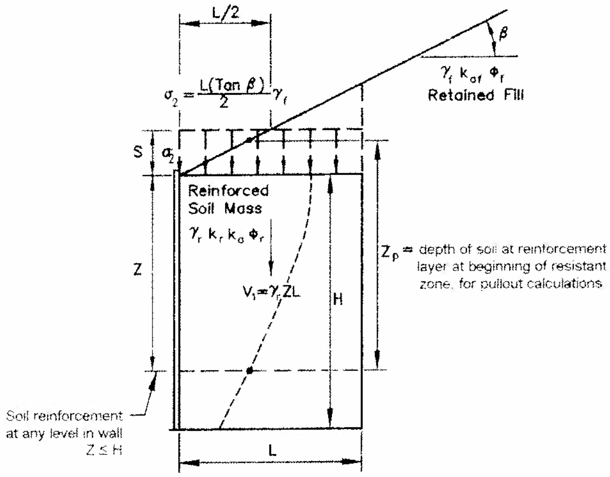



Max Stress: $S = (1/2)L\tan\beta$

$\sigma_v = \gamma_r Z + (1/2)L(\tan\beta)\gamma_f$

Determine $k_{af}$ using a slope angle of $\beta$

Determine $k_r$ from Figure 3

Pullout: $\sigma_v = \gamma_r Z_p$ and $Z_p \ge Z + S$

Note: $H$ is the total height of the wall at the face.

**Figure 11.10.6.2.1-2 Calculation of Vertical Stress for Sloping Backslope Condition for Internal Stability Analysis.**

**For the Simplified Method,** the lateral earth pressure coefficient $k_r$ is determined by applying a multiplier to the active earth pressure coefficient, $k_a$. **The $k_a$ multiplier for the Simplified Method shall be determined as shown in Figure 3. For assessment of reinforcement pullout, the Simplified Method multiplier for steel strip walls shall be used for all steel reinforced walls. For reinforcement rupture, the multiplier applicable to the specific type of steel reinforcement shall be used. For the Coherent Gravity Method, the lateral earth pressure coefficient used for internal stability design of steel reinforced MSE wall systems shall be determined as shown in Figure 4. For geosynthetic reinforced wall systems, $k_a$ is used throughout the wall height. For both methods, $k_a$ shall be** determined using Eq. 3.11.5.3-1, assuming no wall friction, i.e., $\delta = \beta$. **For the Coherent Gravity Method, $k_0$ shall be determined using Eq. 3.11.5.2-1.**

Since it is assumed that $\delta = \beta$, and $\beta$ is assumed to always be zero for internal stability, for a vertical wall, the Coulomb equation simplifies mathematically to the simplest form of the Rankine equation.

The applied factored load to the reinforcements, $T_{max}$, shall be determined using a load per unit of wall width basis as follows:

$$T_{max} = \sigma_H S_v \quad (11.10.6.2.1\text{-}2)$$

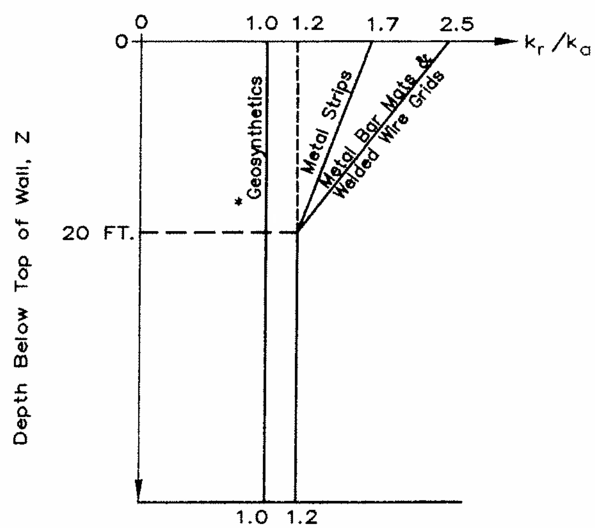
where:

$\sigma_H$ = factored horizontal soil stress at the reinforcement (ksf)

$S_v$ = vertical spacing of the reinforcement (ft.)

A vertical spacing, $S_v$, greater than 2.7 ft. should not be used without full scale wall data (e.g., reinforcement loads and strains, and overall deflections) that support the acceptability of larger vertical spacing.

Live loads shall be positioned for extreme force effect. The provisions of Article 3.11.6 shall apply.




*Does not apply to polymer strip reinforcement

**Figure 11.10.6.2.1-3 Variation of the Coefficient of Lateral Stress Ratio $k_r/k_a$ with Depth in a Mechanically Stabilized Earth Wall.**

$$k_a = \tan^2\left(45 - \frac{\phi'_f}{2}\right) \quad (C11.10.6.2.1\text{-}1)$$

If the wall face is battered, the following simplified form of the Coulomb equation can be used:

$$k_a = \frac{\sin^2(\theta + \phi'_f)}{\sin^3\theta\left(1 + \frac{\sin\phi'_f}{\sin\theta}\right)^2} \quad (C11.10.6.2.1\text{-}2)$$

with variables as defined in Figure 3.11.5.3-1.

Based on Figure 3, the $k_a$ multiplier is a function of the reinforcement type and the depth of the reinforcement below the wall top. Multipliers for other reinforcement types can be developed as needed through analysis of measurements of reinforcement load and strain in full scale structures.

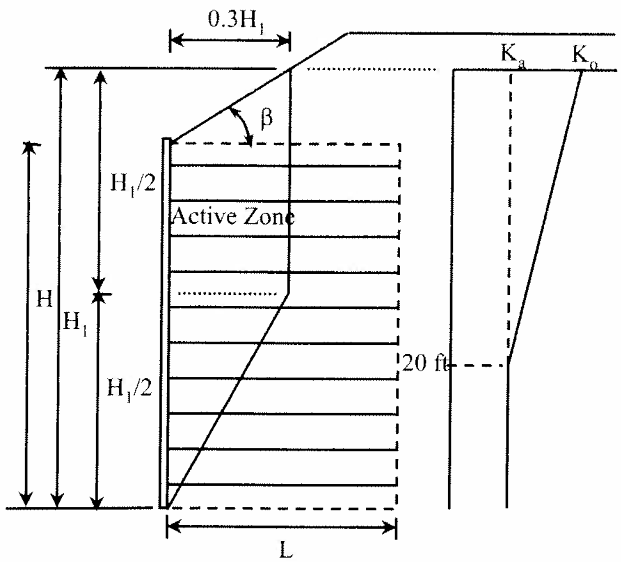



**Figure 11.10.6.2.1-4 Determination of Lateral Earth Pressure Coefficients for Internal Stability Design of Steel Reinforced MSE Walls Using the Coherent Gravity Method.**

*11.10.6.2.2 Reinforcement Loads at Connection to Wall Face*

The factored tensile load applied to the soil reinforcement connection at the wall face, $T_o$, shall be equal to the maximum factored reinforcement tension, $T_{max}$, for all wall systems regardless of facing and reinforcement type.

**11.10.6.3 Reinforcement Pullout**

*11.10.6.3.1 Boundary Between Active and Resistant Zones*

The location of the zone of maximum stress for inextensible and extensible wall systems, i.e., the boundary between the active and resistant zones, is determined as shown in Figure 1. For all wall systems, the zone of maximum stress shall be assumed to begin at the back of the facing elements at the toe of the wall.

For extensible wall systems with a face batter of less than 10° from the vertical, the zone of maximum stress should be determined using the Rankine method. Since the Rankine method cannot account for wall face batter or the effect of concentrated surcharge loads above the reinforced backfill zone, the Coulomb method shall be used for walls with extensible reinforcement in cases of significant batter, defined as 10° from vertical or more, and concentrated surcharge loads to determine the location of the zone of maximum stress.

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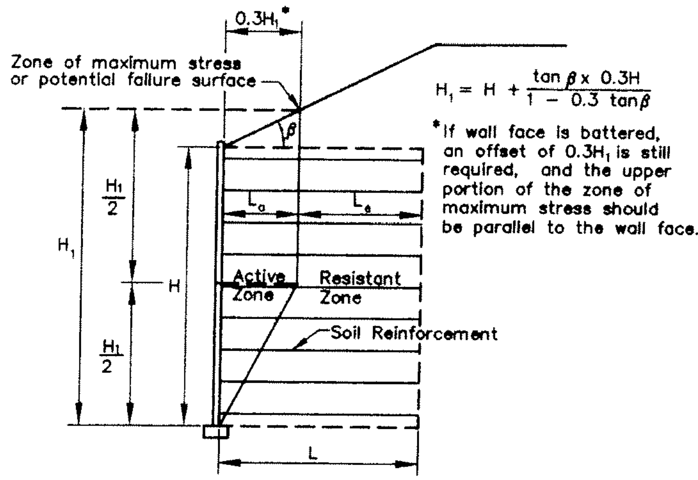
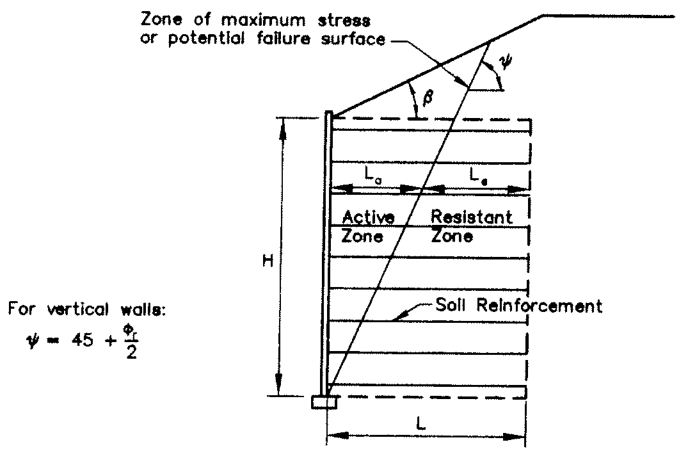



(a) Inextensible Reinforcements




For walls with a face batter 10° or more from the vertical,

$$\tan(\Psi - \phi_r) = \frac{-\tan(\phi_r - \beta) + \sqrt{\tan(\phi_r - \beta)\left[\tan(\phi_r - \beta) + \cot(\phi_r + \theta - 90)\right]\left[1 + \tan(\delta + 90 - \theta)\cot(\phi_r + \theta - 90)\right]}}{1 + \tan(\delta + 90 - \theta)\left[\tan(\phi_r - \beta) + \cot(\phi_r + \theta - 90)\right]}$$

with $\delta = \beta$ and all other variables defined in Figure 3.11.5.3-1.

(b) Extensible Reinforcements

**Figure 11.10.6.3.1-1 Location of Potential Failure Surface for Internal Stability Design of MSE Walls.**

*11.10.6.3.2 Reinforcement Pullout Design*

The reinforcement pullout resistance shall be checked at each level against pullout failure. Only the effective pullout length which extends beyond the theoretical failure surfaces in Figure 11.10.6.3.1-1 shall be used in this calculation. A minimum length, $L_e$, in the resistant zone of 3.0 ft. shall be used. The total length of reinforcement required for pullout is equal to $L_a + L_e$ as shown in Figure 11.10.6.3.1-1.

Note that traffic loads are neglected in pullout calculations (see Figure 11.10.6.2.1-1.

The effective pullout length shall be determined using the following equation:

*C11.10.6.3.2*

$$L_e \geq \frac{T_{max}}{\phi F^* \alpha \sigma_v C R_c} \qquad (11.10.6.3.2\text{-}1)$$

where:

$L_e$ = length of reinforcement in resisting zone (ft.)

$T_{max}$ = applied factored load in the reinforcement from Eq. 11.10.6.2.1-2 (kips/ft.)

$\phi$ = resistance factor for reinforcement pullout from Table 11.5.6-1 (dim.)

$F^*$ = pullout friction factor (dim.)

$\alpha$ = scale effect correction factor (dim.)

$\sigma_v$ = unfactored vertical stress at the reinforcement level in the resistant zone (ksf)

$C$ = overall reinforcement surface area geometry factor based on the gross perimeter of the reinforcement and is equal to 2 for strip, grid and sheet-type reinforcements, i.e., two sides (dim.)

$R_c$ = reinforcement coverage ratio from Article 11.10.6.4.1 (dim.)

$F^*$ and $\alpha$ shall be determined from product-specific pullout tests in the project backfill material or equivalent soil, or they can be estimated empirically/theoretically.

For standard backfill materials (see *AASHTO LRFD Bridge Construction Specifications,* Article 7.3.6.3), with the exception of uniform sands, i.e., coefficient of uniformity $C_u = D_{60}/D_{10} < 4$, in the absence of test data it is acceptable to use conservative default values for F* and $\alpha$ as shown in Figure 1 and Table 1. For ribbed steel strips, if the specific $C_u$ for the wall backfill is unknown at the time of design, a $C_u$ of 4.0 should be assumed for design to determine $F^*$.

**Table 11.10.6.3.2-1 Default Values for the Scale Effect Correction Factor, α.**

| REINFORCEMENT TYPE | DEFAULT VALUE FOR α |
|---|---|
| All Steel Reinforcements | 1.0 |
| Geogrids | 0.8 |
| Geotextiles | 0.6 |

For grids, the spacing between transverse grid elements, $S_t$, shall be uniform throughout the length of the reinforcement rather than having transverse grid members concentrated only in the resistant zone.

$F^* \alpha \sigma_v C L_e$ is the ultimate pullout resistance $P_r$ per unit of reinforcement width.

Pullout testing and interpretation procedures (and direct shear testing for some parameters), as well as typical empirical data, are provided in Appendix A of FHWA Publication No. FHWA-NHI-00-043 (*Elias et al. 2001*).

Recent experience with pullout test results on new geogrids coming into the market has indicated that some materials have pullout values that are lower than the previous $F^*$ default value of 0.8 tan $\phi$. Data obtained by D'Appolonia (*1999*) also indicates that 0.8Tan $\phi$ is closer to a mean value rather than a default lower bound value for geogrids. The default values for other reinforcement types shown in Figure 1 are more representative of lower bound values. The $F^*$ default value has thus been lowered to a more conservative value of 0.67 tan $\phi$ in consideration of these results.

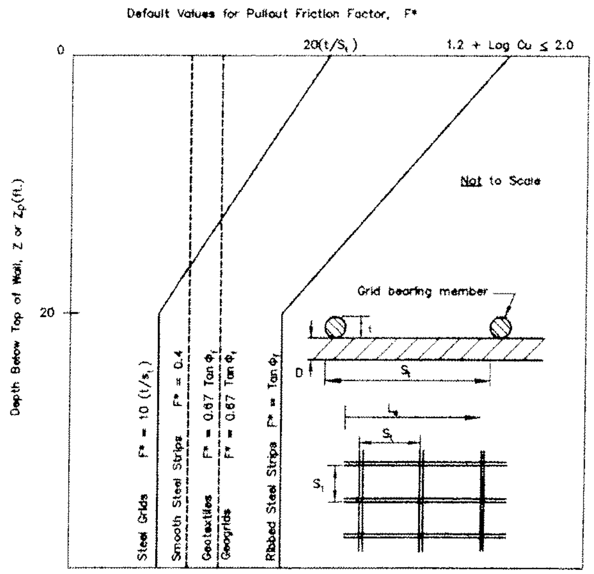



**Figure 11.10.6.3.2-1 Default Values for the Pullout Friction Factor, *F**.**

These pullout calculations assume that the factored long-term strength of the reinforcement (see Article 11.10.6.4.1) in the resistant zone is greater than $T_{max}$.

### 11.10.6.4 Reinforcement Strength

#### *11.10.6.4.1 General*

The reinforcement strength shall be checked at every level within the wall, both at the boundary between the active and resistant zones (i.e., zone of maximum stress), and at the connection of the reinforcement to the wall face, for applicable strength limit states as follows:

At the zone of maximum stress:

$$T_{max} \leq \phi T_{al} R_c \qquad (11.10.6.4.1\text{-}1)$$

where:

$T_{max}$ = applied factored load to the reinforcement determined from Eq. 11.10.6.2.1-2 (kips/ft.)

$\phi$ = resistance factor for reinforcement tension, specified in Table 11.5.6-1 (dim.)

$T_{al}$ = nominal long-term reinforcement design strength (kips/ft.)

$R_c$ = reinforcement coverage ratio specified in Article 11.10.6.4.1 (dim.)

$T_{al}$ shall be determined as specified in Article 11.10.6.4.3a for steel reinforcement and Article 11.10.6.4.3b for geosynthetic reinforcement.

#### *C11.10.6.4.1*

The serviceability limit state is not specifically evaluated in current practice to design backfill reinforcement for internal stability. A first order estimate of lateral deformation of the entire wall structure, however, can be obtained as shown in Article 11.10.4.2.

At the connection with the wall face:

$$T_o \leq \phi T_{ac} R_c \quad (11.10.6.4.1\text{-}2)$$

where:

$T_o$ = applied factored load at reinforcement/facing connection specified in Article 11.10.6.2.2 (kips/ft.)

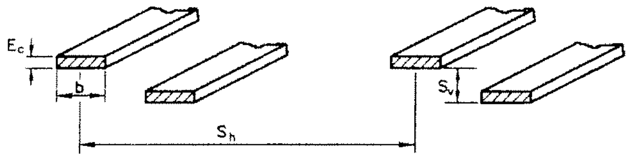
$\phi$ = resistance factor for reinforcement tension in connectors specified in Table 11.5.6-1 (dim.)

$T_{ac}$ = nominal long-term reinforcement/facing connection design strength (kips/ft.)

$R_c$ = reinforcement coverage ratio specified in Article 11.10.6.4.1 (dim.)

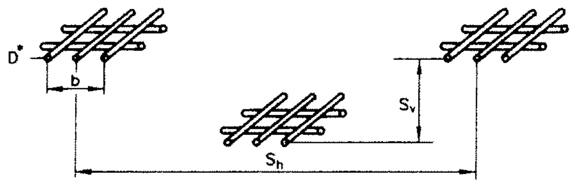
$T_{ac}$ shall be determined at the wall face connection as specified in Article 11.10.6.4.4a for steel reinforcement and Article 11.10.6.4.4b for geosynthetic reinforcement. The difference in the environment occurring immediately behind the wall face relative to the environment within the reinforcement backfill zone and its effect on the long-term durability of the reinforcement/connection shall be considered when determining $T_{ac}$.

$T_{al}$ shall be determined on a long-term strength per unit of reinforcement width basis and multiplied by the reinforcement coverage ratio $R_c$ so that it can be directly compared to $T_{max}$ which is determined on a load per unit of wall width basis (this also applies to $T_{ac}$ and $T_o$). For discrete, i.e., not continuous, reinforcements, such as steel strips or bar mats, the strength of the reinforcement is converted to a strength per unit of wall width basis as shown in Figures 1 and 2. For continuous reinforcement layers, $b = 1$ and $R_c = 1$.




$A_c = bE_c$

$E_c$ = strip thickness corrected for corrosion loss.




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

$D^*$ = diameter of bar or wire corrected for corrosion loss.

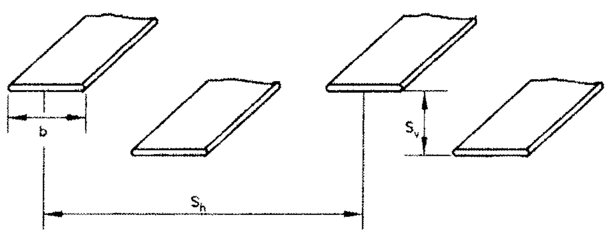
b = unit width of reinforcement (if reinforcement is continuous count number of bars for reinforcement width of 1 unit).

$R_c$ = reinforcement coverage ratio = $\frac{b}{S_h}$

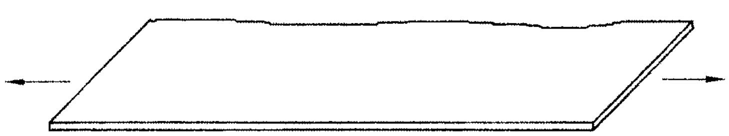
Use $R_c$ = 1 for continuous reinforcement (i.e., $S_h$ = b = 1 unit width).

**Figure 11.10.6.4.1-1 Reinforcement Coverage Ratio for Metal Reinforcement.**

Discontinuous Geosynthetic Sheets:




Continuous Geosynthetic reinforcement sheets:



$R_c$ = reinforcement coverage ratio = $\frac{b}{S_h}$

Use $R_c$ = 1 for continuous geosynthetic sheets (i.e., $S_h$ = b = 1 unit width)

**Figure 11.10.6.4.1-2 Reinforcement Coverage Ratio for Geosynthetic Reinforcement.**

*11.10.6.4.2 Design Life Considerations*

The provisions of Article 11.5.1 shall apply.

*11.10.6.4.2a Steel Reinforcements*

Steel soil reinforcements shall comply with the provisions of *AASHTO LRFD Bridge Construction Specifications*, Article 7.6.4.2, Steel Reinforcements.

The structural design of steel soil reinforcements and connections shall be made on the basis of a thickness, $E_c$, as follows:

$$E_c = E_n - E_s \quad (11.10.6.4.2a\text{-}1)$$

where:

$E_c$ = thickness of metal reinforcement at end of service life as shown in Figure 11.10.6.4.1-1 (mil.)

$E_n$ = nominal thickness of steel reinforcement at construction (mil.)

$E_s$ = sacrificial thickness of metal expected to be lost by uniform corrosion during service life of structure (mil.)

For structural design, sacrificial thicknesses shall be computed for each exposed surface as follows, assuming that the soil backfill used is nonaggressive:

- Loss of galvanizing = 0.58 mil./yr. for first 2 years
  = 0.16 mil./yr. for subsequent years
- Loss of carbon steel = 0.47 mil./yr. after zinc depletion

Soils shall typically be considered nonaggressive if they meet the following criteria:

- pH = 5 to 10
- Resistivity ≥3000 ohm-cm
- Chlorides ≤100 ppm
- Sulfates ≤200 ppm
- Organic Content ≤1 percent

*C11.10.6.4.2a*

Corrosion loss rates summarized in Yannas (*1985*) and supplemented by field data developed under other FHWA research studies have been used to establish the sacrificial thicknesses herein.

The backfill specifications contained in *AASHTO LRFD Bridge Construction Specifications*, Section 7, for MSE structures using steel reinforcements present minimum electrochemical requirements, which will generally ensure a mild to moderate potential for corrosion. Where deicing salts are used, adequate drainage provisions for salt laden runoff is required. In some cases, an impervious membrane may be required between the pavement structure and the select backfill. Criteria for evaluating potential corrosion losses are given in Elias (*1990*).

These sacrificial thicknesses account for potential pitting mechanisms and much of the uncertainty due to data scatter, and are considered to be maximum anticipated losses for soils which are defined as nonaggressive.

Recommended test methods for soil chemical property determination include AASHTO T 289-91 I for pH, AASHTO T 288-91 I for resistivity, AASHTO T 291-91 I for chlorides and AASHTO T 290-91 I for sulfates.

These sacrificial thickness requirements are not applicable for soils which do not meet one or more of the nonaggressive soil criteria. Additionally, these sacrificial thickness requirements are not applicable in applications where:

- The MSE wall will be exposed to a marine or other chloride rich environment,
- The MSE wall will be exposed to stray currents such as from nearby underground power lines or adjacent electric railways,
- The backfill material is aggressive, or
- The galvanizing thickness is less than specified in these guidelines.

If the resistivity is greater than or equal to 5000 ohm-cm, the chlorides and sulfates requirements may be waived. For bar mat or grid-type reinforcements, the sacrificial thickness listed above shall be applied to the radius of the wire or bar when computing the cross-sectional area of the steel remaining after corrosion losses.

Transverse and longitudinal grid members shall be sized in accordance with ASTM A 185. The transverse wire diameter shall be less than or equal to the longitudinal wire diameter.

Galvanized coatings shall be a minimum of 2 oz./ft.$^2$ or 3.4 mils. in thickness, applied in conformance to AASHTO M 111 (ASTM A 123) for strip-type reinforcements or ASTM A 641 for bar mat or grid-type steel reinforcement.

Each of these situations creates a special set of conditions which should be specifically analyzed by a corrosion specialist. Alternatively, noncorrosive reinforcing elements can be considered. Furthermore, these corrosion rates do not apply to other metals. The use of alloys such as aluminum and stainless steel is not recommended.

Requiring the transverse wire diameter to be less than or equal to the longitudinal wire diameter will preclude local overstressing of the longitudinal wires.

Corrosion-resistant coatings should generally be limited to galvanization.

There is insufficient evidence at this time regarding the long-term performance of epoxy coatings for these coatings to be considered equivalent to galvanizing. If epoxy-type coatings are used, they should meet the requirements of ASTM A 884 for bar mat and grid reinforcements, or AASHTO M 284 for strip reinforcements, and have a minimum thickness of 16 mils.

*11.10.6.4.2b Geosynthetic Reinforcements*

Within specific limits of wall application, soil conditions, and polymer type, strength degradation due to environmental factors can be anticipated to be minimal and relatively consistent from product-to-product, and the impact of any degradation which does occur will be minimal. This allows application of a single default reduction factor, *RF*, to the ultimate tensile strength to account for long-term strength losses, as described in Article 11.10.6.4.3b.

Where wall application limits, soil aggressiveness and polymer requirements are consistent with the conditions below, a single default reduction factor specified herein may be used:

- Poor performance of failure will not have severe consequences
- The soil is considered nonaggressive
- The polymer material meets the requirements provided in Table 1

1) *Structure Application Issues*: Identification of applications for which the consequences of poor performance or failure are severe shall be as described in Article 11.5.1. In such applications, a single default reduction factor shall not be used for final design.

2) *Determination of Soil Aggressiveness*: Soil aggressiveness for geosynthetics shall be assessed based on the soil pH, gradation, plasticity, organic content, and in-ground temperature. Soil shall be defined as nonaggressive if the following criteria are met:

*C11.10.6.4.2b*

The durability of geosynthetic reinforcement is influenced by environmental factors such as time, temperature, mechanical damage, stress levels and chemical exposure, e.g., oxygen, water, and pH, which are the most common chemical factors. Microbiological attack may also affect certain polymers, although not most polymers used for carrying load in soil reinforcement applications. The effects of these factors on product durability are dependent on the polymer type used, i.e., resin type, grade, additives, and manufacturing process, and the macrostructure of the reinforcement. Not all of these factors will have a significant effect on all geosynthetic products. Therefore, the response of geosynthetic reinforcements to these long-term environmental factors is product specific.

- pH, as determined by AASHTO T 289-91, I = 4.5 to 9 for permanent applications and 3 to 10 for temporary applications,

- Maximum soil particle size is less than 0.75 in., unless full scale installation damage tests are conducted in accordance with ASTM D 5818,

- Soil organic content, as determined by AASHTO T 267-86 for material finer than the 0.0787 in. (No. 10) sieve ≤1 percent, and

- Design temperature at wall site:
  ≤ 86°F for permanent applications
  ≤ 95°F for temporary applications

Soil backfill not meeting these requirements as provided herein shall be considered to be aggressive. The environment at the face, in addition to that within the wall backfill, shall be evaluated, especially if the stability of the facing is dependent on the strength of the geosynthetic at the face, i.e., the geosynthetic reinforcement forms the primary connection between the body of the wall and the facing.

The chemical properties of the native soil surrounding the mechanically stabilized soil backfill shall also be considered if there is potential for seepage of groundwater from the native surrounding soils to the mechanically stabilized backfill. If this is the case, the surrounding soils shall also meet the chemical criteria required for the backfill material if the environment is to be considered nonaggressive, or adequate long-term drainage around the geosynthetic reinforced mass shall be provided to ensure that chemically aggressive liquid does not enter into the reinforced backfill.

The effective design temperature is defined as the temperature which is halfway between the average yearly air temperature and the normal daily air temperature for the warmest month at the wall site. Note that for walls which face the sun, it is possible that the temperature immediately behind the facing could be higher than the air temperature. This condition should be considered when assessing the design temperature, especially for wall sites located in warm, sunny climates.

3) *Polymer Requirements*: Polymers which are likely to have good resistance to long-term chemical degradation shall be used if a single default reduction factor is to be used, to minimize the risk of the occurrence of significant long-term degradation. The polymer material requirements provided in Table 1 shall, therefore, be met if detailed product specific data as described in Elias et al. (*2001*) and Elias (*2000*) is not obtained. Polymer materials not meeting the requirements in Table 1 may be used if this detailed product specific data extrapolated to the design life intended for the structure are obtained.

Guidelines for product-specific studies to determine RF are provided in Elias et al. (*2001*) and Elias (*2000*).

For applications involving:

- severe consequences of poor performance or failure,

- aggressive soil conditions,

- polymers not meeting the specific requirements set in Table 1, or
- a desire to use an overall reduction factor less than the default reduction factor recommended herein,

then product-specific durability studies shall be carried out prior to product use to determine the product-specific long-term strength reduction factor, *RF*. These product-specific studies shall be used to estimate the short-term and long-term effects of these environmental factors on the strength and deformational characteristics of the geosynthetic reinforcement throughout the reinforcement design life.

**Table 11.10.6.4.2b-1 Minimum Requirements for Geosynthetic Products to Allow Use of Default Reduction Factor for Long-Term Degradation.**

| Polymer Type | Property | Test Method | Criteria to Allow Use of Default *RF* |
|---|---|---|---|
| Polypropylene | UV Oxidation Resistance | ASTM D4355 | Minimum 70% strength retained after 500 hrs. in weatherometer |
| Polyethylene | UV Oxidation Resistance | ASTM D4355 | Minimum 70% strength retained after 500 hrs. in weatherometer |
| Polyester | Hydrolysis Resistance | Intrinsic Viscosity Method (ASTM D4603) and GRI Test Method GG8, or Determine Directly Using Gel Permeation Chromatography | Minimum Number Average Molecular Weight of 25000 |
| Polyester | Hydrolysis Resistance | GRI Test Method GG7 | Maximum of Carboxyl End Group Content of 30 |
| All Polymers | Survivability | Weight per Unit Area (ASTM D5261) | Minimum 270 $g/m^2$ |
| All Polymers | % Post-Consumer Recycled Material by Weight | Certification of Materials Used | Maximum of 0% |

*11.10.6.4.3 Design Tensile Resistance*

*11.10.6.4.3a Steel Reinforcements*

The nominal reinforcement tensile resistance is determined by multiplying the yield stress by the cross-sectional area of the steel reinforcement after corrosion losses (see Figure 11.10.6.4.1-1). The loss in steel cross-sectional area due to corrosion shall be determined in accordance with Article 11.10.6.4.2a. The reinforcement tensile resistance shall be determined as:

$$T_{al} = \frac{A_c F_y}{b} \qquad (11.10.6.4.3a\text{-}1)$$

where:

$T_{al}$ = nominal long-term reinforcement design strength (kips/ft.)

$F_y$ = minimum yield strength of steel (ksi)

$A_c$ = area of reinforcement corrected for corrosion loss (Figure 11.10.6.4.1-1) (in.$^2$)

$b$ = unit width of reinforcement (Figure 11.10.6.4.1-1) (ft.)

*11.10.6.4.3b Geosynthetic Reinforcements*

The nominal long-term reinforcement tensile strength shall be determined as:

$$T_{al} = \frac{T_{ult}}{RF} \qquad (11.10.6.4.3b\text{-}1)$$

where:

$$RF = RF_{ID} \times RF_{CR} \times RF_D \qquad (11.10.6.4.3b\text{-}2)$$

and:

$T_{al}$ = nominal long-term reinforcement design strength (kips/ft.)

$T_{ult}$ = minimum average roll value (MARV) ultimate tensile strength (kips/ft.)

$RF$ = combined strength reduction factor to account for potential long-term degradation due to installation damage, creep and chemical aging (dim.)

$RF_{ID}$ = strength reduction factor to account for installation damage to reinforcement (dim.)

$RF_{CR}$ = strength reduction factor to prevent long-term creep rupture of reinforcement (dim.)

$RF_D$ = strength reduction factor to prevent rupture of reinforcement due to chemical and biological degradation (dim.)

Values for $RF_{ID}$, $RF_{CR}$, and $RF_D$ shall be determined from product specific test results as specified in Article 11.10.6.4.2b. Even with product specific test results, neither $RF_{ID}$ nor $RF_D$ shall be less than 1.1.

*C11.10.6.4.3b*

$T_{al}$ is the long-term tensile strength required to prevent rupture calculated on a load per unit of reinforcement width basis. $T_{ult}$ is the ultimate tensile strength of the reinforcement determined from wide width tensile tests specified in ASTM D 4595 for geotextiles and ASTM D 6637 for geogrids. The value selected for $T_{ult}$ is the minimum average roll value (MARV) for the product to account for statistical variance in the material strength.

Guidelines for determination of $RF_{ID}$, $RF_{CR}$, and $RF_D$ from product-specific data are provided in Elias et al. (*2001*) and Elias (*2001*).

For wall applications which are defined as not having severe consequences should poor performance or failure occur, having nonaggressive soil conditions, and if the geosynthetic product meets the minimum requirements listed in Table 1, the long-term tensile strength of the reinforcement may be determined using a default reduction factor for *RF* as provided in Table 1 in lieu of product-specific test results.

**Table 11.10.6.4.3b-1 Default and Minimum Values for the Total Geosynthetic Ultimate Limit State Strength Reduction Factor, *RF*.**

| Application | Total Reduction Factor, *RF* |
|---|---|
| All applications, but with product-specific data obtained and analyzed in accordance with Elias (*2001*) and Elias et al. (*2001*) | All reduction factors shall be based on product specific data. Neither $RF_{ID}$ nor $RF_D$ shall be less than 1.1. |
| Permanent applications not having severe consequences should poor performance or failure occur, nonaggressive soils, and polymers meeting the requirements listed in Table 11.10.6.4.2b-1 | 7.0 |
| Temporary applications not having severe consequences should poor performance or failure occur, nonaggressive soils, and polymers meeting the requirements listed in Table 11.10.6.4.2b-1 provided product specific data are not available | 3.5 |

*11.10.6.4.4 Reinforcement/Facing Connection Design Strength*

*11.10.6.4.4a Steel Reinforcements*

Connections shall be designed to resist stresses resulting from active forces, $T_o$, in Article 11.10.6.2.2, as well as from differential movements between the reinforced backfill and the wall facing elements.

Elements of the connection which are embedded in the facing element shall be designed with adequate bond length and bearing area in the concrete to resist the connection forces. The capacity of the embedded connector shall be checked by tests as required in Article 5.11.3. Connections between steel reinforcement and the wall facing units, e.g., welds, bolts, pins, etc., shall be designed in accordance with Article 6.13.3.

Connection materials shall be designed to accommodate losses due to corrosion in accordance with Article 11.10.6.4.2a. Potential differences between the environment at the face relative to the environment within the reinforced soil mass shall be considered when assessing potential corrosion losses.

*11.10.6.4.4b Geosynthetic Reinforcements*

The portion of the connection embedded in the concrete facing shall be designed in accordance with Article 5.11.3.

The nominal long-term geosynthetic connection strength $T_{ac}$ on a load per unit reinforcement width basis shall be determined as follows:

$$T_{ac} = \frac{T_{ult} \times CR_{cr}}{RF_D} \quad (11.10.6.4.4b\text{-}1)$$

where:

$T_{ac}$ = nominal long-term reinforcement/facing connection design strength per unit of reinforcement width at a specified confining pressure (kips/ft.)

$T_{ult}$ = minimum average roll value (MARV) ultimate tensile strength of soil reinforcement (kips/ft.)

$CR_{cr}$ = long-term connection strength reduction factor to account for reduced ultimate strength resulting from connection (dim.)

$RF_D$ = reduction factor to prevent rupture of reinforcement due to chemical and biological degradation (Article 11.10.6.4.3b) (dim.)

*C11.10.6.4.4b*

The long-term creep reduced geosynthetic strength at the connection with the wall facing is obtained by reducing $T_{ult}$ by $CR_{cr}$ using the connection/seam strength determined in accordance with long-term connection strength test protocol as described in Appendix A of Elias et al. (*2001*). The connection test is similar in nature to a wide width tensile test (ASTM D 4595 or ASTM D 6637), except that one end of the reinforcement material is sandwiched between two courses of concrete blocks to form one of the grips. This protocol consists of a series of connection creep tests carried out over an extended period of time to evaluate the potential for creep rupture at the connection. $CR_{cr}$ is taken as the creep reduced connection strength, $T_{crc}$, extrapolated to the specified design life, divided by the ultimate wide width tensile strength (ASTM D 4595 or D 6637) for the reinforcement material lot used for the connection strength testing, $T_{lot}$.

$CR_{cr}$ may also be obtained from short-term connection test (ASTM D4884 for seam connections, or NCMA Test Method SRWU-1 in Simac et al. (*1993*) for segmental concrete block connections) results, which are to obtain a short-term ultimate connection strength reduction factor $CR_u$. $C_{ru}$ is taken as the ultimate connection strength $T_{ultconn}$ from SRWU-1 or ASTM D 4884, divided by $T_{lot}$ as described above. In this case, $CR_u$ must be further reduced by the creep reduction factor $RF_{CR}$ (Article 11.10.6.4.3b) in order to account for the potential of creep rupture as follows:

$$CR_{cr} = \frac{CR_u}{RF_{CR}} \quad (C11.10.6.4.4b\text{-}1)$$

For reinforcements connected to the facing through embedment between facing elements, e.g., segmental concrete block faced walls, the capacity of the connection is conceptually governed by one of two failure modes: rupture, or pullout of the reinforcement. This is consistent with the evaluation of internal wall stability in the reinforced backfill zone, where both the rupture and pullout mode of failure must be considered.

The objective of the connection design is to assess the long-term capacity of the connection. If rupture is the mode of failure, the long-term effects of creep and durability on the geosynthetic reinforcement at the connection, as well as on the connector materials, must be taken into account, as the capacity of the connection is controlled by the reinforcement or connector long-term strength. If pullout is the mode of failure, the capacity of the connection is controlled by the frictional interface between the facing blocks and the geosynthetic reinforcement. It is assumed for design that this interface is not significantly affected by time dependent mechanisms such as creep or chemical degradation. This again is consistent with the design of the soil reinforcement within the wall backfill. The load bearing fibers or ribs of the geosynthetic do not necessarily have to experience rupture in the connection test for the mode of failure to be rupture. If the connector is a material that is susceptible to creep, failure of the connectors between blocks due to creep rupture of the connector could result in long-term connection strength losses. In these cases, the value of $CR_{cr}$ and $RF_D$ to be used in Eq. 1 should be based on the durability of the connector, not the geosynthetic.

Regardless of the failure mode, the long-term connection test referenced in Elias et al. (*2001*) addresses the long-term capacity of the connection. Eq. C1 above should also be considered to conservatively apply to both failure modes, if the long-term connection test is not performed.

If the connectors between blocks are intended to be used for maintaining block alignment during wall construction and are not intended for long-term connection shear capacity, the alignment connectors should be removed before assessing the connection capacity for the selected block-geosynthetic combination. If the pins or other connection devices are to be relied upon for long-term capacity, the durability of the connector material must be established.

Values for $RF_{CR}$ and $RF_D$ shall be determined from product-specific test results, except as otherwise specified herein. The environment at the wall face connection may be different than the environment away from the wall face in the wall backfill. This shall be considered when determining $RF_{CR}$ and $RF_D$.

$CR_{cr}$ shall be determined at the anticipated vertical confining pressure at the wall face between the facing blocks. The vertical confining pressure shall be calculated using the Hinge Height Method as shown in Figure 1 for a face batter, $\omega$, of greater than 8°. $T_{ac}$ should not be greater than $T_{al}$.

Guidelines for determining $RF_{CR}$ and $RF_D$ from product-specific data are provided in Elias et al. (*2001*) and Elias (*2001*). The use of default reduction factors may be acceptable where the reinforcement load is maximum, i.e., in the middle of the wall backfill, and still not be acceptable at the facing connection if the facing environment is defined as aggressive.

Geosynthetic walls may be designed using a flexible reinforcement sheet as the facing using only an overlap with the main soil reinforcement. The overlaps shall be designed using a pullout methodology. By replacing $T_{max}$ with $T_o$, Eq. 11.10.6.3.2-1 may be used to determine the minimum overlap length required, but in no case shall the overlap length be less than 3.0 ft. If tan ρ is determined experimentally based on soil to reinforcement contact, tan ρ shall be reduced by 30 percent where reinforcement to reinforcement contact is anticipated.

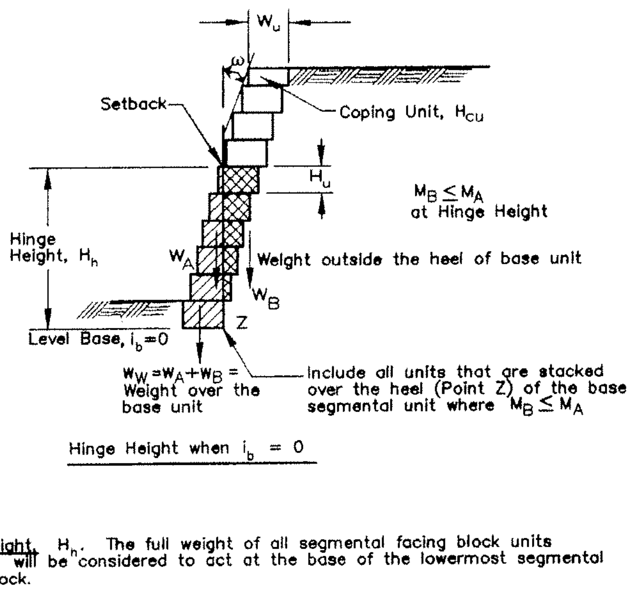



**Figure 11.10.6.4.4b-1 Determination of Hinge Height for Segmental Concrete Block Faced MSE Walls.**

The hinge height, $H_h$, shown in Figure 1, shall be determined as:

$$H_h = 2\left[\left(W_u - G_u - 0.5H_u \tan i_h\right)\cos i_b\right]/\tan\left(\omega + i_b\right) \quad (11.10.6.4.4b\text{-}1)$$

where:

$H_u$ = segmental facing block unit height (ft.)

$W_u$ = segmental facing block unit width, front to back (ft.)

$G_u$ = distance to the center of gravity of a horizontal segmental facing block unit, including aggregate fill, measured from the front of the unit (ft.)

$\omega$ = wall batter due to setback per course (°)

$H$ = total height of wall (ft.)

$H_h$ = hinge height (ft.)

## 11.10.7 Seismic Design

### 11.10.7.1 External Stability

Stability determinations shall be made by applying the sum of static forces, the horizontal inertial force, $P_{IR}$, and 50 percent of the dynamic horizontal thrust, $P_{AE}$ to the wall. The locations of $P_{AE}$ and $P_{IR}$ shall be taken as illustrated in Figure 1. These forces are combined with the static forces, factored in accordance with Article 3.4.1. The dynamic horizontal thrust, $P_{AE}$, shall be evaluated using the pseudo-static Mononobe-Okabe method and shall be applied to the back surface of the reinforced fill at the height of $0.6H$ from the base and the horizontal inertial force shall be applied at the center of dynamic mass of the structure. $A_m$, the maximum acceleration coefficient at the wall centroid, except as noted in Article C11.10.7.1, shall be determined as:

$$A_m = (1.45 - A)A \tag{11.10.7.1-1}$$

where:

$A$ = maximum earthquake acceleration coefficient (Article 3.10.2) (dim.)

Values of $P_{AE}$ and $P_{IR}$ for structures with horizontal backfill may be determined using the following simplified equations:

$$P_{AE} = 0.375\gamma_{EQ}A_m\gamma_s H^2 \tag{11.10.7.1-2}$$

$$P_{IR} = 0.5\gamma_{EQ}A_m\gamma_s H^2 \tag{11.10.7.1-3}$$

where:

$\gamma_{EQ}$ = load factor for $EQ$ loads from Table 3.4.1-1 (dim.)

$A_m$ = maximum wall acceleration coefficient at the centroid of the wall mass (dim.)

$\gamma_s$ = soil unit weight (kcf)

$H$ = height of wall (ft.)

For structures with sloping backfills, the inertial force, $P_{IR}$, shall be based on an effective mass having a height $H_2$ and a base width equal to 0.5 $H_2$ determined as follows:

$$H_2 = H + \frac{0.5H\tan(\beta)}{\left[1 - 0.5\tan(\beta)\right]} \tag{11.10.7.1-4}$$

### C11.10.7.1

The equation for $P_{AE}$ was developed assuming a friction angle of 30°. $P_{AE}$ may be calculated using the Mononobe-Okabe method, with the horizontal acceleration $k_h$ equal to $A_m$ and $k_v$ equal to zero, multiplied by the load factor $\gamma_{EQ}$.

The seismic earth pressure calculated using the Mononabe-Okabe method may be reduced in accordance with Article 11.6.5 for walls which can displace laterally. If it is desired to apply the procedures provided in Article C11.6.5 to account for the effect of lateral displacement in reducing seismic lateral earth pressures, calculate $A_m$ as follows:

$$A_m = (1.45 - k_h)k_h \tag{C11.10.7.1-1}$$

Where $k_h$ is obtained from Eq. C11.6.5-1.

It is recommended that this reduced acceleration value only be used for external stability calculations, including wall inertial forces as well as seismic earth pressures behind the wall, to be consistent with the concept of the MSE wall behaving as a rigid block. Internally, the lateral deformation response of the MSE wall is much more complex, and it is not clear at this time how much the acceleration coefficient could decrease due to the allowance of some lateral deformation during seismic loading internally in the MSE wall.

If either $A$ or $k_h$ is greater than 0.45$g$, set $A_m$=$A$ or $A_m$=$k_h$, respectively, as this equation has not been specifically validated for higher accelerations, and a value of $A_m$ less than $A$ or $k_h$ would result.

The seismic design procedures provided herein do not directly account for the lateral deformation that may occur during large earthquake seismic loading. It is, therefore, recommended that if the anticipated ground acceleration is greater than 0.29$g$, a detailed lateral deformation analysis of the structure during seismic loading should be performed, such as a Newmark sliding analysis or numerical modeling.

where:

$\beta$ = slope of backfill (°)

$P_{IR}$ for sloping backfills shall be determined as:

$$P_{IR} = P_{ir} + P_{is} \qquad (11.10.7.1\text{-}5)$$

where:

$$P_{ir} = 0.5\gamma_{EQ} A_m \gamma_s H_2 H \qquad (11.10.7.1\text{-}6)$$

$$P_{is} = 0.125\gamma_{EQ} A_m \gamma_s (H_2)^2 \tan(\beta) \qquad (11.10.7.1\text{-}7)$$

where:

$P_{ir}$ = the inertial force caused by acceleration of the reinforced backfill (kips/ft.)

$P_{is}$ = the inertial force caused by acceleration of the sloping soil surcharge above the reinforced backfill (kips/ft.)

The width of mass contributing to $P_{IR}$ shall be equal to $0.5H_2$. $P_{IR}$ shall act at the combined centroid of $P_{ir}$ and $P_{is}$.

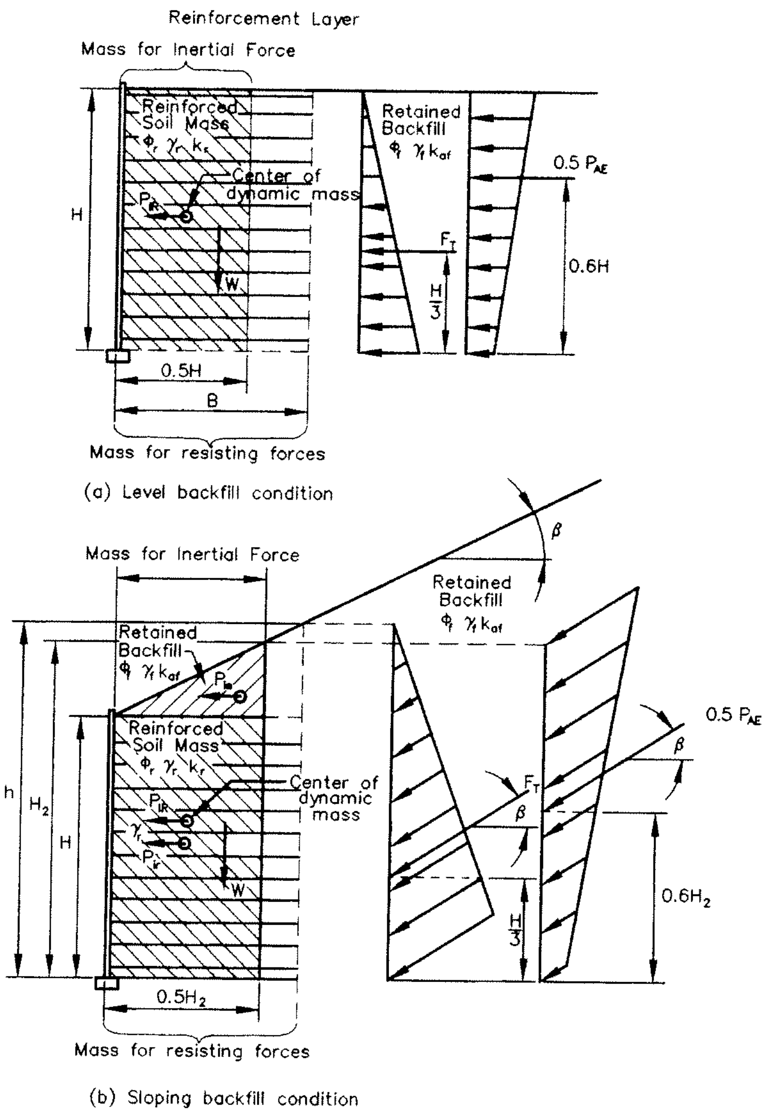



**Figure 11.10.7.1-1 Seismic External Stability of a MSE Wall.**

### 11.10.7.2 Internal Stability

### C11.10.7.2

Reinforcements shall be designed to withstand horizontal forces generated by the internal inertia force, $P_i$, and the static forces. The total inertia force, $P_i$, per unit length of structure shall be considered equal to the mass of the active zone times the maximum wall acceleration coefficient $A_m$. This inertial force shall be distributed to the reinforcements proportionally to their resistant areas on a load per unit width of wall basis as follows:

$$T_{md} = \gamma P_i \frac{L_{ei}}{\sum_{i=1}^{m}(L_{ei})} \qquad (11.10.7.2\text{-}1)$$

where:

$T_{md}$ = factored incremental dynamic inertia force at Layer $i$ (kips/ft.)

$\gamma$ = load factor for $EQ$ loads from Table 3.4.1-1 (dim.)

$P_i$ = internal inertia force due to the weight of backfill within the active zone, i.e., the shaded area on Figure 1 (kips/ft.)

= $A_m W_a$
where $W_a$ is the weight of the active zone and $A_m$ given by Eq. 11.10.7.1-1

$L_{ei}$ = effective reinforcement length for layer $i$ (ft.)

The total factored load applied to the reinforcement on a load per unit of wall width basis as shown in Figure 1 is determined as follows:

$$T_{total} = T_{max} + T_{md} \tag{11.10.7.2-2}$$

where:

$T_{max}$ = the factored static load applied to the reinforcements determined using Eq. 11.10.6.2.1-2.

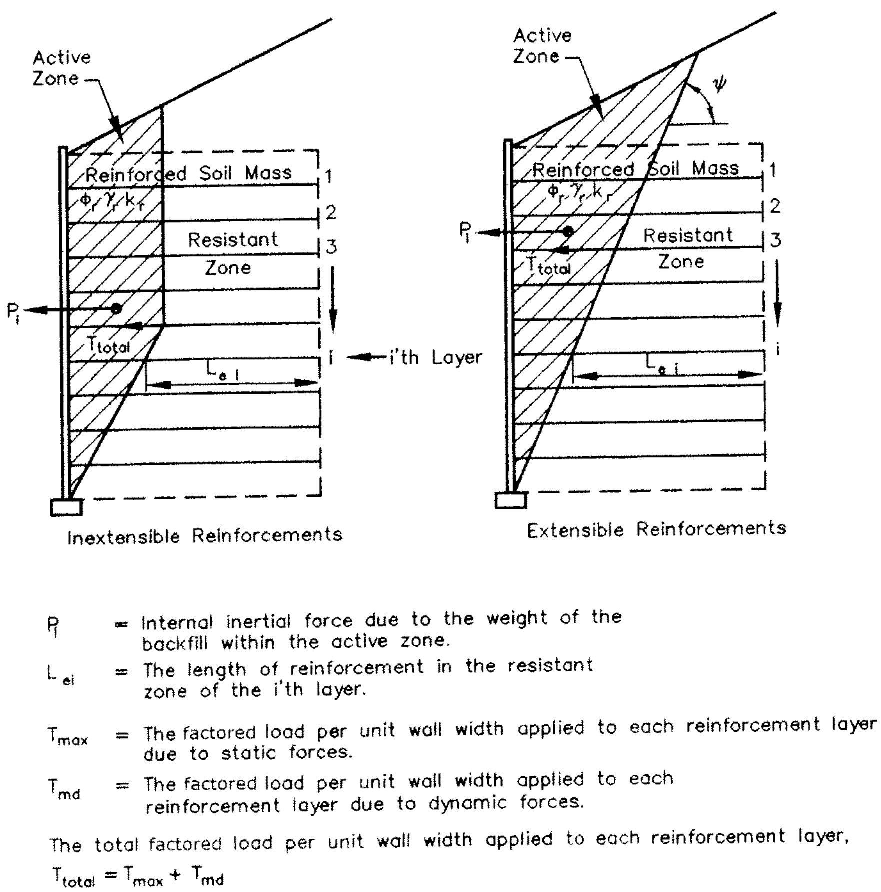



**Figure 11.10.7.2-1 Seismic Internal Stability of a MSE Wall.**

For geosynthetic reinforcement rupture, the reinforcement shall be designed to resist the static and dynamic components of the load determined as:

For the static component:

$$S_{rs} \geq \frac{T_{max} RF}{\phi R_c} \qquad (11.10.7.2\text{-}3)$$

For the dynamic component:

$$S_{rt} \geq \frac{T_{md} RF_{ID} RF_D}{\phi R_c} \qquad (11.10.7.2\text{-}4)$$

where:

$\phi$ = resistance factor for combined static/earthquake loading from Table 11.5.6-1 (dim.)

$S_{rs}$ = ultimate reinforcement tensile resistance required to resist static load component (kips/ft.)

$S_{rt}$ = ultimate reinforcement tensile resistance required to resist dynamic load component (kips/ft.)

$R_c$ = reinforcement coverage ratio specified in Article 11.10.6.4.1 (dim.)

$RF$ = combined strength reduction factor to account for potential long-term degradation due to installation damage, creep, and chemical aging specified in Article 11.10.6.4.3b (dim.)

$RF_{ID}$ = strength reduction factor to account for installation damage to reinforcement specified in Article 11.10.6.4.3b (dim.)

$RF_D$ = strength reduction factor to prevent rupture of reinforcement due to chemical and biological degradation specified in Article 11.10.6.4.3b (dim.)

The required ultimate tensile resistance of the geosynthetic reinforcement shall be determined as:

$$T_{ult} = S_{rs} + S_{rt} \qquad (11.10.7.2\text{-}5)$$

For pullout of steel or geosynthetic reinforcement:

$$L_e \geq \frac{T_{total}}{\phi (0.8F^{*} \alpha \sigma_v C R_c)} \qquad (11.10.7.2\text{-}6)$$

The reinforcement must be designed to resist the dynamic component of the load at any time during its design life. Design for static loads requires the strength of the reinforcement at the end of the design life to be reduced to account for creep and other degradation mechanisms. Strength loss in polymeric materials 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. The resistance of the reinforcement to the static component of load, $T_{max}$, must, therefore, 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.

where:

$L_e$ = length of reinforcement in resisting zone (ft.)

$T_{total}$ = maximum factored reinforcement tension from Eq. 2 (kips/ft.)

$\phi$ = resistance factor for reinforcement pullout from Table 11.5.6-1 (dim.)

$F^*$ = pullout friction factor (dim.)

$\alpha$ = scale effect correction factor (dim.)

$\sigma_v$ = unfactored vertical stress at the reinforcement level in the resistant zone (ksf)

$C$ = overall reinforcement surface area geometry factor (dim.)

$R_c$ = reinforcement coverage ratio specified in Article 11.10.6.4.1 (dim.)

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

### 11.10.7.3 Facing Reinforcement Connections

### C11.10.7.3

Facing elements shall be designed to resist the seismic loads determined as specified in Article 11.10.7.2, i.e., $T_{total}$. Facing elements shall be designed in accordance with applicable provisions of Sections 5, 6, and 8 for reinforced concrete, steel, and timber, respectively.

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

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

For the static component of the load:

$$S_{rs} \geq \frac{T_{max} RF_D}{0.8\phi C R_{cr} R_c} \qquad (11.10.7.3\text{-}1)$$

For the dynamic component of the load:

$$S_{rt} \geq \frac{T_{md} RF_D}{0.8\phi CR_u R_c} \qquad (11.10.7.3\text{-}2)$$

where:

$S_{rs}$ = ultimate reinforcement tensile resistance required to resist static load component (kip/ft.)

$T_{max}$ = applied load to reinforcement (kip/ft.)

$RF_D$ = reduction factor to prevent rupture of reinforcement due to chemical and biological degradation specified in Article 11.10.6.4.4b (dim.)

$\phi$ = resistance factor from Table 11.5.6-1 (dim.)

$CR_{cr}$ = long-term connection strength reduction factor to account for reduced ultimate strength resulting from connection (dim.)

$R_c$ = reinforcement coverage ratio from Article 11.10.6.4.1 (dim.)

$S_{rt}$ = ultimate reinforcement tensile resistance required to resist dynamic load component (kip/ft.)

$T_{md}$ = factored incremental dynamic inertia force (kip/ft.)

$CR_u$ = short-term reduction factor to account for reduced ultimate strength resulting from connection as specified in Article C11.10.6.4.4b (dim.)

For mechanical connections that do not rely on a frictional component, the 0.8 multiplier may be removed from Eqs. 1 and 2.

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

$$T_{ult} = S_{rs} + S_{rt} \qquad (11.10.7.3\text{-}3)$$

For structures in seismic performance Zones 3 or 4, facing connections in segmental block faced walls shall use shear resisting devices between the facing blocks and soil reinforcement such as shear keys, pins, etc., and shall not be fully dependent on frictional resistance between the soil reinforcement and facing blocks.

For steel reinforcement connections, resistance factors for combined static and seismic loads may be increased by 33 percent of factors used for static loading. Based on these resistance factors, the available factored connection strength must be greater than $T_{total}$.

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.

## 11.10.8 Drainage

Internal drainage measures shall be considered for all structures to prevent saturation of the reinforced backfill and to intercept any surface flows containing aggressive elements.

MSE walls in cut areas and side-hill fills with established groundwater levels shall be constructed with drainage blankets in back of, and beneath, the reinforced zone.

For MSE walls supporting roadways which are chemically deiced in the winter, an impervious membrane may be required below the pavement and just above the first layer of soil reinforcement to intercept any flows containing deicing chemicals. The membrane shall be sloped to drain away from the facing to an intercepting longitudinal drain outletted beyond the reinforced zone. Typically, a roughened surface PVC, HDPE or LLDPE geomembrane with a minimum thickness of 30 mils. should be used. All seams in the membrane shall be welded to prevent leakage.

## 11.10.9 Subsurface Erosion

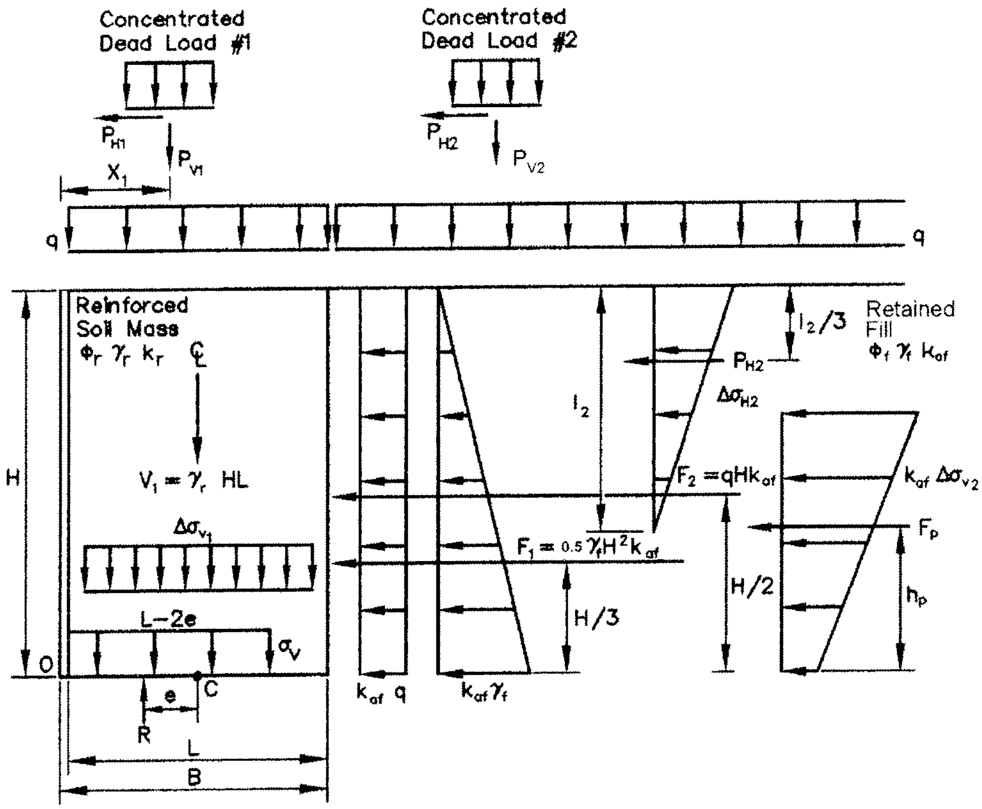
The provisions of Article 11.6.3.5 shall apply.

## 11.10.10 Special Loading Conditions

### 11.10.10.1 Concentrated Dead Loads

The distribution of stresses within and behind the wall resulting from concentrated loads applied to the wall top or behind the wall shall be determined in accordance with Article 3.11.6.3.

Figure 1 illustrates the combination of loads using superposition principles to evaluate external and internal wall stability. Depending on the size and location of the concentrated dead load, the location of the boundary between the active and resistant zones may have to be adjusted as shown in Figure 2.




Note: These equations assume that concentrated dead load #2 is located within the active zone behind the reinforced soil mass.

Note: For relatively thick facing elements, (e.g., segmental concrete facing blocks), it is acceptable to include the facing dimensions and weight in sliding, overturning, and bearing capacity calculations (i.e., use $B$ in lieu of $L$).

Note: $P_{V1}$, $P_{H1}$, $\Delta\sigma_{v1}$, $\Delta\sigma_{v2}$, $\Delta\sigma_{H2}$, and $l_2$ are as determined from Figures 3.11.6.3-1 and 3.11.6.3-2, and $F_p$ results from $P_{V2}$ (i.e., $K\Delta\sigma_{v2}$ from Figure 3.11.6.3-1. $H$ is the total wall height at the face.

**Figure 11.10.10.1-1 Superposition of Concentrated Dead Loads for External and Internal Stability Evaluation.**

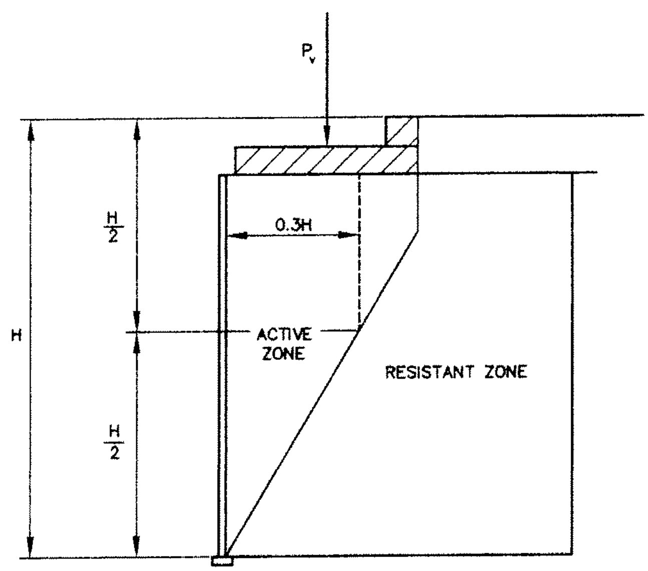



**Figure 11.10.10.1-2 Location of Maximum Tensile Force Line in Case of Large Surcharge Slabs (Inextensible Reinforcements).**

### 11.10.10.2 Traffic Loads and Barriers

Traffic loads shall be treated as uniform surcharge loads in accordance with the criteria outlined in Article 3.11.6.2. The live load surcharge pressure shall not be less than 2.0 ft. of earth. Parapets and traffic barriers, constructed over or in line with the front face of the wall, shall be designed to resist overturning moments by their own mass. Base slabs shall not have any transverse joints, except construction joints, and adjacent slabs shall be joined by shear dowels. The upper layer(s) of soil reinforcements shall have sufficient tensile capacity to resist a concentrated horizontal load of $\gamma P_H$ where $P_H = 10$ kips distributed over a barrier length of 5.0 ft. This force distribution accounts for the local peak force in the soil reinforcements in the vicinity of the concentrated load. This distributed force would be equal to $\gamma P_{H1}$ where $P_{H1} = 2.0$ kips/ft. and is applied as shown in Figure 3.11.6.3-2a. $\gamma P_{H1}$ would be distributed to the reinforcements assuming $b_f$ equal to the width of the base slab. Adequate space shall be provided laterally between the back of the facing panels and the traffic barrier/slab to allow the traffic barrier and slab to resist the impact load in sliding and overturning without directly transmitting load to the top facing units.

For checking pullout safety of the reinforcements, the lateral traffic impact load shall be distributed to the upper soil reinforcement using Figure 3.11.6.3-2a, assuming $b_f$ equal to the width of the base slab. The full-length of reinforcements shall be considered effective in resisting pullout due to the impact load. The upper layer(s) of soil reinforcement shall have sufficient pullout capacity to resist a horizontal load of $\gamma P_{H1}$ where $P_{H1} = 10.0$ kips distributed over a 20.0 ft. base slab length.

Due to the transient nature of traffic barrier impact loads, when designing for reinforcement rupture, the geosynthetic reinforcement must be designed to resist the static and transient (impact) components of the load as follows:

For the static component, see Eq. 11.10.7.2-3.

For the transient components,

$$\Delta\sigma_H S_v \le \frac{\phi S_{rt} R_c}{RF_{ID} RF_D} \qquad (11.10.10.2\text{-}1)$$

where:

$\Delta\sigma_H$ = traffic barrier impact stress applied over reinforcement tributary area per Article 11.10.10.1 (ksf)

$S_v$ = vertical spacing of reinforcement (ft.)

$S_{rt}$ = ultimate reinforcement tensile resistance required to resist dynamic load component (kips/ft.)

### C11.10.10.2

The force distribution for pullout calculations is different than that used for tensile calculations because the entire base slab must move laterally to initiate a pullout failure due to the relatively large deformation required.

Refer to C11.10.7.2 which applies to transient loads, such as impact loads on traffic barriers, as well as earthquake loads.

$R_c$ = reinforcement coverage ratio from Article 11.10.6.4.1 (dim.)

$RF_{ID}$ = strength reduction factor to account for installation damage to reinforcement from Article 11.10.6.4.3b (dim.)

$RF_D$ = strength reduction factor to prevent rupture of reinforcement due to chemical and biological degradation from Article 11.10.6.4.3b (dim.)

The reinforcement strength required for the static load component must be added to the reinforcement strength required for the transient load component to determine the required total ultimate strength using Eq. 11.10.7.3-3.

Parapets and traffic barriers shall satisfy crash testing requirements as specified in Section 13. The anchoring slab shall be strong enough to resist the ultimate strength of the standard parapet.

Flexible post and beam barriers, when used, shall be placed at a minimum distance of 3.0 ft. from the wall face, driven 5.0 ft. 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 as described in Article 11.10.10.4. The upper two rows of reinforcement shall be designed for an additional horizontal load $\gamma P_{HI}$, where $P_{HI}$ = 300 lbs. per linear ft. of wall, 50 percent of which is distributed to each layer of reinforcement.

### 11.10.10.3 Hydrostatic Pressures

For structures along rivers and streams, a minimum differential hydrostatic pressure equal to 3.0 ft. of water shall be considered for design. This load shall be applied at the high-water level. Effective unit weights shall be used in the calculations for internal and external stability beginning at levels just below the application of the differential hydrostatic pressure.

### C11.10.10.3

Situations where the wall is influenced by tide or river fluctuations may require that the wall be designed for rapid drawdown conditions, which could result in differential hydrostatic pressure considerably greater than 3.0 ft., or alternatively rapidly draining backfill material such as shot rock or open graded coarse gravel can be used as backfill. Backfill material meeting the gradation requirements in the *AASHTO LRFD Bridge Construction Specifications* for MSE structure backfill is not considered to be rapid draining.

### 11.10.10.4 Obstructions in the Reinforced Soil Zone

If the placement of an obstruction in the wall soil reinforcement zone such as a catch basin, grate inlet, signal or sign foundation, guardrail post, or culvert cannot be avoided, the design of the wall near the obstruction shall be modified using one of the following alternatives:

### C11.10.10.4

1) Assuming reinforcement layers must be partially or fully severed in the location of the obstruction, design the surrounding reinforcement layers to carry the additional load which would have been carried by the severed reinforcements.

Field cutting of longitudinal or transverse wires of metal grids, e.g., bar mats, should not be allowed unless one of the alternatives in Article 11.10.10.4 is followed and compensating adjustment is made in the wall design.

2) Place a structural frame around the obstruction capable of carrying the load from the reinforcements in front of the obstruction to reinforcements connected to the structural frame behind the obstruction as illustrated in Figure 1.

3) If the soil reinforcements consist of discrete strips and depending on the size and location of the obstruction, it may be possible to splay the reinforcements around the obstruction.

Typically, the splay of reinforcements is limited to a maximum of 15°.

For Alternative 1, the portion of the wall facing in front of the obstruction shall be made stable against a toppling (overturning) or sliding failure. If this cannot be accomplished, the soil reinforcements between the obstruction and the wall face can be structurally connected to the obstruction such that the wall face does not topple, or the facing elements can be structurally connected to adjacent facing elements to prevent this type of failure.

For the second alternative, the frame and connections shall be designed in accordance with Section 6 for steel frames.

Note that it may be feasible to connect the soil reinforcement directly to the obstruction depending on the reinforcement type and the nature of the obstruction.

For the third alternative, the splay angle, measured from a line perpendicular to the wall face, shall be small enough that the splaying does not generate moment in the reinforcement or the connection of the reinforcement to the wall face. The tensile resistance of the splayed reinforcement shall be reduced by the cosine of the splay angle.

If the obstruction must penetrate through the face of the wall, the wall facing elements shall be designed to fit around the obstruction such that the facing elements are stable, i.e., point loads should be avoided, and such that wall backfill soil cannot spill through the wall face where it joins the obstruction. To this end, a collar next to the wall face around the obstruction may be needed.

If driven piles or drilled shafts must be placed through the reinforced zone, the recommendations provided in Article 11.10.11 shall be followed.

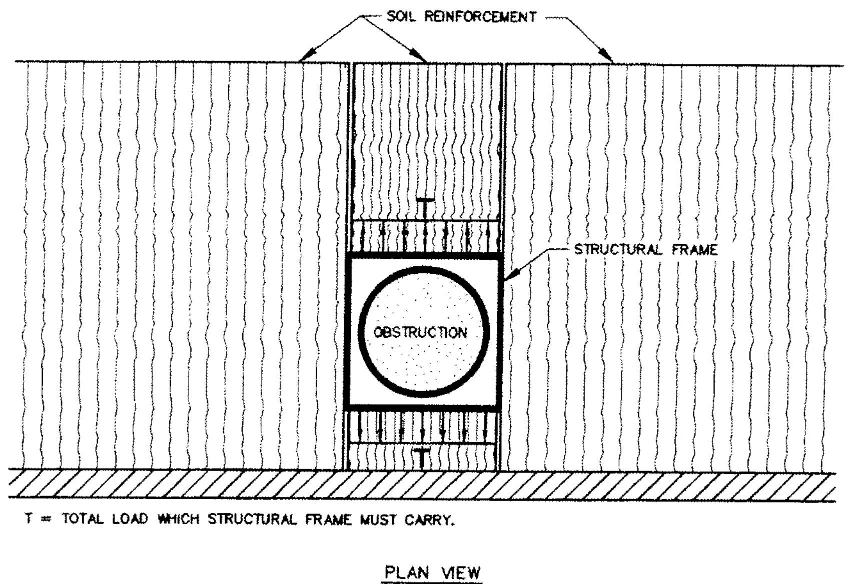



**Figure 11.10.10.4-1 Structural Connection of Soil Reinforcement Around Backfill Obstructions.**

### 11.10.11 MSE Abutments

Abutments on MSE walls shall be proportioned to meet the criteria specified in Article 11.6.2 through 11.6.6.

The MSE wall below the abutment footing shall be designed for the additional loads imposed by the footing pressure and supplemental earth pressures resulting from horizontal loads applied at the bridge seat and from the backwall. The footing load may be distributed as described in Article 11.10.10.1.

The factored horizontal force acting on the reinforcement at any reinforcement level, $T_{max}$, shall be taken as:

$$T_{max} = \sigma_{Hmax} S_v \qquad (11.10.11\text{-}1)$$

where:

$\sigma_{Hmax}$ = factored horizontal stress at layer $i$, as defined by Eq. 2 (ksf)

$S_v$ = vertical spacing of reinforcement (ft.)

Horizontal stresses in abutment reinforced zones shall be determined by superposition as follows, and as specified in Article 11.10.10.1:

$$\sigma_{Hmax} = \gamma_p(\sigma_v k_r + \Delta\sigma_v k_r + \Delta\sigma_H) \qquad (11.10.11\text{-}2)$$

where:

$\gamma_p$ = load factor for vertical earth pressure in Table 3.4.1-2

### C11.10.11

$\Delta\sigma_H$ = magnitude of lateral pressure due to surcharge (ksf)

$\sigma_v$ = vertical soil stress over effective base width $(B-2e)$ (ksf)

$\Delta\sigma_v$ = vertical soil stress due to footing load (ksf)

$k_r$ = earth pressure coefficient varying as a function of $k_a$ as specified in Article 11.10.6.2.1

$k_a$ = active earth pressure coefficient specified in Article 3.11.5.8

The effective length used for calculations of internal stability under the abutment footing shall be as described in Article 11.10.10.1 and Figure 11.10.10.1-2.

The minimum distance from the centerline of the bearing on the abutment to the outer edge of the facing shall be 3.5 ft. The minimum distance between the back face of the panel and the footing shall be 6.0 in.

Where significant frost penetration is anticipated, the abutment footing shall be placed on a bed of compacted coarse aggregate 3.0 ft. thick as described in Article 11.10.2.2.

The density, length, and cross-section of the soil reinforcements designed for support of the abutment shall be carried on the wingwalls for a minimum horizontal distance equal to 50 percent of the height of the abutment.

In pile or drilled shaft supported abutments, the horizontal forces transmitted to the deep foundation elements shall be resisted by the lateral capacity of the deep foundation elements by provision of additional reinforcements to tie the drilled shaft or pile cap into the soil mass, or by batter piles. Lateral loads transmitted from the deep foundation elements to the reinforced backfill may be determined using a P-Y lateral load analysis technique. The facing shall be isolated from horizontal loads associated with lateral pile or drilled shaft deflections. A minimum clear distance of 1.5 ft. shall be provided between the facing and deep foundation elements. Piles or drilled shafts shall be specified to be placed prior to wall construction and cased through the fill if necessary.

The equilibrium of the system should be checked at each level of reinforcement below the bridge seat.

Due to the relatively high bearing pressures near the panel connections, the adequacy and ultimate capacity of panel connections should be determined by conducting pullout and flexural tests on full-sized panels.

The minimum length of reinforcement, based on experience, has been the greater of 22.0 ft. or $0.6\,(H + d) + 6.5$ ft. The length of reinforcement should be constant throughout the height to limit differential settlements across the reinforced zone. Differential settlements could overstress the reinforcements.

The permissible level of differential settlement at abutment structures should preclude damage to superstructure units. This subject is discussed in Article 10.6.2.2. In general, abutments should not be constructed on mechanically stabilized embankments if anticipated differential settlements between abutments or between piers and abutments are greater than one-half the limiting differential settlements described in Article C10.5.2.2.

Moments should be taken at each level under consideration about the centerline of the reinforced mass to determine the eccentricity of load at each level. A uniform vertical stress is then calculated using a fictitious width taken as $(B-2e)$, and the corresponding horizontal stress should be computed by multiplying by the appropriate coefficient of lateral earth pressure.

## 11.11 PREFABRICATED MODULAR WALLS

### 11.11.1 General

Prefabricated modular systems may be considered where conventional gravity, cantilever or counterfort concrete retaining walls are considered.

### C11.11.1

Prefabricated modular wall systems, whose elements may be proprietary, generally employ interlocking soil-filled reinforced concrete or steel modules or bins, rock filled gabion baskets, precast concrete units, or dry cast segmental masonry concrete units (without soil reinforcement) which resist earth pressures by acting as gravity retaining walls. Prefabricated modular walls may also use their structural elements to mobilize the dead weight of a portion of the wall backfill through soil arching to provide resistance to lateral loads. Typical prefabricated modular walls are shown in Figure C1.

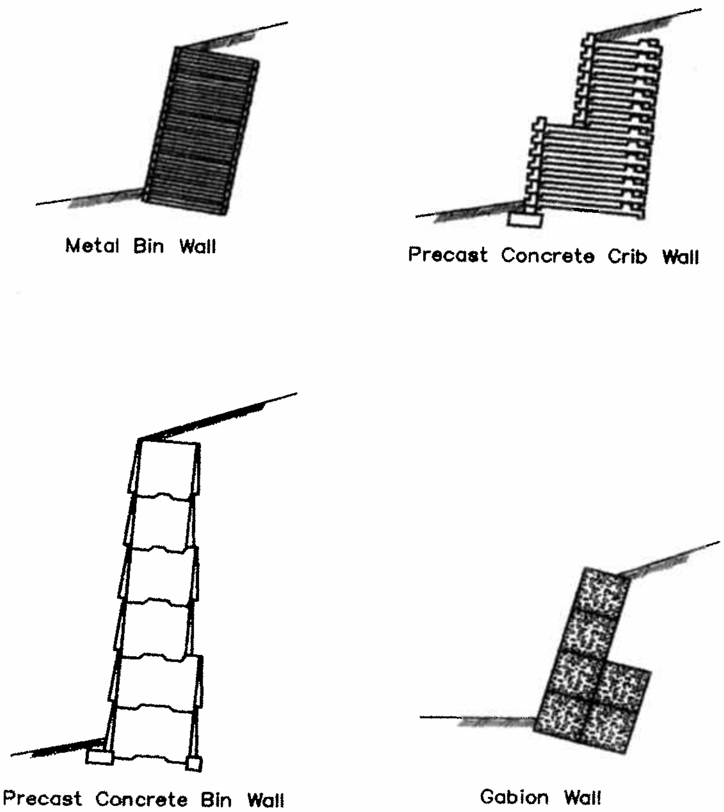



**Figure C11.11.1-1 Typical Prefabricated Modular Gravity Walls.**

Prefabricated modular wall systems shall not be used under the following conditions:

- On curves with a radius of less than 800 ft., unless the curve can be substituted by a series of chords.

- Steel modular systems shall not be used where the groundwater or surface runoff is acid contaminated or where deicing spray is anticipated.

### 11.11.2 Loading

The provisions of Articles 11.6.1.2 and 3.11.5.9 shall apply, except that shrinkage and temperature effects need not be considered.

### 11.11.3 Movement at the Service Limit State

The provisions of Article 11.6.2 shall apply as applicable.

### C11.11.3

Calculated longitudinal differential settlements along the face of the wall should result in a slope less than 1/200.

### 11.11.4 Safety Against Soil Failure

#### 11.11.4.1 General

For sliding and overturning stability, the system shall be assumed to act as a rigid body. Determination of stability shall be made at every module level.

Passive pressures shall be neglected in stability computations, unless the base of the wall extends below the depth of maximum scour, freeze-thaw, or other disturbance. For these cases only, the embedment below the greater of these depths may be considered effective in providing passive resistance.

#### 11.11.4.2 Sliding

The provisions of Article 10.6.3.3 shall apply.

Computations for sliding stability may consider that the friction between the soil-fill and the foundation soil, and the friction between the bottom modules or footing and the foundation soil are effective in resisting sliding. The coefficient of sliding friction between the soil-fill and foundation soil at the wall base shall be the lesser of $\phi_f$ of the soil fill and $\phi_f$ of the foundation soil. The coefficient of sliding friction between the bottom modules or footing and the foundation soil at the wall base shall be reduced, as necessary, to account for any smooth contact areas.

In the absence of specific data, a maximum friction angle of 30° shall be used for $\phi_f$ for granular soils. Tests should be performed to determine the friction angle of cohesive soils considering both drained and undrained conditions.

#### 11.11.4.3 Bearing Resistance

The provisions of Article 10.6.3 shall apply.

Bearing resistance shall be computed by assuming that dead loads and earth pressure loads are resisted by point supports per unit length at the rear and front of the modules or at the location of the bottom legs. A minimum of 80 percent of the soil weight inside the modules shall be considered to be transferred to the front and rear support points. If foundation conditions require a footing under the total area of the module, all of the soil weight inside the modules shall be considered.

#### C11.11.4.3

Concrete modular systems are relatively rigid and are subject to structural damage due to differential settlements, especially in the longitudinal direction. Therefore, bearing resistance for footing design should be determined as specified in Section 10.6.

#### 11.11.4.4 Overturning

The provisions of Article 11.6.3.3 shall apply.

A maximum of 80 percent of the soil-fill inside the modules is effective in resisting overturning moments.

#### C11.11.4.4

The entire volume of soil within the module cannot be counted on to resist overturning, as some soil will not arch within the module. If a structural bottom is provided to retain the soil within the module, no reduction of the soil weight to compute overturning resistance is warranted.

#### 11.11.4.5 Subsurface Erosion

Bin walls may be used in scour-sensitive areas only where their suitability has been established. The provisions of Article 11.6.3.5 shall apply.

#### 11.11.4.6 Overall Stability

The provisions of Article 11.6.2.3 shall apply.

#### 11.11.4.7 Passive Resistance and Sliding

The provisions of Articles 10.6.3.3 and 11.6.3.6 shall apply, as applicable.

### 11.11.5 Safety Against Structural Failure

#### 11.11.5.1 Module Members

Prefabricated modular units shall be designed for the factored earth pressures behind the wall and for factored pressures developed inside the modules. Rear face surfaces shall be designed for both the factored earth pressures developed inside the modules during construction and the difference between the factored earth pressures behind and inside the modules after construction. Strength and reinforcement requirements for concrete modules shall be in accordance with Section 5.

Strength requirements for steel modules shall be in accordance with Section 6. The net section used for design shall be reduced in accordance with Article 11.10.6.4.2a.

Factored bin pressures shall be the same for each module and shall not be less than:

$$P_b = \gamma \gamma_s b \qquad (11.11.5.1\text{-}1)$$

where:

$P_b$ = factored pressure inside bin module (ksf)

$\gamma_s$ = soil unit weight (kcf)

$\gamma$ = load factor for vertical earth pressure specified in Table 3.4.1-2

#### C11.11.5.1

Structural design of module members is based on the difference between pressures developed inside the modules (bin pressures) and those resulting from the thrust of the backfill. The recommended bin pressure relationships are based on relationships obtained for long trench geometry, and are generally conservative.

$b$ = width of bin module (ft.)

Steel reinforcing shall be symmetrical on both faces unless positive identification of each face can be ensured to preclude reversal of units. Corners shall be adequately reinforced.

### 11.11.6 Seismic Design

The provisions of Article 11.6.5 shall apply.

### 11.11.7 Abutments

Abutment seats constructed on modular units shall be designed by considering earth pressures and supplemental horizontal pressures from the abutment seat beam and earth pressures on the backwall. The top module shall be proportioned to be stable under the combined actions of normal and supplementary earth pressures. The minimum width of the top module shall be 6.0 ft. The centerline of bearing shall be located a minimum of 2.0 ft. from the outside face of the top precast module. The abutment beam seat shall be supported by, and cast integrally with, the top module. The front face thickness of the top module shall be designed for bending forces developed by supplemental earth pressures. Abutment beam-seat loadings shall be carried to foundation level and shall be considered in the design of footings.

Differential settlement provisions, specified in Article 11.10.4, shall apply.

### 11.11.8 Drainage

In cut and side-hill fill areas, prefabricated modular units shall be designed with a continuous subsurface drain placed at, or near, the footing grade and outletted as required. In cut and side-hill fill areas with established or potential groundwater levels above the footing grade, a continuous drainage blanket shall be provided and connected to the longitudinal drain system.

For systems with open front faces, a surface drainage system shall be provided above the top of the wall.

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# APPENDIX A11 SEISMIC DESIGN OF ABUTMENTS AND GRAVITY RETAINING STRUCTURES

## A11.1 GENERAL

The numerous case histories of damage to or failure of bridges induced by abutment failure or displacement during earthquakes have clearly demonstrated the need for careful attention to abutment design and detailing in seismic areas. Damage is typically associated with fill settlement or slumping, displacements induced by high seismically caused lateral earth pressures, or the transfer of high longitudinal or transverse inertia forces from the bridge structure itself. Settlement of abutment backfill, severe abutment damage, or bridge deck damage induced by the movement of abutments may cause loss of bridge access; hence abutments must be considered a vital link in the overall seismic design process for bridges.

The nature of abutment movement or damage during past earthquakes has been well documented in the literature. Evans (*1971*) examined the abutments of 39 bridges within 30 miles (48.3 km) of the 1968 M7 Inangahua earthquake in New Zealand, of which 23 showed measurable movement and 15 were damaged. Movements of free-standing abutments followed the general pattern of outward motion and rotation about the top after contact with and restraint by the superstructures. Fill settlements were observed to be 10 to 15 percent of the fill height. Damage effects on bridge abutments in the M7.1 Madang earthquake in New Guinea, reported by Ellison (*1971*), were similar; abutment movement, as much as 20.0 in. (500 mm), was noted. Damage to abutments in the 1971 San Fernando earthquake is described by Fung et al. (*1971*). Numerous instances of abutment displacement and associated damage have been reported in publications on Niigata and Alaskan earthquakes. However, these failures were primarily associated with liquefaction of foundation soils.

Design features of abutments vary tremendously and depend on the nature of the bridge site, foundation soils, bridge span length, and load magnitudes. Abutment types include free-standing gravity walls, cantilever walls, tied back walls, and monolithic diaphragms. Foundation support may use spread footings, vertical piles, or battered piles, whereas connection details to the superstructure may incorporate roller supports, elastomeric bearings, or fixed bolted connections. Considering the number of potential design variables, together with the complex nature of soil abutment superstructure interaction during earthquakes, it is clear that the seismic design of abutments necessitates many simplifying assumptions.

### A11.1.1 Free-Standing Abutments

For free-standing abutments, such as gravity or cantilever walls, which are able to yield laterally during an earthquake, i.e., superstructure supported by bearings that are able to slide freely, the well-established Mononobe-Okabe pseudo-static approach, outlined below, is widely used to compute earth pressures induced by earthquakes.

For free-standing abutments in highly seismic areas, design of abutments to provide zero displacement under peak ground accelerations may be unrealistic, and design for an acceptable small lateral displacement may be preferable. A recently developed method for computing the magnitude of relative wall displacement during a given earthquake is outlined in this Article. On the basis of this simplified approach, recommendations are made for the selection of a pseudo-static seismic coefficient and the corresponding displacement level for a given effective peak ground acceleration.

#### A11.1.1.1 Mononobe-Okabe Analysis

The method most frequently used for the calculation of the seismic soil forces acting on a bridge abutment is a static approach developed in the 1920s by Mononobe (*1929*) and Okabe (*1926*). The Mononobe-Okabe analysis is an extension of the Coulomb sliding-wedge theory, taking into account horizontal and vertical inertia forces acting on the soil. The analysis is described in detail by Seed and Whitman (*1970*) and Richards and Elms (*1979*). The following assumptions are made:

1. The abutment is free to yield sufficiently to enable full soil strength or active pressure conditions to be mobilized. If the abutment is rigidly fixed and unable to move, the soil forces will be much higher than those predicted by the Mononobe-Okabe analysis.

2. The backfill is cohesionless, with a friction angle of $\phi$.

3. The backfill is unsaturated, so that liquefaction problems will not arise.

Equilibrium considerations of the soil wedge behind the abutment, as shown in Figure 1, then lead to a value, $E_{AE}$, of the active force exerted on the soil mass by the abutment and vice versa. When the abutment is at the point of failure $E_{AE}$ is given by the expression:

$$E_{AE} = \frac{1}{2}\gamma H^2 (1-k_v) K_{AE} \tag{A11.1.1.1-1}$$

where the seismic active pressure coefficient $K_{AE}$ is

$$K_{AE} = \frac{\cos^2(\phi-\theta-\beta)}{\cos\theta\cos^2\beta\cos(\delta+\beta+\theta)} \times \left[1 + \sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta-i)}{\cos(\delta+\beta+\theta)\cos(i-\beta)}}\right]^{-2} \tag{A11.1.1.1-2}$$

and where

$\gamma$ = unit weight of soil (kcf)

$H$ = height of soil face (ft.)

$\phi$ = angle of friction of soil (°)

$\theta$ = arc tan $(k_h/(1-k_v))$ (°)

$\delta$ = angle of friction between soil and abutment (°)

$k_h$ = horizontal acceleration coefficient (dim.)

$k_v$ = vertical acceleration coefficient (dim.)

$i$ = backfill slope angle (°)

$\beta$ = slope of wall to the vertical, negative as shown (°)

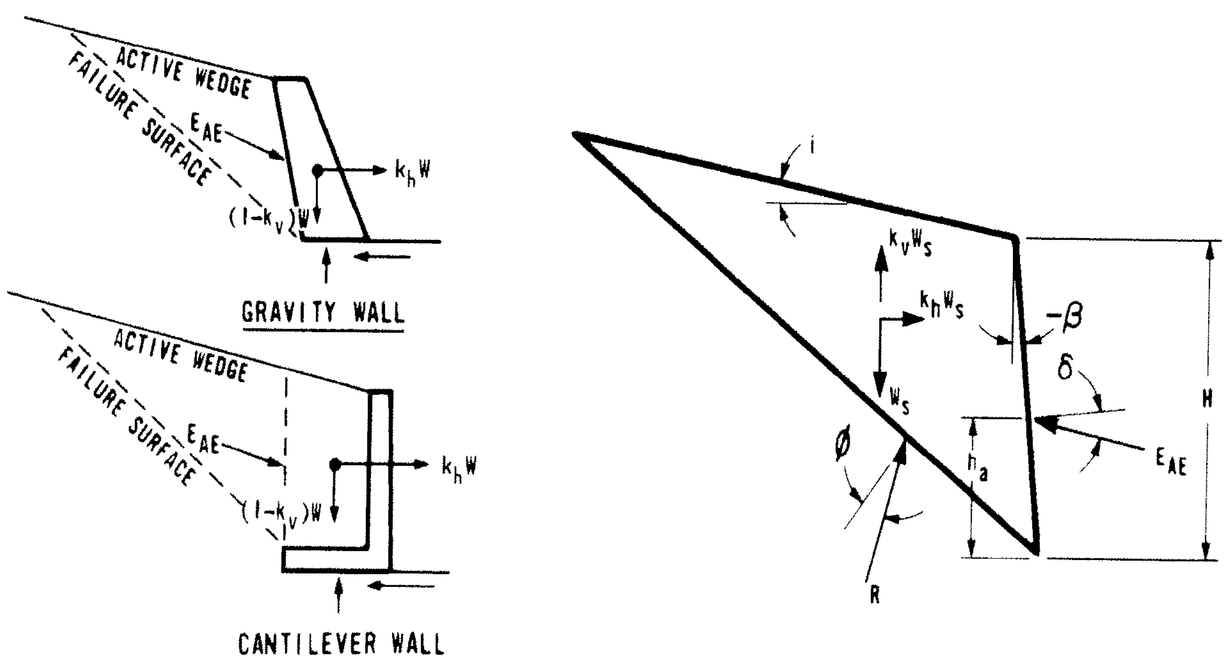


**Figure A11.1.1.1-1 Active Wedge Force Diagram.**

The equivalent expression for passive force if the abutment is being pushed into the backfill is:

$$E_{PE} = \frac{1}{2}\gamma H^2 (1-k_v) K_{PE} \tag{A11.1.1.1-3}$$

where:

$$K_{PE} = \frac{\cos^2(\phi-\theta+\beta)}{\cos\theta\cos^2\beta\cos(\delta-\beta+\theta)} \times \left[1-\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta+i)}{\cos(\delta-\beta+\theta)\cos(i-\beta)}}\right]^{-2} \tag{A11.1.1.1-4}$$

As the seismic inertial angle $\theta$ increases, the values of $K_{AE}$ and $K_{PE}$ approach each other and, for vertical backfill, become equal when $\theta = \phi$.

Despite the relative simplicity of the approach, the accuracy of Eq. 1 has been substantiated by model tests (*Seed and Whitman 1970*), and back calculation from observed failures of flood channels walls (*Clough and Fragaszy 1977*). In the latter case, however, the displacements were large; and this, as will be seen, can modify the effective values of $k_h$ at which failure occurs.

The value of $h_a$, the height at which the resultant of the soil pressure acts on the abutment, may be taken as $H/3$ for the static case with no earthquake effects involved. However, it becomes greater as earthquake effects increase. This has been shown empirically by tests and theoretically by Wood (*1973*), who found that the resultant of the dynamic pressure acted approximately at midheight. Seed and Whitman have suggested that $h$ could be obtained by assuming that the static component of the soil force (computed from Eq. 1 with $\theta = k_v = 0$) acts at $H/3$ from the bottom of the abutment, whereas the additional dynamic effect should be taken to act at a height of $0.6H$. For most purposes, it is sufficient to assume $h = H/2$ with a uniformly distributed pressure.

Although the Mononobe-Okabe expression for active thrust is easily evaluated for any particular geometry and friction angle, the significance of the various parameters is not obvious. Figure 2 shows the variation of $K_{AE}$ against $k_h$ for different values of $\phi$ and $k_v$; obviously $K_{AE}$ is very sensitive to the value of $\phi$. Also, for a constant value of $\phi$, $K_{AE}$ doubles as $k_h$ increases from 0.0 to 0.35 for 0.0 vertical acceleration, and thereafter it increases more rapidly.

In order to evaluate the increase in soil active pressure due to earthquake effects more easily, $K_{AE}$ can be normalized by dividing by its static value $K_A$ to give a thrust factor

$$F_T = \frac{K_{AE}}{K_A} \tag{A11.1.1.1-5}$$

Whereas Figure 2 shows that $K_{AE}$ is sensitive to changes in the soil friction angle, the plots of $F_T$ against $\phi$ in Figure 3 indicate that the value of $\phi$ has little effect on the thrust factor until quite suddenly, over a short range of $\phi$, $F_T$ increases rapidly and becomes infinite for specific critical values of $\phi$. The reason for this behavior may be determined by examining Eq. 2. The contents of the radical must be positive for a real solution to be possible, and, for this, it is necessary that:

$$\phi \geq i + \theta = i + \arctan\left(\frac{k_h}{1-k_v}\right) \tag{A11.1.1.1-6}$$

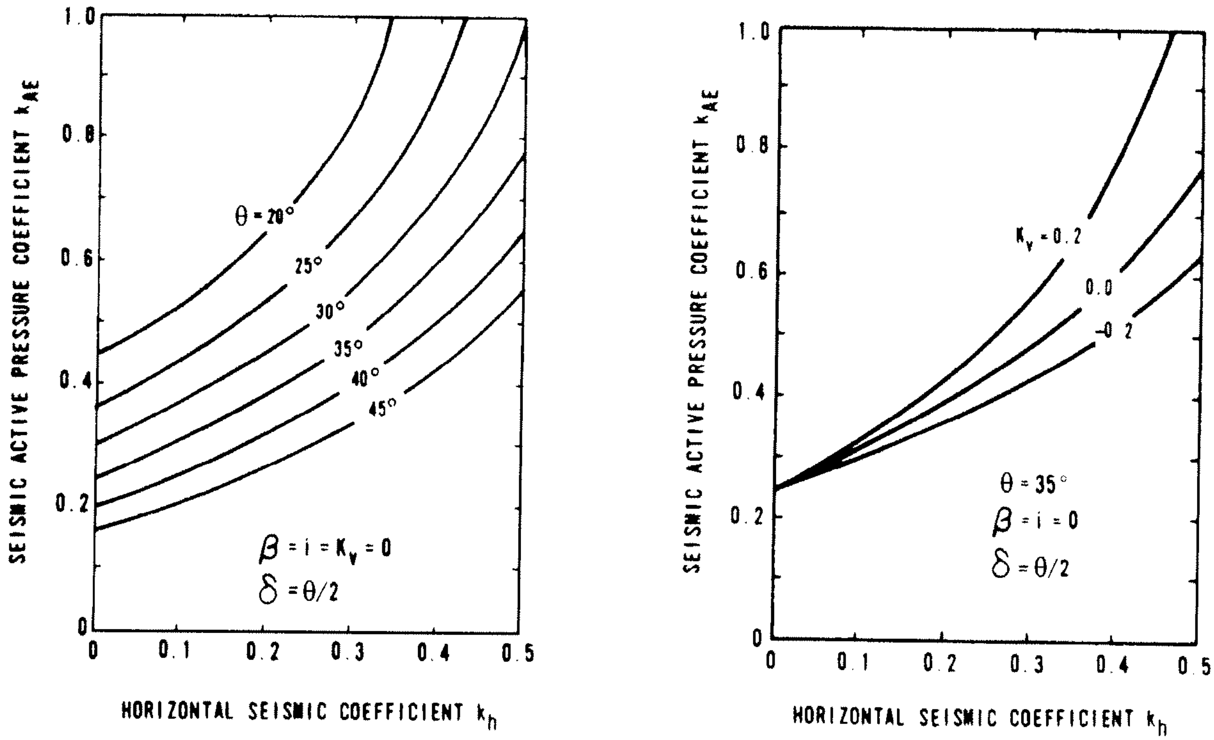


**Figure A11.1.1.1-2 Effects of Seismic Coefficient and Soil Friction Angle on Seismic Active Pressure Coefficient.**

This condition could also be thought of as specifying a limit to the horizontal acceleration coefficient that could be sustained by any structure in a given soil. The limiting condition is:

$$k_h \le (1 - k_v)\tan(\phi - i) \qquad \text{(A11.1.1.1-7)}$$

For 0.0 vertical acceleration and backfill angle and for a soil friction angle of 35°, the limiting value of $k_h$ is 0.7. This is a figure of some interest in that it provides an absolute upper-bound for the seismic acceleration that can be transmitted to any structure whatsoever built on soil with the given strength characteristics.

Figure 4 shows the effect on $F_T$ of changes in the vertical acceleration coefficient $k_v$. Positive values of $k_v$ have a significant effect for values of $k_h$ greater than 0.2. The effect is greater than ten percent above and to the right of the dashed line. As is to be expected from Eq. 4, $K_{AE}$ and $F_T$ are also sensitive to variations in backfill slope, particularly for higher values of horizontal acceleration coefficient when the limit, implied by Eq. 4, is approached. This effect is shown in Figure 5.

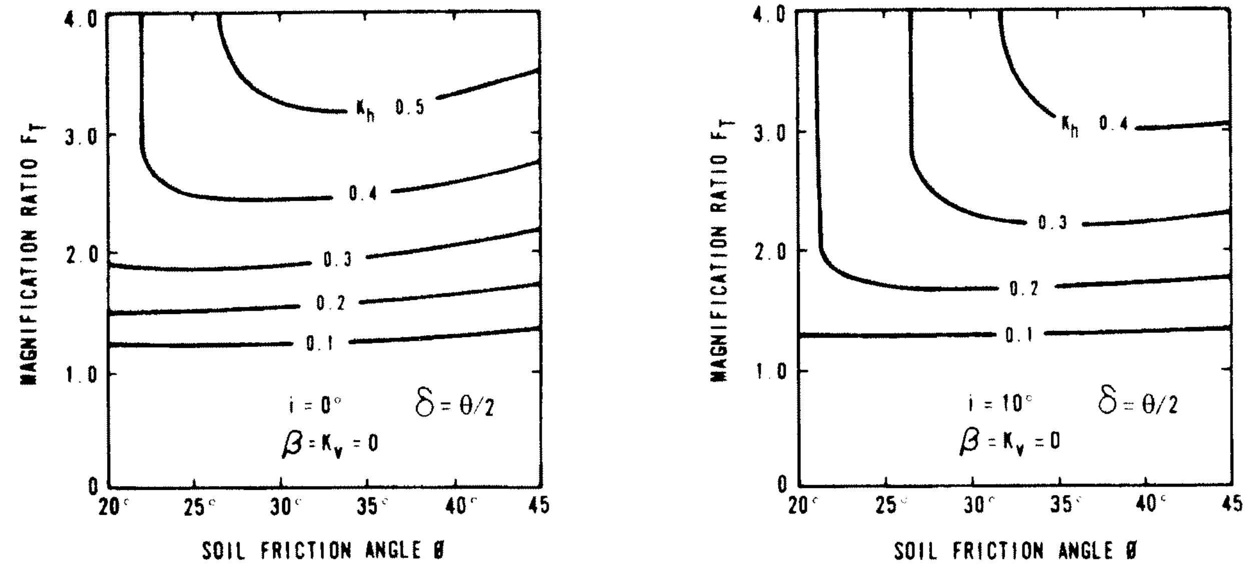


**Figure A11.1.1.1-3 Influence of Soil Friction Angle on Magnification Ratio.**

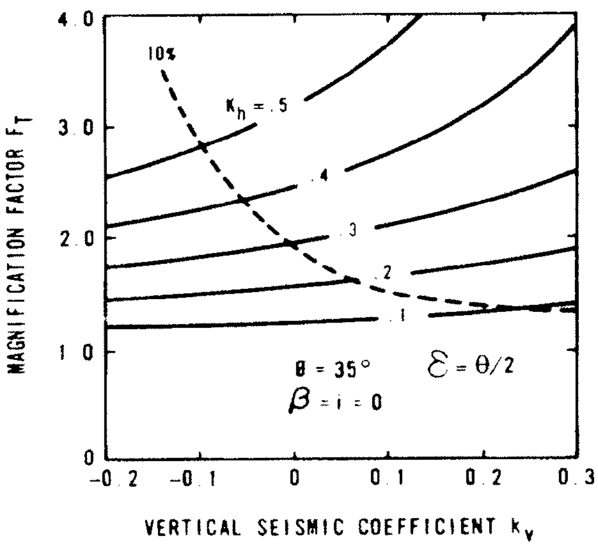


**Figure A11.1.1.1-4 Influence of Vertical Seismic Coefficient on Magnification Ratio.**

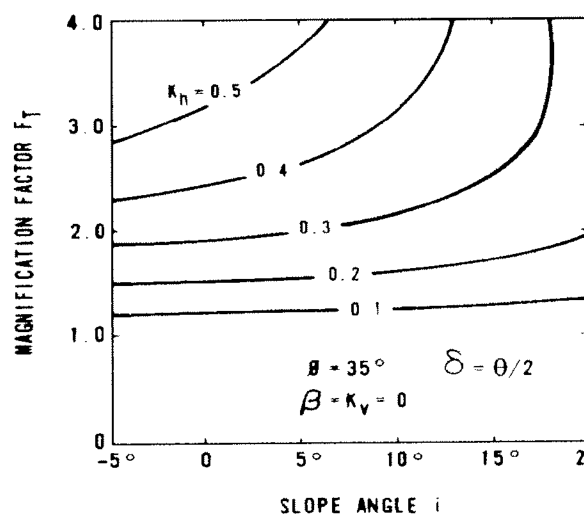


**Figure A11.1.1.1-5 Influence of Backfill slope Angle on Magnification Ratio.**

The effects of abutment inertia are not taken into account in the Mononobe-Okabe analysis. Many current procedures assume that the inertia forces due to the mass of the abutment itself may be neglected in considering seismic behavior and seismic design. This is not a conservative assumption, and for those abutments relying on their mass for stability, it is also an unreasonable assumption in that to neglect the mass is to neglect a major aspect of their behavior. The effects of wall inertia are discussed further by Richards and Elms (*1979*), who show that wall inertia forces should not be neglected in the design of gravity-retaining walls.

### A11.1.1.2 Design For Displacement

If peak ground accelerations are used in the Mononobe-Okabe analysis method, the size of gravity-retaining structures will often be excessively great. To provide a more economic structure, design for a small tolerable displacement, instead of no displacement, may be preferable.

Tests have shown that a gravity-retaining wall fails in an incremental manner in an earthquake. For any earthquake ground motion, the total relative displacement may be calculated using the sliding block method suggested by Newmark (*1965*). The method assumes a displacement pattern similar to that of a block resting on a plane rough horizontal surface subjected to an earthquake, with the block being free to move against frictional resistance in one direction only. Figure 1 shows how the relative displacement relates to the acceleration and velocity time histories of soil and wall. At a critical value of $k_h$, the wall is assumed to begin sliding; relative motion will continue until wall and soil velocities are equal. Figures 2 and 3 show the results by Richards and Elms (*1979*) of a computation of wall displacement for $k_h = 0.1$ for the El Centro 1940 N-S record.

Newmark computed the maximum displacement response for four earthquake records and plotted the results after scaling the earthquakes to a common maximum acceleration and velocity. Franklin and Chang (*1977*) repeated the analysis for a large number of both natural and synthetic records and added their results to the same plot. Upper-bound envelopes for their results are shown in Figure 4. All records were scaled to a maximum acceleration coefficient of 0.5 and a maximum velocity, $V$, of 30.0 in./sec. The maximum resistance of coefficient $N$ is the maximum acceleration coefficient sustainable by a sliding block before it slides. In the case of a wall designed using the Mononobe-Okabe method, the maximum coefficient is, of course, $k_h$.

Figure 4 shows that the displacement envelopes for all the scaled records have roughly the same shape.

An approximation of the curves for relatively low displacements is given by the relations, expressed in any consistent set of units,

$$d = 0.087\frac{V^2}{Ag}\left(\frac{N}{A}\right)^{-4} \tag{A11.1.1.2-1}$$

where $d$ is the total relative displacement of a wall subjected to an earthquake ground motion whose maximum acceleration coefficient and maximum velocity are $A$ and $V$, respectively. This displacement is drawn as a straight line on Figure 4. Because this expression has been derived from envelope curves, it will overestimate $d$ for most earthquakes.

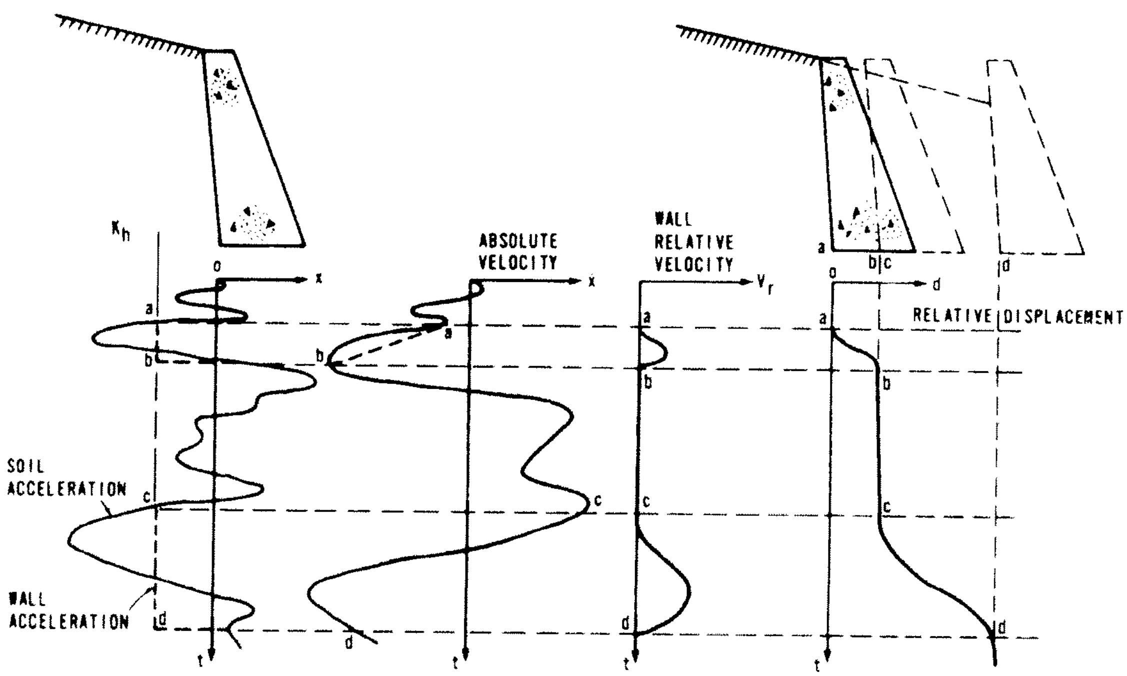


**Figure A11.1.1.2-1 Relation between Relative Displacement and Acceleration and Velocity Time Histories of Soil and Wall.**

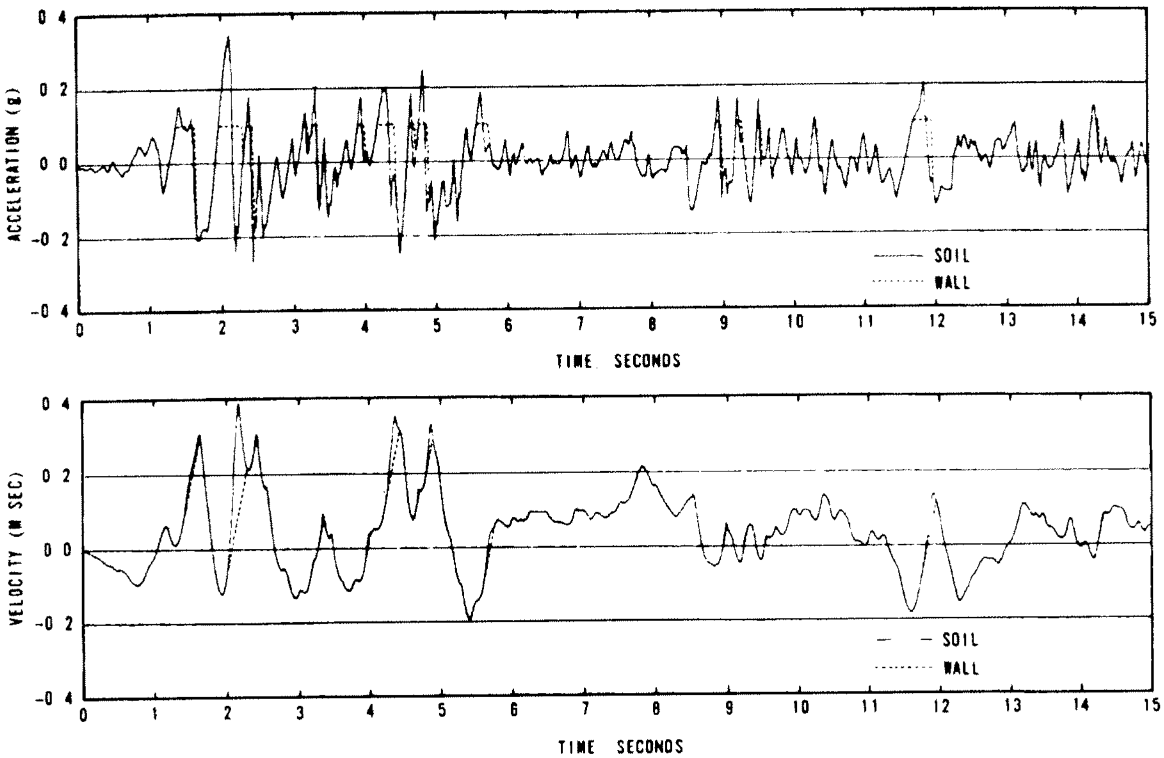


Figure A11.1.1.2-2 Acceleration and Velocity Time Histories of Soil and Wall (*El Centro 1940 N-S Record*).

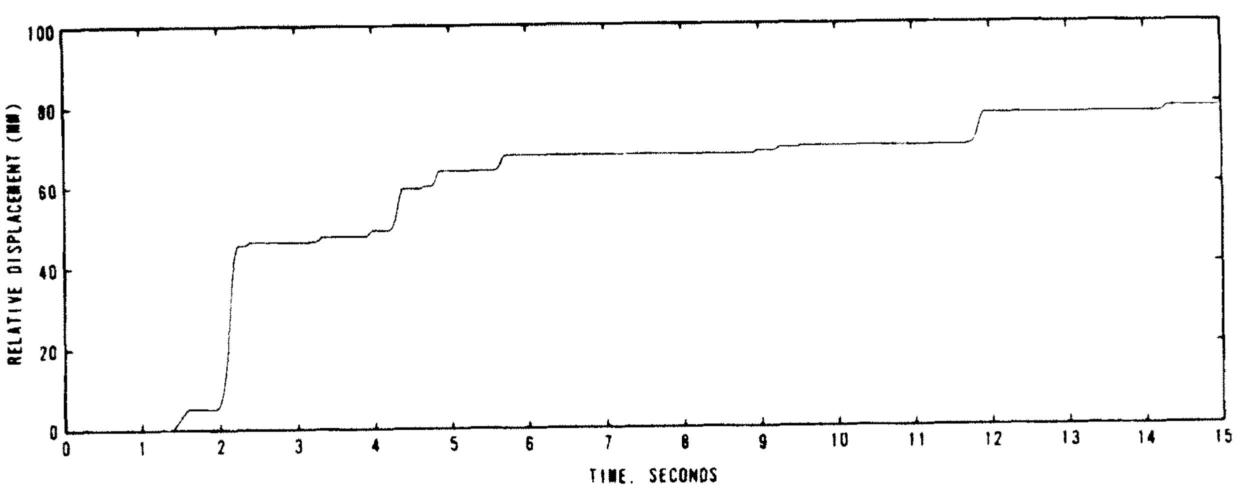


Figure A11.1.1.2-3 Relative Displacement of Wall (*El Centro 1940 N-S Record*).

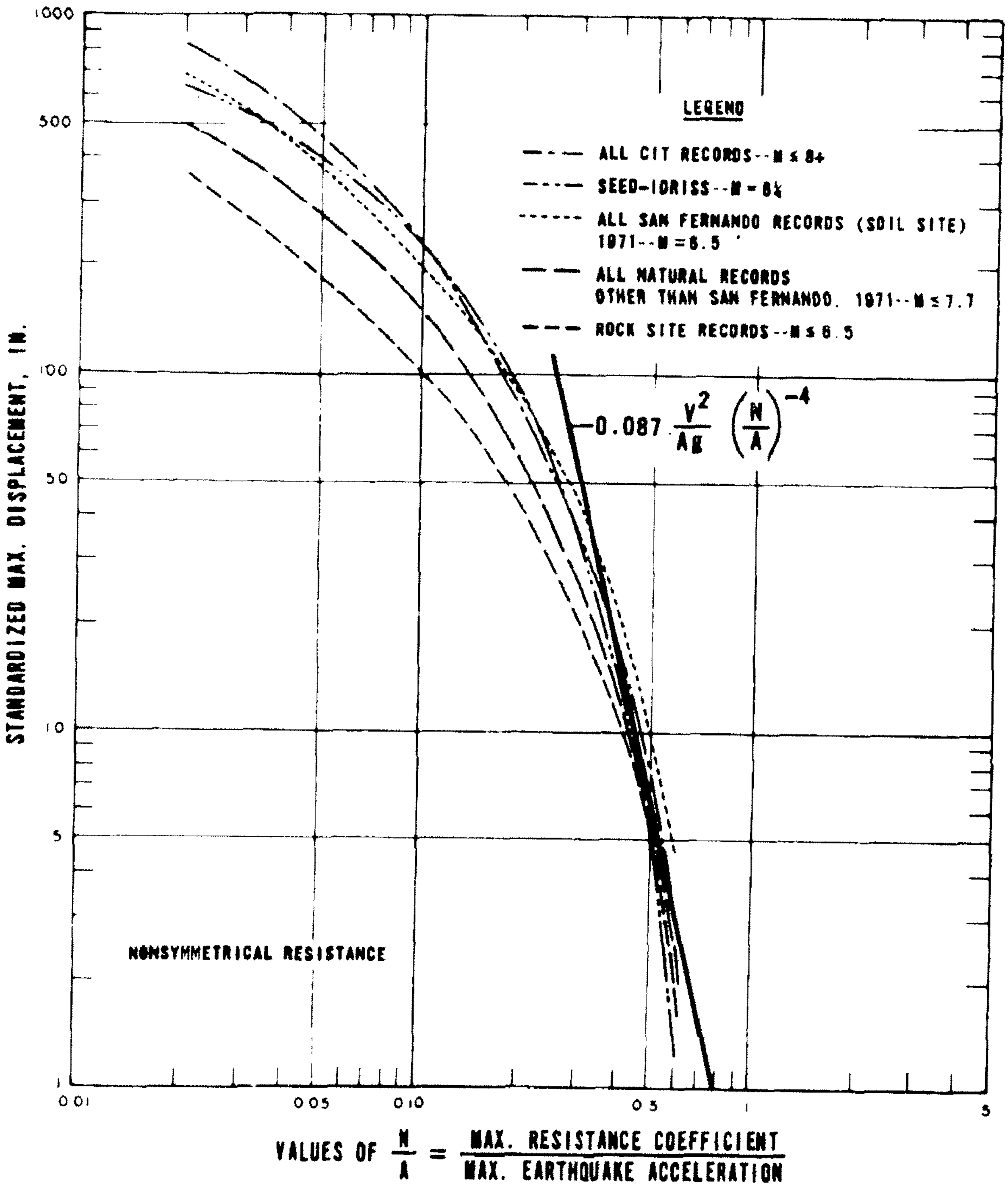


**Figure A11.1.1.2-4 Upper-Bound Envelope Curves of Permanent Displacements for all Natural and Synthetic Records Analyzed by Franklin and Chang (*1977*).**

One possible design procedure would be to choose a desired value of maximum wall displacement, $d$, together with appropriate earthquake parameters and to use Eq. 1 to derive a value of the seismic acceleration coefficient for which the wall should be designed. The wall connections, if any, could then be detailed to allow for this displacement.

By applying the above procedure to several simplified examples, Elms and Martin (*1979*) have shown that a value of $k_h = A/2$ is adequate for most design purposes, provided that allowance is made for an outward displacement of the abutment of up to $10A$ in.

For bridges classified as Seismic Zones 3 or 4, more detailed consideration of the mechanism of transfer of structural inertia forces through bridge bearings to free-standing abutments is required, particularly for bridges classified as Seismic Zone 4, where continued bridge accessibility after a major earthquake is required.

For sliding steel bearings or pot bearings, force diagrams describing limiting equilibrium conditions for simple abutments are shown in Figure 5. Where bearings comprise unconfined elastomeric pads, the nature of the forces transferred to the abutment becomes more complex because such bearings are capable of transferring significant force. The magnitude of the force initially depends on the relative movement between the superstructure and the abutment, and force magnitudes can become quite large before slip will occur.

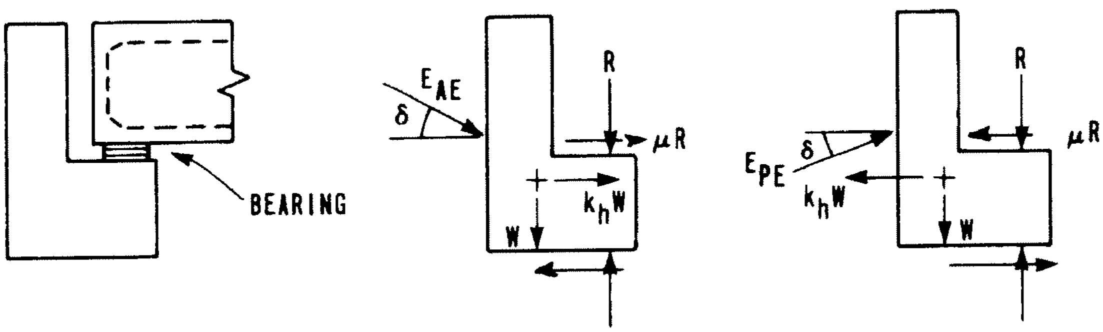


**Figure A11.1.1.2-5 Force Diagrams including Bearing Friction.**

For bridges classified as Seismic Zone 4, additional consideration should be given to the use of linkage bolts and buffers to minimize damage. A typical abutment support detail used by the New Zealand Ministry of Works is shown in Figure 6. It may be seen that linkage bolts are incorporated to prevent spans from dropping off supports. The rubber rings act as buffers to prevent impact damage in the event that the lateral displacement clearance provided is inadequate. The knock-off backwall accommodates differential displacement between the abutment and superstructure, with minimum structural damage. A more typical design provision in United States practice is to seal the gap between superstructure and abutment with bitumen to minimize impact damage. It must be recognized, however, that in this case some damage and possible abutment rotation will occur in strong earthquakes.

In Figures 6 and 7, the use of a settlement or approach slab, which has the effect of providing bridge access in the event of backfill settlement, is also noted. The slab also provides an additional abutment friction anchorage against lateral movement.

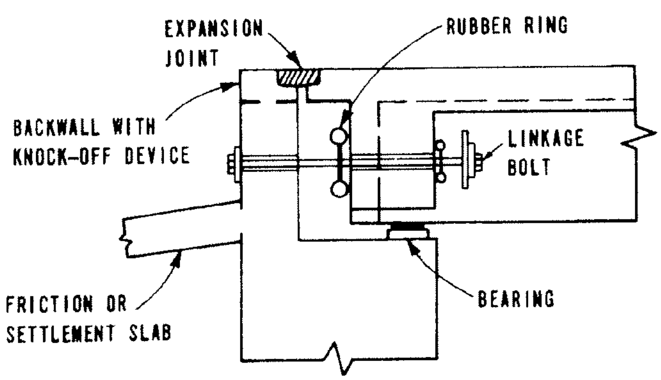


**Figure A11.1.1.2-6 Possible Abutment Details.**

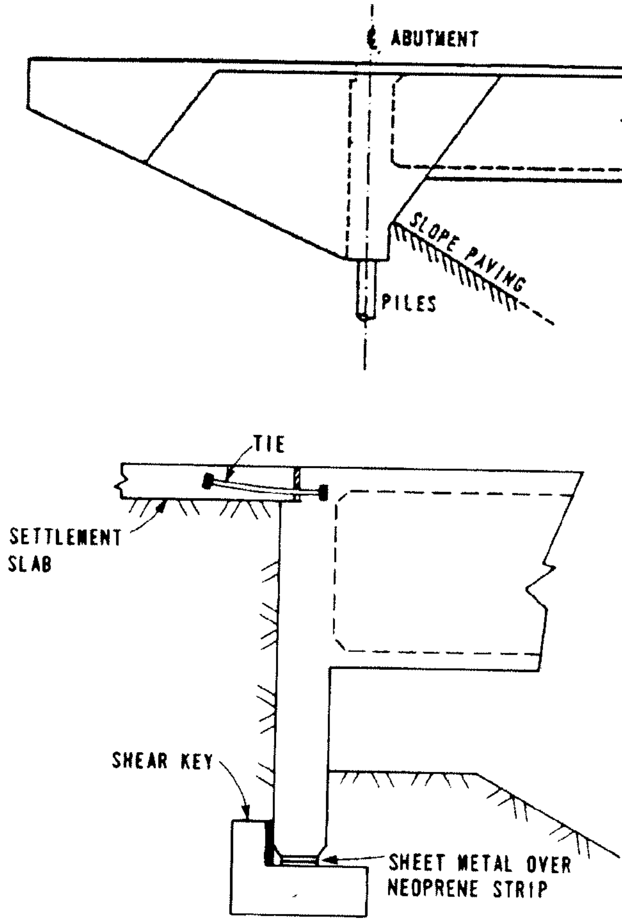


**Figure A11.1.1.2-7 Typical Monolithic Abutment.**

### A11.1.1.3 Nonyielding Abutments

As previously noted, the Mononobe-Okabe analysis assumes that the abutment is free to laterally yield a sufficient amount to mobilize peak soil strengths in the soil backfill. For granular soils, peak strengths can be assumed to be mobilized if deflections at the top of the wall are about 0.5 percent of the abutment height. For abutments restrained against lateral movement by tiebacks or batter piles, lateral pressures induced by inertia forces in the backfill will be greater than those given by a Mononobe-Okabe analysis. Simplified elastic solutions presented by Wood (*1973*) for rigid nonyielding walls also indicate that pressures are greater than those given by Mononobe-Okabe. The use of a factor of 1.5 in conjunction with peak ground accelerations is suggested for design where doubt exists that an abutment can yield sufficiently to mobilize soil strengths.

## A11.1.2 Monolithic Abutments

Monolithic or end diaphragm abutments, such as shown in Figure A11.1.1.2-7, are commonly used for single and for two span bridges in California. As shown, the end diaphragm is cast monolithically with the superstructure and may be directly supported on piles, or provision may be made for beam shortening during post-tensioning. The diaphragm acts as a retaining wall with the superstructure acting as a prop between abutments.

Such abutments have performed well during earthquakes, avoiding problems such as backwall and bearing damage associated with yielding abutments and reducing the lateral load taken by columns or piers. On the other hand, higher longitudinal and transverse superstructure inertia forces are transmitted directly into the backfill, and provision must be made for adequate passive resistance to avoid excessive relative displacements.

Although free-standing or seat-type abutments allow the engineer more control over development of soil forces, the added joint introduces a potential collapse mechanism into the structure. To avoid this collapse mechanism, monolithic abutments are particularly recommended for bridges classified as Seismic Zone 4. Although damage may be heavier than that for free-standing abutments because of the higher forces transferred to backfill soils, with adequate abutment reinforcement, the collapse potential is low. In making estimates of monolithic abutment stiffness and associated longitudinal displacements during transfer of peak earthquake forces from the structure, it is recommended that abutments be proportioned to restrict displacements to 3.5 in. or less in order to minimize damage.

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