SECTION 11

WALLS, ABUTMENTS, AND PIERS

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_c	=	cross-sectional area of reinforcement unit (in.^{2}) (11.10.6.4.1)
A_S	=	peak seismic ground acceleration coefficient modified by short-period site factor (11.6.5) (C11.8.6) (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 defined as ratio of the particle size of soil that is 60 percent finer in size (D_{60}) to the particle size of soil that is ten percent finer in size (D_{10}) (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 = 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 = eccentricity of load from centerline of foundation (ft) (11.10.8)
F_p = static lateral force due to a concentrated surcharge load (kips/ft) (11.6.5.1)
F_T = resultant force of active lateral earth pressure (kips/ft) (11.6.3.2)
F_v = site class adjustment factor for the 1-sec. spectral acceleration (dim.) (A11.5)
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) (11.6.5.1)
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) (11.10.7.1)
h_a distance between the base of the wall, or the mudline in front of the wall, and the resultant active seismic earth pressure force (ft) (A11.3.1)
h_i = height of reinforced soil zone contributing horizontal load to reinforcement at level i (ft) (11.10.6.2.1)
h_p = vertical distance between the wall base and the static surcharge lateral force F_p (ft) (11.6.5.1)
i = backfill slope angle (degrees) (A11.3.1)
i_b = slope of facing base downward into backfill (degrees) (11.10.6.4.4b)
K = seismic passive pressure coefficient (dim.) (A11.3.1)
K_{AE} = seismic active pressure coefficient (dim.) (A11.3.1)
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_{h0} = horizontal seismic acceleration coefficient at zero displacement (dim.) (11.6.5.2)
k_v = vertical seismic acceleration coefficient (dim.) (11.6.5.3)
k_r = horizontal earth pressure coefficient of reinforced fill (dim.) (11.10.5.2)
k_y = yield acceleration in sliding block analysis that results in sliding of the wall (dim) (A11.5)
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)
M = moment magnitude of design earthquake (dim.) (A11.5)
$MARV$ = 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 or standard penetration resistance from SPT (kips/ft or blows/ft, respectively) (11.6.3.2) (A11.5)
n = total number of reinforcement layers in the wall (dim) (11.10.7.2)
P_{AE} = dynamic active horizontal thrust, including static earth pressure (kips/ft) (11.10.7.1)
P_a = resultant active earth pressure force per unit width of wall (kips/ft) (11.8.6.2)
P_b = pressure inside bin module (ksf) (11.10.5.1)
PGA = peak ground acceleration (dim.) (11.6.5.1)
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_{PE} = dynamic passive horizontal thrust, including static earth pressure (kips/ft) (11.8.6.2)
P_r = ultimate soil reinforcement pullout resistance per unit of reinforcement width (kips/ft) (11.10.6.3.2)
P_{seis} = total lateral force applied to a wall during seismic loading (kips/ft) (11.6.5.1)
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)
PVG = peak ground velocity (in./sec.) (A11.5)
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)
S_1 = 1-sec. spectral acceleration coefficient (dim.) (A11.5)
T_{ac} = nominal long-term reinforcement/facing connection design strength (kips/ft) (11.10.6.4.1)
$T_{a\ell}$ = 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_s = fundamental period of wall (sec.) (A11.5)
T_{total} = total load on reinforcement layer (static & dynamic) per unit width of wall (kips/ft) (11.10.7.2)
V_s = shear wave velocity of soil behind wall (ft/sec.) (A11.5)
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_s = weight of the soil that is immediately above the wall, including the wall heel (kips/ft) (11.6.5.1)
W_u = unit width of segmental facing (ft) (11.10.2.3.2)
W_w = weight of the wall (kips/ft) (11.6.5.1)
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)
α = scale effect correction factor, or wall height acceleration reduction factor for wave scattering (dim.) (11.10.6.3.2) (A11.5)
β = inclination of ground slope behind face of wall (degrees) (11.5.5)
γ_{EQ} = load factor for live load applied simultaneously with seismic loads in Article 3.4.1 (dim.) (11.6.5)
γ_P = load factor for vertical earth pressure in Article 3.4.1 (dim.) (11.10.6.2.1)
γ_s = soil unit weight (kcf)
γ'_s = effective soil unit weight (kcf) (C11.8.4.1)
γ_r = unit weight of reinforced fill (kcf) (11.10.5.2)
γ_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)
δ = wall-backfill interface friction angle (degrees) (11.5.5)
δ_{max} = maximum displacement (ft) (11.10.4.2)
δ_R = relative displacement coefficient (11.10.4.2)
θ = wall batter from horizontal (degrees) (11.10.6.2.1)
θ_{MO} = arc tan $[k_h/(1-k_v)]$ for M-O analysis (degrees) (11.6.5.3)
ρ = soil-reinforcement angle of friction (degrees) (11.10.5.3)
ϕ = resistance factor (11.5.4)
ϕ_f = internal friction angle of foundation or backfill soil (degrees) (11.10.2)
ϕ_r = internal friction angle of reinforced fill (degrees) (11.10.5.2)
ϕ'_f = effective internal friction angle of soil (degrees) (11.8.4.1)
σ_H = factored horizontal stress at reinforcement level (ksf) (11.10.6.2.1)
σ_{Hmax} = maximum stress in soil reinforcement in abutment zones (11.10.8)
σ_v = vertical stress in soil (ksf) (11.10.6.2.1)
σ_{VI} = vertical soil stress over effective base width (ksf) (11.10.8)
τ_n = nominal anchor bond stress (ksf) (11.9.4.2)
ω = wall batter due to setback of segmental facing units (degrees) (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 Berg et al. (2009).

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.

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.

C11.5.1

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

- Bearing resistance failure,
- Lateral sliding,
- Loss of base contact due to eccentric loading,
- Pullout failure of anchors or soil reinforcements, and
- Structural failure.

11.5.4—Extreme Event Limit State

11.5.4.1—General Requirements

Abutments, walls, and piers shall be investigated at the extreme event limit state for:

- Overall stability failure,
- Bearing resistance failure,
- Lateral sliding,
- Loss of base contact due to eccentric loading,

- Pullout failure of anchors or soil reinforcements, and
- Structural failure.

The site-adjusted peak ground acceleration, A_s (i.e., $F_{pga} \times PGA$, as specified in Article 3.10.3.2), used for seismic design of retaining walls shall be determined in accordance with Article 3.10.

The levels of peak ground acceleration at the ground surface in some areas will be low enough that a check on seismic loading is not required as other limit states will control the design.

11.5.4.2—Extreme Event I, No Analysis

A seismic design shall not be considered mandatory for walls located in Seismic Zones 1 through 3, or for walls at sites where the site adjusted peak ground acceleration, A_s, is less than or equal to 0.4g, unless one or more of the following is true:

- Liquefaction induced lateral spreading or slope failure, or seismically induced slope failure, due to the presence of sensitive clays that lose strength during the seismic shaking, may impact the stability of the wall for the design earthquake.
- The wall supports another structure that is required, based on the applicable design code or specification for the supported structure, to be designed for seismic loading and poor seismic performance of the wall could impact the seismic performance of that structure.

The no-seismic-analysis option should be limited to internal and external seismic stability design of the wall. If the wall is part of a bigger slope, overall seismic stability of the wall and slope combination should still be evaluated.

These no-seismic-analysis provisions shall not be considered applicable to walls functioning as support piers for bridges.

C11.5.4.2

Article 11.5.4.2, related to specific seismic zones, may also be considered applicable to the corresponding Seismic design categories (SDC) A, B, and C, if using AASHTO*'s Guide Specifications for LRFD Seismic Bridge Design*.

A summary of previous performance of walls in earthquakes, as well as key research findings that provide support to the provisions in Article 11.5.4.2, are provided in Appendix A11. In general, wall performance in past earthquakes has been very good, even in the largest, most damaging earthquakes, and cases where either wall collapse or severe wall displacements have occurred are rare. For those cases where collapse or severe displacement of walls did occur, those cases were mostly limited to situations where significant liquefaction occurred, where soil conditions behind or below the wall were very poor (e.g., soft silts and clays, marginally stable soils, water build up behind the wall) and ground accelerations were high, or where the wall was subjected to direct shear displacement of the fault. Furthermore, most of those failures were limited to walls that were very old. These wall failure situations are all well outside the limits specified in Article 11.5.4.2 where these specifications allow the designer to not conduct a detailed wall seismic design. However, walls meeting the requirements in Article 11.5.4.2 that allow a seismic analysis to not be conducted have demonstrated consistently good performance in past earthquakes.

Based on previous experience, walls that form tunnel portals have tended to exhibit more damage due to earthquakes than free standing walls. It is likely that the presence of the tunnel restricts the ability of the portal wall to move, increasing the seismic forces to which the wall is subjected. Therefore, a more detailed seismic analysis of tunnel portal walls should be considered even if the walls meet all the other no seismic analysis conditions specified in Article 11.5.4.2.

For walls that cross an active fault which could result in significant differential movement within the wall, a detailed seismic analysis should be considered even if the wall is located in Seismic Zones 1, 2, or 3.

Examples of other structures include bridges (e.g., the abutment foundation), buildings, pipelines or major utilities, pipe arches, or dams. If the wall supports another wall, a seismic design is not required for the lower wall, provided that the upper and lower wall can be designed as a single tiered structure and the limitations on the tiered structure for these provisions in Article 11.5.4.2, if in Seismic Zone 3 or lower, are met.

If the wall has abrupt changes in its alignment geometry (e.g., corners and short radius turns at an enclosed angle of 120 degrees or less), a seismic analysis of the wall should be considered for Seismic Zone 2 or higher. Based on past experience in earthquakes, wall corners tend to attract greater loads than free standing walls with generally straight alignments and have therefore suffered greater damage. The seismic details discussed in Articles 11.6.5.6 and 11.10.7.4 and their commentary will help to reduce the potential problems at corners that have occurred in past earthquakes. Note that the corner or abrupt alignment change enclosed angle as defined in Article 11.5.4.2 can either be internal or external to the wall.

A seismic analysis should be considered for Seismic Zone 2 or higher if either of the following is greater than 30 ft:

- The exposed wall height plus the average depth over the width of the wall of any soil surcharge present, or
- For tiered walls the sum of the exposed height of all the tiers plus the average soil surcharge depth, is greater than 30 ft.

A seismic analysis should be considered if in Seismic Zone 2 or higher, and if, for gravity and semigravity walls, the wall backfill does not meet the requirements of Article 7.3.6.3 of the *AASHTO LRFD Bridge Construction Specifications*, due to the possibility that the backfill will not be adequately drained to prevent water build-up in the backfill.

For Seismic Zone 2 or higher, if a seismic design is not conducted, it is still important to use good seismic details as specified in Articles 11.6.5.6 and Article 11.10.7.4.

If the wall is part of a bigger slope that potentially could fail during seismic loading, the overall seismic stability of the wall and slope as defined in Article 11.6.2.3 should be evaluated, as specified in Articles 11.5.4.1 and 11.5.8. If the wall is determined to have only a minor destabilizing effect on the overall stability of the slope during seismic loading, for example, a wall placed within a large slope or existing landslide that is marginally stable during static loading, it may not be practical to design the wall to be stable for overall stability for the Extreme Event I limit state. Addressing the landslide overall stability during seismic loading should be considered a separate effort not specifically addressed by these Specifications.

11.5.5—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 11.5.6.

The factored resistance, R_R, calculated for each applicable limit state shall be the nominal resistance, R_n, multiplied by an appropriate resistance factor, ϕ, specified in Table 11.5.7-1.

C11.5.5

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.

11.5.6—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.

C11.5.6

Figures C11.5.6-1 and C11.5.6-2 show the typical application of load factors to produce the total extreme factored force effect for external stability of retaining walls for the strength limit state. 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 C11.5.6-3. 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. Figure C11.5.6-3 is also applicable to seismic loading (i.e., Extreme Event I), except that the load factor for live load surcharge is γ_{EQ} instead of *LL*.

Figure C11.5.6-4 shows the typical application of load factors to produce the total extreme factored force effect for external stability of retaining walls for the Extreme Event I limit state.

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

The subscripts $_V$ and $_H$ in Figure C11.5.6-4 denote vertical and horizontal components, respectively, of each force.

For the Extreme Event I limit state, the peak seismic lateral pressures acting on the wall should not be based on the maximum ground water elevation due to the low probability that the design peak seismic acceleration would be combined with the maximum ground water level. Instead, it is more appropriate to use the time-averaged mean groundwater elevation or a reasonable engineering estimate of this elevation.

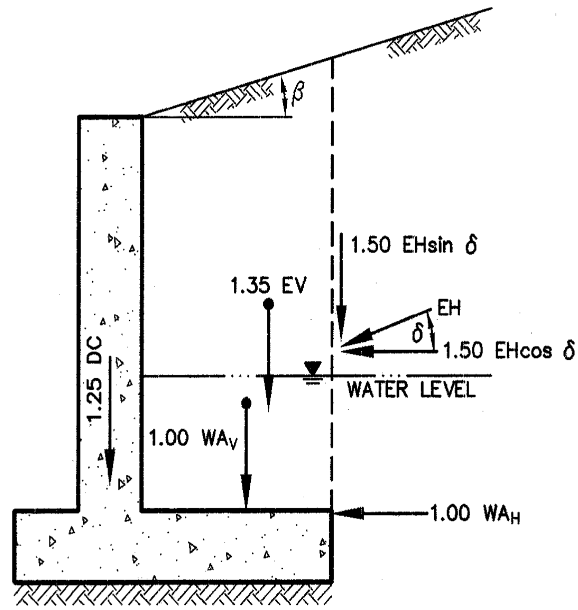


Figure C11.5.6-1—Typical Application of Load Factors for Bearing Resistance

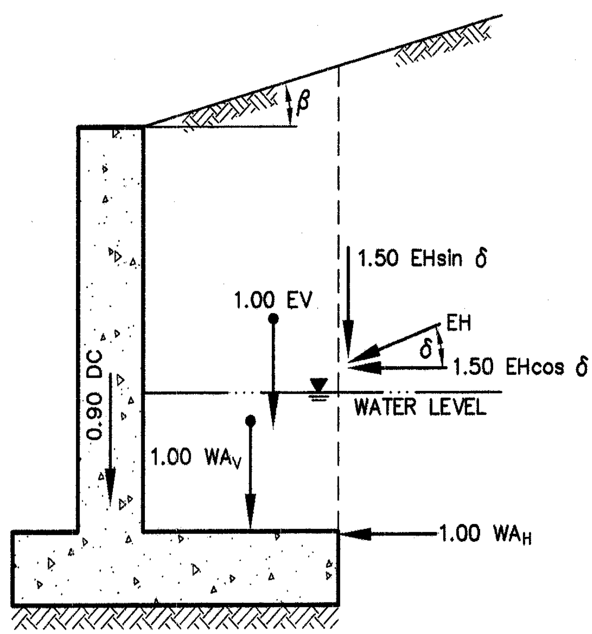


Figure C11.5.6-2—Typical Application of Load Factors for Sliding and Eccentricity

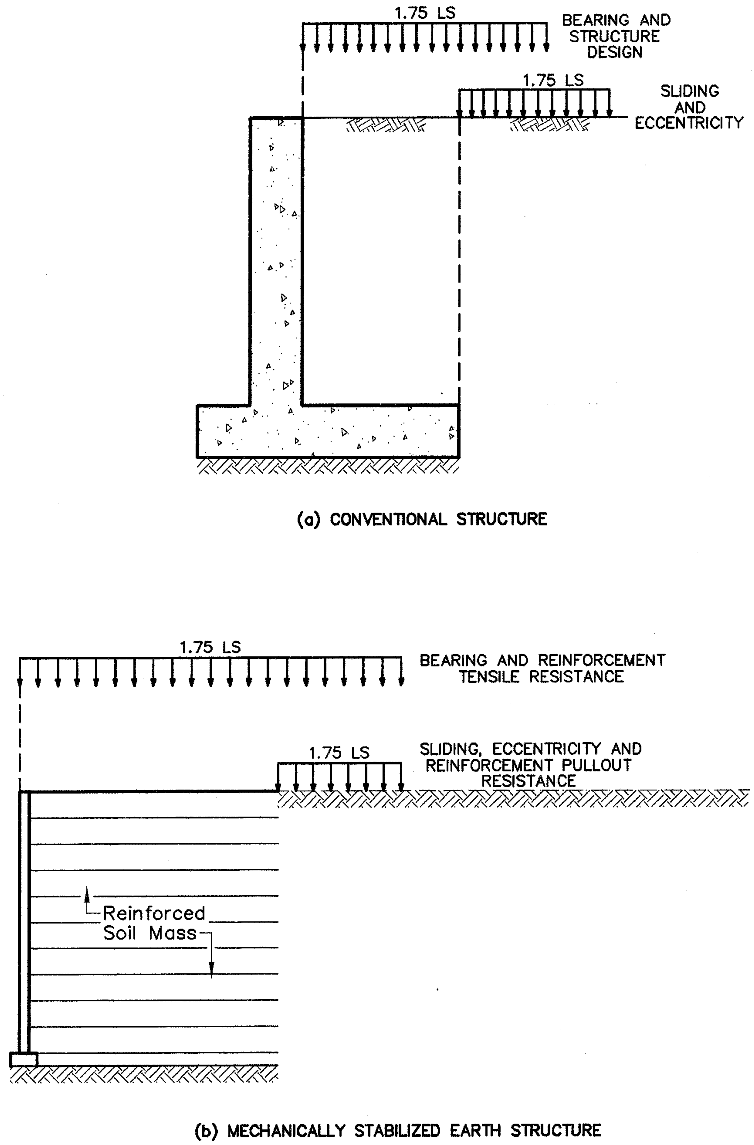


Figure C11.5.6-3—Typical Application of Live Load Surcharge

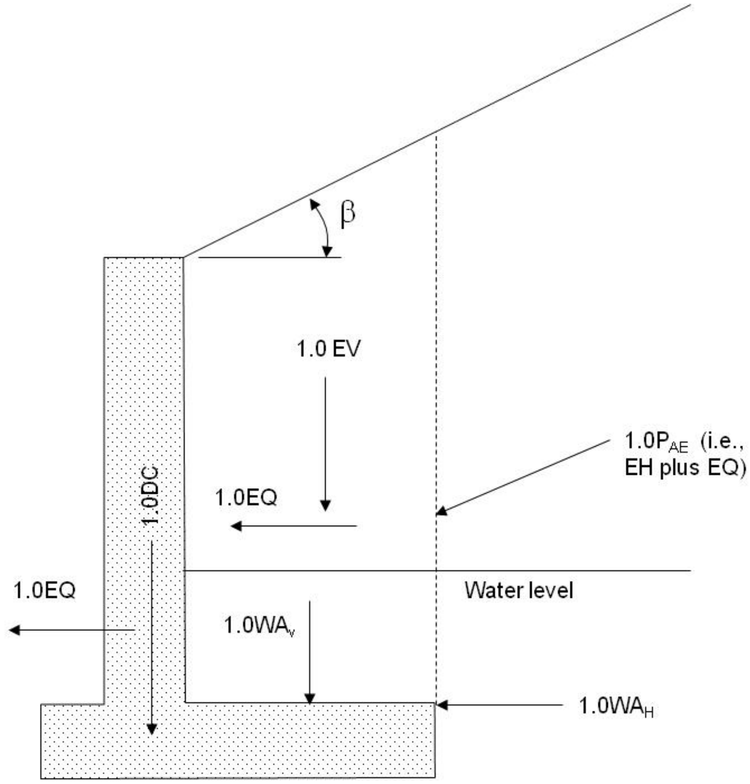


Figure C11.5.6-4—Typical Application of Load Factors for Bearing and Sliding Resistance and for Eccentricity in the Extreme Event I Limit State

For seismic loading effects on lateral earth pressure, the seismic load factor shall be applied to the entire lateral earth pressure load created by the earth mass retained by the wall or abutment. For any surcharge loads acting on the wall (e.g., *ES*) in combination with seismic load, *EQ*, the load factor for seismic loads, shall be applied.

Seismic loading of an earth mass retained by a wall is calculated using an extension of Coulomb theory or by limit equilibrium slope stability methods. The seismic loading causes the active soil wedge to increase, resulting in increased total load. The static loading cannot be separated from the seismic loading in this analysis, other than by artificial means through subtracting the static earth pressure from the total earth pressure calculated for seismic loading. Past allowable stress design practice has been to apply a single reduced safety factor to the entire lateral earth load combination. Therefore, one seismic load factor (typically a load factor of 1.0) is applied to the total earth pressure that occurs during seismic loading.

Regarding other loads acting in combination with the seismic loading and earth pressure, the load combination philosophy described for earth pressure also applies to be consistent with past allowable stress design practice for a no collapse design objective.

11.5.7—Resistance Factors—Service and Strength

Resistance factors for the service limit states shall be taken as 1.0, except as provided for overall stability in Article 11.6.2.3.

For the strength limit state, the resistance factors provided in Table 11.5.7-1 shall be used for wall design, unless region specific values or substantial successful experience is available to justify higher values.

C11.5.7

The resistance factors given in Table 11.5.7-1, 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 11.5.7-1 were based on direct correlation to allowable stress design, the differences between the resistance factors for tensile

Resistance factors for geotechnical design of foundations that may be needed for wall support, unless specifically identified in Table 11.5.7-1, are as specified in Tables 10.5.5.2.2-1, 10.5.5.2.3-1, and 10.5.5.2.4-1.

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 11.5.7-1.

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.

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.

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.

Region-specific resistance factor values should be determined based on substantial statistical data combined with calibration or substantial successful experience to justify higher values. Smaller resistance factors should be used if site or material variability is anticipated to be unusually high or if design assumptions are required that increase design uncertainty that has not been mitigated through conservative selection of design parameters. See Allen et al. (2005) for additional guidance on calibration of resistance factors.

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 11.5.7-1—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 • Cohesive soils • Rock	0.65 [1] 0.70 [1] 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 A615 bars) • High strength steel (e.g., ASTM A722 bars)	0.90 [3] 0.80 [3]
Flexural capacity of vertical elements		0.90
Mechanically Stabilized Earth Walls, Gravity Walls, and Semigravity Walls		
Bearing resistance	• Gravity and semigravity walls • MSE walls	0.55 0.65
Sliding		1.0
Tensile resistance of metallic reinforcement and connectors	Strip reinforcements [4] • Static loading Grid reinforcements [4] [5] • Static loading	 0.75 0.65
Tensile resistance of geosynthetic reinforcement and connectors	• Static loading	0.90
Pullout resistance of tensile reinforcement	• Static loading	0.90
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.

11.5.8—Resistance Factors—Extreme Event Limit State

Unless otherwise specified, all resistance factors shall be taken as 1.0 when investigating the extreme event limit state.

For overall stability of the retaining wall when earthquake loading is included, a resistance factor, ϕ, of 0.9 shall be used. For bearing resistance, a resistance factor of 0.8 shall be used for gravity and semigravity walls and 0.9 for MSE walls.

For tensile resistance of metallic reinforcement and connectors, when earthquake loading is included, the following resistance factors shall be used:

- Strip reinforcements, $\phi = 1.0$
- Grid reinforcement, $\phi = 0.85$

Table 11.5.7-1 Notes 4 and 5 also apply to these resistance factors for metallic reinforcements.

For tensile resistance of geosynthetic reinforcement and connectors, a resistance factor, ϕ, of 1.20 shall be used.

For pullout resistance of metallic and geosynthetic reinforcement, a resistance factor, ϕ, of 1.20 shall be used.

C11.5.8

A resistance factor of 1.0 is recommended for the extreme event limit state in view of the unlikely occurrence of the loading associated with the design earthquake. The choice of 1.0 is influenced by the following factors:

- For competent soils that are not expected to lose strength during seismic loading (e.g., due to liquefaction of saturated cohesionless soils or strength reduction of sensitive clays), the use of static strengths for seismic loading is usually conservative, as rate-of-loading effects tend to increase soil strength for transient loading.
- Earthquake loads are transient in nature and hence, if soil yield occurs, the net effect is an accumulated small deformation as opposed to foundation failure. This assumes that global stability is adequate.

Using a resistance factor of 1.0 for soil assumes ductile behavior. While this is a correct assumption for many soils, it is inappropriate for brittle soils where there is a significant post-peak strength loss (e.g., stiff over-consolidated clays, sensitive soils). For such conditions, special studies will be required to determine the appropriate combination of resistance factor and soil strength.

For bearing resistance, a slightly lower resistance factor of 0.8 is recommended for gravity and semigravity walls and 0.9 for MSE walls to reduce the possibility that a bearing resistance failure could occur before the wall moves laterally in sliding, reducing the likelihood of excessive wall tilting or collapse, consistent with the design objective of no collapse.

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 C11.6.1.1-1. 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.

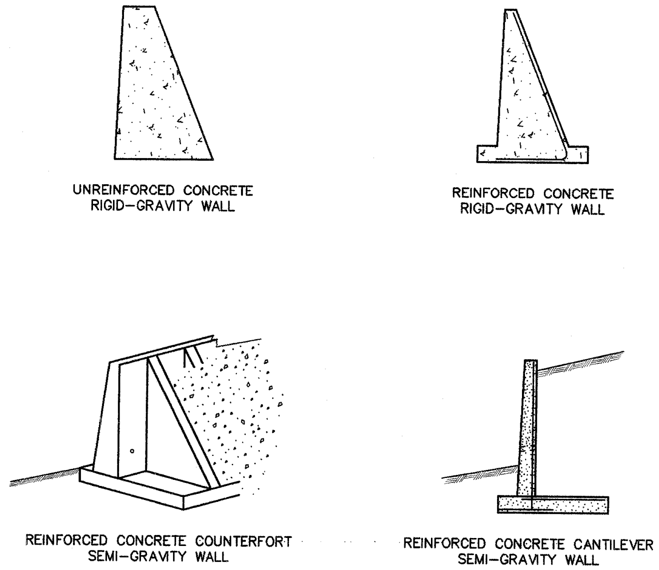


Figure C11.6.1.1-1—Typical Rigid Gravity and Semigravity Walls

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;
- 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.

C11.6.1.2

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, ϕ, 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

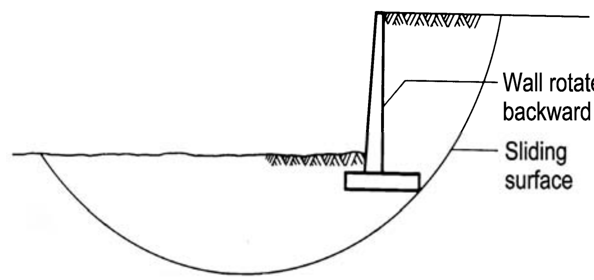


Figure C11.6.2.3-1—Retaining Wall Overall Stability Failure

Figure C11.6.2.3-1 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.

Available slope stability programs produce a single factor of safety, *FS*. The specified resistance factors are essentially the inverse of the *FS* that should be targeted in the slope stability program.

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 11.6.3.2-1.

  The vertical stress shall be calculated as follows:

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

  where:

  ΣV = the summation of vertical forces, and the other variables are as defined in Figure 11.6.3.2-1

- 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 11.6.3.2-2. 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 11.6.3.2-2. If the resultant is outside the middle one-third of the base:

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

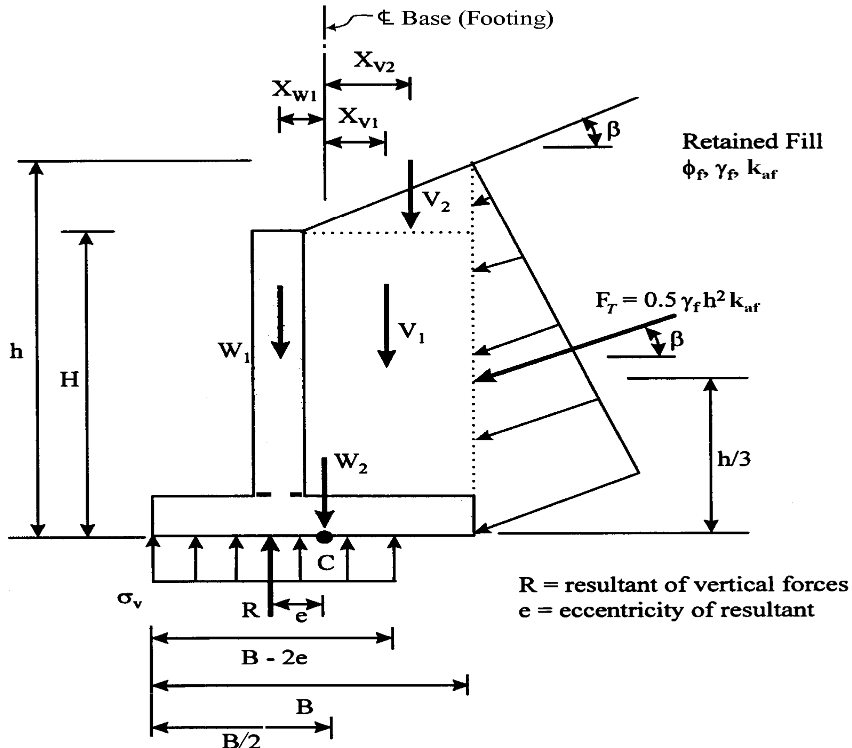
$$\sigma_{vmin} = 0 \qquad (11.6.3.2\text{-}5)$$

  where the variables are as defined in Figure 11.6.3.2-2.

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.

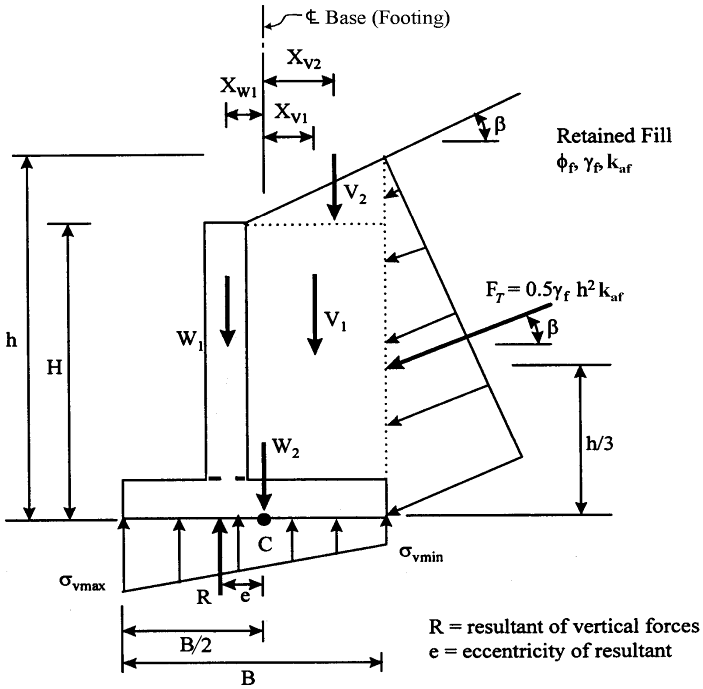
See Article C11.5.5 for application of load factors for bearing resistance and eccentricity.



Summing moments about Point C:

$$e = \frac{\left(F_T Cos\beta\right)h/3 - \left(F_T \sin\beta\right)B/2 - V_1X_{V1} - V_2X_{V2} + W_1X_{W1}}{V_1 + V_2 + W_1 + W_2 + F_T \sin\beta}$$

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



If e > B/6, σ_{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—Eccentricity Limits

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

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

C11.6.3.3

The specified criteria for the location of the resultant, coupled with investigation of the bearing pressure, replace the investigation of the ratio of stabilizing moment to overturning moment. Location of the resultant within the middle two-thirds 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

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

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.

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 for Abutments and Conventional Retaining Walls

11.6.5.1—General

Rigid gravity and semigravity retaining walls and abutments shall be designed to meet overall stability, external stability, and internal stability requirements during seismic loading. The procedures specified in Article 11.6.2.3 for overall stability, Article 11.6.3 for bearing stability, and Article 10.6.3.4 for sliding stability shall be used but including seismically induced earth pressure and inertial forces and using Extreme Event I limit state load and resistance factors as specified in Article 11.5.8.

For seismic eccentricity evaluation of walls with 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 $\gamma_{EQ} = 1.0$. For values of γ_{EQ} between 0.0 and 1.0, the resultant location restriction shall be obtained by linear interpolation of the values given in this Article.

C11.6.5.1

The estimation of seismic design forces should account for wall inertia forces in addition to the equivalent static-forces. For semigravity walls in which the footing protrudes behind the back of the wall face (i.e., the heel), the weight of the soil located directly above the heel of the footing should be included in the calculated wall inertial force.

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 in accordance with Article 14.6.3.

For bridge abutments, the abutment seismic design should be conducted in accordance with Articles 5.2 and 6.7 of AASHTO's *Guide Specifications for LRFD Seismic Bridge Design* but with the following exceptions:

- k_h should be determined as specified in Article 11.6.5.2 and
- Lateral earth pressures should be estimated in accordance with Article 11.6.5.3.

To evaluate safety against structural failure (i.e., internal stability) for seismic design, the structural design of the wall elements shall comply with the provisions of Sections 5, 6, 7, and 8.

The total lateral force to be applied to the wall due to seismic and earth pressure loading, P_{seis}, should be determined considering the combined effect of P_{AE} and P_{IR}, in which:

$$P_{IR} = k_h(W_w + W_s) \qquad (11.6.5.1\text{-}1)$$

and where:

P_{AE} = dynamic lateral earth pressure force
P_{IR} = horizontal inertial force due to seismic loading of the wall mass
k_h = seismic horizontal acceleration coefficient
W_w = the weight of the wall
W_s = the weight of soil that is immediately above the wall, including the wall heel

To investigate the wall stability considering the combined effect of P_{AE} and P_{IR} and considering them not to be concurrent, the following two cases should be investigated:

- Combine 100 percent of the seismic earth pressure P_{AE} with 50 percent of the wall inertial force P_{IR} and
- Combine 50 percent of P_{AE} but no less than the static active earth pressure force (i.e., F_1 in Figure 11.10.5.2-1), with 100 percent of the wall inertial force P_{IR}.

The most conservative result from these two analyses should be used for design of the wall. Alternatively, if approved by the Owner, more sophisticated numerical methods may be used to investigate nonconcurrence. For competent soils that do not lose strength under seismic loading, static strength parameters should be used for seismic design.

- For cohesive soils, total stress strength parameters based on undrained tests should be used during the seismic analysis.
- For clean cohesionless soils, the effective stress friction angle should be used.

The static lateral earth pressure force acting behind the wall is already included in P_{AE} (i.e., P_{AE} is the combination of the static and seismic lateral earth pressure). See Articles 3.11.6.3 and 11.10.10.1 for definition of terms in Figure 11.6.5.1-1 not specifically defined in this Article.

Since P_{AE} is the combined lateral earth pressure force resulting from static earth pressure plus dynamic effects, the static earth pressure as calculated based on the lateral earth pressure coefficient K_a should not be added to the seismic earth pressure calculated in Article 11.6.5.3. The static lateral earth pressure coefficient, K_a, is, in effect, increased during seismic loading to K_{AE} (see Article 11.6.5.3) due to seismically induced inertial forces on the active wedge, and the potential increase in the volume of the active wedge itself due to flattening of the active failure surface. P_{AE} does not include any additional lateral forces caused by permanent surcharge loads located above the wall (e.g., the static force F_p, and the dynamic force $k_hW_{\text{surcharge}}$ in Figure 11.6.5.1-1, in which $W_{\text{surcharge}}$ is the weight of the surcharge). If the generalized limit equilibrium method (GLE) is used to calculate seismic lateral earth pressure on the wall, the effect of the surcharge on the total lateral force acting on the wall during seismic loading may, however, be taken directly into account when determining P_{AE}. Note that the inertial force due to the weight of the concentrated surcharge load, $k_hW_{\text{surcharge}}$, and the static force F_p are separate and both act during seismic loading. They must therefore both be included in the seismic wall stability analysis. F_p is calculated as specified in Article 3.11.6.

For evaluating external stability of the wall and for evaluating safety against structural failure of the wall (internal stability), the simplest design approach that will ensure a safe result is to combine the total seismic earth pressure force with the inertial response of the wall section, assuming both are in phase. This approach is conservative in that the peak inertial response of the wall mass is not likely to occur at the same time as the peak seismic active pressure. Previous design practice, at least for MSE walls, has been to combine the full wall inertial force with only 50 percent of the dynamic increment of the total earth pressure (i.e., $P_{AE} - P_A$) to account for this lack of concurrence in the design forces.

Research using centrifuge testing of reduced scale walls by Al Atik and Sitar (2010) indicated that these two seismic forces are out of phase, in that when dynamic earth pressure was at its maximum, the wall inertial force was at its minimum and was very close to zero. When the wall inertial force was at its maximum, the total seismic earth pressure (i.e., P_{AE}) was close to its static value. They also indicated, however, that more coincidence between these two forces may still be possible for some wall configurations and ground motions. Nakamura (2006) made similar observations regarding lack of concurrence of these forces based on dynamic centrifuge testing he conducted. This research indicates that treating the two forces as nonconcurrent is justified in most cases.

- For sensitive cohesive soils or saturated cohesionless soils, the potential for earthquake-induced strength loss shall be addressed in the analysis.

See Al Atik and Sitar (2010) and Nakamura (2006) for examples of the application of numerical methods to investigate this issue of nonconcurrent forces.

The inertial force associated with the soil mass on the wall heel behind the retaining wall is not added to the active seismic earth pressure when structurally designing the retaining wall. The basis for excluding this inertial force is that movement of this soil mass is assumed to be in phase with the structural wall system with the inertial load transferred through the heel of the wall. Based on typical wave lengths associated with seismic loading, this is considered a reasonable assumption. However, the inertial force for the soil mass over the wall heel is included when determining the external stability of the wall.

Additional discussion and guidance on the selection of soil parameters for seismic design of walls and the potential consideration of soil cohesion are provided by Anderson et al. (2008).

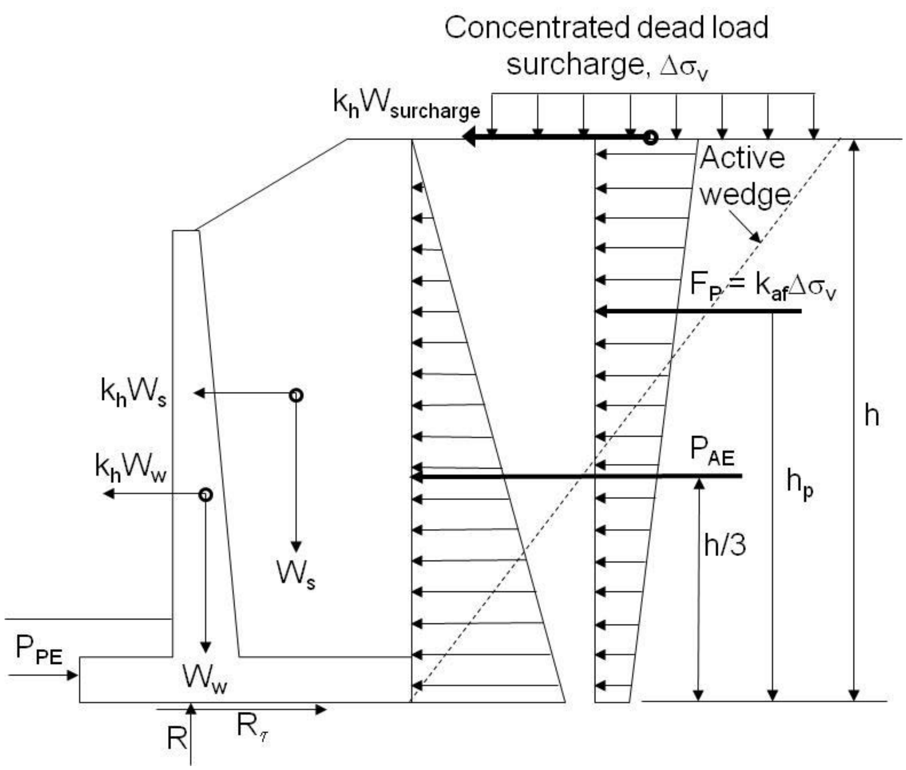


Figure 11.6.5.1-1—Seismic Force Diagram for Gravity Wall External Stability Evaluation

11.6.5.2—Calculation of Seismic Acceleration Coefficients for Wall Design

11.6.5.2.1—Characterization of Acceleration at Wall Base

The seismic horizontal acceleration coefficient (k_h) for computation of seismic lateral earth pressures and loads shall be determined on the basis of the *PGA* at the ground surface (i.e., $k_{h0} = F_{pga}\,PGA = A_s$, where k_{h0} is the seismic horizontal acceleration coefficient assuming zero wall displacement occurs). The acceleration coefficient determined at the original ground surface should be considered to be the acceleration coefficient acting at the wall base. For walls founded on Site Class A or B soil (hard or soft rock), k_{h0} shall be based on 1.2 times the site-adjusted peak ground acceleration coefficient (i.e., $k_{h0} = 1.2F_{pga}PGA$).

The seismic vertical acceleration coefficient, k_v, should be assumed to be zero for the purpose of calculating lateral earth pressures, unless the wall is significantly affected by near fault effects (see Article 3.10), or if relatively high vertical accelerations are likely to be acting concurrently with the horizontal acceleration.

C11.6.5.2.1

A_s is determined as specified in Article 3.10.

In most situations, vertical and horizontal acceleration are at least partially out of phase. Therefore, k_v is usually rather small when k_h is near its maximum value. The typical assumption is to assume that k_v is zero for wall design.

11.6.5.2.2—Estimation of Acceleration Acting on Wall Mass

The seismic lateral wall acceleration coefficient, k_h, shall be determined considering the effects of wave scattering or ground motion amplification within the wall and the ability of the wall to displace laterally. For wall heights less than 60.0 ft, simplified pseudostatic analyses may be considered acceptable for use in determining the design wall mass acceleration. For wall heights greater than 60.0 ft, special dynamic soil structure interaction design analyses should be performed to assess the effect of spatially varying ground motions within and behind the wall and lateral deformations on the wall mass acceleration.

The height of wall, *h*, shall be taken as the distance from the bottom of the heel of the retaining structure to the ground surface directly above the heel.

If the wall is free to move laterally under the influence of seismic loading and if lateral wall movement during the design seismic event is acceptable to the Owner, k_{h0} should be reduced to account for the allowed lateral wall deformation. The selection of a maximum acceptable lateral deformation should take into consideration the effect that deformation will have on the stability of the wall under consideration, the desired seismic performance level, and the effect that deformation could have on any facilities or structures supported by the wall. Where the wall is capable of displacements of 1.0 to 2.0 in. or more during the design seismic event, k_h may be reduced to $0.5k_{h0}$ without conducting a deformation analysis using the Newmark

C11.6.5.2.2

The designer may use k_h for wall design without accounting for wave scattering and lateral deformation effects; however, various studies have shown that the ground motions in the mass of soil behind the wall will often be lower than k_{h0} at the ground surface, particularly for taller walls. However, in some cases, it is possible to have amplification of the ground motion in the wall relative to the wall base ground motion.

The desired performance of walls during a design seismic event can range from allowing limited damage to the wall or displacement of the wall to requiring damage-free, post-earthquake conditions. In many cases, a well-designed gravity or semigravity wall could slide several inches and perhaps even a foot or more, as well as tilt several degrees, without affecting the function of the wall or causing collapse, based on past performance of walls in earthquakes. However, the effect of such deformation on the facilities or structures located above, behind, or in front of the wall must also be considered when establishing an allowable displacement.

Recent work completed as part of NCHRP Report 611 (Anderson et al., 2008) concluded that, when using the Newmark method, the amount of permanent ground displacement associated with $k_h = 0.5k_{h0}$ will in most cases be less than 1.0 to 2.0 in. (i.e., use of $k_h = 0.5k_{h0}$ provides conservative results).

Details of specific simplified procedures that may be used to estimate wave scattering effects and lateral wall deformations to determine k_h are provided in method (Newmark, 1965) or a simplified version of it. This reduction in k_h shall also be considered applicable to the investigation of overall stability of the wall and slope.

A Newmark sliding block analysis or a simplified form of that type of analysis should be used to estimate lateral deformation effects, unless the Owner approves the use of more sophisticated numerical analysis methods to establish the relationship between k_h and the wall displacement. Simplified Newmark analyses should only be used if the assumptions used to develop them are valid for the wall under consideration.

Appendix A11. Those simplified procedures include Kavazanjian et al. (2003), Anderson et al. (2008), and Bray et al. (2009, 2010). Additional background needed to conduct a full Newmark sliding block analysis is also provided in Appendix A11.

Alternate Methods of Estimating Permanent Displacement

The simplified, Newmark Method-based equations given above present a relatively quick method of estimating the yield acceleration for a given maximum acceptable displacement or, alternatively, the displacements that will occur if the capacity to demand (*C*/*D*) ratio for a limiting equilibrium stability analysis is less than 1.0. Alternatively, two-dimensional numerical methods that allow seismic time history analyses may be used to estimate permanent displacements. Such models require considerable expertise in the set-up and interpretation of model results, particularly relative to the selection of strength parameters consistent with seismic loading. For this reason, use of this alternate approach should be adopted only with the Owner's concurrence.

11.6.5.3—Calculation of Seismic Active Earth Pressures

Seismic active and passive earth pressures for gravity and semigravity retaining walls shall be determined following the methods described in this Article. Site conditions, soil and retaining wall geometry, and the earthquake ground motion determined for the site shall be considered when selecting the most appropriate method to use.

The seismic coefficient (k_h) used to calculate seismic earth pressures shall be the site-adjusted peak ground surface acceleration identified in Article 11.6.5.2.1 (i.e., A_s) after adjustments for 1) spectral or wave scattering effects and 2) limited amounts of permanent deformation as determined appropriate for the wall and anything the wall movement could affect (Article 11.6.5.2.2). The vertical acceleration coefficient (k_v) should be assumed to be zero for design as specified in Article 11.6.5.2.1.

For seismic active earth pressures, either the Mononobe-Okabe (M-O) Method or the Generalized Limit Equilibrium (GLE) Method should be used. For wall geometry or site conditions for which the M-O Method is not suitable, the GLE Method should be used.

The M-O Method shall be considered acceptable for determination of seismic active earth pressures only where:

- The material behind the wall can be reasonably approximated as a uniform, cohesionless soil within a zone defined by a 3H:1V wedge from the heel of the wall,
- The backfill is not saturated and in a loose enough condition such that it can liquefy during shaking, and

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The suitability of the method used to determine active and passive earth seismic pressures should be determined after a review of features making up the static design, such as backfill soils and slope above the retaining wall. These conditions, along with the ground motion for a site, will affect the method selection.

The complete M-O equation is provided in Appendix A11. The M-O equation for seismic active earth pressure is based on the Coulomb earth pressure theory and is therefore limited to design of walls that have homogeneous, dry cohesionless backfill. The M-O equation has been shown to be most applicable when the backfill is homogenous and can be characterized as cohesionless.

Another important limitation of the M-O equation is that there are combinations of acceleration and slope angle in which real solutions to the equation are no longer possible or that result in values that rapidly approach infinity. The contents of the radical in this equation must be positive for a real solution to be possible. In past practice, when the combination of acceleration and slope angle results in a negative number within the radical in the equation, rather than allowing that quantity to become negative, it was artificially set at zero. While this practice made it possible to get a value of K_{AE}, it also tended to produce excessively conservative results. Therefore, in such cases it is better to use an alternative method.

- The combination of peak ground acceleration and backslope angle do not exceed the friction angle of the soil behind the wall, as specified in Eq. 11.6.5.3-1.

$$\phi \geq i + \theta_{MO} = i + \arctan\left(\frac{k_h}{1-k_v}\right) \qquad (11.6.5.3\text{-}1)$$

where:

ϕ = the wall backfill friction angle
i = backfill slope angle (degrees)
k_h = the horizontal acceleration coefficient
k_v = the vertical acceleration coefficient

Once K_{AE} is determined, the seismic active force, P_{AE}, shall be determined as:

$$P_{AE} = 0.5\ \gamma\ h^2\ K_{AE} \qquad (11.6.5.3\text{-}2)$$

where:

K_{AE} = seismic active earth pressure coefficient (dim)
γ = the soil unit weight behind the wall (kips/ft^3)
h = the total wall height, including any soil surcharge present, at the back of the wall

The external active force computed from the generalized limit equilibrium method, distributed over the wall height h, shall be used as the seismic earth pressure.

The equivalent pressure representing the total static and seismic active force (P_{AE}) as calculated by either method should be distributed using the same distribution as the static earth pressure used to design the wall when used for external stability evaluations, as illustrated in Figure 11.6.5.1-1, but no less than H/3. For the case when a sloping soil surcharge is present behind the wall face (h in Figure 11.6.5.1-1), this force shall be distributed over the total height, h.

For complex wall systems or complex site conditions, with the owner's approval, dynamic numerical soil structure interaction (SSI) methods should also be considered.

For many situations, gravity and semigravity walls are constructed by cutting into an existing slope where the soil properties differ from the backfill that is used behind the retaining wall. In situations where soil conditions are not homogeneous and the failure surface is flatter than the native slope, seismic active earth pressures computed for the M-O equation using the backfill properties may no longer be valid, particularly if there is a significant difference in properties between the native and backfill soils.

However, the M-O Method has been used in past design practice for estimating seismic earth pressures for many of these situations due to lack of an available alternative. Various approaches to force the method to be usable for such situations have been used, such as estimating some type of average soil property for layered soil conditions or limiting the acceleration to prevent the radical in the equation from being negative, among others. With the exception of seismic passive pressure estimation, this practice has typically resulted in excessively conservative designs and it is not recommended to continue this practice.

The GLE Method consists of conducting a seismic slope stability analysis in which k_h is used as the acceleration coefficient, typically using a computer program in which the applied force necessary to maintain equilibrium (i.e., a capacity/demand ratio of 1.0) under seismic loading is determined. This force is P_{AE}. Specific procedures used to conduct this method are provided in Appendix A11. The GLE Method should be used when the M-O Method is not suitable due to the wall geometry, seismic acceleration level, or site conditions.

The Coulomb Wedge Equilibrium Method, also referred to as the trial wedge method, as described in Peck et al. (1974) and Caltrans (2010), may also be used for situations when the M-O method is not suitable but a hand calculation method is desired, provided that the soil conditions are not too complex (e.g., layered soil conditions behind the wall). Other than the potential ability to use the trial wedge method as a hand calculation method, it has no real advantages over the GLE method.

Recent studies have indicated that classic limit equilibrium based methods such as the M-O, GLE, and the Coulomb Wedge Equilibrium methods may be overly conservative even if the limitations listed above are considered. See Bray et al. (2010) and Lew et al. (2010a, 2010b) with regard to the generation of seismic earth pressures behind walls and the applicability of the Mononobe-Okabe or similar method.

For cases in which the wall seismic design result appears to be excessively conservative relative to past experience in earthquakes, other than taking advantage of the no seismic analysis provisions in Article 11.5.4.2, there are no simple solutions; numerical dynamic soil structure interaction (SSI) modeling may need to be considered. See Bray et al. (2010) for an example. Dynamic numerical SSI solutions may also be needed for more complex wall systems and for walls in which the seismic loading is severe. Due to the complexities of such analyses, an independent peer review of the analysis and results is recommended.

Past practice for locating the resultant of the static and seismic earth pressure for external wall stability has been to either assume a uniform distribution of lateral earth pressure for the combined static plus seismic stress or, if the static and seismic components of earth pressure are treated separately, using an inverted trapezoid for the seismic component, with the seismic force located at 0.6h above the wall base, and combining that force with the normal static earth pressure distribution (Seed and Whitman, 1970). More recent research indicates the location of the resultant of the total earth pressure (static plus seismic) should be located at h/3 above the wall base (Clough and Fragaszy, 1977; Al Atik and Sitar, 2010; Bray et al., 2010; and Lew et al., 2010a and b). See Appendix A11 for additional discussion on this issue. As a minimum, the combined resultant of the active and seismic earth pressure (i.e., P_{AE}) should be located no lower, relative to the wall base, than the static earth pressure resultant. However, a slightly higher combined static/seismic resultant location (e.g., 0.4h to 0.5h) may be considered, since there is limited evidence the resultant could be higher, especially for walls in which the impact of failure is relatively high.

Most natural cohesionless soils have some fines content that contributes cohesion, particularly for short-term loading conditions. Similarly, cohesionless backfills are rarely fully saturated and partial saturation provides for some apparent cohesion, even for most clean sands. The effects of cohesion, whether actual or apparent, are an important issue to be considered in practical design problems.

The M-O equation has been extended to c-ϕ soils by Prakash and Saran (1966), where solutions were obtained for cases including the effect of tension cracks and wall adhesion. Similar solutions have also been discussed by Richards and Shi (1994) and Chen and Liu (1990).

Results of analyses by Anderson et al. (2008) show a significant reduction in the seismic active pressure for small values of cohesion. From a design perspective, this means that even a small amount of cohesion in the soil could reduce the demand required for retaining wall design.

From a design perspective, the uncertainties in the amount of cohesion or apparent cohesion make it difficult to explicitly incorporate the contributions of cohesion in many situations, particularly in cases where clean backfill materials are being used, regardless of the potential benefits of apparent cohesion that could occur if the soil is partially saturated. Realizing these uncertainties, the following guidelines are suggested.

- Where cohesive soils are being used for backfill or where native soils have a clear cohesive strength component, the designer should give consideration to incorporating some effects of cohesion in the determination of the seismic coefficient.
- If the cohesion in the soil behind the wall results primarily from capillarity stresses, especially in relatively low fines content soils, it is recommended that cohesion be neglected when estimating seismic earth pressure.

The groundwater within the active wedge or submerged conditions (e.g., as in the case of a retaining structure in a harbor or next to a lake or river) can influence the magnitude of the seismic active earth pressure. The time-averaged mean groundwater elevation should be used when assessing groundwater effects.

If the soil within the wedge is fully saturated, then the total unit weight (γ_t) should be used to estimate the earth pressure when using the M-O Method, under the assumption that the soil and water move as a unit during seismic loading. This situation will apply for soils that are not free draining.

If the backfill material is a very open granular material, such as quarry spalls, it is possible that the water will not move with the soil during seismic loading. In this case, the effective unit weight should be used in the pressure determination and an additional force component due to hydrodynamic effects should be added to the wall pressure. Various methods are available to estimate the hydrodynamic pressure (see Kramer, 1996). Generally, these methods involve a form of the Westergaard solution.

11.6.5.4—Calculation of Seismic Earth Pressure for Nonyielding Abutments and Walls

For abutment walls and other walls that are considered nonyielding, the value of k_h used to calculate seismic earth pressure shall be increased to $1.0k_{h0}$, unless the Owner approves the use of more sophisticated numerical analysis techniques to determine the seismically induced earth pressure acting on the wall, considering the ability of the wall to yield in response to lateral loading. In this case, k_h should not be corrected for wall displacement, since displacement is assumed to be zero. However, k_h should be corrected for wave scattering effects as specified in Article 11.6.5.2.2.

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The lateral earth pressure calculation methodologies provided in Article 11.6.5.3 assume that the abutment or wall is free to laterally yield a sufficient amount to mobilize peak soil strengths in the backfill. Examples of walls that may be nonyielding are integral abutments, abutment walls with structural wing walls, tunnel portal walls, and tied back cylinder pile walls. For granular soils, peak soil strengths can be assumed to be mobilized if deflections at the wall top are about 0.5 percent of the abutment or wall height. For walls restrained from movement by structures, batter piles, or anchors, lateral forces induced by backfill inertial forces could be greater than those calculated by M-O or GLE methods of analysis. Simplified elastic solutions presented by Wood (1973) for rigid nonyielding walls also indicate that pressures are greater than those given by M-O and GLE analysis. These solutions also indicate that a higher resultant location for the combined effect of static and seismic earth pressure of $h/2$ may be warranted for nonyielding abutments and walls and should be considered for design. The use of a factor of 1.0 applied to k_{h0} is recommended for design where doubt exists that an abutment or wall can yield sufficiently to mobilize backfill soil strengths. In general, if the lack of ability of the wall to yield requires that the wall be designed for K_0 conditions for the strength limit state, then a k_h of $1.0k_{h0}$ should be used for seismic design.

Alternatively, numerical methods may be used to better quantify the yielding or nonyielding nature of the wall and its effect on the seismic earth pressures that develop, if approved by the Owner.

11.6.5.5—Calculation of Seismic Passive Earth Pressure

For estimating seismic passive earth pressures, wall friction and the deformation required to mobilize the passive resistance shall be considered and a log spiral design methodology shall be used. The M-O Method shall not be used for estimating passive seismic earth pressure.

Seismic passive earth pressures shall be estimated using procedures that account for the friction between the retaining wall and the soil, the nonlinear failure surface that develops in the soil during passive pressure loading, and for wall embedment greater than or equal to 5.0 ft, the inertial forces in the passive pressure zone in front of the wall from the earthquake. For wall embedment depths less than 5.0 ft, passive pressure should be calculated using the static methods provided in Section 3.

In the absence of any specific guidance or research results for seismic loading, a wall interface friction equal to two-thirds of the soil friction angle should be used when calculating seismic passive pressures.

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The seismic passive earth pressure becomes important for walls that develop resistance to sliding from the embedded portion of the wall. For these designs, it is important to estimate passive pressures that are not overly conservative or unconservative for the seismic loading condition. This is particularly the case if displacement-based design methods are used but it can also affect the efficiency of designs based on limit-equilibrium methods.

If the depth of embedment of the retaining wall is less than 5.0 ft, the passive pressure can be estimated using static methods given in Section 3 of these Specifications. For this depth of embedment, the inertial effects from earthquake loading on the development of passive pressures will be small.

For greater depths of embedment, the inertial effects of ground shaking on the development of passive pressures should be considered. This passive zone typically extends three to five times the embedment depth beyond the face of the embedded wall.

Shamsabadi et al. (2007) have developed a methodology for estimating the seismic passive pressures while accounting for wall friction and the nonlinear failure surface within the soil. Appendix A11 of this Section provides charts based on this development for a wall friction of two-thirds of the soil friction angle (ϕ) and a range of seismic coefficients, ϕ values, and soil cohesion (c).

The seismic coefficient used in the passive seismic earth pressure calculation is the same value as used for the seismic active earth pressure calculation. Wave scattering reductions are also appropriate to account for incoherency of ground motions in the soil if the depth of the passive zone exceeds 20.0 ft. For most wall designs the difference between the seismic coefficient behind the wall relative to seismic coefficient of the soil in front of the wall is too small to warrant use of different values.

The M-O equation for seismic passive earth pressure is not recommended for use in determining the seismic passive pressure, despite its apparent simplicity. For passive earth pressure determination, the M-O equation is based on the Coulomb method of determining passive earth pressure; this method can overestimate the earth pressure in some cases.

A key consideration during the determination of static and seismic passive pressures is the wall friction. Common practice is to assume that some wall friction will occur for static loading. The amount of interface friction for static loading is often assumed to range from 50 percent to 80 percent of the soil friction angle. Similar guidance is not available for seismic loading.

Another important consideration when using the seismic passive earth pressure is the amount of deformation required to mobilize this force. The deformation to mobilize the passive earth pressure during static loading is usually assumed to be large—typically 2 percent to 6 percent of the embedded wall height. Similar guidance is not available for seismic loading and therefore the normal approach during design for seismic passive earth pressures is to assume that the displacement to mobilize the seismic passive earth pressure is the same as for static loading.

11.6.5.6—Wall Details for Improved Seismic Performance

Details that should be addressed for gravity and semigravity walls in seismically active areas, defined as Seismic Zone 2 or higher, or a peak ground acceleration A_s greater than 0.15g, include the following:

- Vertical Slip Joints, Expansion Joints, and Vertical Joints between an Abutment Curtain Wall and the Free-Standing Wall: Design to prevent joint from opening up and allowing wall backfill to flow through the open joint without sacrificing the joint's ability to slip to allow differential vertical movement. This also applies to joints at wall corners. Compressible joint fillers, bearing pads, and sealants should be used to minimize damage to facing units due to shaking. The joint should also be designed in a way that allows a minimum amount of relative movement between the adjacent facing units to prevent stress build-up between facing units during shaking (Extreme Event I), as well as due to differential deformation between adjacent wall sections at the joint for the service and strength limit states.

- *Coping at Wall Top:* Should be used to prevent toppling of top facing units and excessive differential lateral movement of the facing.

- *Wall Corners and Abrupt Facing Alignment Changes:* Should be designed for the potential for higher loads to develop during shaking than would be determined using two-dimensional analysis. Wall corners and short radius turns are defined as having an enclosed angle of 120 degrees or less.

- *Wall Backfill Stability:* Backfill should be well graded and angular enough to interlock/bind together well to minimize risk of fill spilling

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These recommended details are based on previous experiences with walls in earthquakes (e.g., Yen et al., 2011). Walls that did not utilize these details tended to have a higher frequency of problems than walls that did utilize these details.

With regard to preventing joints from opening up during shaking, this can be addressed through use of a backup panel placed behind the joint, a slip joint cover placed in front of the joint, or the placement of the geotextile strip behind the facing panels to bridge across the joint. The special units should allow differential vertical movement between facing units to occur while maintaining the functionality of the joint. The amount of overlap between these joint elements and the adjacent facing units is determined based on the amount of relative movement between facing units that is anticipated in much the same way that the bridge seat width is determined for bridges.

Little guidance on the amount of overlap between the backing panel and the facing panels is available for walls but past practice has been to provide a minimum overlap of 2.0 to 4.0 in. A geotextile strip may also be placed between the backfill soil and the joint or joint and backing panel combination. Typical practice has been to use a minimum overlap of the geotextile beyond the edges of the joint of 6.0 to 9.0 in. and the geotextile is usually attached to the back of the panel using adhesive. Typically, a Class 1 or Class 2 high elongation (>50 percent strain at peak strength) drainage geotextile in accordance with AASHTO M 288 is used. Similarly, this technique may be applied to the joint between the facing units and protrusions through the wall facing.

For wall corners, not cast monolithically, a special facing unit formed to go across the corner, providing overlap with adjacent panels, should be used. Regarding the design of wall corners and abrupt changes in the facing alignment, both static and seismic earth pressure through open wall joints.

- *Wall Backfill Silt and Clay Content:* Wall backfills classified as a silt or clay should in general not be used in seismically active areas.
- *Structures and Foundations within the Wall Active Zone:* The effect of these structures and foundations on the wall seismic loading shall be evaluated and the wall designed to take the additional load.
- *Protrusions through the Wall Face:* The additional seismic force transmitted to the wall, especially the facing, through the protruding structure (e.g., a culvert or drainage pipe) shall be evaluated. The effect of differential deformation between the protrusion and the wall face shall also be considered. Forces transmitted to the wall face by the protruding structure should be reduced through the use of compressible joint filler or bearing pads and sealant.

loading may be greater than what would be determined from two-dimensional analysis. Historically, corners and abrupt alignment changes in walls have had a higher incidence of performance problems during earthquakes than relatively straight sections of the wall alignment, as the corners tend to attract dynamic load and increased earth pressures. This should be considered when designing a wall corner for seismic loading.

Note that the corner or abrupt alignment change enclosed angle as defined in the previous paragraph can either be internal or external to the wall.

With regard to wall backfill materials, walls that have used compacted backfills with high silt or clay content have historically exhibited more performance problems during earthquakes than those that have utilized compacted granular backfills. This has especially been an issue if the wall backfill does not have adequate drainage features to keep water out of the backfill and the backfill fully drained. Also, very uniform clean sand backfill, especially if it lacks angularity, has also been problematic with regard to wall seismic performance. The issue is how well it can be compacted and remain in a compacted state. A backfill soil coefficient of uniformity of greater than 4 is recommended and, in general, the backfill particles should be classified as subangular or angular rather than rounded or subrounded. The less angular the backfill particles, the more well graded the backfill material needs to be.

For additional information on good wall details, see Berg et al. (2009). While this reference is focused on MSE wall details, similar details could be adapted for gravity and semigravity walls.

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.

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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 15 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 10.7.2.2 and 10.8.2.1 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.

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 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.

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 degrees − $\phi'_f/2$) from the horizontal in soil or 45 degrees 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_{a1}.
- 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:

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

where:

γ = load factor for horizontal earth pressure, EH (dim.)
k_{a2} = the active earth pressure coefficient for soil 2 (dim.)
γ'_{s1} = the effective soil unit weight for soil 1 (kcf)
H = the design height of the wall (ft)
ϕ = the resistance factor for passive resistance in front of the wall (dim.)
k_{p2} = the passive earth pressure coefficient for soil 2 (dim.)
γ'_{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, $D = 1.2D_o$, 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.

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:

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

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.

- For simple spans with soil arching:

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

- For continuous spans without soil arching:

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

- For continuous spans with soil arching:

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

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. C11.8.5.2-1 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. C11.8.5.2-2 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. C11.8.5.2-3 and C11.8.5.2-4 are applicable for facing which is continuous over several vertical supports, e.g., reinforced shotcrete or concrete.

11.8.6—Seismic Design of Nongravity Cantilever Walls

11.8.6.1—General

The effect of earthquake loading shall be investigated using the Extreme Event I limit state of Table 3.4.1-1 with resistance factor ϕ=1.0 and load factor γ_p =1.0 and an accepted methodology, with the exception of overall stability of the wall, in which case a resistance factor of 0.9 should be used as specified in Article 11.5.8.

The seismic analysis of the nongravity cantilever retaining wall shall demonstrate that the cantilever wall will maintain overall stability and withstand the seismic earth pressures induced by the design earthquake without excessive structural moments and shear on the cantilever wall section. Limit equilibrium methods or numerical displacement analyses shall be used to confirm acceptable wall performance.

Design checks should also be performed for failures below the excavation level but through the structure. These analyses should include the contributions of the structural section to slope stability. If the structural contribution to resistance is being accounted for in the stability assessment, the moments and shears developed by the structural section should be checked to confirm that specified structural limits are not exceeded.

C11.8.6.1

During seismic loading, the nongravity cantilever wall develops resistance to load through the passive resistance of the soil below the excavation depth. The stiffness of the structural wall section above the excavation depth must be sufficient to transfer seismic forces from the soil behind the wall, through the structural section, to the soil below. The seismic evaluation of the nongravity cantilever wall requires, therefore, determination of the demand on the wall from the seismic active earth pressure and the capacity of the soil from the seismic passive soil resistance.

For flexible cantilevered walls, forces resulting from wall inertia effects may be ignored in estimating the seismic design forces. However, for very massive nongravity cantilever wall systems, such as tangent or secant pile walls, wall mass inertia effects should be included in the seismic analysis of the wall.

Two types of stability checks are conducted for the nongravity cantilever wall: global stability and internal stability. In contrast to gravity and semigravity walls, sliding, overturning, and bearing stability are not design considerations for this wall type. By sizing the wall to meet earth pressures, the equilibrium requirements for external stability are also satisfied.

The global stability check for seismic loading involves a general slope failure analysis that extends below the base of the wall. Typically, the embedment depth of the wall is 1.5 to 2 times the wall height above the excavation level. For these depths, global stability is not normally a concern, except where soft layers are present below the toe of the wall.

The global stability analysis is performed with a slope stability program. The failure surfaces used in the analysis should normally extend below the depth of the structure member.

Internal stability for a nongravity cantilever wall refers to the moments and shear forces developed in the wall from the seismic loads.

11.8.6.2—Seismic Active Lateral Earth Pressure

Lateral earth pressures and inertial forces for seismic design of nongravity cantilever walls shall be determined as specified in Article 11.6.5. The resulting active seismic earth pressure shall be distributed as specified in Article 11.6.5.3, above the excavation level as shown in Figure 11.8.6.2-1.

To reduce the lateral seismic acceleration coefficient k_{h0} for the effects of horizontal wall displacement in accordance with Article 11.6.5.2.2, analyses shall demonstrate that the displacements associated with the yield acceleration do not result in any of the following:

C11.8.6.2

In most situations, the nongravity cantilever wall moves enough during seismic loading to develop seismic active earth pressures; however, the amount of movement may not be the 1.0 to 2.0 in. necessary to allow reduction in the seismic coefficient by 50 percent, unless analyses demonstrate that permanent wall movements will occur without damaging the wall components. Beam-column analyses involving *p-y* modeling of the vertical wall elements will usually be required to make this assessment.

If the effect of cohesion in reducing the seismic active earth pressure acting on the wall is considered,

- Yield of structural members making up the wall, such as with a pile-supported wall,
- Loads applied to the lateral support systems (e.g., ground anchors in anchored wall systems; see Article 11.9.6) that exceed the available factored resistance, and
- Unacceptable deformation or damage to any facilities located in the vicinity of the wall.

the reduction in earth pressure due to cohesion should not be combined with a reduction in earth pressure due to horizontal wall displacement.

As described in Article 11.6.5.3, an alternate approach for determining the seismic active earth pressure involves use of the generalized limit equilibrium method. If used for the design of a nongravity cantilever wall, the geometry of the slope stability model should extend from the ground surface to the bottom or toe of the sheet pile or other nongravity cantilever walls in which the wall is continuous both above and below the excavation line in front of the wall. For soldier pile walls, the analysis extends to the excavation level. The seismic active pressure is determined as described in Appendix A11.

The static lateral earth pressure force acting behind the wall is already included in P_{AE} (i.e., P_{AE} is the combination of the static and seismic lateral earth pressure). See Articles 3.11.6.3 and 11.10.10.1 for definition of terms in Figure 11.8.6.2-1 not specifically defined in this Article.

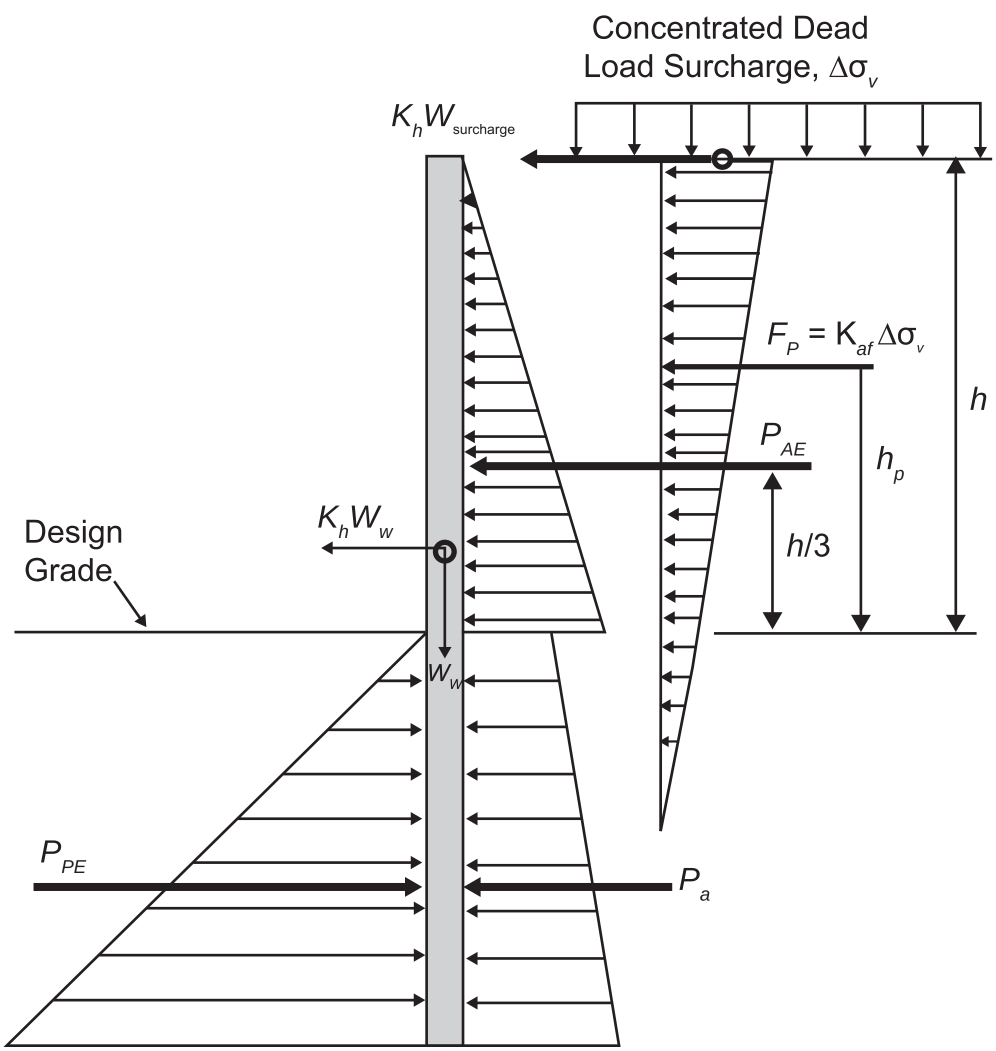


Figure 11.8.6.2-1—Seismic Force Diagram for Nongravity Cantilever Wall External Stability Evaluation

11.8.6.3—Seismic Passive Lateral Earth Pressure

The method used to compute the seismic passive pressure shall consider wall interface friction, the nonlinear failure surface that develops during passive pressure loading, and the inertial response of the soil within the passive pressure wedge for depths greater than 5.0 ft. Cohesion and frictional properties of the soil shall be included in the determination. Passive pressure under seismic loading shall be determined as specified in Article 11.6.5.5.

In the absence of any specific guidance or research results for seismic loading, a wall interface friction equal to two-thirds of the soil friction angle should be used when calculating seismic passive pressures.

The seismic passive pressure shall be applied as a triangular pressure distribution similar to that for static loading. The amount of displacement to mobilize the passive pressure shall also be considered in the analyses.

The peak seismic passive pressure should be based on:

- The time-averaged mean groundwater elevation,
- The full depth of the below-ground structural element, not neglecting the upper 2.0 ft of soil as typically done for static analyses,
- The strength of the soil for undrained loading, and
- The wall friction in the passive pressure estimate taken as two-thirds times the soil strength parameters from a total stress analysis.

In the absence of specific guidance for seismic loading, a reduction factor of 0.67 should be applied to the seismic passive pressure during the seismic check to limit displacement required to mobilize the passive earth pressure.

C11.8.6.3

The effects of live loads are usually neglected in the computation of seismic passive pressure.

Reductions in the seismic passive earth pressure may be warranted to limit the amount of deformation required to mobilize the seismic passive earth pressure, if a limit equilibrium method of analysis is used, to make sure that the wall movement does not result in the collapse of the wall or of structures directly supported by the wall. However, a passive resistance reduction factor near 1.0 may be considered if, in the judgment of the engineer, such deformations to mobilize the passive resistance would not result in wall or supported structure collapse.

If the nongravity cantilever wall uses soldier piles to develop reaction to active pressures, adjustments must be made in the passive earth pressure determination to account for the three-dimensional effects below the excavation level as soil reactions are developed. In the absence of specific seismic studies dealing with this issue, it is suggested that methods used for static loading be adopted. One such method, documented in the California Department of Transportation (Caltrans) *Shoring Manual* (2010), suggests that soldier piles located closer than three pile diameters be treated as a continuous wall. For soldier piles spaced at greater distances, the approach in the *Shoring Manual* depends on the type of soil:

- For cohesive soils, the effective pile width that accounts for arching ranges from one pile diameter for very soft soil to two diameters for stiff soils.
- For cohesionless soils, the effective width is defined as $0.08\phi B$ up to three pile diameters. In this relationship, ϕ is the soil friction angle and B is the soldier pile width.

During seismic loading, the inertial response of the soil within the passive pressure failure wedge will decrease the soil resistance during a portion of each loading cycle. Figures provided in Appendix A11 can be used to estimate the passive soil resistance for different friction values and normalized values of cohesion. A preferred methodology for computing seismic earth pressures with consideration of wall friction, nonlinear soil failure surface, and inertial effects involves use of the procedures documented by Shamsabadi et al. (2007).

11.8.6.4—Wall Displacement Analyses to Determine Earth Pressures

Numerical displacement analyses, if used, shall show that moments, shear forces, and structural displacements resulting from the peak ground surface accelerations are within acceptable levels. These analyses shall be conducted using a model of the wall system that includes the structural stiffness of the wall section, as well as the load displacement response of the soil above and below the excavation level.

C11.8.6.4

Numerical displacement methods offer a more accurate and preferred method of determining the response of nongravity cantilever walls during seismic loading. Either of two numerical approaches can be used. One involves a simple beam-column approach; the second involves the use of a two-dimensional computer model. Both approaches need to appropriately represent the load displacement behavior of the soil and the structural members during loading. For soils, this includes nonlinear stress-strain effects; for structural members, consideration must be given to ductility of the structure, including the use of cracked versus uncracked section properties if concrete structures are being used.

Beam-Column Approach

The pseudostatic seismic response of a nongravity cantilever wall can be determined by representing the wall in a beam-column model with the soil characterized by *p-y* springs. This approach is available within commercially available computer software. The total seismic active pressure above the excavation level is used for wall loading. Procedures given in Article 11.8.6.2 should be used to make this estimate.

For this approach, the p-y curves below the excavation level need to be specified. For discrete structural elements (e.g.., soldier piles), conventional *p-y* curves for piles may be used. For continuous walls or walls with pile elements at closer than 3 diameter spacing, *p*- and *y*-modifiers have been developed by Anderson et al. (2008) to represent a continuous (sheet pile or secant pile) retaining wall. The procedure involves:

- Developing conventional isolated pile *p-y* curves using a 4.0-ft diameter pile following API (1993) procedures for sands or clays.
- Normalizing the isolated *p-y* curves by dividing the *p* values by 4.0 ft.
- Applying the following *p*- and *y*-multipliers, depending on the type of soil, in a conventional beam-column analysis.

Soil Type	*p*-multiplier	*y*-multiplier
Sand	0.5	4.0
Clay	1.0	4.0

It should be noted that the starting point of using a 4.0 ft diameter pile has nothing to do with the actual diameter of the vertical elements in the wall. It is simply a starting point in the procedure to obtain p-y curves that are applicable to a wall. The p-y curves obtained in the final step of this process are intended to be applicable to a continuous wall.

Supporting information for the development and use of the *p*-y approach identified above is presented in Volume 1 of NCHRP 611 Report (Anderson et al., 2008). The earth pressure used as the load in the beam column analysis is determined from one of the limit equilibrium methods, including M-O with or without cohesion or the generalized limit equilibrium procedure, as discussed in Article 11.6.5. The benefit of the *p-y* approach is that it enforces compatibility of deflections, earth pressure, and flexibility of the wall system. The method is in contrast to the limit equilibrium method in which the effects of the wall flexibilities are ignored. This is very important for the seismic design and performance of the wall during seismic event. The deformation and rotation of the wall can easily be captured using the *p-y* approach.

Finite Difference or Finite Element Modeling

Pseudostatic or dynamic finite element or finite difference procedures in computer programs can also be used to evaluate the seismic response of nongravity cantilever walls during seismic loading. For two-dimensional models, it may be necessary to "smear" the stiffness of the structural section below the excavation level to adjust the model to an equivalent two-dimensional representation if the below-grade portion of the wall is formed from discrete piles (e.g., soldier piles).

The finite difference or finite element approach to evaluating wall response will involve a number of important assumptions; therefore, this approach should be discussed with and agreed to by the Owner before being adopted. As part of the discussions, the possible limitations and the assumptions being made for the model should be reviewed.

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 11.9.1-1, 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.

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.

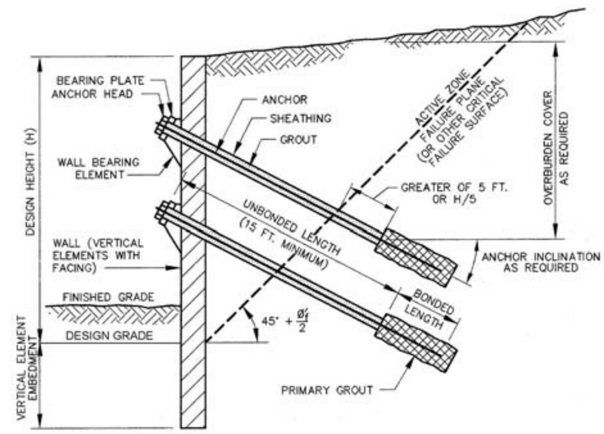


Figure 11.9.1-1—Anchored Wall Nomenclature and Anchor Embedment Guidelines

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 10.6.2.2, 10.7.2.2, and 10.8.2.1 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 C11.9.3.1-1 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 C11.9.3.1-1 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.

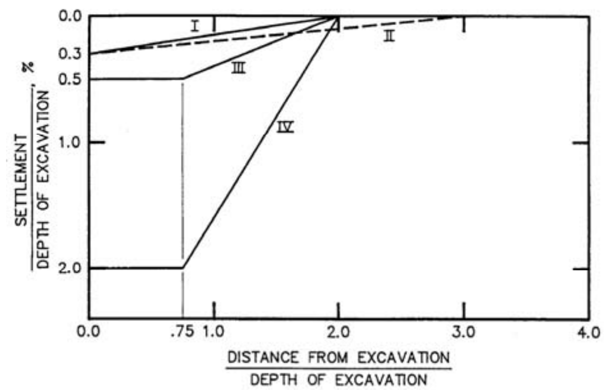
Where noted in the definition of the various curves in Figure C11.9.3.1-1, the basal heave ratio, RBH, shall be taken as:

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

where:

S_u = undrained shear strength of cohesive soil (ksf)
γ_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, σ_v', however, should be calculated with reference to the elevation of the midheight of the exposed wall, with β and σ_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:

$$Q_R = \phi Q_n = \phi \pi d \tau_a L_b \qquad (11.9.4.2\text{-}1)$$

where:

ϕ = resistance factor for anchor pullout (dim.)

Q_n = nominal anchor pullout resistance (kips)

d = diameter of anchor drill hole (ft)

τ_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 C11.9.4.2-1, C11.9.4.2-2, and C11.9.4.2-3 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, τ_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 V. Stiff 2.5–4.0	0.6 to 2 1.5 to 3.6
Medium Plasticity Clay	Stiff 1.0–2.5 V. Stiff 2.5–4.0	2.0 to 5.2 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[a]	Presumptive Ultimate Unit Bond Stress, τ_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

[a] 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, τ_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 C11.9.4.2-1 through C11.9.4.2-3 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 11.5.7-1, 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 C11.9.4.2-1 through C11.9.4.2-3. Use of the resistance factors in Table 11.5.7-1 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 C11.9.4.2-1 through C11.9.4.2-3 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 C11.9.4.2-1 through C11.9.4.2-3 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.

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.

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,

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 degrees 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

- 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.

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

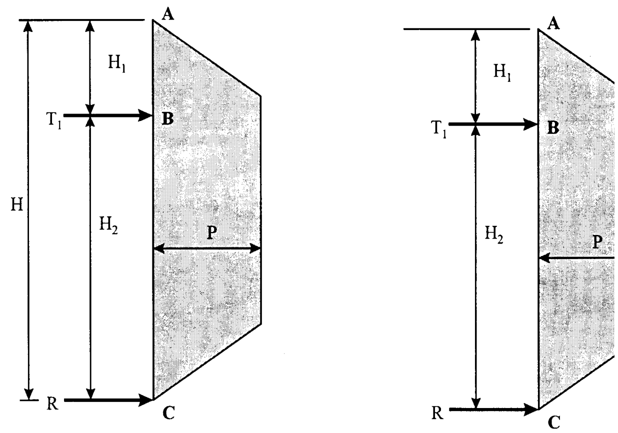
The horizontal component of anchor design force shall be computed using the provisions of Article 11.9.2 and any other horizontal pressure components acting on the wall in Article 3.11. The total anchor design force shall be determined based on the anchor inclination. The horizontal anchor spacing and 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 C11.5.9.1-1 and C11.5.9.1-2. 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.



Tributary area method

T_1 = Load over length $H_1 + H_2/2$

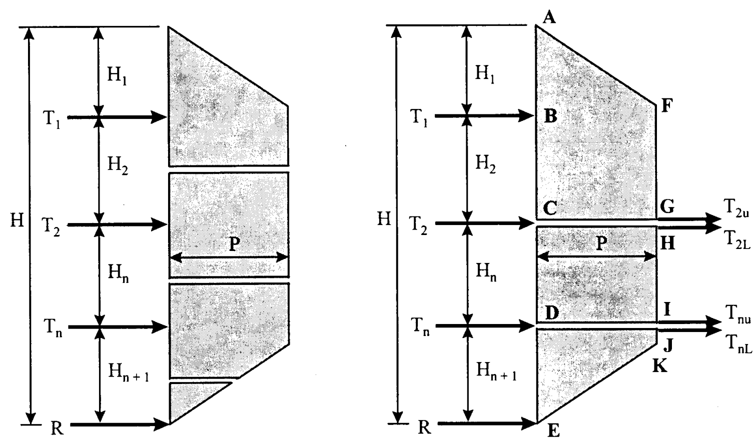
R = Load over length $H_2/2$

Hinge method

T_1 Calculated from $\Sigma M_C = 0$

R = Total earth pressure – T_1

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 except as modified in this Article.

The seismic analysis of the anchored retaining wall shall demonstrate that the anchored wall can maintain overall stability and withstand the seismic earth pressures induced by the design earthquake without exceeding the capacity of the anchors or the structural wall section supporting the soil. Limit equilibrium methods or numerical displacement analyses shall be used to confirm acceptable wall performance.

Anchors shall be located behind the limit equilibrium failure surface for seismic loading. The location of the failure surface for seismic loading shall be established using methods that account for the seismic coefficient and the soil properties (i.e., c and ϕ) within the anchored zone.

C11.9.6

See Article C11.8.6.

The seismic design of an anchored wall involves many of the same considerations as the nongravity cantilever wall. However, the addition of one or more anchors to the wall introduces some important differences in the seismic design check as identified in this Article.

The earth pressures above the excavation level result from the inertial response of the soil mass behind the wall. In contrast to a nongravity cantilever wall, the soil mass includes anchors that have been tensioned to minimize wall deflections under static earth pressures. During seismic loading, the bars or strands making up the unbonded length of the anchor are able to stretch under the imposed incremental seismic loads. In most cases, the amount of elastic elongation in the strand or bar under the incremental seismic load is sufficient to develop seismic active earth pressures but may not be sufficient to allow the horizontal seismic acceleration coefficient, k_{h0}, and associated earth pressure to be reduced to account for permanent horizontal wall displacement. The ability of the wall to deform laterally should be specifically investigated before reducing k_{h0} to account for horizontal wall displacement.

The passive pressure for the embedded portion of the soldier pile or sheet pile wall also plays a part in the stability assessment, as it helps provide stability for the portion of the wall below the lowest anchor. This passive pressure is subject to seismically induced inertial forces that will reduce the passive resistance relative to the static capacity of the pile or wall section. Most often, the embedded portion of the pile involves discrete structural members spaced at 8.0 to 10.0 ft; however, the embedded portion could also involve a continuous wall in the case of a sheet pile or secant pile wall.

Anchors should be located behind the failure surface associated with the calculation of P_{AE}. The location of this failure surface can be determined using either the wedge equilibrium or the generalized limit equilibrium (slope stability) method. Note that this failure surface will likely be flatter than the requirements for anchor location under static loading. When using the wedge equilibrium or the generalized limit equilibrium method, P_{AE} and its associated critical surface should be determined without the anchor forces.

Once the location of the anchor bond zone is defined, an external stability check should be conducted with the anchor forces included, using the anchor test load to define ultimate anchor capacities. This check is performed to confirm that the *C*/*D* ratio is greater than 1.0. Under this loading condition, the critical surface will flatten and could pass through or behind some anchors. However, as long as the *C*/*D* ratio is greater than 1.0, the design is satisfactory.

If the *C*/*D* ratio is less than 1.0, either the unbonded length of the anchor must be increased or the length of the grouted zone must be lengthened. The design check would then be repeated.

The global stability check is performed to confirm that a slope stability failure does not occur below the anchored wall; external stability is checked to confirm the anchors will have sufficient reserve capacity to meet seismic load demands; and internal stability is checked to confirm that moments and shear forces within the structural members, including the anchor strand or bar tensile loads and the head connection, are within acceptable levels for the seismic load.

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 degrees 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:

- Where utilities other than highway drainage are to be constructed within the reinforced zone unless access is provided to utilities without disrupting

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 C11.10.1-1.

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 future access to utilities may be gained without disrupting reinforcements and where leakage from utilities would not create detrimental hydraulic reinforcements and breakage or rupture of utility lines will not have a detrimental effect on the stability of the structure.

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

conditions or degrade reinforcements, utilities in the reinforced zone may be acceptable.

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.

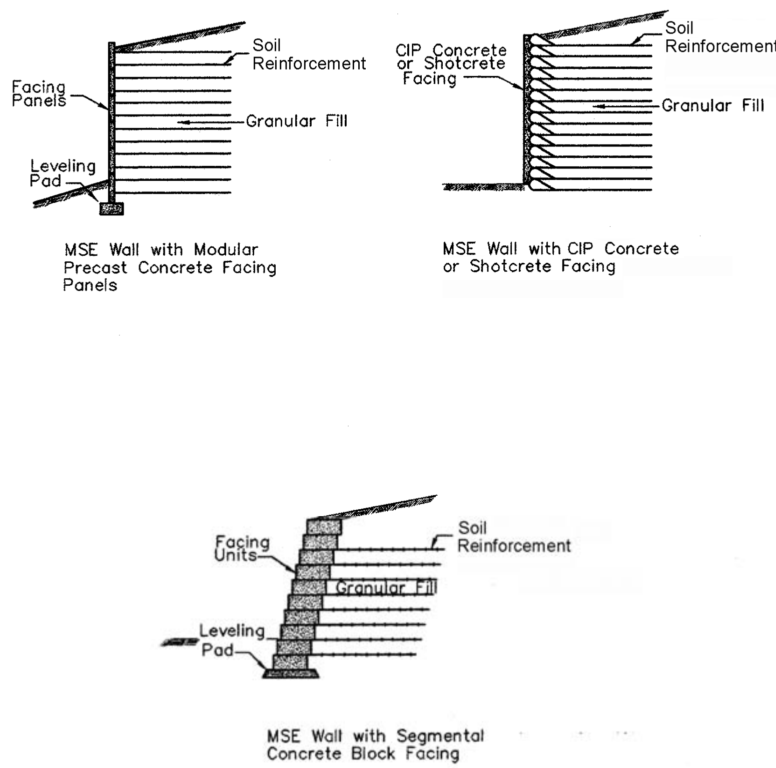


Figure C11.10.1-1—Typical Mechanically Stabilized Earth Walls

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-NHI-10-024 (Berg et al., 2009).

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

Compound stability should also be evaluated for these complex MSE wall systems (see Article 11.10.4.3).

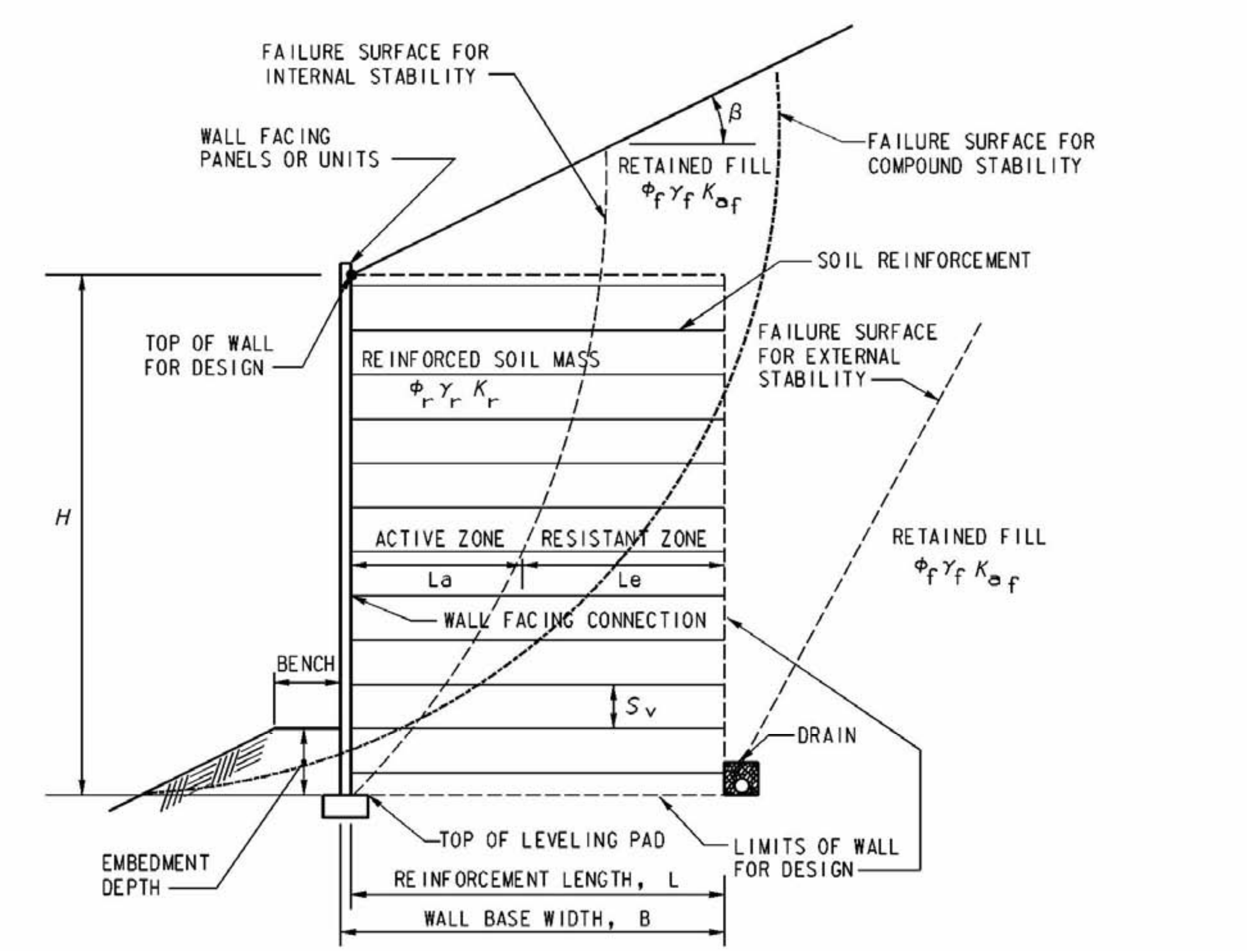
materials used as soil reinforcements as well as the soil mechanics which govern MSE wall behavior.

11.10.2—Structure Dimensions

An illustration of the MSE wall element dimensions required for design is provided in Figure 11.10.2-1.

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.

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.

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.

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-NHI-10-024 (Berg et al., 2009).

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.

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.

C11.10.2.2

The minimum embedment guidelines provided in Table C11.10.2.2-1 may be used to preclude local bearing resistance failure under the leveling pad or footing due to higher vertical stresses transmitted by the facing.

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
	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, Wu 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 1.5Wu 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, Wu, 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 C11.10.4.1-1.

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

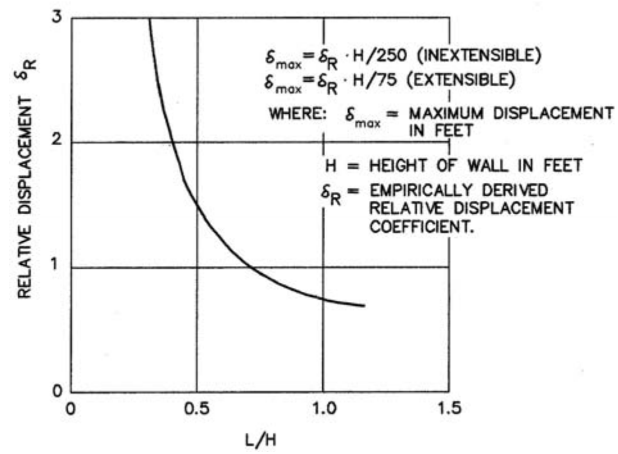
For MSE walls with full height precast concrete facing panels, total settlement should be limited to 2.0 in., and the limiting differential settlement should be 1/500. For walls with segmental concrete block facings, the limiting differential settlement should be 1/200. For walls with welded wire facings 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 C11.10.4.2-1. If significant vertical settlement is anticipated or heavy surcharges are present, lateral displacements could be considerably greater. Figure C11.10.4.2-1 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 additional explanation on how to use this figure, see Berg et al. (2009).

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 11.10.4.3-1 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.

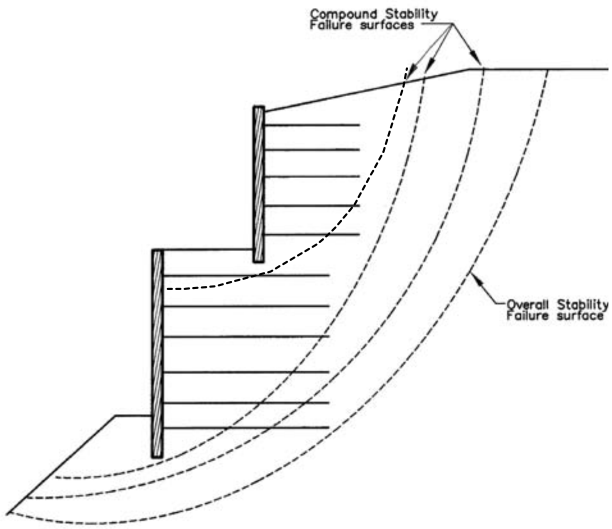


Figure 11.10.4.3-1—Overall and Compound Stability of Complex MSE Wall Systems

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 degrees 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 11.10.5.2-1. 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.

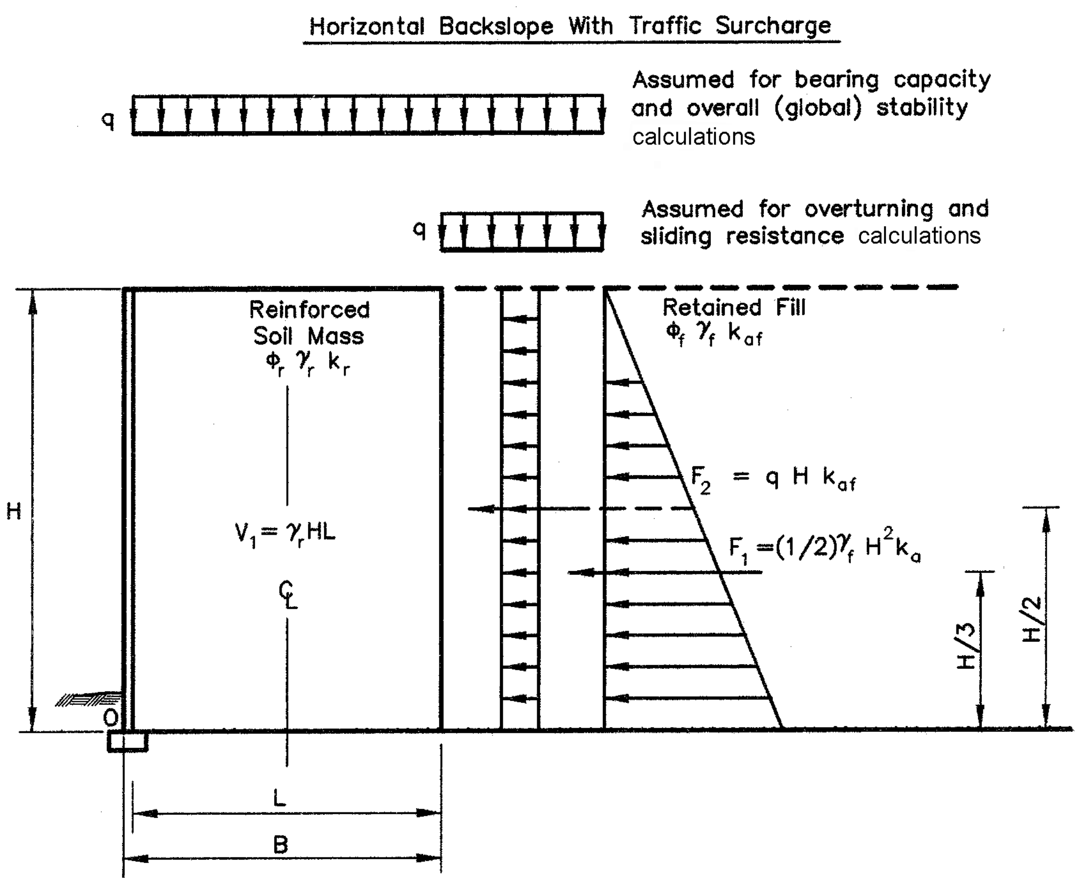


Figure 11.10.5.2-1—External Stability for Wall with Horizontal Backslope and Traffic Surcharge

11.10.5.3—Sliding

The provisions of Article 10.6.3.4 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 ϕr of the reinforced fill and ϕf of the foundation soil. For continuous reinforcements, e.g., grids and sheets, the angle of sliding friction shall be taken as the lesser of ϕr, ϕf and ρ, where ρ is the soil-reinforcement interface friction angle. In the absence of specific data, a maximum friction angle, ϕf, of 30 degrees and a maximum soil-reinforcement interface angle, ρ, of 2/3 ϕf 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 degrees, unless the specific project select backfill is tested for frictional strength by triaxial or direct shear testing methods, AASHTO T 296 and T 297 or T 236, respectively. A design friction angle of greater than 40 degrees shall not be used with the Simplified Method even if the measured friction angle is greater than 40 degrees.

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 degrees 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, σ_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:

γ_P = the load factor for vertical earth pressure EV from Table 3.4.1-2

k_r = horizontal pressure coefficient (dim.)

σ_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.

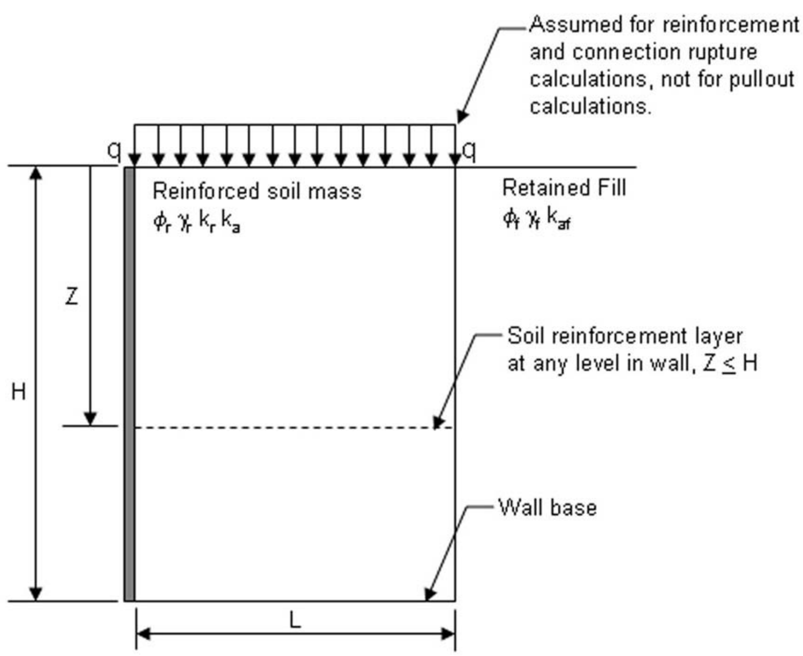
The use of EV for the load factor in this case for both methods (i.e., the Simplified and Coherent Gravity Methods) should be considered an interim measure until research is completed to quantify load prediction bias and uncertainty.

$\Delta\sigma_H$ = 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 11.10.6.2.1-1 and 11.10.6.2.1-2. 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.

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 σ_v; (2) for checking pullout, determine T_{max} with live load surcharge excluded from the calculation of σ_v.

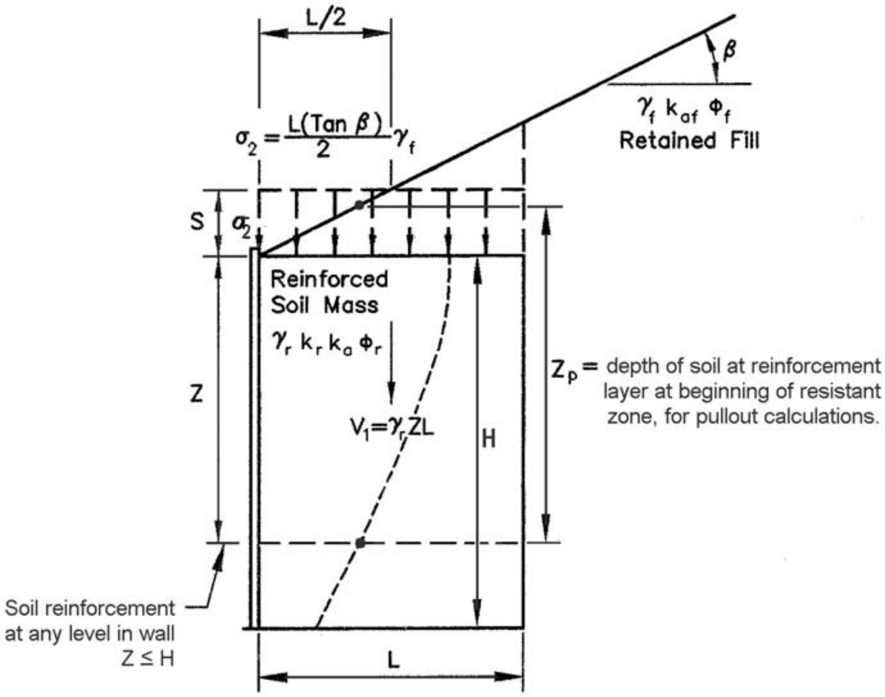


Max Stress: $\sigma_v = \gamma_r Z + q + \Delta\sigma_v$

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



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

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

Determine k_{af} using a slope angle of β

Determine k_r from Figure 11.10.6.2.1-3

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

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

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

For the Simplified Method, the lateral earth pressure coefficient k_r is determined by applying a multiplier to the active earth pressure coefficient, k_a. The k_a multiplier for the Simplified Method shall be determined as shown in Figure 11.10.6.2.1-3. For assessment of reinforcement pullout, the Simplified Method multiplier for steel strip walls shall be used for all steel reinforced walls. For reinforcement rupture, the multiplier applicable to the specific type of steel reinforcement shall be used. For the Coherent Gravity Method, the lateral earth pressure coefficient used for internal stability design of steel reinforced MSE wall systems shall be determined as shown in Figure 11.10.6.2.1-4. For geosynthetic reinforced wall systems, k_a is used throughout the wall height. For both methods, k_a shall be determined using Eq. 3.11.5.3-1, assuming no wall friction, i.e., $\delta = \beta$. For the Coherent Gravity Method, k_0 shall be determined using Eq. 3.11.5.2-1.

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

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

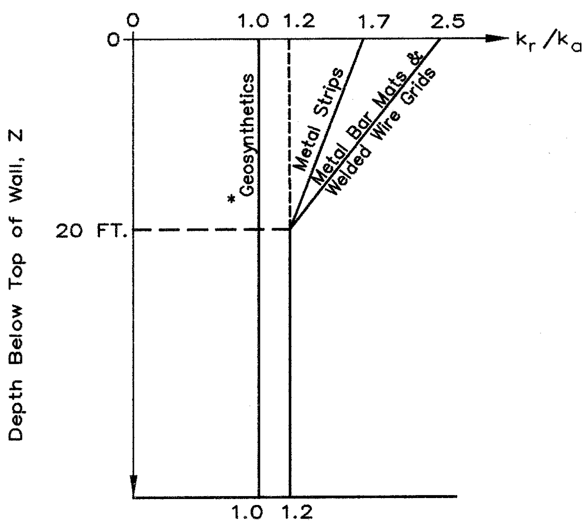
$$T_{\max} = \sigma_H S_v \tag{11.10.6.2.1-2}$$

where:

σ_H = factored horizontal soil stress at the reinforcement (ksf)
S_v = vertical spacing of the reinforcement (ft)

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

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



*Does not apply to polymer strip reinforcement

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

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

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

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

with variables as defined in Figure 3.11.5.3-1.

Based on Figure 11.10.6.2.1-3, the k_a multiplier is a function of the reinforcement type and the depth of the reinforcement below the wall top. Multipliers for other reinforcement types can be developed as needed through analysis of measurements of reinforcement load and strain in full scale structures.

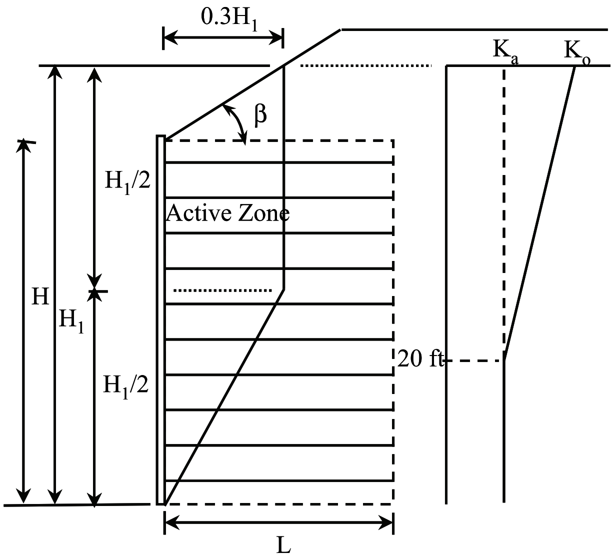


Figure 11.10.6.2.1-4—Determination of Lateral Earth Pressure Coefficients for Internal Stability Design of Steel Reinforced MSE Walls Using the Coherent Gravity Method

11.10.6.2.2—Reinforcement Loads at Connection to Wall Face

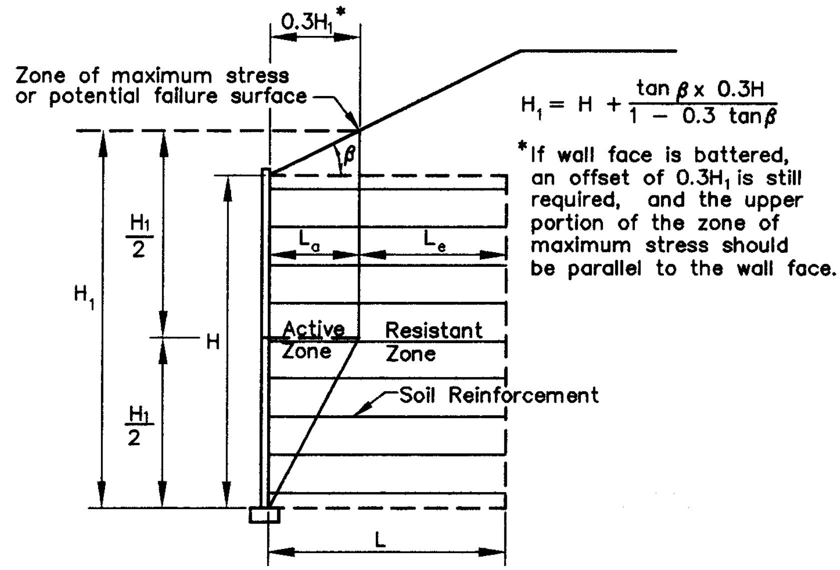
The factored tensile load applied to the soil reinforcement connection at the wall face, T_o, shall be equal to the maximum factored reinforcement tension, T_{max}, for all wall systems regardless of facing and reinforcement type.

11.10.6.3—Reinforcement Pullout

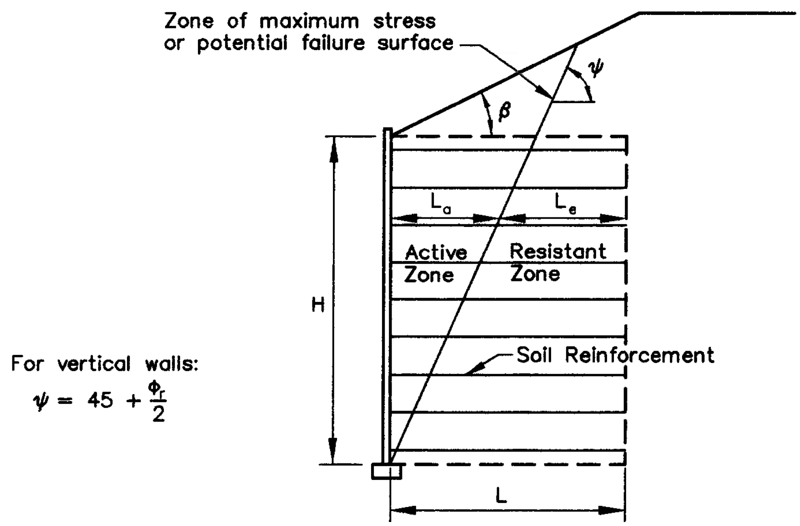
11.10.6.3.1—Boundary between Active and Resistant Zones

The location of the zone of maximum stress for inextensible and extensible wall systems, i.e., the boundary between the active and resistant zones, is determined as shown in Figure 11.10.6.3.1-1. For all wall systems, the zone of maximum stress shall be assumed to begin at the back of the facing elements at the toe of the wall.

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



(a) Inextensible Reinforcements



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

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

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

(b) Extensible Reinforcements

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

11.10.6.3.2—Reinforcement Pullout Design

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

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

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

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

where:

L_e = length of reinforcement in resisting zone (ft)

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

ϕ = resistance factor for reinforcement pullout from Table 11.5.7-1 (dim.)

F^* = pullout friction factor (dim.)

α = scale effect correction factor (dim.)

σ_v = unfactored vertical stress at the reinforcement level in the resistant zone (ksf)

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

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

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

For standard backfill materials (see *AASHTO LRFD Bridge Construction Specifications,* Article 7.3.6.3), with the exception of uniform sands, i.e., coefficient of uniformity $C_u=D_{60}/D_{10} < 4$, in the absence of test data it is acceptable to use conservative default values for F* and α as shown in Figure 11.10.6.3.2-1 and Table 11.10.6.3.2-1. For ribbed steel strips, if the specific C_u for the wall backfill is unknown at the time of design, a C_u of 4.0 should be assumed for design to determine F^*.

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

Reinforcement Type	Default Value for α
All Steel Reinforcements	1.0
Geogrids	0.8
Geotextiles	0.6

C11.10.6.3.2

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

Pullout testing and interpretation procedures (and direct shear testing for some parameters), as well as typical empirical data, are provided in Appendix A of FHWA-NHI-10-025 (Berg et al., 2009).

Recent experience with pullout test results on new geogrids coming into the market has indicated that some materials have pullout values that are lower than the previous F^* default value of 0.8 tan ϕ. Data obtained by D'Appolonia (1999) also indicates that 0.8 tan ϕ is closer to a mean value rather than a default lower bound value for geogrids. The default values for other reinforcement types shown in Figure 11.10.6.3.2-1 are more representative of lower bound values. The F^* default value has thus been lowered to a more conservative value of 0.67 tan ϕ in consideration of these results.

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

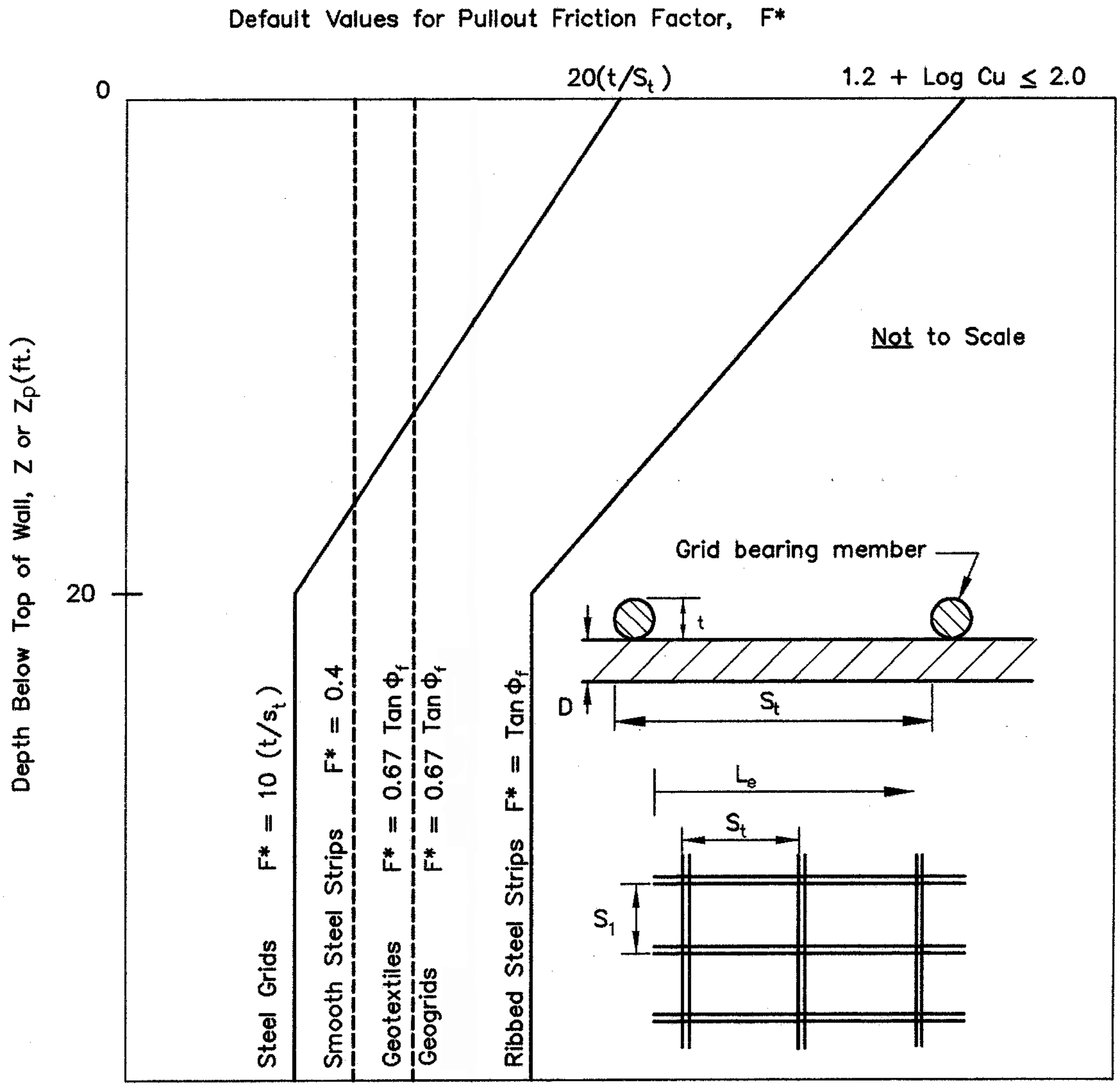


Figure 11.10.6.3.2-1—Default Values for the Pullout Friction Factor, F^*

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

11.10.6.4—Reinforcement Strength

11.10.6.4.1—General

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

At the zone of maximum stress:

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

where:

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

ϕ = resistance factor for reinforcement tension, specified in Table 11.5.7-1 (dim.)

$T_{a\ell}$ = nominal long-term reinforcement design strength (kips/ft)

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

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

At the connection with the wall face:

$$T_o \le \phi T_{ac} R_c \qquad (11.10.6.4.1\text{-}2)$$

where:

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

ϕ = resistance factor for reinforcement tension in connectors specified in Table 11.5.7-1 (dim.)

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

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

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

C11.10.6.4.1

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

$T_{a\ell}$ shall be determined on a long-term strength per unit of reinforcement width basis and multiplied by the reinforcement coverage ratio R_c so that it can be directly compared to T_{max} which is determined on a load per unit of wall width basis (this also applies to T_{ac} and T_o). For discrete, i.e., not continuous, reinforcements, such as steel strips or bar mats, the strength of the reinforcement is converted to a strength per unit of wall width basis as shown in Figures 11.10.6.4.1-1 and 11.10.6.4.1-2. For continuous reinforcement layers, $b = 1$ and $R_c = 1$.

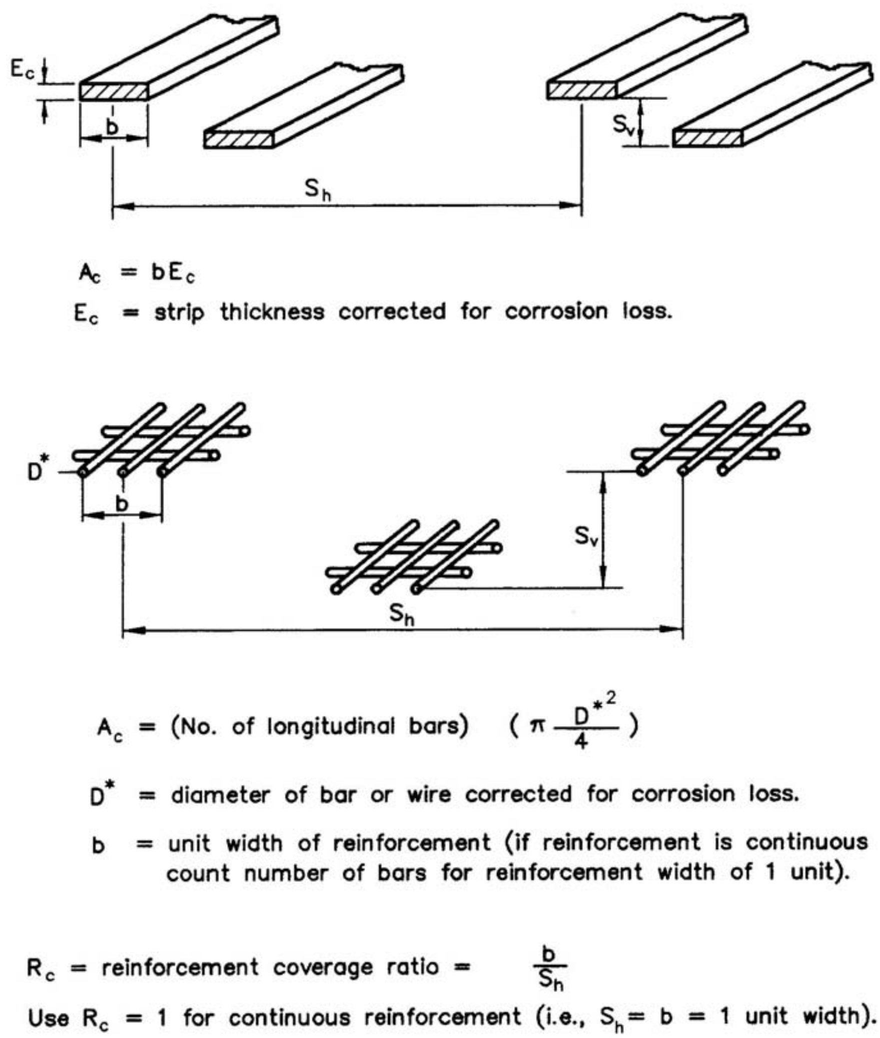
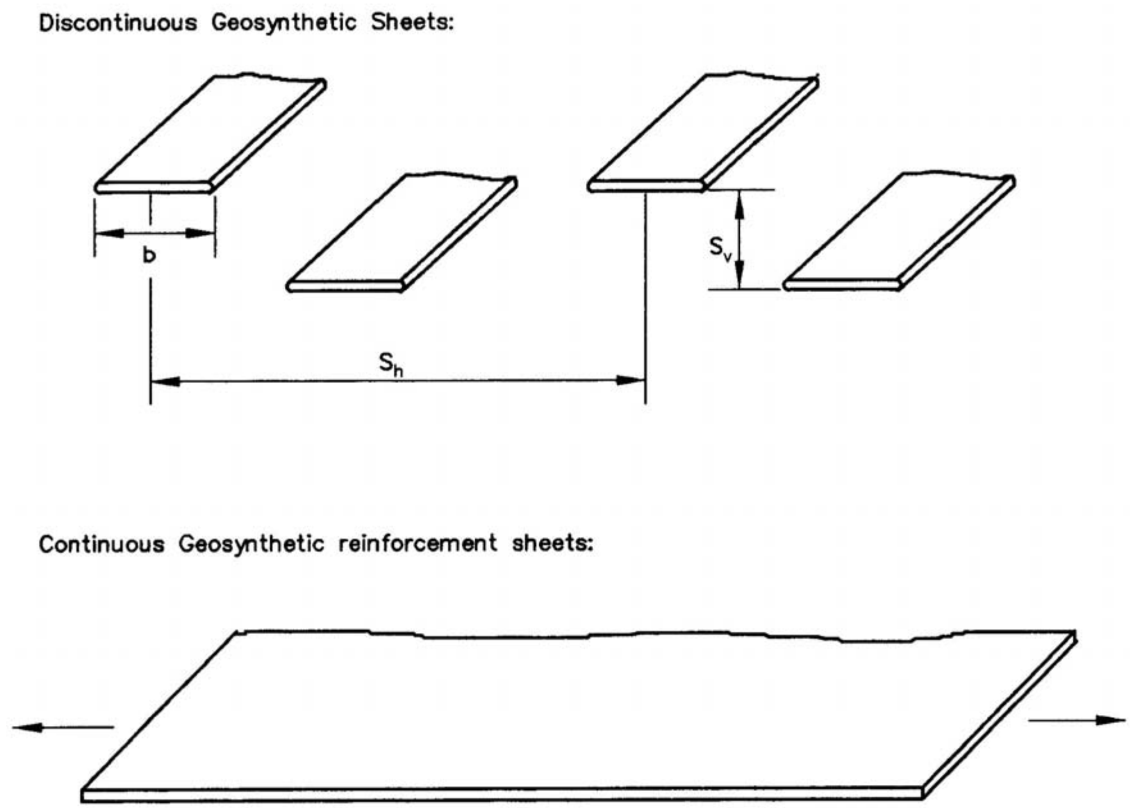


Figure 11.10.6.4.1-1—Reinforcement Coverage Ratio for Metal Reinforcement



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

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

Figure 11.10.6.4.1-2—Reinforcement Coverage Ratio for Geosynthetic Reinforcement

11.10.6.4.2—Design Life Considerations

The provisions of Article 11.5.1 shall apply.

11.10.6.4.2a—Steel Reinforcements

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

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

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

where:

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

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

C11.10.6.4.2a

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

The backfill specifications contained in *AASHTO LRFD Bridge Construction Specifications*, Section 7, for MSE structures using steel reinforcements present minimum electrochemical requirements, which will generally ensure a mild to moderate potential for corrosion. Where deicing salts are used, adequate drainage provisions for salt laden runoff is required. In some cases, an impervious membrane may be required between the pavement structure and the select backfill. Criteria for evaluating potential corrosion losses are given in Elias et. al (2009).

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

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

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

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

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

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

Transverse and longitudinal grid members shall be sized in accordance with ASTM A185. The transverse wire diameter shall be less than or equal to the longitudinal wire diameter.

Galvanized coatings shall be a minimum of 2 oz./ft^2 or 3.4 mils. in thickness, applied in conformance to AASHTO M 111M/M 111 (ASTM A123/A 123M) for strip-type reinforcements or ASTM A641 for bar mat or grid-type steel reinforcement.

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

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

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

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

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

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

Corrosion-resistant coatings should generally be limited to galvanization.

There is insufficient evidence at this time regarding the long-term performance of epoxy coatings for these coatings to be considered equivalent to galvanizing. If epoxy-type coatings are used, they should meet the requirements of ASTM A884 for bar mat and grid reinforcements, or AASHTO M 284M/M 284 for strip reinforcements, and have a minimum thickness of 16 mils.

11.10.6.4.2b—Geosynthetic Reinforcements

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

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

- Poor performance of failure will not have severe consequences
- The soil is considered nonaggressive
- The polymer material meets the requirements provided in Table 11.10.6.4.2b-1

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

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

- pH, as determined by AASHTO T 289, $I = 4.5$ to 9 for permanent applications and 3 to 10 for temporary applications,
- Maximum soil particle size is less than 0.75 in., unless full scale installation damage tests are conducted in accordance with ASTM D5818,
- Soil organic content, as determined by AASHTO T 267 for material finer than the 0.0787 in. (No. 10) sieve ≤1 percent, and
- Design temperature at wall site:
  - ≤ 86°F for permanent applications
  - ≤ 95°F for temporary applications

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

C11.10.6.4.2b

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

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

reinforcement forms the primary connection between the body of the wall and the facing.

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

3) *Polymer Requirements*: Polymers which are likely to have good resistance to long-term chemical degradation shall be used if a single default reduction factor is to be used, to minimize the risk of the occurrence of significant long-term degradation. The polymer material requirements provided in Table 11.10.6.4.2b-1 shall, therefore, be met if detailed product specific data as described in AASHTO PP 66 and Elias, et al. (2009) is not obtained. Polymer materials not meeting the requirements in Table 11.10.6.4.2b-1 may be used if this detailed product specific data extrapolated to the design life intended for the structure are obtained.

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

For applications involving:

- Severe consequences of poor performance or failure,
- Aggressive soil conditions,
- Polymers not meeting the specific requirements set in Table 11.10.6.4.2b-1, or
- A desire to use an overall reduction factor less than the default reduction factor recommended herein,

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

Guidelines for product-specific studies to determine *RF* are provided in Elias et al. (2009) and AASHTO PP 66, a provisional standard that is based on WSDOT Standard Practice T925 (WSDOT, 2009). Independent product-specific data from which *RF* may be determined can be obtained from the AASHTO National Transportation Product Evaluation Program (NTPEP) website at http://www.ntpep.org.

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

Polymer Type	Property	Test Method	Criteria to Allow Use of Default *RF*
Polypropylene	UV Oxidation Resistance	ASTM D4355	Minimum 70% strength retained after 500 hrs. in weatherometer
Polyethylene	UV Oxidation Resistance	ASTM D4355	Minimum 70% strength retained after 500 hrs. in weatherometer
Polypropylene	Thermo-Oxidation Resistance	ENV ISO 13438:1999, Method A	Minimum 50% strength retained after 28 days
Polyethylene	Thermo-Oxidation Resistance	ENV ISO 13438:1999, Method B	Minimum 50% strength retained after 56 days
Polyester	Hydrolysis Resistance	Intrinsic Viscosity Method (ASTM D4603) and GRI Test Method GG8, or Determine Directly Using Gel Permeation Chromatography	Minimum Number Average Molecular Weight of 25000
Polyester	Hydrolysis Resistance	ASTM D7409	Maximum of Carboxyl End Group Content of 30
All Polymers	Survivability	Weight per Unit Area (ASTM D5261)	Minimum 270 g/m^2
All Polymers	% Post-Consumer Recycled Material by Weight	Certification of Materials Used	Maximum of 0%

11.10.6.4.3—Design Tensile Resistance

11.10.6.4.3a—Steel Reinforcements

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

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

where:

$T_{a\ell}$ = nominal long-term reinforcement design strength (kips/ft)
F_y = minimum yield strength of steel (ksi)
A_c = area of reinforcement corrected for corrosion loss (Figure 11.10.6.4.1-1) (in.^{2})
b = unit width of reinforcement (Figure 11.10.6.4.1-1) (ft)

11.10.6.4.3b—Geosynthetic Reinforcements

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

C11.10.6.4.3b

$T_{a\ell}$ is the long-term tensile strength required to prevent rupture calculated on a load per unit of reinforcement width basis. T_{ult} is the ultimate tensile

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

where:

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

and:

$T_{a\ell}$ = nominal long-term reinforcement design strength (kips/ft)
T_{ult} = minimum average roll value (MARV) ultimate tensile strength (kips/ft)
RF = combined strength reduction factor to account for potential long-term degradation due to installation damage, creep and chemical aging (dim.)
RF_{ID} = strength reduction factor to account for installation damage to reinforcement (dim.)
RF_{CR} = strength reduction factor to prevent long-term creep rupture of reinforcement (dim.)
RF_D = strength reduction factor to prevent rupture of reinforcement due to chemical and biological degradation (dim.)

strength of the reinforcement determined from wide width tensile tests specified in ASTM D4595 for geotextiles and ASTM D6637 for geogrids. The value selected for T_{ult} is the minimum average roll value (MARV) for the product to account for statistical variance in the material strength.

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

Guidelines for determination of RF_{ID}, RF_{CR}, and RF_D from product-specific data are provided in AASHTO PP 66 and Elias et al. (2009). PP 66 is based on WSDOT Standard Practice T925 (WSDOT, 2009). Independent product-specific data from which RF_{ID}, RF_{CR}, and RF_D may be determined can be obtained from the AASHTO National Transportation Product Evaluation Program (NTPEP) website at http://www.ntpep.org.

Note that RF_D is generally not based on long-term performance testing unless the soil is considered to be chemically aggressive. Instead, for typical soil defined as chemically nonaggressive, the index tests and criteria identified in Table 11.10.6.4.2b-1 are used to establish a default value for RF_D that can be used in combination with the product specific values of RF_{ID} and RF_{CR} to determine a product specific value of RF to use for design. For products meeting the requirements in Table 11.10.6.4.2b-1 used in chemically nonaggressive soil, a default value of RF_D of 1.3 may be used (AASHTO, 2010; WSDOT, 2009; Berg, et al., 2009). Additional guidance on the selection of RF_D is provided in Berg, et al. (2009).

For wall applications which are defined as not having severe consequences should poor performance or failure occur, having nonaggressive soil conditions, and if the geosynthetic product meets the minimum requirements listed in Table 11.10.6.4.3b-1, the long-term tensile strength of the reinforcement may be determined using a default reduction factor for RF as provided in Table 11.10.6.4.3b-1 in lieu of product-specific test results.

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

Application	Total Reduction Factor, *RF*
All applications, but with product-specific data obtained and analyzed in accordance with AASHTO PP 66	All reduction factors shall be based on product specific data. Neither RF_{ID} nor RF_D shall be less than 1.1.
Permanent applications not having severe consequences should poor performance or failure occur, nonaggressive soils, and polymers meeting the requirements listed in Table 11.10.6.4.2b-1	7.0
Temporary applications not having severe consequences should poor performance or failure occur, nonaggressive soils, and polymers meeting the requirements listed in Table 11.10.6.4.2b-1 provided product-specific data are not available	3.5

11.10.6.4.4—Reinforcement/Facing Connection Design Strength

11.10.6.4.4a—Steel Reinforcements

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

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

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

11.10.6.4.4b—Geosynthetic Reinforcements

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

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

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

where:

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

C11.10.6.4.4b

The long-term creep reduced geosynthetic strength at the connection with the wall facing is obtained by reducing T_{ult} by CR_{cr} using the connection/seam strength determined in accordance with long-term connection strength test protocol as described in Appendix A of Elias et al. (2001). The connection test is similar in nature to a wide width tensile test (ASTM D4595 or ASTM D6637), except that one end of the reinforcement material is sandwiched between two courses of concrete blocks to form one of the grips. This protocol consists of a series of connection creep tests carried out over an extended period of time to evaluate the potential for creep rupture at the connection. CR_{cr} is taken as the creep reduced connection strength, T_{crc}, extrapolated to the specified design life, divided by the ultimate wide width tensile strength (ASTM D4595 or D6637) for the reinforcement material lot used for the connection strength testing, T_{lot}.

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

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

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

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

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

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

The objective of the connection design is to assess the long-term capacity of the connection. If rupture is the mode of failure, the long-term effects of creep and durability on the geosynthetic reinforcement at the connection, as well as on the connector materials, must be taken into account, as the capacity of the connection is controlled by the reinforcement or connector long-term strength. If pullout is the mode of failure, the capacity of the connection is controlled by the frictional interface between the facing blocks and the geosynthetic reinforcement. It is assumed for design that this interface is not significantly affected by time dependent mechanisms such as creep or chemical degradation. This again is consistent with the design of the soil reinforcement within the wall backfill. The load bearing fibers or ribs of the geosynthetic do not necessarily have to experience rupture in the connection test for the mode of failure to be rupture. If the connector is a material that is susceptible to creep, failure of the connectors between blocks due to creep rupture of the connector could result in long-term connection strength losses. In these cases, the value of CR_{cr} and RF_D to be used in Eq. C11.10.6.4.4b-1 should be based on the durability of the connector, not the geosynthetic.

Regardless of the failure mode, the long-term connection test referenced in Elias et al. (2001) addresses the long-term capacity of the connection. Eq. C11.10.6.4.4b-1 above should also be considered to conservatively apply to both failure modes, if the long-term connection test is not performed.

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

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

CR_{cr} shall be determined at the anticipated vertical confining pressure at the wall face between the facing blocks. The vertical confining pressure shall be calculated using the Hinge Height Method as shown in Figure 11.10.6.4.4b-1 for a face batter, ω, of greater than 8 degrees. T_{ac} should not be greater than $T_{a\ell}$.

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

Requirements for determining RF_{CR} and RF_D from product-specific data are provided in Article 11.10.6.4.3b and its commentary. The use of default reduction factors may be acceptable where the reinforcement load is maximum, i.e., in the middle of the wall backfill, and still not be acceptable at the facing connection if the facing environment is defined as aggressive.

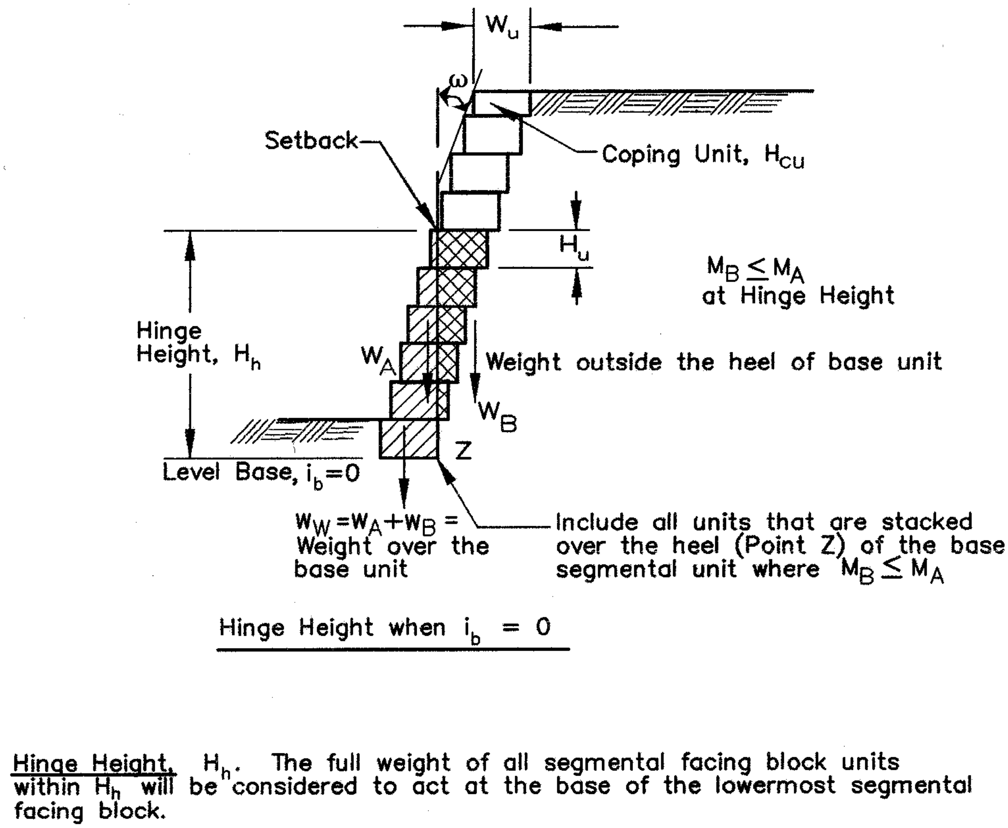


Figure 11.10.6.4.4b-1—Determination of Hinge Height for Segmental Concrete Block Faced MSE Walls

The hinge height, H_h, shown in Figure 11.10.6.4.4b-1, shall be determined as:

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

where:

H_u = segmental facing block unit height (ft)

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

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

ω = wall batter due to setback per course (degrees)

H = total height of wall (ft)

H_h = hinge height (ft)

11.10.7—Seismic Design of MSE Walls

11.10.7.1—External Stability

External stability evaluation of MSE walls for seismic loading conditions shall be conducted as specified in Article 11.6.5, except as modified in this Article for MSE wall design.

Wall mass inertial forces (P_{IR}) shall be calculated based on an effective mass having a minimum width equal to the structural facing width (W_u) plus a portion of the reinforced backfill equal to 50 percent of the effective height of the wall. For walls in which the wall backfill surface is horizontal, the effective height shall be taken equal to H in Figure 11.10.7.1-1. For walls with sloping backfills, the inertial force, P_{IR}, shall be based on an effective mass having a height H_2 and a base width equal to a minimum of 0.5 H_2, in which H_2 is determined as follows:

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

where:

β = slope of backfill (degrees)

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

$$P_{IR} = P_{ir} + P_{is} \qquad (11.10.7.1\text{-}2)$$

where:

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

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

P_{IR} shall act at the combined centroid of reinforced wall mass inertial force, P_{ir}, and the inertial force resulting from the mass of the soil surcharge above the reinforced wall volume, P_{is}. P_{ir} shall include the inertial force from the wall facing. The determination of the MSE wall inertial forces shall be as illustrated in Figure 11.10.7.1-1.

C11.10.7.1

Since the reinforced soil mass is not really a rigid block, the inertial forces generated by seismic shaking are unlikely to peak at the same time in different portions of the reinforced mass when reinforcing strips or layers start becoming very long, as in the case of MSE walls with steep backslopes in moderately- to-highly seismic areas. This introduces excessive conservatism if the full length of the reinforcing strips is used in the inertia determination. Past design practice, as represented in previous editions of these Specifications, recommended that wall mass inertial force be limited to a soil volume equal to 50 percent of the effective height of the wall.

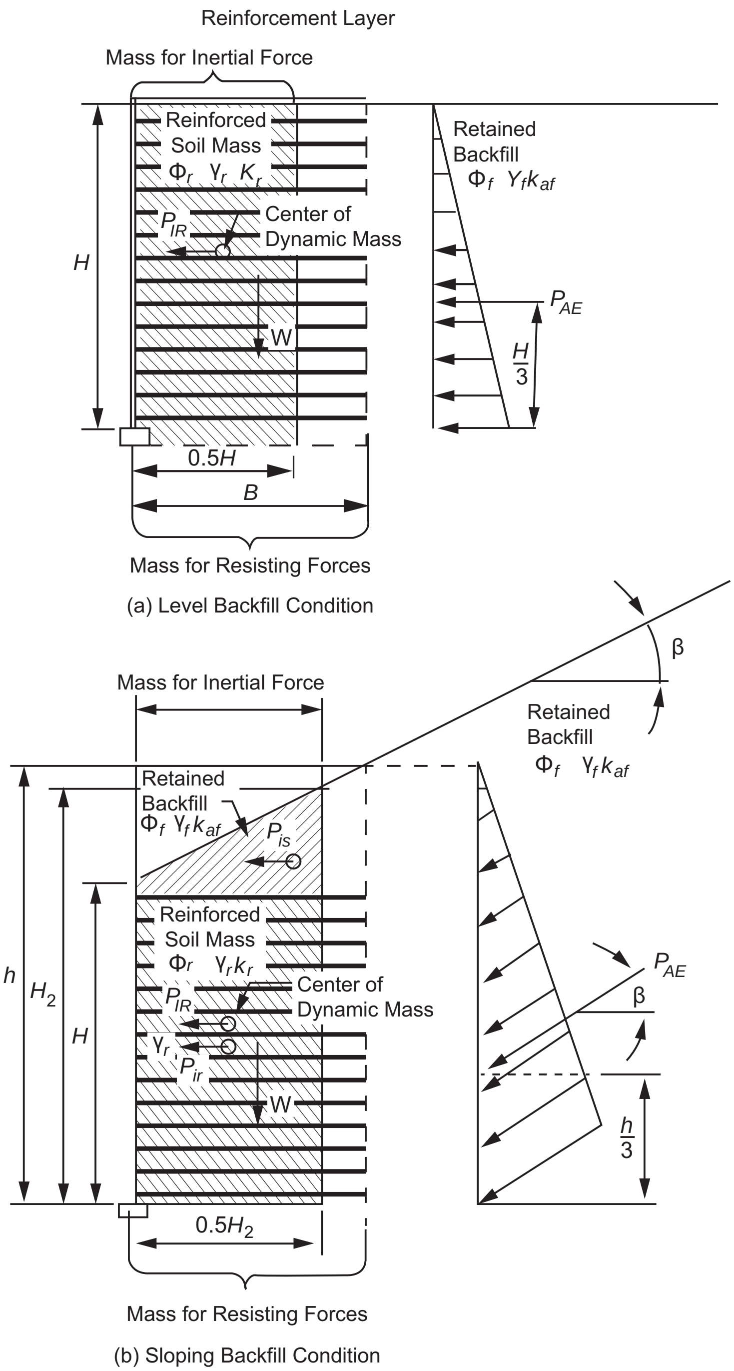


Figure 11.10.7.1-1—Seismic External Stability of an MSE Wall

11.10.7.2—Internal Stability

Reinforcements shall be designed to withstand horizontal forces generated by the internal inertia force, P_i, and the static forces. The total inertia force, P_i, per unit length of structure shall be considered equal to the mass of the active zone times the wall acceleration coefficient, k_h, reduced for lateral displacement of the wall during shaking. The reduced acceleration coefficient, k_h, should be consistent with the value of k_h used for external stability.

C11.10.7.2

In past design practice, as presented in previous editions of these Specifications, the design method for seismic internal stability assumes that the internal inertial forces generating additional tensile loads in the reinforcement act on an active pressure zone that is assumed to be the same as that for the static loading case. A bilinear zone is defined for inextensible reinforcements such as metallic strips and a linear zone for extensible strips.

For walls with inextensible (e.g., steel) reinforcement, this inertial force shall be distributed to the reinforcements proportionally to their resistant areas on a load per unit width of wall basis as follows:

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

For walls with extensible reinforcement, this inertial force shall be distributed uniformly to the reinforcements on a load per unit width of wall basis as follows:

$$T_{md} = \gamma\left(\frac{P_i}{n}\right) \qquad (11.10.7.2\text{-}2)$$

where:

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

γ = load factor for EQ loads from Table 3.4.1-1 (dim.)

P_i = internal inertia force due to the weight of backfill within the active zone, i.e., the shaded area on Figure 11.10.7.2-1 (kips/ft)

K_hW_a = where W_a is the weight of the active zone and K_h is calculated as specified in Article 11.6.5.1.

n = total number of reinforcement layers in the wall (dim)

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

This pressure distribution should be determined from the total inertial force using k_h (after reduction for wave scattering and lateral displacement).

The total factored load applied to the reinforcement on a load per unit of wall width basis as shown in Figure 11.10.7.2-1 is determined as follows:

$$T_{total} = T_{max} + T_{md} \qquad (11.10.7.2\text{-}3)$$

where:

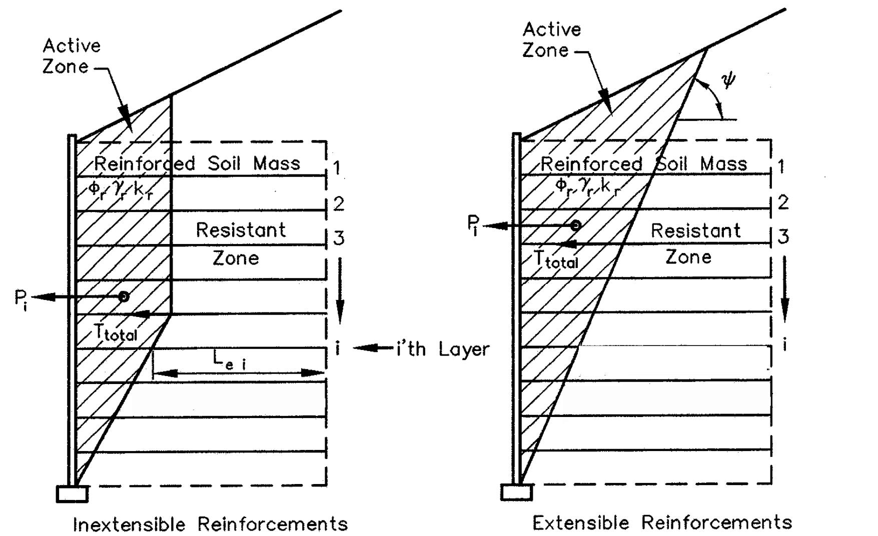
T_{max} = the factored static load applied to the reinforcements determined using Eq. 11.10.6.2.1-2.

Whereas it could reasonably be anticipated that these active zones would extend outwards for seismic cases, as for M-O analyses, results from numerical and centrifuge models indicate that the reinforcement restricts such outward movements and only relatively small changes in location are seen.

In past design practice, as presented in previous editions of these Specifications, the total inertial force is distributed to the reinforcements in proportion to the effective resistant lengths, L_{ei}. This approach follows the finite element modeling conducted by Segrestin and Bastick (1988) and leads to higher tensile forces in lower reinforcement layers.

In the case of internal stability evaluation, Vrymoed (1989) used a tributary area approach that assumes that the inertial load carried by each reinforcement layer increases linearly with height above the toe of the wall for equally spaced reinforcement layers. A similar approach was used by Ling et al. (1997) in limit equilibrium analyses as applied to extensible geosynthetic reinforced walls. This concept would suggest that longer reinforcement lengths could be needed at the top of walls with increasing acceleration levels, and the AASHTO approach could be unconservative, at least for geosynthetic reinforced walls. Numerical modeling of both steel and geosynthetic reinforced walls by Bathurst and Hatami (1999) shows that the distribution of the reinforcement load increase caused by seismic loading tends to become more uniform with depth as the reinforcement stiffness decreases, resulting in a uniform distribution for geosynthetic reinforced wall systems and a triangular distribution for typical steel reinforced wall systems. Hence, the Segrestin and Bastick (1988) method has been preserved for steel reinforced wall systems and, for geosynthetic reinforced wall systems, a uniform load distribution approach is specified.

With regard to the horizontal acceleration coefficient, k_h, past editions of these Specifications have not allowed k_h to be reduced to account for lateral deformation. Based on the excellent performance of MSE walls in earthquakes to date, it appears that this is likely a conservative assumption and it is therefore reasonable to allow reduction of k_h for internal stability design corresponding to the lateral displacement permitted in the design of the wall for external stability.



ψ = angle of active zone boundary as determined from Figure 11.10.6.3.1-1.

P_i = Internal inertial force due to the weight of the backfill within the active zone.

L_{ei} = The length of reinforcement in the resistant zone of the i'th layer.

T_{max} = The factored load per unit wall width applied to each reinforcement layer due to static forces.

T_{md} = The factored load per unit wall width applied to each reinforcement layer due to dynamic forces.

The total factored load per unit wall width applied to each reinforcement layer,

$T_{total} = T_{max} + T_{md}$

Figure 11.10.7.2-1—Seismic Internal Stability of an MSE Wall

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

For the static component:

$$S_{rs} \geq \frac{T_{max} RF}{\phi R_c} \quad (11.10.7.2\text{-}4)$$

For the dynamic component:

$$S_{rt} \geq \frac{T_{md} RF_{ID} RF_D}{\phi R_c} \quad (11.10.7.2\text{-}5)$$

where:

ϕ = resistance factor for combined static/earthquake loading from Table 11.5.7-1 (dim.)

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

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

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

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

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

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

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

$$T_{ult} = S_{rs} + S_{rt} \quad (11.10.7.2\text{-}6)$$

For pullout of steel or geosynthetic reinforcement:

$$L_e \geq \frac{T_{total}}{\phi\,(0.8F^* \alpha \sigma_v C R_c)} \quad (11.10.7.2\text{-}7)$$

The reinforcement must be designed to resist the dynamic component of the load at any time during its design life. Design for static loads requires the strength of the reinforcement at the end of the design life to be reduced to account for creep and other degradation mechanisms. Strength loss in polymeric materials due to creep requires long term, sustained loading. The dynamic component of load for seismic design is a transient load and does not cause strength loss due to creep. The resistance of the reinforcement to the static component of load, T_{max}, must, therefore, be handled separately from the dynamic component of load, T_{md}. The strength required to resist T_{max} must include the effects of creep, but the strength required to resist T_{md} should not include the effects of creep.

where:

L_e = length of reinforcement in resisting zone (ft)
T_{total} = maximum factored reinforcement tension from Eq. 11.10.7.2-2 (kips/ft)
ϕ = resistance factor for reinforcement pullout from Table 11.5.7-1 (dim.)
F^* = pullout friction factor (dim.)
α = scale effect correction factor (dim.)
σ_v = unfactored vertical stress at the reinforcement level in the resistant zone (ksf)
C = overall reinforcement surface area geometry factor (dim.)
R_c = reinforcement coverage ratio specified in Article 11.10.6.4.1 (dim.)

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

11.10.7.3—Facing Reinforcement Connections

C11.10.7.3

Facing elements shall be designed to resist the seismic loads determined as specified in Article 11.10.7.2, i.e., T_{total}. Facing elements shall be designed in accordance with applicable provisions of Sections 5, 6, and 8 for reinforced concrete, steel, and timber, respectively, except that for the Extreme Event I limit state, all resistance factors should be 1.0, unless otherwise specified for this limit state.

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

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

For the static component of the load:

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

For the dynamic component of the load:

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

where:

S_{rs} = ultimate reinforcement tensile resistance required to resist static load component (kip/ft)
T_{max} = applied load to reinforcement (kip/ft)
RF_D = reduction factor to prevent rupture of reinforcement due to chemical and biological degradation specified in Article 11.10.6.4.4b (dim.)
ϕ = resistance factor from Table 11.5.7-1 (dim.)
CR_{cr} = long-term connection strength reduction factor to account for reduced ultimate strength resulting from connection (dim.)
R_c = reinforcement coverage ratio from Article 11.10.6.4.1 (dim.)
S_{rt} = ultimate reinforcement tensile resistance required to resist dynamic load component (kip/ft)
T_{md} = factored incremental dynamic inertia force (kip/ft)
CR_u = short-term reduction factor to account for reduced ultimate strength resulting from connection as specified in Article C11.10.6.4.4b (dim.)

For mechanical connections that do not rely on a frictional component, the 0.8 multiplier may be removed from Eqs. 11.10.7.3-1 and 11.10.7.3-2.

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

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

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

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

Some judgment may be required to determine whether or not a specific shear resisting device or combination of devices is sufficient to meet this requirement in Seismic Performance Zones 3 and 4. The ability of the shear resisting device or devices to keep the soil reinforcement connected to the facing, should vertical acceleration significantly reduce the normal force between the reinforcement and the facing blocks, should be evaluated. Note that in some cases, coarse angular gravel placed within the hollow core of the facing blocks, provided that the gravel can remain interlocked during shaking, can function as a shear restraining device to meet the requirements of this Article.

11.10.7.4—Wall Details for Improved Seismic Performance

The details specified in Article 11.6.5.6 for gravity walls should also be addressed for MSE walls in seismically active areas, defined as Seismic Zone 2 or higher. The following additional requirements should also be addressed for MSE walls:

- *Second Stage Fascia Panels*: The connections used to connect the fascia panels to the main gravity wall structure should be designed to minimize movement between panels during shaking.
- *Soil Reinforcement Length*: A minimum soil reinforcement length of 0.7*H* should be used. A greater soil reinforcement length in the upper 2 to 4.0 ft of wall height (a minimum of two reinforcement layers) should also be considered to improve the seismic performance of the wall. If the wall is placed immediately in front of a very steep slope, existing shoring, or permanent wall, the reinforcement within the upper 2.0 to 4.0 ft of wall height (a minimum of two reinforcement layers, applicable to wall heights of 10.0 ft or more) should be extended to at least 5.0 ft behind the steep slope or existing wall.
- *Wall Corners and Abrupt Facing Alignment Changes*: Should be designed using specially formed facing units to bridge across the corner and overlap with the adjacent wall facing units to prevent the corner from opening up during shaking. Wall corners should also be designed for the potential for higher loads to develop than would be determined using two-dimensional analysis. Wall corners and short radius turns are defined as having an enclosed angle of 120 degrees or less.

C11.10.7.4

These recommended details are based on previous experiences with walls in earthquakes (e.g., see Yen et al., 2011). Walls that did not address these details tended to have a higher frequency of problems than walls that did consider these details.

With regard to preventing joints from opening up during shaking, corners details, and details for addressing protrusions through the wall face, Article C11.6.5.6 applies. For panel-faced MSE walls placed against a cast-in-place (CIP) concrete curtain wall or similar structure, a 4.0-in. lip on the CIP structure to cover the joint with the MSE wall facing has been used successfully.

Regarding the design of wall corners and abrupt changes in the facing alignment (e.g., corners and short radius turns at an enclosed angle of 120 degrees or less), both static and seismic earth pressure loading may be greater than what would be determined from two-dimensional analysis. Historically, corners and abrupt alignment changes in walls have had a higher incidence of performance problems during earthquakes than relatively straight sections of the wall alignment, as the corners tend to attract dynamic load and increased earth pressures. This should be considered when designing a wall corner for seismic loading. For that portion of the corner or abrupt wall facing alignment change where the soil reinforcement cannot achieve its full length required to meet internal stability requirements, the end of the reinforcement layer should be structurally tied to the back of the adjacent panel. Reinforcement layers should be placed in both directions. In addition, the special corner facing element should also have reinforcement layers attached to it to provide stability for the corner panel. The reinforcement layers that are tied to both sides of the corner should be designed for the higher earth pressures considering the corner as a bin structure.

Note that the corner or abrupt alignment change enclosed angle as defined in the previous paragraph can either be internal or external to the wall.

With regard to wall backfill materials, the provisions of Article 11.6.5.6 shall apply.

When structures and foundations within the active zone of the reinforced wall backfill are present significant wall movements and damage have occurred during earthquakes due to inadequate reinforcement length behind the facing due to the presence of a foundation, drainage structure, or other similar structure. The details provided in Article 11.10.10.4 are especially important to implement for walls subjected to seismic loading.

Past experience with second stage precast incremental facing panels indicates that performance problems can occur if the connections between the panels and the first stage wall can rotate or otherwise have some looseness, especially if wall settlement is not complete. Therefore, incremental second stage facia panels should be avoided for walls located in seismically active areas. Full height second stage precast or cast-in-place concrete panels have performed more consistently, provided the panels are installed after wall settlement is essentially complete.

A minimum soil reinforcement length of $0.7H$ has been shown to consistently provide good performance of MSE walls in earthquakes. Extending the upper two layers of soil reinforcement a few feet behind the $0.7H$ reinforcement length has in general resulted in modest improvement in the wall deformation in response to seismic loading, especially if higher silt content backfill must be used. If MSE walls are placed in front of structures or hard soil or rock steep slopes that could have different deformation characteristics than the MSE wall reinforced backfill, there is a tendency for a crack to develop at the vertical or near-vertical boundary of the two materials. Soil reinforcements that extend an adequate distance behind the boundary have been shown to prevent such a crack from developing. It is especially important to extend the length of the upper reinforcement layers if there is inadequate room to have a reinforcement length of $0.7H$ in the bottom portion of the wall, provided the requirements of Article 11.10.2.1 and commentary are met.

For additional information on good wall details for MSE walls, see Berg et al. (2009).

11.10.8—Drainage

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

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

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

11.10.9—Subsurface Erosion

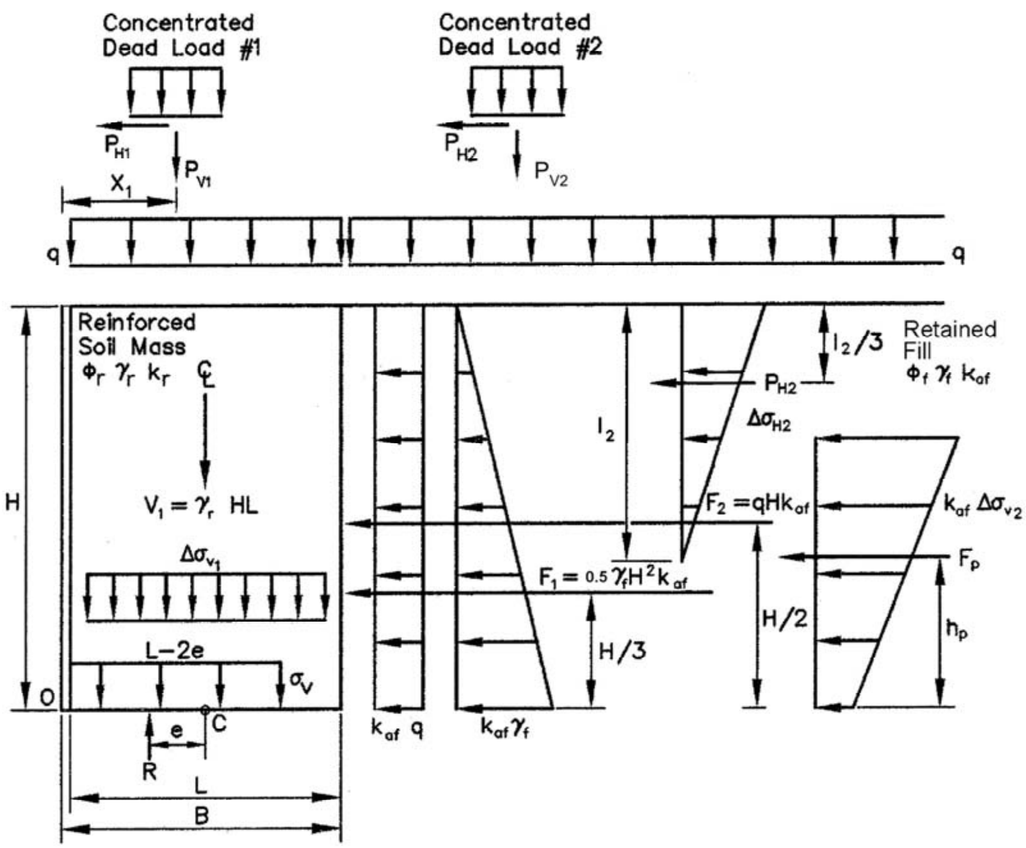
The provisions of Article 11.6.3.5 shall apply.

11.10.10—Special Loading Conditions

11.10.10.1—Concentrated Dead Loads

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

Figure 11.10.10.1-1 illustrates the combination of loads using superposition principles to evaluate external and internal wall stability. Depending on the size and location of the concentrated dead load, the location of the boundary between the active and resistant zones may have to be adjusted as shown in Figure 11.10.10.1-2.



Notes:

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

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

P_{V1}, P_{H1}, $\Delta\sigma_{v1}$, $\Delta\sigma_{v2}$, $\Delta\sigma_{H2}$, and I_2 are as determined from Figures 3.11.6.3-1 and 3.11.6.3-2, and F_p results from P_{V2} (i.e., $K\Delta\sigma_{v2}$ from Figure 3.11.6.3-1. H is the total wall height at the face. h_p is the distance between the centroid of the trapezoidal distribution shown and the bottom of that distribution.

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

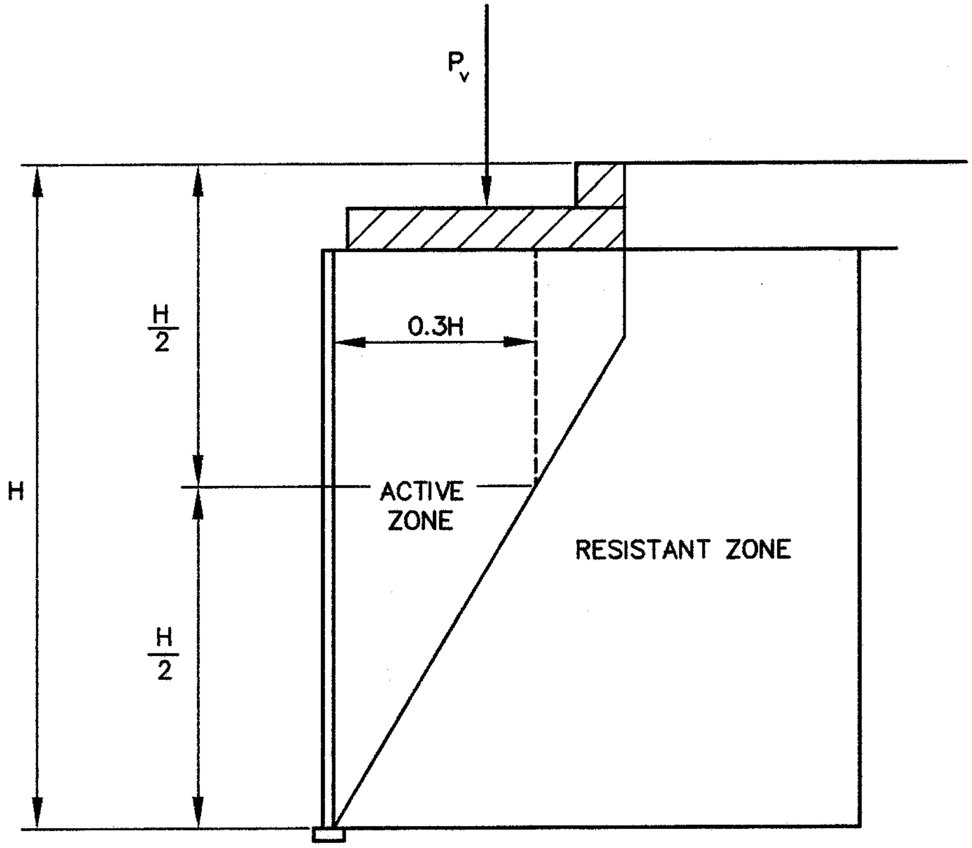


Figure 11.10.10.1-2—Location of Maximum Tensile Force Line in Case of Large Surcharge Slabs (Inextensible Reinforcements)

11.10.10.2—Traffic Loads and Barriers

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

For checking pullout safety of the reinforcements, the lateral traffic impact load shall be distributed to the upper

C11.10.10.2

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

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

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

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

For the transient components:

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

where:

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

S_v = vertical spacing of reinforcement (ft)

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

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

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

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

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

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

Flexible post and beam barriers, when used, shall be placed at a minimum distance of 3.0 ft from the wall face, driven 5.0 ft below grade, and spaced to miss the reinforcements where possible. If the reinforcements cannot be missed, the wall shall be designed accounting for the presence of an obstruction as described in Article 11.10.10.4. The upper two rows of reinforcement shall be designed for an additional horizontal load γP_{H1}, where P_{H1} = 300 lbs. per linear ft of wall, 50 percent of which is distributed to each layer of reinforcement.

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

11.10.10.3—Hydrostatic Pressures

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

C11.10.10.3

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

11.10.10.4—Obstructions in the Reinforced Soil Zone

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

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

2) Place a structural frame around the obstruction capable of carrying the load from the reinforcements in front of the obstruction to reinforcements connected to the structural frame behind the obstruction as illustrated in Figure 11.10.10.4-1.

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

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

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

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

C11.10.10.4

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

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

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

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

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

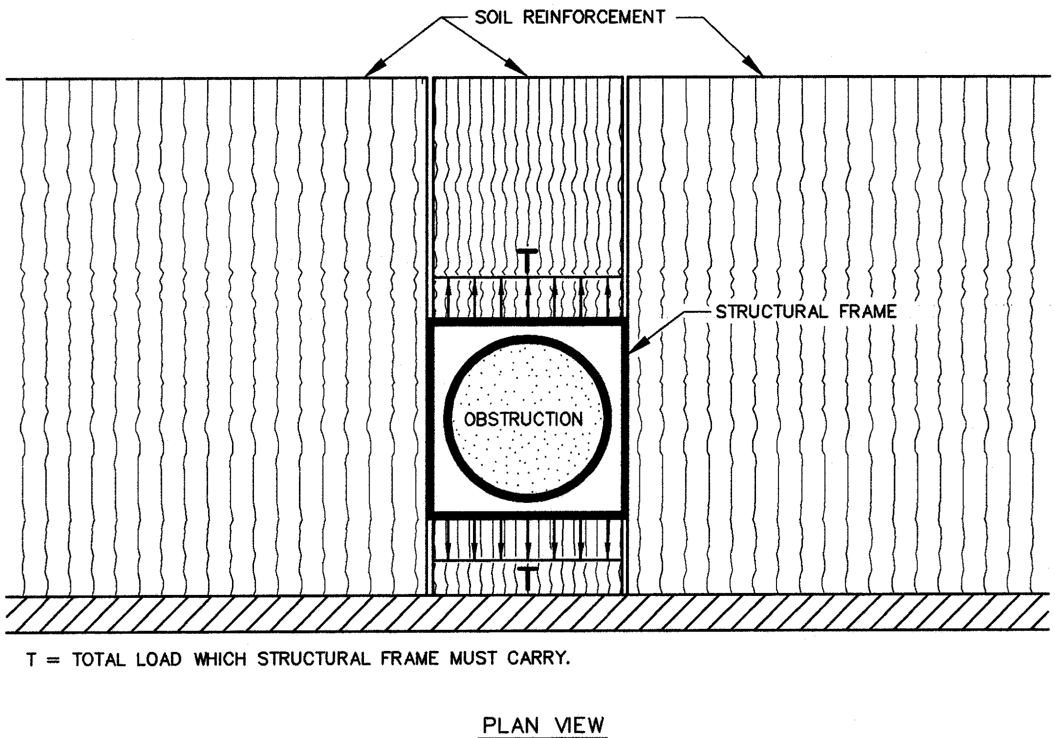


Figure 11.10.10.4-1—Structural Connection of Soil Reinforcement around Backfill Obstructions

11.10.11—MSE Abutments

C11.10.11

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

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

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

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

where:

σ_{Hmax} = factored horizontal stress at layer i, as defined by Eq.11.10.11-2 (ksf)

S_v = vertical spacing of reinforcement (ft)

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

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

where:

γ_p = load factor for vertical earth pressure in Table 3.4.1-2
$\Delta\sigma_H$ = magnitude of lateral pressure due to surcharge (ksf)
σ_v = vertical soil stress over effective base width ($B - 2e$) (ksf)
$\Delta\sigma_v$ = vertical soil stress due to footing load (ksf)
k_r = earth pressure coefficient varying as a function of k_a as specified in Article 11.10.6.2.1
k_a = active earth pressure coefficient specified in Article 3.11.5.8

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

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

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

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

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

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

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

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

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

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

11.11—PREFABRICATED MODULAR WALLS

11.11.1—General

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

C11.11.1

Prefabricated modular wall systems, whose elements may be proprietary, generally employ interlocking soil-filled reinforced concrete or steel modules or bins, rock filled gabion baskets, precast concrete units, or dry cast segmental masonry concrete units (without soil reinforcement) which resist earth pressures by acting as gravity retaining walls. Prefabricated modular walls may also use their structural elements to mobilize the dead weight of a portion of the wall backfill through soil arching to provide resistance to lateral loads. Typical prefabricated modular walls are shown in Figure C11.11.1-1.

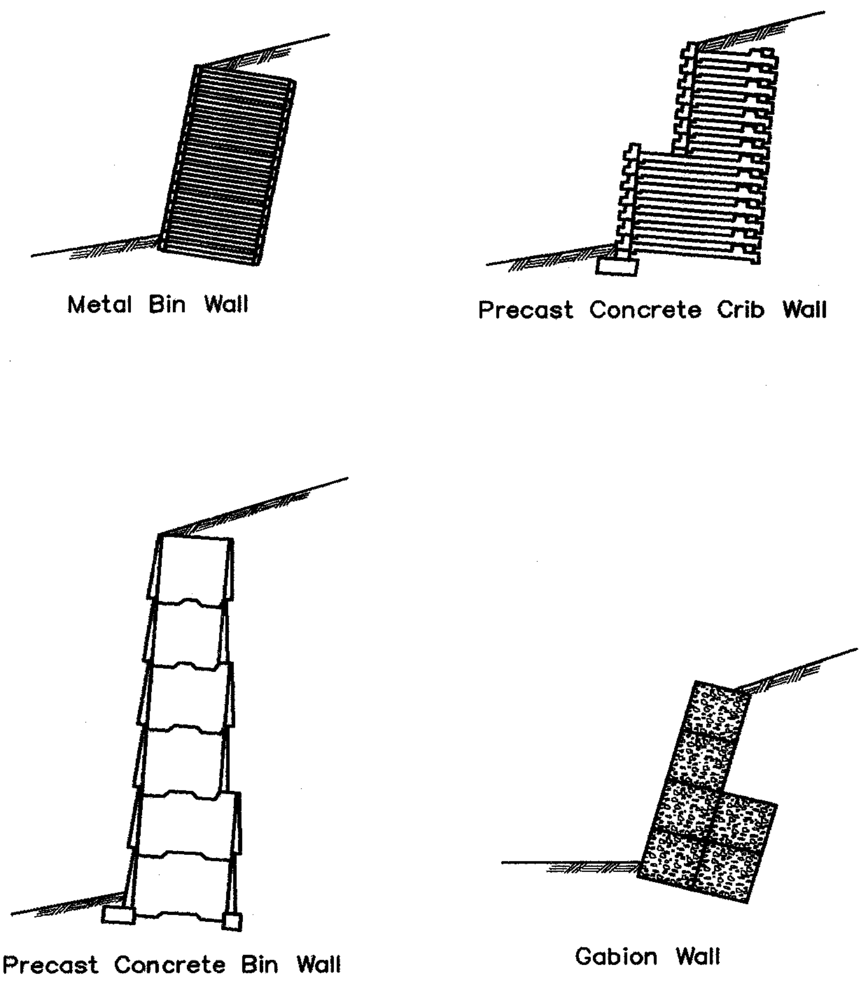


Figure C11.11.1-1—Typical Prefabricated Modular Gravity Walls

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

- On curves with a radius of less than 800 ft, unless the curve can be substituted by a series of chords.
- Steel modular systems shall not be used where the groundwater or surface runoff is acid contaminated or where deicing spray is anticipated.

11.11.2—Loading

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

11.11.3—Movement at the Service Limit State

The provisions of Article 11.6.2 shall apply as applicable.

C11.11.3

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

11.11.4—Safety against Soil Failure

11.11.4.1—General

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

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

11.11.4.2—Sliding

The provisions of Article 10.6.3.4 shall apply.

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

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

11.11.4.3—Bearing Resistance

The provisions of Article 10.6.3 shall apply.

Bearing resistance shall be computed by assuming

C11.11.4.3

Concrete modular systems are relatively rigid and are subject to structural damage due to differential that dead loads and earth pressure loads are resisted by point supports per unit length at the rear and front of the modules or at the location of the bottom legs. A minimum of 80 percent of the soil weight inside the modules shall be considered to be transferred to the front and rear support points. If foundation conditions require a footing under the total area of the module, all of the soil weight inside the modules shall be considered.

settlements, especially in the longitudinal direction. Therefore, bearing resistance for footing design should be determined as specified in Article 10.6.

11.11.4.4—Overturning

The provisions of Article 11.6.3.3 shall apply.

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

C11.11.4.4

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

11.11.4.5 —Subsurface Erosion

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

11.11.4.6—Overall Stability

The provisions of Article 11.6.2.3 shall apply.

11.11.4.7—Passive Resistance and Sliding

The provisions of Articles 10.6.3.4 and 11.6.3.6 shall apply, as applicable.

11.11.5—Safety against Structural Failure

11.11.5.1—Module Members

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

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

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

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

where:

C11.11.5.1

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

P_b = factored pressure inside bin module (ksf)
γ_s = soil unit weight (kcf)
γ = load factor for vertical earth pressure specified in Table 3.4.1-2
b = width of bin module (ft)

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

11.11.6—Seismic Design for Prefabricated Modular Walls

The provisions of Article 11.6.5 shall apply.

C11.11.6

The prefabricated modular wall develops resistance to seismic loads from both the geometry and the weight of the wall section. The primary design issues for seismic loading are global stability, external stability (i.e., sliding, overturning, and bearing), and internal stability. External stability includes the ability of each lift within the wall to also meet external stability requirements. Interlocking between individual structural sections and the soil fill within the wall needs to be considered in this evaluation.

The primary difference for this wall type relative to a gravity or semigravity wall is that sliding and overturning can occur at various heights between the base and top of the wall, as this class of walls typically uses gravity to join sections of the wall together.

The interior of the prefabricated wall elements is normally filled with soil; this provides both additional weight and shear between structural elements. The contributions of the earth, as well as the batter on the wall, need to be considered in the analysis.

Similar to the other external stability checks, the overall (global) stability check needs to consider failure surfaces that pass through the wall section, as well as below the base of the wall. The check on stability at midlevel must consider the contributions of both the soil within the wall and any structural interlocking that occurs for the particular modular wall type.

When checking stability at the mid level of a wall, the additional shear resistance from interlocking of individual wall components will depend on the specific wall type. Usually, interlocking resistance between wall components is provided by the wall supplier.

11.11.7—Abutments

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

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

11.11.8—Drainage

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

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

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APPENDIX A11—SEISMIC DESIGN OF RETAINING STRUCTURES

A11.1—GENERAL

This Appendix provides information that supplements the provisions contained in Section 11 regarding the design walls and free standing abutments for seismic loads. Detailed design methodology is provided for the calculation of seismic earth pressures, both active and passive. Design methodology is also provided for the estimation of deformation effects on the seismic acceleration a wall will experience.

A11.2—PERFORMANCE OF WALLS IN PAST EARTHQUAKES

Even as early as 1970, Seed and Whitman (1970) concluded that "many walls adequately designed for static earth pressures will automatically have the capacity to withstand earthquake ground motions of substantial magnitudes and, in many cases, special seismic provisions may not be needed." Seed and Whitman further indicated that this statement applies to gravity and semigravity walls with peak ground accelerations up to 0.25g. More recently, Bray et al. (2010) and Lew et al. (2010a, 2010b) indicate that lateral earth pressure increases due to seismic ground motion are likely insignificant for peak ground accelerations of 0.3g to 0.4g or less, indicating that walls designed to resist static loads (i.e., the strength and service limit states) will likely have adequate stability for the seismic loading case, especially considering that load and resistance factors used for Extreme Event I limit state design are at or near 1.0.

Following the 1971 San Fernando earthquake, Clough and Fragaszy (1977) assessed damage to floodway structures, consisting of reinforced concrete cantilever (vertical) walls structurally tied to a floor slab forming a continuous U-shaped structure. They found that no damage was observed where peak ground accelerations along the structures were less than 0.5g. However, damage and wall collapse was observed where accelerations were higher than 0.5*g* or localized damage where the structures crossed the earthquake fault and the damage was quite localized. They noted that while higher strength steel rebar was used in the actual structure than required by the static design, the structure was not explicitly designed to resist seismic loads. Gazetas et al. (2004) observed that cantilever semigravity walls with little or no soil surcharge exposed to shaking in the 1999 Athens earthquake performed well for peak ground accelerations up to just under 0.5*g* even though the walls were not specifically designed to handle seismic loads. Lew et al. (1995) made similar observations with regard to tied back shoring walls in the 1994 Northridge Earthquake and Tatsuoka (1996) similarly observed good wall performance for MSE type gravity walls in the 1995 Kobe earthquake. See Bray et al. (2010), Lew et al. (2010a, 2010b), and Al Atik and Sitar (2010) for additional background on observed wall performance and the generation of seismic earth pressures.

Walls meeting the requirements in Article 11.5.4.2 that allow a seismic analysis to not be conducted have demonstrated consistently good performance in past earthquakes. For wall performance in specific earthquakes, see the following:

- Gravity and semigravity cantilever walls in the 1971 San Fernando Earthquake (Clough and Fragaszy, 1977).
- Gravity and semigravity cantilever walls in the 1999 Athens Earthquake (Gazetas et al., 2004).
- Soil nail walls and MSE walls in the 1989 Loma Prieta, California earthquake (Vucetic et al., 1998 and and Collin et al.,1992, respectively).
- MSE walls in the 1994 Northridge, California earthquake (Bathurst and Cai, 1995).
- MSE walls and reinforced concrete gravity walls in the 1995 Kobe, Japan earthquake (Tatsuoka et al., 1996).
- MSE walls and concrete gravity and semigravity walls in the 2010 Maule, Chile earthquake (Yen et al., 2011).
- Summary of the performance of various types of walls (Koseki et al., 2006).
- Reinforced earth walls withstand Northridge Earthquake (Frankenberger et al., 1996).
- The Performance of Reinforced Earth Structures in the Vicinity of Kobe during the Great Hanshin Earthquake (Kobayashi et al, 1996).
- Evaluation of Seismic Performance in Mechanically Stabilized Earth Structures (Sankey et al., 2001).

However, there have been some notable wall failures in past earthquakes. For example, Seed and Whitman (1970) indicated that some concrete gravity walls and quay walls (both gravity structures and anchored sheet pile nongravity cantilever walls), in the great Chilean Earthquake of 1960 and in the Niigata, Japan Earthquake of 1964, suffered severe displacements or even complete collapse. In most of those cases, significant liquefaction behind or beneath the wall was the likely cause of the failure. Hence, Article 11.5.4.2 specifies that a seismic analysis should be performed if liquefaction or severe strength loss in sensitive clays can cause instability of the wall. Seed and Whitman (1970) indicate, however, that collapse of walls located above the water table has been an infrequent occurrence.

Tatsuoka et al. (1996) indicated that several of the very old (1920s to 1960s) unreinforced masonry gravity walls and concrete gravity structures exposed to strong shaking in the 1995 Kobe Japan earthquake did collapse. In those cases, collapse was likely due to the presence of weak foundation soils that had inadequate bearing and sliding resistance and, in a few cases, due to the presence of a very steep sloping surcharge (e.g., $1.5H$:$1V$) combined with poor soil conditions. Soil liquefaction may have been a contributing factor in some of those cases. These wall collapses were mostly located in the most severely shaken areas (e.g., as high as $0.6g$ to $0.8g$). As noted previously, Clough and Fragaszy (1977) observed concrete cantilever walls supporting open channel floodways that had collapsed where peak ground accelerations were $0.5g$ or more in the 1971 San Fernando earthquake. However, in that case, soil conditions were good. All of these wall cases where collapse or severe damage/deformations occurred are well outside of the conditions and situations where Article 11.5.4.2 allows the seismic design of walls to be waived.

Setting the limit at $0.4g$ for the Article 11.5.4.2 no seismic analysis provision represents a reasonable compromise between observations from laboratory modeling and full-scale wall situations (i.e., lab modeling indicates that seismic earth pressures are very low, below $0.4g$, and walls in actual earthquakes start to have serious problems, including collapses even in relatively good soils, when the acceleration is greater than $0.5g$ and the wall has not been designed for the full seismic loading). However, if soil strength loss and flow due liquefaction or strength loss in sensitive silts and clays occurs, wall collapse can occur at lower acceleration values. Note that for the lab model studies, the $0.4g$ limit represents the limit at which significant seismic earth pressure does not appear to develop. However, for walls with a significant structural mass, the inertial force on the wall mass itself can still occur at accelerations less than $0.4g$. At $0.4g$, the combination of seismic earth pressure and wall inertial force is likely small enough still to not control the forces in the wall and its stability, provided the wall mass is not large. For typical gravity walls, the wall mass would not be large enough to offset the lack of seismically increased earth pressure below 0.4g. A possible exception regarding wall mass inertial forces is reinforced soil walls, though that inertial mass consists of soil within the reinforced soil zone. However, due to their flexibility, reinforced soil walls perform better than reinforced concrete walls, so the inertial mass issue may not be as important for that type of wall. Note that experience with walls in actual earthquakes in which the walls have not been designed for seismic loads is limited. So while all indications are that major wall problems do not happen until the acceleration is greater than A_s of $0.5g$, the majority of those walls where such observations could be made have been strengthened to resist some degree of seismic loading. If walls are not designed for seismic loads, it is reasonable to back off a bit from the observed 0.5g threshold. Hence, $0.4g$ represents a reasonable buffer relative to potential severe wall damage or collapse as observed for walls in earthquakes at $0.5g$ or more.

Based on previous experience, walls that form tunnel portals have tended to exhibit more damage due to earthquakes than free-standing walls. It is likely that the presence of the tunnel restricts the ability of the portal wall to move, increasing the seismic forces to which the wall is subjected. Hence, a seismic design is recommended in such cases.

A11.3—CALCULATION OF SEISMIC ACTIVE PRESSURE

Seismic active earth pressures have historically been estimated using the Mononabe-Okabe Method. However, this method is not applicable in some situations. More recently, Anderson et al. (2008) have suggested a generalized limit equilibrium method (GLE) that is more broadly applicable. Both methods are provided herein. Specifications which should be used to select which method to use are provided in Article 11.6.5.3.

A11.3.1—Mononobe-Okabe Method

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

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

2. The backfill is cohesionless, with a friction angle of ϕ.

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

The M-O Method is illustrated in Figure A11.3.1-1 and the equation used to calculate K_{AE} follows the figure.

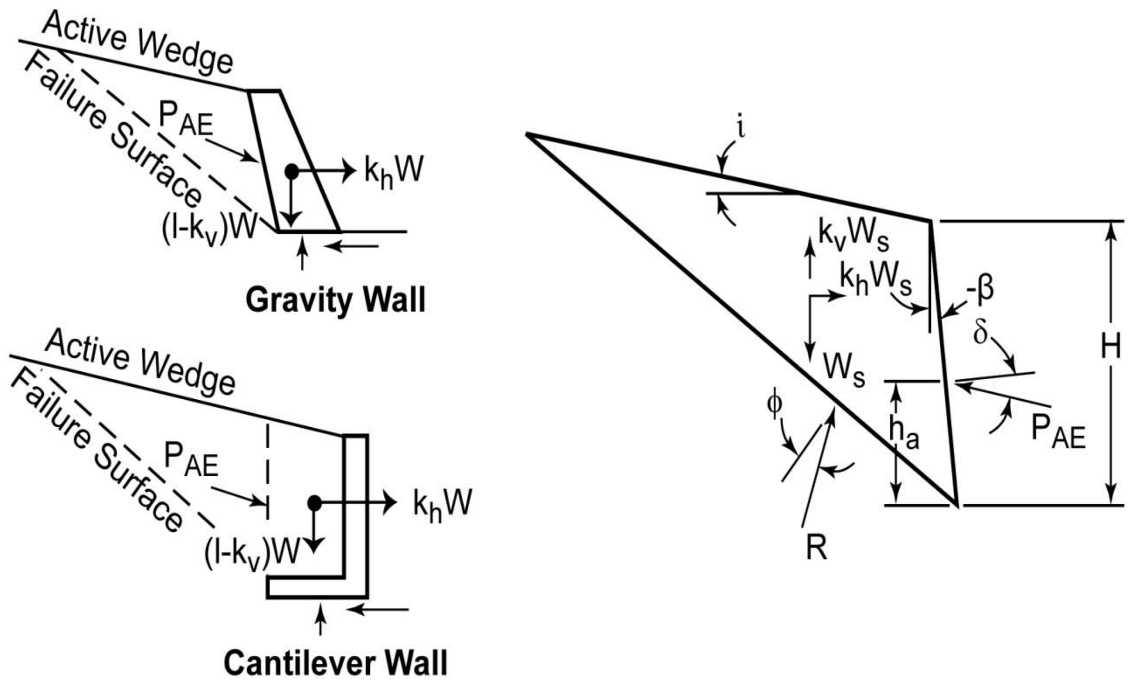


Figure A11.3.1-1—Mononobe-Okabe Method Force Diagrams

$$K_{AE} = \frac{\cos^2(\phi - \theta_{MO} - B)}{\cos\theta_{MO}\cos^2\beta\cos(\delta+\beta+\theta_{MO})} \times \left[1 - \sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta_{MO}-i)}{\cos(\delta+\beta+\theta_{MO})\cos(i-\beta)}}\right]^{-2} \quad \text{(A11.3.1-1)}$$

where:

K_{AE} = seismic active earth pressure coefficient (dim)
γ = unit weight of soil (kcf)
H = height of wall (ft)
h = height of wall at back of wall heel considering height of sloping surcharge, if present (ft)
ϕ_f = friction angle of soil (degrees)
θ_{MO} = arc tan $[k_h/(1 - k_v)]$ (degrees)
δ = wall backfill interface friction angle (degrees)
k_h = horizontal seismic acceleration coefficient (dim.)
k_v = vertical seismic acceleration coefficient (dim.)
i = backfill slope angle (degrees)
β = slope of wall to the vertical, negative as shown (degrees)

In discussion of the M-O method to follow, H and h should be considered interchangeable, depending on the type of wall under consideration (see Figure A11.3.1-1).

Mononobe and Matsuo (1932) originally suggested that the resultant of the active earth pressure during seismic loading remain the same as for when only static forces are present (i.e., H/3 or h/3). However, theoretical considerations by Wood (1973), who found that the resultant of the dynamic pressure acted approximately at midheight and empirical considerations from model studies summarized by Seed and Whitman (1970) who suggested that h_a could be obtained by assuming that the static component of the soil force acts at H/3 from the bottom of the wall and the additional dynamic effect acts at a height of 0.6H, resulted in increasing the height of the resultant location above the wall base. Therefore, in past practice, designers have typically assumed that $h_a = H/2$ with a uniformly distributed pressure. Note that if the wall has a protruding heel or if the wall is an MSE wall then replace H with h in the preceding discussion.

Back analysis of full-scale walls in earthquakes, however, indicates earth pressure resultants located higher than $h/3$ will overestimate the force, resulting in a prediction of wall failure when in reality the wall performed well (Clough and Fragaszy, 1977). Recent research indicates the location of the resultant of the total earth pressure (static plus seismic) should be located one-third up from the wall based on centrifuge model tests on gravity walls (Al Atik and Sitar, 2010; Bray et al., 2010; and Lew et al., 2010). However, recent work by others (Nakamura, 2006) also indicates that the resultant location could be slightly higher, depending on the specifics of the ground motion and the wall details.

A reasonable approach is to assume that for routine walls, the combined static/seismic resultant should be located at the same location as static earth pressure resultant but no less than $h/3$. Because there is limited evidence that in some cases the combined static/seismic resultant location could be slightly higher than the static earth pressure resultant, a slightly higher resultant location (e.g., $0.4h$ to $0.5h$) for seismic design of walls for which the impact of wall failure is relatively high should be considered. However, for routine wall designs, a combined static/seismic resultant location equal to that used for static design (e.g., $h/3$) is sufficient.

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

A11.3.2—Modification of Mononabe-Okabe Method to Consider Cohesion

The M-O equation for seismic active earth pressure determination has many limitations, as discussed in Anderson et al. (2008). These limitations include the inability to account for cohesion that occurs in the soil. This limitation has been addressed by rederiving the seismic active earth pressure using a Coulomb-type wedge analysis. Generally, soils with more than 15 percent fines content can be assumed to be undrained during seismic loading. For this loading condition, total stress soil parameters, γ and c, should be used.

Eq. A11.3.2-1 that is provided by Anderson et al. (2008), and Figure A11.3.2-1 shows the terms in the equation. This equation is very simple and practical for the design of the retaining walls and the equation has been calibrated with slope stability computer programs.

$$P_{AE} = \frac{W\left[(1-k_v)\tan(\alpha-\phi)+k_h\right]-CL\left[\sin\alpha\tan(\alpha-\phi)+\cos\alpha\right]-C_A H\left[\tan(\alpha-\phi)\cos\omega+\sin\omega\right]}{\left[1+\tan(\delta+\omega)\tan(\alpha-\phi)\right]*\cos(\delta+\omega)} \quad \text{A11.3.2-1}$$

The only variables in Eq. A11.3.2-1 are the failure plane angle α and the trial wedge surface length L. Values of friction angle (ϕ), seismic horizontal coefficient (k_h), seismic vertical coefficient (k_v), soil cohesion (C), soil wall adhesion (C_a), soil wall friction (δ), and soil wall angle (ω) are defined by the designer on the basis of the site conditions and the U.S. Geological Survey seismic hazard maps shown in Section 3.

The recommended approach in this Section is to assume that $k_v = 0$, and k_h = the PGA adjusted for site effects (i.e., A_s, $k_{h0,}$ or k_h, or some combination thereof, if the wall is greater than 20.0 ft in height and horizontal wall displacement can occur and is acceptable). A 50 percent reduction in the resulting seismic coefficient is used when defining k_h if 1.0 to 2.0 in. of permanent ground deformation is permitted during the design seismic event. Otherwise, the peak ground acceleration coefficient should be used. Eq. A11.3.2-1 can be easily calculated in a spreadsheet. Using a simple spreadsheet, the user can search for the angle α and calculate maximum value of P_{AE}.

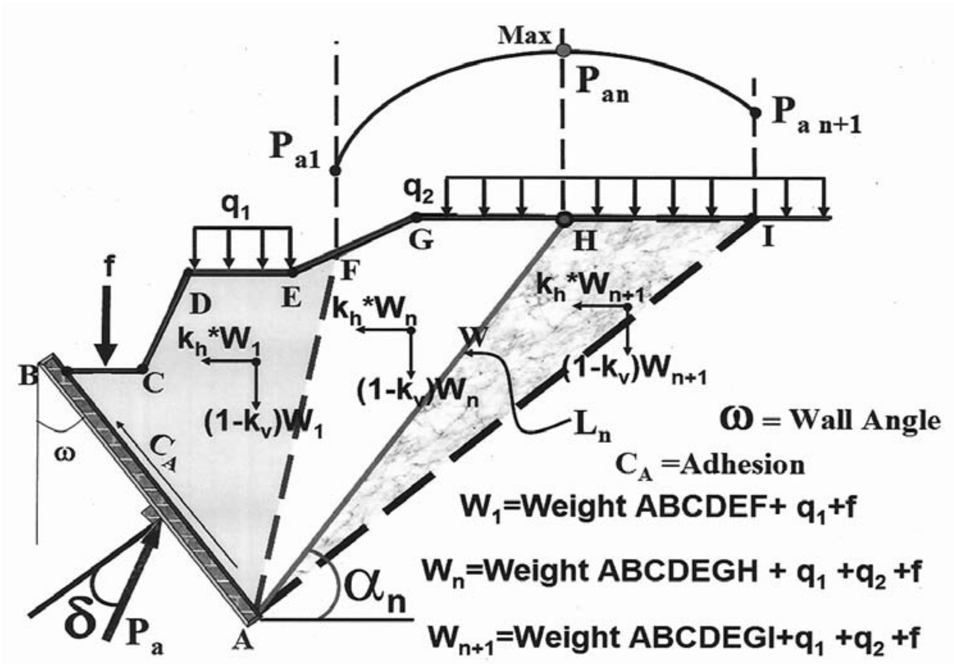


Figure A11.3.2-1—Active Seismic Wedge

The following charts were developed using Eq. A11.3.2-1. These charts are based on level ground behind the wall and a wall friction (δ) of 0.67ϕ. Generally, for active pressure determination, the wall interface friction has a minor effect on the seismic pressure coefficient. However, Eq. A11.3.2-1, the generalized limit equilibrium method, or the charts can be rederived for the specific interface wall friction if this effect is of concern or interest.

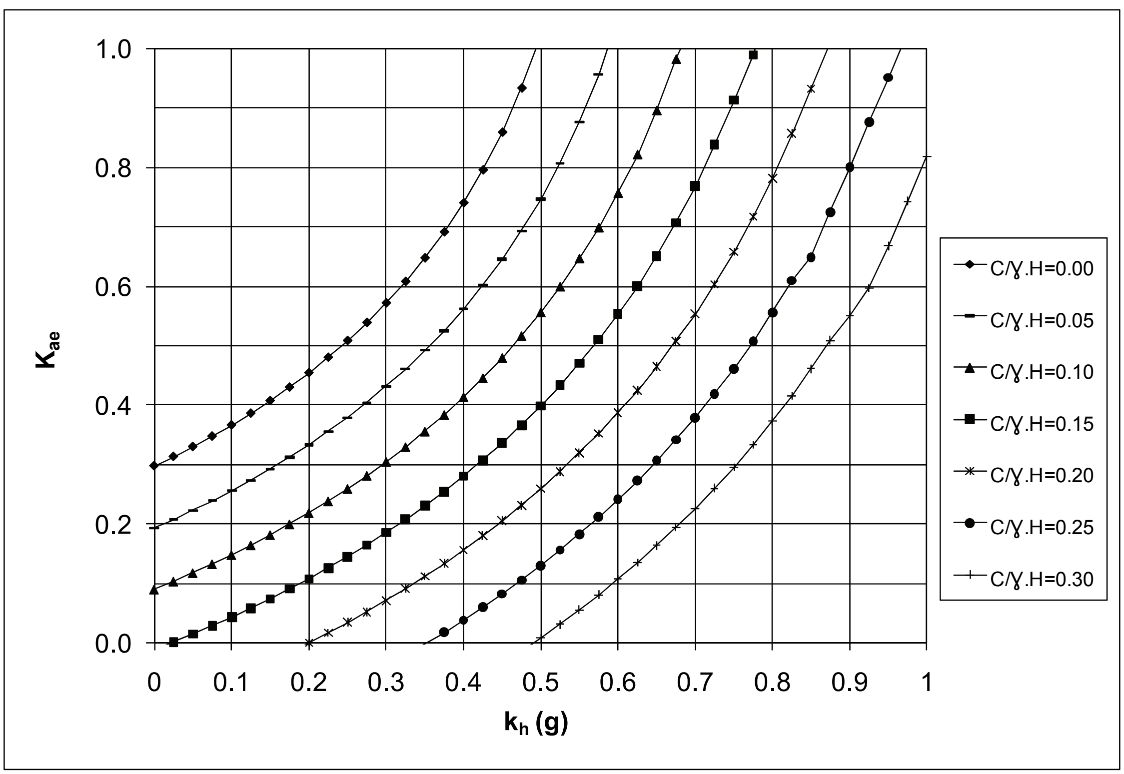


Figure A11.3.2-2—Seismic Active Earth Pressure Coefficient for ϕ = 30 degrees (c = soil cohesion, γ = soil unit weight, and H = retaining wall height)

Note: $k_h = A_s = k_{h0}$ for wall heights greater than 20 ft. This could be H or h as defined in Figure A11.3.1-1.

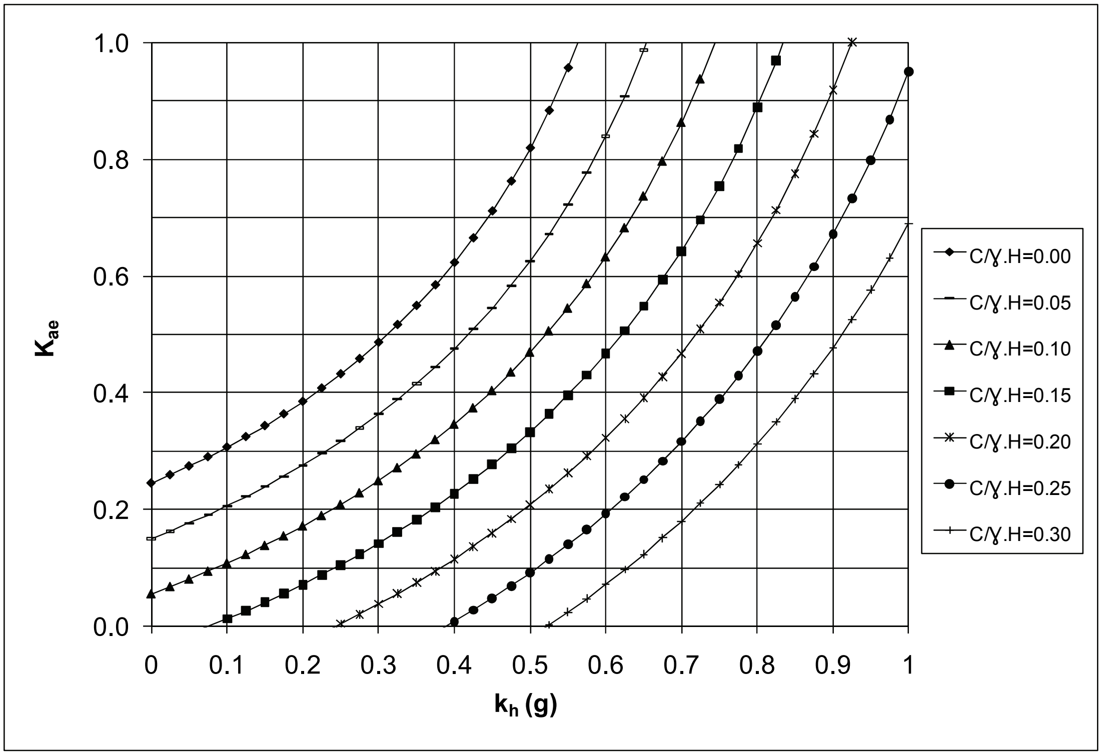


Figure A11.3.2-3—Seismic Active Earth Pressure Coefficient for ϕ = 35 degrees (*c* = soil cohesion, γ = soil unit weight, and *H* = retaining wall height)

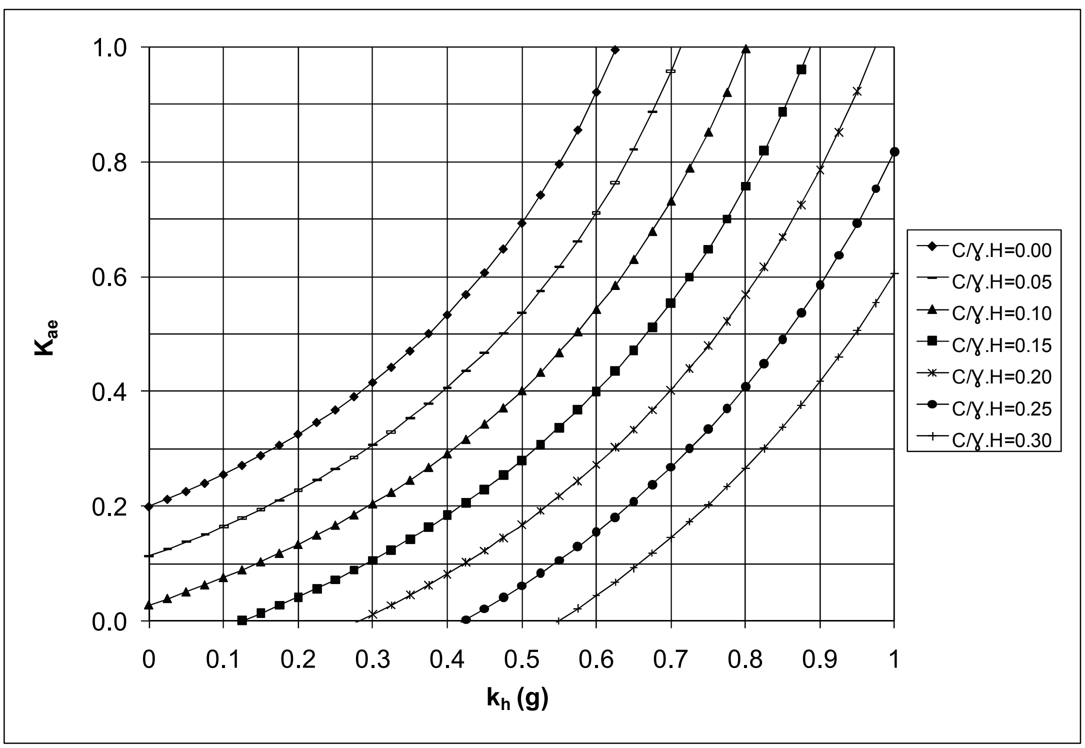


Figure A11.3.2-4—Seismic Active Earth Pressure Coefficient for ϕ = 40 degrees (*c* = soil cohesion, γ = soil unit weight, and *H* = retaining wall height)

A11.3.3—Generalized Limit Equilibrium (GLE) Method

In some situations, the M-O equation is not suitable due to the geometry of the backfill, the angle of the failure surface relative to the cut slope behind the wall, the magnitude of ground shaking, or some combination of these factors (see Article C11.6.5.3). In such situations, a generalized limit equilibrium method involving the use of a computer program for slope stability is likely to be more suitable for determining the earth pressures required for retaining wall design.

Steps in the generalized limit equilibrium (GLE) analysis are as follows:

- Set up the model geometry, groundwater profile, and design soil properties. The internal vertical face at the wall heel or the plane where the earth pressure needs to be calculated should be modeled as a free boundary.
- Choose an appropriate slope stability analysis method. Spencer's method generally yields good results because it satisfies the equilibrium of forces and moments.
- Choose an appropriate sliding surface search scheme. Circular, linear, multi-linear, or random surfaces can be examined in many commercial slope stability analysis programs.
- Apply the earth pressure as a boundary force on the face of the retained soil. For seismic cases, the location of the force may be initially assumed at 1/3*H*) of the retained soil. However, different application points between 1/3*H* and 0.6*H* from the base may be examined to determine the maximum seismic earth pressure force. The angle of applied force depends on assumed friction angle between the wall and the fill soil (typically $2/3\phi_f$ for rigid gravity walls) or the fill friction angle (semigravity walls). If static (i.e., nonseismic) forces are also needed, the location of the static force is assumed at one-third from base (1/3*H*, where *H* is retained soil height).
- Search for the load location and failure surface giving the maximum load for limiting equilibrium (capacity-to-demand ratio of 1.0, i.e., $FS = 1.0$).
- Verify design assumptions and material properties by examining the loads on individual slices in the output as needed.

Additional discussion and guidance regarding this approach is provided in NCHRP Report 611 (Anderson et al., 2008).

A11.4—SEISMIC PASSIVE PRESSURE

This Section provides charts for determination of seismic passive earth pressures coefficients for a soil with both cohesion and friction based on the log spiral method. These charts were developed using a pseudostatic equilibrium method reported in Anderson et al. (2008). The method includes inertial forces within the soil mass, as well as variable soil surface geometries and loads.

Equations used in this approach are given below. Figure A11.4-1 defines the terms used in the equation.

$$dE_i = \frac{W_i(1-K_v)\left[\tan(\alpha_i+\phi)-K_h\right]+C\,L_i\left[\sin\alpha_i\tan(\alpha_i+\phi)+\cos\alpha_i\right]}{\left[1-\tan\delta_i\tan(\alpha_i-\phi)\right]*\cos\delta_i} \qquad \text{(A11.4-1)}$$

$$P_P n = \frac{\sum_{1}^{i} dE}{\left[1-\tan\delta_w\tan(\alpha_w-\phi)\right]*\cos\delta_w} \qquad \text{(A11.4-2)}$$

$$K_P n = \frac{2P_P}{\gamma h^2} \qquad \text{(A11.4-3)}$$

where ϕ is the soil friction angle, c is the cohesion, and δ is wall interface friction.

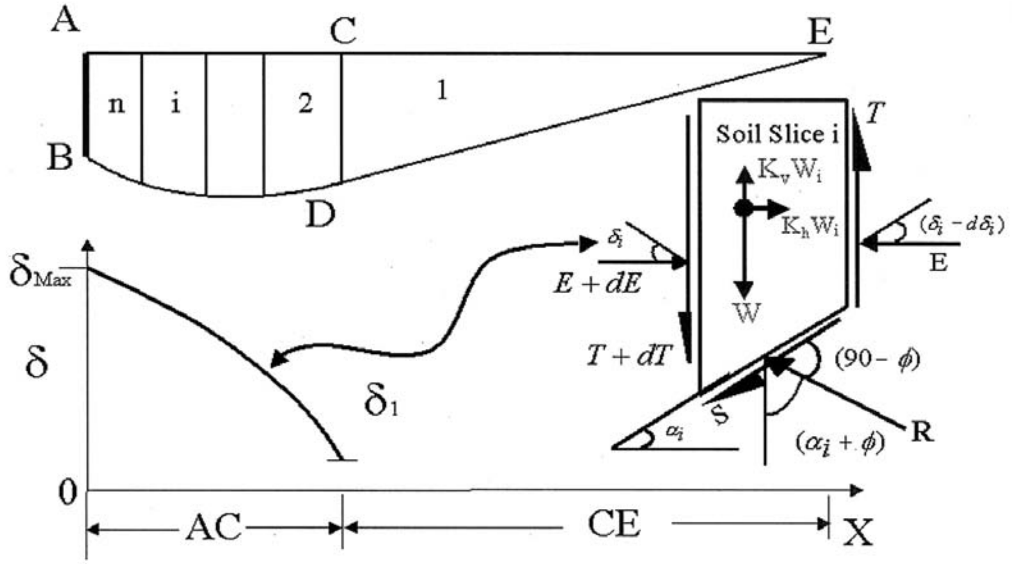
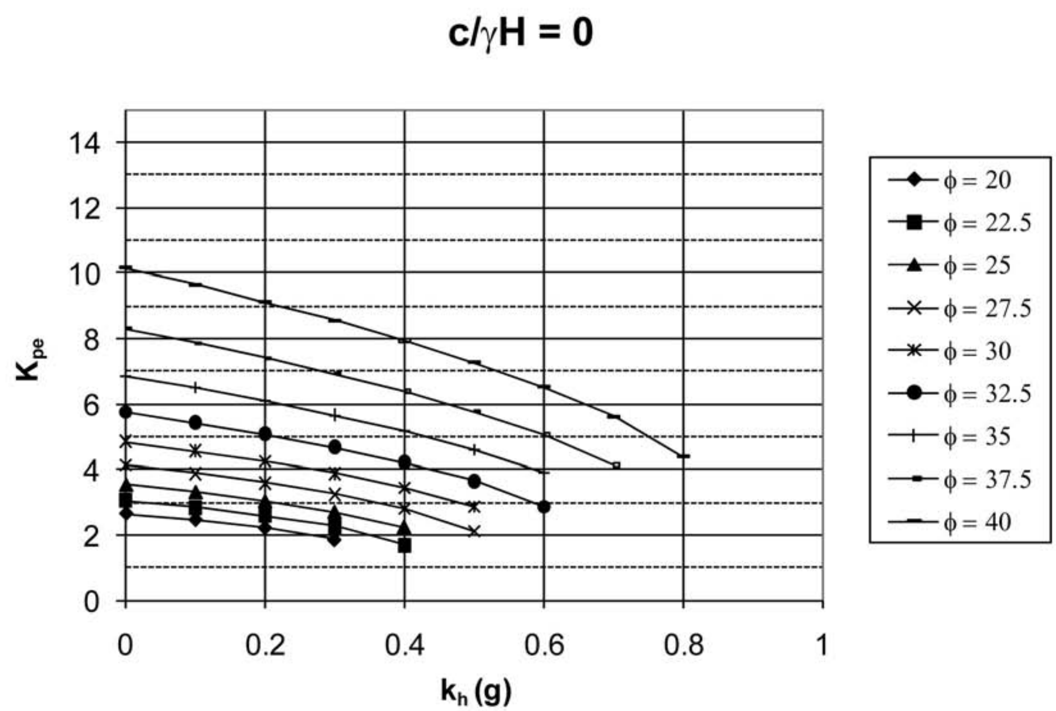


Figure A11.4-1—Limits and Shape Seismic Interslice Force Function (reported in Anderson et al., 2008)

As shown, the method of analysis divides the sliding mass of the backfill into many slices. It is assumed that the shear forces dissipate from a maximum at the wall face (*AB*) to the induced seismic shear forces at the face (*CD*) of the first slice as seen in Figure A11.4-1.

The methodology described above was used to develop a series of charts (Figures A11.4-2 through A11.4-4) for a level backfill condition. These charts can be used to estimate the seismic passive pressure coefficient. The interface friction for these charts is 0.67ϕ. These procedures and charts can be used to estimate the seismic passive coefficient for other interface conditions and soil geometries.



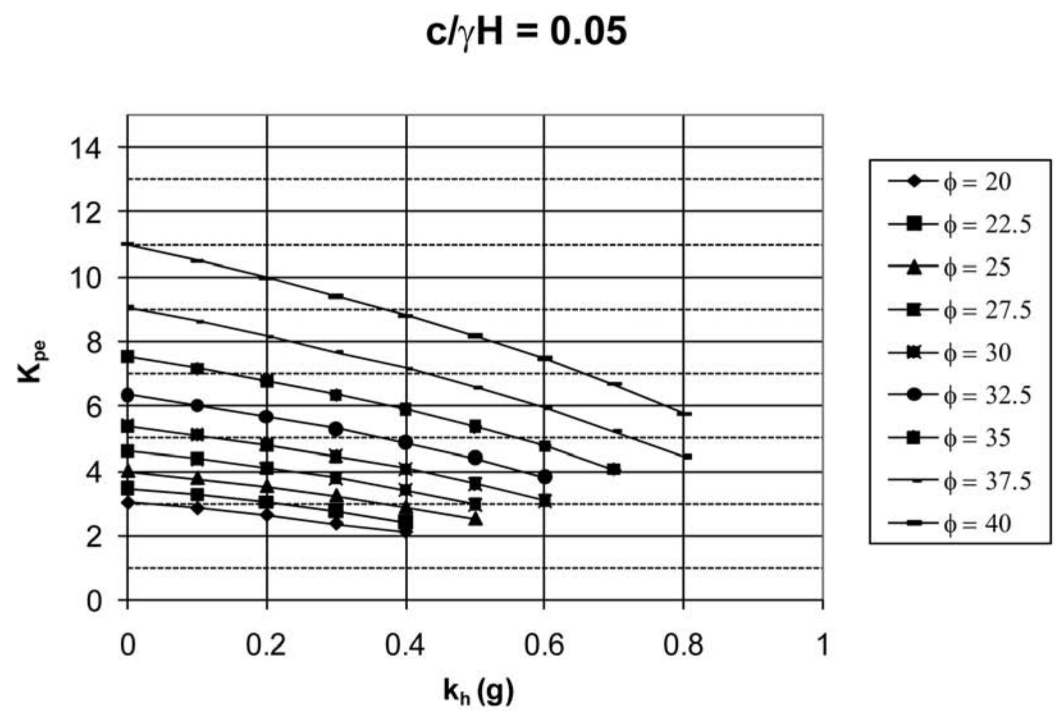
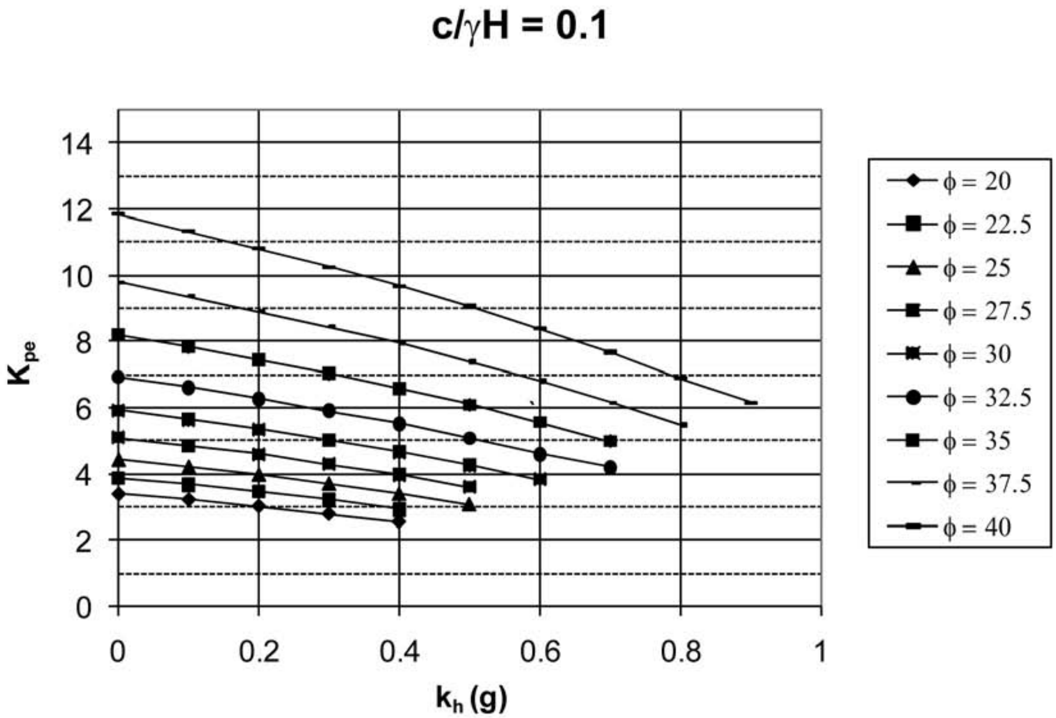


Figure A11.4-2—Seismic Passive Earth Pressure Coefficient Based on Log Spiral Procedure for $c/\gamma H$ = 0 and 0.05 (c = soil cohesion, γ = soil unit weight, and H = height or depth of wall over which the passive resistance acts)

Note: $k_h = A_s = k_{ho}$ for wall heights greater than 20 ft.



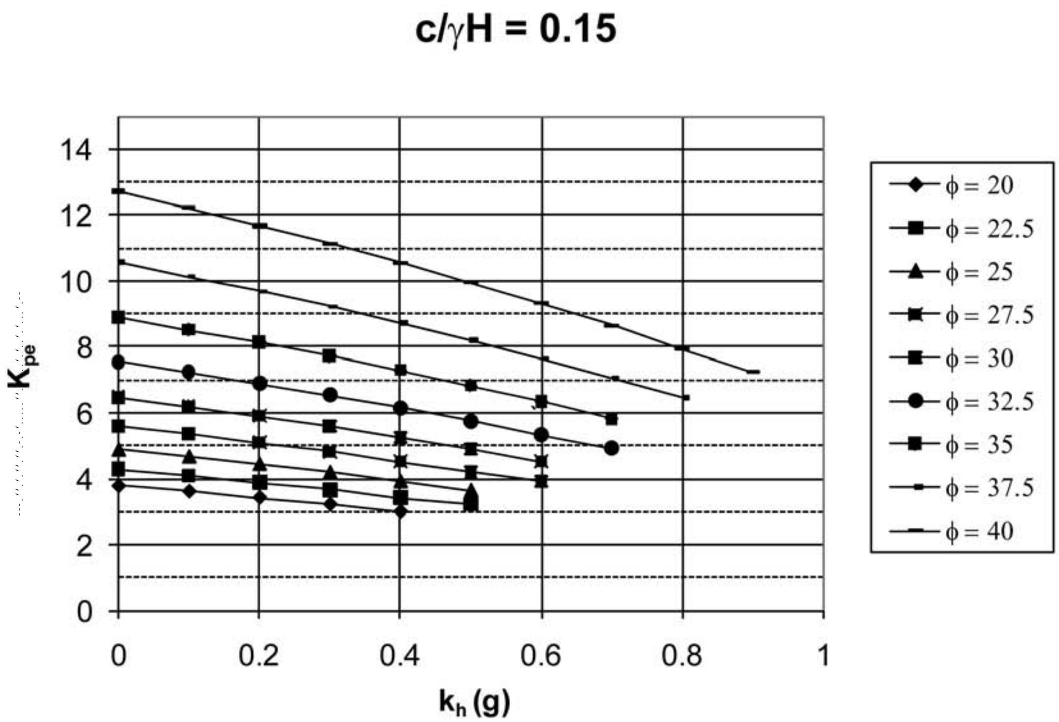
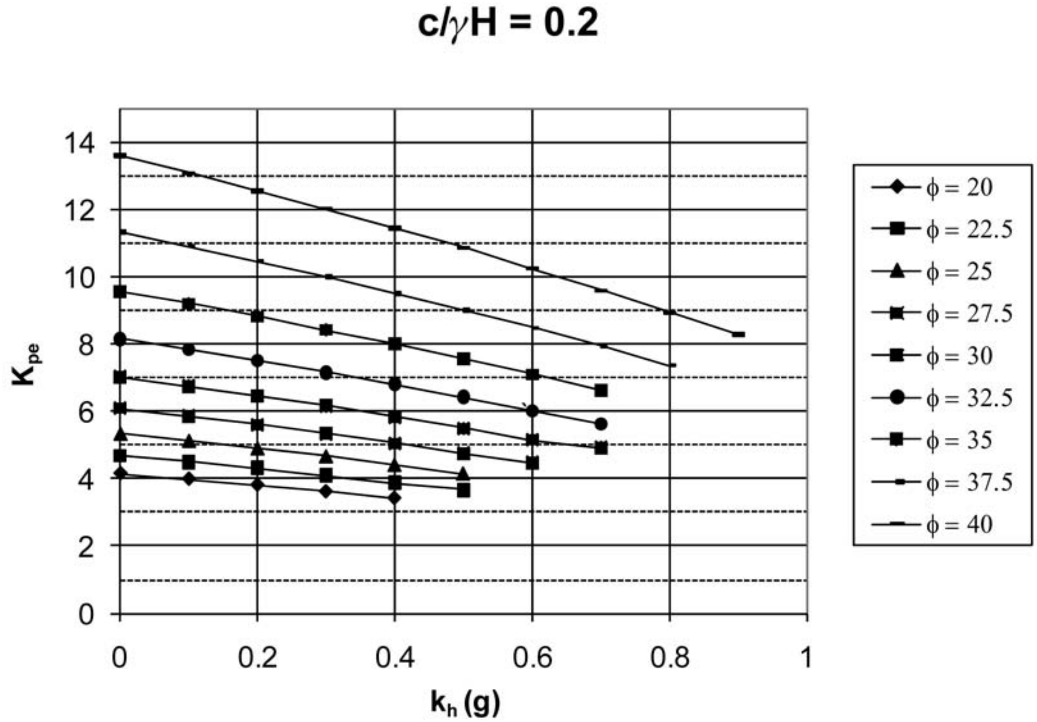


Figure A11.4-3—Seismic Passive Earth Pressure Coefficient Based on Log Spiral Procedure for *c*/γ*H* = 0.1 and 0.15 (*c* = soil cohesion, γ = soil unit weight, and *H* = retaining wall height or depth of wall over which the passive resistance acts)



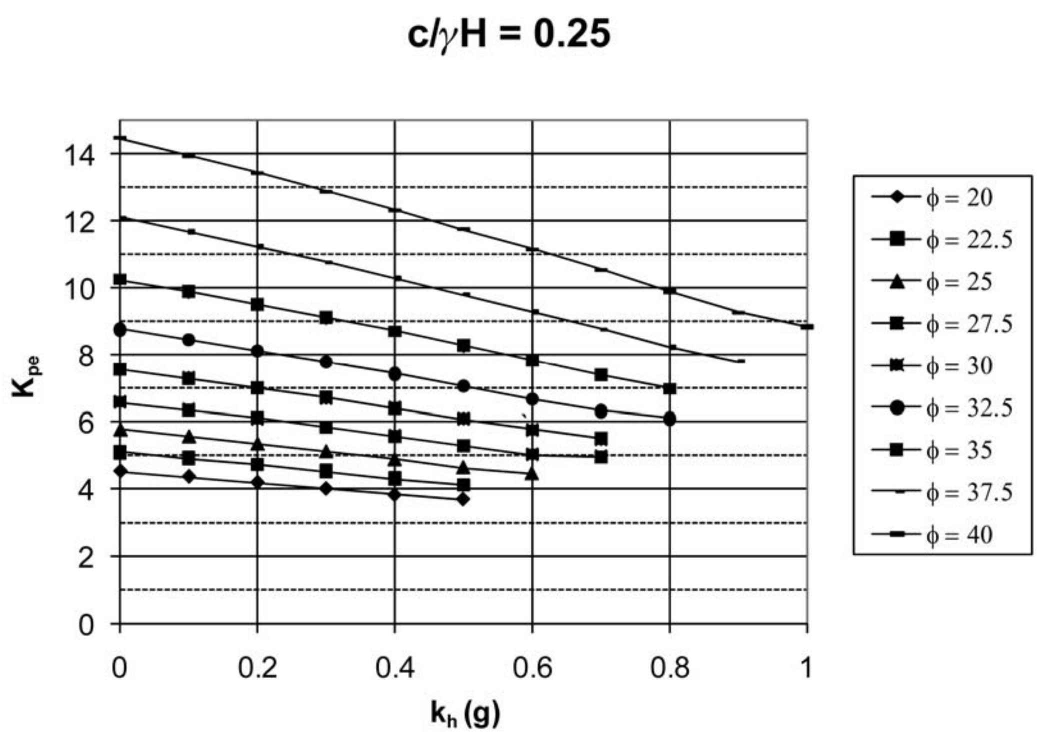


Figure A11.4-4—Seismic Passive Earth Pressure Coefficient Based on Log Spiral Procedure for $c/\gamma H$ = 0.2 and 0.25 (c = soil cohesion, γ = soil unit weight, and H = retaining wall height or depth of wall over which the passive resistance acts)

A11.5—ESTIMATING WALL SEISMIC ACCELERATION CONSIDERING WAVE SCATTERING AND WALL DISPLACEMENT

The seismic acceleration acting on a wall during an earthquake is affected by both wave scattering and wall displacement (see Article 11.6.5.2 and commentary).

With regard to the effects of wall deformation during shaking, the Newmark sliding block concept (Newmark, 1965) was originally developed to evaluate seismic slope stability in terms of earthquake-induced slope displacement as opposed to a factor of safety against yield under peak slope accelerations. The concept is illustrated in Figure A11.5-1, where a double integration procedure on accelerations exceeding the yield acceleration of the slope leads to an accumulated downslope displacement.

The concept of allowing gravity walls to slide during earthquake loading and displacement-based design (i.e., using a Newmark sliding block analysis to compute displacements when accelerations exceed the horizontal limiting equilibrium, yield acceleration for the wall-backfill system) was introduced by Richards and Elms (1979). Based on this concept, Elms and Martin (1979) suggested that a design acceleration coefficient of 0.5would be adequate for limit equilibrium pseudostatic design, provided allowance be made for a horizontal wall displacement of 10 *PGA* in inches. The *PGA* term in Elms and Martin is equivalent to the F_{PGA} *PGA* or k_{h0} in these Specifications.

For many situations, Newmark analysis or simplifications of it (e.g., displacement design charts or equations based on the Newmark analysis method for certain typical cases, or the use of $k_h = 0.5k_{h0}$) are sufficiently accurate. However, as the complexity of the site or the wall-soil system increases, more rigorous numerical modeling methods may become necessary.

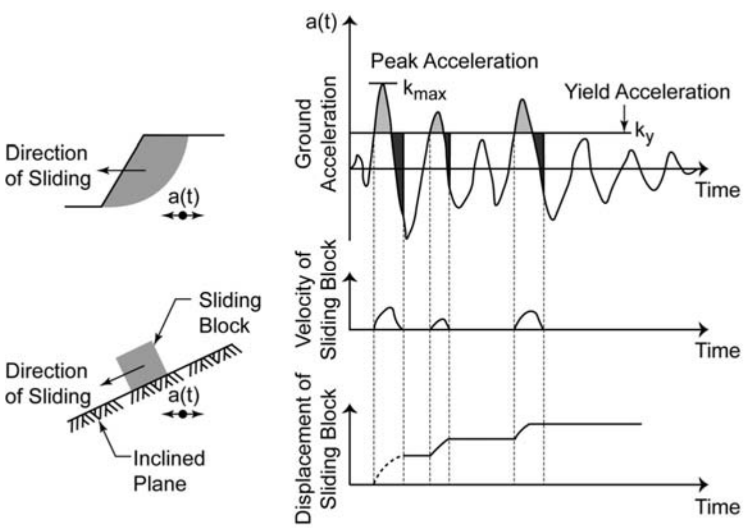


Figure A11.5-1—Newmark Sliding Block Concept

To assess the effects of wave scattering and lateral deformation on the design acceleration coefficient, k_h, three simplified design procedures to estimate the acceleration coefficient are provided in detail in the sub-sections that follow. The first method (Kavazanjian et al., 1997) does not directly address wave scattering and, since wave scattering tends to reduce the acceleration, the first method is likely conservative. The second and third methods account for both wave scattering and wall deformation but are considerably more complex than the first method. With regard to estimation of wave scattering effects, the second method (Anderson et al. 2008) uses a simplified model that considers the effect of the soil mass, but not specifically the effect of the wall as a structure, whereas the third method (Bray et al., 2010) provides a simplified response spectra for the wall, considering the wall to be a structure with a fundamental period. With regard to the effect of lateral wall deformation on the wall acceleration, both methods are based on many Newmark analyses, using those analyses to develop empirical relationships between the yield acceleration for the wall and the soil it retains and the amount of deformation that occurs. The Anderson et al. (2008) method estimates the wall deformation for input yield acceleration, peak ground acceleration, and peak ground velocity, whereas the third method (Bray et al. 2010) estimates the reduced acceleration, k_h, for a specified deformation and spectral acceleration at a specified period. The three alternative design procedures should not be mixed together in any way.

A11.5.1—Kavazanjian et al., (1997)

Kavazanjian et al. (1997) provided the following simplified relationship based on Newmark sliding analysis, assuming that the velocity, in the absence of information on the time history of the ground motion, is equal to 30A:

$$k_h = 0.74\, A_S \left(\frac{A_S}{d} \right)^{0.25} \tag{A11.5.1-1}$$

where:

A_S = earthquake ground acceleration coefficient as specified in Eq. 3.10.4.2-2 (dim.)
k_h = horizontal seismic acceleration coefficient (dim.)
d = lateral wall displacement (in.)

This equation was included in past editions of these Specifications. This equation should not be used for displacements of less than 1.0 in. or greater than approximately 8 in., as this equation is an approximation of a more rigorous Newmark analysis. 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. This method may be more conservative than the more complex methods that follow. Note that this method does not address wave scattering within the wall, which in most cases will be conservative.

A11.5.2—NCHRP Report 611—Anderson et al. (2008)

For values of h (as defined in Article 11.6.5.2.2) greater than 20.0 ft but less than 60.0 ft, the seismic coefficient used to compute lateral loads acting on a freestanding retaining wall may be modified to account for the effects of spatially varying ground motions behind the wall, using the following equation:

$$k_h = \alpha k_{h0} \tag{A11.5.2-1}$$

where:

K_{h0} = αk_{h0}
α = wall height acceleration reduction factor to account for wave scattering

For Site Class C, D, and E:

$$\alpha = 1 + 0.01h\left(0.5\beta - 1\right) \tag{A11.5.2-2}$$

where:

h = wall height (ft)
β = $F_v S_1 / k_{h0}$
S_1 = spectral acceleration coefficient at 1 sec
F_v = site class adjustment factor

For Site Classes A and B (hard and soft rock foundation soils), note that k_{h0} is increased by a factor of 1.2 as specified in Article 11.6.5.2.1. Eq. A11.5.2-1 provides the value of k_h if only wave scattering is considered and not lateral wall displacement.

For wall heights greater than 60.0 ft, special seismic design studies involving the use of dynamic numerical models should be conducted. These special studies are required in view of the potential consequences of failure of these very tall walls, as well as limitations in the simplified wave scattering methodology.

The basis for the height-dependent reduction factor described above is related to the response of the soil mass behind the retaining wall. Common practice in selecting the seismic coefficient for retaining wall design has been to assume rigid body soil response in the backfill behind a retaining wall. In this approach the horizontal seismic coefficient (k_{h0}) is assumed equal to the $F_{PGA}PGA$ when evaluating lateral forces acting on an active pressure failure zone. Whereas this assumption may be reasonable for wall heights less than about 20.0 ft, for higher walls, the magnitude of accelerations in soils behind the wall will vary spatially as shown schematically in Figure A11.5.2-1.

The nature and variation of the ground motions within a wall is complex and could be influenced by the dynamic response of the wall-soil system to the input earthquake ground motions. In addition to wall height, the acceleration distribution will depend on factors such as the frequency characteristics of the input ground motions, the stiffness contrast between backfill and foundation soils, the overall stiffness and damping characteristics of the wall, and wall slope. From a design standpoint, the net effect of the spatially varying ground motions can be represented by an averaging process over a potential active pressure zone, leading to a time history of average acceleration and hence a maximum average acceleration or seismic coefficient as shown in Figure A11.5.2-1.

To evaluate this averaging process, the results of a series of analytical studies are documented in NCHRP Report 611 (Anderson et al., 2008). An evaluation of these results forms the basis for the simplified Eqs. A11.5.2-1 and A11.5.2-2. The analytical studies included wave scattering analyses assuming elastic soil media using different slope heights, with slopes ranging from near vertical for short walls to significantly battered for tall walls, as well as slopes more typical of embankments (3*H*:1*V*) and with a range of earthquake time histories. The properties of the continuum used for these analyses were uniform throughout and therefore did not consider the potential effect of impedance contrasts between different materials (i.e., the properties of the wall vs. that of the surrounding soil). The acceleration time histories simulated spectral shapes representative of Western United States (WUS) and Central and Eastern United States (CEUS) sites and reflected different earthquake magnitudes and site conditions.

Additional height-dependent, one-dimensional SHAKE (Schnaebel et al., 1972) analyses were also conducted to evaluate the influence of nonlinear soil behavior and stiffness contrasts between backfill and foundation soils. These studies were also calibrated against finite element studies for MSE walls documented by Segrestin and Bastick (1988), which form the basis for the average maximum acceleration equation (a function of A_s) given in previous editions of these Specifications. The results of these studies demonstrate that the ratio of the maximum average seismic coefficient (k_h) to A_s (the α factor) is primarily dependent on the wall or slope height and the shape of the acceleration spectra (the β factor). The acceleration level has a lesser effect.

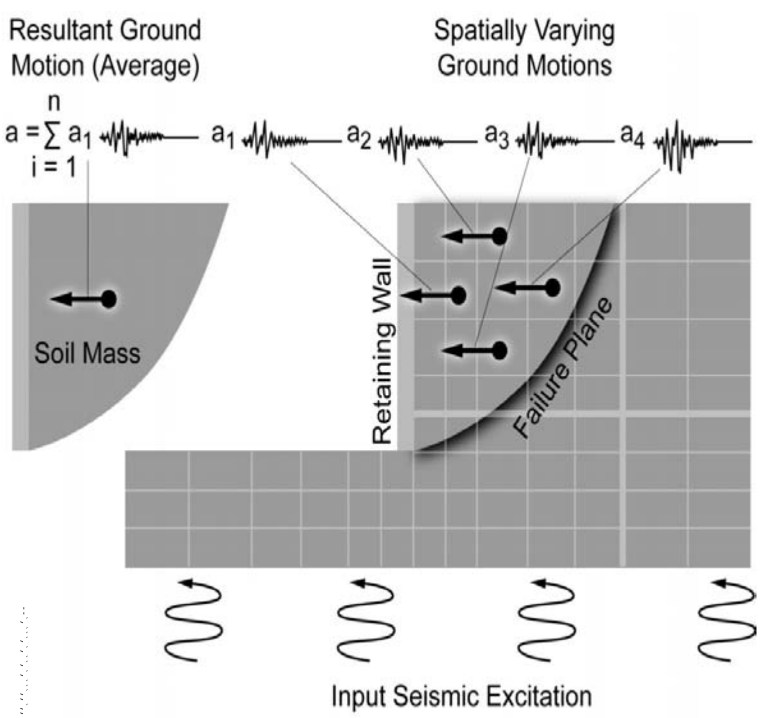


Figure A11.5.2-1—Average Seismic Coefficient Concept

Sliding block displacement analyses were conducted as part of NCHRP Report 611 (Anderson et al., 2008) using an extensive database of earthquake records. The objective of these analyses was to establish updated relationships between wall displacement (d) and the following three terms: the ratio k_y/k_{h0}, k_{h0} as determined in Article 11.6.5.2.1, and *PGV*. Two broad groups of ground motions were used to develop these equations, CEUS and WUS, as shown in Figure A11.5.2-2 (Anderson et al., 2008). Regressions of those analyses result in the following equations that can be used to estimate the relationship between wall displacement and acceleration.

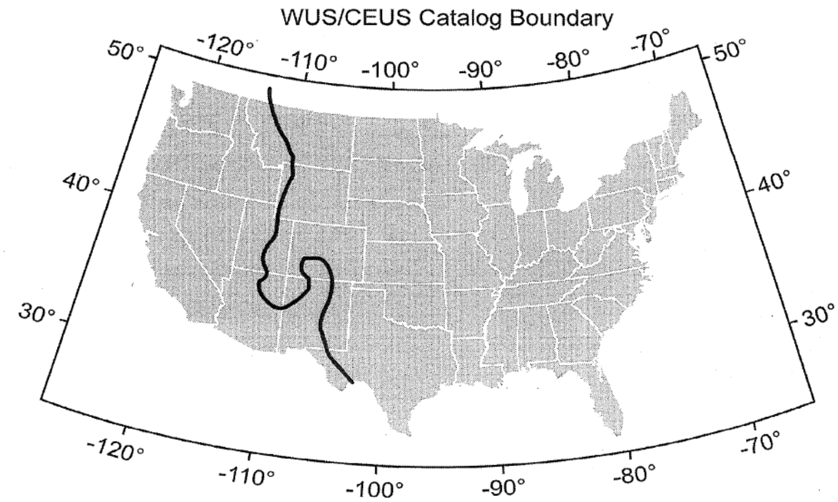


Figure A11.5.2-2—Boundary between WUS and CEUS Ground Motions

For all sites except CEUS rock sites (Categories A and B), the mean displacement (in.) for a given yield acceleration may be estimated as:

$$\log d = -1.51 - 0.74\log\left(\frac{k_v}{k_{h0}}\right) + 3.27\log\left(\frac{1-k_y}{k_{h0}}\right) - 0.80\log k_{h0} + 1.59\log\left(PGV\right) \quad \text{(A11.5.2-3)}$$

where:

k_y = yield acceleration

For CEUS rock sites (Categories A and B), this mean displacement (in.) may be estimated as:

$$\log d = -1.31 - 0.93\log\left(\frac{k_v}{k_{h0}}\right) + 4.52\log\left(1-\frac{k_v}{k_{h0}}\right) - 0.46\log\left(k_{h0}\right) + 1.12\log\left(PGV\right) \quad \text{(A11.5.2-4)}$$

Note that the above displacement equations represent mean values.

In Eqs. A11.5.2-3 and A11.5.2-4 it is necessary to estimate the peak ground velocity (*PGV*) and the yield acceleration (k_y). Values of *PGV* may be determined using the following correlation between *PGV* and spectral ordinates at 1 sec (S_1).

$$PGV\text{ (in./sec)} = 38F_vS_1 \quad \text{(A11.5.2-5)}$$

where S_1 is the spectral acceleration coefficient at 1 sec and F_v is the site class adjustment factor.

The development of the *PGV*-S_1 correlation is based on a simplification of regression analyses conducted on an extensive earthquake database established from recorded and synthetic accelerograms representative of both rock and soil conditions for WUS and CEUS. The study is described in NCHRP Report 611 (Anderson et al., 2008). It was found that earthquake magnitude need not be explicitly included in the correlation, as its influence on *PGV* is captured by its influence on the value of S_1. The equation is based on the mean from the simplification of the regression analysis.

Values of the yield acceleration (k_y) can be established by computing the seismic coefficient for global stability that results in a capacity to demand (*C*/*D*) ratio of 1.0 (i.e., for overall stability of the wall/slope, the *FS* = 1.0). A conventional slope stability program is normally used to determine the yield acceleration. For these analyses, the total stress (undrained) strength parameters of the soil should usually be used in the stability analysis. See guidance on the use of soil cohesion for seismic analyses discussed in Article 11.6.5.3 and its commentary.

Once k_y is determined, the combined effect of wave scattering and lateral wall displacement *d* on k_h is determined as follows:

$$k_h = \alpha k_y \quad \text{(A11.5.2-6)}$$

A11.5.3—Bray et al. (2010), and Bray and Travasarou (2009)

The Bray et al. (2010) method (see also Bray and Travasarou, 2009) for estimating the value of k_h applied to the wall mass considers both the wave scattering and lateral deformation of the wall. The method was developed using 688 ground motion records. The method characterizes the ground motion using a spectral acceleration at five percent damping, the moment magnitude, M, as a proxy for duration of shaking, the fundamental period of the wall, T_s, and the lateral wall deformation allowed during shaking. In this method, k_h is determined as follows:

$$k_h = \exp\left(\frac{-a+\sqrt{b}}{0.66}\right) \tag{A11.5.3-1}$$

where:

a = $2.83 - 0.566ln(S_a)$
b = $a^2 - 1.33[ln(d) + 1.10 - 3.04ln(S_a) + 0.244(ln(S_a))^2 - 1.5T_s - 0.278(M-7) - \varepsilon]$
S_a = the five percent damped spectral acceleration coefficient from the site response spectra
d = the maximum wall displacement allowed, in centimeters
M = the moment magnitude of the design earthquake
T_s = the fundamental period of the wall
ε = a normally distributed random variable with zero mean and a standard deviation of 0.66.

ε should be set equal to zero to estimate k_h considering D_a to be a mean displacement. To calculate the fundamental period of the wall, T_s, use the following equation:

$$T_s = 4H'/V_s \tag{A11.5.3-2}$$

where:

H' = 80 percent of the height of the wall, as measured from the bottom of the heel of the wall to the ground surface directly above the wall heel (or the total wall height at the back of the reinforced soil zone for MSE walls)
V_s = the shear wave velocity of the soil behind the wall

Note that V_s and H' must have consistent units. Shear wave velocities may be obtained from in-situ measurements or through the use of correlations to the Standard Penetration Resistance (*SPT*) or cone resistance (q_c). An example of this type of correlation for granular wall backfill materials is shown in Eq. A11.5.3-3 (Imai and Tonouchi, 1982).

$$V_s = 107N^{0.314} \tag{A11.5.3-3}$$

where:

N = the Standard Penetration Resistance (*SPT*) of the fill material, uncorrected for overburden pressure but corrected for hammer efficiency.

The spectral acceleration, S_a, is determined at a degraded period of $1.5T_s$ from the five percent damped response spectra for the site (i.e., either the response spectra determined using the general procedure or using a site-specific response spectra).

To estimate lateral wall displacement for a given acceleration value, see Bray et al. (2010) and Bray and Travasarou (2009) for details.

A11.6—APPENDIX REFERENCES

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