APPENDIX C

MSE WALLS

GEOTECHNICAL DESIGN MANUAL

January 2022

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APPENDIX C

MECHANICALLY STABILIZED EARTH WALL DESIGN GUIDELINES

C.1 INTRODUCTION

This Appendix outlines SCDOT's design methodology for MSE Walls. MSE wall structures are internally stabilized, flexible gravity, 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 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 (see Chapter 18 for ERS selection criteria). If an MSE wall is appropriate, determine any aesthetic requirements, the geometry, the external loading conditions, the performance criteria, and any construction constraints. Aesthetic requirements include the finish and color of the facing material. 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 elevation of the 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 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 reinforced 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 in STS SC-M-713 (latest version) for *Mechanically Stabilized Earth (MSE) Walls*.

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 general 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 written approval from the OES/GDS. The length of reinforcement (L) is measured from the back of MSE wall panels. For modular block type MSE walls the length of reinforcement (B) is measured from the front face of the modular blocks. The minimum reinforcement length is 0.7H (B) or 8 feet whichever is greater. MSE wall structures with sloping surcharge fills or other concentrated loads will generally require longer reinforcement lengths of 0.8H (B) to 1.1H (B). 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 concerns.

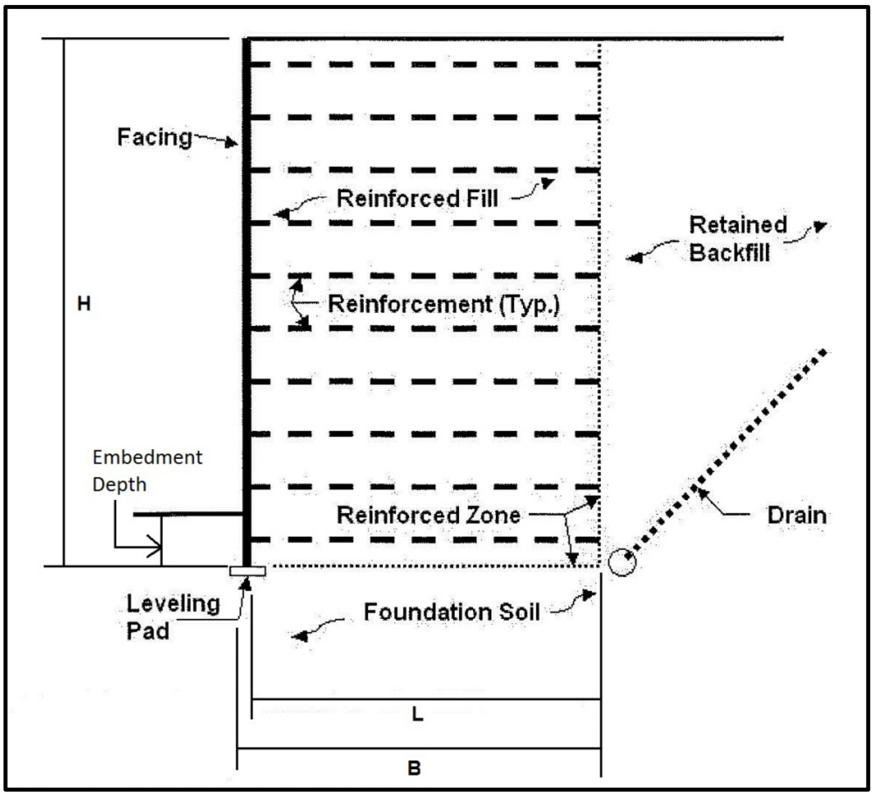


Figure C-1, General MSE Wall Schematic (Modified Berg, Christopher, and Samtani – Volume I (2009))

The top of the leveling pad will require a minimum embedment below finished grade in front of the wall of 2 feet. Greater embedment depths may be required due to bearing capacity, settlement, stability, erosion, or scour requirements. The MSE Wall leveling pad shall be located below the bottom of all utilities, ditches or other buried structures located in front of the wall. If the utility, ditch, or other buried structure is located more than 4 feet plus two times the depth of the bottom of the utility, ditch or other buried structure excavation in front of the wall, then a greater embedment depth will not be required. 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-1. 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.

Table C-1, Minimum MSE Wall Embedment Depth Based on Local Bearing Capacity

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

C.5 NOMINAL LOADS

The next step is the development of unfactored and factored loads on an MSE wall. The determination of these external loads is normally performed by the GEOR.

C.5.1 Unfactored Load Estimate

In this step, the GEOR 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 (horizontal and vertical earth pressures) and any surcharge loadings. There are 3 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 as well as the required load factors. If a bridge is to be supported by shallow foundations that bear on top of the MSE wall, then loads induced by the foundations must be included as specialized dead loads in the design of the MSE wall.

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 10° batter) with horizontal backfill is calculated according to the procedures provided in Chapter 18. The K_a values used in this Appendix are based on Coulomb earth pressure theory. When considering live loads 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).

$$\boldsymbol{F_T = \frac{1}{2} * \left(\gamma_f * h^2 * K_a\right)} \qquad \textbf{Equation C-1}$$

$$F_{TH} = F_T * \cos\delta \text{ and } F_{TV} = F_T * \sin\delta$$ **Equation C-2**

$$F_q = q * h * K_a$$ **Equation C-3**

$$F_{qH} = F_q * \cos\delta \text{ and } F_{qV} = F_q * \sin\delta$$ **Equation C-4**

Where,

γ_f = Unit weight of retained fill material
δ = 2/3 * ϕ of either reinforced soil or retained fill, whichever is smaller
h = Height of MSE wall above leveling pad (H in Figure C-2)
K_a = Active earth pressure coefficient, determined in accordance with Chapter 18 using the retained fill material properties (k_{af} in Figure C-2 and C-3)
q = Surcharge load over retained fill
F_T = Earth pressure induced by retained fill
F_{TH} = Horizontal component of earth pressure induced by retained fill
F_{TV} = Vertical component of earth pressure induced by retained fill
F_q = Earth pressure induced by live load surcharge
F_{qH} = Horizontal component of earth pressure induced by live load surcharge
F_{qV} = Vertical component of earth pressure induced by live load surcharge

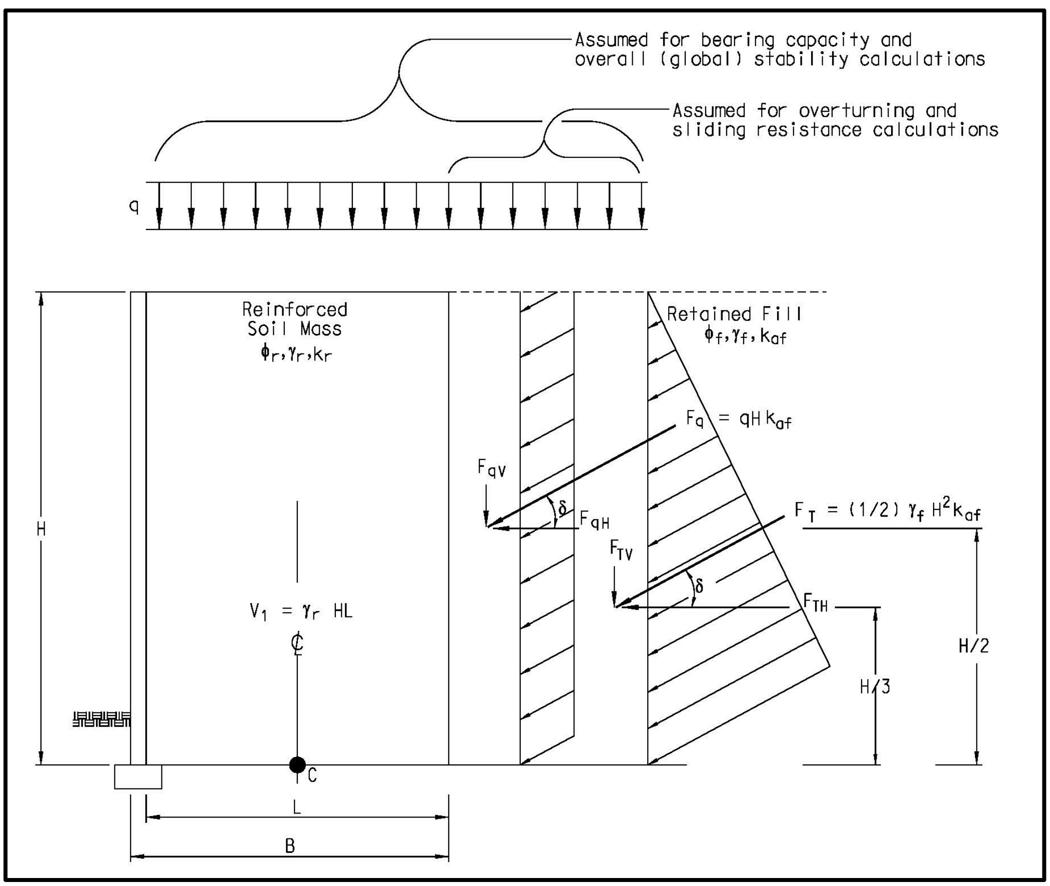


Figure C-2, MSE Wall Earth Pressure for Horizontal Backslope With Traffic Surcharge (modified AASHTO LRFD Specifications)

C.5.1.2 Sloping Backslope

K_a changes when there is a slope behind the MSE wall. K_a is determined in Chapter 18 and is based on Coulomb earth pressure theory. The force on the rear of the reinforced soil mass (P_a) and the resulting horizontal (P_H) and vertical (P_V) forces are determined from the following equations:

$$F_T = \frac{1}{2} * \left(\gamma_f * h^2 * K_a\right) \qquad \textbf{Equation C-5}$$

$$F_{TH} = F_T * \cos\delta \qquad \textbf{Equation C-6}$$

$$F_{TV} = F_T * \sin\delta \qquad \textbf{Equation C-7}$$

Where,

δ = 2/3 * ϕ of either reinforced soil or retained fill, whichever is smaller
h = Total height of wall including vertical projection of slope above wall (see Figure C-3)
K_a = Active earth pressure coefficient, determined in accordance with Chapter 18 using the retained fill material properties
F_T = Earth pressure induced by retained fill (P_a in Figure C-4)
F_{TH} = Horizontal component of earth pressure induced by retained fill (P_h in Figure C-4)
F_{TV} = Vertical component of earth pressure induced by retained fill (P_v in Figure C-4)

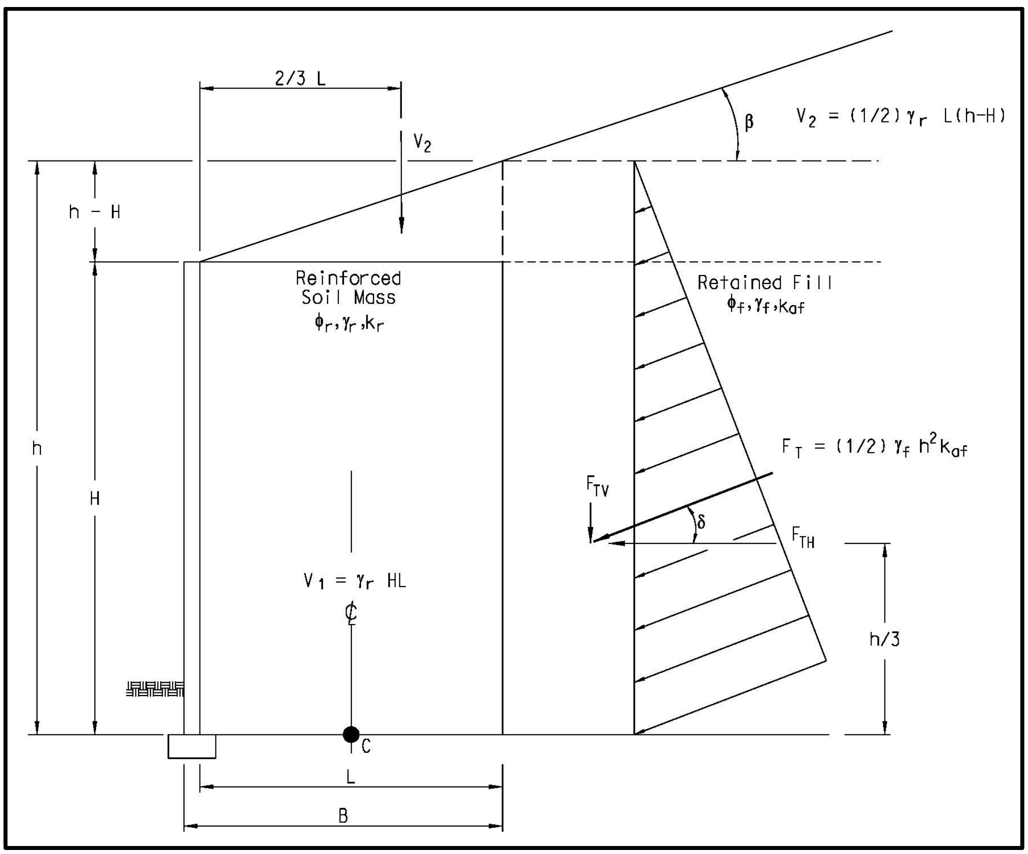


Figure C-3, MSE Wall Earth Pressure for Sloping Backfill (modified AASHTO LRFD Specifications)

C.5.1.3 Broken Backslope

For broken backslopes (see Figure C-4), the K_a is determined as indicated in Chapter 18 and is based on Coulomb earth pressure theory. The AASHTO LRFD Specifications have altered how the K_a from Coulomb earth pressure theory is developed for a broken backslope. As can be seen in Figure C-4 there are 3 cases for use in determining K_a for use in the design of MSE walls with broken backslopes. The cases are delineated on the ratio L_s to h, where L_s is the horizontal distance the broken backslope extends from the end of the reinforced soil mass and h is the vertical distance from the top of the leveling pad (see Figure C-1) to a horizontal line drawn from where the end of the reinforced soil mass intersects the backslope (see Figure C-4).

C.5.1.3.1 Case 1

Case 1 (① in Figure C-4) is the condition when L_s is greater than or equal to h ($L_s \geq h$). This case is similar to and designed as an MSE wall with a sloping backslope that is infinite as discussed in Section C.5.1.2. In determining the Coulomb active earth pressure coefficient, $\beta = \beta$ and is termed $K_{a\text{-Infinite}}$.

C.5.1.3.2 Case 3

Case 3 (③ in Figure C-4) is the condition when L_s is less than or equal to 0 ($L_s \leq 0$) (i.e., slope breaks above the reinforced soil mass (see Figure C-4)). This case is similar to and designed as an MSE wall with a horizontal backslope (Section C.5.1.1 with a traffic surcharge equal to 0 (i.e., q = 0)). In determining the Coulomb active earth pressure coefficient, $\beta = 0$ and is termed $K_{a\text{-Level}}$.

C.5.1.3.3 Case 2

Case 2 (② in Figure C-4) is more complicated, since L_s is greater than 0, but less than h ($0 < L_s < h$). This case is between Case 1 and Case 3 as far as the development of the Coulomb active earth pressure. The AASHTO LRFD Specifications recommend the following equation be used to develop K_a for Case 2.

$$\boldsymbol{K_{a-2}} = \left(\frac{\boldsymbol{L_s}}{\boldsymbol{h}}\right) * \left(\boldsymbol{K_{a-Infinite}} - \boldsymbol{K_{a-Level}}\right) + \boldsymbol{K_{a-Level}} \qquad \textbf{Equation C-8}$$

Using the K_a developed from 1 of the 3 cases previously discussed P_a, P_H, and P_V are determined as indicated in Equations C-5 through C-7. Where, P_a is the force acting on the rear of the MSE wall.

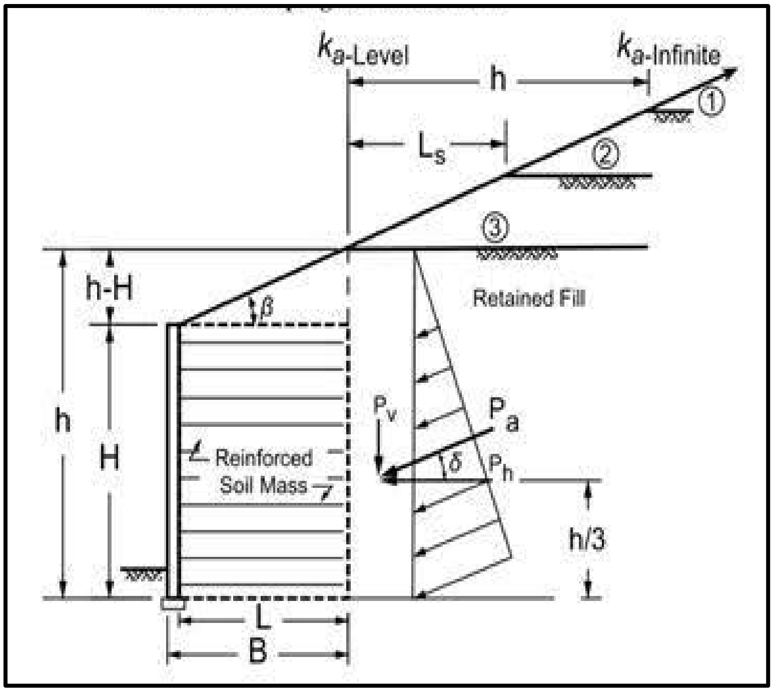
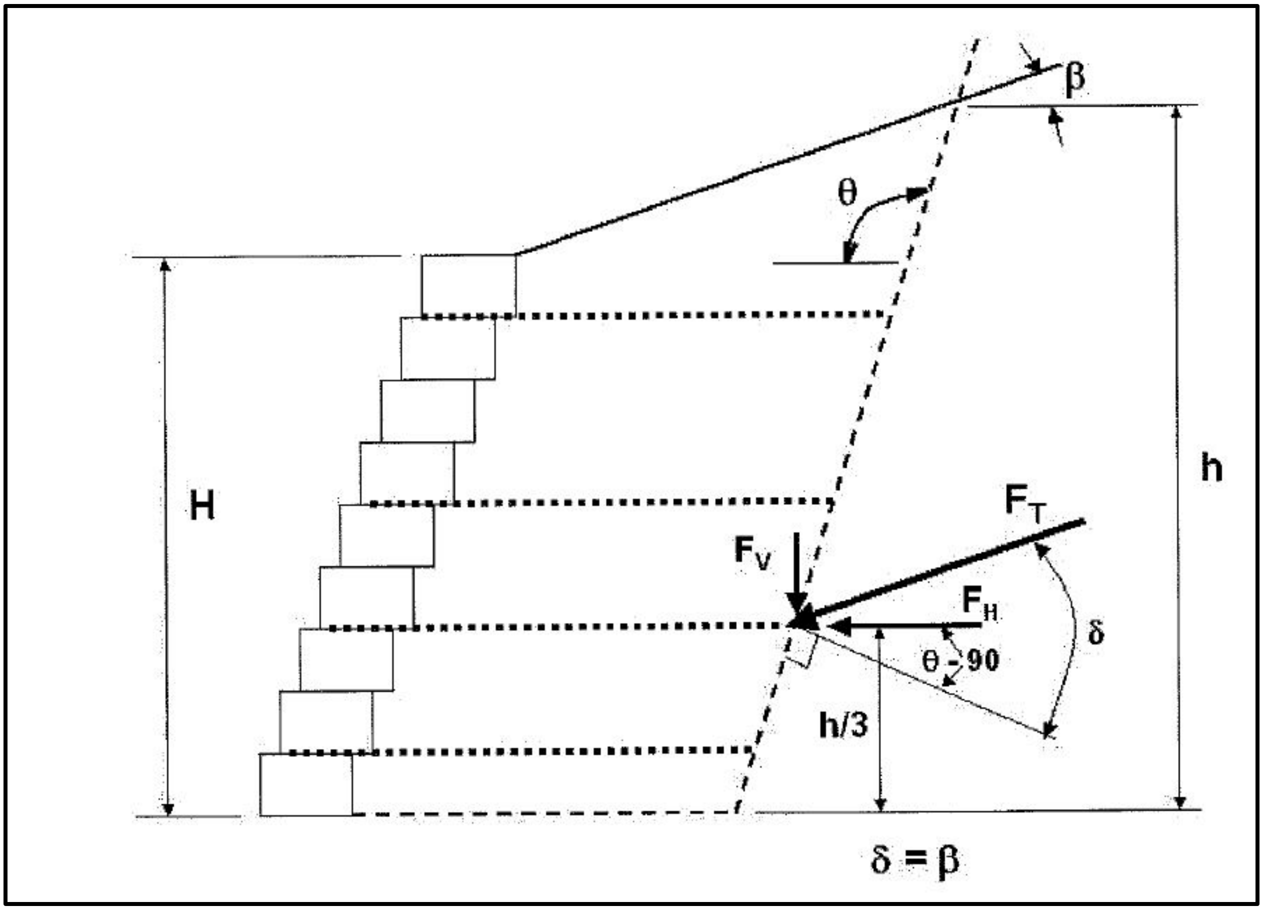


Figure C-4, MSE Wall Earth Pressure for Broken Backslope (AASHTO LRFD Specifications)

C.5.1.4 Battered Wall with or without Backslope

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

> For an inclined front face and reinforced zone (i.e., batter) equal to or greater than 10° from the vertical, the coefficient of earth pressure can be calculated using the procedures contained in Chapter 18, where θ is the face inclination from horizontal, and β is the surcharge slope angle as shown in Figure C-5. The wall friction angle δ is assumed to be equal to β.



θ ≥ 100°

Figure C-5, Notation for Coulomb Active Earth Pressure (Berg, et al. – Vol. I (2009))

C.6 LOAD COMINBATION SUMMARY

Portions of the following Section of this Appendix are adopted directly from Tanyu, Sabatini, and Berg (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 Tanyu, et al. (2008):

> …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, γ_{EV} is selected as the minimum value from *Table 8-6* (i.e., γ_{EV} = 1.00) and when checking bearing resistance, γ_{EV} is selected as the maximum value from *Table 8-6* (i.e., γ_{EV} = 1.35).

C.7 EXTERNAL STABILITY

The external stability analysis checks eccentricity (Section C.7.1), sliding (Section C.7.2), bearing resistance (C.7.3), and overall (global) stability (Section C.10). The determination of external stability is typically performed by SCDOT or its GEC and is performed for all appropriate limit states (see Chapter 8). The following Sections of this Appendix are adopted directly from the AASTHO LRFD Specifications and Berg, et al. – Vol. I (2009) 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.

C.7.1 Eccentricity

Eccentricity as used in this Section is concerned with overturning centered on the junction of the MSE wall face and the leveling pad. According to AASHTO LRFD Specifications:

> Reinforced soil walls are in general too flexible to fail due to excessive eccentricity (i.e., overturning). However, meeting the eccentricity requirements *typically used* for gravity walls…will keep the reinforced soil from being too flexible in its response to lateral earth pressure and other lateral loads that may be present behind the reinforced soil wall.

Therefore,

> For foundations on soil, the location of the resultant of the reaction forces shall be within the middle two-thirds *(2/3)* of the base width.
>
> For foundations on rock, the location of the resultant of the reaction forces shall be within the middle nine-tenths *(9/10)* of the base width.
>
> For *EE I* eccentricity evaluation of walls with foundations on soil and rock, the location of the resultant of the reaction forces shall be in the middle two-thirds *(2/3)* of the base for γ_{EQ} = 0.0… *It is noted that γ_{EQ} = 0.0 for all SCDOT projects unless otherwise specified by SCDOT.*

Combining the requirements of Berg, et al. – Vol. I (2009) and AASHTO LRFD Specifications leads to the following equation for an MSE wall with horizontal backslope and traffic surcharge (see Figure C-2):

Equation C-9

$$e_c = \frac{\left[\gamma_{EH-MAX}*(F_{TH})*\left(\frac{H}{3}\right)+\gamma_{LS}*(F_{qH})*\left(\frac{H}{2}\right)\right]-\left[\gamma_{EH-MIN}*(F_{TV})*\left(\frac{L}{2}\right)+\gamma_{LS}*(F_{qV})*\left(\frac{L}{2}\right)\right]}{\gamma_{EV-MIN}*V_1+\gamma_{EH-MIN}*F_{TV}+\gamma_{LS}*F_{qV}}$$

Using the same sources leads to the following equation for an MSE wall with a sloping backslope (see Figure C-3):

$$e_c = \frac{\left[\gamma_{EH-MAX}*(F_{TH})*\left(\frac{h}{3}\right)\right]-\left[\gamma_{EH-MIN}*(F_{TV})*\left(\frac{L}{2}\right)+\gamma_{EV-MIN}*(V_2)*\left(\frac{L}{6}\right)\right]}{\gamma_{EV-MIN}*V_1+\gamma_{EV-MIN}*V_2+\gamma_{EH-MAX}*F_{TV}} \quad \textbf{Equation C-10}$$

C.7.2 External Sliding Stability

Check external sliding stability of the MSE wall. According to Berg, et al. – Vol. I (2009):

Check the preliminary sizing with respect to sliding of the reinforced zone where the resisting force is the lesser of shear resistance along the base of the wall or of a weak layer near the base of the MSE wall, and the sliding force is the horizontal component of the thrust on the vertical plane at the back of the wall (see Figures *C-2* through *C-4*). The live load surcharge is not considered as a stabilizing force when checking sliding, i.e., the sliding stability check only applies *to* the live load above the retained backfill, as shown in Figure *C-2*. The driving forces generally include factored horizontal loads due to earth, water, seismic and surcharges.

Sliding resistance along the base of the wall is evaluated using the same procedures as for spread footings on soil as *indicated in Chapter 15*. The factored resistance against failure by sliding (R_r) can be estimated by:

$$\boldsymbol{R_r = \varphi_\tau * R_\tau}$$ **Equation C-11**

Where,

φ_τ = Resistance factor for shear resistance between soil and foundation *(equal to 1.0 for sliding of, see Chapter 9)*
R_τ = Nominal sliding resistance between reinforced fill and foundation soil

Note that any soil passive resistance at the toe due to embedment is ignored due to the potential for the soil to be removed through natural or manmade processes during its service life (e.g., erosion, utility installation, etc.). Also, passive resistance is usually not available during construction. The shear strength of the facing system is also conservatively neglected.

Calculation steps and equations to compute sliding for *2* typical cases follow. These equations should be extended to include other loads and geometries, for other cases, such as additional live and dead load *and* surcharge loads.SOIL

1. Calculate nominal thrust, per unit width, acting on the back of the reinforced zone.

   <u>Wall with Horizontal Backfill:</u> (see Figure *C-2*)
   The *horizontal component of the* retained backfill resultant, F_{TH}*, is determined using Equation C-2*.
   For a uniform surcharge, the *horizontal component of the* resultant, F_{qH}*, is determined using Equation C-4*.

   <u>Wall with Sloping Backfill:</u> (see Figure *C-3*)
   Calculate *horizontal component of the* retained backfill force resultant per unit width, P_H, *using Equation C-6*.

   <u>Wall with Broken Backslope:</u> (see Figure C-4)
   Use the correct case indicated above and the correct horizontal components indicated.

2. Calculate the nominal and factored horizontal driving forces. For a horizontal backslope and uniform live load surcharge:

$$\sum F = F_{TH} + F_{qH} \quad \textbf{Equation C-12}$$

$$P_d = \gamma_{EH} * F_{TH} + \gamma_{LS} * F_{qH} \quad \textbf{Equation C-13}$$

   For a sloping backfill, *see Equation C-6, therefore*:

$$P_d = \gamma_{EH} * P_H \quad \textbf{Equation C-14}$$

   Use the maximum EH load factor (*see Chapter 9*) in these equations because it creates the maximum driving force effect for the sliding *check*.

3. Determine the most critical frictional properties at the base. Choose the minimum soil friction angle, ϕ for *3* possibilities:

   i. Sliding along the foundation soil, if its shear strength (based on $c'_r + \tan\phi'_f$ and/or c_u for cohesive soils) is smaller than that of the reinforced fill material shear strength ($\tan\phi'_r$).
   ii. Sliding along the reinforced fill (ϕ'_r).
   iii. For sheet type reinforcement, sliding along the weaker of the upper and lower soil-reinforcement interfaces. The soil-reinforcement friction angle, ρ, should preferably be measured by means of interface direct shear tests. In absence of testing, it may be taken as $(2/3)\tan\phi'_r$.

4. Calculate the nominal components of resisting force and the factored resisting force per unit length of wall. For a horizontal backslope and uniform live load surcharge, the live load is excluded since it increases sliding stability:

$$R_r = [\gamma_{EV} * (V_1) + \gamma_{EH} * F_{TV}] * \mu \quad \textbf{Equation C-15}$$

   For a sloping backfill condition:

$$R_r = [\gamma_{EV} * (V_1 + V_2) + \gamma_{EH} * F_{TV}] * \mu \quad \textbf{Equation C-16}$$

$$V_1 = \gamma_r * H * L \quad \textbf{Equation C-17}$$

$$V_2 = \frac{\gamma_r * (h - H) * L}{2} \quad \textbf{Equation C-18}$$

   Where,

   γ_f = Unit weight of retained fill material
   H = Total wall height above the leveling pad (see Figure *C-2*)

h = Total height of wall including vertical projection of slope above wall (see Figure *C-3*)
L = Length of the reinforced soil mass (see Figure *C-2*)
F_{TV} = Vertical component of earth pressure induced by retained fill (P_v in Figure *C-4*) (see Equation *C-7*)
μ = Minimum soil friction angle ϕ [$\tan\phi'_f$, $\tan\phi'_r$, or (for continuous reinforcement) $\tan\rho$]

Forces V_1 and V_2 are applied through the centroid of the respective soil masses.

External loads that increase sliding resistance should only be included if those loads are permanent.

Use the minimum EV load factor (*see Chapter 9*) in these equations because it results in minimum resistance for the sliding *check*.

5. Compare factored sliding resistance, R_r, to the factored driving force P_d, to check that resistance is greater.

6. Check the capacity demand ratio (CDR) for sliding, CDR = R_r/P_d. If the CDR < 1.0 increase the reinforcement length, L, and repeat the calculations.

C.7.3 <u>Bearing Resistance</u>

The bearing resistance of the soil beneath the MSE Wall is the next design check. According to Berg, et al. – Vol. I (2009):

> Two modes of bearing capacity failure exist, general shear failure and local shear failure. Local shear is characterized by a punching or squeezing of the foundation soil when soft or loose soils exist below the wall.

Bearing calculations require both a *Strength* limit state and *Service* limit state calculation. Strength limit calculations check that the factored bearing pressure is less than the factored bearing resistance. Service limit calculations are used to compute nominal bearing pressure for use in settlement calculations. It should be noted that the weight and width of the wall facing is typically neglected in the calculations. The bearing check applies live load above both the reinforced zone and the retained backfill, as shown in Figure *C-2*.

<u>General Shear:</u> To prevent bearing failure on a uniform foundation soil, it is required that the factored vertical pressure at the base of the wall, as calculated with the uniform Meyerhof-type distribution, does not exceed the factored bearing resistance of the foundation soil:

$$q_r \geq q_{uniform} = \sigma_{V-F} \qquad \textbf{Equation C-19}$$

The uniform vertical pressure is calculated as:

$$q_{uniform} = \sigma_{V-F} = \frac{\sum V}{L-2e_B}$$ **Equation C-20**

This step requires *the* computation of the eccentricity value. Also note that the bearing check applies the live load above both the reinforced zone and the retained backfill, as shown in Figure *C-2*. In addition to walls founded on soil, a uniform vertical pressure is also used for walls founded on rock due to the flexibility of MSE walls and their limited ability to transmit moment (Article C11.10.5.4 (*AASHTO LRFD Specifications*)).

Calculation steps for MSE walls with either a horizontal backslope and uniform live load surcharge *or* for sloping backfills follows. Again, note that these equations should be extended to include loads and geometries, for other cases.

1. Calculate the eccentricity, e_B, of the resulting force at the base of the wall. *Note,* the e_C value from the eccentricity step *check* cannot be used. Calculate e_B with factored loads *(see Figure C-6)*. For a wall with horizontal backslope and uniform live load surcharge centered about the reinforced zone, the eccentricity is equal to:

Equation C-21

$$e_B = \frac{\left[\gamma_{EH-MAX}*F_{TH}*\left(\frac{H}{3}\right)+\gamma_{LS}*F_{qH}*\left(\frac{H}{2}\right)\right]-\left[\gamma_{EH-MAX}*(F_{TV})*\left(\frac{L}{2}\right)+\gamma_{LS}*\left(F_{qV}\right)*\left(\frac{L}{2}\right)\right]}{\gamma_{EV-MAX}*V_1+\gamma_{LS}*q*L+\gamma_{EV-MAX}*F_{TV}+\gamma_{LS}*F_{qV}}$$

Where the terms used were previously defined. The maximum load factors for γ_{EH} and γ_{EV} are used to be consistent with the computation for σ_v (below) where the maximum load factors results in the maximum vertical stress.

For walls with sloping backfill *see the following equation (see Figure C-6)*. Again, note that these equations should be extended to include other loads and geometries, for other cases.

Equation C-22

$$e_B = \frac{\left[\gamma_{EH-MAX}*(F_{TH})*\left(\frac{h}{3}\right)\right]-\left[\gamma_{EH-MAX}*(F_{TV})*\left(\frac{L}{2}\right)+\gamma_{EV-MAX}*(V_2)*\left(\frac{L}{6}\right)\right]}{\gamma_{EV-MAX}*V_1+\gamma_{EV-MAX}*V_2+\gamma_{EH-MAX}*F_{TV}}$$

Note that when checking the various load factors, and load combinations, the value of eccentricity, e_B, will vary. Also note that when the calculated eccentricity, e_B, is negative, a value of 0 should be carried in the design stress equation, i.e., set L' = L, per AASHTO C11.10.5.4 (*AASHTO LRFD Specifications*).

2. Calculate the factored vertical stress σ_{V-F} at the base assuming Meyerhof-type distribution. For a horizontal backslope and uniform live load surcharge the factored bearing pressure is:

$$\sigma_{V-F} = \frac{\gamma_{EV-MAX} * V_1 + \gamma_{LS} * q * L}{L - 2e_B}$$ **Equation C-23**

This approach, proposed originally by Meyerhof, assumes that a stress distribution due to eccentric loading can be approximated by a uniform stress distribution over a reduced area at the base of the wall. This area is defined by a width equal to the wall width minus twice the eccentricity as shown in Figures *C-6* and *C-7*. The effect of eccentricity and load inclination is addressed with the use of the effective width, L-2e$_B$, in lieu of the full width, L.

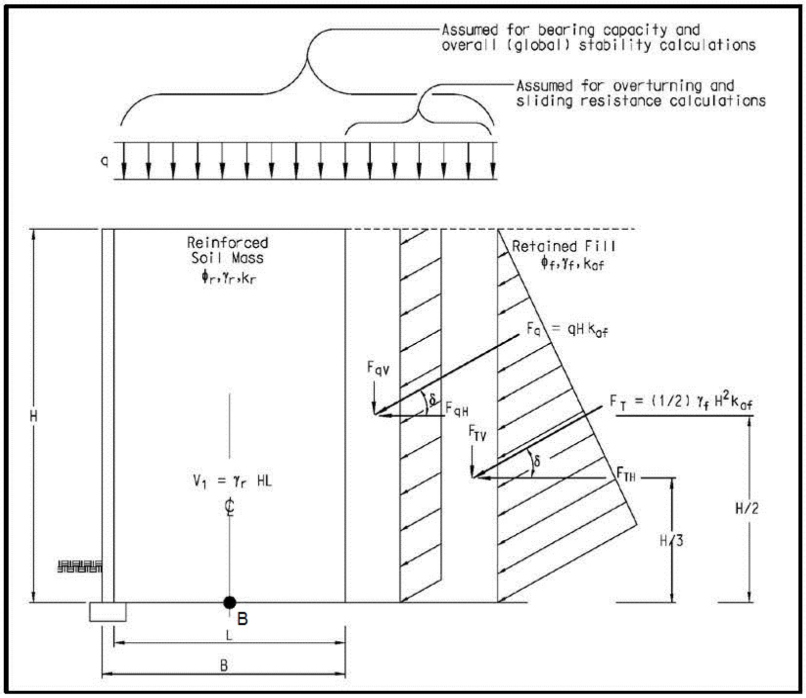


Figure C-6, MSE Wall Eccentricity Check for Horizontal Backslope (Berg, et al. – Vol. I (2009))

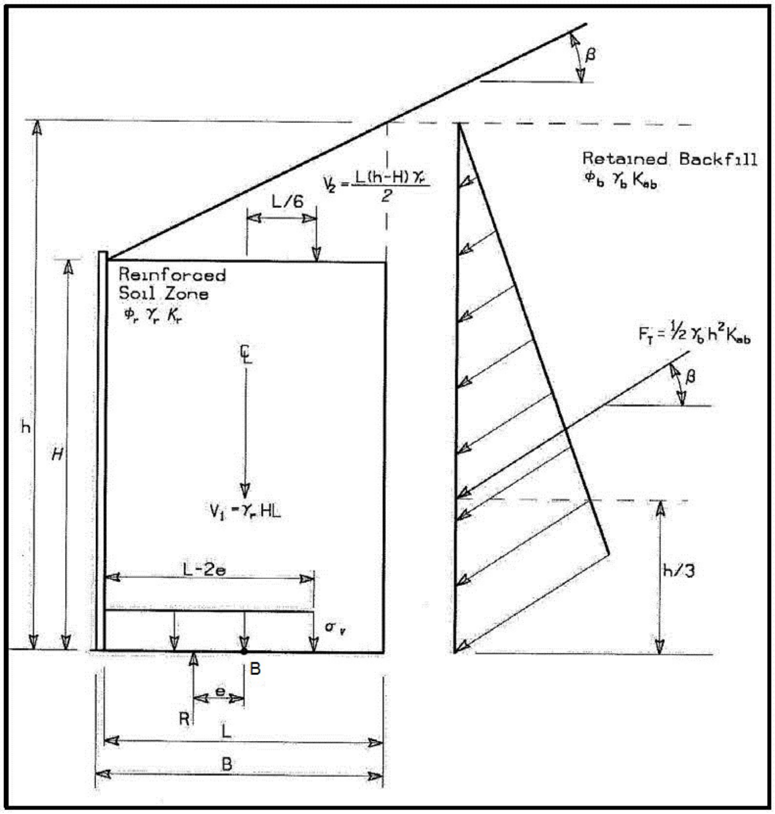


Figure C-7, MSE Wall Eccentricity Check for Sloping Backfill (Berg, et al. – Vol. I (2009))

For wall with sloping backfill the factored bearing stress is:

$$\boldsymbol{\sigma_{V-F}} = \frac{\gamma_{EV-MAX} * V_1 + \gamma_{EV-MAX} * V_2 + \gamma_{EH-MAX} * P_V}{L - 2e_B} \qquad \textbf{Equation C-24}$$

Note the (L-2e_B) is set equal to L when the value of eccentricity is negative. A negative value of eccentricity may be found for some extreme geometries, e.g., a wall section with very long reinforcements and a steep, infinite backslope. Note that when checking the various load factors and load combinations the value of eccentricity, e_B, will vary and a critical value must be determined by comparisons of applicable load combinations.

Where applicable, in the computation of bearing stress, $\sigma_{V\text{-}F}$, include the influence of factored surcharge and factored concentrated loads. Maintain consistency with loads and load factors used in the eccentricity calculation and corresponding bearing stress calculation.

3. *Determine q_n per Chapter 15.*

4. Check that factored bearing resistance is greater than the factored bearing stress, i.e., $q_r \geq \sigma_{V\text{-}F}$. The factored bearing resistance (q_r) is given as:

$$q_r = \varphi * q_n \qquad \textbf{Equation C-25}$$

5. As indicated previously, $\sigma_{V\text{-}F}$ can be decreased and q_r increased by lengthening the reinforcements, though only marginally. The nominal bearing resistance often may be increased by additional subsurface investigation and better definition of the foundation soil properties. If adequate support conditions cannot be achieved or lengthening reinforcements significantly increases costs, improvement of the foundation soil may be considered (*see Chapter 19*).

Local Shear, Punching Shear and Lateral Squeeze. Local shear is a transition between general shear and punching shear, which can occur in loose or compressible soils or in weak soils under slow (drained) loading. If local shear or punching shear is possible, Section 10.6.3.1.2b of *AASHTO LRFD Specifications* requires the use of reduced shear strength parameters for *calculating the* nominal bearing resistance. The reduced effective stress cohesion, c* is set equal to 0.67c'. The reduced effective stress soil friction angle, ϕ^* is set equal to $\tan^{-1}*(0.67\tan\phi'_f)$.

Lateral squeeze is a special case of local shear that can occur when bearing on a weak cohesive soil layer overlying a firm soil layer. Lateral squeeze failure results in significant horizontal movement of the soil under the structure*. Lateral squeeze shall be determined as indicated in Chapter 17.*

If adequate support conditions cannot be achieved, either the soft soils should be removed and replaced with more suitable material or ground improvement of the foundation soils maybe required. Local shear, as well as bearing on 2-layered soil systems in undrained and drained loading, *is* addressed in Section 10.3.6.1.2 of *AASHTO LRFD Specifications*.

C.7.4 Vertical Displacement

MSE wall structures can move vertically due to static and seismic loads. The movements caused by static loads (Service limit state) are discussed here. See Chapter 14 for guidance on seismically (EE I limit state) 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-2 shall be used as typical limiting differential settlement tolerances along the MSE wall facing for MSE wall structures with precast panel facings.

Table C-2, Limiting Differential Settlement for MSE Wall Systems with Precast Concrete Panel Facing (along facing)

Panel Joint Width	Limiting Differential Settlement
≥ 1/2" *	1/300
< 1/2" *	1/600
Full Height Panel	1/600

*__Note:__ Relatively square facing panels

MSE wall structures with modular concrete block facings are typically restricted to a limiting differential settlement of 1/240 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 may be needed. Prefabricated Vertical Drains may be required to accelerate the consolidation settlement if construction time is limited. Walls shall be checked 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. According to Berg, et al. – Vol. I (2009), "Where the anticipated settlements and their duration, cannot be accommodated by these measures, consideration must be given to … the implementation of 2-phased *(-staged)* construction in which the first phase *(stage)* facing is typically a wire facing."

C.8 INTERNAL STABILITY

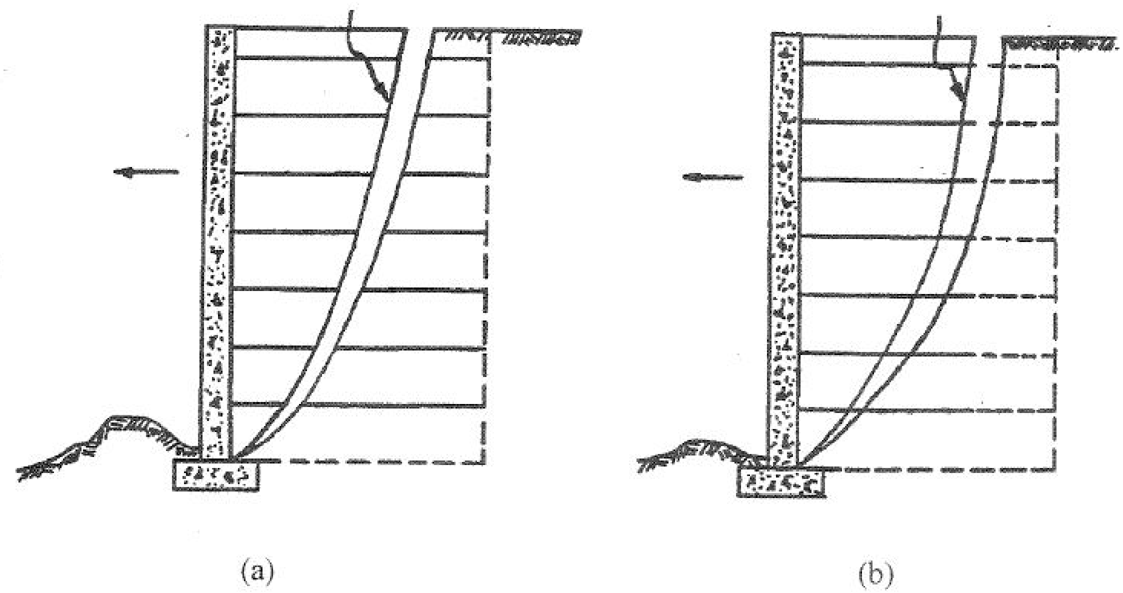
The internal stability analysis is the 7th step of the design process provided in Chapter 18. These analyses are typically performed by the MSE wall supplier or manufacturer and reviewed/checked during the shop plan process. According to AASHTO LRFD Specifications:

> The load in the reinforcement shall be determined at two critical locations:
>
> - The zone of maximum stress, which is assumed to occur at the boundary between the active and resistant zone *(see Figure C-9)*
> - The connection at the wall face

According to Berg, et al. Vol I (2009) in the zone of maximum stress there are two conditions that need to be accounted:

- The tensile forces (and, in the case of rigid reinforcements, the shear forces) in the inclusions become so large that the inclusions elongate excessively or break, leading to large movements and/or possible collapse of the structure. This mode of failure is called failure by elongation or breakage of the reinforcements *(see Figure C-8a)*.
- The tensile forces in the reinforcements become larger than the pullout resistance, leading to large movements and/or possible collapse of the structure. This mode of failure is called failure by pullout *(see Figure C-8b)*.

The process of sizing and designing to preclude internal failure, therefore, consists of determining the maximum developed tension forces, their location along a locus of critical slip surfaces and the resistance provided by the reinforcements both in pullout capacity and tensile strength. Internal stability also includes an evaluation of serviceability requirements such as tolerable lateral movement of supported structures and control of downdrag stress on reinforcement connections.



(a) Tension Failure (b) Pullout Failure

Figure C-8, MSE Wall Internal Failure Mechanisms (Tanyu, et al. (2008))

C.8.1 Select Type of Reinforcement

The first step in internal stability design is the selection of the type of reinforcement (i.e., inextensible (metallic) or extensible (geosynthetic)). Berg, et al. – Vol. I (2009) describes the selection process as:

Soil reinforcements are either inextensible (i.e., mostly metallic) or extensible (i.e., mostly polymeric materials). The internal wall design model varies by material type due to their extensibility relative to soil at failure. Therefore, the choice of material type should be made at this step of the design. The variations are: whether life prediction is based on metal corrosion or polymeric degradation; critical failure plane geometry assumed for design; and lateral stress used for design. Distinction can be made between the characteristics on inextensible and extensible reinforcements, as follows:

Design Methods, Inextensible (e.g., Metallic) Reinforcements

The current method of limit equilibrium analysis uses a coherent gravity structure approach to determine external stability of the reinforced mass, similar to the analysis for any conventional or traditional gravity structure. For internal stability evaluations, it considers a bi-linear failure surface that divides the reinforced zone in active and resistant zones and requires that an equilibrium state be achieved for successful design.

The lateral earth pressure distribution for **external stability** is assumed to be based on Coulomb's method with a wall friction angle δ. For **internal stability** lateral pressure varying from a multiple of K_a to an active earth pressure state, K_a is used for design. Previous research (FHWA RD-89-043, *Christopher, et al. (1990)*) has focused on developing the state of stress for internal stability, as a function of K_a, type of reinforcement used (geotextile, geogrid, metal strip or metal grid), and depth. The results from these and more recent *(Allen, Christopher, Elias, and DiMaggio (2001))* efforts have been synthesized in a simplified method, which will be used throughout this *Appendix*.

Design Methods, Extensible (e.g., Geosynthetic) Reinforcements
For **external stability** calculations, the current method assumes an earth pressure distribution consistent with the method used for inextensible reinforcements.

For **internal stability** computations using the simplified method, the internal coefficient of earth pressure is again a function of the type of reinforcement, where the minimum coefficient (K_a) is used for walls constructed with continuous sheets of geotextiles and geogrids. For internal stability, a Rankine failure surface is considered, because the extensible reinforcements can elongate more than the soil, before failure, and do not significantly modify the shape of the soil failure surface.

C.8.2 Critical Slip Surface Location

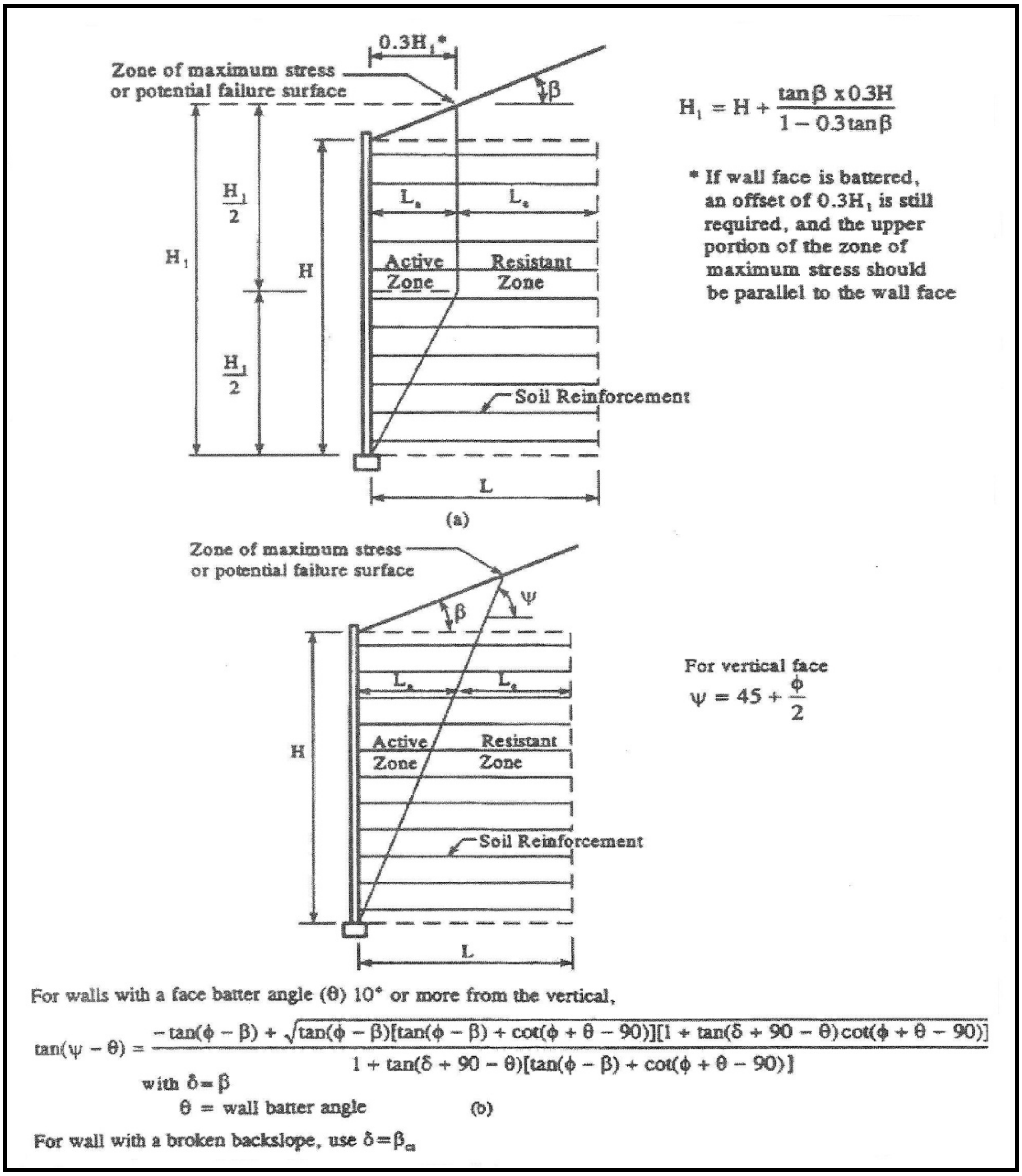
The second step in internal stability design is selecting the location of the critical slip surface. Berg, et al. – Vol. I (2009) describes the critical slip surface location selection process as:

The critical slip surface in a simple reinforced soil wall is assumed to coincide with the locus of the maximum tensile force, T_{MAX}, in each reinforcement layer. The shape and location of the critical failure surface is based upon instrumented structures and theoretical studies.

The critical failure surface has been assumed to be approximately bilinear in the case of inextensible reinforcements (Figure *C-9a*), approximately linear in the case of extensible reinforcements (Figure *C-9b*), and passes through the toe of the wall in both cases.

When failure develops, the reinforcement may elongate and be deformed at its intersection with the failure surface. As a result, the tensile force in the reinforcement would increase and rotate. Consequently, the component in the direction of the failure surface would increase and the normal component may increase or decrease. Elongation and rotation of the reinforcements may be negligible for stiff inextensible reinforcements such as steel strips but may be significant with geosynthetics. Any reinforcement rotation is ignored for internal wall stability calculations with the simplified method. However, reinforcement rotation may be considered in compound slope stability analysis (see *Berg, et al. – Vol. I (2009)*).

For extensible reinforcements, the Coulomb earth pressure relationship shown in Figure *C-9b* should be used to define the failure surface, per AASHTO Figure 11.10.6.3.1-1 (*AASHTO LRFD Specifications*), where the wall front batter from vertical is greater than *or equal to* 10 degrees. *For walls with a front batter from the vertical to less than 10° from vertical (i.e., θ = 90° to 100° in Figure C-9), use the Rankine earth pressure relationship (see Chapter 18).*



(a) Inextensible reinforcements (b) Extensible reinforcements

Figure C-9, Potential Failure Surface Location for Internal Stability of MSE Walls (Berg, et al. – Vol. I (2009))

C.8.3 Define Unfactored Loads

The third step in internal stability design is defining the unfactored loads to be used in design. According to Berg, et al. – Vol. I (2009):

> The primary sources of internal loading of an MSE wall is the earth pressure from the reinforced fill and any surcharge loadings on top of the reinforced zone. The unfactored loads for MSE walls may include loads due to vertical earth pressure (EV), live load surcharge (LS), and earth surcharge (ES). Water, seismic, and vehicle impact loads should also be evaluated as appropriate.
>
> Research studies (Collin (1986), Christopher, et al. (1990), Allen, et al. (2001)) have indicated that the maximum tensile force is primarily related to the type of reinforcement in the MSE wall, which, in turn, is a function of the modulus, extensibility and density of reinforcement. Based on this research, a relationship between the type of the reinforcement and the overburden stress has been developed, and shown in Figure *C-10*. The K_r/K_a ratio for metallic (inextensible) reinforcements decreases from the top of the reinforced wall fill to a constant value 20 feet below this elevation. In contrast to inextensible reinforcements, the K_r/K_a for extensible (e.g., geosynthetic) reinforcement is a constant. Note that the resulting K_r/K_a ratio is referenced to the top of the wall at the face, excluding copings and appurtenances (i.e., the top of the reinforced soil zone at the face) for both walls with level and sloping backfills. The K_r/K_a starting elevation for an MSE wall supporting a spread footing bridge abutment is the top of the backfill.

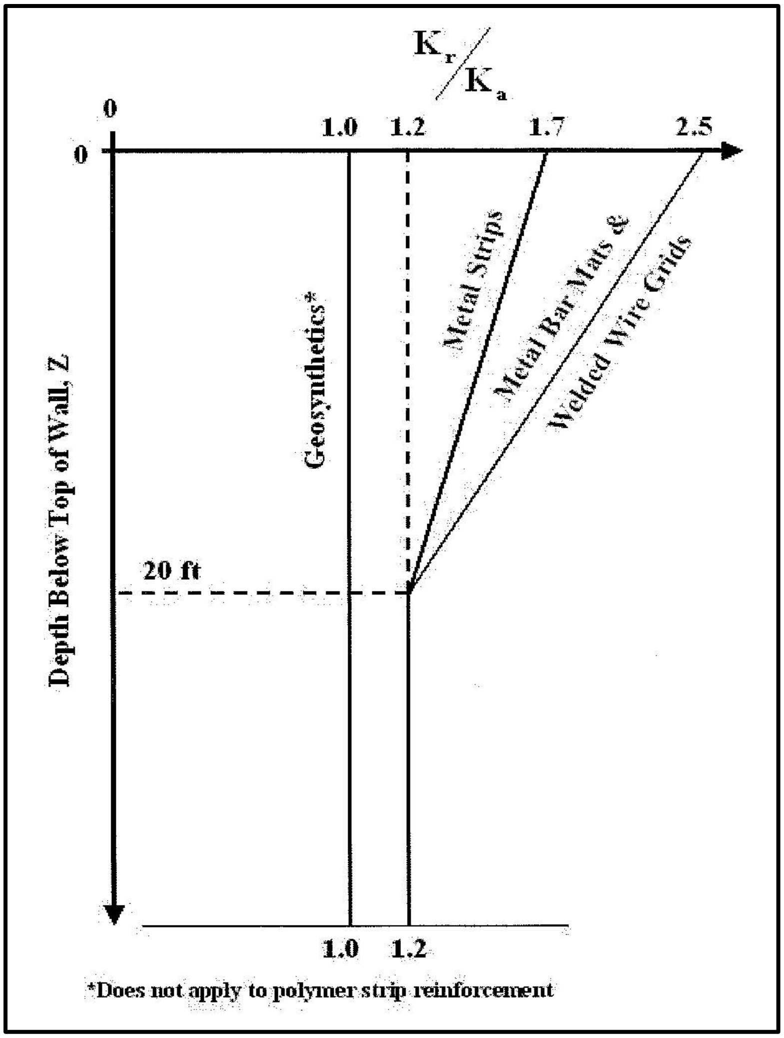


Figure C-10, Variation of the Coefficient of Lateral Stress Ratio with Depth (Berg, et al. – Vol. I (2009))

Figure *C-10* was prepared by back analysis of the lateral stress ratio K_r from available field data where stresses in the reinforcements were measured and normalized as a function of the Rankine active earth pressure coefficient, K_a. The Rankine active earth pressure theory assumes lateral pressure is independent of backfill slope and interface friction. The ratios shown in Figure *C-10* correspond to 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 (γH). This provides a simplified evaluation method for all cohesionless *(i.e., Sand-Like soils)* reinforced fill walls. Future data may lead to modifications in Figure *C-10*, including relationships for newly developed reinforcement types, effect of full height panels, etc. These relationships can be developed by instrumenting structures and using numerical models to verify the K_r/K_a ratio for routine and complex walls.

The lateral earth pressure coefficient K_r is determined by applying a multiplier to the active earth pressure coefficient. The active earth pressure coefficient is determined using a Coulomb earth pressure relationship, assuming no wall friction and an β angle equal to *0.0* (i.e., equivalent to the Rankine earth pressure coefficient (*see Chapter 18 for determination of active earth pressure coefficient*).

For wall face batters equal to or greater than 10° from the vertical, the following simplified form of the Coulomb equation can be used:

$$K_a = \frac{\sin^2(\theta+\phi_r')}{\sin^3\theta * \left(1+\frac{\sin\phi_r'}{\sin\theta}\right)} \qquad \textbf{Equation C-26}$$

Where,

θ = *Inclination of back of the facing as measured from the horizontal starting in front of the wall, as shown in Figure C-5, note that* θ *is greater than 100°*

ϕ'_r = *Angle of internal friction of reinforced zone*

Commentary C11.10.6.2.1 *(AASHTO LRFD Specifications)* states that the above equation can be used for battered walls. The 10° value recommendation is consistent with the equation to determine the failure surface location for walls with 10° or greater batter [*C11.10.6.2.1 (AASHTO LRFD Specifications)*].

The stress, σ_2, due to a sloping backfill on top of an MSE wall can be determined as shown in Figure *C-11*. An equivalent soil height, S_{eq}, is computed based upon slope geometry. The value of S_{eq} should not exceed the slope height for broken back sloping fills. A reinforcement length of 0.7H is used to compute the sloping backfill stress, σ_2, on the soil reinforcement, as a greater length would only have minimal effect on the reinforcement. The vertical stress is equal to the product *of the* equivalent soil height and the reinforced fill unit weight and is uniformly applied across the top of the MSE zone. *See Step 3 of Calculate Horizontal Stress in the next Section for an explanation of why the reinforced fill unit weight is used in this calculation.*

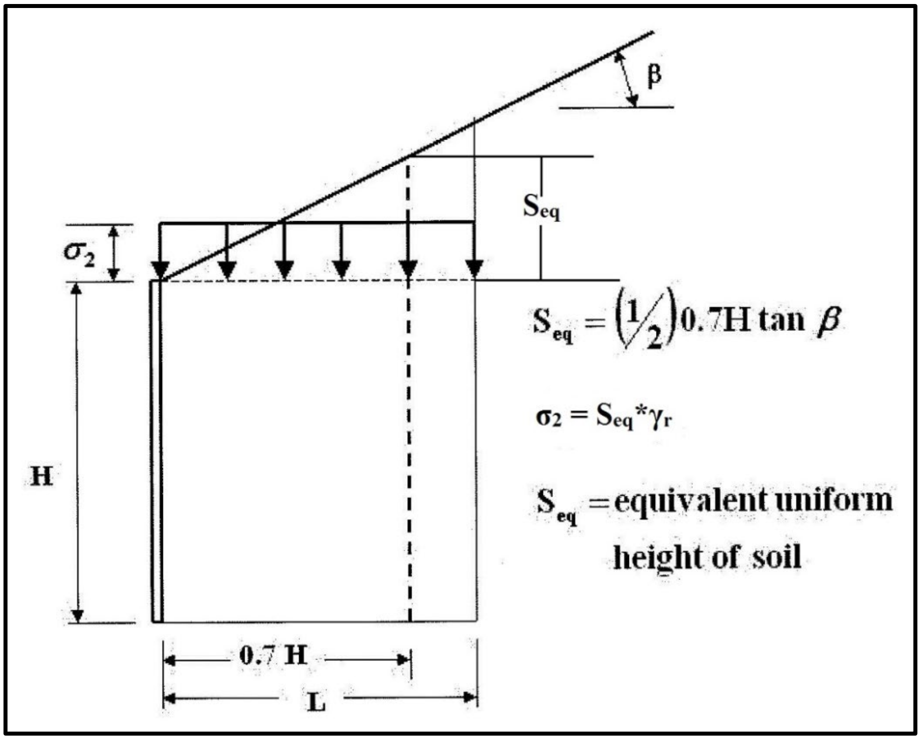


Figure C-11, Calculation of σ_2 for Sloping Backfill for Internal Stability (modified Berg, et al. – Vol. I (2009))

C.8.4 Establish Vertical Layout of Soil Reinforcements

The fourth step in internal stability design is defining the vertical layout of the soil reinforcement. Berg, et al. – Vol. I (2009) describes the vertical layout of the soil reinforcement as follows:

> Use of a constant reinforcement section and spacing for the full height of the wall usually gives more reinforcement in the upper portion of the wall than is required for stability. Therefore, a more economical design may be possible by varying the reinforcing density with depth. However, to provide a coherent reinforced soil zone, vertical spacing of reinforcement should not exceed 32 inches.
>
> There are generally *2* practical ways to accomplish this for MSE walls:
>
> - For reinforcements consisting of strips, grids, or mats, used with segmental precast concrete facings, the vertical spacing is maintained constant and the reinforcement density is increased with depth by increasing the number and/or the size of the reinforcements. For instance, the typical horizontal spacing of 2-inch X 5/32-inch strips is 30 inches, but this can be decreased by adding horizontal reinforcement locations.
>
> - For continuous sheet reinforcements, made of geotextiles or geogrids, a common way of varying the reinforcement density T_{al}/S_v is to change the vertical spacing S_v, especially if wrapped facing is used, because it easily accommodates spacing variations. The range of acceptable spacing is governed by consideration of placement and compaction of the backfill (e.g., S_v taken as 1, 2 or 3 times the compacted lift thickness). The

reinforcement density T_{al}/S_v can also be varied by changing the strength (T_{al}) especially if wrapped facing techniques requiring a constant wrap height are used.

Low-to-medium-height walls (e.g., < 16 ft) are usually constructed with *1* strength geosynthetic. Taller walls use multiple strength geosynthetics. For example the 41-foot high Seattle preload wall used *4* strengths of geotextiles *(Allen, Christopher, and Holtz (1992))*. A maximum spacing of 16 inches is typical for wrapped faced geosynthetic walls, although a smaller spacing may be desirable to minimize bulging.

For walls constructed with modular blocks, the maximum vertical spacing of reinforcement should be limited to *2* times the block depth (front face to back face) or 32 inches, whichever is less, to assure construction and long-term stability. The top row of reinforcement should be limited to 1.5 the block depth (e.g., *1* unit plus a cap unit).

For large face units, such as 3 feet by 3 feet gabions, a vertical spacing equal to the face height (3 feet) is typically used. The spacing slightly exceeds the limit noted above, but this may be offset by the contributions of the large facing unit to internal (i.e., bulging) stability.

C.8.5 Factored Tensile Forces in Reinforcement Layers

The fifth step in internal stability design is determining the tensile forces in the reinforcement layers. This determination consists of 2 sub steps, first, calculating the horizontal stress on each reinforcement layer and then determining the maximum tension (T_{MAX}). Berg, et al. – Vol. I (2009) describes determining the factored tensile forces as follows:

Calculate Horizontal Stress

For internal stability analysis, the distribution of horizontal stress, σ_H, is first established. The horizontal stress at any given depth within the reinforced soil zone is expressed as follows:

$$\boldsymbol{\sigma_H = K_r * \sigma_v + \Delta\sigma_H} \qquad \textbf{Equation C-27}$$

P_{TMAX-D} in Equation C-31

Where,

K_r = Coefficient of lateral earth pressure in the reinforced soil zone (see Figure C-10)
σ_v = Factored vertical pressure at depth of interest
$\Delta\sigma_H$ = Supplemental factored horizontal stress due to external surcharges

For internal stability analysis, the following assumptions are made in the computation of factored vertical pressure, σ_v:

1. Vertical pressure due to the weight of the reinforced soil zone is assigned a load type “EV” with a corresponding (maximum) load factor, $\gamma_{P\text{-}EV}$ = 1.35. The maximum load factor of 1.35, and not the minimum load factor of 1.00, is always used to find the critical stress.

2. Any vertical surcharge above the reinforced soil zone is due to soil or considered as an equivalent soil surcharge is assigned a load type “EV”. In this scenario, a live load traffic surcharge that is represented by an equivalent uniform soil surcharge of height h_{eq} is assumed as a load type “EV”. This is in contrast to the external stability analysis where the live load traffic surcharge is assumed as a load type “LS” because in external stability analysis the MSE wall is assumed to be a rigid block. For internal analysis, the assumption of load type “EV” is used so that the amount of soil reinforcement within the reinforced soil zone is approximately the same as obtained using past working stress design approach (i.e., calibration by fitting).

3. The unit weight of the equivalent soil surcharge is assumed to be the same as the unit weight of the reinforced soil zone, γ_r, which is generally greater than or equal to the unit weight of the retained backfill.

4. Any vertical surcharge that is due to non-soil source is assigned a load type “ES”. Example of such a load is the bearing pressure under a spread footing on top of the reinforced soil zone. However, the application of the load factor of $\gamma_{P\text{-}ES}$ = 1.50 that is assigned to load type “ES” is a function of how the vertical pressures are computed as follows:

   - If the vertical pressures are based on nominal (i.e., unfactored) loads, then use $\gamma_{P\text{-}ES}$ = 1.50.
   - If the vertical pressures were based on factored loads, then use $\gamma_{P\text{-}ES}$ = 1.00. This is because once the loads are factored they should not be factored again.

   It is recommended that the factored vertical pressure be evaluated using both the above approaches and the larger value chosen for analysis.

The supplemental factored horizontal pressure, $\Delta\sigma_H$, could be from a variety of sources. Two examples of supplemental horizontal pressures are as follows:

1. Horizontal pressures due to the horizontal (shear) stresses at the bottom of a spread footing on top of the reinforced soil zone.

2. Horizontal pressures from deep foundation elements extending through the reinforced soil zone.

Supplemental horizontal pressures are assigned a load type “ES” since they represent surcharges on or within the reinforced soil zone. However, similar to the vertical pressures due to non-soil loads, the application of the maximum load factor of $\gamma_{P\text{-}ES}$ = 1.50 that is assigned to the load type “ES” is a function of how the horizontal pressures are computed as follows:

- If the horizontal pressures are based on nominal (i.e., unfactored) loads, the use $\gamma_{ES\text{-}MAX}$ = 1.50.
- If the horizontal pressures were based on factored loads, then use $\gamma_{P\text{-}ES}$ = 1.00. This is because once the loads are factored they should be not factored again.

As with vertical pressure, it is recommended that the factored horizontal pressure be evaluated using both of the approaches and the larger value chosen for analysis.

Berg, et al. – Vol. I (2009) provides 4 examples of MSE wall configurations ranging from simple to complex geometries as application of the above guidance. In addition, see Chapter 8 for the definition of each load type.

Calculate Maximum Tension (T_{MAX})

Calculate the maximum factored tension T_{MAX} in each reinforcement layer per unit width of wall based on the vertical spacing S_v from:

$$\boldsymbol{T_{MAX} = \sigma_H * S_v}$$ **Equation C-28**

The term S_v is equal to the vertical reinforcement spacing for a layer where vertically adjacent reinforcements are equally spaced from the layer under construction. In this case, σ_H, calculated at the level of the reinforcement, is at the center of the contributory height. The contributory height is defined as the midpoint between vertically adjacent reinforcement elevations, except for the top and bottom layers reinforcement.

For the top and bottom layers of reinforcement, S_v is the distance from top or bottom of wall, respectively, to the midpoint between the first and second layer (from top or bottom of wall, respectively) of reinforcement. S_v distances are illustrated in Figure *C-12*.

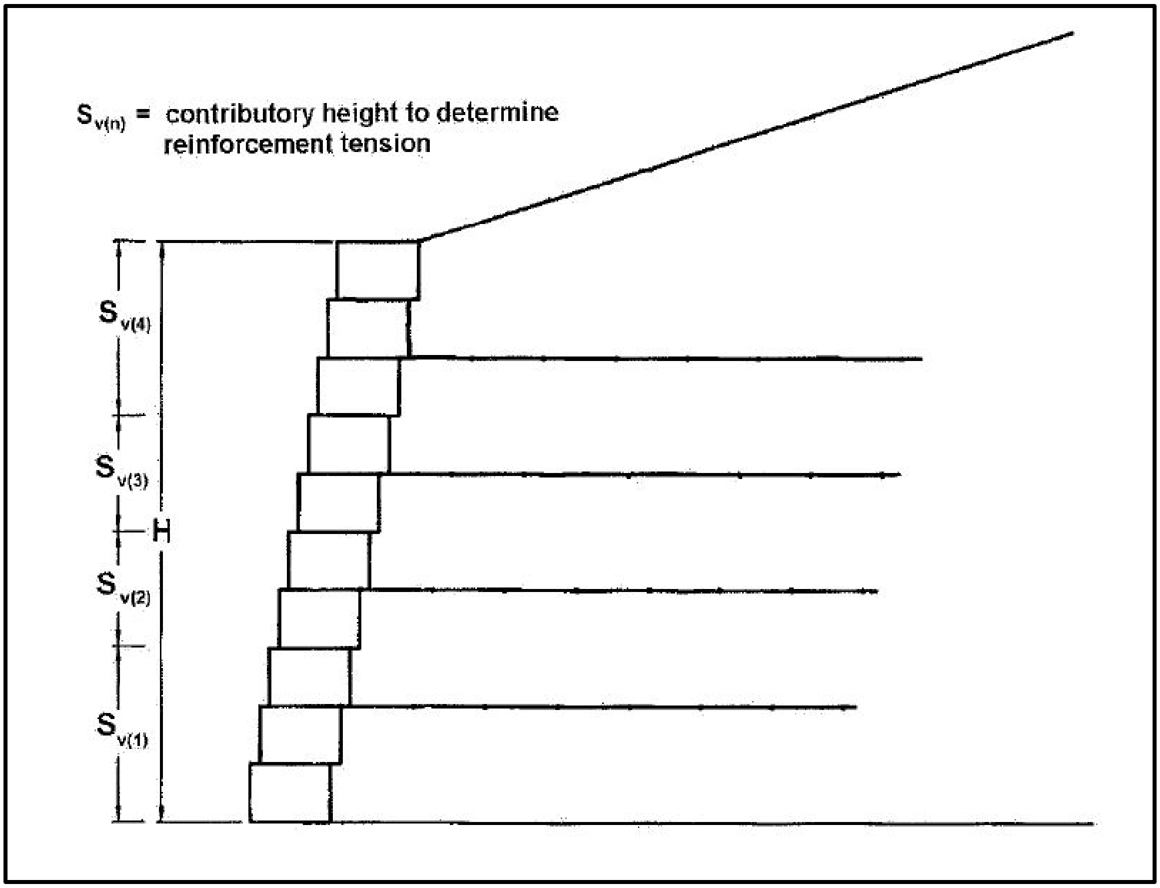


Figure C-12, Reinforcement Load Contributory Height (Berg, et al. – Vol. I (2009))

The maximum reinforcement tension, T_{MAX}, for the top and bottom layers of reinforcement, and for intermediate layers that do not have equally spaced adjacent layers, is calculated as the product of the contributory height and the average factored horizontal stress acting upon that contributory height. The average stress can be calculated based upon the tributary trapezoidal area (i.e., average of the stress at the top and at the bottom of the contributory height) or at the midpoint of the contributory height, as illustrated in Figure *C-12*.

Alternatively, for discrete reinforcements (metal strips, bar mats, geogrids, etc.), T_{MAX} (force per unit width) may be calculated at each level as $P_{TMAX\text{-}UWR}$ in terms of force per unit width of reinforcement as:

$$P_{TMAX-UWR} = \frac{\sigma_H * S_v}{R_c} \qquad \textbf{Equation C-29}$$

$$R_c = \frac{b}{S_h} \qquad \textbf{Equation C-30}$$

Where,

R_c = Ratio of gross width of strip, sheet or grid to the center-to-center horizontal spacing between strips, sheets, or grids; *R_c = 1 for sheet reinforcement; for discrete elements (i.e., strip or bar mat) see Equation C-30*

b = Gross width of the reinforcing element (see Figure C-13)
S_h = Center-to-center horizontal spacing between reinforcements (see Figure C-13)

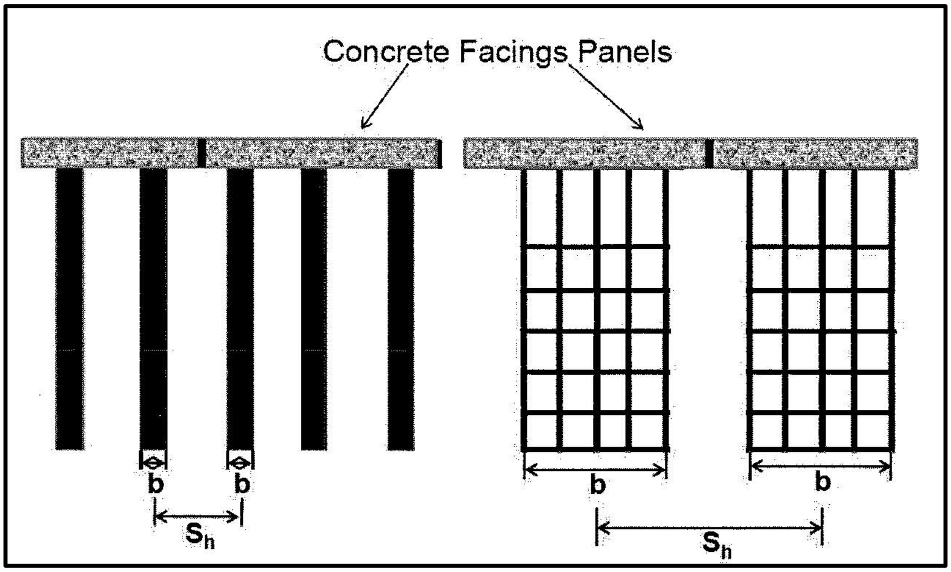


Figure C-13, Coverage Ratio
(Berg, et al. – Vol. I (2009))

For discrete reinforcements of known spacing and segmental precast concrete facing of known panel dimensions, T_{MAX} (force per unit width) can alternately be calculated per discrete reinforcement, P_{TMAX-D}, per panel width, defined as:

$$P_{TMAX-D} = \frac{\sigma_H * S_v * W_P}{N_P}$$ **Equation C-31**

Where,

P_{TMAX-D} = Maximum factored load in discrete reinforcement element
W_P = Width of panel
N_P = Number of discrete reinforcements per panel width

C.8.6 Soil Reinforcement Resistance

The sixth step in internal stability design is determining the soil reinforcement resistance. Berg, et al. – Vol. I (2009) describes determining the soil reinforcement resistance as follows:

The factored soil resistance is the product of the nominal long-term strength, coverage ratio, and applicable resistance factor, φ. The resistance factors for tensile rupture in MSE wall soil reinforcements are summarized in *Chapter 9*. The factored tensile resistance, T_r, is equal to:

$$T_r = \varphi * T_{al}$$ **Equation C-32**

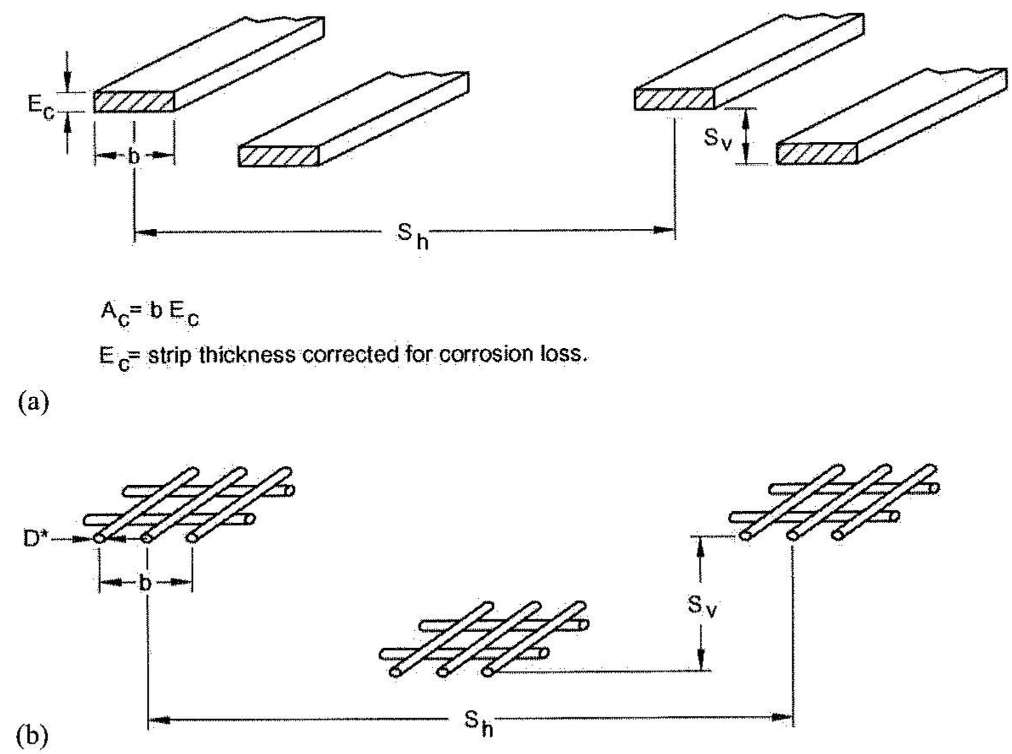
T_{al} and T_r may be expressed in terms of strength per unit width or wall, per reinforcement element, or per unit reinforcement width.

Chapter 9 indicates the resistance factors to be used in the internal design of MSE walls. It is noted that the extreme event resistance factors include both EE I and EE II limit state checks. Further, the EE II limit state check includes the impact on the traffic barrier if the traffic barrier is rigidly connected to the MSE wall and relies on the MSE wall to resist the loading. If however, the traffic barrier is designed to not impart any loading on the MSE wall then the EE II limit state check for the MSE wall is not required.

The development of T_{al} is discussed in the following sections as described by Berg, et al. – Vol. I (2009) for both inextensible (metallic) and extensible (geosynthetic) reinforcements,

> The structural design properties of reinforcement materials are a function of geometric characteristics, strength and stiffness, durability, and material type. The *2* most commonly used reinforcement materials, steel *(inextensible)* and geosynthetics *(extensible)* must be considered separately.
>
> *Strength Properties of Inextensible Reinforcements*
> For *inextensible* reinforcements, the design life is achieved by reducing the cross-sectional area of the reinforcement used in design calculations by the anticipated corrosion losses over the design life period as follows;
>
> $$E_c = E_n - E_R \qquad \textbf{Equation C-33}$$
>
> *Where,*
> *E_c = Thickness of the reinforcement at the end of the design life*
> *E_n = Nominal thickness at construction*
> *E_R = Sacrificial thickness of metal expected to be lost by uniform corrosion during the service life of the structure*
>
> The nominal long-term tensile strength of the reinforcement, T_{al}, is obtained for steel strips and grids as shown in the following equations. T_{al} in units of force per unit width is used to provide a unified strength approach, which can be applied to any reinforcement. Tensile strength of a known steel or grid reinforcement can also be expressed in terms of the tensile load carried by the reinforcement, P_{tal}. The designed designation of reinforcement tensile strength (T_{al} or P_{tal}) varies depending on whether one is designing with a known system, designing with an undefined reinforcement, checking a design layout, performing connection design or performing reinforcement pullout calculations. Thus, nominal tensile strength may be calculated and expressed in the following terms:
>
> $$T_{al} = \frac{F_y * A_c}{b} \qquad \textbf{Equation C-34}$$
>
> $$P_{tal} = F_y * A_c \qquad \textbf{Equation C-35}$$
>
> Where,
> b = Gross width of strip, sheet or grid (see Figure *C-14*)
> F_y = Yield stress of steel

A_c = Design cross section area of the steel, defined as the original cross section area minus corrosion losses anticipated to occur during the design life of the wall



$$A_c = (\text{No. of longitudinal bars}) \times \left(\frac{(D^*)^2}{4}\right)$$

D* = Diameter of bar or wire corrected for corrosion loss

b = Unit width of reinforcement (if reinforcement is continuous count the number of bars for reinforcement width of 1 unit)

Figure C-14, Geometric Configuration of Metallic Reinforcement (Berg, et al. – Vol. I (2009))

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

The quantities needed to determine A_c for steel strips and grids are shown in Figure *C-14*. The use of hardened and otherwise low strain (very high strength) steels may increase the potential for catastrophic failure; therefore, a lower resistance factor may be warranted with such materials. *The use of a lower resistance factor shall be approved in writing by the OES/GDS and OES/SDS prior to completing design*.

For metallic reinforcement, the life of the structure will depend on the corrosion resistance of the reinforcement. Practically all the metallic reinforcement used in construction of embankments and walls, whether they are strips, bar mats, or wire mesh, are made of galvanized mild steel. Woven meshes with PVC coatings provide some corrosion protection, provided the coating is not significantly damaged during construction. Epoxy coatings can be used for corrosion protection, but are susceptible to construction damage, which can significantly reduce the coatings effectiveness. *However, the use of either PVC or epoxy coated reinforcement in MSE walls shall not be allowed on SCDOT projects. This is based on anecdotal evidence that reinforcements coated with these materials do not appear to achieve the required design life required.*

Extensive studies have been made to determine the rate of corrosion of galvanized mild steel bars or strips buried in different types of soils commonly used in reinforced soil. Based on these studies, deterioration of steel strips, mesh, bars, and mats can be estimated and accounted for by using increased metal thickness.

The majority of MSE walls constructed to date have used galvanized steel *reinforcement* and backfill materials with low corrosive potential. A <u>minimum</u> galvanization coating of *either* 2.0 oz/ft^2 or 0.0034 inches (3.4 mils) thickness is required per Article 11.10.6.4.2a (*AASHTO LRFD Specifications*). Galvanization shall be applied in accordance with AASHTO M111 - *Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products* (ASTM A123 – *Standard Specification for Zinc (Hot-Dipped Galvanized) Coatings on Iron and Steel Products*) for strip type, bar mat, or grid type reinforcements and ASTM A153 –*Standard Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware* for accessory parts such as bolts and tie strips. Galvanization shall be applied after fabrication in accordance with ASTM A123. The zinc coating provides a sacrificial anode that corrodes while protecting the base metal. Galvanization also assists in preventing the formation of pits in the base metal during the first years of aggressive corrosion (which can occur in non-galvanized or “black” steel). After the zinc is oxidized (consumed) corrosion of the base metal starts.

The ASTM and AASHTO standards for galvanization provide different required minimum galvanization coating thickness as a function of the bar or wire thickness. However, as noted previously *AASHTO LRFD Specifications* require a minimum thickness of 3.4 mils for MSE walls. Galvanization requirements using this minimum and AASHTO M111 are summarized in Table *C-3*.

Table C-3, Minimum Galvanization Thickness by Steel Thickness (*modified* Berg, et al. – Vol. I (2009))

Category	Steel Thickness	Minimum Galvanization Thickness
Strip	< ¼ inch (6.4 mm)	3.4 mils (85 μm)
	> ¼ inch (6.4mm)	3.9 mils (100 μm)
Wire[1]	All diameters	3.4 mils (85 μm)
[1]For bar mats fabricated from uncoated steel wire. *After AASHTO M111 and ASTM A123*		

The corrosion rates presented in Table *C-4* are suitable for conservative design *and are only applicable to galvanized steel*. These values assume a moderately corrosive backfill material having the controlled electro-chemical property limits *presented in STS SC-M-713 (latest version) for Mechanically Stabilized Earth (MSE) Walls*.

Table C-4, Steel Corrosion Rates for Moderately Corrosive Reinforced Fill (Berg, et al. – Vol. I (2009))

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

[1]after zinc depletion

For a more detailed discussion of corrosion requirements, refer to Elias, Fishman, Christopher and Berg (2009).

Strength Properties of Extensible Reinforcement

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

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

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

$$T_{al} = \frac{T_{ult}}{RF} = \frac{T_{ult}}{RF_{ID} * RF_{CR} * RF_{D}} \qquad \textbf{Equation C-36}$$

Where,

T_{ult} = Ultimate tensile strength (strength per unit width)
RF = Product of all applicable reduction factors
RF_{ID} = Installation damage reduction factor
RF_{CR} = Creep reduction factor
RF_{D} = Durability reduction factor

RF_{ID}, RF_{CR}, and RF_D reflect actual long-term strength losses, analogous to loss of steel strength due to corrosion. This long-term geosynthetic reinforcement strength loss concept is illustrated in Figure *C-15*. As shown in the figure, some strength losses occur immediately upon installation, and others occur throughout the design life of the reinforcement. Much of the long-term strength loss does not begin to occur until near the end of the reinforcement design life.

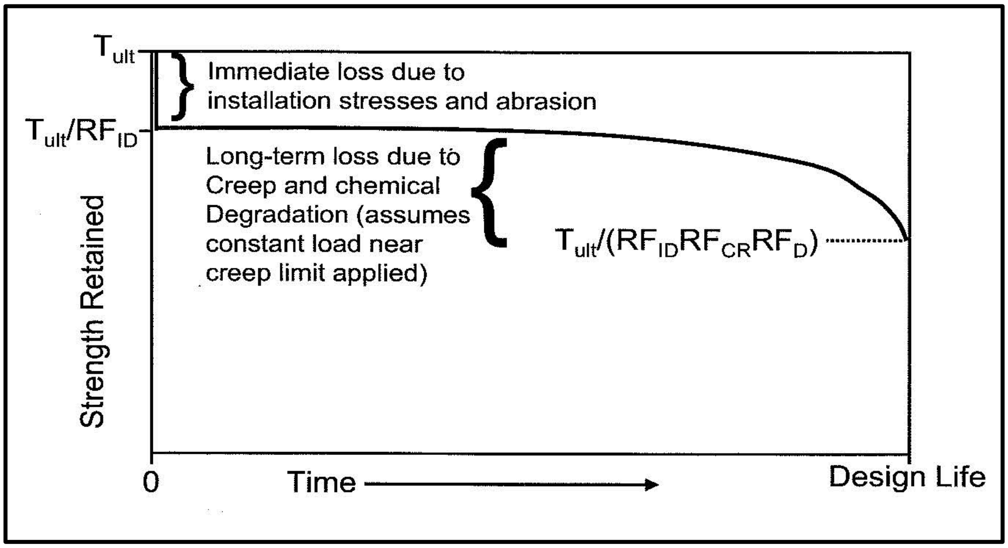


Figure C-15, Long-Term Geosynthetic Reinforcement Strength Concept (Berg, et al. – Vol. I (2009))

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

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

The determination of reduction factors for each geosynthetic product and product line requires extensive field and/or laboratory testing which can take a year or more to complete. Background regarding the determination of each long-term strength reduction factor is briefly summarized as follows:

<u>*Ultimate Tensile Strength, T_{ult}*</u> – The value selected for T_{ult}, for design purposes, is the minimum average roll value (MARV) for the product. *The tensile strength of the reinforcement is determined from wide strip tests for geotextiles per ASTM D4595 – Standard Test Method for Tensile Properties of Geotextiles by the Wide-*

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

Installation Damage Reduction Factor, RF_{ID} – Damage during handling and construction, such as from abrasion and wear, punching and tear or scratching, notching, and cracking may occur in geosynthetics. These types of damage can only be avoided by using care during handling and construction. Construction equipment should not travel directly on geosynthetic materials.

Damage during reinforced fill placement and compaction operations is a function of the severity of loading imposed on the geosynthetic during construction operations and the size and angularity of the reinforced fill. For MSE walls and RSS construction, lightweight, low strength geotextiles and geogrids should be avoided to minimize damage with ensuing loss of strength.

Protocols for field testing for this reduction factor are detailed in *Elias, et al. (2009)* and in ASTM D5818 – *Standard Practice for Exposure and Retrieval of Samples to Evaluate Installation Damage of Geosynthetics*. These protocols require that the geosynthetic material be subjected to a reinforced fill placement and compaction cycle, consistent with field practice. The ratio of the initial strength, to the strength of retrieved samples defines this reduction factor. For reinforcement applications, a minimum weight of 8.0 oz/yd^2 for geotextiles is recommended to minimize installation damage. This roughly corresponds to a Class 1 geotextile in AASHTO M288 – *Standard Specification for Geotextile Specification for Highway Applications*. In general, the combination of geosynthetic reinforcement, and backfill placement and gradation characteristics, should not result in a value of RF_{ID} greater than 1.7. If testing indicates that RF_{ID} will be greater than 1.7 (i.e., an approximate 40 percent strength loss), then that combination of geosynthetic and backfill conditions should not be used, as this or greater levels of damage will cause the remaining strength to be highly variable and therefore not adequately reliable for design.

Table *C-5* provides a summary of typical RF_{ID} values for a range of soil gradations and geosynthetic types.

Table C-5, Installation Damage Reduction Factors, RF_{ID}
(Berg, et al. – Vol. I (2009))

Geosynthetic	Type 1 Backfill Max. Size 4-inch D_{50} about 1-1/4- inch	Type 2 Backfill Max. Size ¾-inch D_{50} about #30
HDPE uniaxial geogrid	1.20 – 1.45	1.10 – 1.20
PP biaxial geogrid	1.20 – 1.45	1.10 – 1.20
PVC coated PET geogrid	1.30 – 1.85	1.10 – 1.30
Acrylic coated PET geogrid	1.30 – 2.05	1.20 – 1.40
Woven geotextiles (PP & PET)[1]	1.40 – 2.20	1.10 – 1.40
Non-woven geotextiles (PP & PET)[1]	1.40 – 2.50	1.10 – 1.40
Slit film woven PP geotexile[1]	1.60 – 3.00	1.10 – 2.00
[1]Minimum weight 8.0 oz/yd^2		

In general, RF_{ID} is strongly dependent on the backfill soil gradation characteristics and its angularity, especially for lighter weight geosynthetics. Provided a minimum of 6 inches of backfill material is placed between the reinforcement surface and the compaction and spreading equipment wheel/tracks, the backfill placement and compaction technique will have a lesser effect on RF_{ID}. Regarding geosynthetic characteristics, the geosynthetic weight/thickness or tensile strength may have a significant effect on RF_{ID}. However, for coated polyester geogrids, the coating thickness may overwhelm the effect of the product unit weight or thickness on RF_{ID}. *A minimum RF_{ID} of 1.1 shall be used to account for testing uncertainties.*

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

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

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

Typical ranges of RF_{CR} as a function of polymer type are provided in *Table C-6*.

Table C-6, Creep Reduction Factors, RF_{CR}
(Berg, et al. – Vol. I (2009))

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

Durability Reduction Factor, RF_D – This reduction factor is dependent on the susceptibility of the geosynthetic to attack by chemicals, thermal oxidation, hydrolysis, environmental stress cracking, and micro-organisms and can vary typically from 1.1 to 2.0.

Typically, polyester products (PET) are susceptible to aging strength reductions due to hydrolysis (water must be available). Hydrolysis and the resulting fiber dissolution are accelerated in alkaline regimes, percent of water saturation in the surrounding soil, and temperature. Polyolefin products (PP and HDPE) are susceptible to aging strength losses due to oxidation (contact with oxygen). The level of oxygen in reinforced fills is a function of soil porosity, groundwater location, and other factors, and has been found to be slightly less than oxygen levels in the atmosphere (21 percent). Therefore, oxidation of geosynthetics in-ground may proceed at a rate equal to those above ground. Oxidation is accelerated by the presence of transition metals (*specifically* Fe, Cu, Mn, Co, Cr) in the reinforced fill as found in acid sulphate soils (e.g., pyrite), slag and cinder fills, other industrial wastes or mine tailings containing transition metals, and elevated temperatures. It should be noted that the resistance of polyolefin geosynthetics to oxidation is primarily a function of the proprietary antioxidant package added to the base resin, which differs for each product brand, even when formulated with the same base resin.

The relative resistance of polymers to these identified regimes is shown in Table *C-7* and a choice can be made, therefore, consistent with the in-ground regimes indicated.

Table C-7, Anticipated Resistance of Polymers to Specific Environments (Berg, et al. – Vol. I (2009))

Soil Environment	Polymer		
	PET	HDPE	PP
Acid Sulphate Sols	NE	ETR	ETR
Organic Soils	NE	NE	NE
Saline Soils pH < 9	NE	NE	NE
Ferruiginous	NE	ETR	ETR
Calcareous Soils	ETR	NE	NE
Modified Soils/Lime, Cement	ETR	NE	NE
Sodic Soils, pH > 9	ETR	NE	NE
Soils with Transition Metals	NE	ETR	ETR
NE = No effect ETR = Exposure Test Required			

Most geosynthetic reinforcement is buried, and therefore ultraviolet (UV) stability is only of concern during construction and when the geosynthetic is used to wrap wall or slope faces. If used in exposed locations, geosynthetics should be protected with coatings or facing units to prevent deterioration. UV tests (ASTM D4355 – *Standard Test Method for Deterioration of Geotextiles by Exposure to Light, Moisture and Heat in a Xenon Arc Type Apparatus*) extended beyond the normal 500 hour test duration should be performed on materials that will be directly exposed for long periods of time (more than several months) in order to evaluate the materials anticipated design life. Vegetative covers can also be considered in the case of open weave geotextiles or geogrids. Thick geosynthetics with UV stabilizers can be left exposed for several years or more without protection; however, long-term maintenance should be anticipated because of both UV deterioration and possible vandalism.

Protocols for testing to obtain this reduction factor have been proposed and are detailed in Elias, et al. (1999). In general, for polyolefins, they consist of oven aging polyolefins (PP and HDPE) samples to accelerate oxidation and measure their strength reduction, as a function of time, temperature, and oxygen concentration. This high temperature data must then be extrapolated to a temperature consistent with field conditions. For polyesters (PET) the aging is conducted in an aqueous media at varying pH and relatively high temperature to accelerate hydrolysis, with data extrapolated to a temperature consistent with field conditions. For more detailed explanations, see Elias, et al. (2009).

Due to the long-term nature of these durability evaluation protocols (2 to 3 years could be required to complete such test), it is generally not practical to conduct such tests for typical geosynthetic reinforcement design, but are generally more suited for research activities. However, short-term index type tests can be conducted as indicators of good long-term durability performance, based on correlation to the long-term research results obtained and reported by Elias, et al. (1999). Such index test results, combined with a criteria applied to the test results that can be considered to indicate good long-term performance, can be used to justify the use of a default value of RF_D that can be used for the determination of T_{al}.

With respect to aging degradation, current research results suggest the following.

Polyester Geosynthetics

PET geosynthetics are recommended for use only in environments characterized by $3 < pH \leq 9$. The reduction factors for PET aging (RF_D) listed in Table *C-8* are developed for a 100-year design life in the absence of long-term product specific testing. Based on these research results, for polyester reinforcements, the *AASHTO LRFD Specifications* recommend a minimum average molecular weight of 25,000 and a maximum carboxyl end group content (CEG) of 30 to allow the use of the default reduction factor for durability.

Table C-8, RF_D for PET
(Berg, et al. – Vol. I (2009))

Product[a]	**RF_D**	
	$5 < pH \leq 8$	**$3^b < pH \leq 5$ $8 < pH \leq 9$**
Geotextiles $M_n < 20{,}000$, $40 < CEG < 50$	1.6	2.0
Coated Geogrids, Geotextiles $M_n > 25{,}000$, $CEG < 30$	1.15	1.3
M_n = Number average molecular weight CEG = Carboxyl end group		
[a]Use of materials outside the indicated molecular property range requires specific product testing. Use of products outside of $3 < pH < 9$ range is not recommended.		
[b]Lower limit of pH for permanent applications is 4.5 and lower limit for temporary applications is 3, per Article 11.10.6.4.2b (AASHTO LRFD Specifications)		

Polyolefin Geosynthetics

To mitigate thermal and oxidative degradative processes, polyolefins (i.e., PP and HDPE) products are stabilized by the addition of antioxidants for both processing stability and long-term functional stability. These antioxidant packages are proprietary to each manufacturer and their type, quantity, and effectiveness varies. Without residual antioxidant protection (after processing), PP products are vulnerable to oxidation and significant strength loss within a projected 75- to 100-year design life at 68° F. Current data suggests that unsterilized PP has a half-life of less than 50 years.

Therefore the anticipated functional life of a PP geosynthetic is to a great extent a function of the type and post-production antioxidant levels, and the rate of subsequent antioxidant consumption. Antioxidant consumption is related to the in-ground oxygen content, which in fills is only slightly less than atmospheric.

A detailed discussion of the effectiveness of oven aging and other protocols to allow estimation of long-term strength loss due to the combination of heat aging and oxidative degradation of various polyolefins is provided in Elias, et al. (1999) and Elias, et al. (2009).

For both polyester and polyolefins, if the index test criteria are met, a default value of RF_D of 1.3 could be used to determine T_{al} for design purposes. These index criteria are summarized in Table *C-9*. If the effective in-soil site temperature is anticipated to be approximately 85° F plus or minus a few degrees, a higher default reduction factor for RF_D should be considered.

Table C-9, Minimum Testing Requirements for use RF_D (modified Berg, et al. – Vol. I (2009))

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

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

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

Environmental stress cracking is an aging phenomenon that is really as much related to creep as it is to durability. In certain environments, such as when surfactants are present, the creep rupture process, through making it easier for the tie molecules to pull out of the crystalline structure, can be accelerated, allowing cracks in the polymer to form, and premature rupture *to occur*. Additional information on this phenomenon is provided in Elias, et al. (2009). For most in ground conditions, the chemicals necessary to cause this to happen are generally not present, and the results from laboratory creep testing are sufficient to address strength loss under constant load.

Note that biological degradation due to micro-organisms is rarely a concern, as most geosynthetic reinforcement products only contain high molecular weight polymers, and the biological agents have great difficulty in finding the molecular chain endings that would allow them to begin consuming the polymer. Therefore, biological degradation is usually not considered in the determination of RF_D. A minimum RF_D of 1.1 shall be used to account for testing uncertainties.

C.8.7 Strength and Number of Soil Reinforcements

The seventh step in internal stability design is determining the grade and number of soil reinforcement elements at each level. Berg, et al. – Vol. I (2009) describes this selection process as follows:

> The soil reinforcement vertical layout, the factored tensile force at each reinforcement level, and the factored soil reinforcement resistance were defined in the previous steps. With this information, select suitable *strength* of reinforcement, *the* number of (e.g., *discrete (strip) or sheet*) reinforcements, for the defined vertical reinforcement layout; then with this layout check pullout *at Strength and Service limit state loads* and, as applicable, *Extreme Event limit state* loadings. Adjust layout if/as necessary.
>
> Stability with respect to breakage of the reinforcements requires that:
>
> $$T_{MAX} \leq T_r \qquad \textbf{Equation C-37}$$
>
> Where T_{MAX} is the maximum factored load in a reinforcement (*Equation C-28*) and T_r is the factored reinforcement tensile resistance (*Equation C-32*).

C.8.8 Calculate Factored Pullout Resistance of Soil Reinforcements

The eighth step in internal stability design is determining the factored pullout resistance of the soil reinforcement elements. Berg, et al. – Vol. I (2009) describes this process as follows:

> Stability with respect to pullout of the reinforcement requires that the factored effective pullout length is greater than or equal to the factored tensile load in the reinforcement, T_{MAX}. Each layer of reinforcement should be checked, as pullout resistance and/or tensile loads may vary with reinforcement layer. Therefore, the following criteria should be satisfied:
>
> $$\varphi * L_e = \frac{T_{MAX}}{F^* * \alpha * \sigma_v * C * R_c} \qquad \textbf{Equation C-38}$$
>
> Where,
>
> L_e = Length of embedment in the resisting zone. Note that the boundary between the resisting and active zones may be modified by concentrated loadings.
>
> φ = Resistance factor for soil reinforcement pullout (*see Chapter 9*)
>
> T_{MAX} = Maximum reinforcement tension (*see Equation C-31, P_{TMAX-D} in Equation C-31*)
>
> F^* = Pullout resistance factor with variation in depth starting at the same elevations as that for K_r/K_a variation (*discussed in C.8.8.2*)
>
> α = Scale correction factor (*discussed in C.8.8.1)*

σ_v = Nominal (i.e., unfactored) vertical stress at the reinforcement level in the resistant zone, including distributed dead load surcharges, neglecting traffic loads (ksf). See Figure *C-16* for computation of horizontal backslope condition and Figure C-17 for the sloping backslope condition.
C = 2 for strip, grid, and sheet type reinforcement
R_c = Coverage ratio (*see Equation C-30*)

The vertical stress, σ_v, used to calculate pullout resistance for level backslope condition shall be determined as shown in Figure C-16 using the following equation.

$$\sigma_v = \gamma_r * Z$$ **Equation C-39**

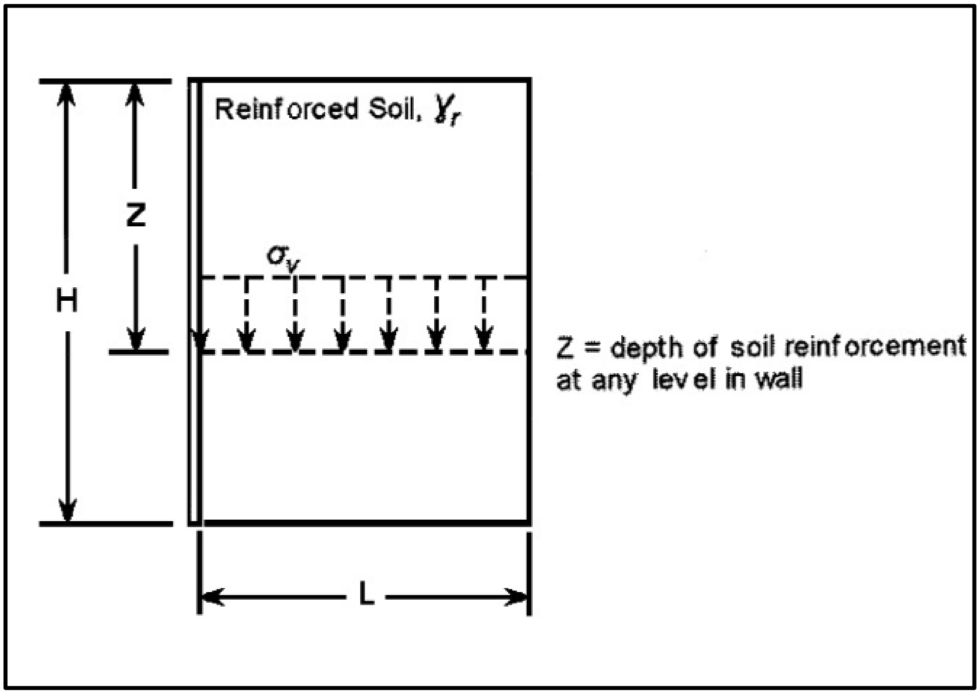


Figure C-16, Calculation of Vertical Stress for Internal Stability Analysis (modified from AASHTO LRFD Specifications (2020))

The vertical stress, σ_v, used to calculate pullout resistance for the sloping backslope condition shall be determined as shown in Figure C-17 using the following equations.

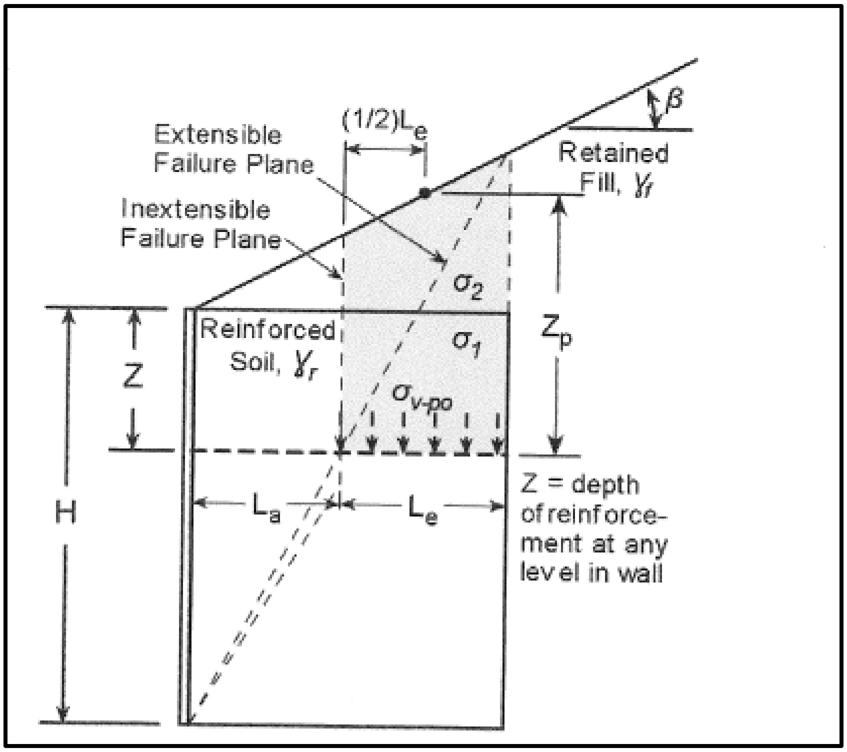


Figure C-17, Calculation of Vertical Confining Pressure beneath Sloping Backfill (AASHTO LRFD Specifications (2020))

$$\sigma_1 = \gamma_r * Z \qquad \textbf{Equation C-40}$$

$$\sigma_2 = \gamma_f * (Z_p - Z) \qquad \textbf{Equation C-41}$$

$$\sigma_v = \sigma_1 + \sigma_2 = \gamma_r * Z + \gamma_f * (Z_p - Z) \qquad \textbf{Equation C-42}$$

Where:

$$Z_p = Z + \left(L_a + \left(\frac{1}{2}\right) L_e\right) tan \qquad \textbf{Equation C-43}$$

Therefore, the required embedment length in the resistance zone (i.e., beyond the potential failure surface) can be determined from:

$$L_e \geq \frac{T_{MAX}}{\varphi * F^* * \alpha * \sigma_v * C * R_c} \geq 3\ ft \qquad \textbf{Equation C-44}$$

If traffic or other live load is present, it is recommended that T_{MAX} be computed with the live loads and that the pullout resistance be computed excluding the live loads. This addresses the possibility of the live loads being present near the front of the wall but not above the reinforcement embedment length. The pullout resistance and the T_{MAX} can be calculated with the live load excluded if it can be shown that the live load will be on the active and resistant zones at the same time or on the resistant zone alone.

Commentary C11.10.6.2.1 *(AASHTO LRFD Specifications)* notes that traffic loads and other live loads are not included for pullout calculations. Therefore, if T_{MAX} calculation for checking the reinforcement and connection strengths included a live load surcharge the value must be recomputed, without the surcharge load.

If the criterion is not satisfied for all reinforcement layers, the reinforcement length has to be increased and/or reinforcement with a greater pullout resistance per unit width must be used, or the reinforcement vertical spacing may be reduced which would reduce T_{MAX}.

The total length of reinforcement, L, required for internal stability is then determined from:

$$L = L_a + L_e$$ **Equation C-45**

Where, L_a is obtained from *Figure C-9* 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:

$$L_a = (H - Z) * tan\left(45 - \frac{\phi'}{2}\right)$$ **Equation C-46**

Where,

Z = Depth of the reinforcement level

For walls with inextensible reinforcement, vertical face and horizontal backfill, from the base up to H/2:

$$L_a = 0.6 * (H - Z)$$ **Equation C-47**

For the upper half of a wall with inextensible reinforcements, vertical face, and horizontal backfill:

$$L_a = 0.3 * H$$ **Equation C-48**

For construction ease, a final uniform length is commonly chosen, based on the maximum length required. 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.8.8.1 Correction Factor (α)

The correction factor (α) depends primarily upon the strain softening of the compacted granular backfill material, the extensibility, and the length of the reinforcement. Typical values of α based on reinforcement type are presented in Table C-10. For inextensible reinforcement, α is approximately 1, but it can be substantially smaller than 1 for extensible reinforcements. The α factor can be obtained from pullout tests on reinforcements with different lengths or derived using analytical or numerical load transfer models which have been "calibrated" through numerical test simulations. In the absence of test data, the values included in Table C-10 should be used for geogrids and geotextiles.

Table C-10, Typical Values of α
(modified Berg, et al. – Vol. I (2009))

Reinforcement Type	α
All steel reinforcements	1.0
Geogrids	0.8
Geotextiles	0.6

C.8.8.2 Pullout Friction Factor (F*)

The pullout friction factor (F*) 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:

$$\boldsymbol{F^* = F_q * \alpha_\beta + \tan\rho} \qquad \textbf{Equation C-49}$$

Where,

F_q = Embedment (or surcharge) bearing capacity factor (see Figure C-18)

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

ρ = Soil-reinforcement interaction friction angle

Equation C-45 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:

$$\boldsymbol{F^* = \tan\rho = 1.2 + \log C_u} \qquad \textbf{Equation C-50}$$

at the top of the structure = 2.0 maximum

$$\boldsymbol{F^* = \tan\phi} \qquad \textbf{Equation C-51}$$

at a depth of 20 feet and below

Where,

ρ = Soil-reinforcement interaction friction angle

C_u = Uniformity coefficient of the backfill (see Chapter 6)
If the specific C_u for the wall backfill is unknown during design, a C_u of 4 should be assumed (i.e., F* = 1.8 at the top of the wall), for backfill meeting the requirements previously provided.

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

At the top of the structure:

$$F^* = F_q * \alpha_\beta = 40\alpha_\beta = 40 * \left(\frac{t}{2S_t}\right) = 20 * \left(\frac{t}{S_t}\right) \quad \textbf{Equation C-52}$$

At a depth of 20 feet and below:

$$F^* = F_q * \alpha_\beta = 20\alpha_\beta = 20 * \left(\frac{t}{2S_t}\right) = 10 * \left(\frac{t}{S_t}\right) \quad \textbf{Equation C-53}$$

Where,
t = 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 (see Figure C-18)

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:

$$F^* = \left(\frac{2}{3}\right) * \tan\phi \quad \textbf{Equation C-54}$$

Where,
ϕ = Peak friction angle of the MSE wall backfill

When used in the above relationship, ϕ is the peak friction angle of the soil which, for MSE walls using select granular backfill, is taken as 36° unless project specific test data substantiates higher values. However, ϕ shall not exceed 38° even if specific test data indicates higher friction values.

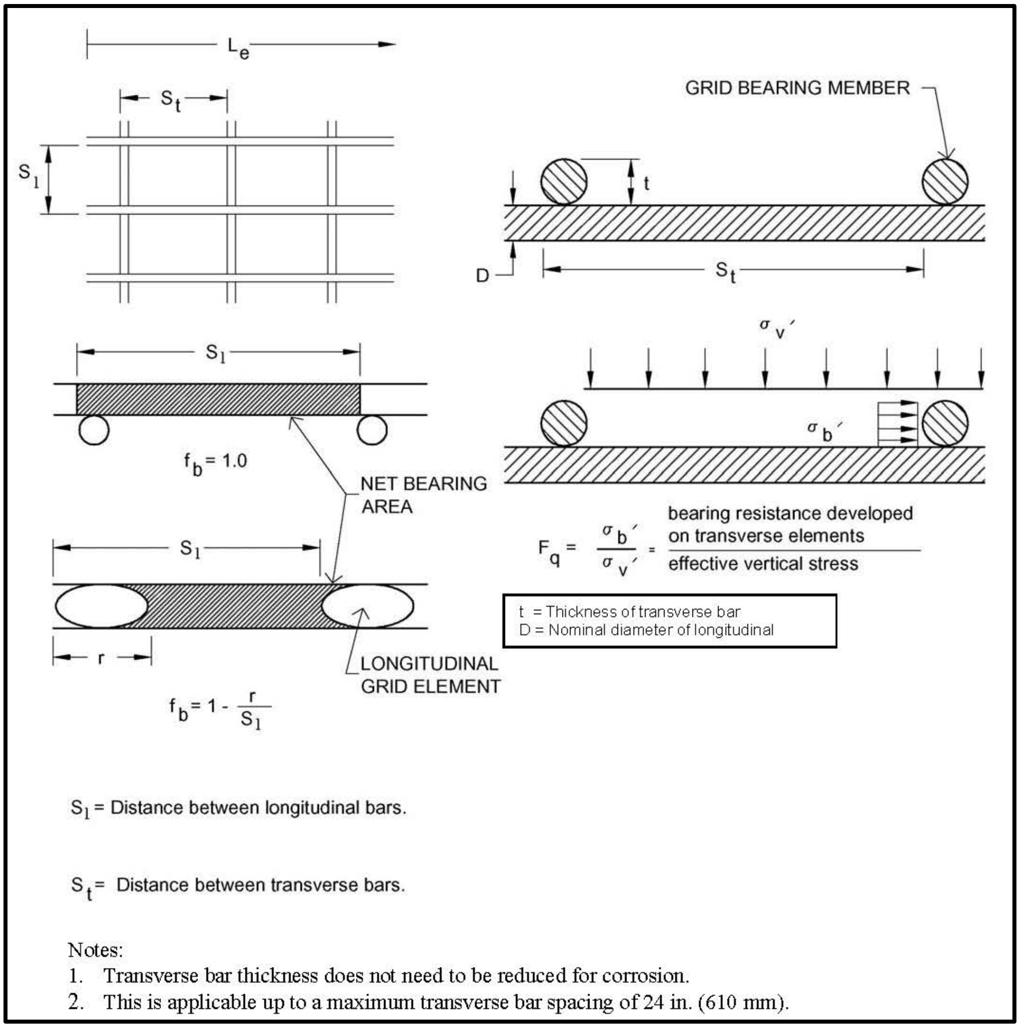


Figure C-18, Grid Dimensions for Pullout Capacity (Berg, et al. – Vol. I (2009))

C.8.9 T_{ac} for Connection Strength

The ninth step in internal stability design is determining the connection strength between the facing elements and the soil reinforcement elements. Berg, et al. – Vol. I (2009) describes this process as follows:

> The connection of the reinforcements with the facing should be designed for T_{MAX} for all limit states. The resistance factors (φ) for the connectors are the same as for the reinforcement strength, and are *provided in Chapter 9.*
>
> Connections to Concrete Panels
> The metallic reinforcements for MSE systems constructed with segmental precast panels are structurally connected to the facing by either bolting the reinforcements to a tie strip cast in the panel or connected with a bar connector to suitable anchorage devices in the panels. The capacity of the embedded connector as an anchorage must be checked by the tests as required by Article 5.11.3 *AASHTO LRFD Specifications* for geometry used. Connections between metallic reinforcements and facing units should be designed in accordance with Article

6.13.5 (*AASHTO LRFD Specifications*), and consider corrosion losses in accordance with Article 11.10.6.4.2A (*AASHTO LRFD Specifications*). The design load at the connection is equal to the maximum load on the reinforcement.

Polyethylene geogrid reinforcements may be structurally connected to segmental precast panels by casting a tab of the geogrid into the panel and connecting to the full length of geogrid with a bodkin joint, as illustrated in Figure *C-19*. The capacity of the embedded connector as an anchorage must be checked by tests as required in Article 5.11.3 *AASHTO LRFD Specifications* for each geometry used. A slat of polyethylene is used for the bodkin. Care should be exercised during construction to eliminate slack from this connection.

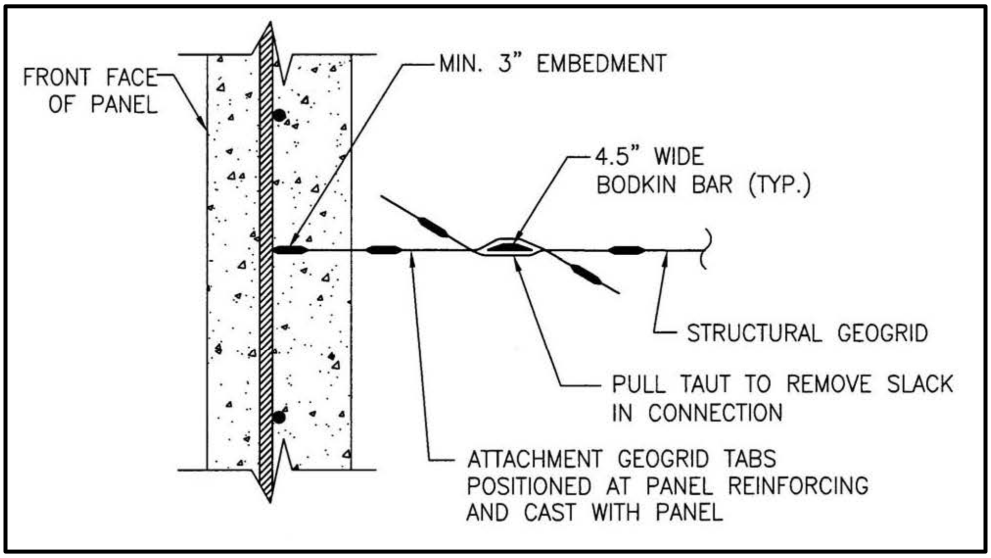


Figure C-19, Bodkin Connection Detail
(Berg, et al. – Vol. I (2009))

Polyester geogrids and geotextiles should not be cast into concrete for connections, due to potential chemical degradation. Other types of geotextiles also are not cast into concrete for connections due to fabrication and field connection requirements.

Connections to MBW Units

MSE walls constructed with Modular Block Wall (MBW) units are connected either by (i) a structural connection subject to verification under Article 5.11.3 (*AASHTO LRFD Specifications*), (ii) friction between units and the reinforcement, including the friction developed from the aggregate contained within the core of the units, or, (iii) a combination of friction and shear from connection devices. This strength will vary with each unit depending on its geometry, unit batter, normal pressure, depth of unit, and unit infill gravel (if applicable). The connection strength is therefore specific to each unit/reinforcement combination and must be developed uniquely by test for each combination.

The nominal long-term connection strength, T_{alc} developed by frictional and/or structural means is determined as follows:

$$\boldsymbol{T_{alc} = \frac{T_{ult} * CR_{CR}}{RF_{Dc}}} \qquad \textbf{Equation C-55}$$

Where,

T_{alc} = Nominal long-term reinforcement/facing connection strength per unit reinforcement width at a specified confining pressure

T_{ult} = Ultimate tensile strength of the geosynthetic soil reinforcement, defined as MARV

RF_{Dc} = Reduction factor to account for chemical and biological degradation at the connection

CR_{CR} = Reduction factor to account for reduced ultimate strength resulting from the connection

CR_{CR} may be obtained from long-term or short-term tests, as described below.

<u>CR_{CR} Defined with Long-Term Testing</u>

A series of connection creep tests are performed over extended periods of time to evaluate creep rupture at the connection. The long-term connection creep rupture data is extrapolated to the specified design life (e.g., 75 years, 100 years*, etc.*) to define the creep reduced connection strength, T_{CRc}, as the specified design life. Details for long-term testing and interpretation of results are presented in Appendix B *of Berg, et al. – Vol. II (2009)*. With this long-term testing, CR_{CR} is defined as follows:

$$\boldsymbol{CR_{CR} = \frac{T_{CRc}}{T_{lot}}} \qquad \textbf{Equation C-56}$$

T_{lot} is the ultimate wide width tensile strength of the reinforcement material roll/lot used for the connection strength testing. The T_{lot} strength, for example, might be 103 percent to 115 percent of the MARV ultimate strength, T_{ult} (or noted $T_{ult\text{-}MARV}$).

<u>CR_{CR} Defined with Short-Term Testing</u>

Short-term (i.e., quick) ultimate strength tests, per ASTM D6638 – *Standard Test Method for Determining Connection Strength Between Geosynthetic Reinforcement and Segmental Concrete Units (Modular Concrete Blocks)*, are used to define an ultimate connection strength, $T_{ultconn}$, at a specified confining pressure. With short-term testing CR_{CR} is defined as follows:

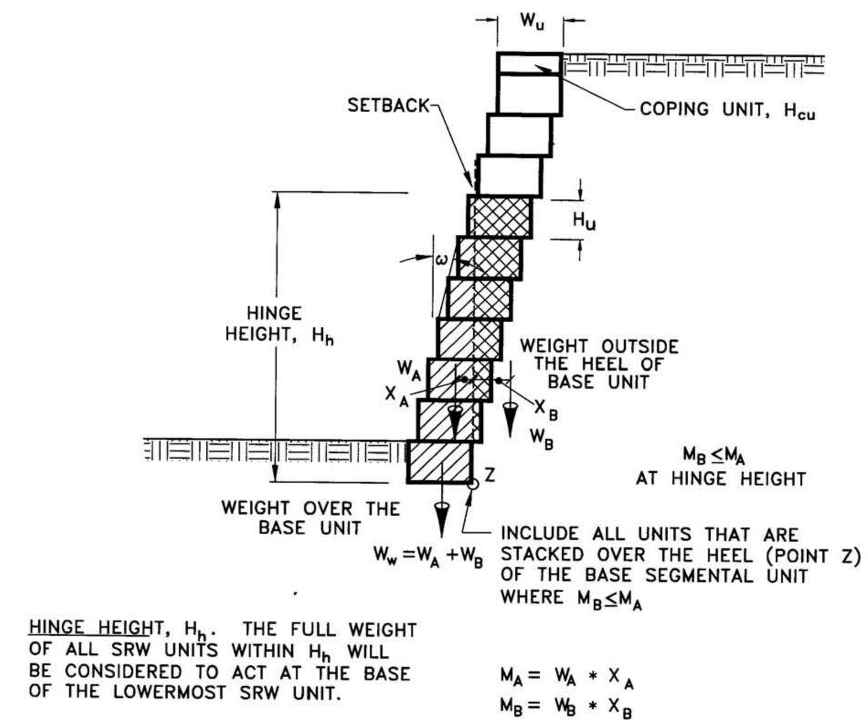
$$\boldsymbol{CR_{CR} = \frac{T_{ultconn}}{RF_{CR} * T_{lot}}} \qquad \textbf{Equation C-57}$$

RF_{CR} is the geosynthetic creep reduction factor (*see above*), and T_{lot} is the ultimate wide width tensile strength of the reinforcement material roll/lot used for the connection strength testing.

Raw data from short-term connection strength laboratory testing should not be used for design. The wall designer should evaluate the data and define the nominal long-term connection strength, T_{alc}. Steps for this data reduction are summarized and discussed in Appendix B *of Berg, et al. – Vol. II (2009)*. An example of reduction of short-term connection strength data is presented in Appendix B *of Berg, et al. – Vol. II (2009)*.

Note that the environment between and directly behind modular blocks at the connection may not be the same as the environment with the reinforced soil zone. Therefore, the long-term environmental aging factor (RF_{Dc}) may be significantly different than that used in computing the nominal long-term reinforcement strength T_{al}.

The connection strength as developed above is a function of normal pressure, which is developed by the weight of the units. Thus, it will vary from a minimum in the upper portion of the structure to a maximum near the bottom of the structure for walls with no batter. Further, since many MBW walls are constructed with a front batter, the column weight above the base of the wall or above any other interface may not correspond to the weight of the facing units above the referenced elevation. The concept is shown in Figure *C-20*, and is termed a hinge height (*Simac, Bathurst, Berg and Lothspeich (1993)*). Hence, for walls with a nominal batter or more than 8 degrees, the normal stress is limited to the lesser of the hinge height or the height of the wall above the interface. This vertical pressure range should be used in developing CR_{CR}. This recommendation is based on research findings that indicated that the hinge height concept is overly conservative for walls with small batters (*Bathhurst, Walters, Vlachopoulos, Burgess and Allan (2000)*).



$$H_h = 2(W_u - G_u)/\tan \omega$$

where:

H_u = SRW unit height (ft)
W_u = SRW unit width, front to back (ft)
G_u = distance to the center of gravity of a horizontal SRW unit including aggregate fill, measured from the front of the unit (ft)
ω = wall batter due to setback per course (deg)
H = total height of wall (ft)
H_h = hinge height (ft)

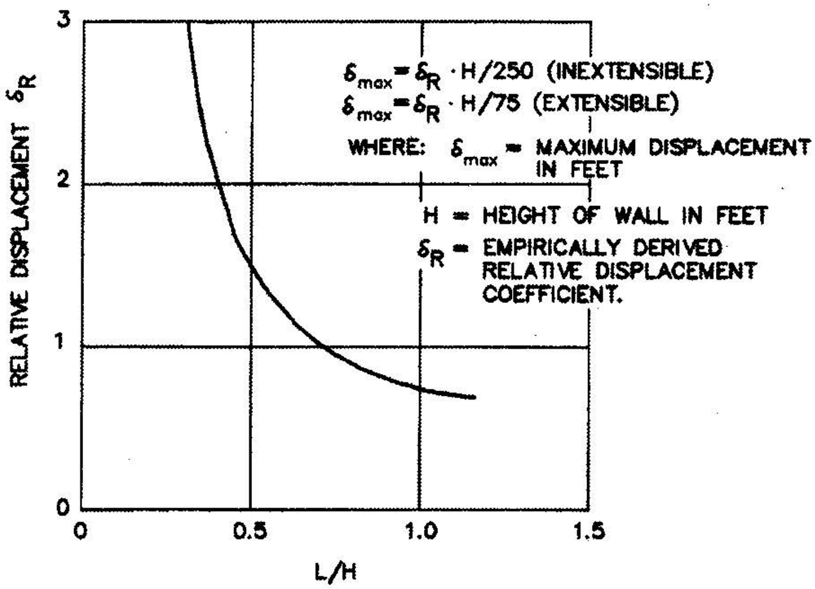
Figure C-20, Hinge Height of Modular Block MSE Walls (Berg, et al. – Vol. I (2009))

C.8.10 <u>Estimation of Lateral Movements</u>

The tenth step in internal stability design is estimating the lateral movements that are anticipated to occur within the reinforced soil zone. These movements are required to fully engage the soil resistance to prevent pullout of the reinforcement. Therefore, the MSE wall face shall be designed and constructed with a positive batter (i.e., the face of the MSE wall shall tilt toward the soil). The required batter shall be clearly indicated on the construction drawings. Berg, et al. – Vol. I (2009) describes lateral movements as follows:

> The evaluation of lateral wall movements in LRFD is the same as in ASD as the deformations are evaluated at the Service I limit state. In general, most internal lateral deformations of an MSE wall face usually occur during construction. Post construction movements, however, may take place due to post construction surcharge loads, settlement of wall fill, or long-term settlement of the foundation soils.

The magnitude of lateral displacement depends on fill placement techniques, compaction effects, reinforcement extensibility, reinforcement length, reinforcement-to-facing connection details, and details of the wall facing. The rough estimate of probable lateral displacements of simple MSE walls that may occur during construction can be estimated based on empirical correlations (*see Figure C-21*).



For L = 0.7 H
Metallic (inextensible) reinforcement ≈ ¾-in. per 10 ft of wall height
Geogrid (moderately extensible) reinforcement ≈1 in. per 10 ft of wall height
Geotextile (extensible) reinforcement ≈1.5 in. per 10 ft of wall height

Based on 20 ft high walls, relative displacement increases approximately 25% for every 400 psf surcharge. Experience indicates that for higher walls, the surcharge effect may be greater.

Note: This figure is only a guide. Actual displacement will depend, in addition to the parameters addressed in the figure, on soil characteristics, compaction effort, and contractor workmanship.

Figure C-21, Empirical Curve for Estimating Lateral Displacement (Berg, et al. – Vol. I (2009))

In general, increasing the length-to-height ratio of reinforcement, from its theoretical lower limit of 0.5H to the *AASHTO LRFD Specification* specified 0.7H decreases the deformation by about 50 percent. For critical structures requiring precise tolerances, such as bridge abutments, more accurate calculations using numerical modeling may be warranted.

A deformation response analysis allows for an evaluation of the anticipated performance of the structure with respect to horizontal (and vertical) displacement. Horizontal deformation analyses are the most difficult and least certain of the performed analyses. In many cases, they are done only approximately. The results may impact the choice of facing, facing connections and backfilling sequences.

C.8.11 <u>Vertical Movement and Bearing Pad Check</u>

The final step in internal stability design is checking the vertical movement and bearing pad. Berg, et al. – Vol. I (2009) describes the vertical movement and bearing pad check as follows:

> Bearing pads are placed in horizontal joints of segmental precast concrete panels in order to allow the panel and the reinforcement to move down with the reinforced fill as it is placed and settles, mitigate downdrag stress, and provide flexibility for differential foundation settlements. Internal settlement within the reinforced fill is practically immediate with some minor movement occurring after construction due to elastic compression in granular materials. The amount of total movement is the combination of the internal movement and external differential movement. The bearing/compression pad thickness and compressibility could be adjusted according to the anticipated movement. Otherwise concrete panel cracking and/or downdrag on connections resulting in bending of the connections and/or out of plane panel movement can occur. Calculation of the external settlement *is discussed previously*. Normally the internal movement is negligible for well graded, granular fill and external movement will usually control the compression pad requirements. However, when using sand type fill and/or marginal fill containing an appreciable amount of fines, the internal movement can be significant and should be calculated to evaluate additional thickness requirements of the bearing pad. Immediate settlement of granular *material* can be calculated *as indicated in Chapter 17*.
>
> The stiffness (axial and lateral), size, and number of bearing pads should be sized such that the final joint opening will be at least 3/4 ± 1/8-inch, unless otherwise shown on the plans. A minimum initial joint width of 3/4-inch is recommended. The stiffness (axial and lateral), size, and number of bearing pads should be checked assuming a vertical loading at a given joint is equal to 2 to 3 times the weight of facing panels directly above that level. Laboratory tests in the form of vertical load-vertical strain and vertical load-lateral strain curves of the bearing pads are required for this check.

C.9 DESIGN OF FACING ELEMENTS

Berg, et al. – Vol. I (2009) indicates that the next major design step for an MSE wall is the facing elements. Precast concrete (panels or full height tilt up panels) or MBW units shall be designed by either the SEOR or the MSE wall supplier's engineer. For the design of concrete, steel, or timber facings, Berg, et al. – Vol. I (2009) indicates the following:

> Facing elements are designed to resist the horizontal forces *developed previously*. Reinforcement is provided to resist the maximum loading conditions at each depth in accordance with structural design requirements in Sections 5, 6, and 8 *of the AASHTO LRFD Specifications*, for concrete, steel, and timber facings, respectively. The embedment of the soil reinforcement to panel connector must be developed by test, to ensure that it can resist the T_{MAX} loads *as required in Section 5 of AASHTO LRFD Specifications*.

Typically, the finish for MSE wall panels is Deep Fractured Fin and for small block the finish is roughened surface (granite) (see SCDOT Standard Drawing 701-950-01). MSE Wall panels and small blocks shall all be natural gray, Federal Standard Color 36173. Alternate finishes are available, see SCDOT Standard Drawings 701-950-02, 701-950-03, 701-975-00, 701-990-01 and 901-990-02. Contact the SCDOT Project Manager to determine if an alternate finish is desired and which one has been selected. Note that an additional drawing sheet may be required to indicate the alternate finish and the layout of the alternate finish.

Berg, et al. - Vol. I (2009) indicates the following with regard to the design of MSE walls with flexible facing elements. The use of flexible facing elements is anticipated for temporary and 2-stage MSE walls.

> Welded wire or similar facing panels should be designed in a manner that prevents the occurrence of excessive bulging as backfill behind the facing elements compresses due to compaction stresses, self-weight of the backfill or lack of section modulus. Bulging at the face between soil reinforcement elements in both the horizontal and vertical direction generally should be limited to 1 to 2 inches as measured from the theoretical wall line. Specification requirements and design detailing to help achieve this tolerance might include limiting the face panel height to 18 inches or less, the placement of a nominal 2-foot wide zone of rockfill or cobbles directly behind the facing, decreasing the vertical and horizontal spacing between reinforcements, increasing the section modulus of the facing material, and/or by providing sufficient overlap between adjacent facing panels. Furthermore, the top of the flexible facing panel at the top of the wall should be attached to a soil reinforcement layer to provide stability to the top of the facing panel.
>
> Geosynthetic facing elements generally should not be left exposed to sunlight (specifically UV radiation) for permanent walls. If geosynthetic facing elements must be left exposed permanently to sunlight, the geosynthetic should be stabilized to be resistant to UV radiation. Furthermore, product specific test data should be provided which can be extrapolated to the intended design life and which proves that the product will be capable of performing as intended in an exposed environment. Alternatively a protective facing should be constructed in addition (e.g., concrete, shotcrete, etc.).

C.10 OVERALL STABILITY

The overall (global) stability is typically determined by the GEOR. This stability shall 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, in overall stability 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 Strength and Service limit states and are normally performed once the initial estimate of the reinforcement length is determined. The Service limit state check is to determine if any movements, lateral or vertical are anticipated occurring under the design loading conditions. 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.

C.11 COMPOUND STABILITY

Prior to submission of the final design plans, a compound global stability analysis shall be performed by the MSE wall supplier. Compound stability analyses and failure surfaces are described by Berg, et al. – Vol. I (2009) as follows:

> Additional slope stability analyses should be performed for MSE walls to investigate potential compound failure surfaces, i.e., failure planes that pass behind or under and through a portion of the reinforced soil zone as illustrated in Figure *C-22*. For simple structures with rectangular geometry, relatively uniform reinforcement spacing, and a near vertical face, compound failures passing both through the unreinforced and reinforced zones will not generally be critical. However, if complex conditions exist such as changes in reinforced soil types or reinforcement lengths, high surcharge loads, seismic loading, sloping faced structures, significant slopes at the toe or above the wall or stacked (tiered) structures, compound failures must be considered.

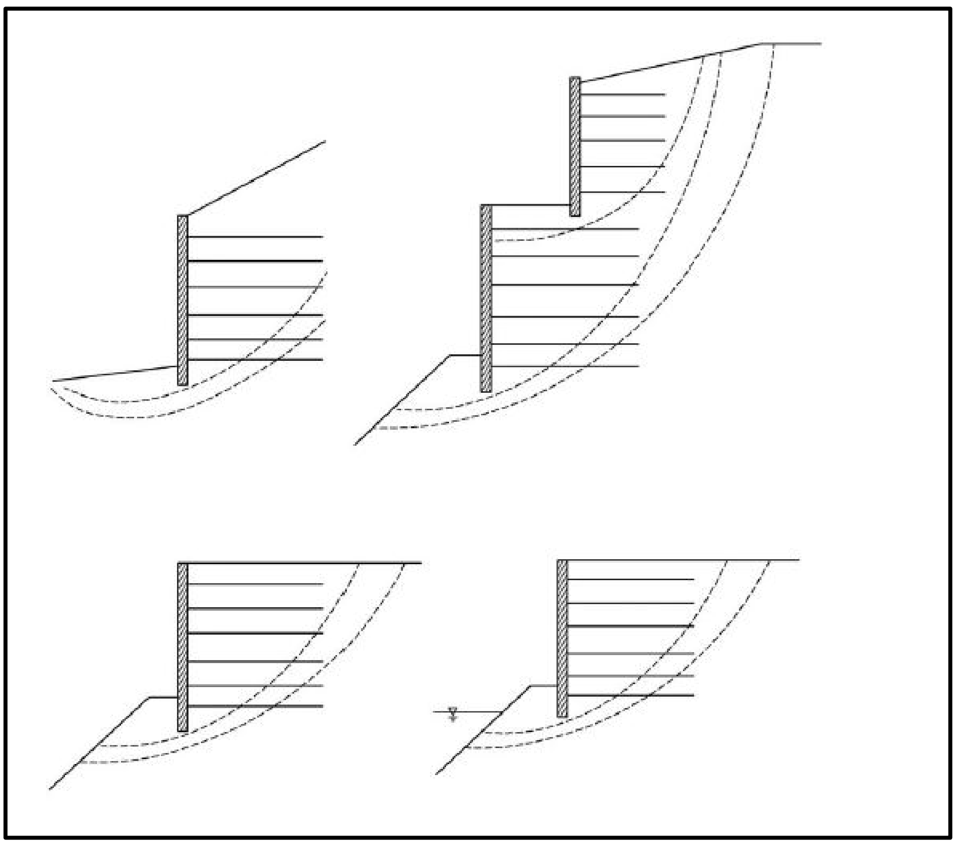


Figure C-22, Compound Stability MSE Wall Geometries (Berg, et al. – Vol. I (2009))

As indicated in Figure C-22, 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. The GEOR will show on the plans the necessary soil parameters for the retained fill and the foundation soils. The compound analysis shall be performed by the MSE wall supplier using the information supplied by the GEOR. In addition, the MSE wall supplier should use the MSEW software package as prepared and provided by ADAMA Engineering, Inc.

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

C.12 WALL DRAINAGE SYSTEM DESIGN

The following Section is adopted directly from Berg, et al. – Vol. I (2009) 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.

C.12.1 Subsurface Drainage

> Subsurface drainage must be addressed in design. The primary component of an MSE wall is soil. Water has a profound effect on this primary component, as it can both decrease the soil shear strength (i.e., resistance) and increase destabilizing forces (i.e., load). Thus, FHWA recommends drainage features be required in all walls unless the engineer determines such feature is, or features are, not required for a specific project or structure.
>
> Drainage design and detailing is addressed in *Berg, et al. – Vol. I (2009)*. Note that MSE walls using free draining reinforced fill do not typically need a full drainage system, but do need a method for discharging water collected within the reinforced wall fill *(see Figure C-23)*. Also note that MSE walls can be designed for water loads, if needed. Basic soil mechanics principles should be used to determine the effect or phreatic surface on wall loads. *See Berg, et al. – Vol. I (2009) for a discussion of the required* design of MSE walls for flood and scour events.

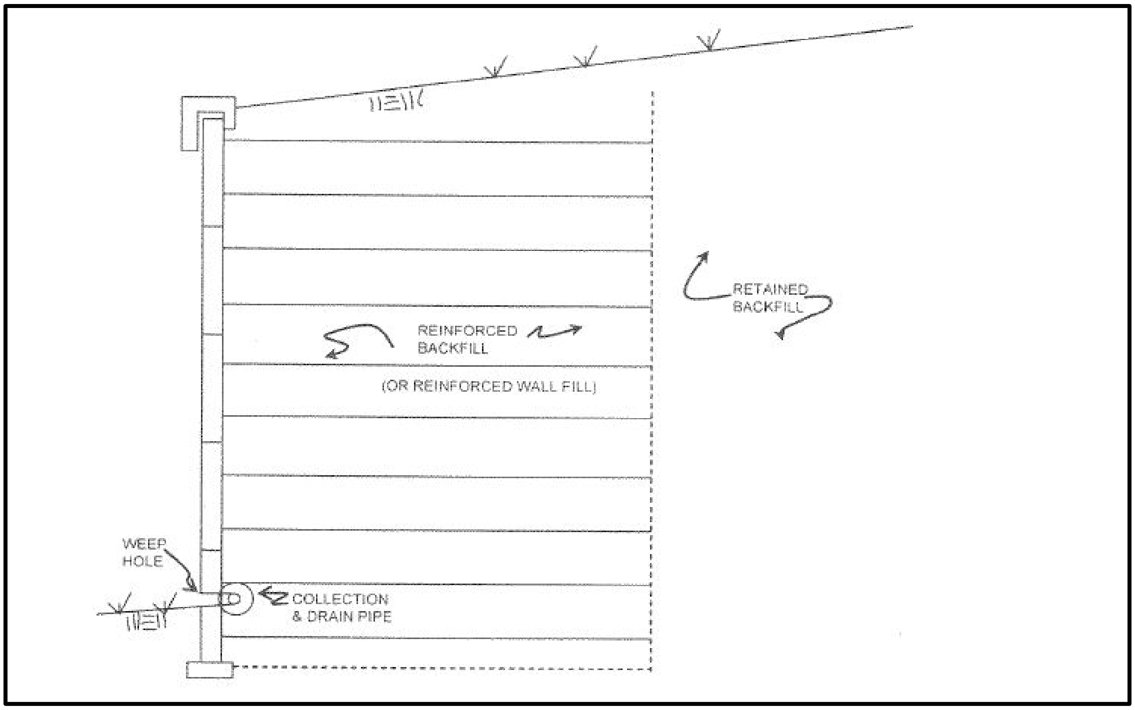


Figure C-23, Drain Immediately Behind MSE Wall Face (Tanyu, et al. (2008))

C.12.2 Surface Water Runoff

Surface drainage is an important aspect of ensuring wall performance and must be addressed during design and during construction. Appropriate drainage measures to prevent surface water from *excessively* infiltrating into the wall fill should be included in the design of a MSE wall structure. Surface drainage design and detailing are addressed in *Berg, et al. – Vol. I (2009)*.

C.12.3 Scour

There are additional detailing considerations for walls that are exposed to potential scour. The wall embedment depth must be below the predicted *or estimated* scour depth. Wall initiation and termination detailing should consider and should be designed to protect from scour *that may be caused by surface water runoff*. Riprap may be used to protect the base and ends of a wall. A coarse stone wall fill may be desired to drain rapidly. The reinforced wall fill at the bottom of the structure may be wrapped with a geotextile filter to minimize loss of fill should scour exceed design predictions. These items are discussed in detail by *Berg, et al. – Vol. I (2009), and should be included on the plans*.

C.12.4 Inundation Design

MSE Walls may be designed for inundation, with permission from the OES/SDS, OES/GDS, OES/HDS, from water that has been determined to be non-aggressive (see Chapter 7 for determination of aggressive versus non-aggressive). Inundation is defined as the process of water entering into the reinforced backfill materials of an MSE Wall, typically from a water level in front of the wall. To prevent the buildup of hydrostatic pressure behind the MSE Wall facing, the reinforced backfill materials in the inundation zone shall consist of stone backfill (see STS SC-M-713 – *Mechanically Stabilized Earth (MSE) Walls*) except Macadam, which is not permitted in the inundation zone. The stone backfill shall extend to 1 foot above the 100-year flood level for non-aggressive water. Granular backfill may be used above the stone backfill; however, a geotextile soil separator is required between the granular backfill and the stone backfill. In addition, the stone backfill shall be encapsulated with a geotextile soil separator to prevent soil from migrating into the stone backfill under certain hydraulic conditions. The use of either metallic or geosynthetic reinforcement is permitted for MSE Walls designed for inundation by non-aggressive water.

The top of the leveling pad shall be placed below the maximum scour depth but no less than 3 feet below the bottom of the stream bed. The excavated area in front of the MSE Wall shall be backfilled with Rip Rap. The Rip Rap shall extend at least 3 feet from the front of the wall toward the centerline of the stream and shall extend at least 3 feet above the leveling pad. The size of the Rip Rap shall be determined by the GEOR in consultation with the HEOR. The Rip Rap shall conform to the requirements of the Standard Specifications. This design approach applies only to those sites where the maximum velocity of the water is less than or equal to 5 feet per second. If the velocity is greater than 5 feet per second, the GEOR in consultation with the HEOR shall provide a recommended design.

The inundation of MSE Walls by water that has been determined to be aggressive is allowed only if the conditions that follow are met. Place MSE Walls 5 feet above the 100-year flood level, in areas where the water has been determined to be aggressive. For these MSE Walls, the use of metallic reinforcement is not allowed. Therefore, the MSE Wall shall use geosynthetic reinforcement within the reinforced backfill. In addition, the geosynthetic reinforcement shall extend the full height and length of the wall. Mixing geosynthetic reinforcement and metallic reinforcement is not allowed either vertically or horizontally. No metallic connectors are allowed within the backfill that may be exposed to water that has been determined to be aggressive. In addition, the reinforced backfill shall be encapsulated with a geotextile soil separator to prevent the retained soil from migrating into the reinforced backfill under certain hydraulic conditions. Further the GEOR should consider the effect of the Extreme Event II hydraulic condition (i.e., the check (500-year) flood) on the reinforced backfill.

If inundation of the MSE Wall is anticipated, the GEOR shall indicate on the MSE Wall drawings whether the water will be non-aggressive or aggressive. The MSE Wall supplier shall be responsible for accounting for the effects of the aggressiveness of the water in the design of the MSE Wall panel. The MSE Wall supplier shall be required to provide a statement and design indicating that the panel was designed for an aggressive environment.

C.13 UTILITIES

No utilities shall be placed within the reinforced zone (see Figure C-1) without written permission of the OES/GDS. If permitted, no utility shall be placed lower than the top layer of reinforcement. All utilities that conduct power shall be sufficiently insulated to prevent stray current from affecting metallic reinforcement. In addition, provide to SCDOT a stamped drawing prepared by a South Carolina licensed engineer providing the details of the power conduit installation and insulation. No force mains (water or wastewater) shall be permitted within the reinforced zone unless a secondary containment system is also provided, including a method to relieve pressure buildup in the secondary containment should the primary utility fail. The exception to this policy is the placement of storm water utilities that are required to permit drainage of the roadway surface. All storm water utilities shall be placed to avoid interference with the reinforcement except where details provided by SCDOT are applied. In addition, all storm water utilities should be designed to inhibit or prevent leaks. Please note that the use of reinforced concrete pipe (RCP) is required along all Interstate (including bridges that cross the Interstate) and SCDOT Evacuation Routes. Regardless of whether the storm water utility is located within the reinforced zone or below the bottom of the wall footprint, the RCP shall meet the requirements of SC-M-714 – *Permanent Pipe Culverts*. A rubber gasket joint material meeting the requirements of ASTM C443 – *Standard Specification for Joints for Concrete Pile and Manholes, Using Rubber Gaskets* including a 13 psi pressure test. For MSE Walls located along non-interstate routes all pipe culvert types are allowed, see SC-M-714. All pipes whether located in the reinforced backfill or below the bottom of the wall footprint shall conform to requirements of SC-M-714. All joint materials shall be appropriate for the pipe culvert being installed. In addition, all joint materials shall include a 13 psi pressure test. Backfill all utility trenches located below the bottom of the reinforced backfill limits with flowable fill regardless of pipe culvert type. The flowable fill shall meet the requirements of SC-M-210 – *Flowable Fill*.

C.14 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 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 should conform to Chapter 10.

C.15 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 including staging 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. Provide an electronic file 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.
12042 SE Sunnyside Road, Suite 711
Clackamas, OR 97015 USA
Tel. (971) 224-4187
adama@geoprograms.com

C.16 PLANS

This Section details the information that should be placed on construction drawings related to MSE walls. The GEOR should review the template drawings available on the SCDOT website at:

https://www.scdot.org/business/geotech.aspx

Select "713 Series – Mechanically Stabilized Earth Walls" in the drop down menu. The requirements for plans for MSE Walls are contained in Chapter 22.

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