

CHAPTER 5 — DESIGN OF MSE WALL COMPONENT

Current design practice for MSE walls does not incorporate the long-term retaining effects of a shoring wall on the design of the MSE wall component.^(1,2) Retaining benefits provided by the shoring wall include reduction of lateral loads on the MSE component and contribution to global stability. This chapter presents design methodology for the MSE wall component of an SMSE wall system.

Where the design method differs from current MSE wall design practice and is an original contribution of this report, boxes are placed around equations and text is in italics; methodology drawn from Elias et al. are cited with the superscript (2).

The most significant contribution of a shoring wall system constructed in conjunction with an MSE wall is its effect on global and external stability. If adequate space is available at the project, the MSE wall component of an SMSE wall system may be designed using standard design procedures with a minimum aspect ratio of 0.7.^(1,2) However, where a shoring system is designed to remain permanently, external and global stability of the composite wall system should take both wall systems into consideration. Design of the MSE wall component of an SMSE wall system should consider:

- Internal stability of the reinforced soil mass with regard to rupture and pullout of reinforcing elements.
- External stability along the MSE wall/shoring wall interface.
- Bearing capacity and settlement of the MSE wall foundation materials.
- Global stability of the composite SMSE wall system.

In contrast to design of a traditional MSE wall, the resistance to sliding and overturning are not evaluated as these are not critical for SMSE wall systems. In addition, a different method is recommended for design of the MSE reinforcements to resist pullout for SMSE wall systems.

When an SMSE wall system is selected as the preferred alternative, the design process is iterative between defining the geometric constraints of the structure and analysis of stability. A number of geometric factors for design of the SMSE wall system should be considered, including reinforcement lengths, toe embedment, and interface geometry. These factors are interrelated, and have a combined effect on the stability of the structure. The optimum combination will provide the most economical construction while meeting the necessary stability criteria.

5.1 POTENTIAL FAILURE MODES

Stability analysis of an SMSE wall system must consider failure modes associated with conventional MSE walls and shoring walls, plus internal failure modes specific to the compound nature of the SMSE wall system. Figure 13 illustrates the various failure modes of the composite SMSE wall system.

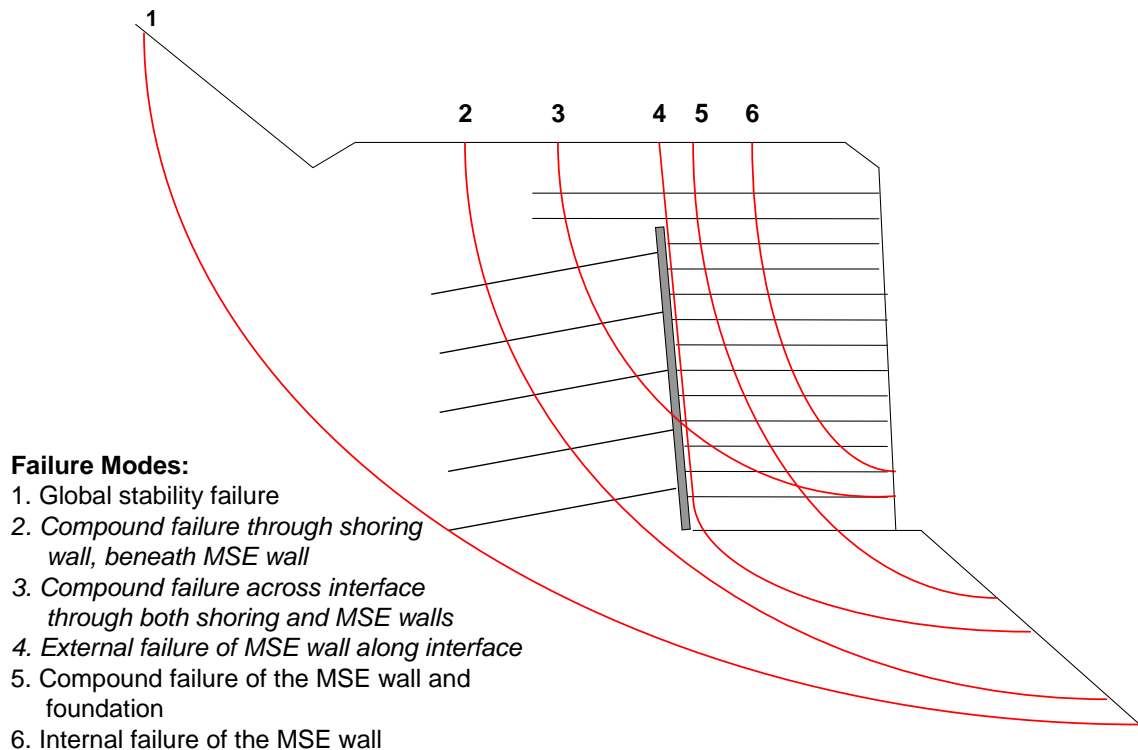


Figure 13. Diagram. SMSE wall system failure modes.

5.1.1 Global Failure

Global failure occurs when a failure surface passes behind and underneath all elements of the compound structure (figure 13, mode 1). In analyzing this type of failure, the SMSE wall system is assumed to act as an intact, impenetrable unit, forcing the shear surface completely outside of the structure. Stability against this type of failure depends on conditions of the foundation and slope below and behind the structure. It is virtually independent of the SMSE wall system strength and structural characteristics, but rather on the geometry of the structure envelope and hillside. In most instances, evaluation must also be made of seismic effects on global stability. Discussion of the global stability evaluation for SMSE wall systems is provided in section 5.5, presented later in this chapter.

5.1.2 Compound Failure of Shoring System and Foundation

Compound failure of the shoring system and foundation occurs when the shear surface intersects the shoring wall, and continues through the foundation below the MSE wall (figure 13, mode 2).

This type of failure may be analyzed using limit equilibrium software which includes elements to model the various wall components, such as Slide 5.0.⁽¹⁵⁾ Analysis programs specific to soil nail wall design such as GoldNail and Snailz do not typically model failure surfaces that exit beneath the shoring wall.^(68,69) Instead, these programs typically model failure surfaces through the toe of the shoring wall, and they do not model the surcharge that is applied by the MSE wall.

5.1.3 Failure Across Interface

Specific to the composite SMSE wall system is a failure across the interface (figure 13, mode 3). In this failure mode, the shear failure surface intersects both the MSE wall and the shoring system. The failure surface may or may not intersect the foundation.

Normally, a structurally sound permanent shoring wall face will effectively preclude such failures. A soil nail shoring wall with a reinforced permanent shotcrete face would be unlikely to experience shear through the shotcrete, if it is properly designed and constructed. In many cases, this failure mode may be excluded from analysis at the discretion of the engineer. However, this may not be the case for some shoring types, such as a tie-back shoring wall with discrete facing panels. Where this failure mode needs to be checked, it can be checked using limit equilibrium software such as Slide 5.0.⁽¹⁵⁾

5.1.4 Interface Shear Failure

Interface shear failure is defined as a failure occurring along the interface between the MSE wall and the shoring system (figure 13, mode 4). This failure mode includes connection failure when connections of the MSE reinforcements to the shoring wall are applied. This failure may extend through the foundation, or it may result in differential deformations between the two wall systems. Similar to a failure across the interface, this failure mode is specific to an SMSE wall system. This failure mode may be checked using limit equilibrium methods.

5.1.5 Compound Failure of MSE Wall and Foundation

Compound failure of the MSE wall and foundation occurs when the shear surface intersects both the MSE wall and the foundation (figure 13, mode 5). This type of failure is representative of the bearing capacity of the foundation materials, and is analyzed using limit equilibrium software such as that discussed above in section 5.1.2.

5.1.6 Internal Failure of the MSE Wall

Internal failure of the MSE component (figure 13, mode 6) is addressed with appropriate backfill materials, suitable vertical spacing of reinforcement, and adequate reinforcement strength and lengths. Evaluation of the internal failure mode of the MSE wall component of an SMSE wall system was one of the primary focuses of the research presented in this report, and design of the MSE wall component to resist internal failure is presented later in this chapter.

5.2 FACTORS OF SAFETY

The recommended minimum factors of safety (FS) for design of the SMSE wall system were modified where appropriate from AASHTO, and are provided below:

- Global stability, FS_g : 1.3 to 1.5.
- Compound stability, FS_c : 1.3.
- Bearing capacity, FS_{bc} : 2.5.
- Seismic stability, FS_{sei} : 75 percent of static FS_g .
- Internal shear capacity, FS_{sc} : 1.5.
- *Interface shear stability (evaluated along the MSE/shoring wall interface), FS_{is} : 1.5.*
- Rupture of reinforcements, FS_r : 1.5.
- *Pullout of reinforcements, FS_p : 1.5 to 2.0 (range of FS to allow the engineer to account for potential reduction in vertical stress in the resistant zone due to arching at the shoring wall/foundation interface, per section 3.2).*
- Connection strength, FS_{cs} : 1.5.

Factors of safety with regard to sliding, overturning, and eccentricity are not provided, as these failure modes are not considered valid for SMSE walls. As discussed in chapter 3, lateral pressures acting on the MSE component are self-induced because the shoring wall effectively reduces external loading, and these self-induced pressures would not realistically induce these modes of failure in walls designed in accordance with the guidelines. Analyses for sliding, overturning, and eccentricity modes of failure, though conducted for traditional MSE walls are not required for SMSE wall design. Refer to AASHTO for a broader discussion of global failure mechanisms acting outside the SMSE wall system.⁽¹⁾

5.3 INTERNAL STABILITY DESIGN

Internal stability design of the MSE component of an SMSE wall system should address the following potential internal failure mechanisms:

- Soil reinforcement rupture (elongation or breakage of the reinforcements).
- Soil reinforcement pullout.

The step-by-step process for internal design of the MSE component is summarized as follows:

- Select the reinforcement type (inextensible or extensible reinforcements) and trial geometry for the MSE wall.
- Estimate the location of the critical failure surface.
- Calculate the maximum tensile force at each reinforcement level for evaluation of reinforcement rupture.
- Calculate the required total tensile capacity of reinforcements in the resistant zone.
- Calculate the pullout capacity at each reinforcement level within the resistant zone with regard to pullout.

Step 1 – Select MSE wall type and trial wall geometry.

The MSE wall system type, including facing, must be selected to complete design. The calculations for internal stability of the MSE component differ somewhat for extensible (geogrid or geotextile) and inextensible (steel) reinforcements, as discussed later in this chapter.

Select vertical reinforcement spacing consistent with the type of MSE facing intended for the application. Keeping in mind that closer reinforcement spacing increases internal stability, as evidenced in centrifuge modeling (appendix B), do not use vertical reinforcement spacing greater than 600 mm. For ease of construction, consider constant vertical reinforcement spacing.

Step 2 – Estimate the location of the critical failure surface.

The critical failure surface can be approximated using Rankine’s active earth pressure theory within the reinforced soil mass, assuming the remaining portion lies along the shoring/MSE interface. Use of the theoretical active failure surface is consistent with current practice for design of MSE walls with extensible reinforcements, and is considered sufficiently conservative for design of SMSE wall systems, based on observations in appendix B. Figure 14 illustrates the conceptualized failure surface for extensible reinforcements. Design for inextensible reinforcements should be conducted using the failure surface illustrated in figure 15, consistent with current design practice.⁽²⁾

As shown in both the extensible and inextensible reinforcement cases, the critical failure surface has been assumed to be bilinear with the lower point passing through the toe of the wall. This assumption is conservative compared to observations from centrifuge modeling (appendix B).

Design for internal stability should conservatively neglect the additional retaining benefits provided by longer upper reinforcement layers (refer to figure 6), and for that reason they are excluded from figures 14 and 15.

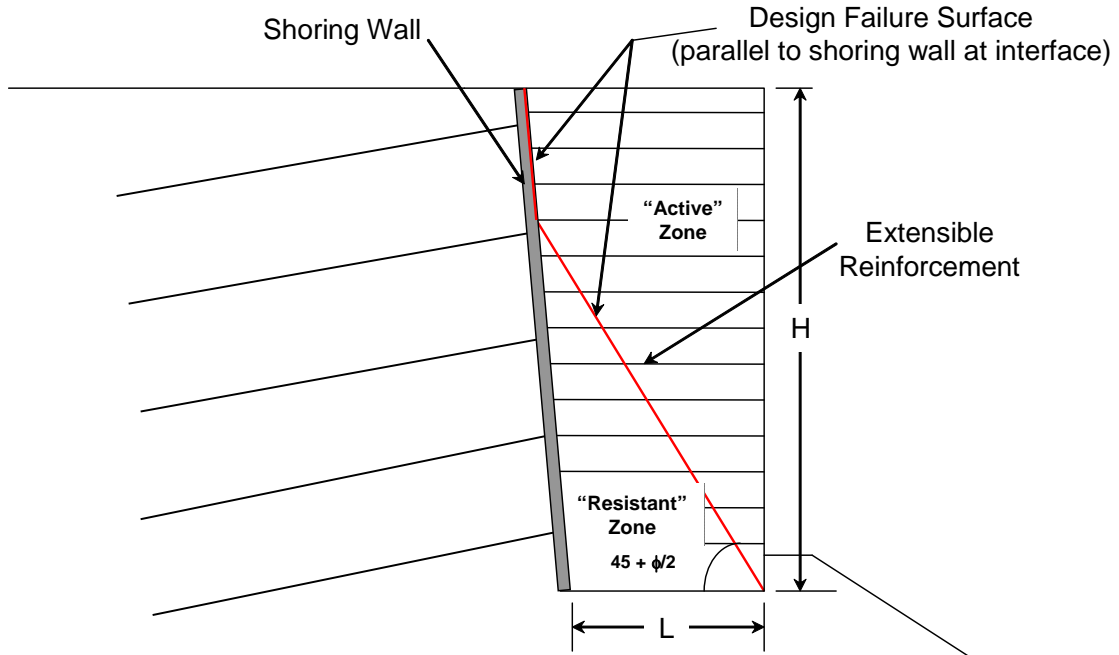


Figure 14. Diagram. Location of potential failure surface for internal stability design of MSE wall component with extensible reinforcements.

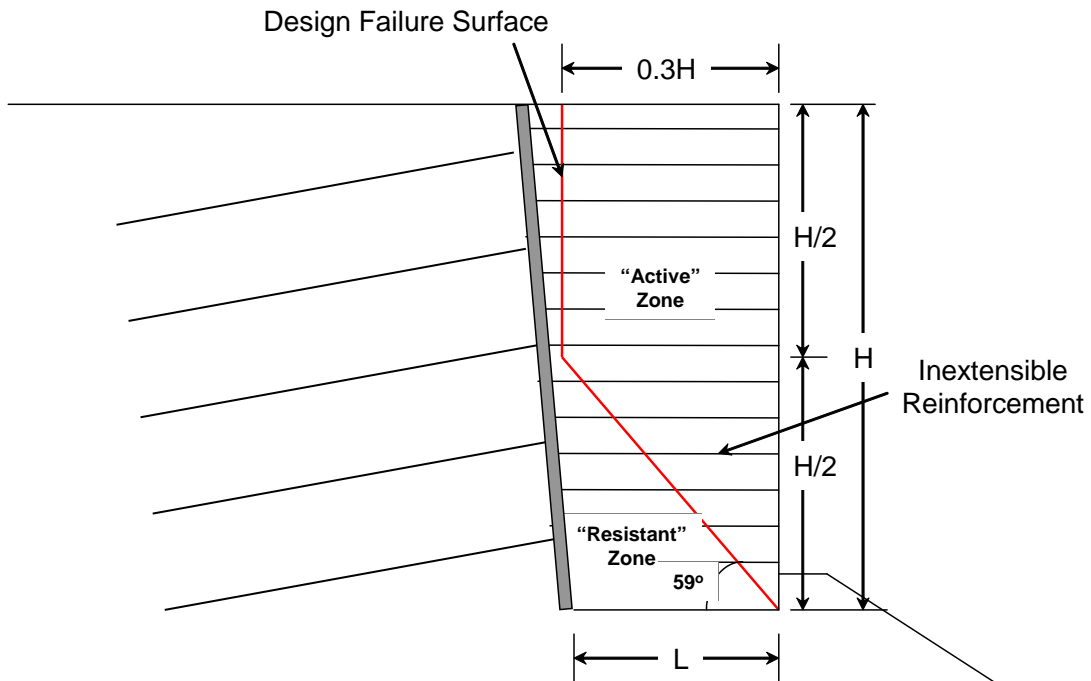


Figure 15. Diagram. Location of potential failure surface for internal stability design of MSE wall component with inextensible reinforcements.

Step 3 – Calculate the internal stability with respect to rupture of the reinforcements.

For SMSE walls, lateral pressures are essentially the result of reaction of the reinforced soil mass against the shoring wall, and are thus internal to the MSE mass. Centrifuge modeling and field-scale testing support this concept. Field-scale test lateral pressure measurements indicate earth pressures along the interface well below theoretical active earth pressure (appendix C). Strain in the reinforcement was also small, even at high surcharge, suggesting relatively low lateral pressures. Lateral earth pressure well below active is also backcalculated from centrifuge testing.

Consistent with current MSE practice, internal design of the MSE wall component requires calculation of lateral stresses, which are dependent on reinforcement type (inextensible versus extensible). The procedure outlined for reinforcement rupture is identical to that presented in Elias et al. for traditional MSE walls.⁽²⁾

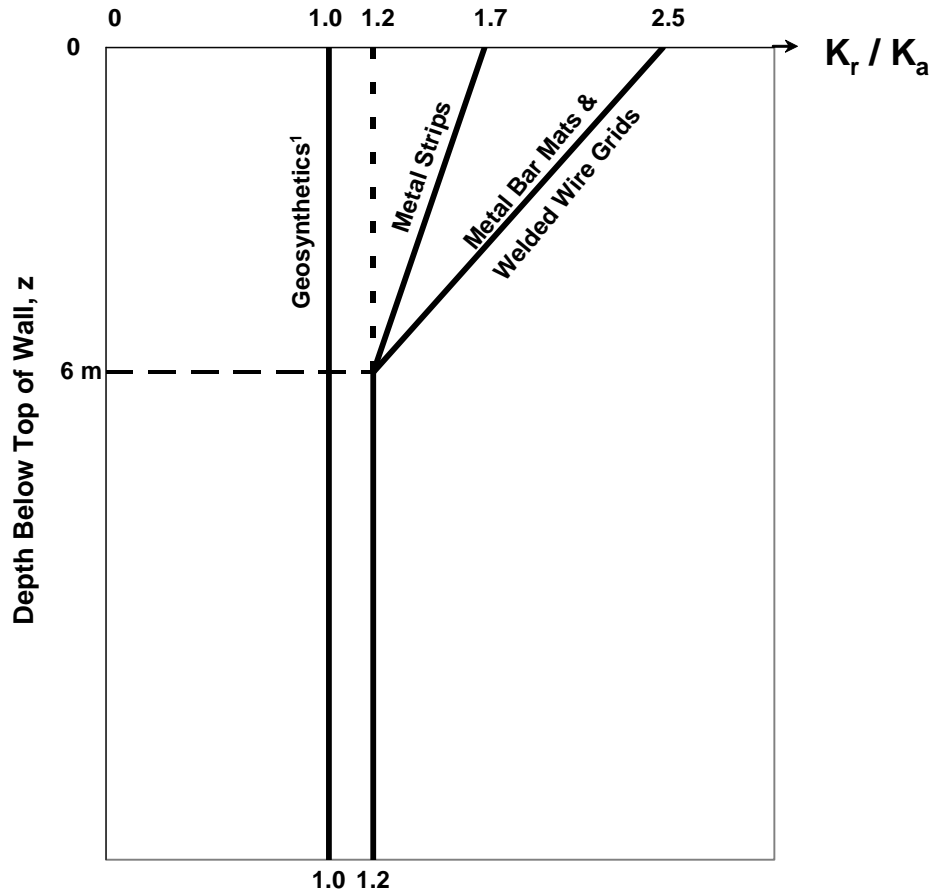
For internal design of the MSE component with extensible reinforcements, active earth pressures are conservatively assumed to apply and the maximum tensile forces acting on each reinforcement layer are calculated using the simplified coherent gravity method. Use of active earth pressure is considered conservative for design of the MSE wall component. Based on observation and testing, coupled with experience, it may be appropriate to re-evaluate this subject in the future and to design based on lower earth pressure.

The relationship between reinforcement type and lateral stress is illustrated in figure 16 for evaluation of the lateral stress ratio, K_r/K_a , where K_r is a lateral earth pressure coefficient appropriate to the reinforcement design, and K_a is the active earth pressure coefficient. As demonstrated in figure 16, the resulting K_r/K_a ratio for inextensible reinforcements decreases from the top of the wall to a constant value of 1.2 below 6 m, while the K_r/K_a ratio for extensible reinforcements is taken as one, regardless of the depth below the top of the wall. The ratios presented in figure 16 assume that the vertical stress is equal to the weight of the overburden (γH), providing a simplified evaluation method for use with cohesionless reinforced fill.

From figure 16, K_r is calculated by applying a multiplier to the active earth pressure coefficient. The active earth pressure coefficient is calculated using the Coulomb earth pressure relationship, assuming that wall friction is zero and that the slope above the wall is horizontal or flat. For a vertical wall, the active earth pressure coefficient reduces to the Rankine equation:

$$K_a = \tan^2 \left(45 - \frac{\phi'}{2} \right) \quad \text{Equation 1.}$$

where ϕ' is the effective friction angle of the reinforced fill.



Notes:

1. Does not include polymer strip reinforcement.

Figure 16. Chart. Variation in lateral stress ratio coefficients with depth in an MSE wall.⁽¹⁾

For wall face batters greater than 8 degrees from vertical, use the following simplified form of the Coulomb equation:

$$K_a = \frac{\sin^2(\theta + \phi')}{\sin^3 \theta \left[1 + \frac{\sin \phi'}{\sin \theta} \right]^2} \quad \text{Equation 2.}$$

where θ is the inclination of the MSE wall facing as defined in figure 17.⁽²⁾

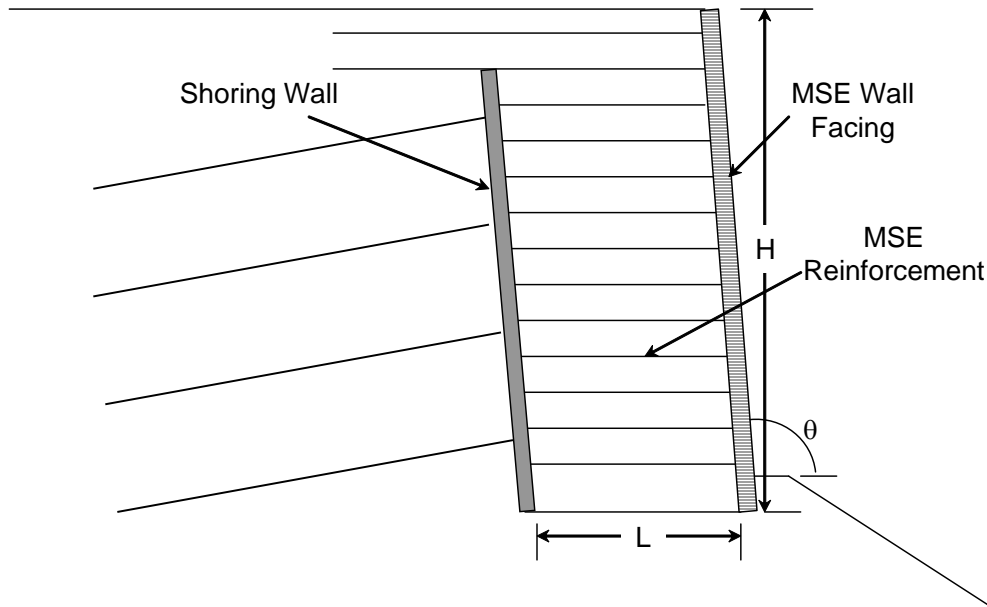


Figure 17. Diagram. Battered MSE wall facing.

At each reinforcement level, calculate the horizontal stresses, σ_h , along the potential failure line from the weight of the reinforced fill (γz), plus uniform surcharge loads (q), and concentrated surcharge loads ($\Delta\sigma_v$ and $\Delta\sigma_h$):

$$\sigma_h = K_r \sigma_v + \Delta\sigma_h \quad \text{Equation 3.}$$

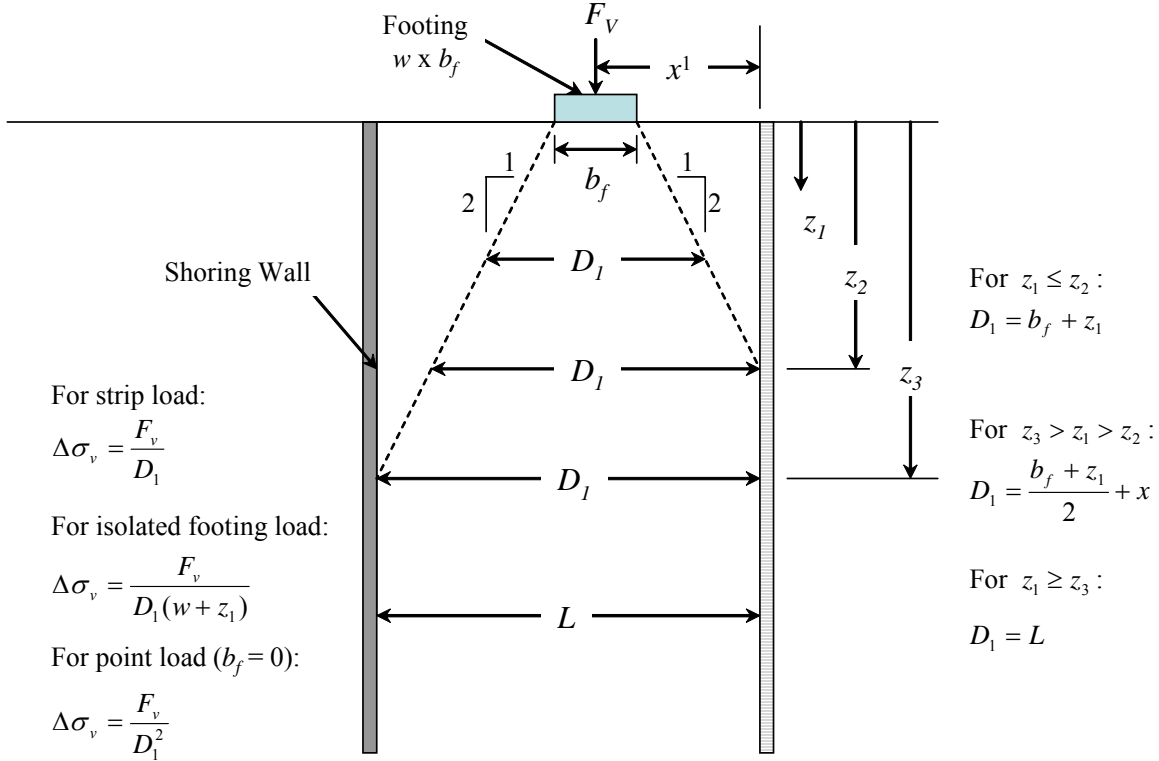
where the vertical stress, σ_v , is calculated:

$$\sigma_v = (\gamma \cdot z) + q + \Delta\sigma_v \quad \text{Equation 4.}$$

and K_r is a function of depth (z) below the top of the wall as shown in figure 16. Use of K_r (greater than or equal to active earth pressure) for computing horizontal stress is considered conservative.

The increment of vertical stress ($\Delta\sigma_v$) due to concentrated vertical loads may be calculated at each reinforcement level using a modified version of the 2:1 method illustrated in figure 18. Figure 18 illustrates a simplified case where the shoring and MSE walls are constructed without a batter, and where the load footing does not straddle the shoring wall. When wall batters are employed, as recommended by these guidelines, the vertical stresses can be estimated by geometrically calculating D_1 at each reinforcement depth. In the case where the footing straddles the shoring wall, D_1 is always greater than z_2 , as defined in the figure. Based on results of field-scale testing (appendix C), the 2:1 method appears to predict the vertical stresses adequately for relatively low footing loads, as would be expected in practice. For high loading situations, the wall designer is cautioned that this method may not produce conservative vertical stresses, as discussed in appendix C.

The increment of horizontal stress ($\Delta\sigma_h$) due to concentrated horizontal loads may be calculated as demonstrated in Elias et al.⁽²⁾ Concentrated horizontal loads result from lateral earth pressures and traffic surcharges acting on footings constructed above the MSE wall component, and lateral forces due to superstructure or other concentrated lateral loads.



Note:

¹ The measurement of x may be from either the face of the MSE wall or the shoring wall, depending on the location of the load footing and the slopes of the various walls.

Figure 18. Diagram. Distribution of stress from concentrated vertical load F_V for internal and external stability calculations.

In each reinforcement layer, calculate the maximum tension per unit width of wall (T_i) based on the vertical reinforcement spacing, s_v :

$$T_i = \sigma_h \cdot s_v \quad \text{Equation 5.}$$

For discrete reinforcements (i.e., metal strips, bar mats, etc.), T_i is calculated by dividing the result of equation 5 by the reinforcement coverage ratio (R_c). R_c is defined by:

$$R_c = \frac{b}{s_h} \quad \text{Equation 6.}$$

where b is the gross width of the strip, sheet or grid and s_h is the center-to-center horizontal spacing between strips, sheets or grids. R_c is equal to one for full coverage of reinforcement.

Next, calculate the internal stability with respect to rupture of the MSE reinforcements. Stability with respect to rupture of each layer requires that:

$$T_{allowable} \geq T_i \quad \text{Equation 7.}$$

where $T_{allowable}$ is the allowable tension force per unit width of the reinforcement. For guidance on determining the long-term allowable strength of the reinforcement, see Elias et al.⁽²⁾

Step 4 – Calculate the required total tensile capacity of MSE reinforcements.

Internal design differs from design of a conventional MSE wall with regard to pullout of the reinforcements. Conventional MSE design requires that each layer of reinforcement resist pullout by extending beyond the estimated failure surface.⁽²⁾ In the case of an SMSE wall system, only the lower reinforcement layers (i.e., those that extend into the resistant zone) are designed to resist pullout for the entire “active” MSE mass.

The required pullout resistance (T_{max}) of the MSE reinforcements within the resistant zone is calculated as the pullout force derived using the simplified free-body diagram presented in figure 19. Figure 19 represents the typical case where the MSE wall component has horizontal backfill and is subjected to a traffic surcharge, q (force per unit length units).

Regardless of whether or not the shoring wall is battered, the engineer should assume development of a tension crack at the MSE/shoring wall interface and that the upper wedge (shown in gray in figure 19) is in equilibrium. As such, the forces N_2 and S_2 shown in figure 19 are ignored. Further, this analysis conservatively excludes extension of the upper layers of reinforcement. For simplicity, the remaining parameters should be assumed the same as represented in figure 19. Concentrated vertical and horizontal loads (F_V and F_H , respectively) are assumed to apply at the centroid of the truncated active failure wedge.

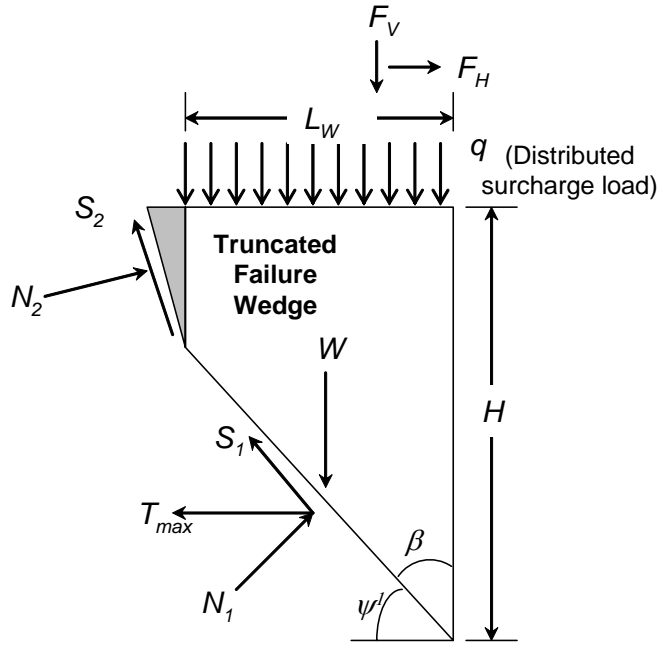
The weight of the upper wedge is insignificant and may also be ignored in the pullout calculation. Therefore, the weight of the active wedge, W , can be calculated as:

$$W = L_w H \gamma - \frac{1}{2} L_w^2 \tan \psi \quad \text{Equation 8.}$$

where H is the height of the MSE wall, γ is the unit weight of the reinforced fill, ψ is the angle defined in figure 19, and L_w is the maximum length of the truncated failure wedge, i.e., reinforced length at the intersection of the shoring wall and active wedge. Assuming that the MSE wall facing is near-vertical, L_w may be estimated as follows:

$$L_w = \frac{L_B v}{v - \tan \psi} \quad \text{Equation 9.}$$

where L_B is width of the MSE wall at the base and v is the vertical component of the shoring wall batter, i.e., $1H:vV$.



Notes:

1. For extensible reinforcements, $\psi = 45 + \phi/2$; for inextensible reinforcements, $\psi = 59$ degrees.
2. Assume tension crack development and neglect forces N_2 and S_2 .
3. Assume upper wedge (shown as gray) is in equilibrium.

Figure 19. Diagram. Free-body diagram for calculation of required tensile capacity in the resistant zone.

Summing force components perpendicular and parallel to the failure surface gives:

$$N_1 = W \sin \beta + qL_W \sin \beta + F_V \sin \beta + T_{max} \cos \beta - F_H \cos \beta \quad \text{Equation 10.}$$

and

$$S_1 = W \cos \beta + qL_W \cos \beta + F_V \cos \beta - T_{max} \sin \beta + F_H \sin \beta \quad \text{Equation 11.}$$

where W is the weight of the active wedge defined in equation 8, N_1 represents the reaction force perpendicular to the failure surface, S_1 represents the shear resistance along the failure surface, T_{max} is the resultant pullout force mobilized by the reinforcement in the resistant zone, and β is the angle as defined in figure 19. At failure, the Mohr-Coulomb failure state defined by:

$$\frac{S_1}{N_1} = \tan \phi'$$

Equation 12.

is applied to the reinforced soil, where ϕ' is the effective friction angle for the reinforced soil. When L_w is less than $H \tan \beta$, the weight of the truncated “active” wedge is given by:

$$W = L_w H \gamma - \frac{\gamma L_w^2}{2 \tan \beta}$$

Equation 13.

where γ is the unit weight of the reinforced soil and H is the height of the MSE wall. Substitution of the expressions for S_1 , N_1 , and W and simplification for T_{max} leads to:

$$T_{max} = \frac{L_w \left(\gamma \left(H - \frac{L_w}{2 \tan \beta} \right) + q \right) + F_v}{\tan(\phi' + \beta)} + F_H \text{ for } L_w \leq H \tan \beta \quad \text{Equation 14.}$$

For the case where $L_w = 0.3H$, the equation may be written:

$$T_{max} = \frac{3H \left(\gamma \left(H - \frac{3H}{20 \tan \beta} \right) + q \right) + F_v}{10 \tan(\phi' + \beta)} + F_H \text{ for } L_w = 0.3H \quad \text{Equation 15.}$$

For the case where $L_w \geq H \tan \beta$, the full active wedge would develop and the forces S_2 and N_2 are zero. In this case, the weight of the “active” wedge is:

$$W = \frac{1}{2} \gamma H^2 \tan \beta$$

Equation 16.

and the expression for T_{max} is:

$$T_{max} = \frac{H \tan \beta (\gamma H + 2q) + 2F_v}{2 \tan(\phi' + \beta)} \text{ for } L_w \geq H \tan \beta \quad \text{Equation 17.}$$

For the above situation, select granular fill having a friction angle of 35 degrees or greater provides an MSE wall section with a minimum aspect ratio of 0.7. Therefore, either the design method presented herein or conventional MSE wall design methods may be employed for internal design of the MSE wall.^(1,2)

Step 5 – Calculate the pullout resistance of MSE reinforcements in the resistant zone.

Calculation of the pullout resistance generally follows traditional design methods.^(1,2) *The primary difference between calculation of pullout resistance for conventional MSE walls and SMSE walls is in the factor of safety. The factor of safety against pullout, FS_p , should be increased from 1.5 to 2.0 for aspect ratios of 0.4 or less due to the potential for arching to develop, as discussed in section 3.2.* Based on the reinforcement spacing(s) selected, calculate the length of embedment (L_{ei}) of each reinforcement layer within the resistant zone:

$$L_{ei} = L - \frac{H - z}{\tan \psi} \quad \text{Equation 18.}$$

where L is the length of the MSE reinforcement at the corresponding reinforcement level, z is the depth to the reinforcement layer from the top of the wall, and ψ is the angle defined in figure 19. When wall batters are employed, geometric evaluation of L_{ei} is required.

At each reinforcement layer within the resistant zone, calculate the pullout resistance, F_{PO} :

$$F_{PO} = \frac{1}{FS_p} F^* \sigma'_{vi} L_{ei} C R_c \alpha \leq T_{allowable} \quad \text{Equation 19.}$$

- where:
- FS_p = Factor of safety against pullout (range of FS to allow engineer to account for potential reduction in vertical stress in the resistant zone due to arching at the shoring wall/foundation interface, per section 3.2).
 - F^* = Pullout resistance factor, discussed later in this section.
 - C = Reinforcement effective unit perimeter (i.e., 2 for strips, grids, and sheets).
 - α = Scale effect correction factor to account for a nonlinear stress reduction over the embedded length of highly extensible reinforcements, based on laboratory data, generally 1.0 for inextensible reinforcements and 0.6 to 1.0 for geosynthetic reinforcements. In the absence of test data, use 0.8 for geogrids and 0.6 for geotextiles.
 - R_c = Coverage ratio (i.e., unity for full coverage).
 - σ'_{vi} = Effective overburden pressure (γz) at the i th reinforcement level, including distributed dead load surcharges, but neglecting traffic live loading.
 - L_{ei} = Length of embedment in the resisting zone at the i th reinforcement level.

The calculated pullout resistance, F_{PO} , is less than or equal to the allowable strength ($T_{allowable}$) of the specified MSE reinforcement.

The pullout resistance factor (F^*) can be estimated using the following general equation, or from laboratory pullout tests:

$$F^* = (F_q \cdot \alpha_\beta) + \tan \rho \quad \text{Equation 20.}$$

where F_q is an embedment bearing capacity factor, α_β is a bearing factor for passive resistance based on the thickness per unit width of the bearing member, and ρ is the soil-reinforcement interaction friction angle. Refer to Elias et al. and AASHTO for more information regarding evaluation of F^* .^(1,2) In the absence of laboratory test data, F^* is commonly estimated as $(2/3)\tan \phi'$ for geotextile reinforcement in granular soil, and $0.8 \tan \phi'$ for geogrid reinforcement in granular soil.⁽¹⁶⁾

For steel ribbed reinforcement, F^* is commonly taken as:

$$F^* = \tan \rho = 1.2 + \log C_u \text{ or } 2.0 \quad \text{Equation 21.}$$

whichever is lesser, at the top of the structure, and

$$F^* = \tan \phi' \quad \text{Equation 22.}$$

for depths greater than or equal to 6 meters. In these equations, C_u is the uniformity coefficient of the backfill (D_{60}/D_{10}). If the specific C_u for the wall backfill is unknown, a C_u of four should be assumed for backfills meeting the requirements of this design guideline (section 3.3.1).

For steel grid reinforcements with transverse spacing (S_t) greater than or equal to 150 mm ($S_t \geq 150$ mm), F^* can be calculated:

$$F^* = F_q \alpha_\beta = 40 \alpha_\beta = 40 \left(\frac{t}{2S_t} \right) = 20 \left(\frac{t}{S_t} \right) \quad \text{Equation 23.}$$

at the top of the structure, and

$$F^* = F_q \alpha_\beta = 20 \alpha_\beta = 20 \left(\frac{t}{2S_t} \right) = 10 \left(\frac{t}{S_t} \right) \quad \text{Equation 24.}$$

for depths greater than or equal to 6 meters. In these equations, t is the thickness of the transverse bar of the grid reinforcement. The transverse spacing should be uniform throughout the length of the reinforcement, and not just concentrated within the resistant zone.

The pullout resistance of the MSE wall component of an SMSE wall system is considered adequate if:

$$T_{\max} \leq \sum F_{PO} \quad \text{Equation 25.}$$

where T_{\max} is calculated as presented in step 4.

5.4 EXTERNAL STABILITY DESIGN

External stability design of the MSE wall component should address bearing capacity and settlement of the foundation materials. Overturning and sliding are not included as failure mechanisms due to stabilization provided by the shoring wall. Hydrostatic forces are eliminated by incorporating internal drainage into the design.

5.4.1 Bearing Capacity

The MSE wall component should be designed for stability against bearing capacity failure. Two modes of bearing capacity failure exist: general shear and local shear failure.

General Shear

To prevent general shear bearing capacity failure, the vertical stress (σ_v) at the base of the wall should not exceed the allowable bearing capacity of the foundation soils:

$$\sigma_v \leq q_a = \frac{q_{ult}}{FS_{bc}} \quad \text{Equation 26.}$$

The vertical stress at the base of the wall is calculated for the MSE wall component of a shored MSE wall using figure 20 for loading from the weight of the reinforced wall and surcharge pressures. Where applicable, the influence of concentrated vertical loading ($\Delta\sigma_v$) should be added, calculated as illustrated in figure 18. The calculation illustrated by figure 20 includes transfer of vertical stress to the shoring wall, where battered. However, it conservatively neglects arching effects near the shoring wall at the base. Based on the field-scale testing, this simplified method of calculating the vertical stress should be conservative for bearing capacity analysis.

The vertical stress, σ_v , acting at the base of the MSE wall component for the case presented in figure 20 with horizontal backfill and traffic surcharge is given by:

$$\sigma_v = \frac{W_1 + (q \cdot L_B)}{L_B} \quad \text{Equation 27.}$$

For relatively thick facing elements (e.g., segmental concrete facing blocks), the facing dimension and weight may be included in the bearing capacity calculations (i.e., use B as defined in figure 20 instead of L_B).

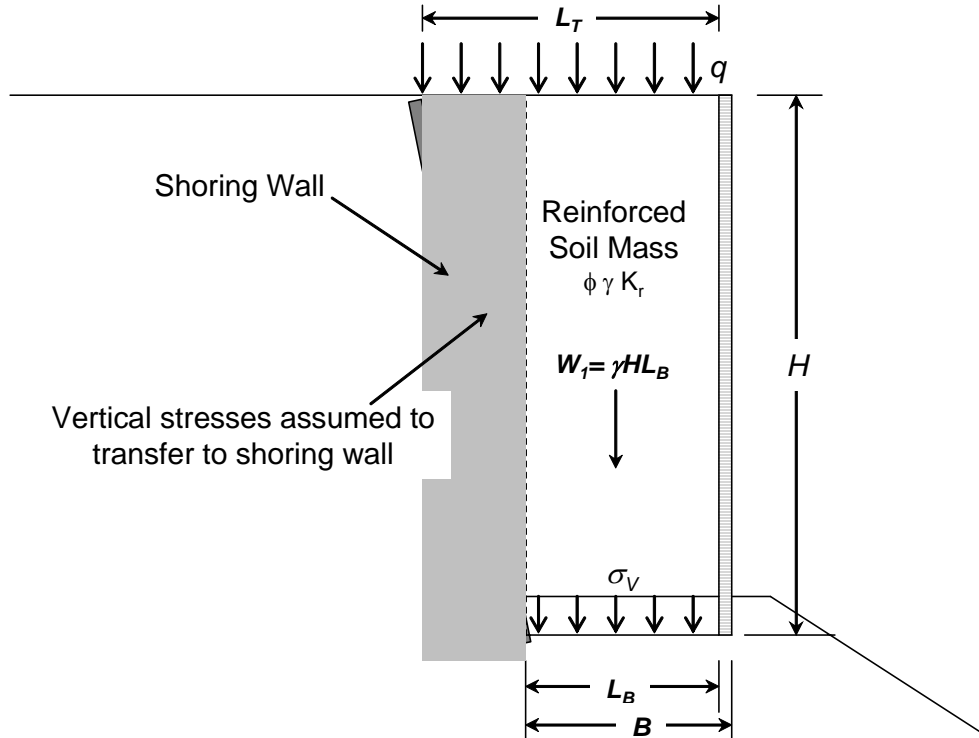


Figure 20. Diagram. Calculation of vertical stress at foundation level.

Calculate the ultimate bearing capacity (q_{ult}) using classical soil mechanics methods:

$$q_{ult} = c_f N_{cq} + 0.5 L_B \gamma_f N_{\gamma q} \quad \text{Equation 28.}$$

where c_f is the cohesion of the foundation soils, γ_f is the unit weight of the foundation soils, and N_{cq} and $N_{\gamma q}$ are dimensionless bearing capacity coefficients. The dimensionless bearing capacity factors can be obtained from figure 4.4.7.1.1.4B of AASHTO for the typical case where the MSE wall component is adjacent to sloping ground.⁽¹⁾ For convenience, this figure has been reproduced in this report as figure 21. Modifications to the equation for q_{ult} for high groundwater level are provided in section 4.4.7.1.1.6 of AASHTO.⁽¹⁾ *No check for eccentricity is recommended, as eccentricity effects are minimal due to the presence of the shoring wall.*

Local Shear

Local shear is characterized by local “squeezing” of the foundation soils when retaining walls are constructed on soft or loose soils (i.e., development of the classic bearing capacity failure surface does not occur). Kimmerling provides guidance for the selection of appropriate reduction factors to the bearing capacity equation when local shear is an issue.⁽¹⁷⁾ Ground improvement of the foundation soils should be incorporated if adequate support conditions are not available.

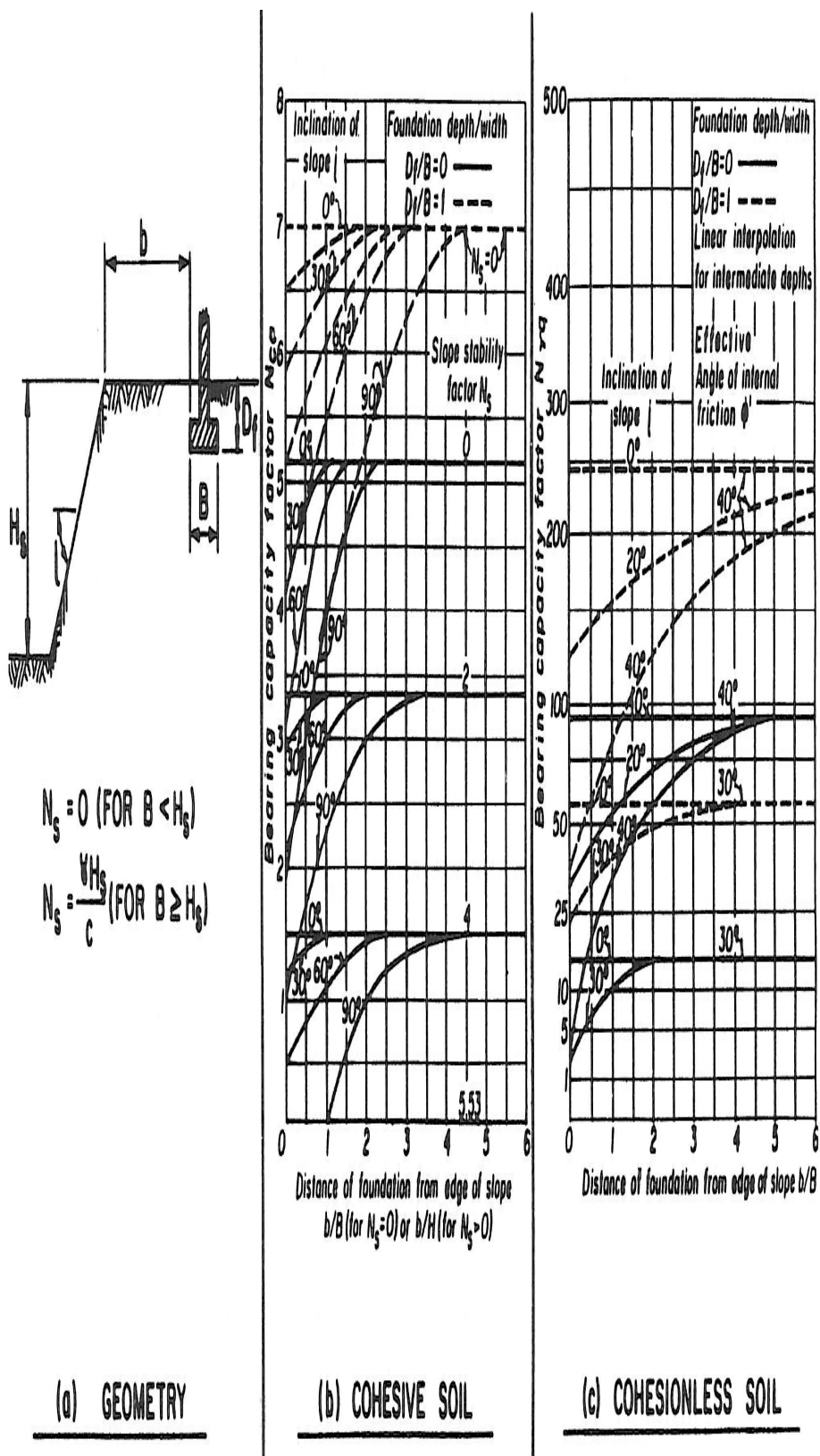


Figure 21. Chart. Modified bearing capacity factors for footing adjacent to sloping ground. ⁽¹⁾

5.4.2 Settlement

Discussion of settlement mechanics and analyses is beyond the scope of this document. For guidance on settlement analysis, see Kimmerling.⁽¹⁷⁾ Unique aspects of SMSE walls in regard to settlement behavior are identified below.

Settlement within the MSE mass itself must be considered. Significant settlement of the MSE mass is not likely to occur where compacted select granular fill is used for the reinforced fill zone. However, a tension crack behind the MSE mass at the top of the wall may result if the reinforced fill zone is constructed of material that does not meet recommended specifications for reinforced fill, as discussed in section 3.3.1.

Settlements external to the MSE mass should be considered. The MSE wall base width, L , may be considerably shorter for the MSE wall component of an SMSE wall system than for a conventional MSE wall. The narrow MSE wall component may be more vulnerable to differential settlement if the foundation is compressible, thus producing an exaggerated outward rotation of the MSE mass and development of a tension crack above the interface. Extended upper reinforcement layers (figure 6) are recommended to mitigate this effect.

Providing nominal facing batters for both the shoring wall interface and MSE wall face are expected to help mitigate differential settlement of the MSE mass. The MSE face should be specified at a practical batter for the type of system and facing contemplated. As discussed in chapter 3, battered construction of the shoring wall at 1H:14V or flatter is recommended.

As discussed in chapter 3, one or more of the following are recommended for SMSE wall design:

- Overlap at least two upper MSE reinforcement layers over the shoring wall section to a minimum length of $0.6H$ or 1.5 m beyond the shoring, whichever is greater. Additional constraints regarding the geometry of the overlapped layers and related guidance are provided in chapter 3.
- Employ mechanical connection between the upper MSE wall reinforcing layers and shoring wall components.
- Use partial shoring construction when only the lower portion of the MSE wall is retained by shoring with longer MSE reinforcements at the upper extent of the wall.

The following additional details may be considered to reduce effects from differential wall settlement behavior:

- Construction of a stepped shoring wall or interface.
- Foundation improvement prior to construction of the MSE component.

5.5 GLOBAL STABILITY DESIGN

As part of the design of the individual MSE wall and shoring components, stability internal to these individual components will have been achieved. However, a global stability evaluation of the SMSE wall system as a compound structure must also be evaluated.

Stability analyses for the SMSE wall system should use conventional limit equilibrium analysis methods. As with any earth stability evaluation, selection of appropriate material parameters is of utmost importance in obtaining a realistic evaluation. In addition, the compound nature of the SMSE wall system requires defining other factors which affect its behavior.

For many applications in steep or mountainous terrain, initial efforts at global stability analysis may produce inadequate factors of safety. A comprehensive treatise on slope stability analyses in these cases is beyond the scope of this report. (Refer to reference 21 for more information.) The following should be considered for projects with difficult global stability issues:

- Confirm the accuracy of soil or rock strength parameters. Consider back-analysis of existing slopes at the site or in the area to check strength parameters.
- If back analysis of empirically stable slopes at the site produces apparent inadequate safety factors, the strength parameters may be overly conservative.
- Consider records or known slope stability experience in the area in conducting back analysis.
- Avoid imposing unrealistic global stability factors of safety on a site with marginally stable slopes. Global stability factors of safety should be consistent with other slope stability factors of safety at the site.

5.5.1 General

Limit equilibrium stability analyses should be conducted to evaluate the stability of the SMSE wall system for the following global failure mechanisms:

- Failure along the shoring/MSE interface.
- Global stability external to the SMSE wall system.

A limit equilibrium computer program such as *Slide*, *UTEXAS*, *SLOPE/W*, *ReSSA*, or others should be used to conduct global stability analyses. (See references 15, 18, 19, and 20.)

Some design codes allow a rigorous approach to modeling stabilizing effects of embedded elements, while others will require the user to apply an effective “cohesion” to model reinforcement effects. The latter should be avoided, except for preliminary designs.

When selecting a method of analysis for a particular wall geometry, it is important to consider the likely shape of the failure surface, and thus whether a circular or noncircular method of analysis is the most appropriate.⁽²¹⁾

Modeling of strength and behavior properties of foundation materials follows conventional practice. Foundations may be composed of soil, rock, or a combination of these, and modeling of the foundation must consider the effects of geologic structure and/or layering conditions. In some cases, special measures may be included to strengthen or improve poor quality foundation conditions. The following sections briefly discuss selection of appropriate foundation parameters. More comprehensive treatment of these topics can be found in Collin et al.⁽²²⁾

Soil Foundations

The character of soil foundations can vary dramatically. As such, the appropriate assumptions for soil behavior are site-specific. Selection of soil parameters for SMSE wall design should be based on the results of a suitably-scoped foundation investigation. The scope of such a study depends on the type and size of the structure and expected variability of foundation conditions. Some general guidelines for geotechnical investigations prior to design of SMSE wall systems are provided in chapter 4.

Soil strength is generally modeled using Mohr-Coulomb criteria, which defines soil strength in terms of the internal friction angle (ϕ) and cohesion (c). These properties are generally derived from laboratory testing of shear strength by using either direct shear or triaxial shear test methods.^(8, 23) In most cases, peak strength is used in the stability analysis. However, in cases where foundation soils have been disturbed, such as by ground movement, a residual friction angle and zero cohesion may be appropriate.

Rock Foundations

Rock foundations can be highly variable in character and the assumptions for strength and stability behavior must be tailored to the specific conditions. Stability analysis of a rock foundation must include consideration of failure along rock discontinuities and within the rock mass itself. Kinematic stability analysis is used to evaluate the potential for failures due to sliding along joints, fractures, or bedding planes. For this type of failure, stability is dependent on the friction along the joint planes and the attitudes of the discontinuities relative to geometry of the foundation slope and structure. Methods such as Markland analysis, using stereographic projection techniques, or commercially available computer programs for block failure analysis are generally used to evaluate these failures.⁽²⁴⁾

Overall rock mass stability can be evaluated using a limit-equilibrium model. In most cases, rock mass strength is characterized by Hoek-Brown criteria, which defines a nonlinear relationship between shear and normal stress.⁽²⁵⁾ Hoek-Brown parameters are generally determined from the information gathered during rock coring and/or detailed rock surface mapping. This type of approach applies to brittle, fractured rock masses. Where the rock mass is weak and more soil-like, a Mohr-Coulomb criterion is generally preferred.

Combined Soil and Rock Foundations

Analysis of a foundation which includes both soil and rock may require a combination of all of the approaches discussed previously. Failure through the foundation may include more than one failure mechanism. For example, in a soil over rock foundation, failure of the foundation could be a combination of a rotational failure in the soil, and kinematic wedge failure in the rock. Most limit equilibrium stability analysis computer programs have the flexibility to model such complex geometries. However, effective use of such models requires a thorough knowledge of the foundation geology and nature of the materials.

5.5.2 MSE Wall/Shoring Interface

A specific characteristic of the SMSE wall system is the creation of an interface between the MSE wall and shoring portions of the composite structure. This interface provides a potential plane of weakness (or failure surface) through the structure. This interface may be constructed in such a way as to increase the resistance to shear along the interface by one of the following methods (discussed in chapter 3):

- Extend upper MSE reinforcements over the top of the shoring wall.
- Construct the shoring wall at a batter or as a stepped structure.
- Provide a mechanical connection between the MSE wall and shoring wall components.

The steps for evaluation of stability along the interface are summarized as follows:

Step 1 – As a first evaluation of interface shear stability, assume zero shear strength along the shoring/MSE interface (i.e., full development of a tension crack). Evaluate stability of the failure mechanism illustrated conceptually in figure 22. If the factor of safety is adequate (i.e., $FS_{is} \geq 1.5$), no further analysis of this failure mechanism is required.

Step 2 – If the factor of safety is not adequate assuming zero shear resistance along the interface, incorporate shear resistance along the interface and re-evaluate. The shear resistance should be estimated as follows:

1. Estimate the interface friction angle, ϕ_i , between the reinforced soil and the shoring wall using table 3.
2. Model the interface with a nominal thickness having an effective friction angle of ϕ_i , (see table 3) and zero cohesion.
3. Conduct limit equilibrium stability analysis forcing failure along the interface, illustrated in figure 23.

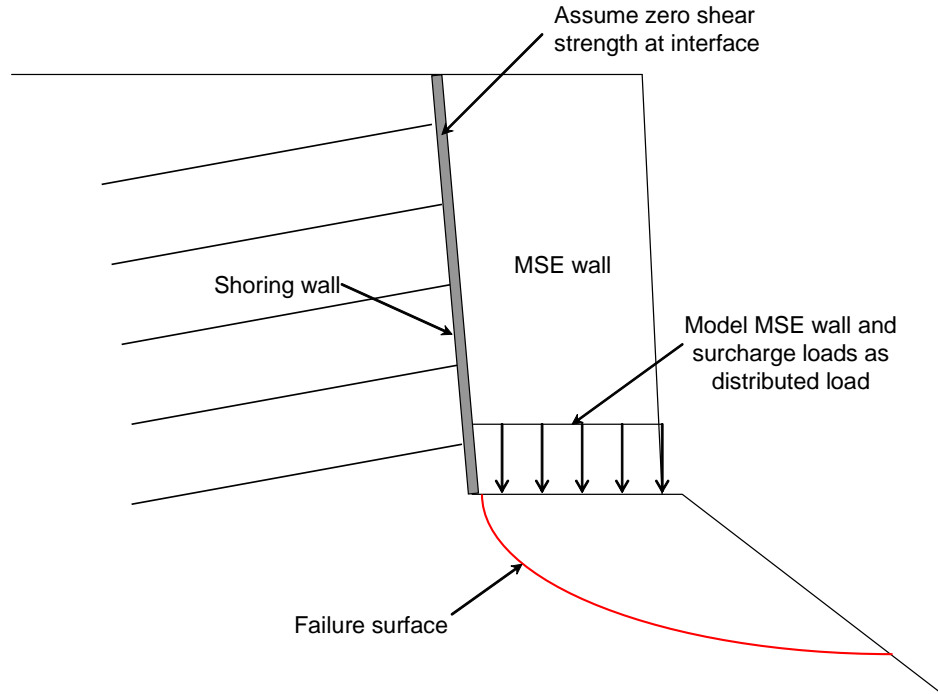


Figure 22. Diagram. Conceptual failure surface and design methodology assuming zero interface shear strength.

Step 3 – If an acceptable factor of safety is still not achieved by considering shear resistance along the interface, then consider the effect of extending the upper reinforcement layers over the shoring wall.

1. Model the MSE wall component, including the geometry of the extended reinforcements. The MSE wall component may be modeled simplistically as a coherent block with cohesion and friction, or rigorously using a sophisticated stability analysis program which enables the use of elements representing the reinforcements
2. Continue modeling the remainder of the interface as indicated in step 2, above.

If adequate stability is not achieved based on this analysis, one or several of the following measures must be taken:

- Employ foundation stabilization measures (e.g., micropiles, stone columns, jet grouting, remove and replace, etc.) to strengthen the foundation.
- Modify SMSE wall system geometry (e.g., wall batter, stepped wall geometry, longer MSE reinforcements, greater setback or embedment, etc.).
- Improve shearing resistance along interface by selecting a different shoring wall surface with higher roughness coefficient or built in irregularities, or using higher strength fill.
- Connect the MSE reinforcement to the shoring system.

Table 3. Interface friction angles.⁽²⁷⁾

Interface Materials	Interface Friction Angle, δ_i
Concrete or masonry against the following foundation materials	
Clean sound rock	35
Clean gravel, gravel-sand mixtures, and coarse sand	29-31
Clean fine-to-medium sand, silty medium-to-coarse sand, and silty or clayey gravel	24-29
Clean fine sand and silty or clayey fine-to-medium sand	19-24
Fine sandy silt and non-plastic silt	17-19
Very stiff clay and hard residual or preconsolidated clay	22-26
Medium-stiff clay, stiff clay and silty clay	17-19
Steel sheet piles against the following soils	
Clean gravel, gravel-sand mixtures, and well-graded rock fill with spall	22
Clean sand, silty sand-gravel mixtures, and single-size hard rock fill	17
Silty sand and gravel or sand mixed with silt or clay	14
Fine sandy silt and non-plastic silt	11
Formed concrete or concrete sheet piles against the following soils	
Clean gravel, gravel-sand mixtures, and well-graded rock fill with spall	22-26
Clean sand, silty-sand-gravel mixtures, and single-size hard rock fill	17-22
Silty sand and gravel or sand mixed with silt or clay	17
Fine sandy silt and non-plastic silt	14
Miscellaneous combinations of structural materials	
Masonry on masonry, igneous, and metamorphic rocks	
Dressed soft rock on dressed soft rock	35
Dressed hard rock on dressed soft rock	33
Dressed hard rock on dressed hard rock	29
Masonry on wood (cross-grain)	26
Steel on steel at sheet-steel interlocks	17

¹ For material not listed, use $\delta_i = 2/3 \phi$

² Angles given are ultimate values that require significant movement before failure occurs.

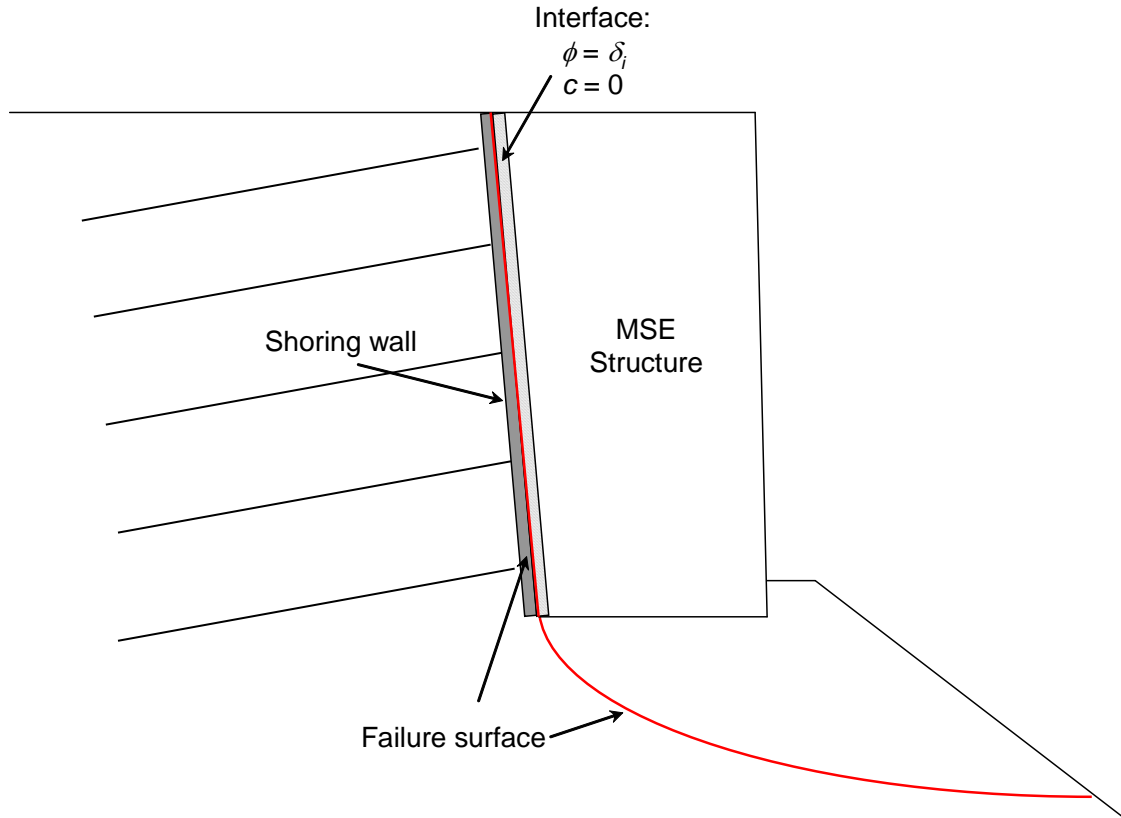


Figure 23. Diagram. Conceptual failure surface to evaluate stability along shoring/MSE interface.

5.5.3 External to SMSE Wall System

Global stability external to the SMSE wall system, illustrated conceptually in figure 24, should be evaluated using limit equilibrium methods. In steep terrain, the factor of safety for the existing slope configuration may be less than the generally accepted range of 1.3 to 1.5, per AASHTO recommendations.⁽¹⁾ Where this is the case, retaining wall system design should not overcompensate to dramatically improve slope stability for the global case, but instead result in a system that provides a nominal increase to the preconstruction factor of safety. However, it should be noted that one advantage of an SMSE wall system is that the shoring component can be designed for slope stabilization prior to construction of the MSE component.

Where the global stability of the SMSE wall system is inadequate, modification of the following design components should be considered:

- Increase level of stabilization provided by shoring wall component (i.e., increase nail length and/or decrease nail spacing for soil nail wall).
- Increase reinforced length of MSE wall component.
- Employ foundation improvement methods.

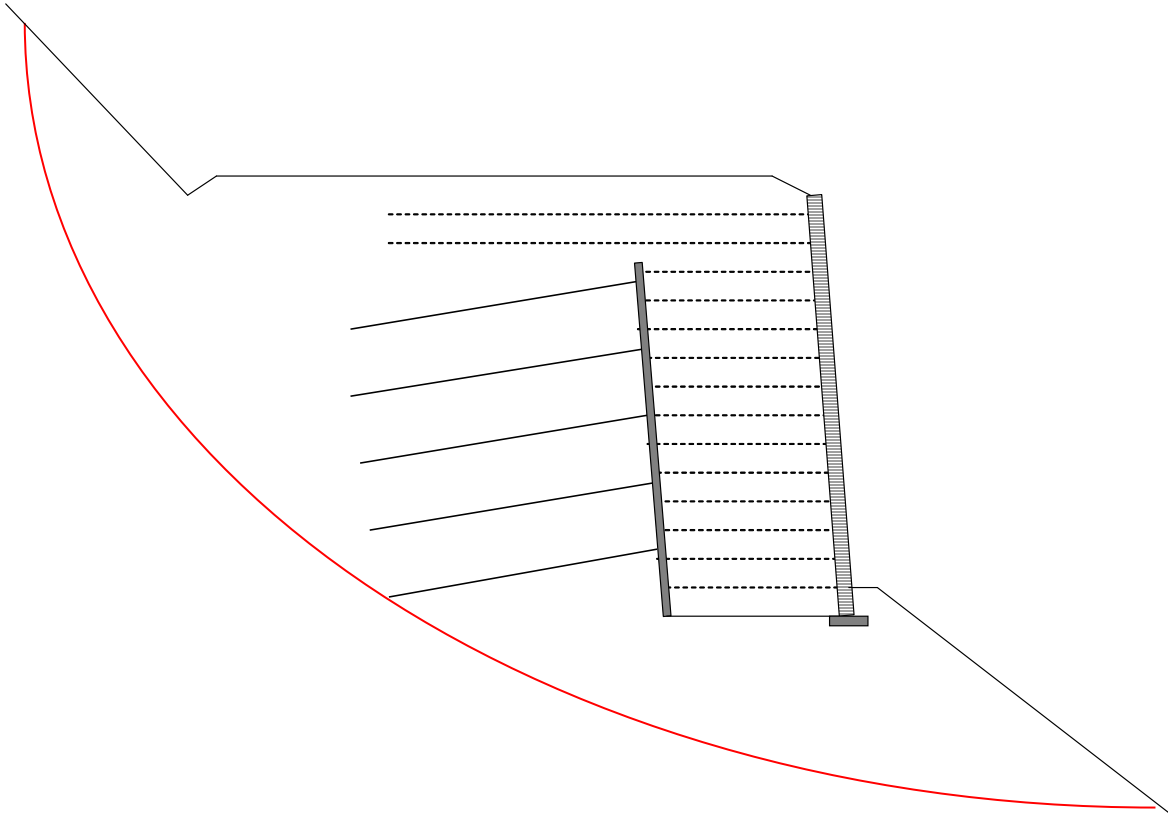


Figure 24. Diagram. Conceptual global stability failure surface.

5.6 SEISMIC STABILITY

Design of both the MSE wall component and the shoring wall component of an SMSE wall system must consider seismic loading. Seismic design of the MSE wall portion should follow the guidelines presented in Elias et al. and AASHTO.^(1,2) A brief discussion is provided in this section.

During an earthquake, the retained fill exerts a dynamic horizontal thrust, P_{AE} , on the SMSE wall system in addition to the static thrust. The reinforced soil mass is subjected to a horizontal inertia force, P_{IR} , equivalent to:

$$P_{IR} = M \cdot A_M \quad \text{Equation 29.}$$

where M is the mass of the active portion of the MSE portion of the SMSE system and A_M is the maximum horizontal acceleration in the reinforced soil wall.

The force, P_{AE} , can be evaluated by the pseudo-static Mononobe-Okabe analysis and added to the static forces acting on the SMSE system.⁽⁶⁰⁾ The dynamic stability with respect to external stability is evaluated considering both the static and dynamic forces. The allowable minimum dynamic factor of safety for external stability is 75 percent of the required static factor of safety.

The internal design of the SMSE system under seismic loads includes an inertial force, P_{IR} , acting horizontally, in addition to the existing static forces. The inertial force will result in increased tensile force on the reinforcement. It is assumed that the location and slope of the maximum tensile force line does not change during seismic loading. Both the tensile strength and pullout capacity of the reinforcement should be evaluated considering both the static and dynamic internal forces. The factor of safety for pullout should be reduced to 75 percent of the static required factor of safety. The allowable strength of the reinforcement is also adjusted to account for the short term duration of the seismic load. See Elias et al. for further guidance.⁽²⁾

5.7 CONNECTION STRENGTH DESIGN

Design of the MSE wall facing connections should be conducted according to AASHTO.⁽¹⁾ Walls designed in accordance with these guidelines will usually not require connection of the MSE mass to the shoring wall. However, where such connections are used, the engineer should consider connection strength, all possible failure modes, corrosion protection, and removal of slack in connection components and MSE reinforcements.

Current practice does not provide the engineer with a rigorous method for calculating the strength of connections between the MSE wall mass and the shoring wall. Research conducted in conjunction with development of these guidelines did not include quantitative strength testing of such connections. However, where connections to the shoring wall are utilized, full strength connections are not likely to be necessary (i.e., connections that develop the full strength of the MSE reinforcing layer). Consideration of failure modes for the wall system that involve connections (such as failure mode 4 from figure 13) may provide the engineer with the connection strength needed.

Corrosion should be considered in the connection design, with protection consistent with similar permanent applications, in accordance with AASHTO.⁽¹⁾ Both design and construction methods should be implemented to minimize slack in the connection.

5.8 MSE WALL BEHAVIOR

When designing an SMSE wall system, consideration should be given to the long-term behavior of each of the individual wall systems. In this section, the behavior of the MSE wall is discussed.

Most lateral displacements occur during construction for MSE walls, and are a function of compaction efforts, vertical reinforcement spacing, reinforcement length, facing connection, facing type, and construction means and methods.⁽²⁾ Anticipated deformations are theoretically greater for MSE walls constructed with extensible (i.e., polymeric) reinforcements than for walls constructed using inextensible (i.e., steel) reinforcements.

With regard to settlement behavior, MSE walls constructed on medium dense to dense granular soils generally exhibit small foundation settlement, occurring mostly during wall construction. Differential settlement is usually only a fraction of the total settlement, but exceptions may occur

where the foundation is highly variable, for example, along the contact between soil and bedrock, or over large boulders. Differential settlement can also occur along steep slopes, where the foundation soils near the face of the wall are significantly less confined than beneath the heel of the wall. Adequate embedment and setback minimize this tendency.

Saturated cohesive soils, however, may exhibit large time-dependent deformations.⁽¹¹⁾ It may be appropriate to wait for most of this settlement to occur prior to placement of final pavement or facing.

If the MSE wall and the shoring system do not employ mechanical connections, or if upper MSE reinforcements do not extend over the shoring wall component, then behavior of the MSE portion of the SMSE wall system will be similar to a conventional MSE wall in many ways. If the foundation is prone to differential settlement, the wall may rotate outward slightly, causing cracking parallel to or along the MSE/shoring interface. The abrupt and relatively rigid interface will tend to focus differential settlement to the area immediately above the shoring, and cracking will likely result. The extended upper reinforcements (to $0.6H$ or longer) will minimize this, as will connections between the reinforcements and the shoring. Connections should be free of slack so that the displacement necessary to mobilize their strength is considerably less than that associated with unacceptable performance at the ground surface or wall face.