CHAPTER 7
DESIGN OF MSE WALLS FOR EXTREME EVENTS

As per AASHTO (2007) an extreme event is one whose recurrence interval can be thought to exceed design life. AASHTO (2007) has two limit states to deal with such events. These limit states are labeled Extreme Event I and Extreme Event II. In the context of MSE walls, the extreme events with the applicable limit state shown in parentheses that require consideration in the design process are as follows:

- Seismic events (Extreme Event I
- Vehicular impact events (Extreme Event II)
- Superflood events and scour (Extreme Event II)

This chapter addresses each of the above extreme events along with a review of the applicable limit state, i.e., Extreme Event I or Extreme Event II.

7.1 SEISMIC EVENTS

Seismic events are analyzed under Extreme Event I limit state as per AASHTO (2007[1]). Seismic events tend to affect both external and internal stability of MSE walls. Guidance for seismic analysis presented in this section is based on Anderson et al. (2008) and Kavazanjian (2009) and represents updated procedures to those in AASHTO (2007).

7.1.1 External Stability

The external stability uses a displacement based approach. The recommended design methodology is presented in the following steps.

Step 1 Establish an initial wall design based on static loading using information in Chapters 4, 5 and 6.

Step 2 Establish the seismic hazard using Article 3.10.2 of AASHTO (2007). Using the 1,000-yr return period seismic hazard maps in AASHTO (2007), estimate the following site-specific values:

- The site peak ground acceleration (PGA), and
- Spectral acceleration at 1-second, S_1

[1] AASHTO 4th Edition 2007 including 2008 and 2009 Interims. 2008 Interims contain significant seismic revisions.

Step 3 For the project under consideration, establish the Site Effects in accordance with Article 3.10.3 of AASHTO (2007). This includes the determination of Site Class as per Article 3.10.3.1 of AASHTO (2007) and Site Factors, F_{pga} and F_v from Tables 3.10.3.2-1 and 3.10.3.2-3, respectively, of AASHTO (2007). The procedure described herein is applicable to Site Classes A, B, C, D and E. For all sites in Site Class F, site-specific geotechnical investigations and dynamic site response analysis should be performed.

Step 4 Determine the maximum accelerations, k_{max}, and peak ground velocity (PGV) as follows:

$$k_{max} = F_{pga}\,(PGA) \tag{7-1}$$

$$PGV\ (in/sec) = 38F_vS_1 \tag{7-2}$$

where F_{pga} and F_v are site factors determined in Step 3 and PGA and S_1 are site peak ground acceleration and spectral acceleration at the 1-second period, respectively, as obtained in Step 2.

Step 5 Using a wall height dependent reduction factor, α, obtain an average peak ground acceleration, k_{av}, within the reinforced soil zone as follows:

$$k_{av} = \alpha k_{max} \tag{7-3}$$

where the value of α is based on the Site Class of the foundation soils as follows:

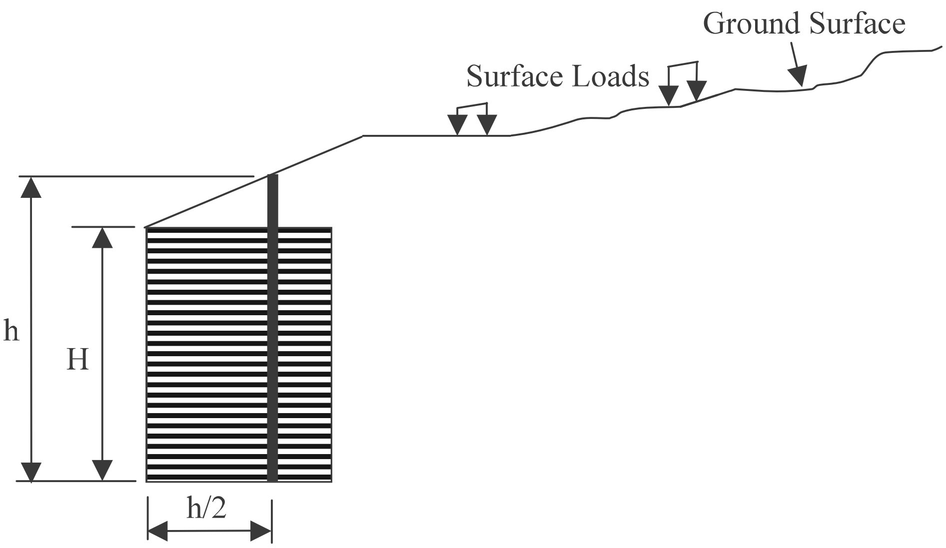
- For Site Class C, D and E (i.e., soils)

$$\boldsymbol{\alpha = 1 + 0.01H\left[0.5\left(\frac{F_vS_1}{k_{max}}\right) - 1\right]} \tag{7-4}$$

  where H is the wall height in feet at the wall face as shown in Figure 7-1.

- For Site Class A and B foundation conditions (i.e., hard and soft rock), the values of α determined by Equation 7-4 should be increased by 20 percent.

For practical purposes, walls less than approximately 20 ft in height and on very firm ground conditions (i.e., Site Class B or C), $k_{av} \approx k_{max}$. For wall heights greater than 100 ft, site-specific geotechnical investigations and dynamic site response analysis should be performed.



$$h = H + \frac{\tan I\,(0.5H)}{(1 - 0.5\tan I)}\ \text{; where I is the backfill slope angle}$$

h/2 is measured from back of wall facing

Figure 7-1. Definition of heights for seismic analyses.

Step 6 Determine the total (static + dynamic) thrust P_{AE} using one of the following two methods:

Method 1: Mononobe-Okabe (M-O) formulation

$$\mathbf{P_{AE} = 0.5(K_{AE})\gamma_b h^2} \tag{7-5}$$

where h is the wall height along the vertical plane within the reinforced soil mass as shown in Figure 7-1, γ_b is the unit weight of the retained fill and K_{AE} is obtained as follows:

$$K_{AE} = \frac{\cos^2(\phi'_b - \xi - 90 + \theta)}{\cos\xi\cos^2(90-\theta)\cos(\delta + 90 - \theta + \xi)\left[1 + \sqrt{\dfrac{\sin(\phi'_b + \delta)\sin(\phi'_b - \xi - I)}{\cos(\delta + 90 - \theta + \xi)\cos(I - 90 + \theta)}}\right]^2} \tag{7-6}$$

where,

$\xi = \tan^{-1}\left(\frac{k_h}{1-k_v}\right)$ with k_h = horizontal seismic coefficient and k_v = vertical seismic coefficient

δ = angle of wall friction = lesser of the angle of friction for the reinforced soil mass (ϕ'_r) and the retained backfill (ϕ'_b)

I = the backfill slope angle = β (see Figure 4-3) {Note: use GLE for broken back slopes, see Comment 2 below}

ϕ'_b = angle of internal friction for retained backfill

θ = the slope angle of the face (see Figure 4-5 in Chapter 4)

To use the Mononobe-Okabe formulation, two seismic coefficient, k_h and k_v, must be defined. It is assumed that these coefficients are applied simultaneously and uniformly to all parts of the structure, i.e., to the reinforced and retained fill. Typically, the vertical seismic coefficient, k_v, is assumed to be zero. The horizontal seismic coefficient, k_h is taken to be equal to k_{max} determined in Step 2.

The total thrust, P_{AE}, calculated as per Equation 7-5 is assumed to act at h/2, i.e., mid-height of the vertical plane of height h shown in Figure 7-1. Therefore, the stress due to thrust P_{AE} is assumed to be distributed uniformly over the height h.

Comments on use of M-O formulation:

1. For backfills that are sloped at 3H:1V or steeper, it may not be possible to obtain a solution for a certain combination of variables in the M-O formulation. This is because the term $\sin(\phi - \xi - I)$ in Equation 7-6 may become negative and represents a limiting condition since at $I = \phi - \xi$ an unstable slope condition occurs (i.e., FS=1 wherein the failure surface coincides with the slope surface. As the limiting condition is approached the earth pressures based on M-O formulation become unrealistically large.

2. M-O formulation is strictly applicable to homogeneous cohesionless soils and may not yield realistic solutions for more complex cases involving (a) soils which derive their shear strength from both cohesion and friction, i.e., c-ϕ soils, (b) non-uniform backslope profiles, and (c) complex surface loadings.

For the cases where M-O formulation leads to unrealistic results, it is recommended that numerical procedures using the same principles of M-O formulation may be used, such as the well-known graphical Culmann method or Coulomb's trial wedge method. However, the more versatile approach for such cases is to utilize the conventional slope stability programs as described in Method 2.

Method 2: Generalized Limit Equilibrium (GLE) slope stability

a. Define the wall geometry, nominal surface loadings (i.e., loadings with load factor = 1.0), groundwater profile, and design soil properties. The plane where the earth pressure needs to be calculated should be modeled as a free boundary. This boundary is a vertical plane located at a distance of h/2 from the back of the wall facing as shown in Figure 7-2.

b. Choose an appropriate slope stability analysis method. Spencer's method generally yields good results because it satisfies the equilibrium of forces and moments.

c. Choose an appropriate sliding surface search scheme, e.g., circular, linear, bi-linear, block, etc.

d. For seismic analysis, use $k_h = k_{av}$ and $k_v = 0$.

e. Apply the earth pressure as a boundary force, P_{AE}, on the face of vertical plane of height h as shown in Figure 7-2. The angle of the applied force with respect to horizontal depends on assumed friction angle between the wall and soil which is lesser of the angle of friction for the reinforced soil mass (ϕ'_r) and the retained backfill (ϕ'_b). Different application points between h/3 and 2h/3 from the base need to be examined to determine the maximum value of P_{AE}. Change the magnitude of the applied load until a capacity:demand ratio (CDR) of 1.0 is obtained i.e., the load and the resistance are balanced. Thus, the force corresponding to a CDR of 1.0 is equal to the total thrust on the retaining structure.

f. Verify design assumptions and material properties by examining the loads on individual slices in the output.

g. Once the maximum value of total thrust, P_{AE}, is determined, apply the force at mid height (h/2) as shown in Figure 7-2 for analysis in following steps.

Step 7. Determine the horizontal inertial force, P_{IR}, of the total reinforced wall mass as follows:

$$P_{IR} = 0.5(k_{av})(W) \tag{7-7}$$

where W is the weight of the full reinforced soil mass and any overlying permanent slopes and/or permanent surcharges within the limits of the reinforced soil mass. The inertial force is assumed to act at the centroid of the mass used to determine the weight W.

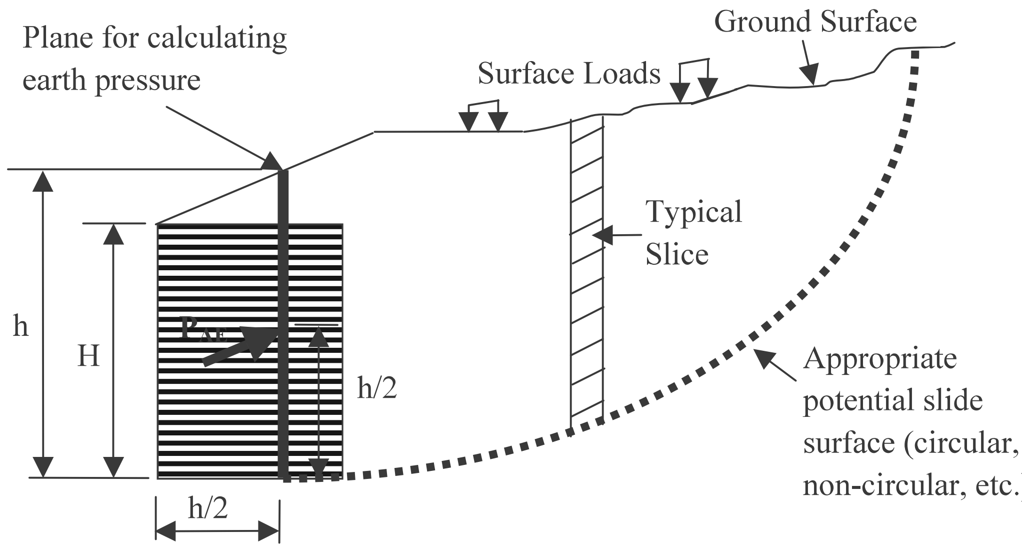


Figure 7-2. Use of a slope stability approach to compute seismic earth pressure.

Step 8 Check the sliding stability using a resistance factor, ϕ_τ, equal to 1.0 and the full, nominal weight of the reinforced zone and any overlying permament sucharges. If the sliding stability is met, the design is satisfactory and go to Step 11. If not, go to Step 9.

Compute the total horizontal force, T_{HF}, is as follows:

- For M-O method:
  T_{HF} = Horizontal component of $P_{AE}(\cos\delta) + P_{IR} + \gamma_{EQ}(q_{LS})K_{AE}H$ + other horizontal nominal forces due to surcharges (with load factor =1.0)

  where, γ_{EQ} is the load factor for live load in Extreme Event I limit state and q_{LL} is the intensity of the live load surcharge.

- For GLE method:
  T_{HF} = Horizontal component of P_{AE} (since all surcharges are included in the slope stability analysis) + P_{IR}

Compute the sliding resistance, R_τ, as follows:

$$R_\tau = \Sigma V\,(\mu)$$

where μ is the minimum of $\tan\phi'_r$, $\tan\phi'_f$ or (for continuous reinforcement) $\tan\rho$ as discussed in Section 4.5.6.a and ΣV is the summation of the vertical forces as follows:

$\Sigma V =$ W + $P_{AE}\sin\delta$ + permanent nominal surcharge loads within the limits of the reinforced soil mass

The sliding stability capacity to demand ratio (CDR) is calculated as follows:

$$CDR_{sliding} = R_\tau / T_{HF}$$

If $CDR_{sliding} > 1$, the design is satisfactory and go to Step 11 otherwise go to Step 9.

Step 9 Determine the wall yield seismic coefficient, k_y, where wall sliding is initiated. This coefficient is obtained by iterative analysis as follows:

a. Determine values of P_{AE} as a function of the seismic coefficient k ($< k_{max}$) as shown in Figure 7-3a.

b. Determine horizontal driving and resisting forces as a function of k (using spreadsheet calculations) and plot as a function of k as shown in Figure 7-3b. The value of k_y corresponds to the point where the two forces are equal, i.e., the CDR against sliding equals 1.0.

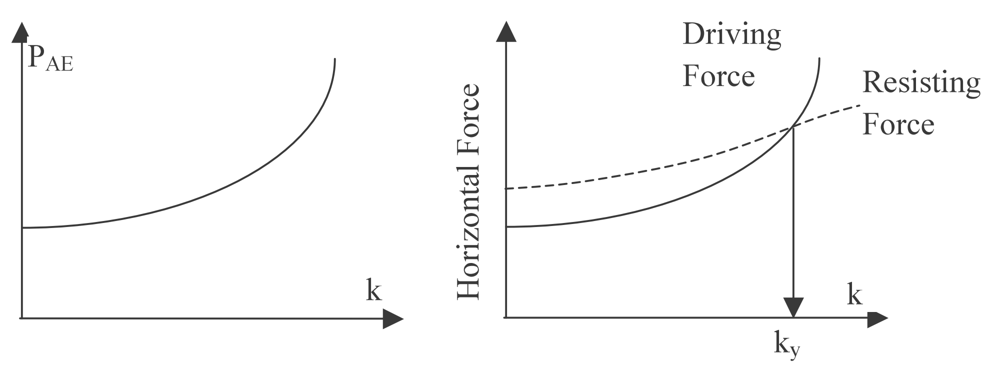


Figure 7-3. Procedure for determination of k_y (Anderson et al., 2008).

Step 10 Determine the wall sliding displacement, d, in inches based on the following relationships between d, k_y/k_{max}, k_{max}, and PGV based on whether the site is located in Western United States (WUS) or Central and Eastern United States (CEUS) as per Figure 7-4:

- <u>For WUS soil and rock sites and CEUS soil sites</u>

$$\log(d) = -1.51 - 0.74\log(k_y/k_{max}) + 3.27\log(1 - k_y/k_{max}) - 0.80\log(k_{max}) + 1.59\log(PGV) \quad (7\text{-}8)$$

- <u>For CEUS rock sites</u>

$$\log(d) = -1.31 - 0.93\log(k_y/k_{max}) + 4.52\log(1 - k_y/k_{max}) - 0.46\log(k_{max}) + 1.12\log(PGV) \quad (7\text{-}9)$$

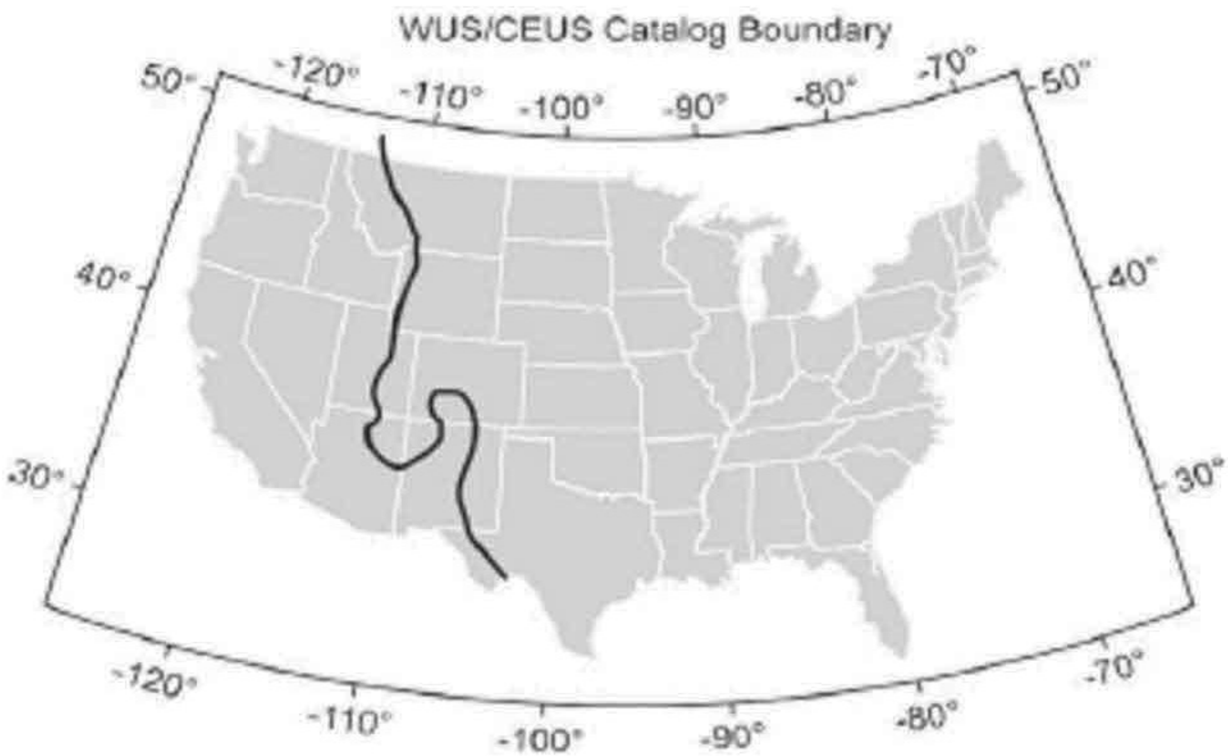


Figure 7-4. Boundary between WUS and CEUS (Anderson et al. 2008).

Step 11 Evaluate the limiting eccentricity and bearing resistance using the same principles discussed in Chapter 4. Include all applicable loads for Extreme Event I. If M-O method is used then add other applicable forces to P_{AE}. If GLE method is used then no additional forces need to be added to P_{AE} since the slope stability analysis includes all applicable forces. Check the limit states using the following criteria:

1. For limiting eccentricity, for foundations on soil and rock, the location of the resultant of the applicable forces should be within the middle two-thirds of the wall base for $\gamma_{EQ} = 0.0$ and within the middle eight-tenths of the wall base for $\gamma_{EQ} = 1.0$. Interpolate linearly between these values as appropriate.

2. For bearing resistance compare the effective uniform bearing pressure to the nominal bearing resistance that is based on the full width of the reinforced zone. A resistance factor of 1.0 is used per Article 10.5.5.3.3 (AASHTO, 2007).

Step 12 If Step 11 criteria are not met, adjust the wall geometry and repeat Steps 6 to 11 as needed.

Step 13 If Step 11 criteria are met, assess acceptability of sliding displacement, d. The amount of displacement which is tolerable will depend on the nature of the wall and what it supports, as well as what is in front of the wall. Typical practice is to limit the lateral displacement in the range of 2.0 in. (50 mm) to 4.0 in (100 mm) assuming that structures on top or at toe of the wall can tolerate such displacements.

7.1.2 Internal Stability

For internal stability, the active wedge is assumed to develop an internal dynamic force, P_i, that is equal to the product of the mass in the active zone and the wall height dependent average seismic coefficient, k_{av}. Thus, P_i is expressed as follows:

$$P_i = k_{av} \, W_a \tag{7-10}$$

where W_a is the soil weight of the active zone as shown by shaded area in Figure 7-5 and k_{av} given by Equation 7-3. The force P_i is assumed to act as shown in Figure 7-5. If the weight of the facing is significant then include it in W_a computation.

The supplementary inertial force, P_i, will lead to dynamic increases in the maximum tensile forces in the reinforcements. Reinforcements should be designed to withstand horizontal forces generated by the internal inertia force, P_i, in addition to the static forces. During the internal stability evaluation, it is assumed that the location and the maximum tensile force lines do not change during seismic loading.

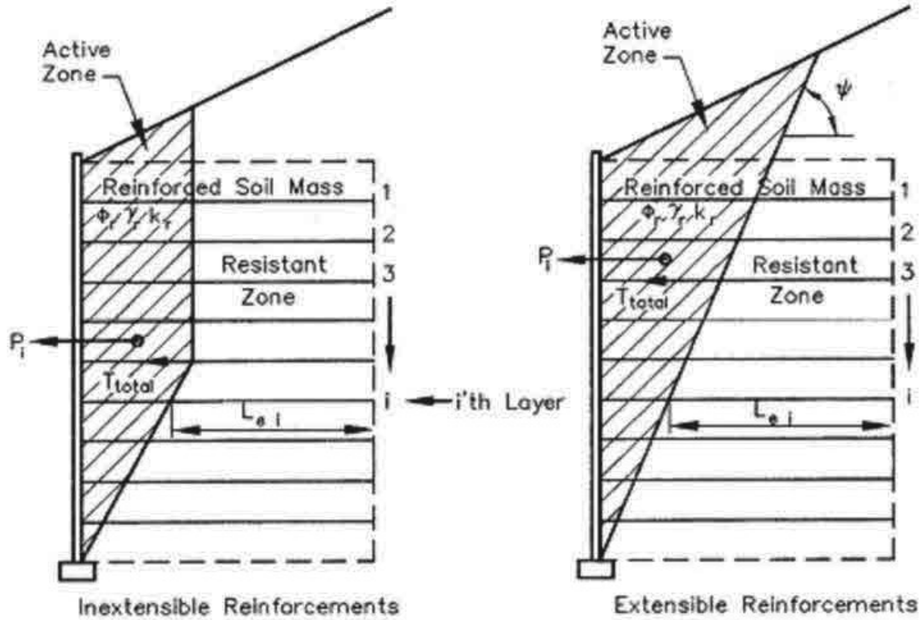


Figure 7-5. Seismic internal stability of a MSE wall.

The inertial force is distributed to the reinforcements equally as follows:

$$T_{md} = \frac{P_i}{n} \tag{7-11}$$

where:

T_{md} = factored incremental dynamic intertia force at layer i

P_i = internal inertia force due to the weight of backfill within the active zone, i.e., the shaded area in Figure 7-5

n = number of soil reinforcement layers within the reinforced soil zone,

The load factor for seismic forces is equal to 1.0. Therefore, the total factored load applied to the reinforcement on a load per unit of wall width basis is determined as follows:

$$T_{total} = T_{max} + T_{md} \tag{7-12}$$

where T_{max} is the factored static load applied to the reinforcements determined using the appropriate equations in Chapters 4 and 6. The reinforcement must be designed to resist the dynamic component of the load at any time during its design life. This includes consideration of both tensile and pullout failures as discussed next.

7.1.2.a Tensile Failure

Design for static loads requires the strength of the reinforcement at the end of the design life to be reduced to account for corrosion for metallic reinforcement, and for creep and other degradation mechanisms for geosynthetic reinforcements. The adjustment for metallic corrosion losses are exactly the same described in Chapter 4 for static analysis. For metallic reinforcements, use the following resistance factors while evaluating tensile failure under combined static and earthquake loading (per Table 11.5.6-1 of AASHTO {2007}):

- Strip reinforcements: 1.00
- Grid reinforcements: 0.85

In contrast, the procedures for geosynthetic do not require a creep reduction for the short duration seismic loading condition and only reductions for geosynthetic degradation losses are required. Strength loss in geosynthetics due to creep requires long-term, sustained loading. The dynamic component of load for seismic design is a transient load and does not cause strength loss due to creep. Therefore, the resistance of the reinforcement to the static component of load, T_{max}, must be handled separately from the dynamic component of load, T_{md}. The strength required to resist T_{max} must include the effects of creep, but the strength required to resist T_{md} should not include the effects of creep. Thus, for geosynthetic reinforcement rupture, the reinforcement is designed to resist the static and dynamic components of the load determined as follows:

<u>For the static component:</u>

$$S_{rs} \geq \frac{T_{max} RF}{\phi R_c} \tag{7-13}$$

<u>For the dynamic component:</u>

$$S_{rt} \geq \frac{T_{md} RF_{ID} RF_D}{\phi R_c} \tag{7-14}$$

where:

ϕ = resistance factor for combined static/earthquake loading = 1.20 from Table 11.5.6-1 of AASHTO (2007)
S_{rs} = ultimate reinforcement tensile resistance required to resist static load component
S_{rt} = ultimate reinforcement tensile resistance required to resist dynamic load component
R_c = reinforcement coverage ratio

RF	=	combined strength reduction factor to account for potential long-term degradation due to installation damage, creep, and chemical aging, equal to RF_{CR} x RF_{ID} x RF_{D} (see Chapter 3)
RF_{ID}	=	strength reduction factor to account for installation damage to reinforcement
RF_{D}	=	strength reduction factor to prevent rupture of reinforcement due to chemical and biological degradation

Using the above equations, the required ultimate tensile resistance of the geosynthetic reinforcement is determined as follows:

$$T_{ult} = S_{rs} + S_{rt} \tag{7-15}$$

7.1.2.b Pullout Failure

For pullout of steel or geosynthetic reinforcement, the following equation is used:

$$L_e \geq \frac{T_{total}}{\phi(0.8F^* \alpha \sigma_v C R_c)} \tag{7-16}$$

where:

L_e	=	length of reinforcement in resisting zone
T_{total}	=	maximum factored reinforcement tension from Equation 7-12
ϕ	=	resistance factor for reinforcement pullout = 1.20 from Table 11.5.6-1 of AASHTO (2007)
F*	=	pullout friction factor
α	=	scale effect correction factor
σ_v	=	unfactored vertical stress at the reinforcement level in the resistant zone
C	=	overall reinforcement surface area geometry factor
R_c	=	reinforcement coverage ratio

For seismic loading conditions, the value of F*, the pullout resistance factor, is reduced to 80 percent of the value used for static design, unless dynamic pullout tests are performed to directly determine the F* value.

7.1.3 Facing Reinforcement Connections

Facing elements are designed to resist the total (static + seismic) factored load, i.e., T_{total}. Facing elements should be designed in accordance with applicable provisions of Sections, 5, 6, and 8 of AASHTO (2007) for reinforced concrete, steel, and timber, respectively.

For segmental concrete block faced walls, the blocks located above the uppermost reinforcement layer should be designed to resist toppling failure during seismic loading.

For geosynthetic connections subjected to seismic loading, the factored long-term connection strength, ϕT_{ac}, must be greater than T_{total} (i.e., $T_{max} + T_{md}$). If the connection strength is partially or fully dependent on friction between the facing blocks and the reinforcement (e.g., MBW facing), the connection strength to resist seismic loads should be reduced to 80 percent of its static value as follows:

<u>For the static component of the load:</u>

$$S_{rs} \geq \frac{T_{max}\, RF_D}{0.8\,\phi\,(CR_{cr})\,R_c} \tag{7-17}$$

<u>For the dynamic component of the load:</u>

$$S_{rt} \geq \frac{T_{md}\, RF_D}{0.8\,\phi(CR_u)\,R_c} \tag{7-18}$$

where:

S_{rs}	=	ultimate reinforcement tensile resistance required to resist static load component
T_{max}	=	applied load to reinforcement
RF_D	=	reduction factor to prevent rupture of reinforcement due to chemical and biological degradation from Chapter 3
ϕ	=	resistance factor = 1.20 applied to both the static and the dynamic components, from Table 11.5.6.4-1 of AASHTO (2007)
CR_{cr}	=	long-term connection strength reduction factor to account for reduced ultimate strength resulting from connection
R_c	=	reinforcement coverage ratio
S_{rt}	=	ultimate reinforcement tensile resistance required to resist dynamic load component
T_{md}	=	factored incremental dynamic inertia force
CR_u	=	short-term reduction factor to account for reduced ultimate strength resulting from connection.

For mechanical connections that do not rely on a frictional component, the 0.8 multiplier is removed from Equations 7-17 and 7-18.

The required ultimate tensile resistance of the geosynthetic reinforcement at the connection is:

$$T_{ult\text{-}conn} = S_{rs} + S_{rt} \tag{7-19}$$

The connection capacity of a facing/reinforcement connection system that is fully dependent on the shear resisting devices for the connection capacity will not be significantly influenced by the normal stress between facing blocks. The percentage of connection load carried by the shear resisting devices relative to the frictional resistance to meet the specification requirements should be determined based on past successful performance of the connection system.

For cases where seismic analysis is required as per Section 4 of AASHTO, facing connections in MBW unit faced walls should use shear resisting devices between the facing blocks and soil reinforcement such as shear keys and structural pins (i.e., pins manufactured from material meeting the design life of the structure, e.g., steel and HDPE) and should not be fully dependent on frictional resistance between the soil reinforcement and facing blocks.

For steel reinforcement connections, AASHTO (2007) recommends that the resistance factors for combined static and seismic loads as follows:

- Strip reinforcements: 1.00
- Grid reinforcements: 0.85

7.2 VEHICULAR IMPACT EVENTS

Traffic railing impact loads are analyzed under Extreme Event II limit state as per Article A13.2 (AASHTO, 2007). Traffic railing impact events tend to affect only the internal stability of MSE walls. Guidance for traffic barrier analysis presented in this section is based on NCHRP 22-20 (Bligh et al., 2009), which is an extension of the previous FHWA (Elias et al., 2001) method based on laboratory and full-scale field tests. Guidance for post and beam railings is based upon AASHTO (2007).

7.2.1 Traffic Barriers

The impact traffic load on barriers constructed over the front face of MSE walls, must be designed to resist the overturning moment by their own mass per Article 11.10.10.2 (AASHTO, 2007).

Static Impact Load

The recommended static impact force is 10,000 lb (45 kN) applied on a barrier with a minimum height of 32 in. (810 mm) above the roadway. Bligh et al. (2009) found that a 10,000 lbs (45 kN) static impact load is equivalent to a dynamic TL-4 railing test level of 54,000 lb (240 kN), as illustrated in Figure 7-6.

The wall design should ensure that the reinforcement does not rupture or pullout during the impact event. Where the impact barrier moment slab is cast integrally with a concrete pavement, the additional force may be neglected. The recommended static impact forces for rupture and for pullout are based upon the recent NCHRP 22-20 project (Bligh et al., 2009) and past practice.

Load Combination and Load Factors

The load factors and load combination for an Extreme Event II are summarized in Table 4-1. A load factor, γ_{P-EV} = 1.35 is used for the static soil load. The traffic surcharge, modeled as an equivalent soil height of 2 ft, also uses the load factor γ_{P-EV} = 1.35 (and not γ = 0.50), for internal stability analysis. The static equivalent impact loads are multiplied by a load factor, γ_{CT} =1.00.

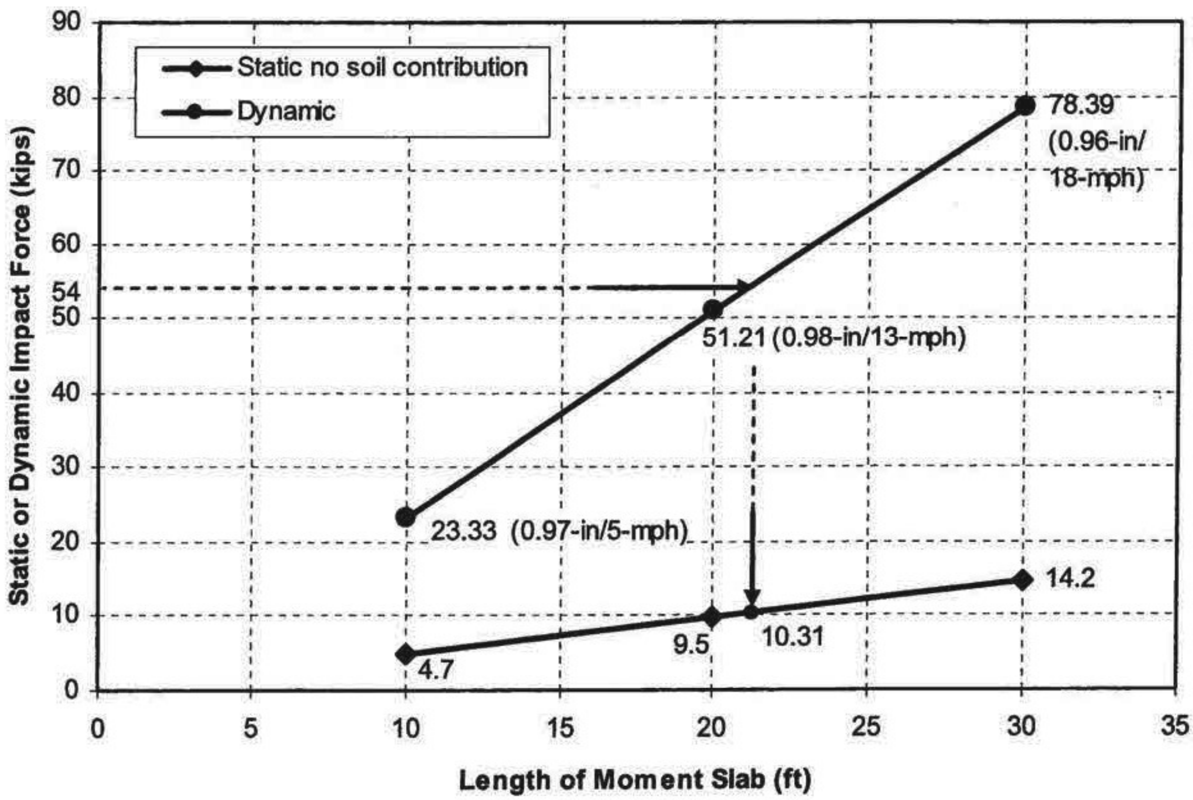


Figure 7-6. Comparison of static and dynamic impact force with 1-inch (25 mm) maximum displacement (Bligh et al., 2009). (1 kip = 4.44 kN; 1 ft = 0.3 m)

Reinforcement Rupture

The static impact force, adds an additional horizontal force to the upper 2 layers of soil reinforcement. It is recommended that the upper layer of soil reinforcement be designed for a rupture impact load equivalent to a static load of 2,300 lb/lft (33.5 kN/m) of wall; and the second layer be designed with a rupture impact load equivalent to a static load of 600 lb/lft (8.8 kN/m). A distribution of stresses, as discussed in Article 11.101.10.2 and illustrated in Figure 3.11.6.3-2 (AASHTO, 2007), is not recommended.

The load factor for impact is equal to 1.0. Therefore, the total factored load applied to the reinforcement on a load per unit of wall width basis is determined as follows:

$$\mathbf{T_{total} = T_{max} + T_I} \qquad (7\text{-}20a)$$

where:

T_I = factored impact load at layer 1 or 2, respectively

T_{MAX} = reinforcement tension from static earth and traffic loads

With terms defined, this equation is:

$$\mathbf{T_{total} = S_V\ K_r\ \gamma_r\ [(Z + h_{eq})\ \gamma_{EV\text{-}MAX}] + t_i\ (\gamma_{CT})} \qquad (7\text{-}20b)$$

where:

t_i = equivalent static load for impact load at layer i, (t_1 = 2,300 lb/lft and t_2 = 600 lb/lft) and other terms as previously defined (Chapter 4 and/or 7).

An example calculation is presented in Appendix E.6. Note that for geosynthetic reinforcements, the nominal strength used to structurally size the reinforcements to resist the impact load is not increased by eliminating the reduction factor for creep, as was done for internal seismic design in Section 7.2.1. This is recommended because full-scale traffic barrier impact testing with geosynthetic soil reinforcement has not been performed to date.

Reinforcement Pullout

The pullout resistance of the soil reinforcement to the impact load is resisted over the full-length of the reinforcements (i.e., L). The traffic surcharge, modeled as an equivalent soil height of 2 ft, is included in the nominal vertical stress, σ_v, for pullout resistance calculation. Pullout is resisted over a greater length of wall than the reinforcement rupture loads. Therefore, for pullout, it is recommended that the upper layer of soil reinforcement be designed for a pullout impact load equivalent to a static load of 1,300 lb/lft (19.0 kN/m) of wall; and the second layer be designed with a pullout impact load equivalent to a static load of 600 lb/lft (8.8 kN/m).

Resistance Factors for Tensile and Pullout Resistance

The resistance factors presented in Table 4-7 for "Combined static/traffic barrier impact" are recommended for Extreme Event II impact loading. (Note that AASHTO does not specifically address tensile resistance factors for impact loading.) The tensile and connection rupture resistance factors are a function of the type of reinforcement.

A pullout resistance factor of 1.00 is recommended for metallic and geosynthetic reinforcements. (Note that AASHTO does not specifically address pullout resistance factors for impact loading.)

Barrier, Coping, and Moment Slab Design

Example traffic barriers are illustrated in Figure 5-2. Typically, the base slab length is 20 ft (6 m) and jointed to adjacent slabs with shear dowels. Parapet reinforcement shall be designed in accordance with AASHTO Section 13 Railings. See NCHRP 22-20 report (Bligh et al., 2009) for barrier, coping, and moment loading recommendations. The anchoring slab shall be strong enough to resist the ultimate strength of the standard parapet, and sized to provide adequate resistance to sliding and overturning.

MSE Facing Panel Design

The upper facing panel must be separated from the barrier slab with 1 to 2 in. (25 to 50 mm) of expanded polystyrene (see Figure 5-2(b)). The distance should be adequate to allow the barrier and slab to resist the impact load in sliding and overturning without loading the facing panel. Separation between the precast facing and cast-in-place resistance slab is required to prevent stressing on the facing panels due to slab curing and shrinking.

7.2.2 Post and Beam Railings

Flexible post and beam barriers, when used, shall be placed at a minimum distance of 3.0 ft (0.9 m) from the wall face, driven 5.0 ft (1.5 m) below grade, and spaced to miss the reinforcements where possible. If the reinforcements cannot be missed, the wall shall be designed accounting for the presence of an obstruction. Each of the upper two rows of reinforcement shall be designed for an additional horizontal load of 150 lb/lft (2.2 kN/m) of wall, for a total additional load of 300 lb/ft (4.4 kN/m).

7.3 SUPERFLOOD EVENTS AND SCOUR

The stability of walls and abutments in areas of turbulent flow must be addressed in design. Wall design should be based on the total scour depths estimated per Article 2.6.4.4.2 (AASHTO, 2007). Scour should be investigated for two flood conditions:

- Design Flood
- Check Flood

The design flood (storm surge, tide, or mixed population flood) is the more severe of the 100-year event or an overtopping flood of lesser recurrence interval. Stability design of the wall should be assessed assuming that the streambed material above the total scour line has been removed. This should be analyzed as a strength limit state.

The check flood (storm surge, tide, or mixed population flood) is the more severe of the 500-year event or an overtopping flood of lesser recurrence interval. Stability design of the wall should be assessed assuming that the streambed material above the total scour line has been removed. This is an extreme event, and the extreme event limit state applies. Resistance factors for this extreme limit state may be taken at 1.0, per Articles 10.6.4 and 10.5.5.3.3 (AASHTO, 2007).