footing width. Unlike the bearing resistance check for Cast-in-place walls founded on rock, the assumption of a uniform base pressure is used for MSE walls founded on rock. The effect of eccentricity, load inclination, and live load surcharges must be included in this check. Effects of live load surcharges are included because they increase the loading on the foundation.

The factored bearing resistance \( q_r \) is given as:

\[
q_r = \varphi q_n
\]

Equation C-13

Where,

\( \varphi = \) Resistance factor (see Chapter 9)
\( q_n = \) Nominal bearing resistance (see Chapter 15)

To check whether the bearing resistance of the MSE wall is adequate, the \( q_r \) computed previously is compared against the following criterion:
\[ q_r \geq q_{\text{uniform}} \] 

Equation C-14

Where,

\( q_{\text{uniform}} = \) Vertical stress for walls on soil foundations, which is calculated assuming a uniform distribution of pressure over an effective base width (\( B' \))

\[ q_{\text{uniform}} = \frac{V_{\text{TOT}}}{B'} \]

Equation C-15

Where,

\( V_{\text{TOT}} = \) Sum of all factored vertical forces acting at the base of the wall (e.g., weight of reinforced fill, live and dead load surcharges, \( \text{etc.} \))

and,

\[ B' = B - 2e \]

Equation C-16

Where,

\( B = \) Length of reinforcement
\( e = \) Eccentricity determined from \( \text{Equation C-6} \), however, for bearing resistance calculations, \( X_o \) is defined as:

\[ X_o = \frac{M_{\text{VTOT}} - M_{\text{HTOT}}}{V_{\text{TOT}}} \]

Equation C-17

Where,

\( M_{\text{VTOT}} = \) Resisting moment due to factored total vertical load based on earth pressure and live load surcharge calculated about the toe of the wall
\( M_{\text{HTOT}} = \) Driving moment due to factored lateral load based on earth pressure and live load surcharge calculated about the toe of the wall

C.6 INTERNAL STABILITY

The internal stability analysis is the ninth step to the thirteenth step of the design process provided in Chapter 18. These analyses are normally performed by the MSE wall supplier or manufacturer. According to **Earth Retaining Structures:**

“To be internally stable, the MSE structure must be coherent and self supporting under the action of its own weight and any externally applied forces. This is accomplished through stress transfer from the soil to the reinforcement. This interaction between the soil and reinforcement improves the tensile properties and creates a composite material with the following characteristics:

- Stress transfer between the soil and reinforcement takes place continuously along the reinforcement; and
- Reinforcements are distributed throughout the soil mass with a degree of regularity and must not be localized.
Figure C-7 illustrates the internal failure mechanisms for MSE walls. At each reinforcement level, the reinforcement must be sized and spaced to preclude rupture under the stress it is required to carry and to prevent pullout from the soil mass. The process of sizing and designing to preclude internal failure, therefore, consists of determining the maximum developed tension forces in the reinforcements (i.e., maximum load), the location of the load against the resistance provided by the reinforcements both in pullout and tension.

![Figure C-7: MSE Wall Internal Failure Mechanisms](Earth Retaining Structures – June 2008)

C.6.1 Critical Failure Surface Location

The location and shape of the theoretical critical failure surface is dependent on the type of reinforcement. Therefore, at this stage of the design, the reinforcement type must be selected first, categorized (i.e., extensible or inextensible) and, then, the potential critical failure surface can be calculated.

When inextensible reinforcements are used, the soil deforms more than the reinforcement. Therefore, the soil strength in this case is measured at low strain. The critical failure surface for this reinforcement type is determined by dividing the reinforced zone into active and resistant zones with a bilinear failure surface as shown in Figure C-8a.

When extensible reinforcements are used, the reinforcement deforms more than the soil. Therefore, it is assumed that the shear strength of the reinforced fill is fully mobilized (residual strength) and active lateral earth pressures are developed. As a result, the critical failure surface for both horizontal and sloping backfill conditions are represented by the Rankine active earth pressure zone as shown in Figure C-8b.
The purpose of this design step is to calculate the maximum factored horizontal stress. It is specifically noted that load factors are typically applied to unfactored loads, not to an unfactored stress (as it is in Equation C-18) below. The AASHTO (2007) LRFD code, however, applies the “load” factor to the unfactored “stress” for this particular design calculation. The factored horizontal stress ($\sigma_H$) at each reinforcement is based on Equation C-18.

$$\sigma_H = \gamma_p (\sigma_v k_r + \Delta \sigma_H)$$

Equation C-18
Where,
\[ \gamma_p = \text{Maximum load factor for vertical earth pressure (EV) (see Chapter 8)} \]
\[ k_r = \text{Lateral earth pressure coefficient from Equation C-20} \]
\[ \sigma_v = \text{Pressure due to resultant of gravity forces from soil unit weight within and immediately above the reinforced wall backfill, and any surcharge loads present} \]
\[ \Delta \sigma_h = \text{Horizontal stress at reinforcement level resulting in a concentrated horizontal surcharge load (refer to Article 11 of AASHTO)} \]

Research studies have indicated that the maximum tensile force is primarily related to the type of reinforcement in the MSE mass, which, in turn, is a function of the modulus extensibility, and density of reinforcement. Based on this research, a relationship between the type of reinforcement and the overburden stress has been developed and is shown in Figure C-9.

![Figure C-9, Variation of the Coefficient of Lateral Stress Ratio (Earth Retaining Structures – June 2008)](image)

Figure C-9 was prepared by back calculation of the lateral stress ratio from available field data where stresses in the reinforcements have been measured and normalized as a function of an active earth pressure coefficient. The lines shown on this figure correspond to usual values representative of the specific reinforcement systems that are known to give satisfactory results, assuming that the vertical stress is equal to the weight of the overburden.

For a vertical wall face (i.e., batters less than 8 degrees from vertical), the active earth pressure coefficient \( K_a \) is determined from Equation C-19. For wall face batters equal to or greater than 8 degrees from the vertical, the simplified form of the Coulomb equation as presented in AASHTO (2007) and shown in Equation C-20, is used to calculate active earth pressure.
\[ K_a = \tan^2 \left( 45 - \frac{\phi_r}{2} \right) \]  
Equation C-19

\[ K_a = \frac{\sin^2 (\theta + \phi_r')}{\sin^3 \theta \left( 1 + \sin \frac{\phi_r'}{\sin \theta} \right)^2} \]  
Equation C-20

Where,
\( \theta \) = Inclination of the back of the facing as measured from the horizontal starting in front of the wall
\( \phi_r' \) = Friction angle of reinforced fill

The value of \( K_a \) in the reinforced soil mass is assumed to be independent of all external loads, even sloping fills. If testing of the site-specific select backfill is not available, the value of \( \phi_r \) used to compute the horizontal stress within the reinforced soil mass should not exceed 34°.

Once the value of \( K_a \) is known, the lateral earth pressure coefficient (\( k_r \)) that is used to compute \( \sigma_{H} \) at each reinforcement level is calculated as:

\[ k_r = \left( \frac{K}{K_a} \right) K_a \]  
Equation C-21

Where,
\( K/K_a \) = From Figure C-9
\( K_a \) = From Equation C-19 or C-20

If present, surcharge load should be added into the estimation of \( \sigma_V \). For sloping soil surfaces above the MSE wall section, the actual surcharge is replaced by a uniform surcharge equal to half of the height of the slope at the back of the reinforcements. For cases where concentrated vertical loads occur, refer to *Earth Retaining Structures* – June 2008 for computation of \( \sigma_V \).

### C.6.3 Maximum Factored Tensile Stress

The maximum tension in each reinforcement layer per unit width of wall (\( T_{\text{max}} \)) based on the reinforcement vertical spacing (\( S_V \)) is calculated as:

\[ T_{\text{max}} = \sigma_{H} S_V \]  
Equation C-22

Where,
\( \sigma_{H} \) = Factored horizontal load calculated using Equation C-18.
$T_{\text{max}}$ may also be calculated at each level for discrete reinforcements (metal strips, bar mats, grids, etc.) per a defined unit width of reinforcement as:

\[ T_{\text{max-R}} = \frac{\sigma_H S_V}{R_c} \]  
Equation C-23

Where,

- $R_c = \text{Reinforcement coverage ratio (b}/S_h) \text{ (see Figure C-10) }$
- $b = \text{Gross width of the reinforcing element}$
- $S_h = \text{Center-to-center horizontal spacing between reinforcements}$

![Diagram](image)

(a) Metal reinforcement  (b) Geosynthetic reinforcement

**Figure C-10, Definitions of $b$, $S_h$ and $S_V$**

(Earth Retaining Structures – June 2008)
C.6.4 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, stress transfer occurs through two mechanisms;

1. Friction along the soil-reinforcement interface (see Figure C-11a)
2. Passive soil resistance or lateral bearing capacity developed along the transverse sections of the reinforcement (see Figure C-11b)

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:

\[
P_r = F \cdot \alpha \sigma_v
\]

Equation C-24

Parameters in Equation C-24 are defined in Equation C-26.

In this design step, the reinforcement pullout resistance is evaluated at each reinforcement level of the MSE wall. The required total length for reinforcement to generate sufficient 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.

The total length of reinforcement \( (L) \) required for internal stability is determined as:

\[
L = L_e + L_a
\]

Equation C-25

Where,

\( L_e \) = Required length of reinforcement in resisting zone (i.e., beyond the potential failure surface)

\( L_a \) = Remainder length of reinforcement

C.6.4.1 Estimating \( L_e \)

The length of reinforcement in the resisting zone \( (L_e) \) is determined using the following equation:
\[ L_c \geq \frac{T_{\text{max}}}{\varphi F^* \alpha \sigma_v \sigma_c R_c} \]  

Equation C-26

Where,

- \( T_{\text{max}} \) = Maximum factored tensile load in the reinforcement (calculated by Equation C-22)
- \( \varphi \) = 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 Equation C-23 and Figure C-10)

C.6.4.2 Correction Factor (\( \alpha \))

The correction factor (\( \alpha \)) depends primarily upon the strain softening of the compacted granular backfill material, and the extensibility, and the length of the reinforcement. Typical values of \( \alpha \) based on reinforcement type are presented in Table C-7. 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 C-7 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>

Table C-7, Typical Values of \( \alpha \)  
(Earth Retaining Structures – June 2008)

C.6.4.3 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:

\[ F^* = F_q \alpha \beta + \tan \rho \]  

Equation C-27
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 C-27 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, $F^\ast$ is commonly estimated as:

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

at the top of the structure = 2.0 maximum

$$F^\ast = \tan \varphi_r$$

at a depth of 20 feet and below

Where,

- $\rho = \text{Interface friction angle mobilized along the reinforcement}$
- $C_u = \text{Uniformity coefficient of the backfill (see Chapter 6)}$
- If the specific $C_u$ for the wall backfill is unknown during design, a $C_u$ of 4 should be assumed (i.e., $F^\ast = 1.8$ 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), $F^\ast$ is a function of a bearing or embedment factor ($F_q$), applied over the contributing bearing factor ($\alpha_\beta$), as follows:

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

at the top of the structure

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

at a depth of 20 feet and below

Where,

- $t = \text{The thickness of the transverse bar}$
\[ S_t = \text{The distance between individual bars in steel grid reinforcement and shall be uniform throughout the length of the reinforcement, rather than having transverse grid members concentrated only in the resistance zone (see Figure C-12)} \]

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:

\[ F^* = 0.67 \tan \omega \]  
Equation C-32

Where,

\[ \omega = \text{Wall fill peak friction angle} \]

When used in the above relationship, \( \omega \) is the peak friction angle of the soil which, for MSE walls using select granular backfill, is taken as 34 degrees unless project specific test data substantiates higher values.

The relationship between \( F^* \) and depth below the top of the wall for different reinforcement types is summarized in Figure C-12.

![Figure C-12](image)

**Figure C-12, Typical Values of \( F^* \)**  
(Earth Retaining Structures – June 2008)

### C.6.4.4 Estimating \( L_a \)

The \( L_a \) is obtained from Figure C-8 for simple structures not supporting concentrated external loads such as bridge abutments. Based on this figure, the following relationships can be obtained for \( L_a \):
• For MSE walls with extensible reinforcement, vertical face, and horizontal backfill:

\[ L_a = (H - Z) \tan \left( 45 - \frac{\phi_r}{2} \right) \]

Equation C-33

Where,
\[ Z = \text{Depth to the reinforcement level} \]

• For walls with inextensible reinforcement form the base up to \( H/2 \):

\[ L_a = 0.6(H - Z) \]

Equation C-34

• For the upper half of a wall with inextensible reinforcements:

\[ L_a = 0.3H \]

Equation C-35

For ease of construction, based on the maximum total length required, a final uniform reinforcement length is commonly chosen. However, if internal stability controls the length, it could be varied from the base, increasing with the height of the wall to the maximum length requirement based on a combination of internal and maximum external stability requirements.

C.6.5 Long-Term Reinforcement Design Strength

In this design step, the maximum factored tensile stress in each reinforcement layer \( (T_{\text{max}}, \text{determined previously}) \) is compared to the nominal long-term reinforcement design strength as presented below:

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

Equation C-36

Where,
\[ \phi = \text{Resistance factor for tensile resistance (see Chapter 9)} \]
\[ R_c = \text{Reinforcement coverage ratio as defined in Equation C-23 and Figure C-10)} \]
\[ T_{al} = \text{Nominal long-term reinforcement design strength} \]

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

C.6.5.1 \( T_{al} \) for Inextensible Reinforcements

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

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

Equation C-37
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 C-38

Where,

\( b \) = Unit width of sheet, grid, bar or mat

\( t_c \) = Thickness at end of design life (see Figure C-13)

\( t_n \) = 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

![Figure C-13, Cross Section Area for Strips (Earth Retaining Structures – June 2008)](image)

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 C-39

Where,

\( N_b \) = Number of bars per unit width \( b \)

\( D^* \) = Bar diameter after corrosion loss (Figure C-14)
When estimating D*, it may be assumed that corrosion losses occur uniformly over the area of the bar.

![Figure C-14, Cross Section Area for Bars](Earth Retaining Structures – June 2008)

C.6.5.2 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 100-year design life, respectively. The designer of an MSE 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.
C.6.5.3  $T_{ai}$ for Extensible Reinforcements

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

$$T_{ai} = \frac{T_{ult}}{RF}$$  \hspace{1cm} \text{Equation C-40}

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

$$RF = RF_{ID}RF_{CR}RF_{D}$$  \hspace{1cm} \text{Equation C-41}

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.

C.6.5.4  $T_{ac}$ for Connection Strength

Allowable connection loads, $T_{ac}$, for various levels of soil reinforcement shall be determined at the anticipated vertical confining pressure at the wall face between the facing blocks. The height of the wall above the wall interface shall be used to compute the vertical confining pressure for walls with a nominal batter of 8 degrees or less. For walls with a nominal batter of more than 8 degrees, the vertical confining pressure is limited to the lesser of the height obtained by the "Hinge Height Method" as described in AASHTO or the height of the wall above the interface.

For metallic soil reinforcement, the factored tensile load applied to the reinforcement connection at the wall face ($T_{ac}$), shall be equal to the maximum factored reinforcement tension, $T_{max}$, as determined previously, for all wall systems regardless of the facing or reinforcement (bar or grid) type.

For geosynthetic soil reinforcement, check that the static allowable connection load, $T_{ac}$, multiplied by the reinforcement coverage ratio, $R_c$, ($R_c=1.0$ for geosynthetic sheet type reinforcement, i.e. geogrid) shall meet or exceed the maximum applied load to the soil reinforcement connection, $T_o$, at each level of reinforcement placement at the wall facing. The static allowable connection load, $T_{ac}$, for geosynthetic reinforcement is determined using the following design methodology.
The allowable connection strength, $T_{ac}$, per unit width of reinforcement at the connection shall be computed by determining the long-term connection strength through laboratory connection strength testing. The long-term connection strength may be determined by either a long-term connection test or by a quick load connection test in accordance with the procedures outlined in Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines FHWA NHI-00-043, March 2001. When the connection system has more than one type of material or component (i.e. connector, geogrid), the long-term connection strength shall be determined from the material or component that produces the lowest connection strength. The allowable connection strength, $T_{ac}$, shall not exceed the allowable tensile load, $T_a$, of the soil reinforcement multiplied by the connection coverage ratio, $C_c$. The connection coverage ratio, $C_c$, is defined as the width of the connectors within a block width divided by the width of the block.

When a long-term connection test is performed, the allowable connection strength is determined by the following equation.

$$T_{ac} = \frac{T_{ult} C_{CR} R_{RF}}{R_{DC}} \leq T_a C_c$$  \hspace{1cm} \text{Equation C-42}

When the quick load connection test is performed, the allowable connection strength is determined by the following equation.

$$T_{ac} = \frac{T_{ult} C_{CR, ult}}{R_{CR, DC}} \leq T_a C_c$$  \hspace{1cm} \text{Equation C-43}

### C.6.5.5 Reduction Factor $RF_{ID}$

$RF_{ID}$ 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 when 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.

### C.6.5.6 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 (maximum 5 years) or 100 years for permanent structures). Typical reduction factors as a function of polymer type are indicated in Table C-8.

<table>
<thead>
<tr>
<th>Polymer Type</th>
<th>RF&lt;sub&gt;CR&lt;/sub&gt;</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>

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<sub>CR</sub> of 5.0 shall be used.

C.6.5.7 Reduction Factor RF<sub>D</sub>

RF<sub>D</sub> 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 have been described in Elias et al. (1997).

C.6.5.8 Connection Strength Reduction Factors CR<sub>CR</sub> and CR<sub>ult</sub>

The long-term connection test strength reduction factor, CR<sub>CR</sub>, accounts for the reduced connection capacity at the end of the design life of the MSE wall structure due to connection failure and also includes reductions in material strength due to creep of the failed connection component. The connection strength reduction factors shall be certified based on the method of long-term connection testing selected. Either a long-term connection test or a quick load connection test may be used to certify the connection strength reduction factors in accordance with the procedures outlined in Appendix “A.3” of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines. When a connection system has more than one type of material or component (i.e. connector, geogrid), the connection strength reduction shall be determined from the material or component that produces the lowest long-term connection strength.

If a long-term connection test is used, the long-term strength reduction factor, CR<sub>CR</sub>, is computed by the following equation:

\[
CR_{CR} = \frac{T_{crc}}{T_{lot}}
\]

Equation C-44

Where,

\( T_{lot} = \) Ultimate wide width tensile strength of the soil reinforcement material lot used for long-term connection strength testing.
$T_{rc} = \text{Ultimate connection strength, } T_{ult}, \text{ at the end of a 1,000 hour connection test for each normal load is then extrapolated to the design life ultimate connection strength.}$

The quick load connection test strength reduction factor, $CR_{ult}$, accounts for the reduced connection capacity due to connection failure, without taking into account the reduction in strength due to creep of the failed connection component. If a quick load connection test is used, the quick load strength reduction factor, $CR_{ult}$, is computed by the following equation:

$$CR_{ult} = \frac{T_{ultconn}}{T_{lot}}$$  \hspace{1cm} \text{Equation C-45}

Where,

- $T_{lot} = \text{Ultimate wide width tensile strength of the soil reinforcement material lot used for quick load connection strength testing}$
- $T_{ultconn} = \text{Ultimate connection strength for each peak connection load at each normal load}$

### C.6.5.9 Creep ($RF_{CRC}$), Durability ($RF_{DC}$), and Combined Creep/Durability ($RF_{CRDC}$) Reduction Factors at Wall Facing Connections

The individual reduction factors for creep, $RF_{CRC}$, and durability, $RF_{DC}$, at the connection shall be documented in accordance with the MSE wall environmental conditions at the connection location. The reinforcement manufacturer shall certify the individual reduction factors for creep ($RF_{CRC}$) in accordance with the same procedures listed for creep reduction factor ($RF_{CR}$) and for durability ($RF_{DC}$) in accordance with the same procedures listed for individual reduction factor for durability ($RF_{D}$).

When long-term connection testing is performed and sufficient documentation is not provided for the durability reduction factor, $RF_{DC}$, the default durability reduction factor, $RF_{D}$, shall be used. When only the quick load connection test is performed and sufficient documentation is not provided for the combined reduction factors for creep and durability ($RF_{CRDC}$), the total default connection factor, $RF_{CDefault}$, shall be 6.0. The combined reduction factor for creep and durability ($RF_{CRDC}$) is computed by the following equation.

$$RF_{CRDC} = RF_{CRC} \cdot RF_{DC}$$  \hspace{1cm} \text{Equation C-46}

The individual reduction factors for creep, $RF_{CRC}$, and for durability, $RF_{DC}$, shall be the certified reduction factors for the environmental conditions that exist at the connection location.

### C.7 OVERALL STABILITY

The overall (global) stability is typically determined by the geotechnical engineer-of-record. This stability can be determined using classical slope stability analyses (see Chapter 17). The failure surfaces may be circular or non-circular and both should be checked. Typically, it is assumed that the failure surface does not pass through the reinforced mass of the MSE structure; therefore, the MSE structure is given strength parameters greater than the retained and foundation soils to prevent the failure plane from passing through the reinforced soil mass. Overall stability analyses are performed for the Service limit state and are normally performed...
once the initial estimate of the reinforcement length is determined from Step 3 (see Section C.4 above). The results of the overall stability analysis can and do affect the reinforcement length used in the design. It should be noted that it is assumed that all MSE walls are free draining and that pore water pressures are not allowed to build up behind the wall.

Prior to submission of the final design plans, 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 MSE wall face or that pass through the retained fill, reinforced soil mass and the foundation soil to exit beyond the toe of the MSE wall. The actual strength parameters that the reinforced soil mass is based on shall be used in the analysis. These analyses can only be performed once a specific MSE wall type is selected. This analysis may be performed by either the geotechnical engineer-of-record or by the MSE wall supplier.

The resistance factors (φ) for global stability analyses are provided in Chapter 9. MSE wall structures are considered Flexible Gravity Retaining Walls.

C.8 WALL DISPLACEMENT ESTIMATION

MSE wall structures can move both vertically and horizontally due to static and seismic loads. The movements caused by static loads are discussed here. See Chapter 14 for guidance on seismically induced movements. Vertical movements (settlement) should be determined using the procedures outlined in Chapter 17. In conditions where the reinforced soil mass will settle more than the face, the reinforcement should be placed on a sloping fill surface, which is higher at the backend of the reinforcement to compensate for the greater vertical movement in this area. The reinforcement connection strength shall be checked if there is any differential settlement between the MSE wall face and the rear of the reinforced soil mass. This differential settlement can induce additional stresses in the connections at the interface between the reinforcement and wall face materials.

Differential settlements perpendicular to the MSE wall facing (along the soil reinforcement) may occur at roadway widening projects. If this type of differential settlement exceeds a ratio of 1/10, the MSE wall suppliers shall be consulted to determine if further analyses are required. The values shown in Table C-9 shall be used as typical limiting differential settlement tolerances along the MSE wall facing for MSE wall structures with precast panel facings.

<table>
<thead>
<tr>
<th>Panel Joint Width</th>
<th>Limiting Differential Settlement</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot; *</td>
<td>1/100</td>
</tr>
<tr>
<td>1/2&quot; *</td>
<td>1/200</td>
</tr>
<tr>
<td>1/8&quot;*</td>
<td>1/300</td>
</tr>
<tr>
<td>Full Height Panel</td>
<td>1/500</td>
</tr>
</tbody>
</table>

*Note: Relatively square facing panels

MSE wall structures with modular concrete block facings are typically restricted to a limiting differential settlement of 1/200 along the MSE wall structure. Temporary MSE wall structures with welded wire mesh facing should be restricted to a limiting differential settlement along the MSE wall facing of 1/50.
Slip joints may be used to maintain MSE wall structures within acceptable differential settlement tolerances. When significant differential settlements are anticipated, ground improvement techniques such as surcharges and wick drains may be required to accelerate the consolidation settlement. Walls shall be designed for any temporary surcharge loading. When long-term settlements are accelerated during construction, temporary wall facings may be required during this accelerated settlement phase followed by installation of permanent facings after the required level of settlement is achieved.

MSE wall structures may experience lateral displacements during construction that may affect the MSE wall performance. These lateral displacements are a function of overall structure stiffness, compaction intensity, soil type, reinforcement length, slack in reinforcement-to-facing connections, and deformability of the facing system. The MSE wall system supplier is responsible for determining the construction batter that is required to meet the geometry requirements shown in the plans. Estimates of lateral wall displacements occurring during construction may be necessary during plan preparation to determine minimum clearances between the wall face and adjacent structures. Based on a reinforcement embedment ratio of 0.7H, the following equations may be used to make a rough estimate of probable lateral displacement, $\delta_{est}$ for inextensible and extensible reinforcement, respectively:

$$\delta_{est} = \frac{H}{250}$$  Equation C-47

$$\delta_{est} = \frac{H}{75}$$  Equation C-48

Where,

$\delta_{est} =$ Estimated lateral displacement (ft)

$H =$ Height of wall (ft)

**C.9 WALL DRAINAGE SYSTEM DESIGN**

The following section is adopted directly from *Earth Retaining Structures – June 2008* and is used with the permission of the US Department of Transportation, Federal Highway Administration. The italics are added to reflect additions or modifications to the selected text and to supply references to this Manual.

A drain at the base of the wall immediately behind the wall facing as shown in Figure C-15, is normally used with an MSE wall. This drain primarily serves to collect surface water that has infiltrated behind the facing and transports the water to an outlet. Outlet may be via weep holes, as shown in Figure C-15, or may be piped to a downslope outlet or to a storm sewer.
This drain may also serve to drain the wall fill if it is relatively free-draining and the wall is used in a fill situation. A drain behind the wall backfill, as illustrated in Figure C-16, should be used when zones of soils (in-situ or retained backfill to wall) are not free-draining relative to one another. A drain is recommended to collect and divert groundwater from the reinforced fill for side hill construction, where in-situ soils are excavated to accommodate the reinforced fill volume. Additionally, a drain at the backcut or at the wall fill and retained backfill interface (for fill situations) is recommended, unless soil permeabilities, filtration, and water flow are analyzed to ensure system is free draining. Soil filtration and permeability requirements must be met between the two adjacent zones of (different) soils to prevent impeded flow.
A geomembrane barrier beneath the pavement structure, sloping to a drain, should be used where significant de-icing salts are used. The purpose of the barrier is to prevent, or minimize, the infiltration of water immediately behind the wall facing. A common detail is shown in Figure C-17.

![Figure C-17, Impervious Geomembrane Details (Earth Retaining Structures – June 2008)](image)

C.10 SEISMIC DESIGN

The seismic external stability design shall conform to the requirements of Chapters 13 and 14. The seismic internal stability calculations shall conform to the requirements contained in the AASHTO LRFD Bridge Design Specifications (Section 11.10 – Mechanically Stabilized Earth Walls), except all accelerations used shall conform to the requirements of this Manual. Additionally, all load and resistance factors shall conform to Chapters 8 and 9 and all displacements shall conform to Chapter 10.

C.11 COMPUTER SOFTWARE

A complete set of the MSE wall system supplier’s design calculations prepared in accordance with this Appendix shall be provided by the MSE wall system supplier. The determination of all loading conditions and assumptions shall be fully documented with all design calculations. Submitted calculations (including computer runs) shall include all load cases that exist during construction and at the end of construction for any surcharges, hydraulic conditions, live loads, combinations, and obstructions within the reinforced backfill. Computer generated designs made by software other than FHWA’s MSEW 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. 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 MSE wall structure. Calculations may be computer generated provided that all input, equations, and assumptions used are shown clearly.
2. A diskette with the input files and the full computer output of the FHWA sponsored computer program MSEW (latest version) for the governing loading condition for each design section of the MSE wall 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

### C.12 REFERENCES

