# SECTION 11: ABUTMENTS, PIERS, AND WALLS

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SECTION 11

# ABUTMENTS, PIERS, AND WALLS

## 11.1 SCOPE

This Section provides requirements for design of abutments and walls. Conventional retaining walls, nongravity cantilevered walls, anchored walls, mechanically stabilized earth (MSE) walls, and prefabricated modular walls are considered.

## 11.2 DEFINITIONS

*Abutment*—A structure that supports the end of a bridge span, and provides lateral support for fill material on which the roadway rests immediately adjacent to the bridge. In practice, different types of abutments may be used. These include:

- *Stub Abutment*—Stub abutments are located at or near the top of approach fills, with a backwall depth sufficient to accommodate the structure depth and bearings which sit on the bearing seat.

- *Partial-Depth Abutment*—Partial-depth abutments are located approximately at middepth of the front slope of the approach embankment. The higher backwall and wingwalls may retain fill material, or the embankment slope may continue behind the backwall. In the latter case, a structural approach slab or end span design must bridge the space over the fill slope, and curtain walls are provided to close off the open area. Inspection access should be provided for this situation.

- *Full-Depth Abutment*—Full-depth abutments are located at the approximate front toe of the approach embankment, restricting the opening under the structure.

- *Integral Abutment*—Integral abutments are rigidly attached to the superstructure and are supported on a spread or deep foundations capable of permitting necessary horizontal movements.

*Anchored Wall*—An earth retaining system typically composed of the same elements as nongravity cantilevered walls, and that derive additional lateral resistance from one or more tiers of anchors.

*Mechanically Stabilized Earth Wall*—A soil-retaining system, employing either strip or grid-type, metallic, or polymeric tensile reinforcements in the soil mass, and a facing element that is either vertical or nearly vertical.

*Nongravity Cantilever Wall*—A soil-retaining system that derives lateral resistance through embedment of vertical wall elements and supports retained soil with facing elements. Vertical wall elements may consist of discrete elements, e.g., piles, drilled shafts or auger-cast piles spanned by a structural facing, e.g., lagging, panels or shotcrete. Alternatively, the vertical wall elements and facing may be continuous, e.g., sheet piles, diaphragm wall panels, tangent-piles, or tangent drilled shafts.

*Pier*—That part of a bridge structure that provides intermediate support to the superstructure. Different types of piers may be used. These include:

- *Solid Wall Piers*—Solid wall piers are designed as columns for forces and moments acting about the weak axis and as piers for those acting about the strong axis. They may be pinned, fixed or free at the top, and are conventionally fixed at the base. Short, stubby types are often pinned at the base to eliminate the high moments which would develop due to fixity. Earlier, more massive designs were considered gravity types.

- *Double Wall Piers*—Double wall piers consist of two separate walls, spaced in the direction of traffic, to provide support at the continuous soffit of concrete box superstructure sections. These walls are integral with the superstructure and must also be designed for the superstructure moments which develop from live loads and erection conditions.

- *Bent Piers*—Bent-type piers consist of two or more transversely spaced columns of various solid cross-sections, and these types are designed for frame action relative to forces acting about the strong axis of the pier. They are usually fixed at the base of the pier and are either integral with the superstructure or with a pier cap at the top. The columns may be supported on a spread- or pile-supported footing, or a solid wall shaft, or they may be extensions of the piles or shaft above the ground line.

- *Single-Column Piers*—Single-column piers, often referred to as "T" or "Hammerhead" piers, are usually supported at the base by a spread-, drilled shaft- or pile-supported footing, and may be either integral with, or provide independent support for, the superstructure. Their cross-section can be of various shapes and the column can be prismatic or flared to form the pier cap or to blend with the sectional configuration of the superstructure cross-section. This type of pier can avoid the complexities of skewed supports if integrally framed into the superstructure and their appearance reduces the massiveness often associated with superstructures.

- *Tubular Piers*—A hollow core section which may be of steel, reinforced concrete or prestressed concrete, of such cross-section to support the forces and moments acting on the elements. Because of their vulnerability to lateral loadings, tubular piers shall be of sufficient wall thickness to sustain the forces and moments for all loading situations as are appropriate. Prismatic configurations may be sectionally precast or prestressed as erected.

*Prefabricated Modular Wall*—A soil-retaining system employing interlocking soil-filled timber, reinforced concrete, or steel modules or bins to resist earth pressures by acting as gravity retaining walls.

*Rigid Gravity and Semi-Gravity (Conventional) Retaining Wall*—A structure that provides lateral support for a mass of soil and that owes its stability primarily to its own weight and to the weight of any soil located directly above its base.

In practice, different types of rigid gravity and semi-gravity retaining walls may be used. These include:

- A *gravity* wall depends entirely on the weight of the stone or concrete masonry and of any soil resting on the masonry for its stability. Only a nominal amount of steel is placed near the exposed faces to prevent surface cracking due to temperature changes.

- A *semi-gravity* wall is somewhat more slender than a gravity wall and requires reinforcement consisting of vertical bars along the inner face and dowels continuing into the footing. It is provided with temperature steel near the exposed face.

- A *cantilever* wall consists of a concrete stem and a concrete base slab, both of which are relatively thin and fully reinforced to resist the moments and shears to which they are subjected.

- A *counterfort* wall consists of a thin concrete face slab, usually vertical, supported at intervals on the inner side by vertical slabs or counterforts that meet the face slab at right angles. Both the face slab and the counterforts are connected to a base slab, and the space above the base slab and between the counterforts is backfilled with soil. All the slabs are fully reinforced.

## 11.3 NOTATION

### 11.3.1 General

$A$ = maximum earthquake acceleration (dim.) (C11.8.6)
$A_c$ = cross-sectional area of reinforcement unit (in.$^2$) (11.10.6.4.1)
$A_m$ = maximum wall acceleration coefficient at the centroid (11.10.7.1)
$B$ = wall base width (ft.) (11.10.2)
$b$ = unit width of reinforcement; width of bin module (ft.) (11.10.6.4.1) (11.11.5.1)
$b_f$ = width of applied footing load (ft.) (11.10.10.2)
$C$ = overall reinforcement surface area geometry factor (dim.) (11.10.6.3.2)
$CR_{cr}$ = long-term connection strength reduction factor to account for reduced ultimate strength resulting from connection (dim.) (11.10.6.4.4b)

$CR_u$ = short-term connection strength reduction factor to account for reduced ultimate strength resulting from the connection (dim.) (11.10.6.4.4b)
$C_u$ = coefficient of uniformity (dim.) (11.10.6.3.2)
$D$ = design embedment depth of vertical element (ft.); diameter of bar or wire (in.) (11.10.6.3.2) (C11.8.4.1)
$D^*$ = diameter of bar or wire corrected for corrosion loss (ft.) (11.10.6.4.1)
$D_o$ = embedment for which net passive pressure is sufficient to provide moment equilibrium (ft.) (C11.8.4.1)
$D_{60}/D_{10}$ = uniformity coefficient of soil defined as ratio of the particle size of soil that is 60 percent finer in size to the particle size of soil that is ten percent finer in size (dim.) (11.10.6.3.2)
$d$ = diameter of anchor drill hole (ft.); the lateral wall displacement (in.); fill above wall (ft.) (C11.6.5) (11.9.4.2) (11.10.8)
$E_c$ = thickness of metal reinforcement at end of service life (mil.) (11.10.6.4.1)
$E_n$ = nominal thickness of steel reinforcement at construction (mil.) (11.10.6.4.2a)
$E_s$ = sacrificial thickness of metal expected to be lost by uniform corrosion during service life (mil.) (11.10.6.4.2a)
$E_{AE}$ = total active static and seismic force (kips/ft.) (A11.1.1.1)
$E_{PE}$ = total passive static and seismic force (kips/ft.) (A11.1.1.1)
$e$ = eccentricity of load from centerline of foundation (ft.) (11.10.8)
$F_T$ = resultant force of active lateral earth pressure (kips/ft.) (11.6.3.2)
$F_y$ = minimum yield strength of steel (ksi) (11.10.6.4.3a)
$F^*$ = reinforcement pullout friction factor (dim.) (11.10.6.3.2)
$G_u$ = distance from center of gravity of a horizontal segmental facing block unit, including aggregate fill, measured from the front of the unit (ft.) (11.10.6.4.4b)
$H$ = height of wall (ft.) (11.9.1)
$H_h$ = hinge height for segmental facing (ft.) (11.10.6.4.4b)
$H_u$ = segmental facing block unit height (ft.) (11.10.6.4.4b)
$H_1$ = equivalent wall height (ft.) (11.10.6.3.1)
$h$ = vertical distance between ground surface and wall base at the back of wall heel (ft.) (11.6.3.2)
$h_i$ = height of reinforced soil zone contributing horizontal load to reinforcement at level $i$ (ft.) (11.10.6.2.1)
$i_b$ = slope of facing base downward into backfill (°) (11.10.6.4.4b)
$k_a$ = active earth pressure coefficient (dim.) (11.8.4.1)
$k_{af}$ = active earth pressure coefficient of backfill (dim.) (11.10.5.2)
$k_h$ = horizontal seismic acceleration coefficient (dim.) (11.8.6)
$k_v$ = vertical seismic acceleration coefficient (dim.) (A11.1.1.1)
$k_{AE}$ = seismic active pressure coefficient (dim.) (A11.1.1.1)
$k_{PE}$ = seismic passive pressure coefficient (dim.) (A11.1.1.1)
$k_r$ = horizontal earth pressure coefficient of reinforced fill (dim.) (11.10.5.2)
$L$ = spacing between vertical elements or facing supports (ft.); length of reinforcing elements in an MSE wall and correspondingly its foundation (ft.) (11.8.5.2) (11.10.2)
$L_a$ = length of reinforcement in active zone (ft.) (11.10.2)
$L_b$ = anchor bond length (ft.) (11.9.4.2)
$L_e$ = length of reinforcement in resistance zone (ft.) (11.10.2)
$L_{ei}$ = effective reinforcement length for layer $i$ (ft.) (11.10.7.2)
$MARV$ = minimum average roll value (11.10.6.4.3b)
$M_{max}$ = maximum bending moment in vertical wall element or facing (kip-ft. or kip-ft./ft.) (11.8.5.2)
$N$ = normal component of resultant on base of foundation (kips/ft.) (11.6.3.2)
$P_{AE}$ = dynamic horizontal thrust (kips/ft.) (11.10.7.1)
$P_b$ = pressure inside bin module (ksf) (11.10.5.1)
$P_H$ = lateral force due to superstructure or other concentrated loads (kips/ft.) (11.10.10.1)
$P_i$ = factored horizontal force per mm of wall transferred to soil reinforcement at level $i$; internal inertial force, due to the weight of the backfill within the active zone (kips/ft.) (11.10.6.2.1) (11.10.7.2)
$P_{IR}$ = horizontal inertial force (kips/ft.) (11.10.7.1)
$P_{ir}$ = horizontal inertial force caused by acceleration of reinforced backfill (kips/ft.) (11.10.7.1)
$P_{is}$ = internal inertial force caused by acceleration of sloping surcharge (kips/ft.) (11.10.7.1)
$P_r$ = ultimate soil reinforcement pullout resistance per unit of reinforcement width (kips/ft.) (11.10.6.3.2)
$P_v$ = load on strip footing (kips/ft.) (11.10.10.1)
$P'_v$ = load on isolated rectangular footing or point load (kips) (11.10.10.1)
$p$ = average lateral pressure, including earth, surcharge and water pressure, acting on the section of wall element being considered (ksf) (11.9.5.2)

$Q_n$ = nominal (ultimate) anchor resistance (kips) (11.9.4.2)
$Q_R$ = factored anchor resistance (kips) (11.9.4.2)
$q_s$ = surcharge pressure (ksf) (11.10.5.2)
$q_{max}$ = maximum unit soil pressure on base of foundation (ksf) (11.6.3.2)
$R$ = resultant force at base of wall (kips/ft.) (11.6.3.2)
$R_{BH}$ = basal heave ratio (C11.9.3.1)
$R_c$ = reinforcement coverage ratio (dim.) (11.10.6.3.2)
$R_n$ = nominal resistance (kips or kips/ft.) (11.5.4)
$R_R$ = factored resistance (kips or kips/ft.) (11.5.4)
$RF$ = combined strength reduction factor to account for potential long-term degradation due to installation damage, creep and chemical/biological aging of geosynthetic reinforcements (dim.) (11.10.6.4.2b)
$RF_c$ = combined strength reduction factor for long-term degradation of geosynthetic reinforcement facing connection (dim.) (11.10.6.4.4b)
$RF_{CR}$ = strength reduction factor to prevent long-term creep rupture of reinforcement (dim.) (11.10.6.4.3b)
$RF_D$ = strength reduction factor to prevent rupture of reinforcement due to chemical and biological degradation (dim.) (11.10.6.4.3b)
$RF_{ID}$ = strength reduction factor to account for installation damage to reinforcement (dim.) (11.10.6.4.3b)
$S_h$ = horizontal reinforcement spacing (ft.) (11.10.6.4.1)
$S_t$ = spacing between transverse grid elements (in.) (11.10.6.3.2)
$S_u$ = undrained shear strength (ksf) (11.9.5.2)
$S_v$ = vertical spacing of reinforcements (ft.) (11.10.6.2.1)
$S_{rs}$ = ultimate reinforcement tensile resistance required to resist static load component (kips/ft.) (11.10.7.2)
$S_{rt}$ = ultimate reinforcement tensile resistance required to resist transient load component (kips/ft.) (11.10.7.2)
$T_{ac}$ = nominal long-term reinforcement/facing connection design strength (kips/ft.) (11.10.6.4.1)
$T_{al}$ = nominal long-term reinforcement design strength (kips/ft.) (11.10.6.4.1)
$T_{crc}$ = creep reduced connection strength per unit of reinforcement width determined from the stress rupture envelope at the specified design life as produced from a series of long-term connection creep tests (kips/ft.) (11.10.6.4.4b)
$T_{lot}$ = ultimate wide width tensile strength per unit of reinforcement width (ASTM D4595 or D6637) for the reinforcement material lot used for the connection strength testing (kips/ft.) (11.10.6.4.4b)
$T_{md}$ = factored incremental dynamic inertia force (kips/ft.) (11.10.7.2)
$T_{ultconn}$ = ultimate connection strength per unit of reinforcement width (kips/ft.) (11.10.6.4.4b)
$T_{ult}$ = ultimate tensile strength of reinforcement (kips/ft.) (11.10.6.4.3b)
$T_{max}$ = applied load to reinforcement (kips/ft.) (11.10.6.2.1)
$T_o$ = factored tensile load at reinforcement/facing connection (kips/ft.) (11.10.6.2.2)
$t$ = thickness of transverse elements (in.) (11.10.6.3.2)
$T_{total}$ = total load on reinforcement layer (static & dynamic) per unit width of wall (kips/ft.) (11.10.7.2)
$V_1$ = weight of soil carried by wall heel, not including weight of soil surcharge (kips/ft.) (11.6.3.2)
$V_2$ = weight of soil surcharge directly above wall heel (kips/ft.) (11.6.3.2)
$W_u$ = unit width of segmental facing (ft.) (11.10.2.3.2)
$W_1$ = weight of wall stem (kips/ft.) (11.6.3.2)
$W_2$ = weight of wall footing or base (kips/ft.) (11.6.3.2)
$x$ = spacing between vertical element supports (ft.) (11.9.5.2)
$Z$ = depth below effective top of wall or to reinforcement (ft.) (11.10.6.2.1)
$Z_p$ = depth of soil at reinforcement layer at beginning of resistance zone for pullout calculation (ft.) (11.10.6.2.1)
$\alpha$ = scale effect correction factor (dim.) (11.10.6.3.2)
$\beta$ = inclination of ground slope behind face of wall (°) (11.5.5)
$\gamma_{EQ}$ = load factor for earthquake loading in Article 3.4.1 (dim.) (11.6.5)
$\gamma_P$ = load factor for vertical earth pressure in Article 3.4.1 (dim.) (11.10.6.2.1)
$\gamma_s$ = soil unit weight (kcf)
$\gamma'_s$ = effective soil unit weight (kcf) (C11.8.4.1)
$\gamma_r$ = unit weight of reinforced fill (kcf) (11.10.5.2)
$\gamma_f$ = unit weight of backfill (kcf) (11.10.5.2)
$\Delta\sigma_H$ = horizontal stress on reinforcement from concentrated horizontal surcharge (ksf); traffic barrier impact stress applied over reinforcement tributary area (ksf) (11.10.6.2.1) (11.10.10.2)
$\Delta\sigma_v$ = vertical stress due to footing load (ksf) (11.10.8)
$\delta$ = wall-backfill interface friction angle (°) (11.5.5)
$\delta_{max}$ = maximum displacement (ft.) (11.10.4.2)

$\delta_R$ = relative displacement coefficient (11.10.4.2)
$\theta$ = wall batter from horizontal (°) (11.10.6.2.1)
$\rho$ = soil-reinforcement angle of friction (°) (11.10.5.3)
$\phi$ = resistance factor (11.5.4)
$\phi_f$ = internal friction angle of foundation or backfill soil (°) (11.10.2)
$\phi_r$ = internal friction angle of reinforced fill (°) (11.10.5.2)
$\phi'_f$ = effective internal friction angle of soil (°) (11.8.4.1)
$\sigma_H$ = factored horizontal stress at reinforcement level (ksf) (11.10.6.2.1)
$\sigma_{Hmax}$ = maximum stress in soil reinforcement in abutment zones (11.10.8)
$\sigma_v$ = vertical stress in soil (ksf) (11.10.6.2.1)
$\sigma_{V1}$ = vertical soil stress over effective base width (ksf) (11.10.8)
$\tau_n$ = nominal anchor bond stress (ksf) (11.9.4.2)
$\omega$ = wall batter due to setback of segmental facing units (°) (11.10.6.4.4b)

## 11.4 SOIL PROPERTIES AND MATERIALS

### 11.4.1 General

Backfill materials should be granular, free-draining materials. Where walls retain in-situ cohesive soils, drainage shall be provided to reduce hydrostatic water pressure behind the wall.

### C11.4.1

Much of the knowledge and experience with MSE structures has been with select, cohesionless backfill as specified in Section 7 of *AASHTO LRFD Bridge Construction Specifications*. Hence, knowledge about internal stress distribution, pullout resistance and failure surface shape is constrained and influenced by the unique engineering properties of granular soils. While cohesive soils have been successfully used, problems including excessive deformation and complete collapse have also occurred. Most of these problems have been attributed to poor drainage. Drainage requirements for walls constructed with poor draining soils are provided in Elias et al. (*2001*).

### 11.4.2 Determination of Soil Properties

The provisions of Articles 2.4 and 10.4 shall apply.

## 11.5 LIMIT STATES AND RESISTANCE FACTORS

### 11.5.1 General

Design of abutments, piers and walls shall satisfy the criteria for the service limit state specified in Article 11.5.2, and for the strength limit state specified in Article 11.5.3.

Abutments, piers and retaining walls shall be designed to withstand lateral earth and water pressures, including any live and dead load surcharge, the self weight of the wall, temperature and shrinkage effects, and earthquake loads in accordance with the general principles specified in this Section.

### C11.5.1

Earth retaining structures shall be designed for a service life based on consideration of the potential long-term effects of material deterioration, seepage, stray currents and other potentially deleterious environmental factors on each of the material components comprising the structure. For most applications, permanent retaining walls should be designed for a minimum service life of 75 years. Retaining wall applications defined as temporary shall be considered to have a service life of 36 months or less.

A greater level of safety and/or longer service life, i.e., 100 years, may be appropriate for walls which support bridge abutments, buildings, critical utilities, or other facilities for which the consequences of poor performance or failure would be severe.

Permanent structures shall be designed to retain an aesthetically pleasing appearance, and be essentially maintenance free throughout their design service life.

Design of walls to be essentially maintenance free does not preclude the need for periodic inspection of the wall to assess its condition throughout its design life.

### 11.5.2 Service Limit States

Abutments, piers, and walls shall be investigated for excessive vertical and lateral displacement, and overall stability, at the service limit state. Tolerable vertical and lateral deformation criteria for retaining walls shall be developed based on the function and type of wall, anticipated service life, and consequences of unacceptable movements to the wall and any potentially affected nearby structures, i.e., both structural and aesthetic. Overall stability shall be evaluated using limit equilibrium methods of analysis.

The provisions of Articles 10.6.2.2, 10.7.2.2, and 10.8.2.2 shall apply to the investigation of vertical wall movements. For anchored walls, deflections shall be estimated in accordance with the provisions of Article 11.9.3.1. For MSE walls, deflections shall be estimated in accordance with the provisions of Article 11.10.4.

### C11.5.2

Vertical wall movements are primarily the result of soil settlement beneath the wall. For gravity and semigravity walls, lateral movement results from a combination of differential vertical settlement between the heel and the toe of the wall and the rotation necessary to develop active earth pressure conditions (see Article C3.11.1).

Tolerable total and differential vertical deformations for a particular retaining wall are dependent on the ability of the wall to deflect without causing damage to the wall elements or adjacent structures, or without exhibiting unsightly deformations.

Surveys of the performance of bridges indicate that horizontal abutment movements less than 1.5 in. can usually be tolerated by bridge superstructures without significant damage, as reported in Bozozuk (*1978*); Walkinshaw (*1978*); Moulton et al. (*1985*); and Wahls (*1990*). Earth pressures used in design of abutments should be selected consistent with the requirement that the abutment should not move more than 1.5 in. laterally.

Regarding impact to the wall itself, differential settlement along the length of the wall and to some extent from front to back of wall is the best indicator of the potential for retaining wall structural damage or overstress. Wall facing stiffness and ability to adjust incrementally to movement affect the ability of a given wall system to tolerate differential movements. The total and differential vertical deformation of a retaining wall should be small for rigid gravity and semigravity retaining walls, and for soldier pile walls with a cast-in-place facing. For walls with anchors, any downward movement can cause significant stress relaxation of the anchors.

MSE walls can tolerate larger total and differential vertical deflections than rigid walls. The amount of total and differential vertical deflection that can be tolerated depends on the wall facing material, configuration and timing of facing construction. A cast-in-place facing has the same vertical deformation limitations as the more rigid retaining wall systems. However, an MSE wall with a cast-in-place facing can be specified with a waiting period before the cast-in-place facing is constructed so that vertical (as well as horizontal) deformations have time to occur. An MSE wall with welded wire or geosynthetic facing can tolerate the most deformation. An MSE wall with multiple precast concrete panels cannot tolerate as much vertical deformation as flexible welded wire or geosynthetic facings because of potential damage to the precast panels and unsightly panel separation.

### 11.5.3 Strength Limit State

Abutments and walls shall be investigated at the strength limit states using Eq. 1.3.2.1-1 for:

- Bearing resistance failure,
- Lateral sliding,
- Excessive loss of base contact,
- Pullout failure of anchors or soil reinforcements, and
- Structural failure.

### 11.5.4 Resistance Requirement

Abutments, piers and retaining structures and their foundations and other supporting elements shall be proportioned by the appropriate methods specified in Articles 11.6, 11.7, 11.8, 11.9, 11.10, or 11.11 so that their resistance satisfies Article 11.5.5.

The factored resistance, $R_R$, calculated for each applicable limit state shall be the nominal resistance, $R_n$, multiplied by an appropriate resistance factor, $\phi$, specified in Table 11.5.6-1.

### C11.5.4

Procedures for calculating nominal resistance are provided in Articles 11.6, 11.7, 11.8, 11.9, 11.10, and 11.11 for abutments and retaining walls, piers, nongravity cantilevered walls, anchored walls, mechanically stabilized earth walls, and prefabricated modular walls, respectively.

### 11.5.5 Load Combinations and Load Factors

Abutments, piers and retaining structures and their foundations and other supporting elements shall be proportioned for all applicable load combinations specified in Article 3.4.1.

### C11.5.5

Figures C1 and C2 show the typical application of load factors to produce the total extreme factored force effect for external stability of retaining walls. Where live load surcharge is applicable, the factored surcharge force is generally included over the backfill immediately above the wall only for evaluation of foundation bearing resistance and structure design, as shown in Figure C3. The live load surcharge is not included over the backfill for evaluation of eccentricity, sliding or other failure mechanisms for which such surcharge would represent added resistance to failure. Likewise, the live load on a bridge abutment is included only for evaluation of foundation bearing resistance and structure design. The load factor for live load surcharge is the same for both vertical and horizontal load effects.

The permanent and transient loads and forces shown in the figures include, but are not limited to:

- Permanent Loads

  $DC$ = dead load of structural components and nonstructural attachments
  $DW$ = dead load of wearing surfaces and utilities
  $EH$ = horizontal earth pressure load
  $ES$ = earth surcharge load
  $EV$ = vertical pressure from dead load of earth fill

- Transient Loads

  $LS$ = live load surcharge
  $WA$ = water load and stream pressure

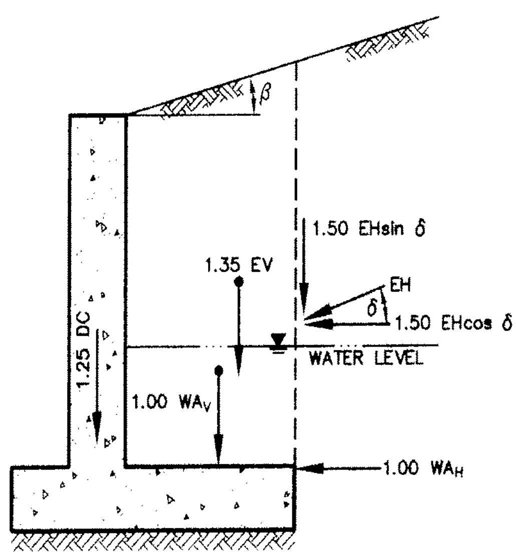



**Figure C11.5.5-1 Typical Application of Load Factors for Bearing Resistance.**

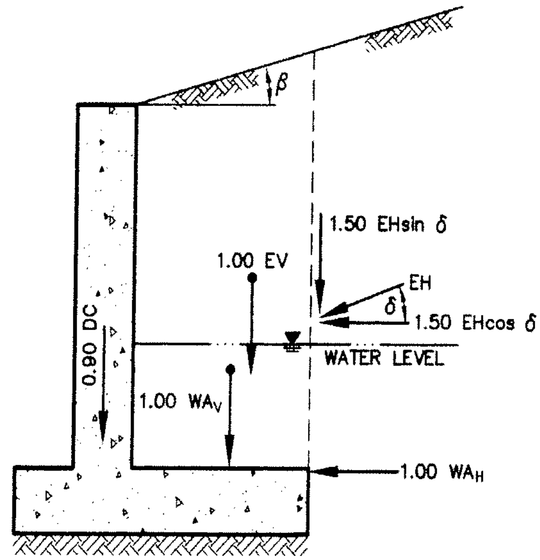



**Figure C11.5.5-2 Typical Application of Load Factors for Sliding and Eccentricity.**

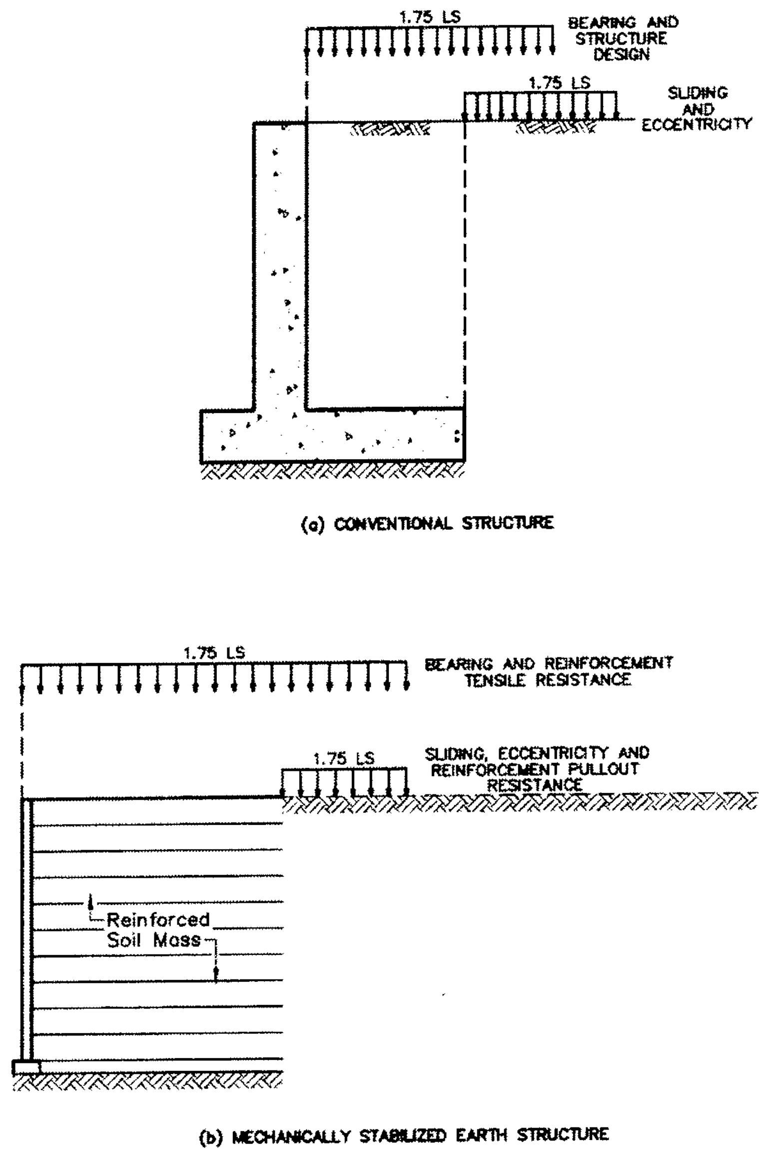



**Figure C11.5.5-3 Typical Application of Live Load Surcharge.**

### 11.5.6 Resistance Factors

Resistance factors for geotechnical design of foundations are specified in Tables 10.5.5.2.2-1, 10.5.5.2.3-1, 10.5.5.2.4-1, and Table 1.

If methods other than those prescribed in these Specifications are used to estimate resistance, the resistance factors chosen shall provide the same reliability as those given in Tables 10.5.5.2.2-1, 10.5.5.2.3-1, 10.5.5.2.4-1, and Table 1.

Vertical elements, such as soldier piles, tangent-piles and slurry trench concrete walls shall be treated as either shallow or deep foundations, as appropriate, for purposes of estimating bearing resistance, using procedures described in Articles 10.6, 10.7, and 10.8.

### C11.5.6

The resistance factors given in Table 1, other than those referenced back to Section 10, were calculated by direct correlation to allowable stress design rather than reliability theory.

Since the resistance factors in Table 1 were based on direct correlation to allowable stress design, the differences between the resistance factors for tensile resistance of metallic versus geosynthetic reinforcement are based on historical differences in the level of safety applied to reinforcement designs for these two types of reinforcements. See Article C11.10.6.2.1 for additional comments regarding the differences between the resistance factors for metallic versus geosynthetic reinforcement.

Some increase in the prescribed resistance factors may be appropriate for design of temporary walls consistent with increased allowable stresses for temporary structures in allowable stress design.

The evaluation of overall stability of walls or earth slopes with or without a foundation unit should be investigated at the service limit state based on the Service I Load Combination and an appropriate resistance factor.

**Table 11.5.6-1 Resistance Factors for Permanent Retaining Walls.**

| WALL-TYPE AND CONDITION | | RESISTANCE FACTOR |
|---|---|---|
| Nongravity Cantilevered and Anchored Walls | | |
| Axial compressive resistance of vertical elements | | Article 10.5 applies |
| Passive resistance of vertical elements | | 0.75 |
| Pullout resistance of anchors [1] | • Cohesionless (granular) soils<br>• Cohesive soils<br>• Rock | 0.65 [1]<br>0.70 [1]<br>0.50 [1] |
| Pullout resistance of anchors [2] | • Where proof tests are conducted | 1.0 [2] |
| Tensile resistance of anchor tendon | • Mild steel (e.g., ASTM A 615 bars)<br>• High strength steel (e.g., ASTM A 722 bars) | 0.90 [3]<br>0.80 [3] |
| Flexural capacity of vertical elements | | 0.90 |
| | | |
| Mechanically Stabilized Earth Walls | | |
| Bearing resistance | | Article 10.5 applies |
| Sliding | | Article 10.5 applies |
| Tensile resistance of metallic reinforcement and connectors | Strip reinforcements [4]<br>• Static loading<br>• Combined static/earthquake loading<br>Grid reinforcements [4] [5]<br>• Static loading<br>• Combined static/earthquake loading | <br>0.75<br>1.00<br><br>0.65<br>0.85 |
| Tensile resistance of geosynthetic reinforcement and connectors | • Static loading<br>• Combined static/earthquake loading | 0.90<br>1.20 |
| Pullout resistance of tensile reinforcement | • Static loading<br>• Combined static/earthquake loading | 0.90<br>1.20 |
| | | |
| Prefabricated Modular Walls | | |
| Bearing | | Article 10.5 applies |
| Sliding | | Article 10.5 applies |
| Passive resistance | | Article 10.5 applies |

(1) Apply to presumptive ultimate unit bond stresses for preliminary design only in Article C11.9.4.2.

(2) Apply where proof test(s) are conducted on every production anchor to a load of 1.0 or greater times the factored load on the anchor.

(3) Apply to maximum proof test load for the anchor. For mild steel apply resistance factor to $F_y$. For high-strength steel apply the resistance factor to guaranteed ultimate tensile strength.

(4) Apply to gross cross-section less sacrificial area. For sections with holes, reduce gross area in accordance with Article 6.8.3 and apply to net section less sacrificial area.

(5) Applies to grid reinforcements connected to a rigid facing element, e.g., a concrete panel or block. For grid reinforcements connected to a flexible facing mat or which are continuous with the facing mat, use the resistance factor for strip reinforcements.

### 11.5.7 Extreme Event Limit State

The applicable load combinations and load factors specified in Table 3.4.1-1 shall be investigated. Unless otherwise specified, all resistance factors shall be taken as 1.0 when investigating the extreme event limit state.

## 11.6 ABUTMENTS AND CONVENTIONAL RETAINING WALLS

### 11.6.1 General Considerations

#### 11.6.1.1 General

Rigid gravity and semigravity retaining walls may be used for bridge substructures or grade separation and are generally for permanent applications.

Rigid gravity and semigravity walls shall not be used without deep foundation support where the bearing soil/rock is prone to excessive total or differential settlement.

#### C11.6.1.1

Conventional retaining walls are generally classified as rigid gravity or semigravity walls, examples of which are shown in Figure C1. These types of walls can be effective for both cut and fill wall applications.

Excessive differential settlement, as defined in Article C11.6.2.2 can cause cracking, excessive bending or shear stresses in the wall, or rotation of the wall structure.

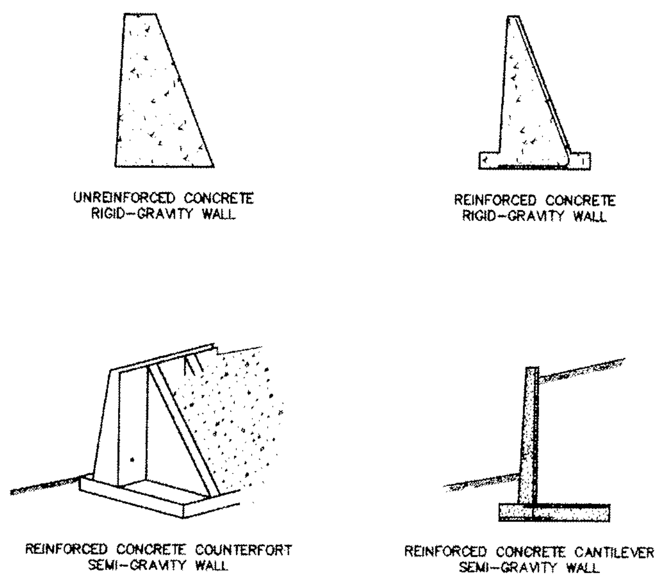



**Figure C11.6.1.1-1 Typical Rigid Gravity and Semigravity Walls.**

#### 11.6.1.2 Loading

Abutments and retaining walls shall be investigated for:

- Lateral earth and water pressures, including any live and dead load surcharge;
- The self weight of the abutment/wall;
- Loads applied from the bridge superstructure;

#### C11.6.1.2

- Temperature and shrinkage deformation effects; and
- Earthquake loads, as specified herein, in Section 3 and elsewhere in these Specifications.

The provisions of Articles 3.11.5 and 11.5.5 shall apply. For stability computations, the earth loads shall be multiplied by the maximum and/or minimum load factors given in Table 3.4.1-2, as appropriate.

The design shall be investigated for any combination of forces which may produce the most severe condition of loading. The design of abutments on mechanically stabilized earth and prefabricated modular walls shall be in accordance with Articles 11.10.11 and 11.11.6.

For computing load effects in abutments, the weight of filling material directly over an inclined or stepped rear face, or over the base of a reinforced concrete spread footing may be considered as part of the effective weight of the abutment.

Where spread footings are used, the rear projection shall be designed as a cantilever supported at the abutment stem and loaded with the full weight of the superimposed material, unless a more exact method is used.

Cohesive backfills are difficult to compact. Because of the creep of cohesive soils, walls with cohesive backfills designed for active earth pressures will continue to move gradually throughout their lives, especially when the backfill is soaked by rain or rising groundwater levels. Therefore, even if wall movements are tolerable, walls backfilled with cohesive soils should be designed with extreme caution for pressures between the active and at-rest cases assuming the most unfavorable conditions. Consideration must be given for the development of pore water pressure within the soil mass in accordance with Article 3.11.3. Appropriate drainage provisions should be provided to prevent hydrostatic and seepage forces from developing behind the wall. In no case shall highly plastic clay be used for backfill.

#### 11.6.1.3 Integral Abutments

Integral abutments shall be designed to resist and/or absorb creep, shrinkage and thermal deformations of the superstructure.

Movement calculations shall consider temperature, creep, and long-term prestress shortening in determining potential movements of abutments.

Maximum span lengths, design considerations, details should comply with recommendations outlined in FHWA Technical Advisory T 5140.13 (*1980*), except where substantial local experience indicates otherwise.

To avoid water intrusion behind the abutment, the approach slab should be connected directly to the abutment (not to wingwalls), and appropriate provisions should be made to provide for drainage of any entrapped water.

#### C11.6.1.3

Deformations are discussed in Article 3.12.

Integral abutments should not be constructed on spread footings founded or keyed into rock unless one end of the span is free to displace longitudinally.

#### 11.6.1.4 Wingwalls

Wingwalls may either be designed as monolithic with the abutments, or be separated from the abutment wall with an expansion joint and designed to be free standing.

The wingwall lengths shall be computed using the required roadway slopes. Wingwalls shall be of sufficient length to retain the roadway embankment and to furnish protection against erosion.

#### 11.6.1.5 Reinforcement

##### *11.6.1.5.1 Conventional Walls and Abutments*

Reinforcement to resist the formation of temperature and shrinkage cracks shall be designed as specified in Article 5.10.8.

##### *11.6.1.5.2 Wingwalls*

Reinforcing bars or suitable rolled sections shall be spaced across the junction between wingwalls and abutments to tie them together. Such bars shall extend into the masonry on each side of the joint far enough to develop the strength of the bar as specified for bar reinforcement, and shall vary in length so as to avoid planes of weakness in the concrete at their ends. If bars are not used, an expansion joint shall be provided and the wingwall shall be keyed into the body of the abutment.

#### 11.6.1.6 Expansion and Contraction Joints

Contraction joints shall be provided at intervals not exceeding 30.0 ft. and expansion joints at intervals not exceeding 90.0 ft. for conventional retaining walls and abutments. All joints shall be filled with approved filling material to ensure the function of the joint. Joints in abutments shall be located approximately midway between the longitudinal members bearing on the abutments.

### 11.6.2 Movement and Stability at the Service Limit State

#### 11.6.2.1 Abutments

The provisions of Articles 10.6.2.4, 10.6.2.5, 10.7.2.3 through 10.7.2.5, 10.8.2.2 through 10.8.2.4, and 11.5.2 shall apply as applicable.

#### 11.6.2.2 Conventional Retaining Walls

The provisions of Articles 10.6.2.4, 10.6.2.5, 10.7.2.3 through 10.7.2.5, 10.8.2.2 through 10.8.2.4, and 11.5.2 apply as applicable.

#### C11.6.2.2

For a conventional reinforced concrete retaining wall, experience suggests that differential wall settlements on the order of 1 in 500 to 1 in 1,000 may overstress the wall.

### 11.6.2.3 Overall Stability

The overall stability of the retaining wall, retained slope and foundation soil or rock shall be evaluated for all walls using limiting equilibrium methods of analysis. The overall stability of temporary cut slopes to facilitate construction shall also be evaluated. Special exploration, testing and analyses may be required for bridge abutments or retaining walls constructed over soft deposits.

The evaluation of overall stability of earth slopes with or without a foundation unit should be investigated at the Service I Load Combination and an appropriate resistance factor. In lieu of better information, the resistance factor, $\phi$, may be taken as:

- Where the geotechnical parameters are well defined, and the slope does not support or contain a structural element........................ 0.75
- Where the geotechnical parameters are based on limited information, or the slope contains or supports a structural element ...................... 0.65

### C11.6.2.3

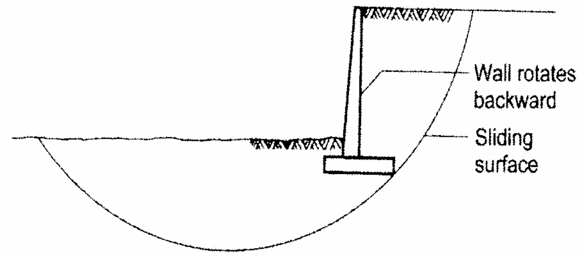



**Figure C11.6.2.3-1 Retaining Wall Overall Stability Failure.**

Figure C1 shows a retaining wall overall stability failure. Overall stability is a slope stability issue, and, therefore, is considered a service limit state check.

The Modified Bishop, simplified Janbu or Spencer methods of analysis may be used.

Soft soil deposits may be subject to consolidation and/or lateral flow which could result in unacceptable long-term settlements or horizontal movements.

**With regard to selection of a resistance factor for evaluation of overall stability of walls, examples of structural elements supported by a wall that may justify the use of the 0.65 resistance factor include a bridge or pipe arch foundation, a building foundation, a pipeline, a critical utility, or another retaining wall. If the structural element is located beyond the failure surface for external stability behind the wall illustrated conceptually in Figure 11.10.2-1, a resistance factor of 0.75 may be used.**

## 11.6.3 Bearing Resistance and Stability at the Strength Limit State

### 11.6.3.1 General

Abutments and retaining walls shall be proportioned to ensure stability against bearing capacity failure, overturning, and sliding. Safety against deep-seated foundation failure shall also be investigated, in accordance with the provisions of Article 10.6.2.5.

### 11.6.3.2 Bearing Resistance

Bearing resistance shall be investigated at the strength limit state using factored loads and resistances, assuming the following soil pressure distributions:

- Where the wall is supported by a soil foundation:

  the vertical stress shall be calculated assuming a uniformly distributed pressure over an effective base area as shown in Figure 1.

The vertical stress shall be calculated as follows:

### C11.6.3.2

See Figure 11.10.10.1-1 for an example of how to calculate the vertical bearing stress where the loading is more complex. Though this figure shows the application of superposition principles to mechanically stabilized earth walls, these principles can also be directly applied to conventional walls.

See Article C11.5.5 for application of load factors for bearing resistance and eccentricity.

$$\sigma_v = \frac{\sum V}{B - 2e} \qquad (11.6.3.2\text{-}1)$$

where:

$\Sigma V$ = the summation of vertical forces, and the other variables are as defined in Figure 1

- Where the wall is supported by a rock foundation:

the vertical stress shall be calculated assuming a linearly distributed pressure over an effective base area as shown in Figure 2. If the resultant is within the middle one-third of the base,

$$\sigma_{vmax} = \frac{\sum V}{B}\left(1 + 6\frac{e}{B}\right) \qquad (11.6.3.2\text{-}2)$$

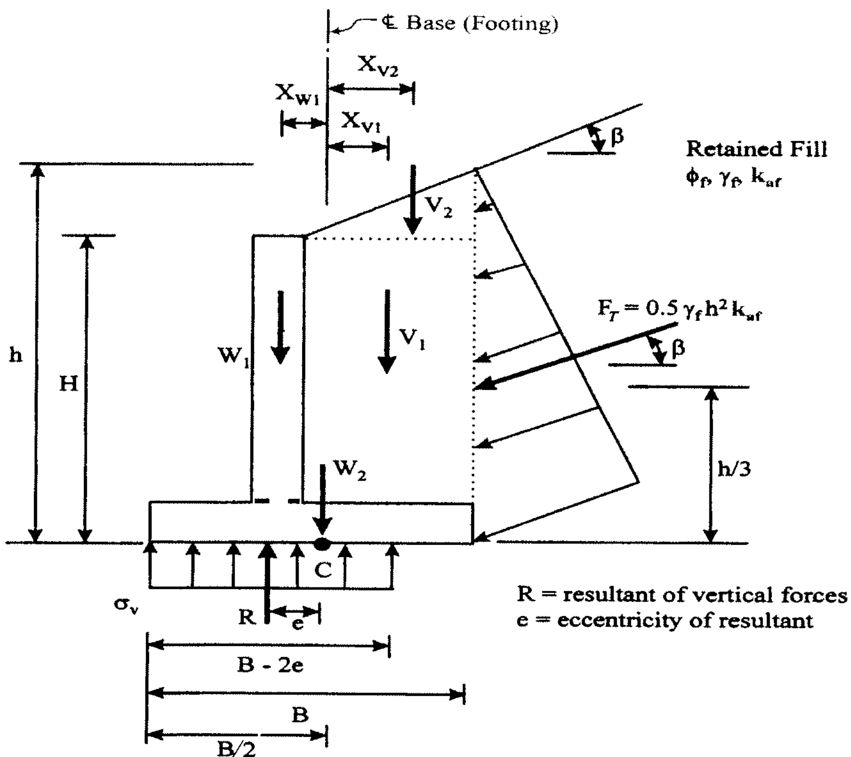
$$\sigma_{vmin} = \frac{\sum V}{B}\left(1 - 6\frac{e}{B}\right) \qquad (11.6.3.2\text{-}3)$$

where the variables are as defined in Figure 2. If the resultant is outside the middle one-third of the base,

$$\sigma_{vmax} = \frac{2\sum V}{3[(B/2) - e)]} \qquad (11.6.3.2\text{-}4)$$

$$\sigma_{vmin} = 0 \qquad (11.6.3.2\text{-}5)$$

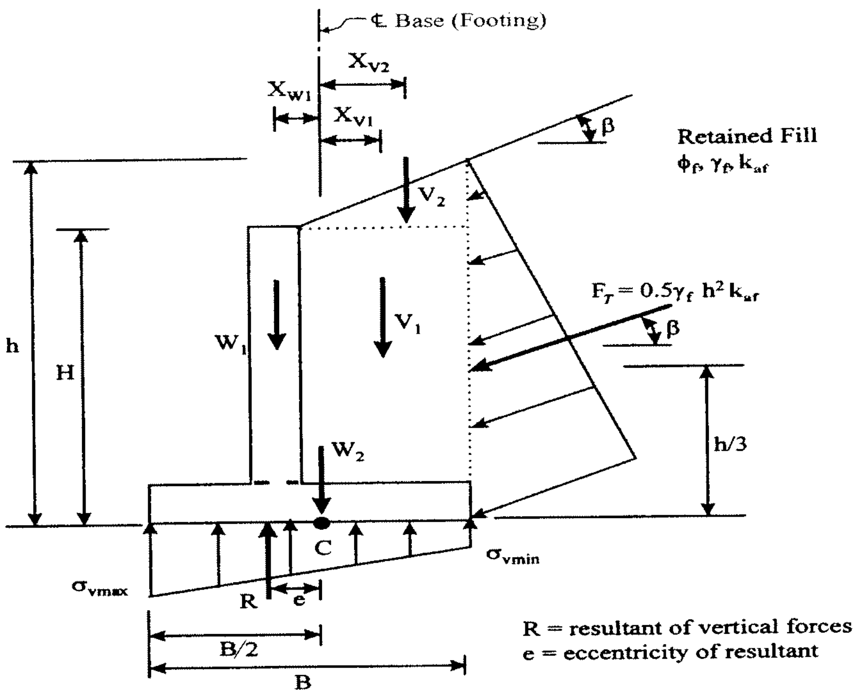
where the variables are as defined in Figure 2.




Summing moments about Point C:

$$e = \frac{\left(F_T Cos\beta\right)h/3 - \left(F_T \sin\beta\right)B/2 - V_1 X_{V1} - V_2 X_{V2} + W_1 X_{W1}}{V_1 + V_2 + W_1 + W_2 + F_T \sin\beta}$$

**Figure 11.6.3.2-1 Bearing Stress Criteria for Conventional Wall Foundations on Soil.**




**If e > B/6, $\sigma_{vmin}$ will drop to zero, and as "e" increases, the portion of the heel of the footing which has zero vertical stress increases.**

**Summing moments about Point C:**

$$e = \frac{\left(F_T Cos\beta\right)h/3 - \left(F_T \sin\beta\right)B/2 - V_1 X_{V1} - V_2 X_{V2} + W_1 X_{W1}}{V_1 + V_2 + W_1 + W_2 + F_T \sin\beta}$$

**Figure 11.6.3.2-2 Bearing Stress Criteria for Conventional Wall Foundations on Rock.**

### 11.6.3.3 Overturning

For foundations on soil, the location of the resultant of the reaction forces shall be within the middle one-half of the base width.

For foundations on rock, the location of the resultant of the reaction forces shall be within the middle three-fourths of the base width.

### C11.6.3.3

The specified criteria for the location of the resultant, coupled with investigation of the bearing pressure, replaces the investigation of the ratio of stabilizing moment to overturning moment. Location of the resultant within the middle one-half of the base width for foundations on soil is based on the use of plastic bearing pressure distribution for the limit state.

### 11.6.3.4 Subsurface Erosion

For walls constructed along rivers and streams, scour of foundation materials shall be evaluated during design, as specified in Article 2.6.4.4.2. Where potential problem conditions are anticipated, adequate protective measures shall be incorporated in the design.

The provisions of Article 10.6.1.2 shall apply.

The hydraulic gradient shall not exceed:

- For silts and cohesive soils: 0.20
- For other cohesionless soils: 0.30

### C11.6.3.4

The measures most commonly used to ensure that piping does not occur are:

- Seepage control,
- Reduction of hydraulic gradient, and
- Protective filters.

Where water seeps beneath a wall, the effects of uplift and seepage forces shall be considered.

Seepage effects may be investigated by constructing a flow net, or in certain circumstances, by using generally accepted simplified methods.

#### 11.6.3.5 Passive Resistance

Passive resistance shall be neglected in stability computations, unless the base of the wall extends below the depth of maximum scour, freeze-thaw or other disturbances. In the latter case, only the embedment below the greater of these depths shall be considered effective.

Where passive resistance is utilized to ensure adequate wall stability, the calculated passive resistance of soil in front of abutments and conventional walls shall be sufficient to prevent unacceptable forward movement of the wall.

The passive resistance shall be neglected if the soil providing passive resistance is, or is likely to become soft, loose, or disturbed, or if the contact between the soil and wall is not tight.

#### C11.6.3.5

Unacceptable deformations may occur before passive resistance is mobilized. Approximate deformations required to mobilize passive resistance are discussed in Article C3.11.1, where $H$ in Table C3.11.1-1 is the effective depth of passive restraint.

#### 11.6.3.6 Sliding

The provisions of Article 10.6.3.4 shall apply.

### 11.6.4 Safety Against Structural Failure

The structural design of individual wall elements and wall foundations shall comply with the provisions of Sections 5, 6, 7, and 8.

The provisions of Article 10.6.1.3 shall be used to determine the distribution of contact pressure for structural design of footings.

### 11.6.5 Seismic Design

The effect of earthquake loading on multi-span bridges shall be investigated using the extreme event limit state of Table 3.4.1-1 with resistance factors $\phi = 1.0$, an accepted methodology in Article 4.7.4.3, and the provisions of Article 3.10.9.2, 3.10.9.3, or 3.10.9.4, as appropriate.

Earthquake loading on single-span bridges shall be investigated in accordance with Articles 4.7.4.2 and 3.10.9.1.

For foundations on soil and rock, the location of the resultant of the reaction forces shall be within the middle two-thirds of the base for $\gamma_{EQ} = 0.0$ and within the middle eight-tenths of the base for $\gamma_{EQ} = 1.0$.

### C11.6.5

In general, the pseudo-static approach developed by Mononobe and Okabe may be used to estimate the equivalent static forces for seismic loads for gravity and semigravity retaining walls. The estimation of seismic design forces should account for wall inertia forces in addition to the equivalent static-forces. For flexible cantilevered walls, forces resulting from wall inertia effects may be ignored in estimating the seismic design forces. Where a wall supports a bridge structure, the seismic design forces should also include seismic forces transferred from the bridge through bearing supports which do not freely slide, e.g., elastomeric bearings. Refer to Appendix A11.

For values of $\gamma_{EQ}$ between 0.0 and 1.0, the restrictions of the location of the resultant shall be obtained by linear interpolation of the values given in this Article.

Where all of the following conditions are met, seismic lateral loads may be reduced as provided in Article C11.6.5, as a result of lateral wall movement due to sliding, from values determined using the Mononobe-Okabe method specified in Appendix A11, Article A11.1.1.1:

- the wall system and any structures supported by the wall can tolerate lateral movement resulting from sliding of the structure.
- the wall base is unrestrained against sliding, other than soil friction along its base and minimal soil passive resistance.
- If the wall functions as an abutment, the top of the wall must also be unrestrained, e.g., the superstructure is supported by sliding bearings.

For overall stability of the retaining wall when earthquake loading is included, a resistance factor, $\phi$, of 0.9 shall be used.

Procedures reducing seismic load due to lateral wall movement are provided in Article A11.1.1.2. In general, this reduction only applies to gravity and semigravity walls. Though the specifications in Article A11.1.1.2 relate to gravity and semigravity walls, these provisions may also apply to other types of walls provided the three conditions listed in Article 11.6.5 are met.

Kavazanjian et al., (*1997*) further simplified the relationship provided in Eq. A11.1.1.2-1 of Appendix A11 as follows, assuming that the velocity, in the absence of information on the time history of the ground motion, is equal to $30A$:

$$k_h = 0.74\,A\left(\frac{A}{d}\right)^{0.25} \qquad \text{(C11.6.5-1)}$$

where:

$A$ = the maximum earthquake acceleration (dim.)

$k_h$ = horizontal seismic acceleration coefficient (dim.)

$d$ = the lateral wall displacement (in.)

This equation should not be used for displacements of less than 1.0 in. or greater than approximately 8.0 in., as this equation is an approximation of a more rigorous Newmark analysis. In general, typical practice among states located in seismically active areas is to design walls for reduced seismic pressures corresponding to 2.0 in. to 4.0 in. of displacement. However, the amount of deformation which is tolerable will depend on the nature of the wall and what it supports, as well as what is in front of the wall.

In addition to whether or not the wall can tolerate lateral deformation, it is recommended that this simplified approach not be used for walls which have a complex geometry, such as stacked walls, MSE walls with trapezoidal sections, or back-to-back walls supporting narrow ramps, for walls which are very tall (over 50.0 ft.), nor for walls where the peak ground acceleration $A$ is $0.3g$ or higher. In such cases, a specialist should be retained to evaluate the anticipated deformation response of the structure, as potentially unacceptable permanent lateral and vertical wall deformations could occur even if design criteria based on this pseudo static approach are met.

### 11.6.6 Drainage

Backfills behind abutments and retaining walls shall be drained or, if drainage cannot be provided, the abutment or wall shall be designed for loads due to earth pressure, plus full hydrostatic pressure due to water in the backfill.

### C11.6.6

Weep holes or geocomposite panel drains at the wall face do not assure fully drained conditions. Drainage systems should be designed to completely drain the entire retained soil volume behind the retaining wall face.

## 11.7 PIERS

### 11.7.1 Load Effects in Piers

Piers shall be designed to transmit the loads on the superstructure, and the loads acting on the pier itself, onto the foundation. The loads and load combinations shall be as specified in Section 3.

The structural design of piers shall be in accordance with the provisions of Sections 5, 6, 7, and 8, as appropriate.

### 11.7.2 Pier Protection

#### 11.7.2.1 Collision

Where the possibility of collision exists from highway or river traffic, an appropriate risk analysis should be made to determine the degree of impact resistance to be provided and/or the appropriate protection system. Collision loads shall be determined as specified in Articles 3.6.5 and 3.14.

#### 11.7.2.2 Collision Walls

Collision walls may be required by railroad owners if the pier is in close proximity to the railroad.

#### C11.7.2.2

Collision walls are usually required by the railroad owner if the column is within 25.0 ft. of the rail. Some railroad owners require a collision wall 6.5 ft. above the top of the rail between columns for railroad overpasses.

#### 11.7.2.3 Scour

The scour potential shall be determined and the design shall be developed to minimize failure from this condition as specified in Article 2.6.4.4.2.

#### 11.7.2.4 Facing

Where appropriate, the pier nose should be designed to effectively break up or deflect floating ice or drift.

#### C11.7.2.4

In these situations, pier life can be extended by facing the nosing with steel plates or angles, and by facing the pier with granite.

## 11.8 NONGRAVITY CANTILEVERED WALLS

### 11.8.1 General

Nongravity cantilevered walls may be considered for temporary and permanent support of stable and unstable soil and rock masses. The feasibility of using a nongravity cantilevered wall at a particular location shall be based on the suitability of soil and rock conditions within the depth of vertical element embedment to support the wall.

### C11.8.1

Depending on soil conditions, nongravity cantilevered walls less than about 10.0 to 15.0 ft. in height are usually feasible, with the exception of cylinder or tangent pile walls, where greater heights can be used.

### 11.8.2 Loading

The provisions of Article 11.6.1.2 shall apply. The load factor for lateral earth pressure (*EH*) shall be applied to the lateral earth pressures for the design of nongravity cantilevered walls.

### C11.8.2

Lateral earth pressure distributions for design of nongravity cantilevered walls are provided in Article 3.11.5.6.

## 11.8.3 Movement and Stability at the Service Limit State

### 11.8.3.1 Movement

The provisions of Articles 10.7.2 and 10.8.2 shall apply. The effects of wall movements on adjacent facilities shall be considered in the selection of the design earth pressures in accordance with the provisions of Article 3.11.1.

### C11.8.3.1

Table C3.11.1-1 provides approximate magnitudes of relative movements required to achieve active earth pressure conditions in the retained soil and passive earth pressure conditions in the resisting soil.

### 11.8.3.2 Overall Stability

The provisions of Article 11.6.2.3 shall apply.

### C11.8.3.2

Use of vertical wall elements to provide resistance against overall stability failure is described in Article C11.9.3.2. Discrete vertical elements penetrating across deep failure planes can provide resistance against overall stability failure. The magnitude of resistance will depend on the size, type, and spacing of the vertical elements.

## 11.8.4 Safety Against Soil Failure at the Strength Limit State

### 11.8.4.1 Overall Stability

The provisions of Article 11.6.2.3 shall apply.

The provisions of Article 11.6.3.5 shall apply.

Vertical elements shall be designed to support the full design earth, surcharge and water pressures between the elements. In determining the embedment depth to mobilize passive resistance, consideration shall be given to planes of weakness, e.g., slickensides, bedding planes, and joint sets that could reduce the strength of the soil or rock determined by field or laboratory tests. Embedment in intact rock, including massive to appreciably jointed rock which should not fail through a joint surface, shall be based on the shear strength of the rock mass.

### C11.8.4.1

Discrete vertical elements penetrating across deep failure planes can provide resistance against failure. The magnitude of resistance will depend on the size, type, and spacing of vertical elements.

The maximum spacing between vertical supporting elements depends on the relative stiffness of the vertical elements. Spans of 6.0 to 10.0 ft. are typical, depending on the type and size of facing.

In determining the embedment depth of vertical wall elements, consideration should be given to the presence of planes of weakness in the soil or rock that could result in a reduction of passive resistance. For laminated, jointed, or fractured soils and rocks, the residual strength along planes of weakness should be considered in the design and, where the planes are oriented at other than an angle of $(45° - \phi'_f/2)$ from the horizontal in soil or 45° from the horizontal in rock toward the excavation, the orientation of the planes should also be considered. Where the wall is located on a bench above a deeper excavation, consideration should be given to the potential for bearing failure of a supporting wedge of soil or rock through intact materials along planes of weakness.

In designing permanent nongravity cantilevered walls with continuous vertical elements, the simplified earth pressure distributions in Figure 3.11.5.6-3 may be used with the following procedure (*Teng 1962*):

- Determine the magnitude of lateral pressure on the wall due to earth pressure, surcharge loads and differential water pressure over the design height of the wall using $k_{af}$.

- Determine the magnitude of lateral pressure on the wall due to earth pressure, surcharge loads and differential water pressure over the design height of the wall using $k_{a2}$.

- Determine in the following equation the value $x$ as defined in Figure 3.11.5.6-3 to determine the distribution of net passive pressure in front of the wall below the design height:

$$x = \left[\gamma k_{a2} \gamma'_{s1} H\right] / \left[\left(\phi k_{p2} - \gamma k_{a2}\right) \gamma'_{s2}\right] \tag{C11.8.4.1-1}$$

where:

$\gamma$ = load factor for horizontal earth pressure, $EH$ (dim.)

$k_{a2}$ = the active earth pressure coefficient for soil 2 (dim.)

$\gamma'_{s1}$ = the effective soil unit weight for soil 1 (kcf)

$H$ = the design height of the wall (ft.)

$\phi$ = the resistance factor for passive resistance in front of the wall (dim.)

$k_{p2}$ = the passive earth pressure coefficient for soil 2 (dim.)

$\gamma'_{s2}$ = the effective soil unit weight for soil 2 (kcf)

- Sum moments about the point of action of $F$ (the base of the wall) to determine the embedment ($D_o$) for which the net passive pressure is sufficient to provide moment equilibrium.

- Determine the depth at which the shear in the wall is zero, i.e., the point at which the areas of the driving and resisting pressure diagrams are equivalent.

- Calculate the maximum bending moment at the point of zero shear.

- Calculate the design depth, $D$=1.2$D_o$, to account for errors inherent in the simplified passive pressure distribution.

### 11.8.5 Safety Against Structural Failure

#### 11.8.5.1 Vertical Wall Elements

Vertical wall elements shall be designed to resist all horizontal earth pressure, surcharge, water pressure, and earthquake loadings.

#### C11.8.5.1

Discrete vertical wall elements include driven piles, drilled shafts, and auger-cast piles, i.e., piles and built-up sections installed in preaugered holes.

Continuous vertical wall elements are continuous throughout both their length and width, although vertical joints may prevent shear and/or moment transfer between adjacent sections. Continuous vertical wall elements include sheet piles, precast or cast-in-place concrete diaphragm wall panels, tangent-piles, and tangent drilled shafts.

The maximum bending moments and shears in vertical wall elements may be determined using the loading diagrams in Article 3.11.5.6, and appropriate load and resistance factors.

#### 11.8.5.2 Facing

The maximum spacing between discrete vertical wall elements shall be determined based on the relative stiffness of the vertical elements and facing, the type and condition of soil to be supported, and the type and condition of the soil in which the vertical wall elements are embedded. Facing may be designed assuming simple support between elements, with or without soil arching, or assuming continuous support over several elements.

If timber facing is used, it shall be stress-grade pressure-treated lumber in conformance with Section 8. If timber is used where conditions are favorable for the growth of decay-producing organisms, wood should be pressure-treated with a wood preservative unless the heartwood of a naturally decay-resistant species is available and is considered adequate with respect to the decay hazard and expected service life of the structure.

#### C11.8.5.2

In lieu of other suitable methods, for preliminary design the maximum bending moments in facing may be determined as follows:

- For simple spans without soil arching:

$$M_{max} = 0.125pL^2 \qquad \text{(C11.8.5.2-1)}$$

- For simple spans with soil arching:

$$M_{max} = 0.083pL^2 \qquad \text{(C11.8.5.2-2)}$$

- For continuous spans without soil arching:

$$M_{max} = 0.1pL^2 \qquad \text{(C11.8.5.2-3)}$$

- For continuous spans with soil arching:

$$M_{max} = 0.083pL^2 \qquad \text{(C11.8.5.2-4)}$$

where:

$M_{max}$ = factored flexural moment on a unit width or height of facing (kip-ft./ft.)

$p$ = average factored lateral pressure, including earth, surcharge and water pressure acting on the section of facing being considered (ksf/ft.)

$L$ = spacing between vertical elements or other facing supports (ft.)

If the variations in lateral pressure with depth are large, moment diagrams should be constructed to provide more accuracy. The facing design may be varied with depth.

Eq. C1 is applicable for simply supported facing behind which the soil will not arch between vertical supports, e.g., in soft cohesive soils or for rigid concrete facing placed tightly against the in-place soil. Eq. C2 is applicable for simply supported facing behind which the soil will arch between vertical supports, e.g., in granular or stiff cohesive soils with flexible facing or rigid facing behind which there is sufficient space to permit the in-place soil to arch. Eqs. C3 and C4 are applicable for facing which is continuous over several vertical supports, e.g., reinforced shotcrete or concrete.

### 11.8.6 Seismic Design

The effect of earthquake loading shall be investigated using the Extreme Event I limit state of Table 3.4.1-1 with resistance factor $\phi$=1.0 and load factor $\gamma_p$ =1.0 and an accepted methodology.

### C11.8.6

In general, the pseudo-static approach developed by Mononobe and Okabe may be used to estimate the equivalent static forces provided the maximum lateral earth pressure, active and passive are computed using a seismic coefficient $k_h$=0.5$A$. Forces resulting from wall inertia effects may be ignored in estimating the seismic lateral earth pressure. Refer to Appendix A.

### 11.8.7 Corrosion Protection

The level and extent of corrosion protection shall be a function of the ground environment and the potential consequences of a wall failure.

### C11.8.7

Corrosion protection for piles and miscellaneous hardware and material should be consistent with the design life of the structure.

### 11.8.8 Drainage

The provisions of Article 3.11.3 shall apply.

Seepage shall be controlled by installation of a drainage medium behind the facing with outlets at or near the base of the wall. Drainage panels shall maintain their drainage characteristics under the design earth pressures and surcharge loadings, and shall extend from the base of the wall to a level 1.0 ft. below the top of the wall.

Where thin drainage panels are used behind walls, and saturated or moist soil behind the panels may be subjected to freezing and expansion, either insulation shall be provided on the walls to prevent freezing of the soil, or the wall shall be designed for the pressures exerted on the wall by frozen soil.

### C11.8.8

In general, the potential for development of hydrostatic pressures behind walls with discrete vertical elements and lagging is limited due to the presence of openings in the lagging, and the disturbance of soil behind lagging as the wall is constructed. However, the potential for leakage through the wall should not be counted upon where the ground water level exceeds one-third the height of the wall because of the potential for plugging and clogging of openings in the wall with time by migration of soil fines. It is probable that, under such conditions, a wall with continuous vertical elements, i.e., a cutoff wall constructed with a drainage system designed to handle anticipated flows will be required.

Water pressures may be considered reduced in design only if positive drainage, e.g., drainage blanket, geocomposite drainage panels, gravel drains with outlet pipes is provided to prevent buildup of hydrostatic pressure behind the wall. Thin drains at the back of the wall face may not completely relieve hydrostatic pressure and may increase seepage forces on the back of the wall face due to rainwater infiltration, Terzaghi and Peck (*1967*), and Cedergren (*1989*). The effectiveness of drainage control measures should be evaluated by seepage analyses.

## 11.9 ANCHORED WALLS

### 11.9.1 General

Anchored walls, whose elements may be proprietary, employ grouted in anchor elements, vertical wall elements and facing.

Anchored walls, illustrated in Figure 1, may be considered for both temporary and permanent support of stable and unstable soil and rock masses.

The feasibility of using an anchored wall at a particular location should be based on the suitability of subsurface soil and rock conditions within the bonded anchor stressing zone.

Where fill is placed behind a wall, either around or above the unbonded length, special designs and construction specifications shall be provided to prevent anchor damage.

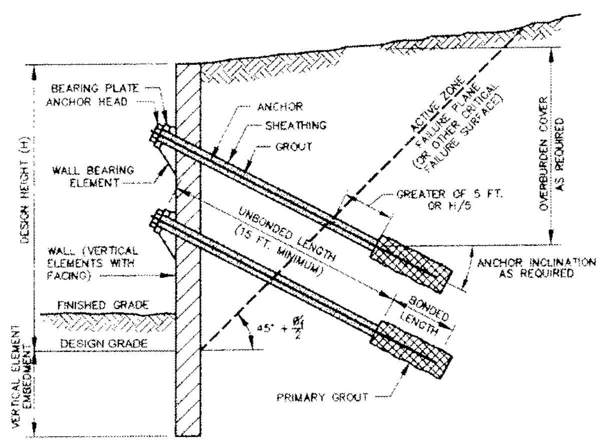



**Figure 11.9.1-1 Anchored Wall Nomenclature and Anchor Embedment Guidelines.**

### C11.9.1

Depending on soil conditions, anchors are usually required for support of both temporary and permanent nongravity cantilevered walls higher than about 10.0 to 15.0 ft.

The availability or ability to obtain underground easements and proximity of buried facilities to anchor locations should also be considered in assessing feasibility.

Anchored walls in cuts are typically constructed from the top of the wall down to the base of the wall. Anchored walls in fill must include provisions to protect against anchor damage resulting from backfill and subsoil settlement or backfill and compaction activities above the anchors.

The minimum distance between the front of the bond zone and the active zone behind the wall of 5.0 ft. or $H/5$ is needed to insure that no load from the bonded zone is transferred into the no load zone due to load transfer through the grout column in the no load zone.

### 11.9.2 Loading

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

### C11.9.2

Lateral earth pressures on anchored walls are a function of the rigidity of the wall-anchor system, soil conditions, method and sequence of construction, and level of prestress imposed by the anchors. Apparent earth pressure diagrams that are commonly used can be found in Article 3.11.5.7 and Sabatini et al. (*1999*).

## 11.9.3 Movement and Stability at the Service Limit State

### 11.9.3.1 Movement

The provisions of Articles 10.6.2, 10.7.2, and 10.8.2 shall apply.

The effects of wall movements on adjacent facilities shall be considered in the development of the wall design.

### C11.9.3.1

Settlement of vertical wall elements can cause reduction of anchor loads, and should be considered in design.

The settlement profiles in Figure C1 were recommended by Clough and O'Rourke (*1990*) to estimate ground surface settlements adjacent to braced or anchored excavations caused during the excavation and bracing stages of construction. Significant settlements may also be caused by other construction activities, e.g., dewatering or deep foundation construction within the excavation, or by poor construction techniques, e.g., soldier pile, lagging, or anchor installation. The field measurements used to develop Figure C1 were screened by the authors to not include movements caused by other construction activities or poor construction techniques. Therefore, such movements should be estimated separately.

Where noted in the definition of the various curves in Figure C1, the basal heave ratio, $R_{BH}$, shall be taken as:

$$R_{BH} = \frac{5.1S_u}{\gamma_s H + q_s} \qquad \text{(C11.9.3.1-1)}$$

where:

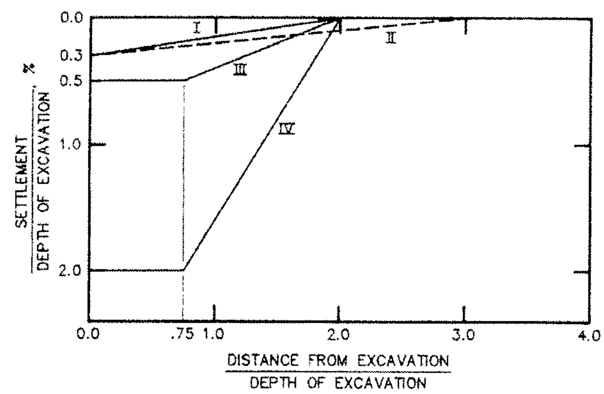
$S_u$ = undrained shear strength of cohesive soil (ksf)

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

$H$ = height of wall (ft.)

$q_s$ = surcharge pressure (ksf)

See Sabatini et al. (*1999*) for additional information on the effect of anchored wall construction and design on wall movement.




| | | |
|---|---|---|
| Curve I | = | Sand |
| Curve II | = | Stiff to very hard clay |
| Curve III | = | Soft to medium clay, $R_{BH} = 2.0$ |
| Curve IV | = | Soft to medium clay, $R_{BH} = 1.2$ |

**Figure C11.9.3.1-1 Settlement Profiles Behind Braced or Anchored Walls (adapted from *Clough and O'Rourke, 1990*).**

#### 11.9.3.2 Overall Stability

The provisions of Article 11.6.2.3 shall apply.

#### C11.9.3.2

Detailed guidance for evaluating the overall stability of anchored wall systems, including how to incorporate anchor forces in limit equilibrium slope stability analyses, is provided by Sabatini et al. (*1999*).

The effect of discrete vertical elements penetrating deep failure planes and acting as in-situ soil improvement may be negligible if the percentage of reinforcement provided by the vertical elements along the failure surface is small. However, it is possible to consider the effect of the discrete vertical elements by modeling the elements as a cohesion along the failure surface, or by evaluating the passive capacity of the elements.

### 11.9.4 Safety Against Soil Failure

#### 11.9.4.1 Bearing Resistance

The provisions of Articles 10.6.3, 10.7.3, and 10.8.3 shall apply.

Bearing resistance shall be determined assuming that all vertical components of loads are transferred to the embedded section of the vertical wall elements.

#### C11.9.4.1

For drilled in place vertical wall elements, e.g., drilled-in soldier piles, in sands, if the β-method is used to calculate the skin friction capacity, the depth $z$ should be referenced to the top of the wall. The vertical overburden stress, $\sigma_v'$, however, should be calculated with reference to the elevation of the midheight of the exposed wall, with β and $\sigma_v'$ evaluated at the midpoint of each soil layer.

### 11.9.4.2 Anchor Pullout Capacity

Prestressed anchors shall be designed to resist pullout of the bonded length in soil or rock. The factored pullout resistance of a straight shaft anchor in soil or rock, $Q_R$, is determined as:

$$Q_R = \phi Q_n = \phi \pi d \tau_a L_b \tag{11.9.4.2-1}$$

where:

$\phi$ = resistance factor for anchor pullout (dim.)

$Q_n$ = nominal anchor pullout resistance (kips)

$d$ = diameter of anchor drill hole (ft.)

$\tau_n$ = nominal anchor bond stress (ksf)

$L_b$ = anchor bond length (ft.)

For preliminary design, the resistance of anchors may either be based on the results of anchor pullout load tests; estimated based on a review of geologic and boring data, soil and rock samples, laboratory testing and previous experience; or estimated using published soil/rock-grout bond guidelines. For final design, the contract documents may require preproduction tests such as pullout tests or extended creep tests on sacrificial anchors be conducted to establish anchor lengths and capacities that are consistent with the contractor's chosen method of anchor installation. Either performance or proof tests shall be conducted on every production anchor to 1.0 or greater times the factored load to verify capacity.

### C11.9.4.2

Anchor pullout capacity is influenced by soil and rock conditions, method of anchor hole advancement, hole diameter, bonded length, grout type and grouting pressure. Information on anchor pullout capacity may be found in Sabatini et al. (*1999*), PTI (*1996*), Cheney (*1984*) and Weatherby (*1982*). As a guide, the presumptive values provided in Tables C1, C2, and C3 may be used to estimate the nominal (ultimate) bond for small diameter anchors installed in cohesive soils, cohesionless soils and rock, respectively. It should be recognized that the values provided in the tables may be conservative.

**Table C11.9.4.2-1 Presumptive Ultimate Unit Bond Stress for Anchors in Cohesive Soils.**

| Anchor/Soil Type (Grout Pressure) | Soil Stiffness or Unconfined Compressive Strength (tsf) | Presumptive Ultimate Unit Bond Stress, $\tau_n$ (ksf) |
|---|---|---|
| Gravity Grouted Anchors (<50 psi) | | |
| Silt-Clay Mixtures | Stiff to Very Stiff 1.0–4.0 | 0.6 to 1.5 |
| Pressure Grouted Anchors (50 psi–400 psi) | | |
| High Plasticity Clay | Stiff 1.0–2.5<br>V. Stiff 2.5–4.0 | 0.6 to 2<br>1.5 to 3.6 |
| Medium Plasticity Clay | Stiff 1.0–2.5<br>V. Stiff 2.5–4.0 | 2.0 to 5.2<br>2.9 to 7.3 |
| Medium Plasticity Sandy Silt | V. Stiff 2.5–4.0 | 5.8 to 7.9 |

**Table C11.9.4.2-2 Presumptive Ultimate Unit Bond Stress for Anchors in Cohesionless Soils.**

| Anchor/Soil Type (Grout Pressure) | Soil Compactness or SPT Resistance[1] | Presumptive Ultimate Unit Bond Stress, $\tau_n$ (ksf) |
|---|---|---|
| Gravity Grouted Anchors (<50 psi) | | |
| Sand or Sand-Gravel Mixtures | Medium Dense to Dense 11–50 | 1.5 to 2.9 |
| Pressure Grouted Anchors (50 psi–400 psi) | | |
| Fine to Medium Sand | Medium Dense to Dense 11–50 | 1.7 to 7.9 |
| Medium to Coarse Sand w/ Gravel | Medium Dense 11–30 | 2.3 to 14 |
| | Dense to Very Dense 30–50 | 5.2 to 20 |
| Silty Sands | — | 3.5 to 8.5 |
| Sandy Gravel | Medium Dense to Dense 11–40 | 4.4 to 29 |
| | Dense to Very Dense 40–50+ | 5.8 to 29 |
| Glacial Till | Dense 31–50 | 6.3 to 11 |

[1] Corrected for overburden pressure.

**Table C11.9.4.2-3 Presumptive Ultimate Unit Bond Stress for Anchors in Rock.**

| Rock Type | Presumptive Ultimate Unit Bond Stress, $\tau_n$ (ksf) |
|---|---|
| Granite or Basalt | 36 to 65 |
| Dolomitic Limestone | 29 to 44 |
| Soft Limestone | 21 to 29 |
| Slates & Hard Shales | 17 to 29 |
| Sandstones | 17 to 36 |
| Weathered Sandstones | 15 to 17 |
| Soft Shales | 4.2 to 17 |

The presumptive ultimate anchor bond stress values presented in Tables C1 through C3 are intended for preliminary design or evaluation of the feasibility of straight shaft anchors installed in small diameter holes. Pressure-grouted anchors may achieve much higher capacities. The total capacity of a pressure-grouted anchor may exceed 500 kips in soil or 2000 to 3000 kips in rock, although such high capacity anchors are seldom used for highway applications. Post-grouting can also increase the load carrying capacity of straight shaft anchors by 20–50 percent or more per phase of post-grouting.

The resistance factors in Table 11.5.6-1, in combination with the load factor for horizontal active earth pressure (Table 3.4.1-2), are consistent with what would be required based on allowable stress design, for preliminary design of anchors for pullout (*Sabatini et al., 1999*). These resistance factors are also consistent with the results of statistical calibration of full scale anchor pullout tests relative to the minimum values of presumptive ultimate unit bond stresses shown in Tables C1 through C3. Use of the resistance factors in Table 11.5.6-1 and the load factor for apparent earth pressure for anchor walls in Table 3.4.1-2, with values of presumptive ultimate unit bond stresses other than the minimum values in Tables C1 through C3 could result in unconservative designs unless the Engineer has previous experience with the particular soil or rock unit in which the bond zone will be established.

Presumptive bond stresses greater than the minimum values shown in Tables C1 through C3 should be used with caution, and be based on past successful local experience, such as a high percentage of passing proof tests in the specified or similar soil or rock unit at the design bond stress chosen, or anchor pullout test results in the specified or similar soil or rock unit. Furthermore, in some cases the specified range of presumptive bond stresses is representative of a range of soil conditions. Soil conditions at the upper end of the specified range, especially if coupled with previous experience with the particular soil unit, may be considered in the selection of anchor bond stresses above the minimum values shown. Selection of a presumptive bond stress for preliminary anchor sizing should consider the risk of failing proof tests if the selected bond stress was to be used for final design. The goal of preliminary anchor design is to reduce the risk of having a significant number of production anchors fail proof or performance tests as well as the risk of having to redesign the anchored wall to accommodate more anchors due to an inadequate easement behind the wall, should the anchor capacities predicted during preliminary design not be achievable.

See Article 11.9.8.1 for guidance on anchor testing.

The anchor load shall be developed by suitable embedment outside of the critical failure surface in the retained soil mass.

Determination of the unbonded anchor length, inclination, and overburden cover shall consider:

- The location of the critical failure surface furthest from the wall,
- The minimum length required to ensure minimal loss of anchor prestress due to long-term ground movements,
- The depth to adequate anchoring strata, as indicated in Figure 11.9.1-1, and
- The method of anchor installation and grouting.

The minimum horizontal spacing of anchors should be the larger of three times the diameter of the bonded zone, or 5.0 ft. If smaller spacings are required to develop the required load, consideration may be given to differing anchor inclinations between alternating anchors.

Significant increases in anchor capacity for anchor bond lengths greater than approximately 40.0 ft. cannot be achieved unless specialized methods are used to transfer load from the top of the anchor bond zone towards the end of the anchor. This is especially critical for strain sensitive soils, in which residual soil strength is significantly lower than the peak soil strength.

Anchor inclination and spacing will be controlled by soil and rock conditions, the presence of geometric constraints and the required anchor capacity. For tremie-grouted anchors, a minimum angle of inclination of about 10° and a minimum overburden cover of about 15.0 ft. are typically required to assure grouting of the entire bonded length and to provide sufficient ground cover above the anchorage zone. For pressure-grouted anchors, the angle of inclination is generally not critical and is governed primarily by geometric constraints, and the minimum overburden cover is typically 6.0–15.0 ft. Steep inclinations may be required to avoid anchorage in unsuitable soil or rock. Special situations may require horizontal or near horizontal anchors, in which case proof of sufficient overburden and full grouting should be required.

The minimum horizontal spacing specified for anchors is intended to reduce stress overlap between adjacent anchors.

Anchors used for walls constructed in fill situations, i.e., bottom-up construction, should be enclosed in protective casing to prevent damage during backfill placement, compaction and settlement.

Selection of anchor type depends on anticipated service life, soil and rock conditions, ground water level, subsurface environmental conditions, and method of construction.

#### 11.9.4.3 Passive Resistance

The provisions of Articles 11.6.3.5, 11.6.3.6, and 11.8.4.1 shall apply.

#### C11.9.4.3

It is recommended in Sabatini et al. (*1999*) that methods such as the Broms Method or the Wang and Reese method be used to evaluate passive resistance and the wall vertical element embedment depth needed. However, these methods have not been calibrated for this application for LRFD as yet.

## 11.9.5 Safety Against Structural Failure

### 11.9.5.1 Anchors

The horizontal component of anchor design force shall be computed using the provisions of Article 11.9.2 and any other horizontal pressure components acting on the wall in Article 3.11. The total anchor design force shall be determined based on the anchor inclination. The horizontal anchor spacing and anchor capacity shall be selected to provide the required total anchor design force.

### C11.9.5.1

Anchor tendons typically consist of steel bars, wires or strands. The selection of anchor type is generally the responsibility of the contractor.

A number of suitable methods for the determination of anchor loads are in common use. Sabatini et al. (*1999*) provides two methods which can be used: the Tributary Area Method, and the Hinge Method. These methods are illustrated in Figures C1 and C2. These figures assume that the soil below the base of the excavation has sufficient strength to resist the reaction force *R*. If the soil providing passive resistance below the base of the excavation is weak and is inadequate to carry the reaction force *R*, the lowest anchor should be designed to carry both the anchor load as shown in the figures as well as the reaction force. See Article 11.8.4.1 for evaluation of passive resistance. Alternatively, soil-structure interaction analyses, e.g., beam on elastic foundation, can be used to design continuous beams with small toe reactions, as it may be overly conservative to assume that all of the load is carried by the lowest anchor.

In no case should the maximum test load be less than the factored load for the anchor.

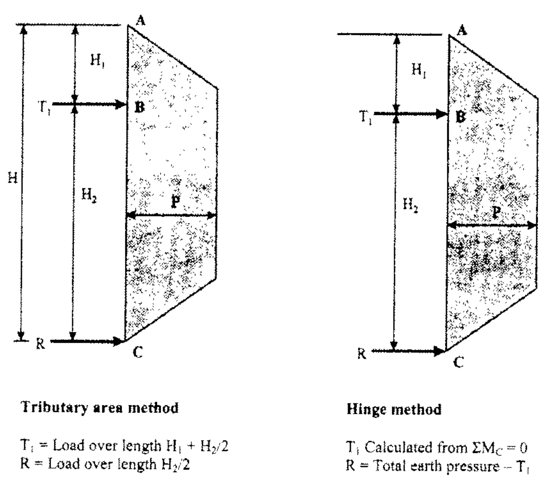
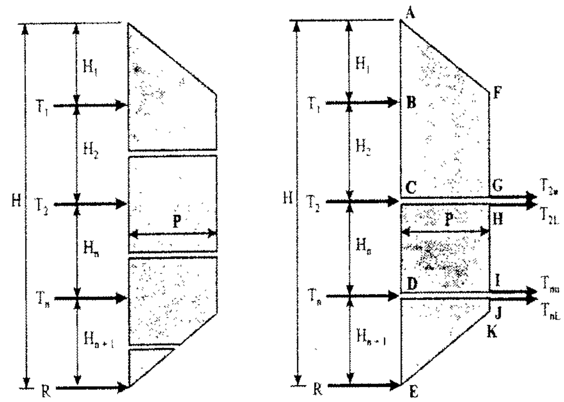



**Figure C11.9.5.1-1 Calculation of Anchor Loads for One-Level Wall after Sabatini et. al. (*1999*).**




**Tributary area method**

$T_1$ = Load over length $H_1 + H_2/2$
$T_2$ = Load over length $H_2/2 + H_n/2$
$T_n$ = Load over length $H_n/2 + H_{n+1}/2$
R = Load over length $H_{n+1}/2$

**Hinge method**

$T_1$ Calculated from $\Sigma M_C = 0$
$T_{2u}$ = Total earth pressure (ABCGF) – $T_1$
$T_{2L}$ = Calculated from $\Sigma M_D = 0$
$T_{nu}$ = Total earth pressure (CDIH) – $T_{2L}$
$T_{nL}$ = Calculated from $\Sigma M_E = 0$
R = Total earth pressure – $T_1 - T_2 - T_n$
$T_2 = T_{2u} = T_{2L}$
$T_n = T_{nu} + T_{nL}$

**Figure C11.9.5.1-2 Calculation of Anchor Loads for Multilevel Wall after Sabatini et. al. (*1999*).**

#### 11.9.5.2 Vertical Wall Elements

Vertical wall elements shall be designed to resist all horizontal earth pressure, surcharge, water pressure, anchor, and seismic loadings, as well as the vertical component of the anchor loads and any other vertical loads. Horizontal supports may be assumed at each anchor location and at the bottom of the excavation if the vertical element is sufficiently embedded below the bottom of the excavation.

#### C11.9.5.2

Discrete vertical wall elements are continuous throughout their length and include driven piles, caissons, drilled shafts, and auger-cast piles, i.e., piles and built-up sections installed in preaugured holes and backfilled with structural concrete in the passive zone and lean concrete in the exposed section of the wall.

Continuous vertical wall elements are continuous throughout both their length and width, although vertical joints may prevent shear and/or moment transfer between adjacent sections. Continuous vertical wall elements include sheet piles, precast or cast-in-place concrete diaphragm wall panels, tangent-piles, and tangent caissons.

For structural analysis methods, see Section 4.

For walls supported in or through soft clays with $S_u < 0.15\gamma_s'H$, continuous vertical elements extending well below the exposed base of the wall may be required to prevent heave in front of the wall. Otherwise, the vertical elements are embedded approximately 3.0 ft. or as required for stability or end bearing.

#### 11.9.5.3 Facing

The provisions of Article 11.8.5.2 shall apply.

### 11.9.6 Seismic Design

The provisions of Article 11.8.6 shall apply.

### C11.9.6

See Article C11.8.6.

### 11.9.7 Corrosion Protection

Prestressed anchors and anchor heads shall be protected against corrosion consistent with the ground and groundwater conditions at the site. The level and extent of corrosion protection shall be a function of the ground environment and the potential consequences of an anchor failure. Corrosion protection shall be applied in accordance with the provisions of *AASHTO LRFD Bridge Construction Specifications*, Section 6, "Ground Anchors."

### C11.9.7

Corrosion protection for piles, wales, and miscellaneous hardware and material should be consistent with the level of protection for the anchors and the design life of the structure.

### 11.9.8 Construction and Installation

#### 11.9.8.1 Anchor Stressing and Testing

All production anchors shall be subjected to load testing and stressing in accordance with the provisions of *AASHTO LRFD Bridge Construction Specifications*, Article 6.5.5, "Testing and Stressing." Preproduction load tests may be specified when unusual conditions are encountered to verify the safety with respect to the design load to establish the ultimate anchor load (pullout test), or to identify the load at which excessive creep occurs.

At the end of the testing of each production anchor, the anchor should be locked off to take up slack in the anchored wall system to reduce post-construction wall deformation. The lock-off load should be determined and applied as described in *AASHTO LRFD Bridge Construction Specifications*, Article 6.5.5.6.

#### C11.9.8.1

Common anchor load tests include pullout tests performed on sacrificial preproduction anchors, and creep, performance, and proof tests performed the production anchors. None of the production anchor tests determine the actual ultimate anchor load capacity. The production anchor test results only provide an indication of serviceability under a specified load. Performance tests consist of incremental loading and unloading of anchors to verify sufficient capacity to resist the test load, verify the free length and evaluate the permanent set of the anchor. Proof tests, usually performed on each production anchor, consist of a single loading and unloading cycle to verify sufficient capacity to resist the test load and to prestress the anchor. Creep tests, recommended for cohesive soils with a plasticity index greater than 20 percent or a liquid limit greater than 50 percent, and highly weathered, soft rocks, consist of incremental, maintained loading of anchors to assess the potential for loss of anchor bond capacity due to ground creep.

Pullout tests should be considered in the following circumstances:

- If the preliminary anchor design using unit bond stresses provided in the tables above indicate that anchored walls are marginally infeasible, requiring that a more accurate estimate of anchor capacity be obtained during wall design. This may occur due to lack of adequate room laterally to accommodate the estimated anchor length within the available right-of-way or easement;

- If the anticipated anchor installation method or soil/rock conditions are significantly different than those assumed to develop the presumptive values in Tables C11.9.4.2-1 through C11.4.9.2-3 and inadequate site specific experience is available to make a reasonably accurate estimate of the soil/rock-grout anchor bond stresses.

The FHWA recommends load testing anchors to 125 percent to 150 percent of the unfactored design load, Cheney (*1984*). Maximum load levels between 125 percent and 200 percent have been used to evaluate the potential for tendon overstress in service, to accommodate unusual or variable ground conditions or to assess the effect of ground creep on anchor capacity. Test load levels greater than 150 percent of the unfactored design load are normally applied only to anchors in soft cohesive soil or unstable soil masses where loss of anchor prestress due to creep warrants evaluation. The area of prestressing steel in the test anchor tendon may require being increased to perform these tests.

Note that the test details provided in the *AASHTO LRFD Bridge Construction Specifications*, Article 6.5.5, at least with regard to the magnitude of the incremental test loads, were developed for allowable stress design. These incremental test loads should be divided by the load factor for apparent earth pressure for anchored walls provided in Table 3.4.1-2 when testing to factored anchor loads.

Typically, the anchor lock-off load is equal to 80 to 100 percent of the nominal (unfactored) anchor load to ensure that the slack in the anchored wall system is adequately taken up so that post-construction wall deformation is minimized. However, a minimum lock-off load of 50 percent is necessary to properly engage strand anchor head wedges.

### 11.9.9 Drainage

The provisions of Article 11.8.8 shall apply.

### C11.9.9

Thin drains at the back of the wall face may not completely relieve hydrostatic pressure and may increase seepage forces on the back of the wall face due to rainwater infiltration, Terzaghi and Peck (*1967*), and Cedergren (*1989*). The effectiveness of drainage control measures should be evaluated by seepage analyses.

## 11.10 MECHANICALLY STABILIZED EARTH WALLS

### 11.10.1 General

MSE walls may be considered where conventional gravity, cantilever, or counterforted concrete retaining walls and prefabricated modular retaining walls are considered, and particularly where substantial total and differential settlements are anticipated.

When two intersecting walls form an enclosed angle of 70° or less, the affected portion of the wall shall be designed as an internally tied bin structure with at-rest earth pressure coefficients.

MSE walls shall not be used under the following conditions:

### C11.10.1

Mechanically stabilized earth (MSE) systems, whose elements may be proprietary, employ either metallic (strip or grid type) or geosynthetic (geotextile, strip, or geogrid) tensile reinforcements in the soil mass, and a facing element which is vertical or near vertical. MSE walls behave as a gravity wall, deriving their lateral resistance through the dead weight of the reinforced soil mass behind the facing. For relatively thick facings, the dead weight of the facing may also provide a significant contribution to the capacity of the wall system. Typical MSE walls are shown in Figure C1.

All available data indicates that corrosion in MSE walls is not accelerated by stray currents from electric rail lines due to the discontinuity of the earth reinforcements in a direction parallel to the source of the stray current. Where metallic reinforcements are used in areas of anticipated stray currents within 200 ft. of the structure, and the metallic reinforcements are continuously connected in a direction parallel to the source of stray currents, a corrosion expert should evaluate the potential need for corrosion control requirements. More detailed information on stray current corrosion issues is provided by Sankey and Anderson (*1999*).

- Where utilities other than highway drainage are to be constructed within the reinforced zone unless access is provided to utilities without disrupting reinforcements and breakage or rupture of utility lines will not have a detrimental effect on the stability of the structure.

Where future access to utilities may be gained without disrupting reinforcements and where leakage from utilities would not create detrimental hydraulic conditions or degrade reinforcements, utilities in the reinforced zone may be acceptable.

- Where floodplain erosion or scour may undermine the reinforced fill zone or facing, or any supporting footing.

The potential for catastrophic failure due to scour is high for MSE walls if the reinforced fill is lost during a scour occurrence. Consideration may be given to lowering the base of the wall or to alternative methods of scour protection, such as sheetpile walls and/or riprap of sufficient size, placed to a sufficient depth to preclude scour.

- With reinforcements exposed to surface or ground water contaminated by acid mine drainage, other industrial pollutants, or other environmental conditions defined as aggressive in Article 7.3.6.3 of the *AASHTO LRFD Bridge Construction Specifications*, unless environmental-specific, long-term corrosion, or degradation studies are conducted.

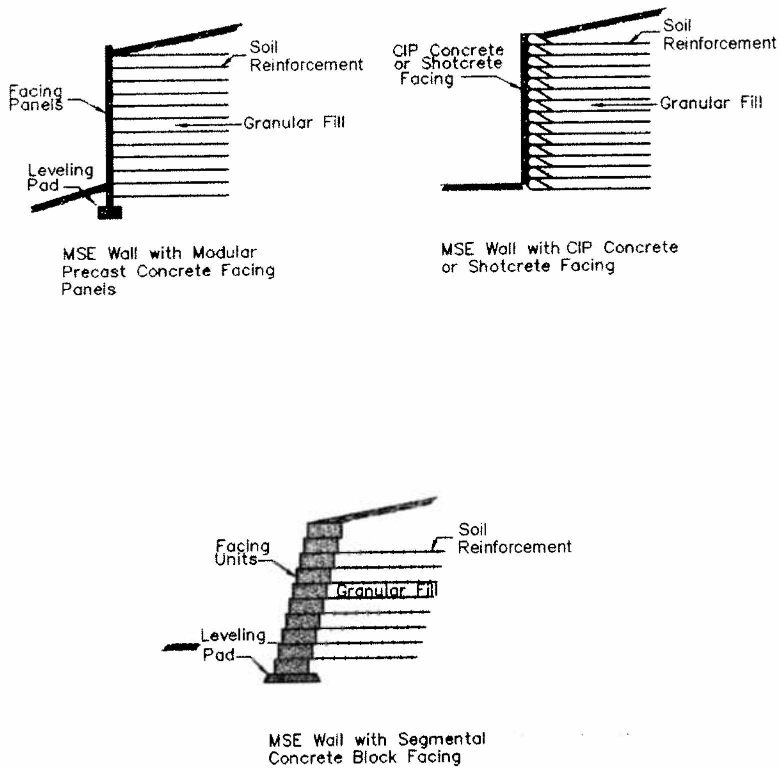



**Figure C11.10.1-1 Typical Mechanically Stabilized Earth Walls.**

MSE walls shall be designed for external stability of the wall system as well as internal stability of the reinforced soil mass behind the facing. Overall and compound stability failure shall be considered. Structural design of the wall facing shall also be considered.

The specifications provided herein for MSE walls do not apply to geometrically complex MSE wall systems such as tiered walls (walls stacked on top of one another), back-to-back walls, or walls which have trapezoidal sections. Design guidelines for these cases are provided in FHWA publication No. FHWA-NHI-00-043 (*Elias et al. 2001*). Compound stability should also be evaluated for these complex MSE wall systems (see Article 11.10.4.3).

For simple structures with rectangular geometry, relatively uniform reinforcement spacing, and a near vertical face, compound failures passing both through the unreinforced and reinforced zones will not generally be critical. However, if complex conditions exist such as changes in reinforced soil types or reinforcement lengths, high surcharge loads, sloping faced structures, a slope at the toe of the wall, or stacked structures, compound failures must be considered.

Internal design of MSE wall systems requires knowledge of short- and long-term properties of the materials used as soil reinforcements as well as the soil mechanics which govern MSE wall behavior.

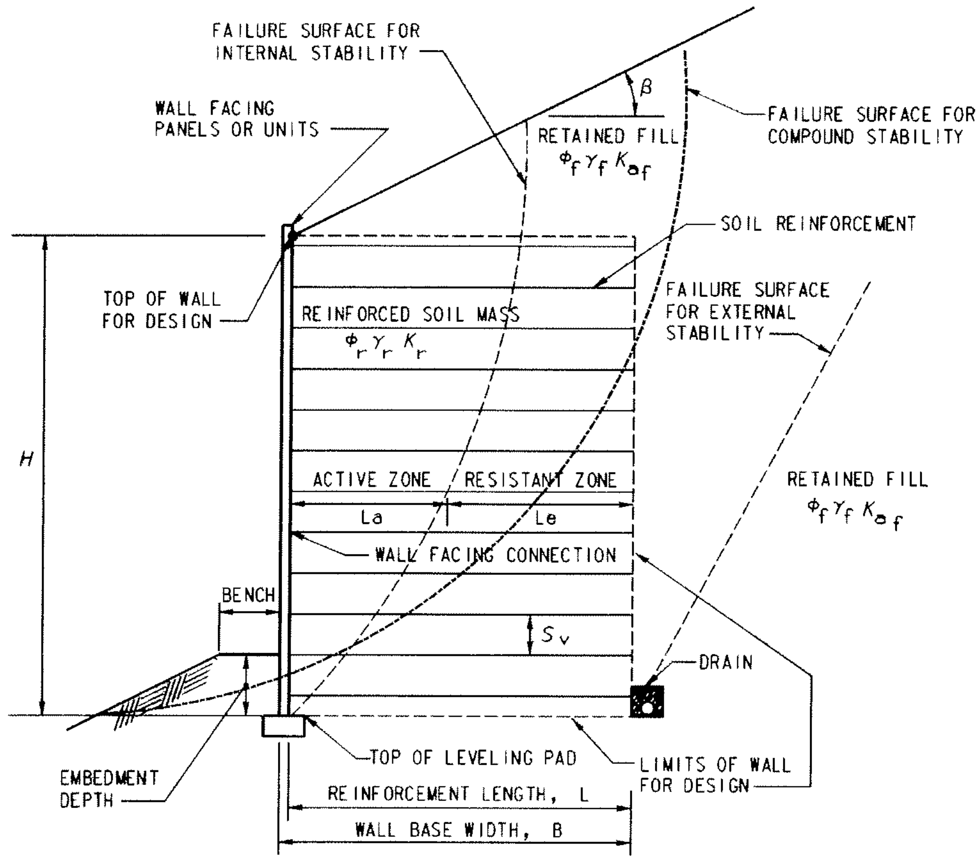
### 11.10.2 Structure Dimensions

An illustration of the MSE wall element dimensions required for design is provided in Figure 1.

The size and embedment depth of the reinforced soil mass shall be determined based on:

- requirements for stability and geotechnical strength, as specified in Article 11.10.5 consistent with requirements for gravity walls,

- requirements for structural resistance within the reinforced soil mass itself, as specified in Article 11.10.6, for the panel units, and for the development of reinforcement beyond assumed failure zones, and
- traditional requirements for reinforcement length not less than 70 percent of the wall height, except as noted in Article 11.10.2.1.




For external and internal stability calculations, the weight and dimensions of the facing elements are typically ignored. However, it is acceptable to include the facing dimensions and weight in sliding and bearing capacity calculations. For internal stability calculations, the wall dimensions are considered to begin at the back of the facing elements.

**Figure 11.10.2-1 MSE Wall Element Dimensions Needed for Design.**

### 11.10.2.1 Minimum Length of Soil Reinforcement

For sheet-, strip-, and grid-type reinforcement, the minimum soil reinforcement length shall be 70 percent of the wall height as measured from the leveling pad. Reinforcement length shall be increased as required for surcharges and other external loads, or for soft foundation soils.

### C11.10.2.1

In general, a minimum reinforcement length of 8.0 ft., regardless of wall height, has been recommended based on historical practice, primarily due to size limitations of conventional spreading and compaction equipment. Shorter minimum reinforcement lengths, on the order of 6.0 ft., but no less than 70 percent of the wall height, can be considered if smaller compaction equipment is used, facing panel alignment can be maintained, and minimum requirements for wall external stability are met.

The requirement for uniform reinforcement length equal to 70 percent of the structure height has no theoretical justification, but has been the basis of many successful designs to-date. Parametric studies considering minimum acceptable soil strengths have shown that structure dimensions satisfying all of the requirements of Article 11.10.5 require length to height ratios varying from $0.8H$ for low structures, i.e., 10.0 ft., to $0.63H$ for high structures, i.e., 40.0 ft.

Significant shortening of the reinforcement elements below the minimum recommended ratio of $0.7H$ may only be considered when accurate, site specific determinations of the strength of the unreinforced fill and the foundation soil have been made. Christopher et al. (*1990*) presents results which strongly suggest that shorter reinforcing length to height ratios, i.e., $0.5H$ to $0.6H$, substantially increase horizontal deformations.

The reinforcement length shall be uniform throughout the entire height of the wall, unless substantiating evidence is presented to indicate that variation in length is satisfactory.

A nonuniform reinforcement length may be considered under the following circumstances:

- Lengthening of the uppermost reinforcement layers to beyond $0.7H$ to meet pullout requirements, or to address seismic or impact loads.
- Lengthening of the lowermost reinforcement layers beyond $0.7H$ to meet overall (global) stability requirements based on the results of a detailed global stability analysis.
- Shortening of the bottom reinforcement layers to less than $0.7H$ to minimize excavation requirements, provided the wall is bearing on rock or very competent foundation soil (see below).

For walls on rock or very competent foundation soil, e.g., SPT > 50, the bottom reinforcements may be shortened to a minimum of $0.4H$ with the upper reinforcements lengthened to compensate for external stability issues in lieu of removing rock or competent soil for construction. Design guidelines for this case are provided in FHWA Publication No. FHWA-NHI-00-043 (*Elias et al. 2001*).

For conditions of marginal stability, consideration must be given to ground improvement techniques to improve foundation stability, or to lengthening of reinforcement.

#### 11.10.2.2 Minimum Front Face Embedment

The minimum embedment depth of the bottom of the reinforced soil mass (top of the leveling pad) shall be based on bearing resistance, settlement, and stability requirements determined in accordance with Section 10.

#### C11.10.2.2

The minimum embedment guidelines provided in Table C1 may be used to preclude local bearing resistance failure under the leveling pad or footing due to higher vertical stresses transmitted by the facing.

Unless constructed on rock foundations, the embedment at the front face of the wall in ft. shall not be less than:

- a depth based on the prevailing depth of frost penetration, if the soil below the wall is frost susceptible, and the external stability requirement, and

- 2.0 ft. on sloping ground (4.0*H*:1*V* or steeper) or where there is potential for removal of the soil in front of the wall toe due to erosion or future excavation, or 1.0 ft. on level ground where there is no potential for erosion or future excavation of the soil in front of the wall toe.

For walls constructed along rivers and streams, embedment depths shall be established at a minimum of 2.0 ft. below potential scour depth as determined in accordance with Article 11.6.3.5.

As an alternative to locating the wall base below the depth of frost penetration where frost susceptible soils are present, the soil within the depth and lateral extent of frost penetration below the wall can be removed and replaced with nonfrost susceptible clean granular soil.

A minimum horizontal bench width of 4.0 ft. shall be provided in front of walls founded on slopes. The bench may be formed or the slope continued above that level as shown in Figure 11.10.2-1.

The lowest backfill reinforcement layer shall not be located above the long-term ground surface in front of the wall.

**Table C11.10.2.2-1 Guide for Minimum Front Face Embedment Depth.**

| Slope in Front of Structures | | Minimum Embedment Depth |
|---|---|---|
| Horizontal | for walls | *H*/20.0 |
| Horizontal | for abutments | *H*/10.0 |
| 3.0*H*:1.0*V* | walls | *H*/10.0 |
| 2.0*H*:1.0*V* | walls | *H*/7.0 |
| 1.5*H*:1.0*V* | walls | *H*/5.0 |

For structures constructed on slopes, minimum horizontal benches are intended to provide resistance to local bearing resistance failure consistent with resistance to general bearing resistance failure and to provide access for maintenance inspections.

### 11.10.2.3 Facing

Facing elements shall be designed to resist the horizontal force in the soil reinforcements at the reinforcement to facing connection, as specified in Articles 11.10.6.2.2 and 11.10.7.3.

In addition to these horizontal forces, the facing elements shall also be designed to resist potential compaction stresses occurring near the wall face during erection of the wall.

The tension in the reinforcement may be assumed to be resisted by a uniformly distributed earth pressure on the back of the facing.

The facing shall be stabilized such that it does not deflect laterally or bulge beyond the established tolerances.

### C11.10.2.3

See Article C3.11.2 for guidance. Additional information on compaction stresses can be found in Duncan and Seed (*1986*) and Duncan et al. (*1991*). Alternatively, compaction stresses can be addressed through the use of facing systems which have a proven history of being able to resist the compaction activities anticipated behind the wall and which have performed well in the long-term.

*11.10.2.3.1 Stiff or Rigid Concrete, Steel, and Timber Facings*

Facing elements shall be structurally designed in accordance with Sections 5, 6, and 8 for concrete, steel, and timber facings, respectively.

The minimum thickness for concrete panels at, and in the zone of stress influence of, embedded connections shall be 5.5 in. and 3.5 in. elsewhere. The minimum concrete cover shall be 1.5 in. Reinforcement shall be provided to resist the average loading conditions for each panel. Temperature and shrinkage steel shall be provided as specified in Article 5.10.8.

The structural integrity of concrete face panels shall be evaluated with respect to the shear and bending moment between reinforcements attached to the facing panel in accordance with Section 5.

For segmental concrete facing blocks, facing stability calculations shall include an evaluation of the maximum vertical spacing between reinforcement layers, the maximum allowable facing height above the uppermost reinforcement layer, inter-unit shear capacity, and resistance of the facing to bulging. The maximum spacing between reinforcement layers shall be limited to twice the width, $W_u$ illustrated in Figure 11.10.6.4.4b-1, of the segmental concrete facing block unit or 2.7 ft., whichever is less. The maximum facing height up to the wall surface grade above the uppermost reinforcement layer shall be limited to $1.5W_u$ illustrated in Figure 11.10.6.4.4b-1 or 24.0 in., whichever is less, provided that the facing above the uppermost reinforcement layer is demonstrated to be stable against a toppling failure through detailed calculations. The maximum depth of facing below the lowest reinforcement layer shall be limited to the width, $W_u$, of the proposed segmental concrete facing block unit.

*C11.10.2.3.1*

The specified minimum panel thicknesses and concrete cover recognize that MSE walls are often employed where panels may be exposed to salt spray and/or other corrosive environments. The minimum thicknesses also reflect the tolerances on panel thickness, and placement of reinforcement and connectors that can reasonably be conformed to in precast construction.

Based on research by Allen and Bathurst (*2001*), facings consisting of segmental concrete facing blocks behave as a very stiff facing, due to the ability of the facing blocks to transmit moment in a vertical direction throughout the facing column, and appear to have even greater stiffness than incremental precast concrete panels.

Experience has shown that for walls with segmental concrete block facings, the gap between soil reinforcement sections or strips at a horizontal level should be limited to a maximum of one block width to limit bulging of the facing between reinforcement levels or build up of unacceptable stresses that could result in performance problems. The ability of the facing to carry moment horizontally to bridge across the gaps in the reinforcement horizontally should be evaluated if horizontally discontinuous reinforcement is used, i.e., a reinforcement coverage ratio $R_c < 1$.

*11.10.2.3.2 Flexible Wall Facings*

If welded wire, expanded metal, or similar facing is used, they shall be designed in a manner which prevents the occurrence of excessive bulging as backfill behind the facing compresses due to compaction stresses or self weight of the backfill. This may be accomplished by limiting the size of individual facing elements vertically and the vertical and horizontal spacing of the soil reinforcement layers, and by requiring the facing to have an adequate amount of vertical slip and overlap between adjacent elements.

The top of the flexible facing at the top of the wall shall be attached to a soil reinforcement layer to provide stability to the top facing.

*C11.10.2.3.2*

Experience has shown that for welded wire, expanded metal, or similar facings, vertical reinforcement spacing should be limited to a maximum of 2.0 ft. and the gap between soil reinforcement at a horizontal level limited to a maximum of 3.0 ft. to limit bulging of the panels between reinforcement levels. The section modulus of the facing material should be evaluated and calculations provided to support reinforcement spacings, which will meet the bulging requirements stated in Article C11.10.4.2.

Geosynthetic facing elements shall not, in general, be left exposed to sunlight (specifically ultraviolet radiation) for permanent walls. If geosynthetic facing elements must be left exposed permanently to sunlight, the geosynthetic shall be stabilized to be resistant to ultraviolet radiation. Product specific test data shall be provided which can be extrapolated to the intended design life and which proves that the product will be capable of performing as intended in an exposed environment.

*11.10.2.3.3 Corrosion Issues for MSE Facing*

Steel-to-steel contact between the soil reinforcement connections and the concrete facing steel reinforcement shall be prevented so that contact between dissimilar metals, e.g., bare facing reinforcement steel and galvanized soil reinforcement steel, does not occur.

A corrosion protection system shall be provided where salt spray is anticipated.

*C11.10.2.3.3*

Steel-to-steel contact in this case can be prevented through the placement of a nonconductive material between the soil reinforcement face connection and the facing concrete reinforcing steel. Examples of measures which can be used to mitigate corrosion include, but are not limited to, coatings, sealants, or increased panel thickness.

## 11.10.3 Loading

The provisions of Article 11.6.1.2 shall apply, except that shrinkage and temperature effects need not be considered to come in contact with steel wall elements.

## 11.10.4 Movement and Stability at the Service Limit State

### 11.10.4.1 Settlement

The provisions of Article 11.6.2 shall apply as applicable.

The allowable settlement of MSE walls shall be established based on the longitudinal deformability of the facing and the ultimate purpose of the structure.

Where foundation conditions indicate large differential settlements over short horizontal distances, vertical full-height slip joints shall be provided.

Differential settlement from the front to the back of the wall shall also be evaluated, especially regarding the effect on facing deformation, alignment, and connection stresses.

### C11.10.4.1

For systems with rigid concrete facing panels and with a maximum joint width of 0.75 in., the maximum tolerable slope resulting from calculated differential settlement may be taken as given in Table C1.

**Table C11.10.4.1-1 Guide for Limiting Distortion for Precast Concrete Facings of MSE Walls.**

| Joint Width (in.) | Limiting Differential Settlement | |
|---|---|---|
| | Area $\leq$ 30 ft.$^2$ | 30 ft.$^2$ $\leq$ Area $\leq$ 75 ft.$^2$ |
| 0.75 | 1/100 | 1/200 |
| 0.50 | 1/200 | 1/300 |
| 0.25 | 1/300 | 1/600 |

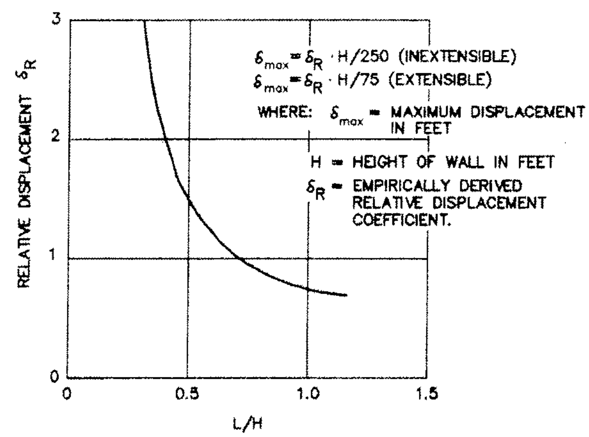
For MSE walls with full height precast concrete facing panels, total settlement should be limited to 2.0 in., and the limiting differential settlement should be 1/500. For walls with segmental concrete block facings, the limiting differential settlement should be 1/200. For walls with welded wire facings or walls in which cast-in-place concrete or shotcrete facing is placed after wall settlement is essentially complete, the limiting differential settlement should be 1/50. These limiting differential settlement criteria consider only structural needs of the facing. More stringent differential settlement criteria may be needed to meet aesthetic requirements.

#### 11.10.4.2 Lateral Displacement

Lateral wall displacements shall be estimated as a function of overall structure stiffness, compaction intensity, soil type, reinforcement length, slack in reinforcement-to-facing connections, and deformability of the facing system or based on monitored wall performance.

#### C11.10.4.2

A first order estimate of lateral wall displacements occurring during wall construction for simple MSE walls on firm foundations can be obtained from Figure C1. If significant vertical settlement is anticipated or heavy surcharges are present, lateral displacements could be considerably greater. Figure C1 is appropriate as a guide to establish an appropriate wall face batter to obtain a near vertical wall or to determine minimum clearances between the wall face and adjacent objects or structures.




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

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

**Figure C11.10.4.2-1 Empirical Curve for Estimating Anticipated Lateral Displacement During Construction for MSE Walls.**

For welded wire or similarly faced walls such as gabion faced walls, the maximum tolerable facing bulge between connections, both horizontally and vertically, with soil reinforcement is approximately 2.0 in. For geosynthetic facings, the maximum facing bulge between reinforcement layers should be approximately 2.75 in. for 1.0 ft. vertical reinforcement spacing to 5.0 in. for 2.0 ft. vertical reinforcement spacing.

#### 11.10.4.3 Overall Stability

The provisions of Article 11.6.2.3 shall apply. Additionally for MSE walls with complex geometrics, compound failure surfaces which pass through a portion of the reinforced soil mass as illustrated in Figure 1 shall be investigated, especially where the wall is located on sloping or soft ground where overall stability may be inadequate. The long-term strength of each backfill reinforcement layer intersected by the failure surface should be considered as restoring forces in the limit equilibrium slope stability analysis.

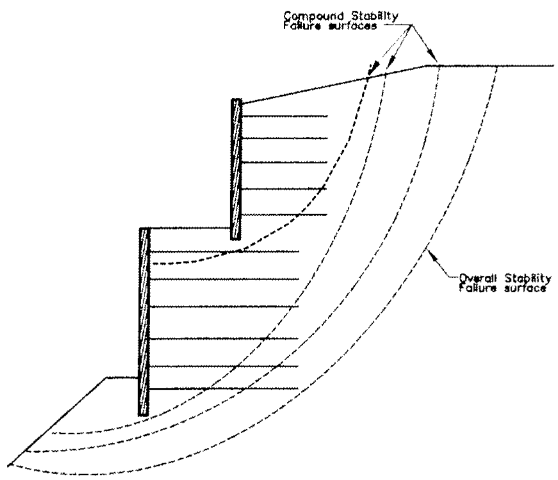



**Figure 11.10.4.3-1 Overall and Compound Stability of Complex MSE Wall Systems.**

### 11.10.5 Safety Against Soil Failure (External Stability)

#### 11.10.5.1 General

MSE structures shall be proportioned to satisfy eccentricity and sliding criteria normally associated with gravity structures.

Safety against soil failure shall be evaluated by assuming the reinforced soil mass to be a rigid body. The coefficient of active earth pressure, $k_a$, used to compute the earth pressure of the retained soil behind the reinforced soil mass shall be determined using the friction angle of the retained soil. In the absence of specific data, a maximum friction angle of 30° may be used for granular soils. Tests should be performed to determine the friction angle of cohesive soils considering both drained and undrained conditions.

#### C11.10.5.1

Eccentricity requirements seldom govern design. Sliding and overall stability usually govern design of structures greater than 30.0 ft. in height, structures constructed on weak foundation soils, or structures loaded with sloping surcharges.

### 11.10.5.2 Loading

Lateral earth pressure distributions for design of MSE walls shall be taken as specified in Article 3.11.5.8. Application of loads for external and internal stability shall be taken as specified in Articles 11.10.5 and 11.10.6, respectively. Application of surcharge loads shall be taken as specified in Article 11.10.11. Application of load factors for these loads shall be taken as specified in Article 11.5.5.

For external stability calculations only, the active earth pressure coefficients for retained backfill, i.e., fill behind the reinforced soil mass, shall be taken as specified in Article 3.11.5.3 with $\delta = \beta$.

Dead load surcharges, if present, shall be taken into account in accordance with Article 11.10.10.

For investigation of sliding stability and eccentricity, the continuous traffic surcharge loads shall be considered to act beyond the end of the reinforced zone as shown in Figure 1. Application of load factors for these loads shall be taken as specified in Article 11.5.5.

### C11.10.5.2

Figures 3.11.5.8.1-1, 3.11.5.8.1-2, and 3.11.5.8.1-3 illustrate lateral earth pressure distributions for external stability of MSE walls with horizontal backslope, inclined backslope, and broken backslope, respectively.

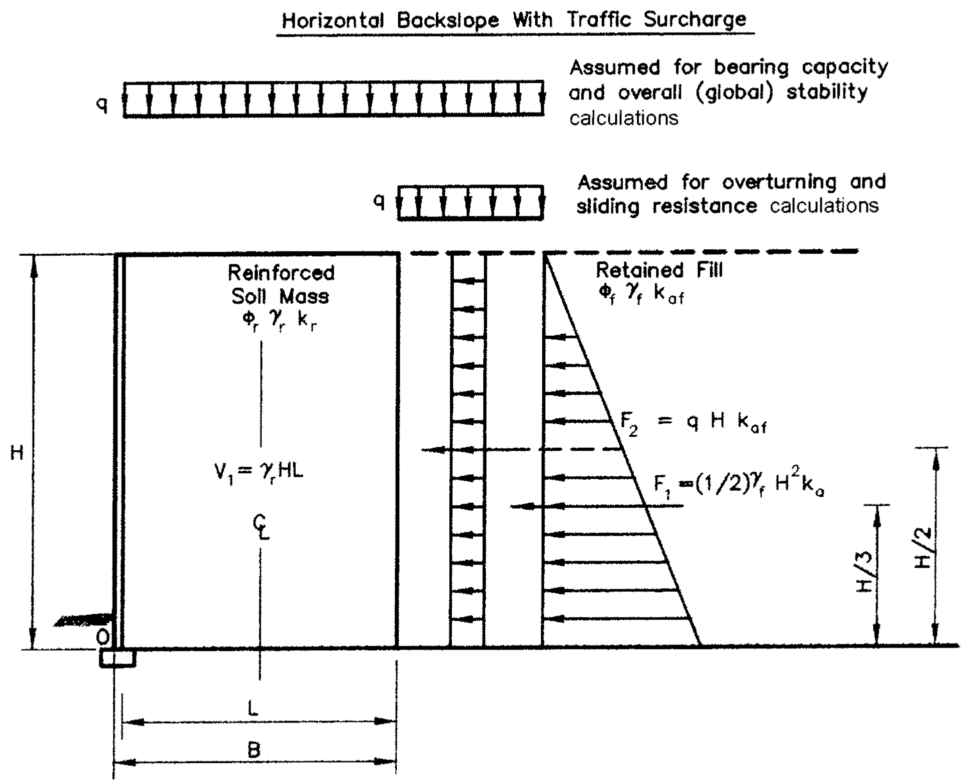



**Figure 11.10.5.2-1 External Stability for Wall with Horizontal Backslope and Traffic Surcharge.**

### 11.10.5.3 Sliding

The provisions of Article 10.6.3.3 shall apply.

The coefficient of sliding friction at the base of the reinforced soil mass shall be determined using the friction angle of the foundation soil. For discontinuous reinforcements, e.g., strips, the angle of sliding friction shall be taken as the lesser of $\phi_r$ of the reinforced fill and $\phi_f$ of the foundation soil. For continuous reinforcements, e.g., grids and sheets, the angle of sliding friction shall be taken as the lesser of $\phi_r$, $\phi_f$ and $\rho$, where $\rho$ is the soil-reinforcement interface friction angle. In the absence of specific data, a maximum friction angle, $\phi_f$, of 30° and a maximum soil-reinforcement interface angle, $\rho$, of 2/3 $\phi_f$ may be used.

### C11.10.5.3

For relatively thick facing elements, it may be desirable to include the facing dimensions and weight in sliding and overturning calculations, i.e., use $B$ in lieu of $L$ as shown in Figure 11.10.5.2-1.

### 11.10.5.4 Bearing Resistance

For the purpose of computing bearing resistance, an equivalent footing shall be assumed whose length is the length of the wall, and whose width is the length of the reinforcement strip at the foundation level. Bearing pressures shall be computed using a uniform base pressure distribution over an effective width of footing determined in accordance with the provisions of Articles 10.6.3.1 and 10.6.3.2.

Where soft soils or sloping ground in front of the wall are present, the difference in bearing stress calculated for the wall reinforced soil zone relative to the local bearing stress beneath the facing elements shall be considered when evaluating bearing capacity. In both cases, the leveling pad shall be embedded adequately to meet bearing capacity requirements.

### C11.10.5.4

The effect of eccentricity and load inclination is accommodated by the introduction of an effective width, $B' = L-2e$, instead of the actual width.

For relatively thick facing elements, it may be reasonable to include the facing dimensions and weight in bearing calculations, i.e., use $B$ in lieu of $L$ as shown in Figure 11.10.2-1.

Note, when the value of eccentricity $e$ is negative: $B' = L$.

Due to the flexibility of MSE walls, a triangular pressure distribution at the wall base cannot develop, even if the wall base is founded on rock, as the reinforced soil mass has limited ability to transmit moment. Therefore, an equivalent uniform base pressure distribution is appropriate for MSE walls founded on either soil or rock.

Concentrated bearing stresses from the facing weight on soft soil could create concentrated stresses at the connection between the facing elements and the wall backfill reinforcement.

### 11.10.5.5 Overturning

The provisions of Article 11.6.3.3 shall apply.

## 11.10.6 Safety Against Structural Failure (Internal Stability)

### 11.10.6.1 General

Safety against structural failure shall be evaluated with respect to pullout and rupture of reinforcement.

A preliminary estimate of the structural size of the stabilized soil mass may be determined on the basis of reinforcement pullout beyond the failure zone, for which resistance is specified in Article 11.10.6.3.

### C11.10.6.1

The resistance factors, specified in Article 11.5.6, are consistent with the use of select backfill in the reinforced zone, homogeneously placed and carefully controlled in the field for conformance with Section 7 of *AASHTO LRFD Bridge Construction Specifications*. The basis for the factors is the successful construction of thousands of structures in accordance with these criteria, and the use of conservative pullout resistance factors representing high confidence limits.

### 11.10.6.2 Loading

The load in the reinforcement shall be determined at two critical locations: the zone of maximum stress and the connection with the wall face. Potential for reinforcement rupture and pullout are evaluated at the zone of maximum stress, which is assumed to be located at the boundary between the active zone and the resistant zone in Figure 11.10.2-1. Potential for reinforcement rupture and pullout are also evaluated at the connection of the reinforcement to the wall facing.

The maximum friction angle used for the computation of horizontal force within the reinforced soil mass shall be assumed to be 34°, unless the specific project select backfill is tested for frictional strength by triaxial or direct shear testing methods, AASHTO T 234-74 and T 236-72, respectively. A design friction angle of greater than 40° shall not be used with the Simplified Method even if the measured friction angle is greater than 40°.

### C11.10.6.2

Loads carried by the soil reinforcement in mechanically stabilized earth walls are the result of vertical and lateral earth pressures, which exist within the reinforced soil mass, reinforcement extensibility, facing stiffness, wall toe restraint, and the stiffness and strength of the soil backfill within the reinforced soil mass. The soil reinforcement extensibility and material type are major factors in determining reinforcement load. In general, inextensible reinforcements consist of metallic strips, bar mats, or welded wire mats, whereas extensible reinforcements consist of geotextiles or geogrids. Inextensible reinforcements reach their peak strength at strains lower than the strain required for the soil to reach its peak strength. Extensible reinforcements reach their peak strength at strains greater than the strain required for soil to reach its peak strength. Internal stability failure modes include soil reinforcement rupture (strength limit state), and excessive reinforcement elongation under the design load (service limit state). The service limit state is not evaluated in current practice for internal stability design. Internal stability is determined by equating the factored tensile load applied to the reinforcement to the factored tensile resistance of the reinforcement, the tensile resistance being governed by reinforcement rupture and pullout.

Analysis of full scale wall data in comparison to the Simplified Method or other widely accepted design methods (see Article 11.10.6.2.1) indicates that these methods will significantly underestimate reinforcement loads if design soil friction angles greater than 40° are used. This recommendation applies to soil friction angles as determined using triaxial or direct shear tests, as the Simplified Method was calibrated using triaxial or direct shear soil strengths (see *Allen et al., 2001*).

*11.10.6.2.1 Maximum Reinforcement Loads*

Maximum reinforcement loads shall be calculated using the Simplified Method **or the Coherent Gravity Method. The Simplified Method shall be considered to apply to both steel and geosynthetic reinforced wall systems. The Coherent Gravity Method shall be applied primarily to steel soil reinforcement systems.** For **the Simplified Method**, the load in the reinforcements shall be obtained by multiplying the vertical earth pressure at the reinforcement by a lateral earth pressure coefficient, and applying the resulting lateral pressure to the tributary area for the reinforcement. **For the Coherent Gravity Method, the load in the reinforcements shall be obtained in the same way as the Simplified Method, except as follows:**

- **The vertical earth pressure at each reinforcement level shall be computed using an equivalent uniform base pressure distribution over an effective width of reinforced wall mass determined in accordance with the provisions of Articles 11.6.3.1 and 11.6.3.2, and**
- **For steel reinforced wall systems, the lateral earth pressure coefficient used shall be equal to $k_0$ at the point of intersection of the theoretical failure surface with the ground surface at or above the wall top, transitioning to $k_a$ at a depth of 20.0 ft below that intersection point, and constant at $k_a$ at depths greater than 20.0 ft. If used for geosynthetic reinforced systems, $k_a$ shall be used throughout the wall height.**

**All other provisions in this article are applicable to both methods.**

Other widely accepted and published design methods for calculation of reinforcement loads may be used at the discretion of the wall owner or approving agency, provided the designer develops method-specific resistance factors for the method employed.

**For the Simplified Method,** factored horizontal stress, $\sigma_H$, at each reinforcement level shall be determined as:

$$\sigma_H = \gamma_P\left(\sigma_v k_r + \Delta\sigma_H\right) \qquad (11.10.6.2.1\text{-}1)$$

where:

$\gamma_P$ = the load factor for vertical earth pressure *EV* from Table 3.4.1-2

$k_r$ = horizontal pressure coefficient (dim.)

$\sigma_v$ = pressure due to resultant of gravity forces from soil self weight within and immediately above the reinforced wall backfill, and any surcharge loads present (ksf)

*C11.10.6.2.1*

**The development of the Simplified Method for estimating reinforcement loads is provided in Allen, et al. (*2001*). The Coherent Gravity Method has been used in MSE wall design practice for many years for steel reinforced wall systems. Detailed procedures for the Coherent Gravity Method are provided in Allen, et al. (*2001*) and in Mitchell and Villet (*1987*). Its application to geosynthetic soil reinforcement systems results in conservative designs.**

The design specifications provided herein assume that the wall facing combined with the reinforced backfill acts as a coherent unit to form a gravity retaining structure. Research by Allen and Bathurst (*2003*) **and Allen et al. (2003)** indicates that reinforcement load is linear with reinforcement spacing to a reinforcement vertical spacing of 2.7 ft or more, though a vertical spacing of this magnitude should not be attempted unless the facing is considered to be adequately stiff to prevent excessive bulging between layers (see Article C11.10.2.3.2).

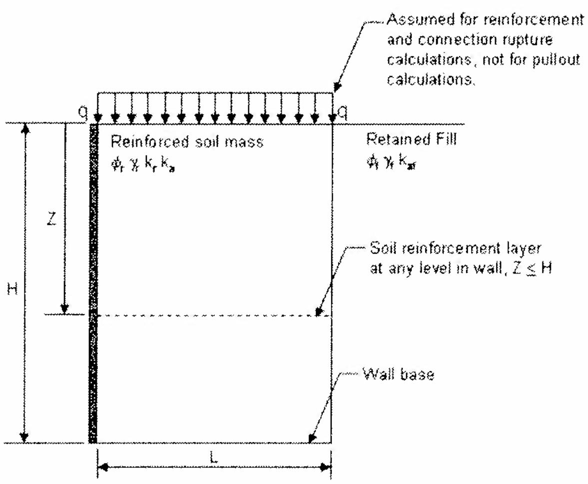
These MSE wall specifications also assume that inextensible reinforcements are not mixed with extensible reinforcements within the same wall. MSE walls which contain a mixture of inextensible and extensible reinforcements are not recommended.

The calculation method for $T_{max}$ is empirically derived, based on reinforcement strain measurements, converted to load based on the reinforcement modulus, from full scale walls at working stress conditions. The load factor *EV*, on the other hand, was determined in consideration of vertical earth pressure exerted by a soil mass without inclusions, and was calibrated to address uncertainties implied by allowable stress design for external stability for walls. *EV* is not directly applicable to internal reinforcement loads in MSE walls, since the calibration of *EV* was not performed with internal stability of a reinforced system in mind.

The use of *EV* for the load factor in this case **for both methods (i.e., the Simplified and Coherent Gravity Methods)** should be considered an interim measure until research is completed to quantify load prediction bias and uncertainty.

$\Delta\sigma_H$ = horizontal stress at reinforcement level resulting from any applicable concentrated horizontal surcharge load as specified in Article 11.10.10.1 (ksf)

**For the Simplified Method,** vertical stress for maximum reinforcement load calculations shall be determined as shown in Figures 1 and 2. **For the Coherent Gravity Method, vertical stress shall be calculated at each reinforcement level using an equivalent uniform base pressure that accounts for load eccentricity caused by the lateral earth pressure acting at the back of the reinforced soil mass above the reinforcement level being considered. This base pressure shall be applied over an effective width of reinforced wall mass determined in accordance with the provisions of Articles 11.6.3.1 and 11.6.3.2. As is true for the Simplified Method, live load is not included in the vertical stress calculation to determine $T_{max}$ for assessing pullout loads when using the Coherent Gravity Method.**




Max Stress: $\sigma_v = \gamma_r Z + q + \Delta\sigma_v$

Pullout: $\sigma_v = \gamma_r Z + \Delta\sigma_v$

Note: $\Delta\sigma_v$ is determined from Figure 11.10.10.1-1
$H$ is the total wall height at the face.

**Figure 11.10.6.2.1-1 Calculation of Vertical Stress for Horizontal Backslope Condition, Including Live Load and Dead Load Surcharges for Internal Stability Analysis.**

Sloping soil surcharges are taken into account through an equivalent uniform surcharge and assuming a level backslope condition. For these calculations, the depth $Z$ is referenced from the top of the wall at the wall face, excluding any copings and appurtenances.

**Note that $T_{max}$, the factored tensile load in the soil reinforcement, must be calculated twice for internal stability design as follows: (1) for checking reinforcement and connection rupture, determine $T_{max}$ with live load surcharge included in the calculation of $\sigma_v$; (2) for checking pullout, determine $T_{max}$ with live load surcharge excluded from the calculation of $\sigma_v$.**