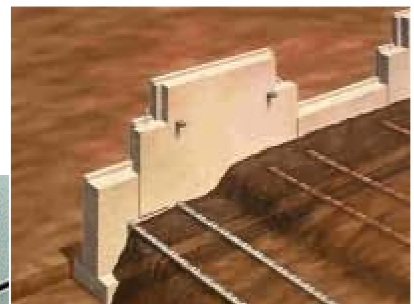
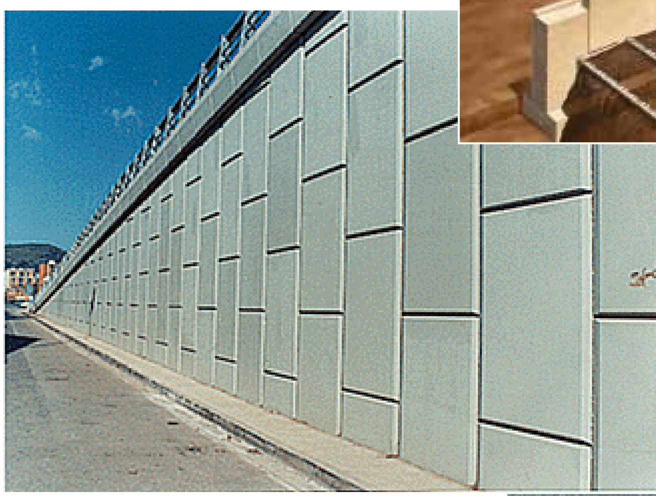
Spreadsheet Design of

Mechanically Stabilized Earth Walls





Spreadsheet Design of

Mechanically Stabilized Earth Walls

Prepared by
PRIME AE Group, Inc.
Harrisburg, Pennsylvania

For

The Pennsylvania Department of Transportation

Central Office

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MSE Wall Design Spreadsheet Capabilities

MSE Wall systems will be designed for two categories:

1. External Stability (deals with composite structure)
    a. Sliding
    b. Bearing Resistance
    c. Overturning (Eccentricity)

2. Internal Stability (deals with soil reinforcement)
    a. Reinforcement Pullout (pullout from reinforced soil mass)
    b. Reinforcement Strength (tension rupture)
    c. Reinforcing to Facing Connection

MSE walls will be investigated for:

- Vertical Pressure from Dead Load of Earth Fill (EV)
- Horizontal Earth Pressure (EH)
- Live Load Traffic Surcharge (LS)
- Earth Surcharge Load (ES) – when applicable
- Horizontal Traffic Impact Loads (CT)
- Self-Weight of the Wall, and Traffic Barriers – when applicable (DC)
- Roadway Surfaces (DW)
- Seismic Conditions, per A11.10.7 (EQ)

Wall Facing Systems:

- Precast Concrete Panels
- Modular Block (not to be confused with Prefabricated Modular Block Walls which rely on gravity to remain stable)
- Welded or Twisted Wire Mesh
- Geotextile Wrap

Soil Reinforcement Types:

- Metal Strip
- Steel Bar Grid Mat
- Welded Wire
- Geosynthetics (Geotextile sheets or Geogrids)

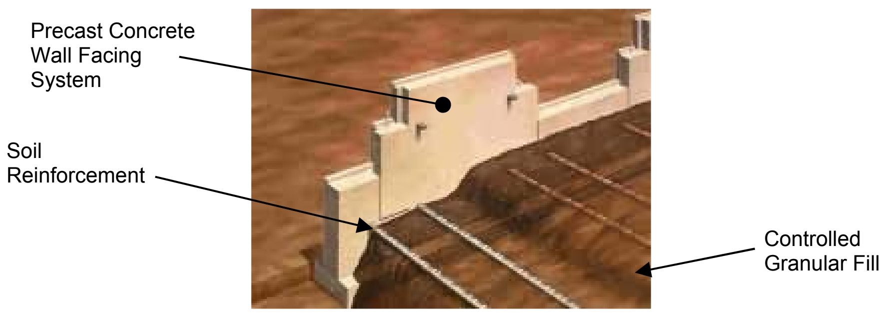
Backfill Conditions:

- Level backfill – with or without Abutment/ Barrier
- Sloping backfill
- Broken backfill – with or without Barrier

Introduction

The intent of this document is to briefly describe Mechanically Stabilized Earth Wall (MSE Wall) technology and to describe/define the methodology, equations and input used for the MSE Wall Design Spreadsheet.

MSE Walls are structures comprised of steel or geosynthetic soil reinforcements connected to a facing system, placed in layers within a controlled granular fill (see below).



The combination of reinforcement and granular fill creates a composite structure that is internally stable as long as sufficient reinforcement is placed within the fill to counteract shear forces. The manner in which stresses are transferred from the soil to the reinforcement depends on the type of MSE wall system used. Most contemporary systems use inextensible reinforcement, such as steel strips, bar mats or welded wire grids, in which the strains required to mobilize the full strength of the reinforcements are much smaller than those required to mobilize the strength of the soil. Extensible reinforcement systems, consisting of geosynthetic materials such as geotextile or geogrid, which require relatively large strains to mobilize the reinforcement strength, produce larger internal deformations. [8]

Originally invented in the late 1960's by Henri Vidal, a French architect and engineer, Reinforced Earth, which consists of soil, steel strip soil reinforcements and precast concrete facing panels was the first MSE system. Since that time other systems utilizing different facing systems (wire and concrete masonry blocks) and different soil reinforcement types (welded wire mesh, geogrids, geotextiles) have been used. [7]

MSE Wall systems are designed for two categories:

1. External Stability (deals with composite structure)
    a. Sliding
    b. Bearing Resistance
    c. Overturning (Eccentricity)
    d. Overall (Global) Stability
2. Internal Stability (deals with soil reinforcement)
    a. Reinforcement Pullout (pullout from reinforced soil mass)
    b. Reinforcement Strength (tension rupture)
    c. Reinforcing to Facing Connection

The weight and dimensions of the wall facing elements are typically ignored for both external and internal stability calculations. However, it is acceptable to include the facing dimensions and weight in the sliding and bearing capacity calculations [1, Fig11.10.2-1]. The spreadsheet considers the weight of the wall facing elements for both sliding stability and bearing capacity calculations.

The following wall facing systems and soil reinforcement types are most commonly used and can be accommodated by the MSE Wall Design Spreadsheet.

Wall Facing Systems:

- Precast Concrete Panels
- Modular Block (not to be confused with Prefabricated Modular Block Walls which rely on gravity to remain stable)
- Welded or Twisted Wire Mesh
- Geotextile Wrap

Soil Reinforcement Types:

- Metal Strip
- Steel Bar Grid Mat
- Welded Wire
- Geosynthetics (Geotextile Sheets or Geogrids)

External and internal stability calculations are separate and independent analyses, and the spreadsheet will therefore have the capability to analyze all combinations of the aforementioned wall facing systems and reinforcing types, in an independent fashion.

Summary of LRFD Methodology for MSE Wall Design

Design Specifications

The MSE Wall Design Spreadsheet will be based on the following:

AASHTO LRFD Bridge Design Specifications, Section 11.10 Mechanically Stabilized Earth Walls, 2010 Fifth Edition, as modified by PennDOT Design Manual Part 4, Part B Design Specifications (DM4), except as noted.

References made to specific sections in the AASHTO LRFD and DM4 code will be prefaced with an "A" and "D", respectively.

General Illustration of MSE Wall Elements

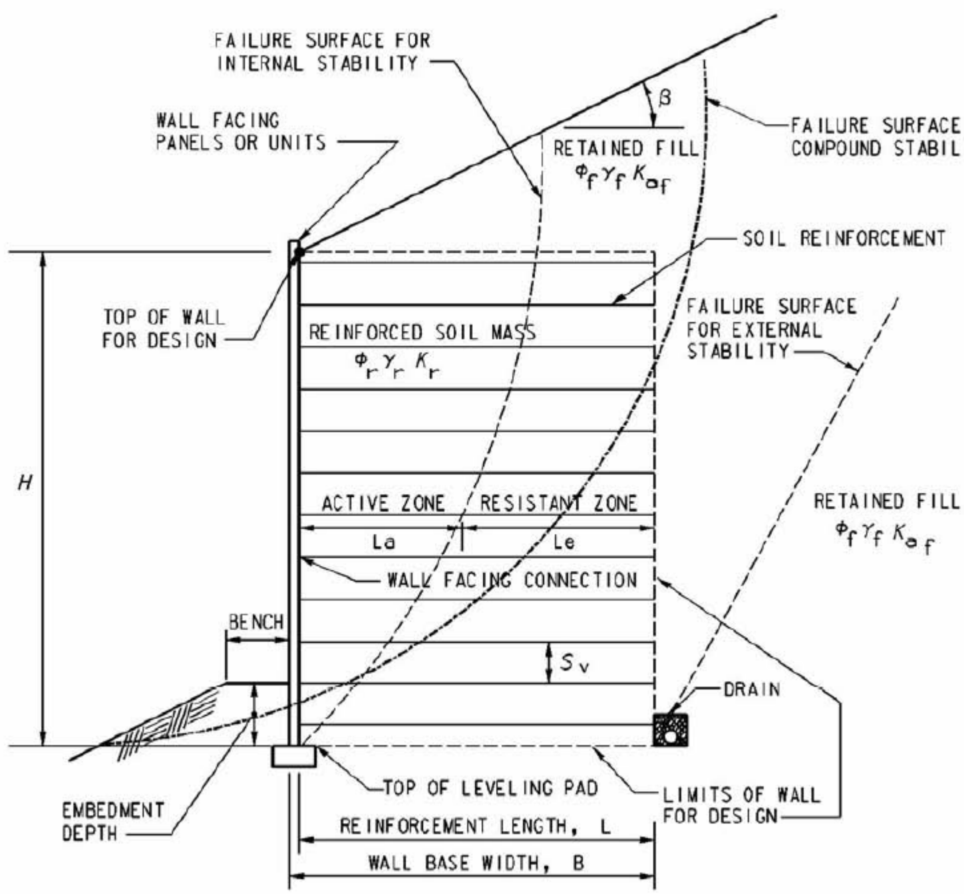


Figure A11.10.2-1 - MSE Wall Element Dimensions Needed for Design

The above illustration depicts MSE wall element dimensions required for design. This is a general illustration and does not identify all facing and reinforcement types or backfill conditions.

Key aspects of the MSE Wall analyses performed by the spreadsheet are governed by specific sections of the AASHTO LRFD code indicated below. More detailed descriptions of the equations and methodology used are offered in the sections that follow this summary.

Structure Dimensions – A11.10.2

A11.10.2.1 – Minimum Length of Soil Reinforcement

A11.10.2.2 – Minimum Front Face Embedment

A11.10.2.3 – Facing per:

- A11.10.6.2.2 Reinforcement Loads at Connection to Wall face
- A11.10.7.3 Facing Reinforcement Connections (Seismic)

Limit States – A11.5 & D11.5

Strength and Service Limit States for Design of MSE Walls

Performance Limit	Strength Limit State	Service Limit State
Sliding	✓	
Bearing Resistance	✓	
Overturning	✓	
Overall Stability	✓	
Rupture of Reinforcing Elements	✓	
Pullout of Reinforcing Elements	✓	
Structural Resistance of Face Elements	✓	
Structural Resistance of Reinforcing to Face Element Connection	✓	
Settlement and Lateral Displacement		✓

External Stability – A11.10.5

A11.10.5.2 & A11.10.10 – Loading

A11.10.4 – Movement and Stability at the Service Limit State

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.

In addition, the foundation should be improved by various improvement techniques such as over-excavation and replacement with compacted backfill using select material (DM4 C11.10.4)

For the purpose of this MSE wall design spreadsheet, it is assumed that the MSE wall will not experience unacceptable settlements or lateral displacements due to assumed relative stiffness of the foundation soil, adequate construction control and sufficient reinforcement length. It is also assumed that the wall will meet the restrictions set forth in D11.9.1 (a) and (b).

A11.10.5.3 – Sliding (per D10.6.3.4)

A11.10.5.4 – Bearing Resistance per:

- A10.6.3.1 Bearing resistance of soil (per D10.6.3.1)
- A10.6.3.2 Bearing resistance of rock (per D10.6.3.2)

A11.10.5.5 – Overturning (Eccentricity) (per A11.6.3.3)

A11.10.4.3 – Overall (Global) Stability (per A11.6.2.3)

Overall stability of the wall, retained slope and foundation soil or rock shall be evaluated using limiting equilibrium methods of analysis (A11.6.2.3). Computer programs such as *STABLE* are typically utilized for this external stability check. Due to the complexity of this type of analysis a check for overall stability is not included in the MSE Wall Spreadsheet.

Internal Stability – A11.10.6

A11.10.6.2 – Loading

A11.10.6.3 – Reinforcement Pullout

A11.10.6.4 – Reinforcement Strength

- A11.10.6.4.2 Design Life Considerations
- A11.10.6.4.2a Steel Reinforcements
- A11.10.6.4.2b Geosynthetic Reinforcements

A11.10.6.4.3 – Design Tensile Resistance

- A11.10.6.4.3a Steel Reinforcements
- A11.10.6.4.3b Geosynthetic Reinforcements

A11.10.6.4.4 – Reinforcement/Facing Connection Design Strength

- A11.10.6.4.4a Steel Reinforcements
- A11.10.6.4.4b Geosynthetic Reinforcements

Seismic Design – A11.10.7

A11.10.7.1 – External Stability

A11.10.7.2 – Internal Stability

A11.10.7.3 – Facing Reinforcement Connections

Special Loading Conditions – A11.10.10

A11.10.10.1 – Concentrated Dead Loads (ES)

A11.10.10.2 – Traffic Loads and Barriers (LS and CT) (per D11.10.10.2)

1.0 LRFD Limit States and Loading

1.1 LOADS (A3.3.2):

MSE walls will be investigated for:

- Vertical Pressure from Dead Load of Earth Fill (ES)
- Horizontal Earth Pressure (EH)
- Live Load Traffic Surcharge (LS)
- Earth Surcharge Load (ES) – when applicable
- Horizontal Traffic Impact Loads (CT)
- Self-Weight of the Wall, and Traffic Barriers – when applicable (DC)
- Roadway Surfaces (DW) – weight of roadway pavements wearing surfaces are all together considered as an (ES) load
- Seismic Conditions, per A11.10.7 (EQ)

1.2 LIMIT STATES (A1.3.2 & D1.3.2):

For design, the resistance and deformation of supporting soil, rock, and structure components must satisfy the following equations.

Strength Limit State: $\Sigma\, \eta_i\, \gamma_i\, Q_i \;\leq\; \phi\, R_n \;=\; R_r$ (A1.3.2.1-1)

Service Limit State: $\Sigma\, \eta_i\, \gamma_i\, \delta_i \;\leq\; \phi\, \delta_n$ [3]

where η_i = 1.0, per D1.3.2.1

The design of MSE walls using LRFD requires evaluation of the external stability of the wall, internal stability of the wall components and wall movements at various Performance Limit States. Based on A11.5 and A11.10 the following table lists design considerations (Performance Limits) and the appropriate Limit States for which they will be evaluated.

Table 1 - Strength and Service Limit States for Design of MSE Walls

Performance Limit	Strength Limit State	Service Limit State
Sliding	✓	
Bearing Resistance	✓	
Overturning	✓	
Overall Stability	✓	
Rupture of Reinforcing Elements	✓	
Pullout of Reinforcing Elements	✓	
Structural Resistance of Face Elements	✓	
Structural Resistance of Reinforcing to Face Element Connection	✓	
Settlement and Lateral Displacement		✓

1.3 LOAD FACTORS & COMBINATIONS (D3.4):

The following table, based on Table D3.4.1.1P-3 and A3.4.1-2 contains load factors and combinations relevant to MSE wall design. Additional load combinations are either redundant or have loadings which are not applicable.

Table 2 - Load Factors and Combinations for MSE Wall Design

Load Factor	SERV I	STR I		STR III		EXTREME I[2]		EXTREME II[3]	
		Min	Max	Min	Max	Min	Max	Min	Max
γ_{DC}	1.0	0.9	1.25	0.9	1.25	0.9	1.25	0.9	1.25
γ_{EV}	1.0	1.0	1.35	1.0	1.35	1.0	1.35	1.0	1.35
γ_{EH}	1.0	1.5	1.5	1.5	1.5	---	---	1.5	1.5
γ_{ESv}[1]	1.0	0.75	1.5	0.75	1.5	0.75	1.5	0.75	1.5
γ_{ESh}[1]	1.0	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
γ_{DWv}[1]	1.0	0.75	1.5	0.75	1.5	0.75	1.5	---	---
γ_{DWh}[1]	1.0	1.5	1.5	1.5	1.5	1.5	1.5	---	---
γ_{LS}[1]	1.0	1.75	1.75	---	---	0.0	0.0	0.50	0.50
γ_{EQ}	---	---	---	---	---	1.0	1.0	---	---
γ_{CT}	---	---	---	---	---	---	---	1.0	1.0

1.The minimum load factor will be used for the vertical component, always in conjunction with the maximum load factor for the corresponding horizontal component.
2.Extreme Event Limit State for seismic loading
3.Extreme Event Limit State for parapet collision force, CT

2.0 Structure Dimensions (A11.10.2)

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. The spreadsheet considers the weight of the wall facing elements for both sliding stability and bearing capacity calculations. For internal stability calculations, the wall dimensions are considered to begin at the back of the facing elements, i.e. the length of the reinforcement.

The size and embedment depth of the reinforced soil will be determined based on requirements for stability and geotechnical strength, structural resistance within the reinforced soil mass, and traditional requirements for reinforcement length discussed in A11.10.2.1.

2.1 MINIMUM LENGTH OF SOIL REINFORCEMENT (A11.10.2.1) (BC-799M)

The minimum length of sheet-, strip-, and grid-type reinforcement shall be 70% of the wall height as measured from the leveling pad. The reinforcement will be increased, as required, for surcharges, other external loads, soft foundation soils, or increased height due to abutment, where applicable. Reinforcement length will be uniform throughout the entire height of the wall.

therefore:

$$L_{\min} = 0.70H$$

2.2 MINIMUM FRONT FACE EMBEDMENT (A11.10.2.2) (BC-799M)

The minimum embedment depth of the top of the leveling pad (see Figure A11.10.2-1) shall be based on bearing resistance, settlement, and stability requirements determined in accordance with AASHTO and DM4, Section 10.

Embedment at front face shall not be less than:

- Depth of frost penetration, if the soil below the wall is frost susceptible, and external stability requirements
- and 2.0 ft on sloping ground (4.0H : 1V 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 (and 2 ft below potential scour depth if constructed adjacent rivers/streams)
- or 3.0 ft per BC-799M

Horizontal bench (see Figure A11.10.2-1):

- 4.0 ft width in front of walls founded on slopes

The following table shall be used as a minimum embedment guideline.

Table 3 – Minimum Embedment

Slope in Front of Structure	Minimum Embedment Depth
Horizontal	H/20.0
3.0H : 1.0V	H/10.0
2.0H : 1.0V	H/7.0
1.5H : 1.0V	H/5.0

(AASHTO Table C11.10.2.2-1 – Guide for Minimum Front Face Embedment Depth)

3.0 External Stability (A11.10.5)

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. A backfill soil friction angle corresponding to 35 pcf/ft of height of lateral earth pressure, based on equivalent fluid method (Rankine Method), shall be used as a minimum in the computation of design earth pressure (plus live load surcharge). For additional limitations, see D11.10.5.1 as follows:

- Saturated soil conditions to be considered in determining external stability of the wall
- Live load surcharge shall be applied from a vertical plane beyond the back of the reinforced zone
- For calculation of the horizontal design forces behind the reinforced soil mass, consider and apply the properties of the random backfill (retained soil) which includes 1 ft of specified backfill material

3.1 LOADING (A11.10.5.2):

3.1.1 MSE Wall Horizontal Earth Pressure (A3.11.5.8):

Based on A3.11.5.8, the resultant force per unit width behind an MSE wall, shown in Figures 1, 2 and 3 and acting at a height of h/3 above the base of the wall, shall be taken as:

$$P_a = 0.5 k_a \gamma_f h^2 \qquad \textit{(A3.11.5.8.1-1)}$$

with the active earth pressure coefficient, k_a, taken as specified in D3.11.5 as:

For horizontal or sloping backfill (Figures 1 & 2):

$$k_a = \cos\beta \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi_f}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi_f}} \qquad \textit{(D3.11.5.8.1-2)}$$

For broken backfill (Figure 3):

$$k_a = \cos B \frac{\cos B - \sqrt{\cos^2 B - \cos^2\phi_f}}{\cos B + \sqrt{\cos^2 B - \cos^2\phi_f}} \qquad \textit{(D3.11.5.8.1-3)}$$

where:

P_a = force resultant of earth pressure on wall, per unit width of wall
β = slope of backfill surface behind MSE wall (Figures 2 and 3)
B = notional slope of backfill behind wall (Figure 3)
γ_f = unit weight of retained backfill/soil
h = height of horizontal earth pressure diagram (Figures 1, 2, and 3)
ϕ_f = internal friction angle of retained soil

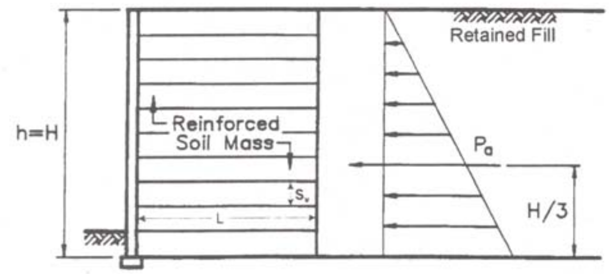


Figure 1. AASHTO Figure 3.11.5.8.1-1 – Earth Pressure Distribution for MSE Wall with Level Backfill Surface

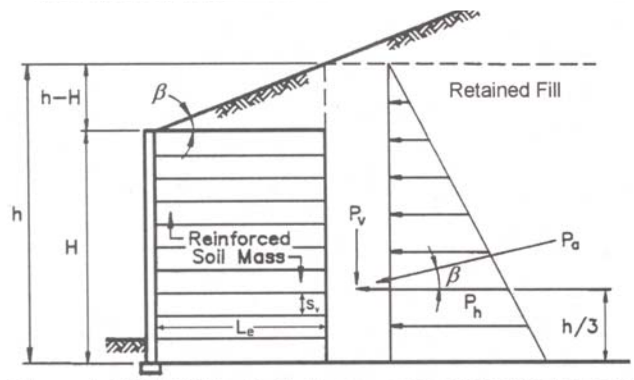


Figure 2. AASHTO Figure 3.11.5.8.1-2 – Earth Pressure for MSE Wall with Sloping Backfill Surface

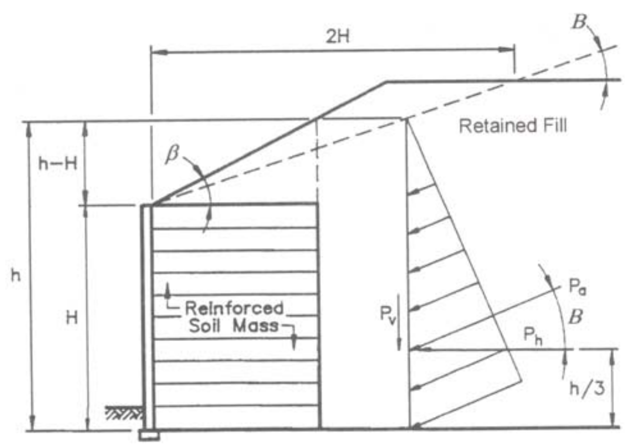


Figure 3. AASHTO Figure 3.11.5.8.1-3 – Earth Pressure Distribution for MSE Wall with Broken Back Backfill Surface

3.1.2 Earth (ES) Surcharge (A11.10.10.1, A3.11.6.3):

Concentrated dead loads (ES) shall be incorporated into the internal and external stability design by using a simplified uniform vertical distribution of 2V:1H. Distribution of stress from concentrated vertical (ES) loads is described in Figure 4. Refer to A3.11.6.3 for further explanation. This loading case would be most applicable for stub abutments on piles supported by MSE walls.

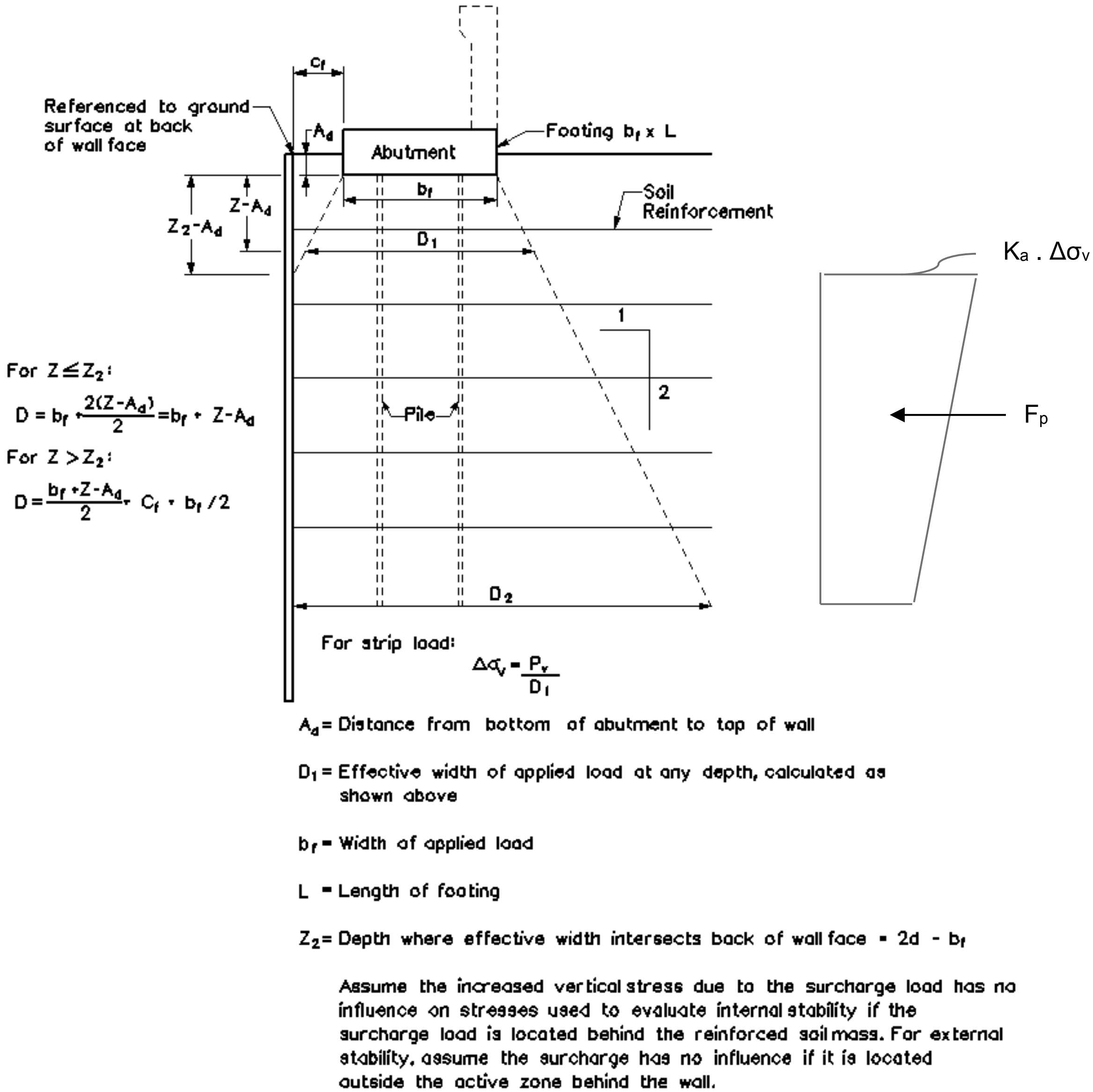


Figure 4. Distribution of Stress from Concentrated Vertical Load P_v for Internal and External Stability Calculations

Additionally, horizontal surcharge loads developed due to the vertical surcharges mentioned above will also be applicable from loads such as: weight of roadway pavement (DW), weight of backfill (ES), and weight of wet concrete footing (P_V). The force F_p shown above depicts the corresponding stress variation. See Figure A11.10.10.1-1

3.1.3 Live Load Traffic (LS) Surcharge (A11.10.10.2, A3.11.6.4 and supplemented by D3.11.6.4):

A live load surcharge will be applied where vehicular traffic load is expected to act on the surface of the backfill based on Figure 5, or as governed laterally by a parapet/barrier. When applicable, traffic LS surcharge will be applied to the reinforced soil mass and the retained fill for bearing capacity and overall stability.

For overturning and sliding resistance, LS will only be applied to the retained fill. The horizontal component of LS may be applied without any vertical component.

It is assumed that traffic surcharge will never be applied to the “sloping” condition, as depicted in Figure 2. An “Abutment” will be applicable for a “Horizontal Backfill” condition only.

The increase in horizontal pressure due to live load surcharge will be estimated as:

$$F_2 = k_{af} qH = k(\gamma_f h_{eq})H = \Delta pH$$ *(F_2 from Figure 5)*

such that:

$$\Delta p = k\gamma_f h_{eq}$$ *(A3.11.6.4-1)*

where:

Δp = constant horizontal earth pressure due to live load surcharge

γ_f = total unit weight of soil for live load surcharge

k = coefficient of lateral earth pressure taken as k_a for MSE walls

h_{eq} = equivalent height of soil for vehicular load as specified per DM4 Table 3.11.6.4-2

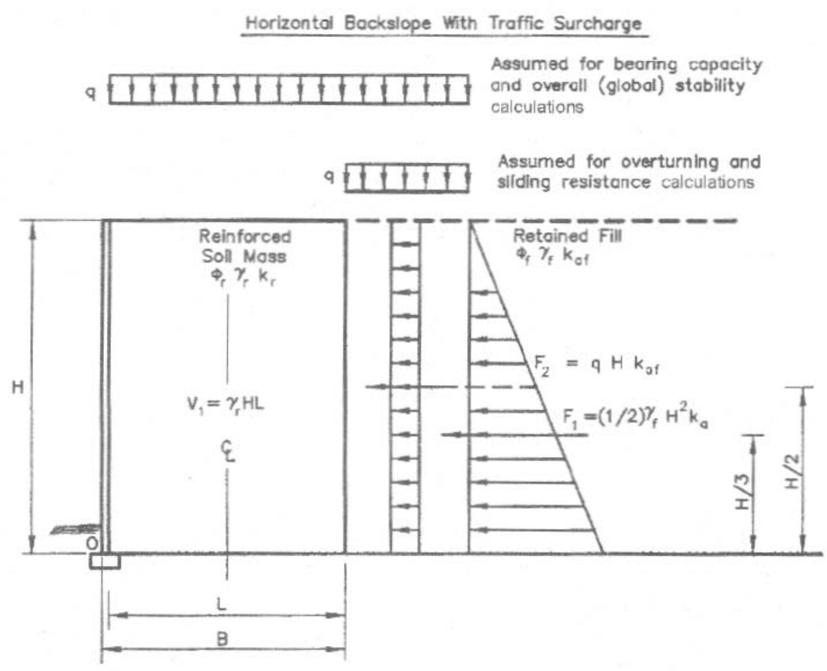


Figure 5. AASHTO Figure 11.10.5.2-1 – External Stability for Wall with Horizontal Backslope and Traffic Surcharge

3.1.4 Horizontal (CT) Collision Loads (D11.10.10.2, A3.11.6.3, Figure A3.11.6.3-2b):

Applied per Figure 6, assuming the horizontal load P_{H2} represents a vehicular collision (CT) load. The footing depicted on the retained fill portion shall represent the parapet to which CT is applied. The parapet bearing pressure will be assumed negligible and will not be considered for external stability calculations.

where:

P_{H2} = assumed vehicular collision (impact) load (CT)

$\Delta\sigma_H$ = horizontal stress due to surcharge load, as defined in Figure 6

c_f = distance from back of wall to the back face of the parapet

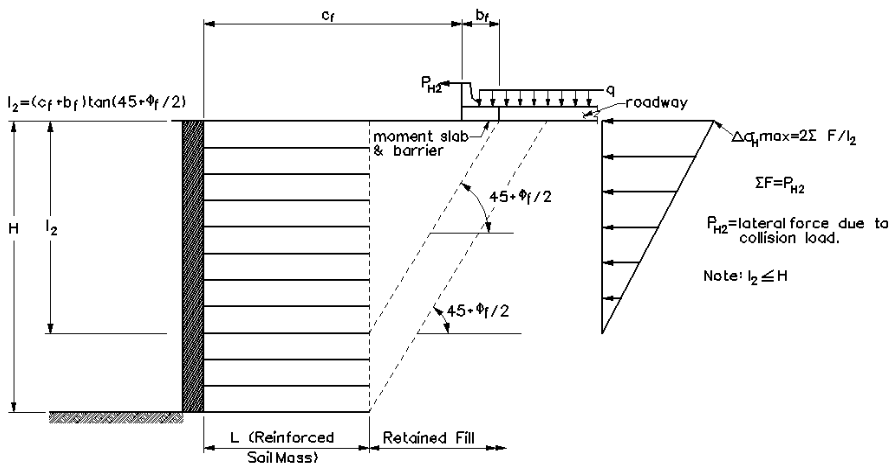


Figure 6. Distribution of Stress from Concentrated Horizontal Loads for External Stability Calculations

When CT is applied (Extreme Event II Limit State), l_2 from Figure 6 will be taken as:

$$l_2 = (c_f + b_f)\tan(45 + \phi_f / 2) \qquad \text{when } c_f <= L$$
$$l_2 = (c_f + b_f - L)\tan(45 + \phi_f / 2) \quad \text{when } c_f > L$$

Horizontal Loads (A3.11.6.3)

The effect of horizontal loads on the wall will be computed based on Article A3.11.6.3. The following forces are distributed according to Figures A3.11.6.3-1, A3.11.6.3-2a and A3.11.6.3-2b and combined:

a) Longitudinal forces acting on the abutment from superstructure (PH1a) (Figure 14)
b) Collision forces on barriers (CT), distributed to the wall as PH1 (Figure 13) and as PH2 (Figure 6)
c) Lateral force effects from vertical surcharge load (ES), weight of wet concrete foundations of abutments on piles (P_V), weight of roadway pavement and wearing surface (DW), and vertical live load surcharge (LS) using active earth pressure coefficient k_a.

Note that the live load surcharge (LS) will be included in Extreme-II Limit State considering CT loads.

3.2 SLIDING (A11.10.5.3 & D10.6.3.4):

The MSE Wall spreadsheet will neglect passive resistance (R_{ep}) in the evaluation of sliding, per D10.6.3.4.

Factored resistance against failure by sliding will be taken as:

$$R_R = \varphi R_n = \varphi_\tau R_\tau \qquad \textit{(D10.6.3.4-1)}$$

where:

φ_τ = resistance factor for shear resistance between soil and foundation specified in Table D10.5.5.2.2-1

R_τ = nominal resistance for sliding between soil and foundation

where R_τ equals:

1. For cohesionless soil or rock:

$$R_\tau = V \tan \delta \qquad \textit{(A10.6.3.4-2)}$$

where:

$\tan \delta$ = $\tan \phi_{fw}$ for sliding of one soil on another or on reinforcement ($\tan \rho$)

ϕ_b = internal friction angle of base soil

ϕ_r = internal friction angle of reinforced fill

ρ = soil-reinforcement interface friction angle ($2/3\phi_b$)

ϕ_{fw} = internal friction angle of weaker soil or ρ

V = total vertical force per unit width

2. For soils exhibiting both frictional and cohesive shear strength components (c-ϕ Soils):

$$R_\tau = V \tan \delta + c_a B' \qquad \textit{(D10.6.3.4-3P)}$$

where:

tan δ = tan ϕ_{fw} for sliding of one soil on another

c_a = adhesion between footing and soil, taken as

$c\ (0.21+0.27/c) \leq 1.0$,

unless better data is available, where c is defined in Section 3.3.1; (c and c_a in tsf)

B' = effective footing width as specified in Section 3.3.1, per A10.6.1.3

V = total vertical force per unit width

3. Foundations on clay, for which the minimum over-excavation and structure backfill is specified in accordance with D10.6.1.9P

Sliding Resistance on clay foundation layer shall be taken as lesser of:

1. The cohesion of the clay, c, or
2. Where footings are supported on at least 6.0 inches of compacted granular material, one-half the normal stress on the interface between footing and soil, as shown in Figure A10.6.3.4-1 for retaining walls.

3.3 BEARING RESISTANCE (A11.10.5.4):

3.3.1 Bearing on soil (A10.6.3.1 & D10.6.3.1):

$q_R = \varphi_b q_n > q$ (Factored Bearing Pressure) *(A10.6.3.1.1-1)*

where φ_b is the bearing resistance factor specified in DM4 Table 10.5.5.2.2-1.

For continuous footings (L > 5B):

General Equation:

$$q_n = cN_c + 0.5\gamma BN_\gamma + \gamma D_f N_q$$ *(A10.6.3.1.2a-1)*

Modified Equation (accounts for footing shape, ground surface slope, and inclined loading):

$$q_n = cN_c s_c i_c + 0.5\gamma BN_\gamma s_\gamma i_\gamma + \gamma D_f N_q s_q i_q$$ *(A10.6.3.1.2a-10P)*

where c refers to the cohesion through which clay primarily develops it's resistance to load, and analogous to ϕ_f for non-cohesive or granular soil.

Bearing Capacity Factors (DM4 C10.6.3.1.2a)

$$N_q = (e^{\pi \tan\phi_f})\tan^2(45 + \phi_f / 2)$$

$$N_c = (Nq - 1)\cot\phi_f \quad \text{(for } \phi_f > 0\text{)}$$

$$N_c = 2 + \pi \quad \text{(for } \phi_f = 0\text{)}$$

$$N_\gamma = 2(N_q + 1)\tan\phi_f$$

Where a slope exists in front of the MSE wall, user input would be necessary for the parameters N_{cq} and $N_{\gamma q}$ in conformance with Section A10.6.3.1.2c. Appropriate values from Figure A10.6.3.1.2c-1 (N_{cq} for Cohesive soils) and Figure A10.6.3.1.2c-2 ($N_{\gamma q}$ for Non-Cohesive soils), in consultation with Geotechnical Engineer, to be substituted.

Eccentric Loading (A10.6.1.3))

$$B' = B - 2e_B$$ *(A10.6.1.3-1)*

$$L' = L - 2e_L$$ *(A10.6.1.3-1)*

Footing Shape Factors (D10.6.3.1.2a)

S_c, S_γ, S_q = for footing shapes other than continuous footings (i.e.. L < 5B), footing shape correction factors as specified in Table A10.6.3.1.2a-3 (dim). For L ≥ 5B, shape factors = 1.0.

Where eccentric loading is present B' and L' will be substituted in place of B and L, respectively, for all equations for bearing in conformance with Section A10.6.3.1.1.

Inclined Loading Factors (A10.6.3.1.2a)

$i_c = i_q - [(1-i_q)/(N_q-1)]$, (for $\phi_f > 0$) *(A10.6.3.1.2a-6)*

$i_c = 1-(nH/cBLN_c)$, (for $\phi_f = 0$) *(A10.6.3.1.2a-5)*

$i_q = [1-(H)/(V+BLc\cot\phi_f)]^n$, Use $i_q = 1.0$ for $\phi_f = 0$ *(A10.6.3.1.2a-7)*

$i_\gamma = [1-(H)/(V+BLc\cot\phi_f)]^{(n+1)}$, Use $i_r = 1.0$ for $\phi_f = 0$ *(A10.6.3.1.2a-8)*

$n = [(2+L/B)/(1+L/B)]\cos^2\theta + [(2+B/L)/(1+B/L)]\sin^2\theta$ *(A10.6.3.1.2a-9)*

Groundwater (DM4 10.6.3.1.2gP, and Figure 7)

a. For $\phi_f < 37°$

$z_w \geq B$: use $\gamma = \gamma_m$ *(D10.6.3.1.2gP-1)*

$z_w < B$: use $\gamma = \gamma' + (z_w/B)(\gamma_m - \gamma')$ *(D10.6.3.1.2gP-2)*

$z_w \leq 0$: use $\gamma = \gamma'$ *(D10.6.3.1.2gP-3)*

b. For $\phi_f \geq 37°$

$z_w \geq D$, $\gamma = \gamma_m$ *(D10.6.3.1.2gP-4)*

$z_w < D$, $\gamma = (2D - z_w)(z_w\gamma_m/D^2) + (\gamma'/D^2)(D-z_w)^2$ *(D10.6.3.1.2gP-5), (A10.6.3.1.2a-9)*

where $D = 0.5B\tan(45^\circ + \phi_f/2)$ *(D10.6.3.1.2gP-6)*

$z_w \leq 0$, $\gamma = \gamma'$ *(D10.6.3.1.2gP-7)*

A negative value for z_w will represent a saturated soil condition above bottom of footing.

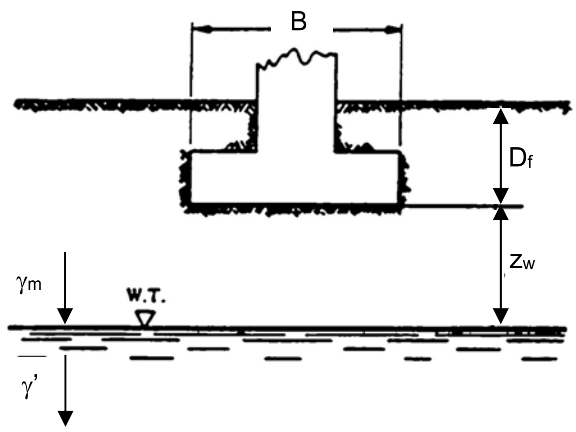


Figure 7. DM4 Figure 10.6.3.1.2gP-1 Definition Sketch for Influence of Groundwater Table on Bearing Capacity

3.3.2 Bearing on rock (D10.6.3.2.2, Semi-Empirical Procedure):

$$q_R = \varphi_b q_n = \varphi_b N_{ms} C_o$$

such that $q_R > V_{tot} / \mathrm{B}'$

where:

C_o = laboratory tested compressive strength of rock sample

N_{ms} = coefficient factor to estimate ultimate bearing resistance of rock (q_n) specified in DM4 Table 10.6.3.2.2-1P

φ_b = bearing capacity resistance factor for foundation on rock specified in DM4 Table 10.5.5.2.2-1.

V_{tot} = total factored vertical load per unit width

B' = effective footing width for load eccentric (short side), as specified in A10.6.1.3

3.4 OVERTURNING (ECCENTRICITY) (A11.10.5.5, A11.6.3.3):

The location of the vertical resultant of the reaction forces (e_B) shall not fall beyond the maximum location (e_{max}):

$$e_B < e_{\max}$$

such that:

1. For foundations on SOIL: the location of the resultant of the reaction forces (e_{maxS}) shall be within the middle one-half of the base width
2. For foundations on ROCK: the location of the resultant of the reaction forces (e_{maxR}) shall be within the middle three-fourths of the base width

where:

$$e_{\max S} = \mathrm{B}/4$$
$$e_{\max R} = 3\mathrm{B}/8$$
$$e_B = \mathrm{B}/2 - X_o \text{ = Eccentricity}$$
$$X_o = (M_{vtot} - M_{htot}) / V_{tot}$$

such that:

M_{vtot} = Total factored overturning moment caused by vertical loads per unit width

M_{htot} = Total factored overturning moment caused by horizontal loads per unit width

V_{tot} = Total factored vertical loads per unit width

3.5 SEISMIC CONSIDERATIONS FOR EXTERNAL STABILITY (A11.10.7.1):

Stability determinations will be made by applying static forces, the horizontal inertial force, P_{IR}, and 50 percent of the dynamic horizontal thrust, P_{AE}, to the wall. P_{AE} will be evaluated using the pseudo-static Mononobe-Okabe method, and applied based on Figure 8 and 9.

P_{IR} and P_{AE} will be determined based the following:

3.5.1 For Horizontal backfill:

$$A_m = (1.45 - A_s)A_s \quad \text{(A11.10.7.1-1)}$$

$$P_{AE} = 0.375\gamma_{EQ} A_m \gamma_s H^2 \quad \text{(A11.10.7.1-2)}$$

$$P_{IR} = 0.5\gamma_{EQ} A_m \gamma_s H^2 \quad \text{(A11.10.7.1-3)}$$

where:

A_s = peak seismic ground acceleration coefficient modified by short-period site factor specified in A3.10.4.

γ_{EQ} = load factor for EQ loads from Table 2

γ_s = soil unit weight (backfill)

H = height of wall

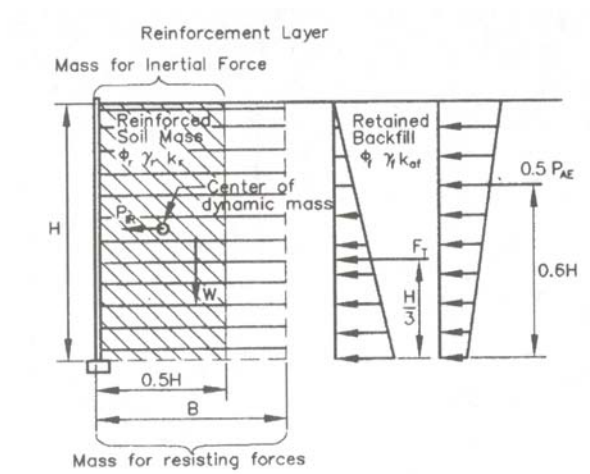


Figure 8. AASHTO Figure 11.10.7.1-1a - Seismic External Stability of a MSE Wall, Level Backfill Condition

3.5.2 For Sloping backfills:

$$P_{IR} = P_{ir} + P_{is} \quad \text{(A11.10.7.1-5)}$$

where P_{ir} is the inertial force caused by acceleration of the reinforced backfill, and P_{is} is the inertial force caused by the acceleration of the sloping soil surcharge above the reinforced backfill:

$$P_{ir} = 0.5\gamma_{EQ} A_m \gamma_s H_2 H \quad \text{(A11.10.7.1-6)}$$

$$P_{is} = 0.125\gamma_{EQ} A_m \gamma_s (H_2)^2 \tan\beta \quad \text{(A11.10.7.1-7)}$$

such that:

β = slope of backfill

$$H_2 = H + \frac{0.5H \tan\beta}{(1 - 0.5\tan\beta)} \quad \text{(A11.10.7.1-4)}$$

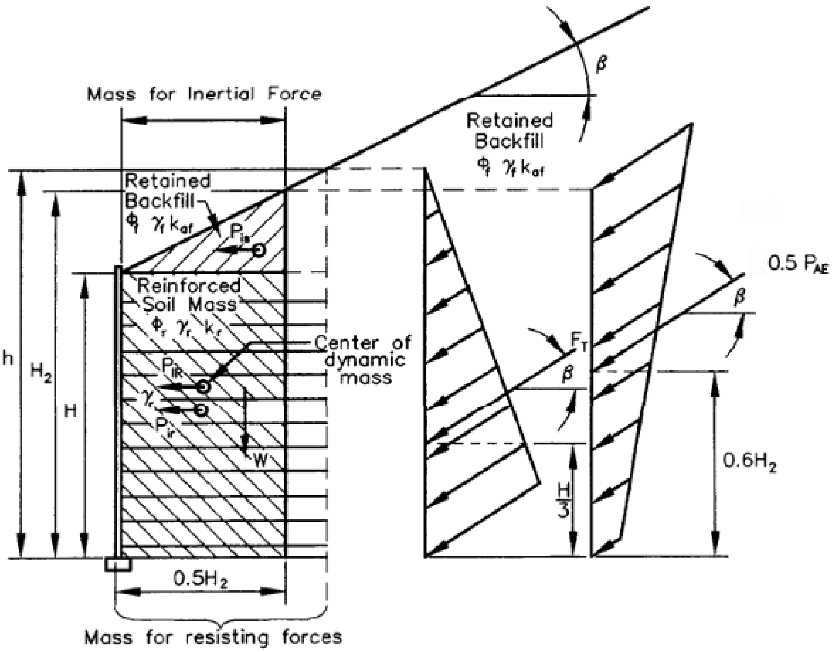


Figure 9. AASHTO Figure 11.10.7.1-1b - Seismic External Stability of a MSE Wall, Sloping Backfill Condition

4.0 Internal Stability (A11.10.6)

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

4.1 LOADING (A11.10.6.2):

The load in the reinforcement shall be determined at two critical locations:

1. the zone of maximum stress
2. the connection with the wall face

The Simplified Method, per A11.10.6.2.1, shall be used to calculate loads.

Potential for reinforcement rupture and pullout are evaluated at the zone of maximum stress, which is assumed to be at the boundary between the active zone and the resistant zone (see Figure A11.10.2-1 in the Summary), and also at the connection of the reinforcement to the wall facing.

Maximum friction angle used for the computation of horizontal force within the reinforced soil mass shall be assumed to be 34 degrees, unless the backfill is tested for frictional strength via triaxial or direct shear testing methods as specified in A11.10.6.2. 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.

NOTE that live load surcharge loads are NEGLECTED in soil reinforcement pullout calculations as per A11.10.6.3.2.

4.1.1 Maximum Reinforcement Loads (A11.10.6.2.1) (D11.10.6.2.1)

Maximum factored reinforcement loads (T_{max}) shall be calculated in the following manner:

$$T_{\max} = \sigma_H S_v \qquad \textit{(A11.10.6.2.1-2)}$$

where:

S_v = vertical spacing of reinforcement (Constant or variable spacing can be handled; the maximum spacing is limited to 2'-0" where Geosynthetic reinforcement is used

σ_H = factored horizontal soil stress at the reinforcement level

equal to:

$$\sigma_H = \gamma_P(\sigma_v k_r + \Delta\sigma_H) \qquad \textit{(A11.10.6.2.1-1)}$$

where:

γ_P = load factor for EV specified in Table 2 specified (maximum) as γ_{EV}, or as specified for per section 4.3.3.

k_r = horizontal pressure coefficient = multiplier from Figure 10 * k_a, where k_a is determined as specified in Section 3.1.1 using DM4 equations D3.11.5.8-2, and D3.11.5.8-3 for horizontal/sloping and broken backfill situations, respectively.

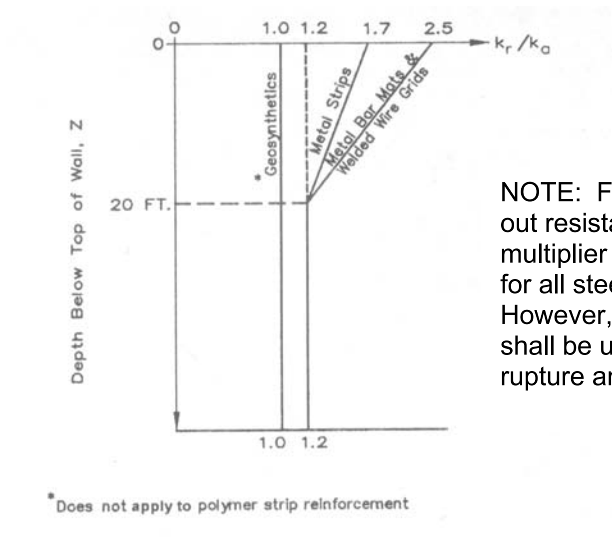


Figure 10. AASHTO Figure 11.10.6.2.1-3 – Variation of the Coefficient of Lateral Stress Ratio Kr/Ka with Depth in a MSE Wall

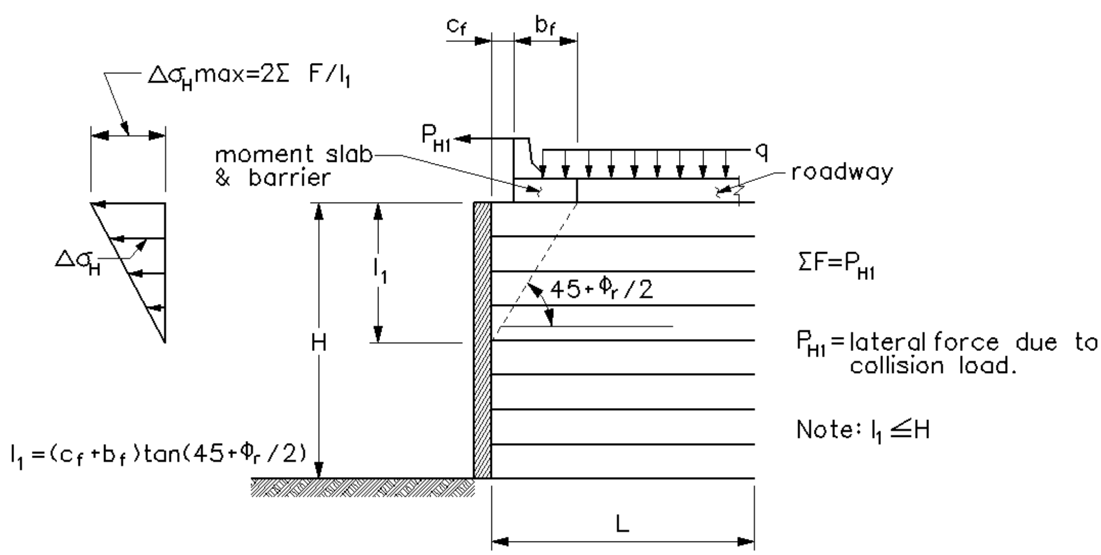


Figure 11. AASHTO Figure 3.11.6.3-2a – Distribution of Stress from Concentrated Horizontal Loads for Internal Stability Calculations

$\Delta\sigma_H$ = horizontal stress at reinforcement level resulting from a concentrated horizontal surcharge load per Figure 11,

OR,

$\Delta\sigma_H$ = 0, in the case of Abutment on piles where the horizontal surcharge loads from backfill behind the abutment (DW,ES, LS) are resisted by the abutment reinforcement only (see Figure 14), with no additional stress in the soil reinforcement

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 calculated based on the following:

1. Horizontal Backslope Condition (see Figure 12):

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

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

Where $\Delta\sigma_v$, if applicable, is determined from Figure 4, Section 3.1.2.

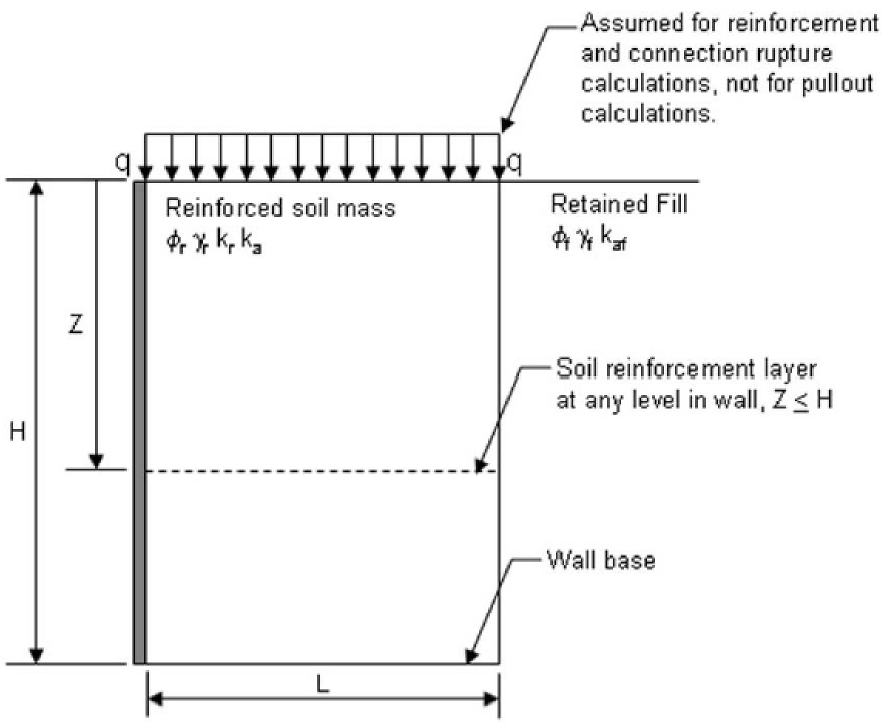


Figure 12. AASHTO Figure 11.10.6.2.1-1 – Calculation of Vertical Stress for Horizontal Backslope Condition, Including Live Load and Dead Load Surcharges for Internal Stability

For the situation where the barrier and slab extend beyond the end of the soil reinforcement (See Figure 13), separate treatment of the internal and external stability is required for the applicable portion of the slab and barrier.

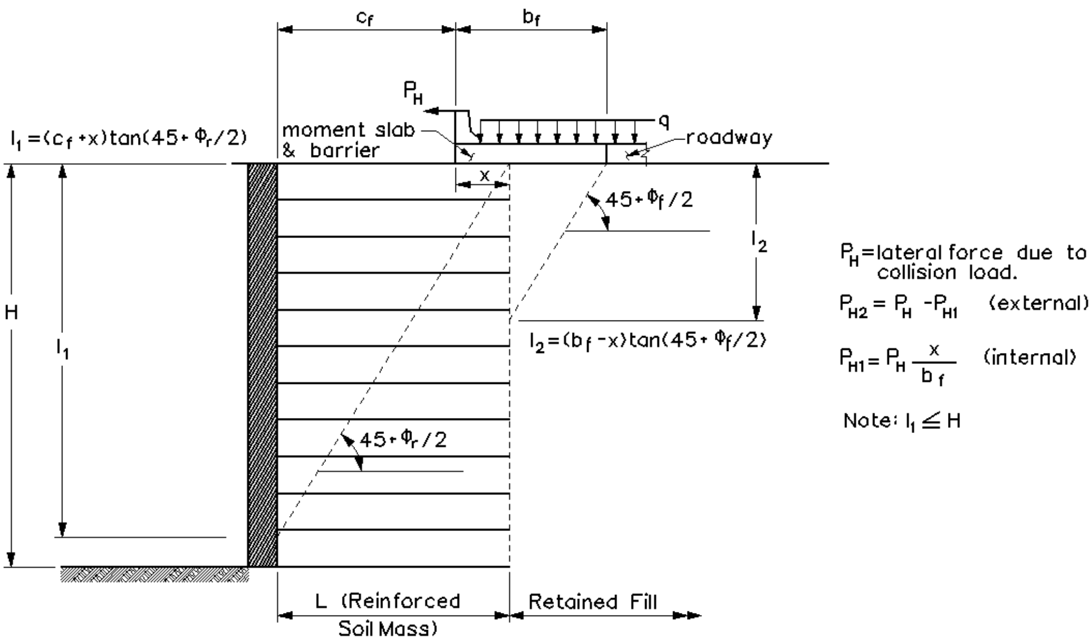


Figure 13. Distribution of Stress from Concentrated Horizontal Loads for combined External and Internal Stability Calculations

Reinforcement behind a stub abutment must be analyzed for internal stability. The analysis is based upon direct loading of the straps in the horizontal direction. σ_H is the result of horizontal loads from the earth load and surcharges behind the abutment together with the superstructure concentrated horizontal beam seat load P_{H1a} (see figure 14).

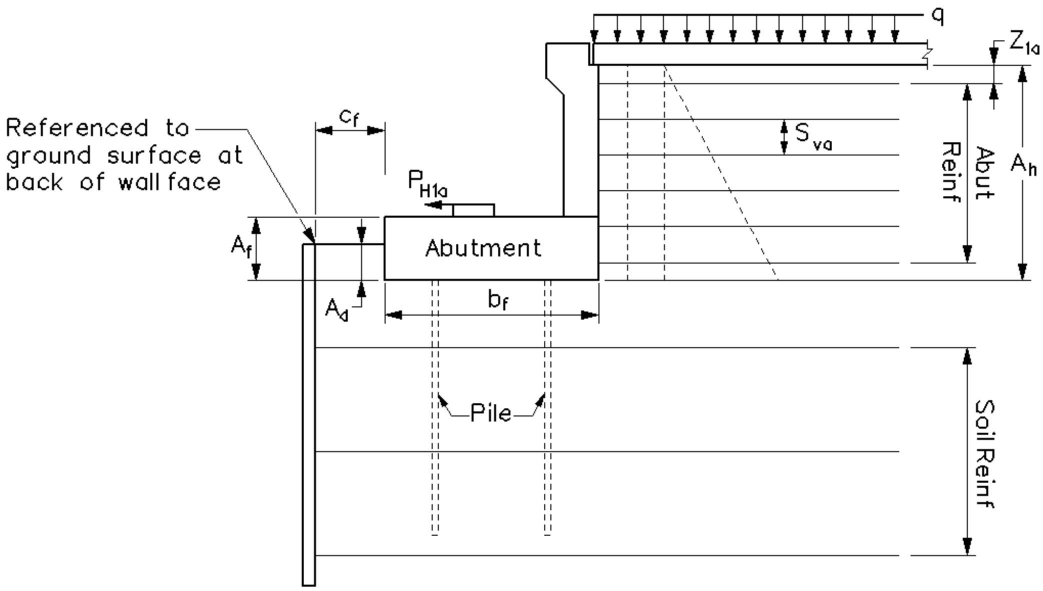


Figure 14– Loads behind the abutment supported on piles (See Figure 4) for internal stability calculations

Where:

P_{H1a}	= Longitudinal superstructure load on the abutment
A_d	= Distance from bottom of abutment to top of wall (Figure 14)
A_f	= Footing thickness (Figure 14)
A_h	= Height of backfill behind the abutment (Figure 14)
S_{va}	= Vertical spacing the reinforcement layers (Figure 14)
Z_{1a}	= Depth of Layer 1 below roadway pavement (Figure 14)

The condition for Abutment with no backwall the lateral forces above the abutment footing are assumed to have no effect on the abutment footing.

2. Sloping Backslope Condition (see Figure 15):

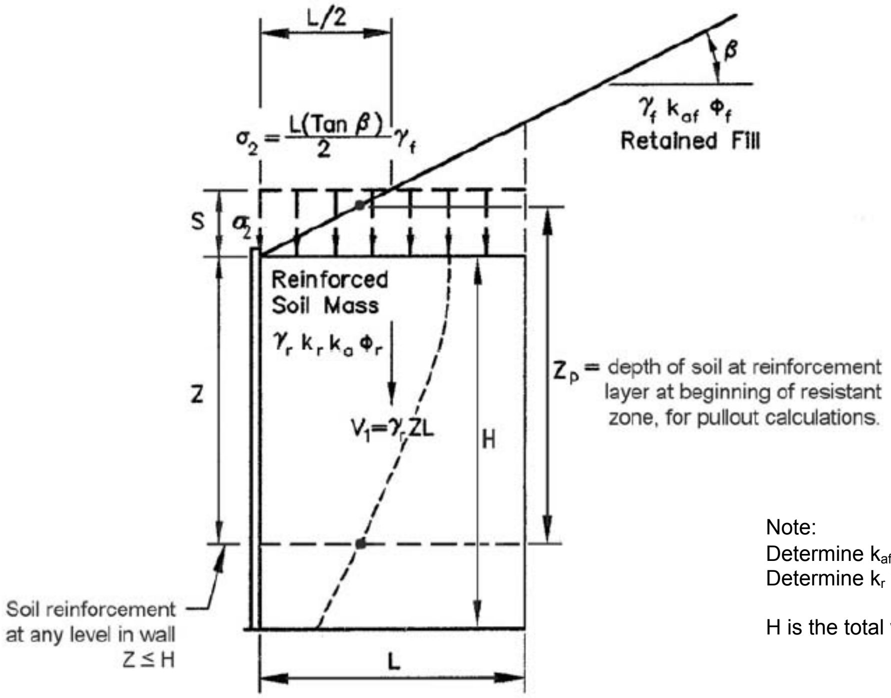
For Max Stress: $S = (1/2)L \tan \beta$

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

For Pullout:

$$\sigma_v = \gamma_r Z_p \qquad \text{when} \quad Z_p \geq Z + S$$

$$\sigma_v = \gamma_r Z + (1/2)L(\tan \beta)\gamma_f \qquad \text{when} \quad Z_p < Z + S$$



Note:
Determine k_{af} using the slope angle β.
Determine k_r from Figure 10.

H is the total wall height at the face.

Figure 15. AASHTO Figure 11.10.6.2.1-2 – Calculation of Vertical Stress for Sloping Backslope Condition

where:

Z = depth below top of wall to reinforcement layer

Z_p = depth of soil at reinforcement layer at beginning of resistance zone for pullout calculations

γ_r = unit weight of reinforced fill

γ_f = unit weight of backfill

4.1.2 Maximum Reinforcement Loads at the Connection to Wall Face (A11.10.6.2.2):

Factored tensile load (T_o) applied to the connection shall equal T_{max} as defined in Section 4.1.1.

4.1.3 Horizontal (CT) Collision Loads (D11.10.10.2)

D11.10.10.2

The soil reinforcement at the upper layer(s) shall be designed to have sufficient tensile and pullout capacity to resist the horizontal parapet collision load, applied per Figure 11 (assuming the horizontal load P_{H1} represents a vehicular collision (CT) load). The footing depicted on the retained fill portion shall represent the parapet to which CT is applied. The parapet weight will be assumed negligible and will not be considered for internal stability calculations.

where:

P_{H1} = assumed vehicular collision (impact) load (CT)

F_1 = lateral force due to earth pressure

F_2 = lateral force due to traffic surcharge, as calculated above in Section 3.1.3

$\Delta\sigma_H$ = horizontal stress due to surcharge load, as defined in Figure 11

c_f = distance from back of wall to the back face of the parapet

When CT Loads are applied (EXT II Limit State), l_1 from Figure 11 will be computed as:

$$l_1 = (c_f + b_f)\tan(45+\phi_r/2)$$

The full length of the reinforcement will be considered effective in resisting the impact horizontal load.

4.2 REINFORCEMENT PULLOUT (A11.10.6.3):

The following pullout calculations assume the factored long-term strength of the reinforcement in the resistant zone is greater than T_{max} per A11.10.6.3.2., except in checking CT loads the top layer/s as specified in Section 4.1.3 above where full length of reinforcement will be considered effective in resisting the impact load.

Reinforcement pullout will be checked at each level for pullout failure. In the case of an MSE wall with Abutment, the reinforcement pull-out is checked for the construction stage with the dead load of the uncured abutment footing applied prior to backfilling for the superstructure (no DW, no LS). However, a 2 feet of additional earth surcharge has been considered for temporary conditions.

Reinforcement pullout is based on the following:

$$L_R = L_a + L_e$$

where:

IF $Z + (H_1 - H) > \frac{H_1}{2}$, $\quad L_a = 0.6H_1 - \left(\frac{Z + (H_1 - H)}{H_1}\right)(0.6H_1)$

ELSE $L_a = 0.3H_1$

and:

$$L_e \geq \frac{T_{\max}}{\varphi F^* \alpha \sigma_v C R_c} \geq 3.0\text{ft} \qquad \text{(A11.10.6.3.2-1)}$$

where:

L_R = total length of reinforcement required

L_a = length of reinforcement in the active zone

L_e = length of reinforcement in the resisting zone, or length of overlap of geosynthetic sheet when used as a facing (4.3.2.2.2)

H & H_1 are defined in Figure 17

Z = depth below top of wall to reinforcement layer

T_{max} = applied factored load in the reinforcement from Section 4.1.1

φ = pullout resistance factor specified in AASHTO Table A11.5.6-1

σ_v = unfactored vertical stress at the reinforcement level in the resistant zone, with no live load surcharge

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

R_c = reinforcement coverage ratio as determined in Section 4.3, Figures 18 and 19 for steel and geosynthetic reinforcement, respectively.

Friction pullout factor (F*) and scale correction factor (α) should be determined from product-specific pullout tests in the project backfill material or equivalent soil, or they can be estimated empirically/theoretically based on the following:

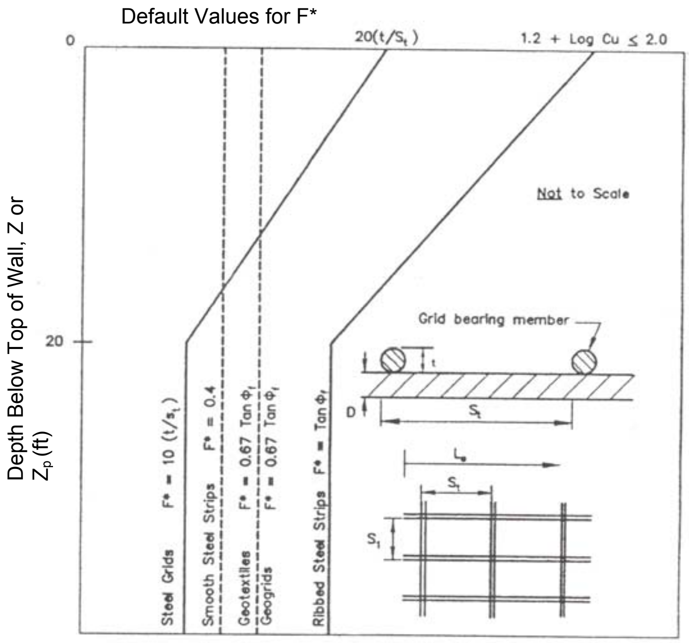
For standard backfill materials as defined in AASHTO LRFD Bridge Construction Specifications, Article 7.3.6.3, with the exception of uniform sands ($C_u < 4$), it is acceptable to use conservative default values for α and F* as shown in Table 4 and Figure 16, respectively.

α = scale effect correction factor

Table 4. AASHTO Table 11.10.6.3.2-1 Default Values for the Scale Effect Correction Factor, α

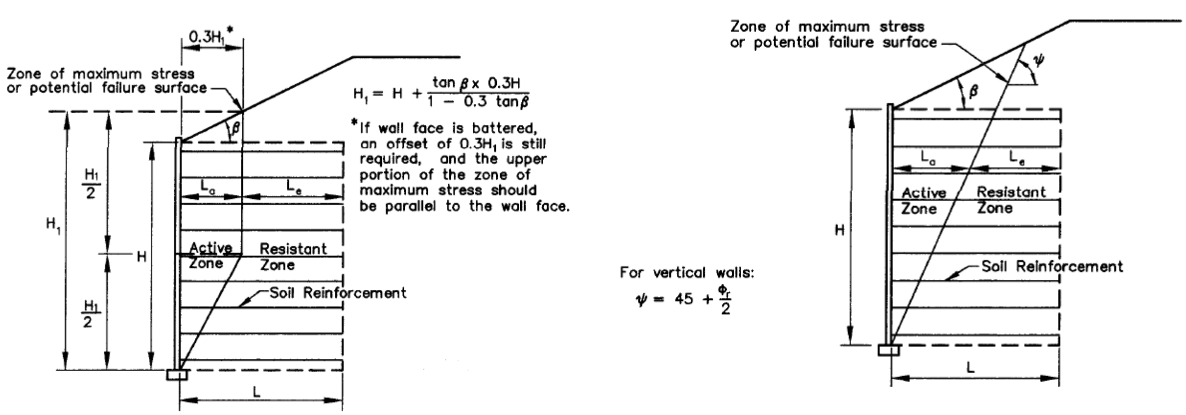
Reinforcement Type	Default Value for α
All Steel Reinforcements	1.0
Geogrids	0.8
Geotextiles	0.6

F* = pullout friction factor



Note: tan Φ_f should be tan Φ_r in this figure.

Figure 16. AASHTO Figure 11.10.6.3.2-1
Default Values for the Pullout Friction Factor



a) Inextensible Reinforcements

b) Extensible Reinforcements

Figure 17. AASHTO Figure 11.10.6.3.1-1 – Location of Potential Failure Surface for Internal Stability Design for MSE Walls

4.3 REINFORCEMENT STRENGTH (A11.10.6.4):

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) (see Figure 17) and at the connection of the reinforcement to the wall face, for applicable strength limit states as follows:

At the zone of maximum stress:

$$T_{\max} \leq \varphi\, T_{al} R_c \qquad \textit{(A11.10.6.4.1-1)}$$

where:

T_{max} = applied factored load in the reinforcement from Section 4.1.1

φ = reinf. tensile resistance factor specified in AASHTO Table 11.5.6-1

T_{al} = nominal long-term reinforcement design strength (for steel see Section 4.3.1.1, and 4.3.1.2 for geosynthetics)

R_c = reinforcement coverage ratio (for steel use Figure 16, for geosynthetics use Figure 19)

At the connection with the wall face:

$$T_o = T_{\max} \leq \varphi\, T_{ac} R_c \qquad \textit{(A11.10.6.4.1-2)}$$

where:

T_o = applied factored load at the reinforcement/wall facing connection = T_{max}, per A11.10.6.2.2

φ = reinf. tensile resistance factor specified in AASHTO Table 11.5.6-1

T_{ac} = nominal long-term reinforcement/wall facing connection design strength

R_c = reinforcement coverage ratio (for steel use Figure 18, for geosynthetics use Figure 19)

The nominal long-term reinforcement design strength (T_{al}), reinforcement/wall facing connection design strength (T_{ac}) and reinforcement coverage ratio (R_c) are dependent upon the type of reinforcing material being used, i.e., steel or geosynthetic (see Sections 4.3.1. and 4.3.2, respectively).

For aid in determining Ec and RF in Section 4.3.1 Steel Reinforcement, and 4.3.2 Geosynthetic Reinforcement, consult AASHTO 11.10.6.4.2 for factors and circumstances causing strength degradation due to corrosion and environmental factors.

4.3.1 Steel Reinforcement:

4.3.1.1 Design Tensile Resistance (A11.10.6.4.3a)

$$T_{al} = \frac{A_c F_y}{b} \qquad \text{(A11.10.6.4.3a-1)}$$

where:

F_y = minimum yield strength of steel

A_c = area of reinforcement corrected for corrosion loss, per Figure 18 below

b = unit width of reinforcement, per Figure 18 below

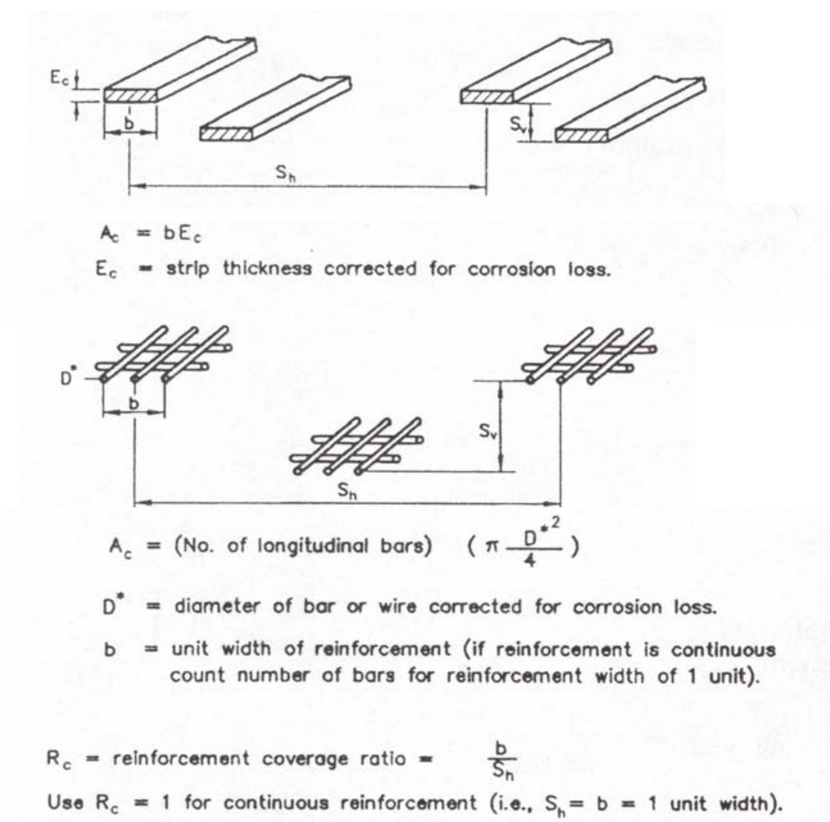


Figure 18. AASHTO Figure 11.10.6.4.1-1 – Reinforcement Coverage Ratio for Metal Reinforcement

4.3.1.2 Reinforcement/Facing Connection Design (A11.10.6.4.4a)

MSE wall system components are proprietary and therefore are engineered prior to use. Consequentially, the connections between the steel reinforcement / facing units (precast panels or modular block), bond length and bearing of elements embedded in the facing unit, and the capacity of the embedded connector are already designed/selected by manufacturers approved by the Department.

When selecting a facing unit from a Department-approved manufacturer/supplier, the facing units' factored connection capacity CC(STR) should be selected based on the following:

$$CC(STR) > T_o = T_{\max}$$

where T_{max} is the applied factored load calculated per Section 4.1.1

In the absence of a value for connection strength CC(STR) during design engineering phase, the reported value of T_{max} may be used in the design of the connection by the supplier.

4.3.2 Geosynthetic Reinforcement:

4.3.2.1 Design Tensile Resistance (A11.10.6.4.3b):

$$T_{al} = \frac{T_{ult}}{RF} \qquad \textit{(A11.10.6.4.3b-1)}$$

where:

$RF = RF_{ID} \; x \; RF_{CR} \; x \; RF_D$ (see A11.10.6.4.3b)

T_{ult} = minimum average roll value (MARV) ultimate tensile strength

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

RF_{ID} = strength reduction factor to account for installation damage to the reinforcement (shall not be less than 1.1)

RF_{CR} = strength reduction factor to prevent long-term creep rupture of reinforcement

RF_D = strength reduction factor to prevent rupture of reinforcement due to chemical and biological degradation (shall not be less than 1.1)

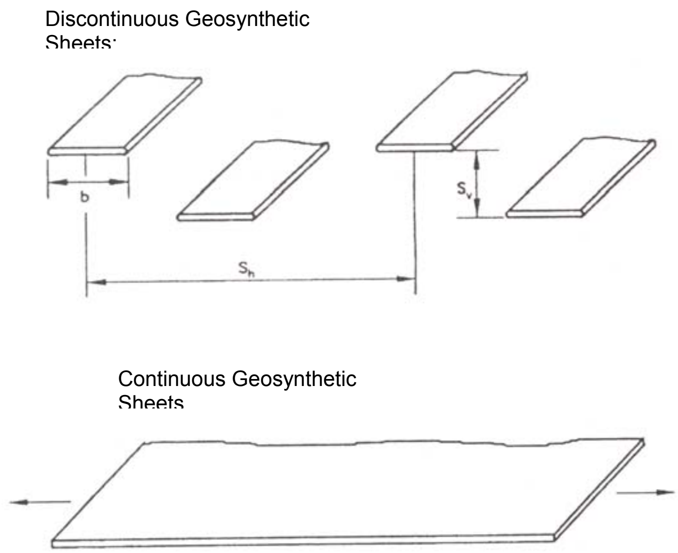
Geosynthetic reinforcement must also be able to resist the static and transient (impact) components in Extreme Event Limit State II for the parapet collision load, CT as follows:

For the static component, see Section 4.4.3.2.1, Equation A11.10.7.2-3.

For the transient components:

$$\Delta\sigma_h S_v \le \frac{\varphi S_{rt} \; R_c}{RF_{ID} RF_D} \qquad \textit{(A11.10.10.2-1)}$$

$\Delta\sigma_h$ = traffic barrier impact stress applied over reinforcement tributary area, per Figure 11



R_c = Reinforcement coverage ratio = b / S_h

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

Figure 19. AASHTO Figure 11.10.6.4.1-2 – Reinforcement Coverage Ratio for Geosynthetic Reinforcement

4.3.2.2 Reinforcement/Facing Connection Design (A11.10.6.4.4b)

4.3.2.2.1 Concrete Facing

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

$$T_{ac} = \frac{T_{ult} CR_{cr}}{RF_D} \qquad \textit{(A11.10.6.4.4b-1)}$$

where:

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

CR_{cr} = long-term connection strength reduction factor to account for reduced ultimate strength resulting from connection (see AASHTO C11.10.6.4.4b)

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

4.3.2.2.2 Geotextile Wrap Facing

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, Equation A11.10.6.3.2-1 can be used to determine the minimum overlap length required, but in no case shall the overlap length be less than 3'.

$$L_{overlap} \geq \frac{T_o}{\varphi F^* \alpha \sigma_v C R_c} \geq 3.0\text{ft}$$

If tan ρ is determined experimentally based on soil to reinforcement contact, tan ρ is reduced by 30 percent for reinforcement-to-reinforcement contact, per the last paragraph of A11.10.6.4.4b. Therefore, F* = 0.7* tan ρ. In the absence of specific data, F* = 2/3 tan ϕ_r.

4.3.3 Redundancy Check (D11.10.6.4.5P)

To protect against catastrophic failure of each discrete facing panels, in the event of failure of one strip or one longitudinal bar per grid mesh, redundancy of the soil reinforcement is checked for pull-out and rupture strengths.

The load factors will be applied as follows:

EH: 1.1
EV: 1.0

This check is not applicable for continuous grid or geotextile sheet reinforcements. This check is not applicable also, for the abutment reinforcements.

T_{max} will be recalculated as follows:

$$T_{max} = 1.1\ \{1.0\ \sigma_v\ k_r\ + \Delta\sigma_H\}\ S_v$$

4.4 SEISMIC CONSIDERATIONS FOR INTERNAL STABILITY (A11.10.7.2):

4.4.1 Loading

Reinforcements shall be designed to withstand horizontal forces generated by the internal inertia force, P_i, and the static forces as follows:

$$P_i = W_a A_m$$

where A_m is the maximum wall acceleration coefficient as determined per Section 3.5.1, and W_a is the weight of the active zone defined in Figure 20.

This inertial force shall be distributed to the reinforcements on a load per unit width of wall basis as follows:

$$T_{md} = \gamma_{EQ} P_i \frac{L_{ei}}{\sum_{i=1}^{m} L_{ei}} \qquad \textit{(A11.10.7.2-1)}$$

where:

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

γ_{EQ} = load factor for EQ loads from Table 2

P_i = inertia force due to the weight of backfill within the active zone, i.e., the shaded area in Figure 20

L_{ei} = effective reinforcement length for layer "i"

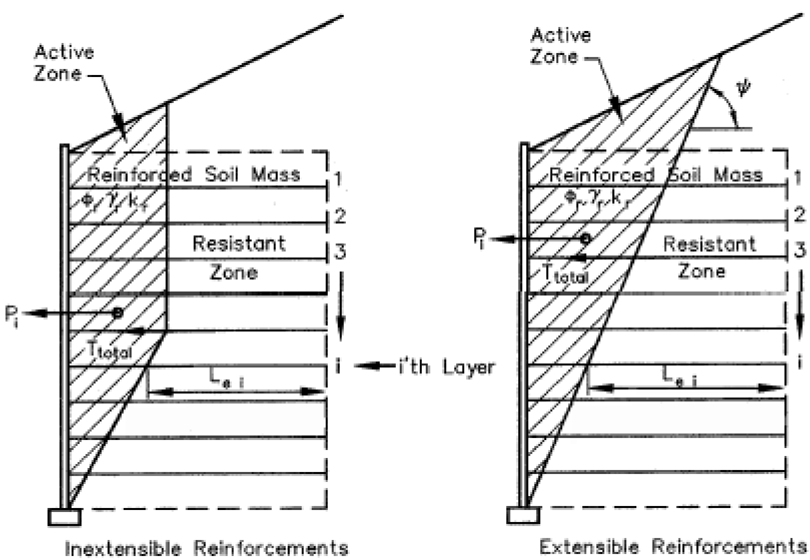


Figure 20. AASHTO Figure 11.10.7.2-1 Seismic Internal Stability of a MSE Wall

The total factored load applied to the reinforcement on a load per unit of wall width basis for the Extreme Event Limit State I will be T_{total} as follows:

$$T_{total} = T_{\max} + T_{md} \qquad \text{(A11.10.7.2-2)}$$

where T_{max} due to static forces is calculated in Section 4.1.1.

4.4.2 Reinforcement Pullout (A11.10.7.2):

For steel or geosynthetic reinforcement the length of the reinforcement in the resisting zone will be determined as follows:

$$L_e \geq \frac{T_{total}}{\varphi(0.8F^{*}\alpha\sigma_v CR_c)} \qquad \text{(A11.10.7.2-6)}$$

where F* is 80 percent of that specified in Section 4.2.

4.4.3 Reinforcement Strength (A11.10.7.2):

Reinforcement strength will be checked at Extreme Event Limit State I at every level, Z, within the wall, typical to Section 4.3, as follows:

4.4.3.1 Steel Reinforcement:

4.4.3.1.1 Design Tensile Resistance:

<u>At the zone of maximum stress</u> (per Section 4.3):

$$T_{total} \leq \varphi\, T_{al} R_c$$

where T_{total} is per Section 4.4.1, and T_{al} is per Section 4.3.1.1.

4.4.3.1.2 Reinforcement/Facing Connection Design (A11.10.6.4.4a):

Typical to Section 4.3.1.2 except that the facing unit's factored connection capacity, CC(EXT), should satisfy the following:

$$CC(EXT) > T_{total}$$

where T_{total} is the applied factored load calculated per Section 4.4.1.

In the absence of a value for connection strength CC(EXT) during design engineering phase, the reported value of T_{total} may be used in the design of the connection by the supplier.

4.4.3.2 Geosynthetic Reinforcement (A11.10.7.2):

4.4.3.2.1 Design Tensile Resistance:

Geosynthetic reinforcement will be designed to resist rupture for the static and dynamic components of T_{total}, as follows:

For the static component (T_{max}):

$$T_{\max} \le \frac{\varphi S_{rs} R_c}{RF} \qquad \text{(A11.10.7.2-3)}$$

For the dynamic component (T_{md}):

$$T_{md} \le \frac{\varphi S_{rt} R_c}{RF_{ID} RF_D} \qquad \text{(A11.10.7.2-4)}$$

where:

φ = reinforcement resistance factor for combined static/earthquake loading specified in AASHTO Table 11.5.6-1

R_c, RF, RF_{ID}, and RF_D are as defined in Section 4.3.2

S_{rs} and S_{rt} are the ultimate reinforcement tensile resistances required to resist the static and dynamic components of the total factored load T_{total} , respectively.

such that:

$$T_{ult} = S_{rs} + S_{rt} \qquad \text{(A11.10.7.2-5)}$$

4.4.3.2.2 Reinforcement/Facing Connection Design (A11.10.7.3):

4.4.3.2.2.1 Concrete Facing

In general, geosynthetic connections subjected to seismic loading must satisfy the following:

$$T_{total} = T_{\max} + T_{md} \le \varphi T_{ac}$$

For connections relying on friction:

For the static component (T_{max}):

$$T_{\max} \le \frac{0.8\,\varphi S_{rs}\, CR_{cr} R_c}{RF_D} \qquad \text{(A11.10.7.3-1)}$$

For the dynamic component (T_{md}):

$$T_{md} \leq \frac{0.8\,\varphi S_{rt}\;CR_u R_c}{RF_D} \qquad \text{(A11.10.7.3-2)}$$

where:

S_{rs}, S_{rt}, φ and RF_D are as specified in Section 4.4.3.2.1

CR_{cr} in Section 4.3.2.2

CR_u = short-term reduction factor to account for reduced ultimate strength resulting from connection (see AASHTO C11.10.6.4.4b)

For mechanical connections:

Remove the 0.8 multiplier from the previous equations specified for connections relying on friction.

4.4.3.2.2.2 Geotextile Wrap Facing

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, Equation A11.10.7.2-6 can be used to determine the minimum overlap length required, but in no case shall the overlap length be less than 3'.

$$L_{overlap} \geq \frac{T_o + T_{md}}{\varphi(0.8 * F^* \alpha \sigma_v CR_c)} \geq 3.0\text{ft}$$

If tan ρ is determined experimentally based on soil to reinforcement contact, tan ρ is reduced by 30 percent for reinforcement-to-reinforcement contact, per the last paragraph of A11.10.6.4.3b. Therefore, F* = 0.7* tan ρ. In the absence of specific data, ρ = 2/3 ϕ_r.

References:

1. AASHTO LRFD Bridge Design Specification, Fifth Edition, 2010
2. Design Manual Part 4, Pennsylvania. Department of Transportation, Publication 15m, May 2012 Edition
3. Load and Resistance Factor Design for Highway Bridge Substructures, Federal Highway Administration, Publication No. FHWA HI-98-032, July 1998
4. MSE Wall Design Spreadsheet, Tony Allen, Washington State Department of Transportation
5. The Reinforced Earth Company, www.recousa.com (photos on cover)
6. FD System International, Directed Fragility Systems, www.directedfragility.com (photos on cover)
7. www.geosource.com/rw/mse.htm
8. Mechanically Stabilized Earth Walls, Transportation Research Board Circular, Number 444, May 1995, ISSN 0097-8515
9. www.groupetai.com/products/trl.html (bottom photo on cover)

Spreadsheet Input	
Value	Units
A	dim
A_c*	in^2
A_d	ft
A_f	ft
A_h	ft
b	in
b_f	ft
c	tsf
c_f	ft
C	dim
C_o	tsf
C_u	dim
CC (STR)	tons/ft
CC (EXT)	tons/ft
CR_{cr}	dim
CT (pullout)	kips/ft
CT	kips/ft
d	ft
D	in
DC	k/ft^3
D_f	ft
DW	ksf
F_y	ksi
h_B	ft
h_{eq}	ft
h_{eq_t}	ft
H	ft
HC	dim
L	ft
L	ft
$L_{overlap}$	ft
Nb	dim
N_{ms}	dim
P_{H1a}	kips/ft
p_w	ft
RF_{ID}	dim
RF_{CR}	dim
RF_D	dim
S_h	ft
S_{rs}	t/ft
S_{rt}	t/ft
S_t	in
S_u	tsf
S_v	ft
S_{va}	ft
Tr	in

Spreadsheet Input	
Value	Units
T_w	ft
t	in
z_w	ft
Z_1	ft
Z_{1a}	ft
β	deg
ϕ_b	deg
ϕ_f	deg
ϕ_r	deg
γ'	pcf
γ_f	pcf
γ_m	pcf
γ_r	pcf
φ	dim
φ_b	dim
φ_s	dim
φ_τ	dim
φ_n	dim
θ	deg
ρ	deg

Calculated Values (Internally, or Explicit)	
Value	Units
A_c	in^2
A_m	dim
B	deg
B	ft
bfe	dim
bfi	dim
c_a	tsf
CR_u	dim
D*	in
e_B	ft
e_L	ft
e_{max}	ft
e_{maxR}	ft
e_{maxS}	ft
F*	dim
F_1	tons/ft
F_2	tons/ft
g	ft/s^2
h	ft
H_1	ft
H_2	ft
i_c	dim
i_q	dim
i_γ	dim
k	dim
k_a	dim
k_{af}	dim
k_r	dim
l_1	ft
l_2	ft
L'	ft
L_a	ft
L_e	ft
L_{ei}	ft
L_R	ft
L_{min}	ft
M_{az}	t /ft
M_{htot}	t-ft/ft
M_{vtot}	t-ft/ft
n	dim
N_c	dim
N_q	dim
N_γ	dim
Δp	tsf
P_a	tons/ft

Calculated Values (Internally, or Explicit)	
Value	Units
P_i	tons/ft
P_{ir}	tons/ft
P_{is}	tons/ft
P_{AE}	tons/ft
P_{H1}	tons/ft
P_{H2}	tons/ft
P_{IR}	tons/ft
$q_{\text{surcharge}}$	tsf
$q_{\text{bearing press}}$	tsf
q_R	tsf
q_s	tsf
q_{ult}	tsf
Q_{ep}	tons/ft
Q_R	tons/ft
Q_τ	tons/ft
R_c	dim
RF	dim
S	ft
S_c	dim
S_q	dim
S_γ	dim
T_{ac}	tons/ft
T_{al}	tons/ft
T_o	tons/ft
T_{md}	tons/ft
T_{max}	tons/ft
T_{total}	tons/ft
T_{ult}	tons/ft
V_{tot}	tons/ft
x	dim
Z	ft
Z_p	ft
α	dim
ϕ_{fw}	deg
γ_{CT}	dim
γ_{DC}	dim
γ_{EH}	dim
γ_{EQ}	dim
γ_{ES}	dim
γ_{EV}	dim
γ_{LS}	dim
η_i	dim
γ_p	dim
φ_{ep}	dim
π	dim

Calculated Values (Internally, or Explicit)	
Value	Units
θ	deg
σ_H	tsf
$\Delta\sigma_H$	tsf
σ_V	tsf
$\Delta\sigma_V$	tsf
σ'_V	tsf
Ψ_r	deg

Notation

A_s = peak seismic ground acceleration coefficient modified by short period site factor per A3.10.4 (3.5.1)
A_c = area of reinforcement corrected for corrosion loss (4.3.1.1)
A_c* = area of reinforcement as determined by corrosion specialist (for aggressive soils only)
Ad = Distance from bottom of abutment to top of wall (4.1.1)
A_f = Footing thickness for stub abutment (4.1.1)
A_h = Height of backfill behind stub abutment (not including roadway)
A_m = maximum wall acceleration coefficient at the centroid of the wall (3.5.1, 4.4.1)
b = unit width of reinforcement (4.3.1.1)
b_f = abutment/slab width above or behind soil reinforcement (3.1)
B = notional slope of backfill behind wall (3.1.1)
B' = effective footing width for load eccentric (short side), as specified in A10.6.1.3 (3.2, 3.3)
B = width of footing (3.3, Fig 7)
c = soil or rock cohesion (3.3.1)
c_a = adhesion between footing and soil (3.2)
c_f = distance from back of wall to the front edge of the abutment (or back face of barrier) (3.1.4)
C = overall reinforcement surface area geometry factor (4.2)
C_o = laboratory tested compressive strength of rock sample (3.3.2)
C_u = coefficient of uniformity for ribbed steel strips (A11.10.6.3.2) (4.2)
CC (STR) = facing unit's factored STR connection capacity as supplied by manufacturer (4.3.1.2, 4.4.3.1.2)
CC (EXT) = facing unit's factored EXT connection capacity as supplied by manufacturer (4.3.1.2, 4.4.3.1.2)
CR_{cr} = long-term correction strength reduction factor to account for reduced ultimate strength resulting from connection (4.3.2.2)
CR_u = short-term reduction factor to account for reduced ultimate strength resulting from connection (4.4.3.2.2)
CT = Collision loads specified in DM4 Section 11.10.10.2
d = distance from back of MSE wall to linear vertical load Pv (A3.11.6.3-1)
D = diameter of bar or wire prior to corrosion loss (Fig 16)
DC = Self-weight of the wall, and traffic barriers when applicable
DW = Roadway surface surcharge.
D* = diameter of bar or wire corrected for corrosion loss (Fig 18)
D_f = depth of base of footing as depicted in (3.3, Fig 7)
e_B = eccentricity of load in the B direction measured from centroid of footing (3.3, 3.4)
e_L = eccentricity of load in the L direction measured from centroid of footing (3.3)
e_{max} = maximum allowable eccentricity of resultant reaction (general) (3.4)
e_{maxR} = maximum allowable eccentricity of resultant reaction on rock (3.4)
e_{maxS} = maximum allowable eccentricity of resultant reaction on soil (3.4)
E_c = thickness of strip reinforcement corrected for section loss, used for determining A_c (4.3.1)
EH = horizontal earth pressure load
EQ = earthquake load
ES = earth surcharge load
EV = vertical pressure from dead load of earth fill
F* = pullout friction factor (4.2)
F_1 = lateral force due to earth pressure (4.1.3)
F_2 = lateral force due to traffic surcharge (3.1.3, 3.1.4, Fig 5 & 6)
F_y = minimum yield strength of steel (4.3.1.1)
F_p = total horizontal surcharge load from vertical surcharge due to wet concrete footing (3.1.2)
h = notional height of earth pressure diagram (3.1.1, Fig 1, 2, 3)
h_{eq} = equivalent height of soil for vehicular load (DM4 Table 3.11.6.2-2) (3.1.3)

h_{eq_t} = equivalent height of soil for temporary construction load (DM4 Table 3.11.6.4-2) (3.1.3)
H = height of MSE wall from top of footing (3.1.3)
H_1 = height from top of MSE wall footing to point of intersection of zone of maximum stress with sloping backfill (4.2, Fig 17)
H_2 = height of effective mass for external seismic stability calculations (3.5.2, Fig 9)
i_c = inclination factor for inclined loading (A10.6.3.1.2a) (3.3)
i_q = inclination factor for inclined loading (A10.6.3.1.2a) (3.3)
i_γ = inclination factor for inclined loading (A10.6.3.1.2a) (3.3)
k = coefficient of lateral earth pressure (general) (3.1.3)
k_a = active earth pressure coefficient (3.1.1)
k_r = horizontal pressure coefficient (4.1.1, Fig 10)
k_{af} = active earth pressure coefficient of backfill (3.1.3, Fig 5)
l_1 = depth from top of wall to point of intersection of bearing pressure from above (4.1.3, Fig 11)
l_2 = depth from top of wall to point of intersection of bearing pressure from above (3.1.4, Fig 6)
L = length of reinforced soil mass (Fig 6)
L = length of wall (3.3)
L' = effective footing length for load eccentric (long side), as specified in A10.6.1.3 (3.3)
L_a = length of reinforcement in the active zone (4.2)
L_e = length of reinforcement in the resistant zone, or length of geosynthetic overlap (4.2, 4.4.2)
L_{ei} = length reinforcement in the resistant zone, i.e., L_e at the i^{th} layer (4.4.1)
$L_{overlap}$ = overlap length of geosynthetic wrap wall face (4.3.2.2.2)
L_R = total length of soil reinforcement required (4.2)
L_{min} = minimum length of soil reinforcement (2.1)
LS = live load surcharge load
M_{htot} = Total factored overturning moment caused by horizontal loads per unit width (3.4)
M_{vtot} = Total factored overturning moment caused by vertical loads per unit width (3.4)
n = exponential factor relating B/L or L/B ratios for inclined loading (A10.6.3.1.2a) (3.3)
N_c = bearing capacity factor (DM4 C10.6.3.1.2a) (3.3)
N_q = bearing capacity factor (DM4 C10.6.3.1.2a) (3.3)
N_{ms} = coefficient factor to estimate ultimate bearing resistance of rock (DM4 Table D10.6.3.2.2-1P) (3.3.2)
N_γ = bearing capacity factor (DM4 C10.6.3.1.2a) (3.3)
Δp = constant horizontal earth pressure due to live load surcharge (3.1.3)
P_a = force resultant of earth pressure on wall, per unit width of wall (3.1.1)
P_{ah}, P_h = horizontal component of P_a earth pressure force (3.1.1)
P_{av}, P_v = vertical component of P_a earth pressure force (3.1.1)
P_i = internal inertia force due to weight of backfill with in the active zone (4.4.1)
P_{ir} = inertial force caused by acceleration of the reinforced backfill (3.5.2, Fig 9)
P_{is} = inertial force caused by acceleration of the sloping surcharge (3.5.2, Fig 9)
P_{AE} = dynamic horizontal thrust (3.5.1)
P_{IR} = horizontal inertial force (3.5.1)
P_{H1} = lateral parapet vehicular collision load (4.1.3, Fig 11)
P_{H1a} = lateral load applied to stub abutment associated with the superstructure (4.1.1, Fig 14)
P_{H2} = lateral parapet vehicular collision load (3.1.4, Fig 6)
q = surcharge pressure (3.1.3, Fig 5)
q = factored bearing pressure (3.3)
q_n = nominal bearing resistance of rock, = $N_{ms}C_o$ (3.3.2)
q_n = nominal bearing resistance of soil (3.3.1)
q_R = factored bearing resistance of soil or rock (3.3.1)
q_{ult} = modified form of bearing capacity equation, q_n, to account for the effects of footing shape, ground surface slope, and inclined loading (3.3) (D10.6.3.1.2a-10P)
R_R = factored resistance against failure by sliding (3.2)
R_τ = nominal shear resistance between soil and foundation (3.2)
R_c = reinforcement coverage ratio (4.3)

RF	=combined strength reduction factor to account for potential long-term degradation due to installation damage, creep and chemical aging (4.3.2.1)
RF_{ID}	= strength reduction factor to account for installation damage to the reinforcement (4.3.2.1)
RF_{CR}	= strength reduction factor to prevent long-term creep rupture of the reinforcement (4.3.2.1)
RF_D	= strength reduction factor to prevent rupture of reinforcement due to chemical and biological degradation (4.3.2.1, 4.3.2.2)
S	= max stress for sloping backslope (4.1.1, Fig 15)
s_c	= footing shape factor (Table A10.6.3.1.2a-3) (3.3)
S_h	= horizontal spacing of soil reinforcement (Fig 18 & 19)
s_q	= footing shape factor (Table A10.6.3.1.2a-3) (3.3)
S_v	= vertical spacing of soil reinforcement (4.1.1, 4.3.2.1, Fig 18 & 19)
S_{va}	= vertical spacing of soil reinforcement behind stub abutment(4.1.1, 4.3.2.1, Fig 14)
S_t	= spacing of transverse reinforcement in Figure 16 (4.2)
s_γ	= footing shape factor (Table A10.6.3.1.2a-3) (3.3)
S_{rs}	= ultimate reinforcement tensile resistance required to resist the static component of T_{total} (4.4.3.2.1)
S_{rt}	= ultimate reinforcement tensile resistance required to resist the dynamic component of T_{total} (4.3.2.1, 4.4.3.2.1)
t	= diameter of transverse reinforcement in Figure 16 (4.2)
Tr	= Total thickness of reinforcing strips
T_{ac}	= nominal long-term reinforcement/wall facing connection design strength (4.3.2.2)
T_{al}	= nominal long-term reinforcement design strength (4.3.1.1, 4.3.2.1)
T_o	= factored tensile load (4.1.2)
T_{md}	= factored incremental dynamic inertia force at layer "i" (4.4.1, 4.4.3.2.1, 4.4.3.2.2)
T_{max}	= maximum factored reinforcement loads (4.1.1, 4.4.1, 4.4.3.2.1, 4.4.3.2.2)
T_{total}	= total factored load applied to the reinforcement for Extreme Event Limit State I (4.4.1, 4.4.2, 4.4.3)
T_{ult}	= minimum average roll value (MARV) ultimate tensile strength (4.3.2.1, 4.3.2.2), ultimate reinforcement tensile resistance required to resist the static and dynamic components of T_{total} (4.4.3.2.1)
T_{wall}	= Thickness of wall facing elements.
V	= total vertical force per unit width (3.2)
W_a	= weight of active zone for seismic loads (4.4.1)
x	= portion of bf over reinforced fill zone (4.1.1, Fig 13)
z_w	= depth from base of footing to highest anticipated groundwater level (3.3, Fig 7)
Z	= depth below top of wall to a reinforcement layer (4.1.1)
Z_{1a}	= depth of layer 1 below roadway pavement (4.1.1)
Z_p	= depth of soil at the reinforcement layer at beginning of resistance zone for pullout calculations (4.1.1)
α	= scale correction factor (4.2)
β	= slope of backfill surface behind MSE wall (3.1.1)
γ	= total unit weight of bearing soil or rock (3.3)
γ'	= saturated unit weight of bearing soil or rock (3.3)
γ_f	= unit weight of retained backfill (Figure 5 (3.1.3), et al.) (4.1.1)
γ_m	= moist unit weight of bearing soil (3.3)
γ_p	= load factor for EV (Table 2, 1.3, 4.1.1)
γ_r	= unit weight of reinforced fill (4.1.1)
γ_s	= (same as γ_f) unit weight of backfill/soil (3.1.1); unit weight of soil used LL surcharge (3.1.3)
γ_{CT}	= load factor for CT specified in Table 2 (1.3)
γ_{DC}	= load factor for DC specified in Table 2 (1.3)

γ_{DWh} = load factor for horizontal component of DW specified in Table 2 (1.3)
γ_{DWv} = load factor for vertical component of DW specified in Table 2 (1.3)
γ_{EH} = load factor for EH specified in Table 2 (1.3)
γ_{EQ} = load factor for EQ specified in Table 2 (1.3)
γ_{ESh} = load factor for horizontal component of ES specified in Table 2 (1.3)
γ_{ESv} = load factor for vertical component of ES specified in Table 2 (1.3)
γ_{EV} = load factor for EV specified in Table 2 (1.3)
γ_{LS} = load factor for LS specified in Table 2 (1.3)
η_i = load modifier (1.2)
φ = resistance factors specified in DM4 Table 10.5.5.2.2-1
φ_b = resistance factor for soil bearing specified in DM4 Table 10.5.5.2.2-1
φ_b = bearing capacity resistance factor for foundation on rock specified in DM4 Table D10.5.5.2.2-1 (3.3.2)
ϕ_b = internal friction angle of base soil (3.2)
φ_τ = resistance factor for sliding between soil and foundation specified in DM4 Table 10.5.5.2.2-1 (3.2)
ϕ_f = AASHTO internal friction angle of retained backfill (3.1). Also AASHTO friction angle for soil bearing (3.2)
ϕ_{fw} = angle of internal friction of weaker soil (3.2)
ϕ_r = internal friction angle of reinforced fill (3.2)
π = 3.14
σ_H = factored horizontal soil stress at the reinforcement (4.1.1)
$\Delta\sigma_H$ = horizontal stress due to surcharge load (3.1.4, Figure 6)
$\Delta\sigma_H$ = horizontal stress at the reinforcement level resulting from a concentrated horizontal surcharge load (4.1.1, 4.3.2.1, Figure 6, Figure 11)
σ_v = pressure due to resultant of gravity forces from soil self-weight (4.1.1)
σ_v = unfactored vertical stress at the reinforcement level in the resistant zone (4.2)
$\Delta\sigma_v$ = vertical stress at the reinforcement resulting from ES load (3.1)

Example Problem Verification Matrix

		Problem Number				
	Problem Components	**#1**	**#2**	**#3**	**#4**	**#5**
Wall Facing Systems	Precast Concrete Panels	✓	✓			
	Modular Block			✓		
	Welded/Twisted Wire				✓	
	Geotextile Wrap					✓
Soil Reinforcement	Metal Strip		✓			
	Steel Bar Grid Mat	✓		✓		
	Welded Wire				✓	
	Geosynthetics					✓
Backfill Conditions	Level Backfill	✓		✓		✓
	Sloping Backfill				✓	
	Broken Backfill		✓			
Units	U.S. Customary	✓	✓		✓	
	Metric			✓		✓
Stability	Internal	✓	✓	✓	✓	✓
	External	✓	✓	✓	✓	✓
Loading	EV	✓	✓	✓	✓	✓
	EH	✓	✓	✓	✓	✓
	LS		✓	✓		
	ES (Stub Abutment Load)	✓				
	CT		✓	✓		
	Seismic	✓	✓	✓	✓	✓
	$DC_{Traffic\ Barrier}$		✓	✓		
	$DC_{Wall\ Self\text{-}Weight}$	✓	✓	✓	✓	✓
	$DW_{Roadway\ Surface}$		✓	✓		