ASSESSMENT AND RATING OF MECHANICALLY STABILIZED EARTH WALLS

by

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To my loving Wife, sons, and family.

Abstract

ASSESSMENT AND RATING OF MECHANICALLY

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Mechanically stabilized earth (MSE) walls have been increasingly utilized in the United States since 1972 and in Texas since 1979. MSE walls provide earth retention for commercial, industrial and transportation projects throughout Texas. Within the transportation industry, the traditional design life of MSE walls is 75 years, with new projects requiring 100 year design life. The millions of square feet of existing MSE walls are generally less than half of the anticipated design life, making identifying and assessing the existing wall inventory a significant asset management component for optimizing maintenance expenditures.

This thesis proposes and applies a system that assesses and rates MSE walls based on as-built design assessment, physical condition, safety impacts, and owner defined elements. The program developed in this thesis has been generally tailored to application in transportation infrastructure with MSE walls

having greater than two years of service, founded in and retaining both cohesive and cohesionless soils.

Table of Contents

LIST OF ILLUSTRATIONS

LIST OF TABLES

Chapter 1

Introduction

1.1 Background

Reinforcing soil, typically with a form of vegetation, is a technique that has been applied throughout human history ranging from using straw to reinforce mud bricks to structures such as the Great Wall of China which has performed for over a millennium. Modern application of reinforcing soil is referred to as mechanically stabilized earth (MSE) walls. Modern MSE wall methods and the use of MSE walls as earth retention structures is attributed Henri Vidal, a French Architect who first published his research in the early 1960's (Berg, Christopher, & Samtani, 2009). MSE wall construction began in the United States in 1972 and the first MSE wall in Texas was constructed in 1979 (Smith & Janacek, 2011). During the nearly forty years of service history, tens of thousands of structures have been constructed safely (Harpstead, Schmidt, & Christopher, 2010); however, MSE wall failures are becoming more common (Bachus & Griffin, 2010), and based on recent construction in North Texas, emergency repair or reconstruction of a wall can be over ten times the cost of a planned stabilization. This apparent trend is cause for concern given that the majority of walls have been in service less than half of the typical 75 to 100 year design life (Berg R. , 2010).

1.2 Problem Statement

MSE walls have been constructed in many countries, with many different methods and in many geologic formations. A general review of case histories of failures indicates a variety of reason for the failures and large variations of timeframe for failure. Many failures occur during or closely following construction; however, failures in stiff fissured clays have been observed up to fifty years after construction (Wright, Zornberg, & Aguettant, 2007). With the large quantities of MSE walls in service representing both a significant investment and potential liability, an asset management system provides a means to minimize the impact of both i) long term maintenance needed to achieve the structures design and ii) development of significant distress and/or failure of the structure. An asset management system typically includes an inventory of the asset, condition assessment of the inventory, and recommended actions (Anderson, Alzamora, & DeMarco, 2008). As described in Section 2.5, the author has observed that the proposed and in-place systems for retaining wall asset management focus on the physical condition of the structure. Physical condition is the primary means for evaluating changes in the wall system; however it inherently assumes that equal distress indicates equal risk. Properly integrating a reliability based as-built design assessment with the physical assessment will achieve a more accurate condition assessment.

1.3 Objectives

This thesis presents a methodology for condition assessment and rating of transportation related MSE walls (with inextensible reinforcement and concrete panel facing) by correlating primary external failure modes identified in reliability based as-built design assessment with physical distress features typical of those failure modes. This approach will provide a means for identifying the likely failure mode affecting the wall and the relative degree to which that failure is developing. Owners are anticipated to have a greater return on investment by allocating maintenance budget based on prioritizing walls demonstrating physical distress associated with the walls probable failure mode (potentially indicating significant serviceability issues or wall failure depending on the long term factor of safety for the probable failure mode) over walls with similar distress that have a higher factor of safety for the associated failure mode. Understanding how distress is associated with a failure mode can also provide targeted maintenance actions that will improve the walls condition not just aesthetically, but structurally.

Prioritization of wall maintenance will be achieved by a rating system that accounts for the walls physical and as-built design condition, observeability of distress development based on failure mode and/or physical monitoring, and consequences of deteriorated wall serviceability and/or wall failure. The rating system will also allow for additional owner specific factors to be introduced to further refine wall priority.

1.4 Organization and summary

Chapter 2 – Summary of findings from a review of historical and current literature on the design and construction of MSE walls, probability and variation as applied to soil parameters, asset management theory, and MSE wall failure modes.

Chapter 3 – Proposed condition assessment program detailing the physical assessment, as-built design analysis, and matrix for correlating the physical distress and probable failure mode(s).

Chapter 4 – Conclusions and recommendations.

Chapter 2

Literature Review

2.1 Introduction

Development of a methodology for assessing MSE walls requires research and understanding of the design of MSE walls, the materials of construction, construction tolerances, and behavior of soils for the in-service condition. The subsequent rating system development incorporated research of literature addressing reliability analysis and existing asset management programs. The following sections summarize information developed in the literature review.

2.2 Design

Figure 2-1 shows the generalized regions of an MSE wall system needed for design. A MSE wall is a flexible gravity wall system that uses its mass and interaction with the foundation soils to resist the lateral force imparted by the soils being retained. The design process associated with sizing the MSE mass for resisting the external force is referred to as external stability. Soil is mechanically stabilized by anchoring reinforcement in the reinforced soil that is connected to a facing system which supports the soil pressure from the active zone of the reinforced soil, subsequently creating a self-supporting soil mass. Design of the anchorage, facing and reinforced mass is known as internal stability. Global stability analysis is another critical design component that evaluates the stability of the entire embankment, MSE wall, and foundation soil system. Consistent with the focus of this thesis, the relatively large quantity of study that has already been performed concerning global stability, and the variety of proprietary considerations for internal stability, the following sections of literature review will focus on external stability design and influencers for MSE walls.

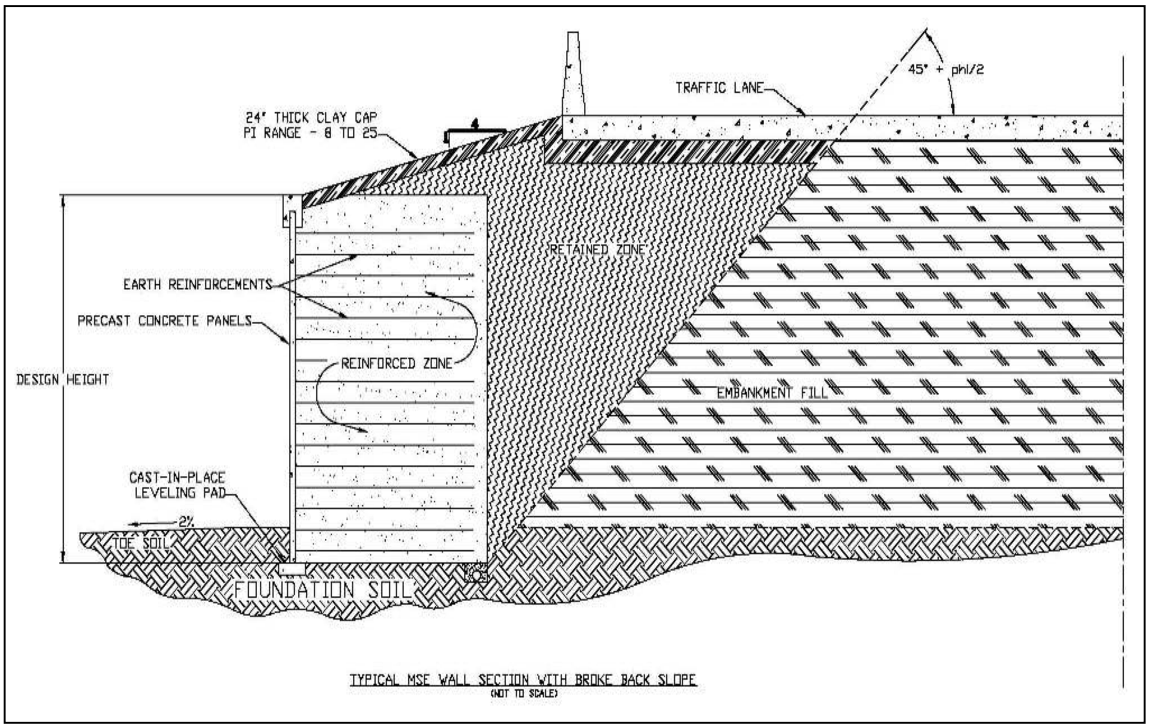


Figure 2-1: Generalized MSE Section

2.3 External Stability

Federal Highway Administration (FHWA) and National Highway Institute (NHI) publication number FHWA-NHI-00-043 (Elias, Christopher, & Berg, 2001) defines four external stability failure modes that are to be analyzed:

- Sliding – Soils retained by the MSE wall impart a lateral load on the wall system. Sliding is the lateral movement of a structure typically associated with movement along the interface of the base of the structure and the foundation soil. Deeper seated planes may occur; however they are typically identified in a block sliding/global stability analysis. Graphically depicted in Figure 2-2.a.
- Overturning and Eccentricity – Described in FHWA-NHI-00-043 as "Limiting the location of the resultant of all forces." Overturning can be generalized as the balance of forces that prevents the wall from rotating about its toe. Eccentricity is the location of the resultant force at the base of the wall when the forces are balanced to prevent overturning. Eccentricity has a substantial impact on the pressure distribution at the base of the wall. Graphically depicted in Figure 2-2.b.
- Bearing capacity – Vertical support of MSE walls by foundation soils. Bearing capacity is generally a function of the foundation

soil properties, the geometry of the MSE wall, and the applied vertical pressure (bearing pressure) at the base of the MSE wall. Graphically depicted in Figure 2-2.c.

- Deep seated stability – Failures associated with soils not in immediate contact with the wall. Also referred to as rotational and overall stability, this failure mode encompasses both the embankment beyond the soils causing lateral pressure and foundation soils beyond the limits evaluated in bearing capacity. Graphically depicted in Figure 2-2.d.

Generally two methods of analysis can be utilized to evaluate the external stability of a MSE wall. FHWA-NHI-00-043 uses an allowable stress design (ASD) methodology that compares the ultimate resistance of the wall system relative to the calculated loads on the system to determine the factor of safety (FS) for a given failure mode. For a FS=1, the system is in theoretical equilibrium, whereas an FS<1 indicates there is more load than there is resistance and that the system will fail. Minimum factors of safety are established for each failure mode. FHWA (Berg, Christopher, & Samtani, 2009) provides a load and resistance factor design (LRFD) method for MSE wall design. Generally, LRFD uses multipliers known as factors to modify the calculated loads and the wall system resistance based on load conditions and the confidence level of the calculated load or resistance.

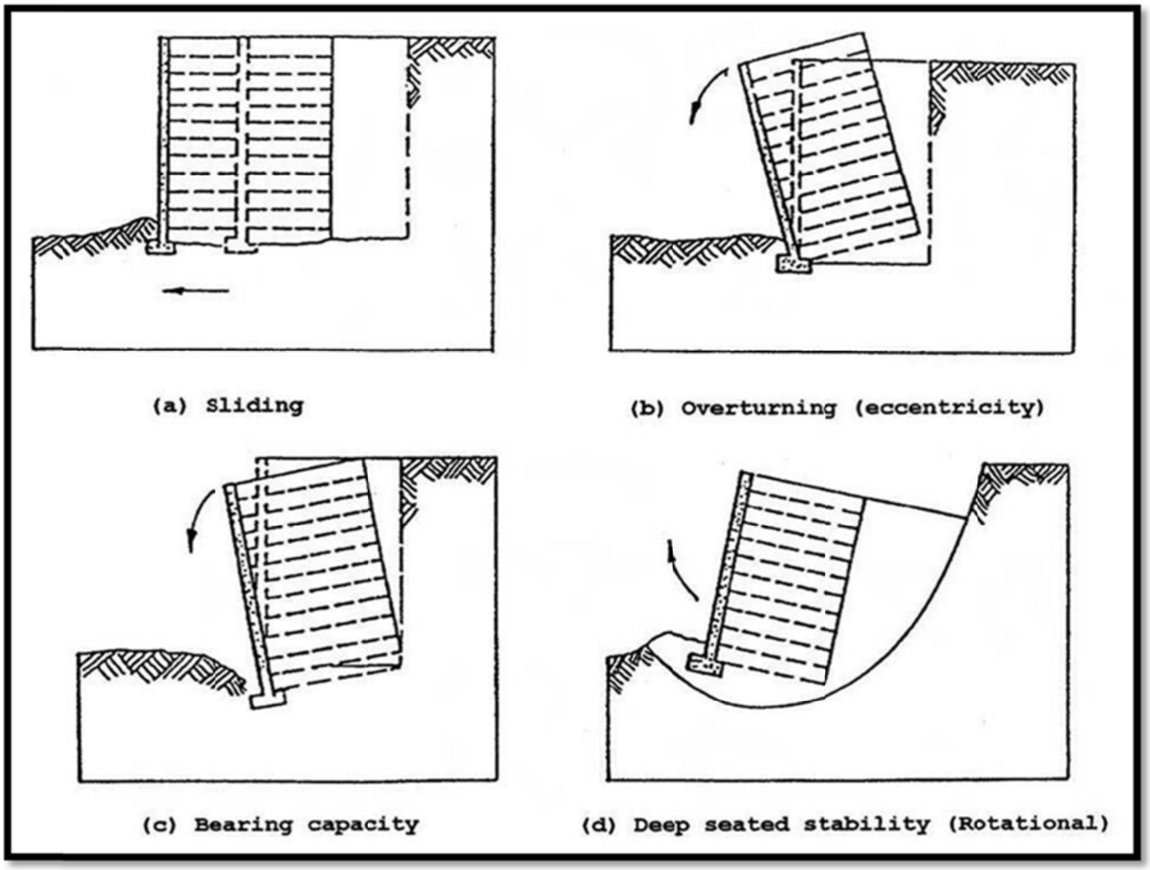


Figure 2-2: External Stability Failure Modes (Elias, Christopher, & Berg, 2001)

2.3.1 Loads for External Stability

Driving forces are calculated based on the soil properties and the geometry of the slope. Figure 2.3 provides a generalized summary of the components of force acting on an MSE wall system.

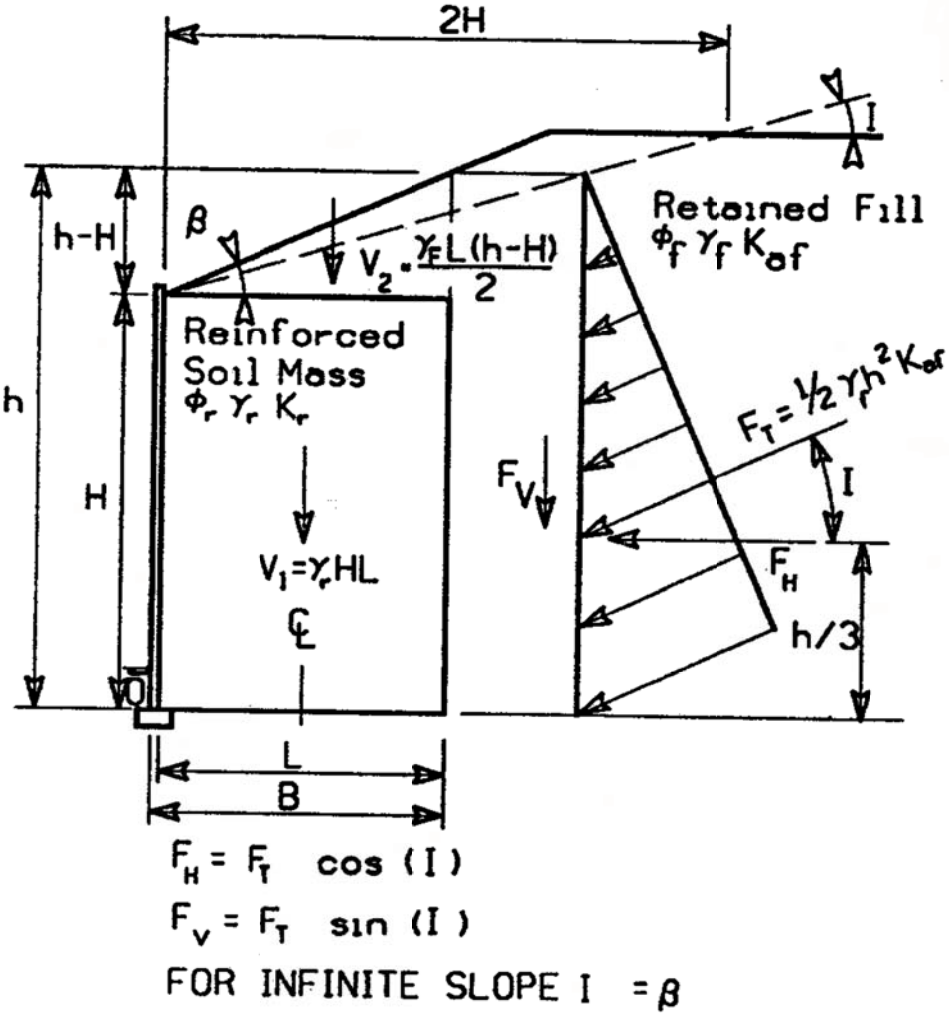


Figure 2-3: Diagram of External Stability Forces

The variables listed in Figure 2-3 are defined as:

- Geometric Variables:
  - H: Wall Height
  - h: Mechanical Wall Height
  - L: Reinforcement Length
  - B: Base Length (L + facing thickness)
  - β: Back Slope Angle
  - I: Equivalent Slope Angle for Broken Back Slopes
- Soil Property Variables:
  - γ_r: Reinforced Fill Unit Weight
  - ϕ_r: Reinforced Fill Internal Angle of Friction
  - k_r: Reinforced Fill Lateral Pressure Coefficient
  - γ_f: Retained Soil Unit Weight
  - ϕ_f: Retained Soil Internal Angle of Friction
  - c_f: Retained Soil cohesion (not shown)
  - k_{af}: Retained Soil Lateral Pressure Coefficient
  - γ_b: Foundation Soil Unit Weight (not shown)
  - ϕ_b: Foundation Soil Internal Angle of Friction (not shown)
  - c_b: Foundation Soil Cohesion (not shown)

2.3.1.1 Lateral Earth Pressure

Several lateral earth pressure theories and methodologies exist; however FHWA uses Rankine's equations for active earth pressure coefficients (Berg, Christopher, & Samtani, 2009). Active earth pressure occurs when a wall has leaned away from the retained soil to the point that a triangular wedge of the retained soil fails (Das, 2007). Soil wedge failure can be modeled using the Mohr's Circle of Stress as shown in Figure 2.4.

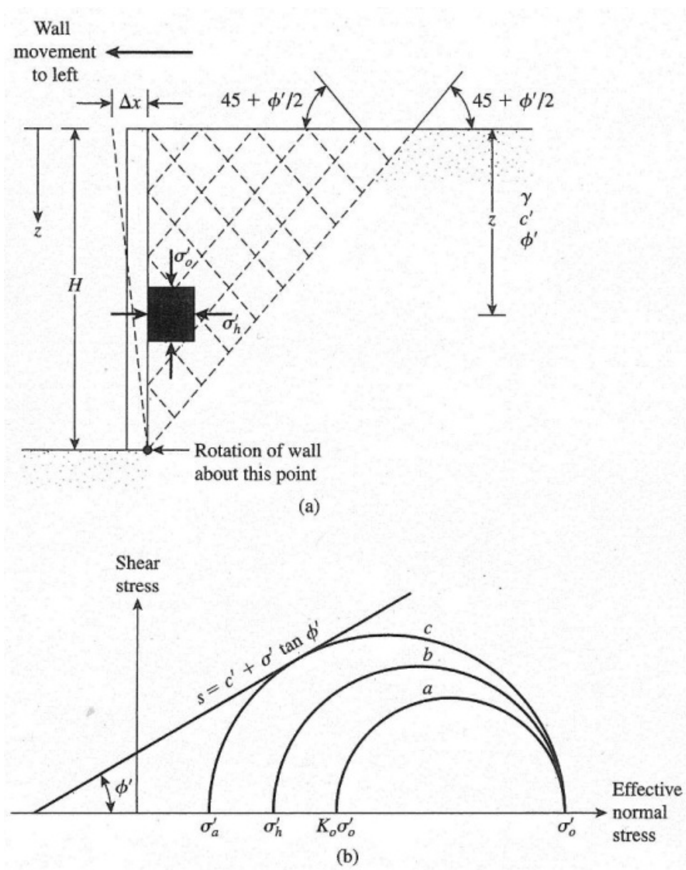


Figure 2-4: Rankine Active Pressure (Figure 7.5 from Das, 2007): (a) idealized wall deflection; (b) chane in stress state with wall deflection

The major principal stress σ'_o is equivalent to the effective overburden pressure of the soil. As the wall deflects, the confining pressure σ'_h is reduced from the initial point of equilibrium $k_o * \sigma'_o$ to the point when the Mohr's circle intersects with the Mohr-Coulomb failure line defined by equation 2.1.

$$s = c' + \sigma' * tan\,\phi' \qquad \text{Equation 2-1}$$

- s - shear strength of the soil
- c' - effective cohesion of the soil
- σ' - effective overburden stress
- ϕ' - effective internal angle of friction of the soil

The minor principal stress at the point of soil wedge failure, σ'_a, is known as the Rankine active pressure (Das, 2007). Das, 2007 derives the general equation for σ'_a, resulting in equation 2.2 for a vertical retaining wall with a frictionless interface on the back of the wall and a horizontal back slope (β=0).

$$\sigma'_a = \sigma'_o k_a - 2c'\sqrt{k_a} \qquad \text{Equation 2-2}$$

where k_a is calculated by:

$$k_a = tan^2(45 - \tfrac{\phi}{2}) \qquad \text{Equation 2-3}$$

Additional forms of the equation for k_a exist that account for changes in wall verticality, slope on the back face of the wall, cohesionless soils, and changes in angle β. For MSE walls, the most typically applied form assumes a vertical wall, vertical back face, frictionless back face, with cohesionless soils at a variable angle I (I= β for infinite slopes). FHWA 2000 calculates this form of k_a as shown in equation 2.4:

$$k_a = \frac{sin^2(\theta + \phi')}{sin^2\theta \sin(\theta - \delta)\left\{1 + \sqrt{\frac{\sin(\phi' + \delta)\sin(\phi' - I)}{\sin(\theta - \delta)\sin(\theta + I)}}\right\}^2} \qquad \text{Equation 2-4}$$

The equation for k_a is further modified when considering retained fill soils with both cohesion and internal friction angle. Two geometric soil behavior properties predicted by Rankine theory and the active pressure state are:

1. For a retained fill with cohesion, a tension crack is likely to form do to the negative force calculated when, for example, $\sigma'_o k_a < 2c'\sqrt{k_a}$. The depth of this crack (z_c) is calculated as
   a. $z_c = \frac{2c'}{\gamma\sqrt{k_a}}$ for β=0
   b. $z_c = \frac{2c'}{\gamma}\sqrt{\frac{1+sin\,\phi'}{1-sin\,\phi'}}$ for β>0
2. At active pressure, the angle of the failure plane (η) within the retained soil mass is a function of the soils internal angle of friction:
   a. $\eta = 45 + \frac{\phi'}{2}$ from the horizontal plane for β=0
   b. $\eta = \frac{\pi}{4} + \frac{\phi'}{2} + \frac{\beta}{2} - \frac{1}{2}\sin^{-1}(\frac{\sin\beta}{\sin\phi'})$ for β>0

Lateral earth pressure is calculated at any depth as $k_a\sigma'_o$ acting at angle I (I = β for infinite slopes). Using the geometric definitions from figure 2.3, the resulting lateral force is calculated:

$$F_T = \frac{1}{2}\gamma_f h^2 k_{af}$$ (Elias, Christopher, & Berg, 2001)

Equation 2-5

Force from earth pressure (F_T) is applied at h/3 above the base of the wall and can be separated into horizontal and vertical components through simple trigonometry.

2.3.1.2 Surcharge Loads

Surcharge loads (q) for retaining walls are loads imparted onto the retained soil that transfer through the retained soil to the retaining wall. Surcharge loads are typically separated into permanent loads (e.g. structures such as buildings, signs, or bridge abutments) and temporary loads (e.g. vehicular traffic, water during a flood stage, and construction loads). Lateral force from surcharge loads is calculated as a function of the surcharge load and the appropriate lateral earth pressure coefficient. When the surcharge load is assumed to be "infinite" in width, the calculation for the surcharge force (F_s) is

$$F_s = qk_a h$$ (Elias, Christopher, & Berg, 2001) Equation 2-6

where F_s acts along angle I at h/2 above the base of the wall. For surcharge loads that have discrete dimensions, the resulting F_s is calculated by using elastic theories such as Boussinesq's distribution. FHWA (Elias, Christopher, &

Berg, 2001) notes that applied surcharge loads whose limits are fully beyond the active failure wedge do "not need to be considered in the external stability calculations." Note it is the Author's interpretation that this statement should only apply to the sliding, overturning/eccentricity, and bearing capacity calculations; while global stability calculations should account for loads beyond the active failure wedge.

2.3.1.3 MSE "Self Weight" Load

MSE walls have many unique components and design features; however the structure basic design concept for external stability is that it is a gravity wall. Das 2007 explains that the stability of a gravity wall is due to the weight of the wall. The mass of material contained vertically in the footprint of the wall contributes to both driving and resisting forces within the MSE wall system. Figure 2.3 separates the "self-weight" loads into the load of the reinforced zone (V_1) and the load of the wedge of soil created on top of the wall when β>0 (V_2). These loads are calculated as shown in equations 2.7 and 2.8.

$V_1 = \gamma_r HL$ (Elias, Christopher, & Berg, 2001) Equation 2-7

$V_2 = \gamma_f L \frac{h-H}{2}$ (Elias, Christopher, & Berg, 2001) Equation 2-8

2.3.1.4 Applied Bearing Pressure

Pressure applied from the MSE wall to the foundation soil is distributed based on the location (eccentricity, e) of the resultant force at the base of the wall. The magnitude of the resultant force is the sum of the vertical loads on the base of the wall. The eccentricity of the force is calculated as the sum of the moments on the wall divided by the resultant force. This calculation yields the eccentricity measured from L/2.

$$e = \frac{F_T(cos\beta)\frac{h}{3}+F_S(cos\beta)\frac{h}{2}-F_T(sin\beta)\frac{L}{2}-F_S^*(sin\beta)\frac{L}{2}-V_2\frac{L}{6}}{V_1+V_2+F_Tsin\beta}$$ (Elias, Christopher, & Berg, 2001) Equation 2-9

Note that F_S^* is the force from applicable permanent surcharge loads only.

FHWA (Elias, Christopher, & Berg, 2001) uses Meyerhof's equivalent uniform distribution of the applied bearing pressure which is calculated using a reduced length L-2e, resulting in an applied bearing pressure for design:

$$\sigma_v = \frac{qL+V_1+V_2+F_Tsin\beta}{L-2e}$$ (Elias, Christopher, & Berg, 2001) Equation 2-10

2.3.2 Resistance for External Stability

With the exception of overturning, resistance to external loads for MSE walls is driven by the foundation soils and foundation interactions. Foundation soils can consist of natural or fill soils and range from homogenous characteristics to layered and/or anisotropic properties. Both FHWA ASD and LRFD manuals neglect wall embedment for resistance calculations, except for bearing capacity when there is excessive embedment (Elias, Christopher, &

Berg, 2001). Excessive embedment is defined as embedment beyond the minimum required embedment (Berg, Christopher, & Samtani, 2009).

2.3.2.1 Bearing Resistance

Foundation soils provide vertical resistance to the applied pressure of the MSE wall. Foundation soil response is generally elasto-plastic in nature, and consideration must be given to multiple geometries of shear failure and the timeframe/magnitude of elasto-plastic movements that occur without shear failure in the soils. Shear failures are classified as general shear, local shear, or punching shears. These shear failures are depicted in figure 2.5.

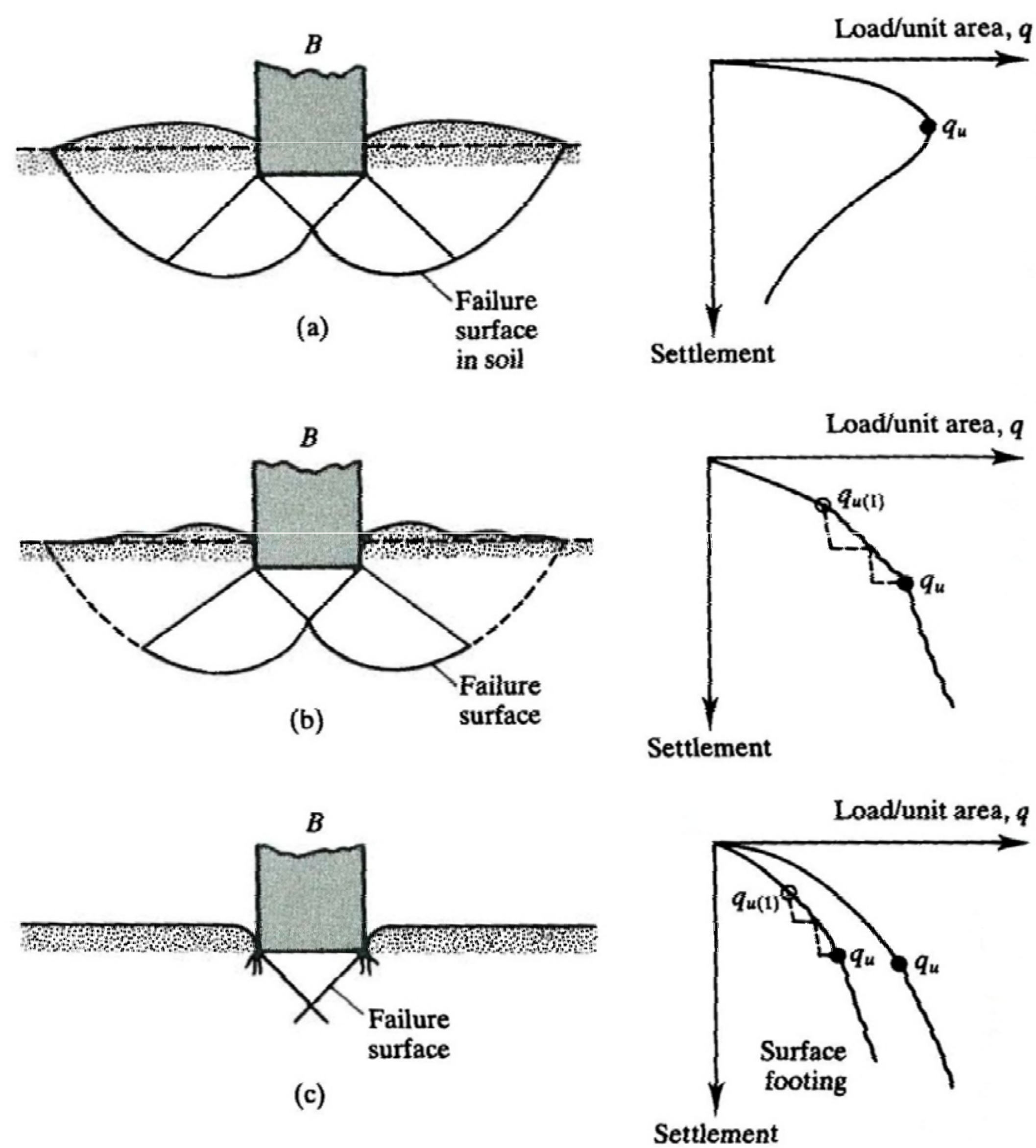


Figure 2-5: Nature of bearing capacity failure in soil: (a) general shear failure; (b) local shear failure; (c) punching shear failure (Das, 2007)

Physical behaviors of these shear failures are (Das, 2007):

- General shear failures (Figure 2-5.a) are most likely to occur in typically stiff clays or dense sands. General shear is characterized by a sudden shearing of the foundation soils that extends to the ground surface. This occurs at an ultimate strength with a subsequent loss in strength during additional deformation.
- Local shear failures (Figure 2-5.b) generally occur in moderately compacted sands and clays and are characterized by increases in the rate of settlement at discrete levels of applied pressure. Typically the failure plane does not reach the ground surface until significant movement of the foundation has occurred. Local shear results in significant deformation, but strength loss is not anticipated.
- Punching shear failures (Figure 2-5.c) are typical in loose and very soft soils. Significant settlement is anticipated in response to loading. Failure is defined as the point where the rate of settlement in response to loading increases and becomes relatively linear. The failure surface will not extend to the ground.

FHWA analyzes general shear and check for local shear conditions (Elias, Christopher, & Berg, 2001) (Berg, Christopher, & Samtani, 2009). Meyerhof's general bearing capacity equation is used to calculate the ultimate bearing resistance (q_u). Equation 2.11 (Berg, Christopher, & Samtani, 2009) assumes a horizontal foundation, no slope in front of the wall, and ground water beyond the influence of the foundation.

$$q_u = c_b N_c + 0.5 L' \gamma_b N_\gamma \qquad \text{Equation 2-11}$$

L' is defined as L-2e with a maximum of L' = L (Berg, Christopher, & Samtani, 2009). N_c, N_γ are bearing capacity factors (Table 2-1)

Table 2-1: Meyerhof's Bearing Capacity Factors

ϕ' (degrees)	N_c	N_q	N_γ	ϕ'	N_c	N_q	N_γ
0	5.14	1.00	0.00	21	15.82	7.07	6.20
1	5.38	1.09	0.07	22	16.88	7.82	7.13
2	5.63	1.20	0.15	23	18.05	8.66	8.20
3	5.9	1.31	0.24	24	19.32	9.60	9.44
4	6.19	1.43	0.34	25	20.72	10.66	10.88
5	6.49	1.57	0.45	26	22.25	11.85	12.54
6	6.81	1.72	0.57	27	23.94	13.20	14.47
7	7.16	1.88	0.71	28	25.8	14.72	16.72
8	7.53	2.06	0.86	29	27.86	16.44	19.34
9	7.92	2.25	1.03	30	30.14	18.40	22.40
10	8.35	2.47	1.22	31	32.67	20.63	25.99
11	8.8	2.71	1.44	32	35.49	23.18	30.22
12	9.28	2.97	1.69	33	38.64	26.09	35.19
13	9.81	3.26	1.97	34	42.16	29.44	41.09
14	10.37	3.59	2.29	35	46.12	33.30	48.03
15	10.98	3.94	2.65	36	50.59	37.75	56.31
16	11.63	4.34	3.06	37	55.63	42.92	66.19
17	12.34	4.77	3.53	38	61.35	48.93	78.03
18	13.1	5.26	4.07	39	67.87	55.96	92.25
19	13.93	5.80	4.68	40	75.31	64.20	109.41
20	14.83	6.40	5.39				

FHWA methodology discounts the benefits of embedment, although it can be included when the embedment exceeds the minimum embedment (FHWA, 2009). Modifications to equation 2.11 for a slope in front of the wall or for high groundwater are provided in 10.6.3.1.2 AASHTO (2007) (Berg, Christopher, & Samtani, 2009).

LRFD checks local shear for soft clays using Equation 2-12 (Berg, Christopher, & Samtani, 2009).

$$\gamma H \leq 3c_b \qquad \text{Equation 2-12}$$

Settlement is the deflection of the foundation soils due to load and is often a controlling factor in allowable bearing capacity determination. Settlements are categorized by the timeframe in which they occur. Elastic settlement is immediate and takes place during and/or immediately after load application. Consolidation settlements occur over a much longer time frame as volumetric change occurs in the soils due to pore-water pressure dissipation (Das 2007). LRFD provides limiting differential settlements for MSE walls (Table 2.2) (Berg, Christopher, & Samtani, 2009); however, these limits only impact the space between the MSE facing panels (panel joints). Total settlement (sum of elastic and inelastic settlements) is not addressed due to the flexibility of MSE wall systems.

Table 2-2: Panel Joint Width for Differential Settlement

Joint Width	Limiting Differential Settlement
20 mm	1/100*
13 mm	1/200
6 mm	1/300

*for differential settlements greater than 1/100, slip joints are recommended

2.3.2.2 Sliding Resistance

MSE wall sliding stability is a function of the horizontal force imparted by the soils and the shear resistance at or near the base of the wall. Slip planes may develop in the reinforced zone at or below the lowest reinforcement layer (for sheet reinforcement), at the interface of the reinforced zone and foundation soil, or in a weak layer in the foundation. Passive resistance from embedment is generally not included in MSE wall design practice (Berg, Christopher, & Samtani, 2009). The total sliding resistance (P_R, Equation 2-13 after (Elias, Christopher, & Berg, 2001)) is the sum of the frictional resistance and the cohesive resistance along the reinforcement length; therefore the critical resistance slip plane will have the lowest resultant shear strength. Frictional resistance is calculated using the friction factor (μ), which is tangent of the internal angle of friction or interface friction for the slip plane being analyzed.

$$P_R = (V_1 + V_2 + F_T sin\beta)\mu \qquad \text{Equation 2-13}$$

Equation 2-13 does not include a term that accounts for cohesion in the foundation soils; however FHWA's ASD (Elias, Christopher, & Berg, 2001) and LRFD (Berg, Christopher, & Samtani, 2009) manuals reference its use when considering sliding along the foundation soils. Equation 2-13 is modified when considering cohesion as shown in Equation 2-14 (Elias, Christopher, & Berg, 2001).

$$P_R = (V_1 + V_2 + F_T sin\beta)\mu + c'L \qquad \text{Equation 2-14}$$

FHWA ASD (Elias, Christopher, & Berg, 2001) discounts vertical contributions of temporary surcharges (Live Loads) for sliding resistance; however permanent surcharge loads may be considered. Das 2007 recommends applying an interface reduction factor for ϕ' (and c') when calculating μ; however FHWA LRFD (Berg, Christopher, & Samtani, 2009) only recommends this reduction when evaluating the interface of sheet type reinforcement.

2.3.2.3 Overturning Resistance

Overturning and limiting eccentricity are closely coupled in their calculation in that both evaluate the balance of moments acting on the MSE wall system. Inspection of Equation2-9 reveals that eccentricity is the moment arm required for a resultant force (the total of all vertical forces acting on the reinforced zone) to balance the moments induced by earth pressures, surcharge pressures, and overburden soil above the reinforced mass (Elias, Christopher, & Berg, 2001). Applying Equation 2-9 and the magnitude of the resultant force to a free body diagram, the sum of moments about the toe of the wall, necessarily equaling zero for a state of equilibrium, can be written as:

$$\sum M = 0 = (F_T + F_S^*)(sin\beta)L + V_2\frac{2L}{3} + V_1\frac{L}{2} - (V_1 + V_2 + F_T sin\beta)(\frac{L}{2} - e) - (F_T\frac{h}{3} + F_S\frac{h}{2})(cos\beta) \quad \text{Equation 2-15}$$

Separating the terms of Equation 2-15 into the form of Equation 2-16 with the resisting moments on the left side of the equation and the driving moments on the right side of the equation:

$$(F_T + F_S^*)(sin\beta)L + V_2\frac{2L}{3} + V_1\frac{L}{2} = (V_1 + V_2 + F_T sin\beta)\left(\frac{L}{2} - e\right) + (F_T\frac{h}{3} + F_S\frac{h}{2})(cos\beta) \quad \text{Equation 2-16}$$

Since the term $(V_1 + V_2 + F_T sin\beta)\left(\frac{L}{2} - e\right)$ represents reaction force from the foundation soils (i.e. it is a passive resistance, not an applied resistance), when e ≥ (L/2), the driving force equals or exceeds the resisting forces, indicating that the wall is unstable in overturning stability. Eccentricity limits are (Berg, Christopher, & Samtani, 2009):

e ≤ L/6 for walls bearing on soils

e ≤ L/4 to walls bearing on rock

The limits on eccentricity also impact bearing capacity as seen in Equation 2-10 and Equation 2-11 by increasing the vertical pressure applied from the wall and simultaneously decreasing the calculated ultimate bearing capacity of the foundation soils.

2.3.3 External Stability Design Procedures

Transportation related MSE wall design is generally performed in accordance with the allowable stress design (ASD) procedures of FHWA-NHI-00-043 (Elias, Christopher, & Berg, 2001) (FHWA ASD) or the load and resistance factor design (LRFD) procedures of FHWA-NHI-10-024 (Berg, Christopher, & Samtani, 2009) (FHWA LRFD) depending on the requirements of the project owner. The primary difference in these documents is the design methodology. ASD methodology uses a factor of safety (FS) to generally account for variations in the system, materials, construction, external factors, and a level of safety in the design. Probability of failure cannot be quantified since ASD does not take specific variations into account. LRFD accounts for variations in load and resistance by using multipliers known as factors. Factors increase or decrease the load or resistance for a specific analytical case. Load factors and resistance factors are developed based on a target reliability index (β). Β is a measure of the overlap of the probability distribution functions for the load and the resistance, allowing for calculation of the probability of failure (Samtani & Sabatini, 2010). The ratio of factored resistance to factored load is the capacity to demand ratio (CDR), which is required to be greater than or equal to one for a balanced system. Resistance factors for MSE wall LRFD design are currently calibrated based on the ASD factors of safety, which limits the influence of soil property variability (Wasman, McVay, Bloomquist, Harrison, & Lai, 2011).

The equations developed in 2.3.1 and 2.3.2 are generally used in both the ASD and LRFD design procedures. Load factors for LRFD equations are shown in Table 2-3 (Berg, Christopher, & Samtani, 2009).

Table 2-3: MSE Wall Load Factors for Permanent Loads

Type of Load	Maximum	Minimum
DC: Component and Attachments	1.25	0.9
EH: Horizontal Earth Pressure		
Active	1.5	0.9
EV: Vertical Earth Pressure		
Overall Stability	1	N/A
Retaining Walls and Abutments	1.35	1
ES: Earth Surcharge	1.5	0.75
Note: May subscript as γ_{EV-MIN}, γ_{EV-MAX}, γ_{EH-MIN}, γ_{EH-MAX}, etc.		

Resistance factors for MSE wall LRFD equations use the terms shown in Table 2-4 (Berg, Christopher, & Samtani, 2009).

Table 2-4: External Stability Resistance Factors for MSE Walls

Stability Mode	Condition	Resistance Factor (ϕ_r)
Bearing Resistance		0.65
Sliding		1
Overall (global) Stability	Where geotechnical parameters are well defined, and the slope does not support or contain a structural element	0.75
	Where geotechnical parameters are based on limited information, and the slope contains or supports a structural element	0.65

Design equations for each failure mode and design method (assuming β>0 and q=0) are:

- Eccentricity

$$e = \frac{F_T(\cos\beta)\frac{h}{3} - F_T(\sin\beta)\frac{L}{2} - V_2\frac{L}{6}}{V_1 + V_2 + F_T\sin\beta} \quad \text{(ASD) Equation 2-17}$$

$$e = \frac{\gamma_{EH-MAX}F_T\cos\beta\left(\frac{h}{3}\right) - \gamma_{EH-MAX}F_T\sin\beta\left(\frac{L}{2}\right) - \gamma_{EV-MIN}V_2\left(\frac{L}{6}\right)}{\gamma_{EV-MIN}V_1 + \gamma_{EV-MIN}V_2 + \gamma_{EH-MAX}F_T\sin\beta} \quad \text{(LRFD) Equation 2-18}$$

- Sliding

$$FS = \frac{(V_1 + V_2 + F_T\sin\beta)\mu}{\frac{1}{2}\gamma_f h^2 k_{af}\cos\beta} \quad \text{(ASD) Equation 2-19}$$

$$CDR = \frac{[\gamma_{EV-MIN}(V_1 + V_2) + \gamma_{EH-MAX}F_T\sin\beta] \times \mu}{\gamma_{EH-MAX}F_T\cos\beta} \quad \text{(LRFD) Equation 2-20}$$

- Bearing (General Shear)

$$FS = \frac{c_b N_c + 0.5L'\gamma_b N_\gamma}{\left(\frac{V_1 + V_2 + F_T\sin\beta}{L - 2e}\right)} \quad \text{(ASD) Equation 2-21}$$

$$CDR = \frac{\emptyset_B(c_b N_c + 0.5L' \gamma_b N_\gamma)}{\left(\frac{\gamma_{EV-MAX}(V_1+V_2)+\gamma_{EH-MAX}F_T sin\beta}{L-2e_B}\right)}$$ (LRFD) Equation 2-22

- $e_B = \frac{\gamma_{EH-MAX}F_T cos\beta\left(\frac{h}{3}\right) - \gamma_{EH-MAX}F_T sin\beta\left(\frac{L}{2}\right) - \gamma_{EV-MAX}V_2(\frac{L}{6})}{\gamma_{EV-MAX}V_1 + \gamma_{EV-MAX}V_2 + \gamma_{EH-MAX}F_T sin\beta}$
- $L' = L - 2e_B, \emptyset_B = Bearing\ resistance\ factor$

FHWA ASD and LRFD manuals do not discuss overturning factor of safety beyond the limiting eccentricity. (AASHTO, 2007) comments that the factor of safety for overturning is replaced by the limiting eccentricity and investigation of bearing pressure. Overturning factor of safety is computed by modifying Equation 2-15. The resulting ratio of resisting moments to driving moments is shown in Equation 2-23:

$$\mathrm{FS} = \frac{(F_T + F_S^*)(sin\beta)L + V_2\frac{2L}{3} + V_1\frac{L}{2}}{(F_T\frac{h}{3} + F_S\frac{h}{2})(cos\beta)}$$ Equation 2-23

2.3.4 Soil Properties and Behavior

The equations reported in Section 2.3 identify that soil parameters, geometry, and surcharge loads are the basic variables in wall design. From a practical perspective, the geometry and surcharge loads are defined by the designers involved in the project, though not specifically the wall designer (Smith & Janacek, 2011). While this demonstrates the importance of proper project communication, the soil parameters in the retained, reinforced, and foundation zones of the MSE wall represent approximately 70% of the variables used in the design equations. As with many design variables, the wall designer makes assumptions about the properties of the soil that will be used for construction; however actual soil properties can vary, sometimes substantially, based on the actual materials of construction. For example, FHWA recommends that reinforced zone soils meet the AASHTO T-27 gradation shown in Table 2-5.

Table 2-5: AASHTO T-27 Gradation (Berg, Christopher, & Samtani, 2009)

U.S. Sieve Size	Percent Passing
4 inch(102 mm)	100
No. 40 (0.425 mm)	0 – 60
No. 200 (0.075 mm)	0 – 15

Assuming the material is placed in a compacted state, the in place unit weight soils with gradations meeting AASHTO T-27 can range from approximately 100 pounds per cubic foot (pcf) to approximately 155 pcf (Naval Facilities Command, 1986) (not accounting for submerged conditions). A wall with a design sliding factor of safety of 1.5 using an average reinforced fill unit weight of 125 pcf that was constructed with a 100 pcf reinforced zone fill would have (assuming all other factors remain equal) an as-built factor of safety of 1.2. Bowles (Bowles, 1996) reports representative friction angle values for common soil types ranging from 0 to 55 degrees, and cohesion tends to have a greater variation the friction angle (Hsu & Nelson, 2002). The following sections summarize the literature reviewed for variations in soil parameters.

2.3.4.1 Soil Properties

Section 2.3.1 summarizes the soil properties used in design of MSE walls. The soils used in each zone of the wall will vary; however each soil will have the basic properties of density (unit weight), friction angle, and cohesion. In addition to the listed properties, consolidation characteristics of the soil mass influence the long term performance of the wall system. The following sections summarize aspects of each of these properties that impact their variability relative to the design and performance of MSE walls.

(1) Unit Weight

Soil density is typically referred to as unit weight in the U.S. design manuals and practice. Unit weight, typically in units of pounds-force per cubic foot, is the weight of a soil sample divided by the volume of a soil sample. Das 2007 details this concept as:

$$\gamma = \frac{W_s + W_w}{V_s + V_w + V_a} \qquad \text{Equation 2-24}$$

Where:

- o W_s = Weight of solids
- o W_w = Weight of water
- o V_s = Volume of soilds
- o V_w = Volume of water
- o V_a = Volume of air

The weight of water relative to the weight of solids is known as the moisture content (in percentage):

$$w = \frac{W_w}{W_s} x100 \qquad \text{Equation 2-25}$$

The volume of voids (V_v) is the sum of V_s and V_a, and the void ratio (e) is defined as

$$e = \frac{V_v}{V_s} \qquad \text{Equation 2-26}$$

Specific gravity (G_s) is the density of the soil relative to the density of water (γ_w). Incorporating G_s, w, and e into the equation for γ provides a general form equation:

$$\gamma = \frac{G_s \gamma_w (1+w)}{1+e} \qquad \text{Equation 2-27}$$

Equation 2-27 identifies that the unit weight of soils is a function of the specific gravity, moisture content, and void ratio. Das (Das, 2007) notes that the range of specific gravity for a given soil type is relatively narrow (with the exception of peat) as shown in Table 2-6 below. It should be noted that when mixed soils are used, Equation 2-27 will have to be adjusted for the components of the soil matrix, or properties for the specific soil blend will have to be developed.

Table 2-6: Specific Gravities of Selected Soils (after Das, 2007)

Type of Soil	Specific Gravity (G_s)	Δ G_s
Quartz Sand	2.64 – 2.66	0.02
Silt	2.67 – 2.73	0.06
Clay	2.70 – 2.90	0.2
Chalk	2.60 – 2.75	0.15
Loess	2.65 – 2.73	0.08
Peat	1.30 – 1.90	0.6

Water content has an intuitive lower bound of zero; however most natural and fill deposits have some water. The upper limit is the saturated state where $V_w = V_v$. For a saturated condition, $e = wG_s$, which modifies Equation 2-27 to

$$\gamma_{saturated} = \frac{G_s\gamma_w + e\gamma_w}{1+e} \qquad \text{Equation 2-28}$$

Soil void ratios generally range from 0.35 < e < 2 for soil in its densest to loosest state (Bowles, 1996), although high organic soils and montmorillonitic clays can reach values of e = 5.2 (Christopher, Schwartz, & Boudreau, 2006). Compaction of the soil (whether from natural processes or construction procedures) and grain size distribution are the significant influencers of void ratio, although electrochemical interactions on the microscopic level impact clays as well.

(2) Shear Strength

Shear strength of a soil is a common engineering definition of the shear stress along the plane of failure for a specific normal load (Holtz & Kovacs, 1981). A linear failure envelope is commonly used to estimate the shear strength. This failure envelope is defined by three components as shown in Equation 2-29.

$$\tau = \sigma \, tan\varphi + c \qquad \text{(Holtz \& Kovacs, 1981) Equation 2-29}$$

Where τ is the shear strength of the soil, σ is the normal stress, and c is the cohesion. Equation 2-29 consists of stress dependent and stress independent variables. The stress dependent term, $\sigma \, tan\varphi$, is the same as the sliding resistance equation, where $tan\varphi$ represents the friction factor within the soils. Cohesion is the “sticking together of like materials” (Holtz & Kovacs, 1981), and is affected by the water content of the soil. Sands are generally cohesionless soils, relying on pressures applied to the soil structure to resist shearing. Clays and silts are generally cohesive soils that have several time dependent properties that are driven by the internal drainage of water within the soil being slow because of the small pore sizes within the soil mass. Clay behavior changes significantly between unsaturated and saturated condition and total stress and effective stress conditions. Both areas of clay variation are primarily driven by the water content, which changes mostly due to environmental factors (Holtz & Kovacs, 1981).

Values for c and φ are measured through laboratory testing. Direct shear testing and triaxial testing are categories of soil tests that measure strain versus variable stress (typical the variable stress is in one direction with constant stress in other directions). Failure of the sample is typically selected at the point of maximum stress that does not cause continued strain of the sample (peak stress). For soils that do not display a peak stress, 15% strain is a typical cutoff of a test. Multiple tests are performed on the same material to develop the failure envelope.

Test selection should account for the soil type, drainage conditions, and field loading condition. Cohesionless materials that are not expected to develop positive or negative pore pressure can be tested using a direct shear. Cohesive soils and soils that develop positive or negative pore pressure are commonly tested using the consolidated-undrained (CU) triaxial test with pore pressure measurements so that total and effective stress parameters can be measured simultaneously. To evaluate local bearing shear in soft soils, an unconfined compression test (UC) or an unconsolidated-undrained (UU) triaxial test may best represent the short term conditions in the field that are associated with local/punching shear in bearing capacity. Anisotropic materials respond differently to stress based on the relative orientation of the stress and the soil structure. Anisotropic materials such as shale can be tested using a direct shear to estimate shear strength for sliding failures, and also using a UC, UU or CU to evaluate bearing capacity and global stability. Although equation 2-29 indicates that the failure envelope is linear, it is often curved for a large range of stress conditions; therefore selection of confining pressures that represent the field conditions is important for gaining applicable test results (Holtz & Kovacs, 1981).

Some observations from the literature reviewed regarding application of test data are:

- Peak friction angle values are often used for design; however these values may overestimate the actual field resistance.
- The sequence in which the soils are loaded, known as the stress path, significantly effects soil strength (Lambe, 1997).

- Cyclic wetting and drying of compacted clays and stiff, fissured clays impacts the long term shear strength of these soils (Wright, Zornberg, & Aguettant, 2007).
- Designs should consider shear strengths for totals stress and effective stress conditions (FHWA, 2009).

(3) Consolidation

Fine grained soils undergo time dependent movements known as consolidation. Consolidation occurs as the initial pore water pressures dissipate after pore pressure builds up in the clay matrix when load is applied. The dissipation of pore pressure allows for the rearranging of soil particles into a more dense state (lower void ratio); resulting in volumetric change and vertical displacement at the surface (Holtz & Kovacs, 1981). The magnitude of consolidation settlement is affected by the additional applied pressure, the previous maximum pressure in the soil mass, and the rate in change of void ratio relative to load (virgin compression, c_c and recompression, c_r). The time rate of consolidation (coefficient of consolidation, c_v), the drainage paths, and the thickness of the soil mass are used to estimate the time over which the consolidation settlement will occur (Holtz & Kovacs, 1981).

2.4 Probability of Failure

Traditional factors of safety (Duncan, 2000) and modern LRFD analysis do not yet account for variation in soil properties (Wasman, McVay, Bloomquist, Harrison, & Lai, 2011). Variations in soil properties are accounted for and described through parametric studies and probability of failure analysis (Chalermyanont & Benson, 2005). Reliability analysis incorporates the soil variations and can be used in conjunction with established design procedures to develop an acceptable design (Duncan, 2000).

Parametric studies are performed by holding all variables except for one constant in order to evaluate the impact of that variable on the overall design. The parametric study for external stability of MSE walls reported by Chalermyanont & Benson in 2005 summarizes that sliding is primarily impacted by variations in the retained zone friction angle, bearing capacity by the retained zone and foundation zone friction angles, and overturning by the retained zone friction angle and the unit weight of the reinforced fill.

Duncan (Duncan, 2000) outlines a method to evaluate the reliability of a factor of safety for a failure mode using the Taylor series method. This method is a first order estimation that uses the mean and standard deviation of each applicable variable to estimate the standard deviation (σ_F) and coefficient of variation (V_F) for the failure mode. A factor of safety for the failure mode is calculated using the most likely value of each parameter (F_{MLV}). V_F and F_{MLV} are then used to determine the probability of failure (P_f) and reliability for the factor of safety using a lognormal distribution.

Standard deviations and most likely values can be estimated using published data, local experience, or analysis of available, appropriate testing data. Local experience and testing data are evaluated by assuming a normal distribution of variable and dividing the distribution into six equal parts (either mathematically or graphically). This estimate is based on 99.73% of all values for a normal distribution falling within 6 standard deviations (Duncan, 2000). Published coefficients of variations can be used to back calculate of standard deviations for specific variables using the equation for coefficient of variation and the most likely value of the variable:

$$\sigma_i = V_{published} \times MLV_i$$ (After Duncan, 2000) Equation 2-30

Table 2-7 summarizes coefficients of variations for MSE wall design variables.

Table 2-7: Summary of Published Coefficients of Variation (after Duncan, 2000; USACE, 2006; and Kim, 2012)

Parameter	Coefficient of Variation	Reference
Unit Weight	0.03	USACE, 2006
Unit Weight (retained soils)	0.1	Kim, 2012
Unit Weight (reinforced zone fill)	0.05	Kim, 2012
Unit Weight	0.03 - 0.07	Duncan, 2000
Buoyant Unit Weight	0 - 0.1	Duncan, 2000
Angle of Internal Friction retained soils	0.1	USACE, 2006
Angle of Internal Friction (Sand) (ϕ)	0.025	Kim, 2012
Interface Friction Angle (Foundation and Reinforced Zone Soils)	0.022	Kim, 2012
Effective Stress Friction Angle	0.02 - 0.13	Duncan, 2000
Cohesion (Undrained Shear Strength) (c)	0.4	USACE, 2006
Undrained Shear Strength	0.13 - 0.40	Duncan, 2000
Wall Friction Angle (d)	0.2	USACE, 2006
Wall Friction Angle (d)	0.25	USACE, 2006
Earth Pressure Coefficient Sand (K)	0.15	USACE, 2006
Dead Load	0.1	USACE, 2006
Live Load	0.25	USACE, 2006
Surcharge Load	0.205	Kim, 2012
Seismic Force	0.3	USACE, 2006

Using the most likely values and standard deviations for each parameter, 2N factor of safety calculations are performed for a failure mode, where N is the number of variables, varying one variable plus one standard deviation ($F_i^{+\sigma}$) and minus one standard deviation ($F_i^{-\sigma}$) while the others variables are held constant. For each pair of $F_i^{+\sigma}$and $F_i^{-\sigma}$ standard deviation calculations, ΔF is calculated as:

$$\Delta F_i = F_i^{+\sigma} - F_i^{-\sigma} \qquad \text{(after Duncan, 2000) Equation 2-31}$$

The standard deviation for the failure mode is then calculated:

$$\sigma_F = \sqrt{(\frac{\Delta F_1}{2})^2 + (\frac{\Delta F_2}{2})^2 + (\frac{\Delta F_i}{2})^2 \ldots + (\frac{\Delta F_N}{2})^2}$$ (after Duncan, 2000)

Equation 2-32

and the coefficient of variation for the failure mode is calculated:

$$V_F = \frac{\sigma_F}{F_{MLV}}$$ (Duncan, 2000) Equation 2-33

Probability of failure is the probability that the factor of safety will fall below 1, which can have a variety of implications ranging from unsatisfactory performance to catastrophic collapse depending on the failure mode and the physical characteristics of the failure. This methodology can be used to calculate the probability that a quantity will exceed a defined limit. Duncan demonstrates this application for settlement using a settlement ratio (SR) of probable settlement divided by most likely settlement. The same approach will be applied to eccentricity in section 3 of this thesis.

Table 2-8 below and probabilities of failure for settlement are calculated by calculating the lognormal reliability index (β_{LN}) using V_F and F_{MLV} or another ratio comparing the limit value and most likely value. β_{LN} for factor of safety calculations is calculated as:

$$\beta_{LN} = \frac{\ln\left(\frac{F_{MLV}}{\sqrt{1+V^2}}\right)}{\sqrt{\ln(1+V^2)}}$$ (Duncan, 2000) Equation 2-34

and for ratios of limiting values and likely values such as settlement:

$$\beta_{LN} = \frac{\ln\left((SR)(\sqrt{(1+V^2)})\right)}{\sqrt{\ln(1+V^2)}}$$ (Duncan, 2000) Equation 2-35

The reliability of β_{LN} is determined using a standard cumulative normal distribution function, and the probability of failure is calculated as one minus reliability.

Table 2-8: Probabilities of Factor of Safety (%) < 1.0 (Lognormal Distribution of Factor of Safety) (after Duncan, 2000)

F_{MLV}	Coefficient of Variation of Factor of Safety (VF)											
	2%	4%	6%	8%	10%	14%	16%	20%	25%	40%	50%	80%
1.05	0.01	0.12	0.22	0.28	0.33	0.39	0.41	0.44	0.47	0.53	0.55	0.61
1.1	0.00	0.01	0.06	0.12	0.18	0.27	0.30	0.35	0.40	0.48	0.51	0.59
1.15	0.00	0.00	0.01	0.04	0.09	0.18	0.21	0.27	0.33	0.43	0.48	0.56
1.16	0.00	0.00	0.01	0.03	0.08	0.16	0.20	0.26	0.32	0.42	0.47	0.56
1.18	0.00	0.00	0.00	0.02	0.05	0.13	0.17	0.23	0.29	0.41	0.45	0.55
1.2	0.00	0.00	0.00	0.01	0.04	0.11	0.14	0.21	0.27	0.39	0.44	0.54
1.25	0.00	0.00	0.00	0.00	0.01	0.06	0.09	0.15	0.22	0.35	0.41	0.51
1.3	0.00	0.00	0.00	0.00	0.01	0.03	0.06	0.11	0.17	0.31	0.37	0.49
1.35	0.00	0.00	0.00	0.00	0.00	0.02	0.04	0.08	0.14	0.28	0.34	0.47
1.4	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.05	0.11	0.25	0.32	0.45
1.5	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.06	0.19	0.27	0.41
1.6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.04	0.15	0.22	0.38
1.7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.12	0.19	0.34
1.8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.09	0.16	0.31
1.9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.07	0.13	0.29
2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.11	0.26
2.2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.08	0.22
2.4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.05	0.19
2.6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.04	0.16
2.8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.13
3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.11

Table 2-9 (U.S. Army Corps of Engineers, 2006) is presented to aid in the understanding of probability values and verbal descriptions of probability.

Table 2-9: Empirical Translations of Verbal Descriptions of Uncertainty

Verbal Description	Probability Equivalent (%)
Virtually Impossible	1
Very Unlikely	10
Unlikely	15
Fairly Unlikely	25
Fair Chance, Even Chance	50
Usually, Likely	75
Probable	80
Very Probable	90
Virtually Certain	99

Probabilistic analysis for geotechnical structures commonly target a reliability index value of 3.0, and critical structures such as bridges use a reliability index of 3.5 for critical components. These β values correspond to probabilities of failure of 0.13% and 0.023% respectively (Kim & Salgado, Load and Resistance Factors for External Stability Checks of Mechanically Stabilized Earth Walls, 2012).

2.5 Asset Management

Wall design life is often specified to be 100 years; however experience with the long term performance of walls is limited. Asset management systems enable owners to better track their inventory and the condition of the walls within the inventory. This knowledge is useful in prioritizing repairs and maintenance to try to minimize excessive wall distress/failure. Repairs of failed transportation system walls often cost millions of dollars (FHWA, 2008). Feasibility of a retaining wall asset management program was studied by the Colorado Department of Transportation (CDOT). The study found a management system feasible and beneficial, providing “efficient preservation of existing walls and the informed selection of designs in new construction…” (Hearn, 2003) Although many benefits exist from implementing an asset management system for retaining walls, limited programs have progressed beyond the concept stage and to the implementation stage (Anderson, Alzamora, & DeMarco, 2008). FHWA identified only three retaining wall asset management systems in use in 2008, the Oregon Department of Transportation, the City of Cincinnati, and the National Park Service (FHWA, 2008). Utah Department of Transportation (UDOT) has also embarked on an inventory and assessment program (Swenson, 2010). Each of the programs follows the asset management methodology:

1. Develop an inventory of the walls
2. Assess the condition of the walls
3. Provide recommendations for maintenance actions and budget
4. Repeat steps 2 and 3 on a regular basis

Each of the systems reviewed primarily focused on inventory data for the wall, physical distress features, and surrounding conditions that would impact wall performance. Notable observations of the systems include:

- The National Park Service system and CDOT study incorporated an evaluation of whether or not the wall met current AASHTO design requirements; however, the design requirements appear to be the traffic safety requirements and materials of wall construction, and not the internal and external stability design (Hearn, 2003).
- The National Park Service system proposed reliability factors to modify condition assessments that may or may not be visible or accurately identifiable. This reliability is associated with the inspectors' ability to physically observe a condition, not the inherent variation of physical values (Anderson, Alzamora, & DeMarco, 2008).
- The National Park Service system produced ratings for each physical wall component and an overall wall condition rating
- UDOT categorized physical distress observations and field influencers (drainage, grading, etc.) based on correlations to modes of failure, providing an adverse performance score for a specific failure mode for each wall.

- Rating values are not consistent between various methods.
- The ratings of the systems do not appear to incorporate consequences of failure.

Brutus and Tauber provide example recommendations for a simplified wall rating scale range and a separate consequence of failure rating in a report prepared for the National Cooperative Highway Research Program (Brutus & Tauber, 2009):

- Numerical Wall Ratings (after Brutus, 2009)
    1. Excellent – No observable indications of significant distress.
    2. Good – Some observable distress, wall is performing as designed.
    3. Fair – Multiple distress observations, deteriorating wall performance.
    4. Poor – Significant levels of deterioration/distress, potential for wall failure.
    5. Critical – Severe levels of deterioration/distress, imminent wall failure.
- Standard Term Wall Ratings (Brutus & Tauber, 2009)

- Good to Excellent – No current distress indicating performance concerns, and no evidence of repairs from previous undocumented distress.
- Fair – Some observed distress in the overall wall and the wall components. Repairs that have occurred have improved wall performance.
- Poor to Critical – Wall movement relative to its surroundings and distress of wall components is readily observable. Previous wall repairs have not improved wall performance.

- Consequence-of-Failure Rating (Brutus & Tauber, 2009)
  - Severe – High probability of:
    - Injury/death from falling debris on heavily traveled routes or collapse of structures supported by the wall (and debris fall associated with the structures collapse).
    - Significant damage to vehicles or structures.
    - Closure of all lanes of a high volume roadway requiring lengthy detours.
  - Significant – Low likelihood of injury, but probable:
    - Significant property damage.

- Interruption of utilities to a large area.
- Prolonged restriction/blockage of access to private/public facilities.
- Long lasting environmental or cultural resource damage.
- Multi-lane closure of high volume roadway.
- Closure of any roadway with no available detour or a lengthy detour.

o Minor

- Low likelihood of injury, vehicular damage, or damage to third parties.
- Full road closures permissible if alternate access is available.
- Closure of a single lane on high volume roadway.

Chapter 3

Wall Assessment and Rating

3.1 Introduction

Using the information gained from the literature review and the Author's experience, this chapter describes the proposed wall assessment and rating system. The assessment consists of both a physical and as-built design assessment. The rating system incorporates the physical and as-built design assessments as well as external influencers to provide an updatable rating for the wall. Recommended actions are provided for each rating level. The following sections detail the proposed system.

3.2 Physical Assessment

Physical condition of the MSE walls should be assessed on a regular basis to update information on the overall condition of the wall. The actual frequency of assessment will vary based on several factors. Recommendations for frequency of physical assessments based on this rating system are provided in Section 3.4.

Key components of a physical assessment program include:

- Accuracy
- Repeatability
- Applicability
- Comprehensiveness
- Simplicity of terms

Visibility

3.2.1 Distress Features

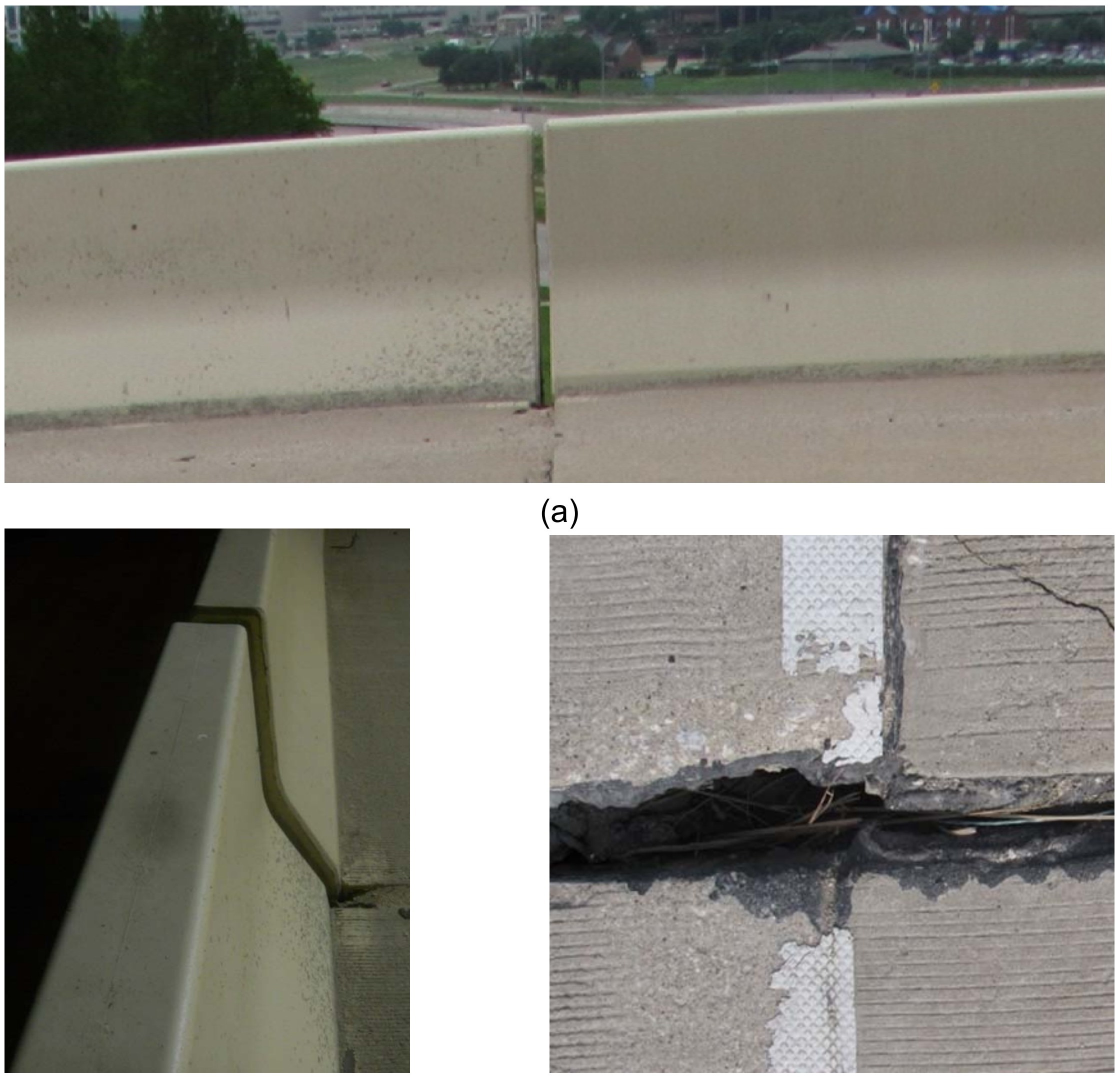
MSE walls can exhibit a variety of distress features that may be directly tied to the wall, such as a cracked panel, or that are indirectly linked to the wall, such as heaving soil at the toe of the wall. Distress features are defined as features indicating degraded performance of the wall. External influencers beyond the general design and construction of the MSE wall that may cause degraded performance are discussed in section 3.1.2. This system separates evaluation of distress features into three zones; Top Slope, Wall Face, and Toe Slope. The locations, magnitudes, and frequency of these distress features provide an indication of the performance of the wall at that point in time.

3.2.1.1 Top Slope

The top slope of the MSE wall consists of the surface of the soil or structure on top of and behind the wall. Common features that indicate wall distress, or that may lead to wall distress include depressions, ponding of water, cracks in the surface of the soil or pavement, areas of erosion, and distress of nearby structures.

Figure 3-1: Out of Plumb Structure in Retained Zone

Distress features in the top slope should be evaluated for horizontal and vertical components of movement. Figure 3-2 indicates horizontal movement of the paving (traffic rail is offset and pavement joints are offset), however there is limited vertical deformation indicated. Note that vertical movements of cracks in slopes will be difficult to identify.



(a)

(b)

(c)

Figure 3-2: Lateral Displacement with no Vertical Displacement: (a) profile view of traffic rail; (b) offset traffic rail; (c) offset pavement joint in-line with offset traffic rail

While the frequency and magnitude of cracks for the entire width of the slope can influence the walls performance (and should be recorded), key locations to evaluate are at the limits of the reinforcement, near the theoretical extension of the active wedge, and at least one wall height (or further if preliminary global stability analysis indicates) beyond the extension of the active wedge. In addition to vertical and horizontal indications of movement, the shape (along the length of the wall) and depth of the crack can provide valuable analytical information.

Figure 3-3: Cracking in Retained Zone Soil

Figure 3-4: Cracking in Retained Zone Pavement

Depressions behind the wall may be shallow and widespread, localized, or intermittent. Care should be taken to evaluate potential sources in order to differentiate between external erosion and mechanically induced depressions (although these will still impact maintenance aspects) versus depressions caused by wall/slope deformation or internal piping/erosion of soils.

Figure 3-5: Depression in Paving

Figure 3-6: Erosion at Adjacent to Wall Flume

3.2.1.2 Wall Face

Assessment of the wall face incorporates the wall panels, joints between panels, and coping. Wall facing distress can be the most easily identifiable, measurable and misleading forms of distress observed in an MSE wall assessment. One of the benefits of MSE walls is the inherent flexibility in most of the systems. This flexibility allows for panel displacement without compromising the integrity of the wall system (Berg, Christopher, & Samtani, 2009).

(1) Background Information

Displaced/damaged panels can easily be viewed as distress in the system; however additional information is needed to understand the degree and impact of this apparent distress. Prior to or in conjunction with assessment of the wall face, the assessor should identify:

- Baseline condition
- Reinforced zone type (rock, sand, or cement stabilized sand)
- Bearing pad type [rubber or high density polyethylene (HDPE)]
- Panel overlap

The original construction of the wall may create the appearance of distress due to the installation tolerances and procedures. Installation tolerances can vary based on the owner and wall designer; however the TxDOT construction manual specifies that wall panel joints openings from 3/8” to 3/4”, and the vertical plumb of the wall face after wall construction is completed should be no greater than 1/2” in 10 feet (TxDOT, 2004). Assessment measurements that exceed these tolerances may indicate distress.

Lateral movements of approximately 3/4” per 10 foot of wall height are anticipated to occur during construction depending on backfill operations, reinforcement type and length, reinforcement connection, construction quality, and the wall facing details (Berg, Christopher, & Samtani, 2009). Additional movements may occur due to settlement, surcharge loads, and degradation of the MSE wall system.

Reinforced zone fills influence distress observed in the wall face based on the type of fill. Typical examples of this influence are:

- Rock in the 3/8” to 3/4” range can wedge between panels, causing potential spalling and cracking.
- Sand (and even some gravels) can be washed out, potentially loosening backfill and decreasing support of the panel (Figure 3-7)
- Cement stabilized sand limits movement of the panels, potentially masking wall movements that are indications of distress

Figure 3-7: Reinforced Zone Material Loss
Through Panel Joints

Two types of bearing pads are typically used by wall manufacturers HDPE and rubber. HDPE bearing pads typically have very small deflections. Rubber bearing pads typically have a greater deflection response to load. The response of rubber pads is evident when different loads are applied across the same panel (e.g., when aesthetic panels are used that have protruding features that create additional weight such as prisms), creating the appearance of panel movement/distress.

Precast panels are manufactured with several types of joints depending on the manufacturer and the purpose of the panel. Two basic joint categories are overlapping and slip joints. Overlapping joints have protrusions that overlap protrusions from adjacent panels. The overlap (and in some cases shear pins connecting panels) appears to have been originally intended to limit differential horizontal movements in panels. Many of the joints today are designed to allow for 1/2” to 3/4” of lateral movement before the panels come into contact with each other. Overlapping joints are also required to provide environmental exposure protection for the bearing pads and filter fabric in the joints. Lateral restraint of panels from the overlap can lead to visible cracking in the panels.

Slip joints are used where large differential movements are anticipated (e.g. when the wall crosses a large box culvert, or at a change to a different wall type). Manufacturers have designed a variety of slip joints that provide the appropriate environmental exposure protection while allowing for vertical and lateral deformations exceeding one (1) inch. Differential movements at slip joints should be noted in wall assessments; however this movement is not necessarily a sign of adverse distress.

Panels may be damaged prior to or during placement. The Author has observed many hairline cracks in panels that have not yet been installed in walls, and chipping of the overlap protrusions can occur during handling or installation. While proper quality control procedures should prevent these panels from being installed, the damaged panels occasionally become a part of the permanent wall.

(2) Common Distress and Maintenance Features

FHWA ASD (Elias, Christopher, & Berg, 2001) Section 9.3.e lists a variety of out of tolerance conditions for panels that are typically identified during or shortly after construction. Many of the conditions identified will be addressed during construction; however some will be left in place or develop at a later date. Based on this list, post-construction wall face distress can be generally categorized into panel joints, individual panels, and wall sections.

Changes in the panel joint spacing is typically the first indicators of MSE wall movement (unless cement stabilized backfill is used). Tightening or widening of these joints beyond tolerance can lead to additional distress in the wall. Inspection of joints will also provide indications of drainage issues such as staining and/or washout material may be present on the joint surface. Material washout and vegetation growth (Figure 3-8) in the joints are also indicators of degraded filter fabric, and potentially degraded reinforced zone backfill.

Figure 3-8: Tree Growth Through Panel Joint

Cracking or spalling of an individual panel may be an indication of wall distress if the damage did not occur during construction. Cracking and spalling are often associated with panel to panel contact due to excessive movements causing panel joints to close, or nominal movements when debris is in the panel joint. Rust staining at a crack or opening could indicate full penetration through the panel and corrosion of the panel reinforcing steel. Assessments should differentiate cracks/spalls that occur between the edge of the panel and the reinforcement connection, and cracks/spalls that encompass a reinforcement connection(s). Cracking of panels that are restrained by something other than an adjacent panel (such as coping, direct connection to a bridge pier, etc.) may indicate overall wall movement Figure 3-9

Figure 3-9: Cracked Panels and Coping

Individual panel batter and/or bulging displacement are often a construction tolerance issue; however localized washout, differential support (direct connection versus standard reinforcement), reinforcement connection corrosion, and excessive settlement are some of the post-construction causes of this distress.

Sections of the wall exhibiting batter and/or bulging issues are generally more indicative of post-construction movement/distress. Differential settlement is also more apparent within a section of a wall (Figure 3-10). Sections of wall exhibiting a type of overall distresses should be delineated for further analysis and evaluation during the physical and design assessments.

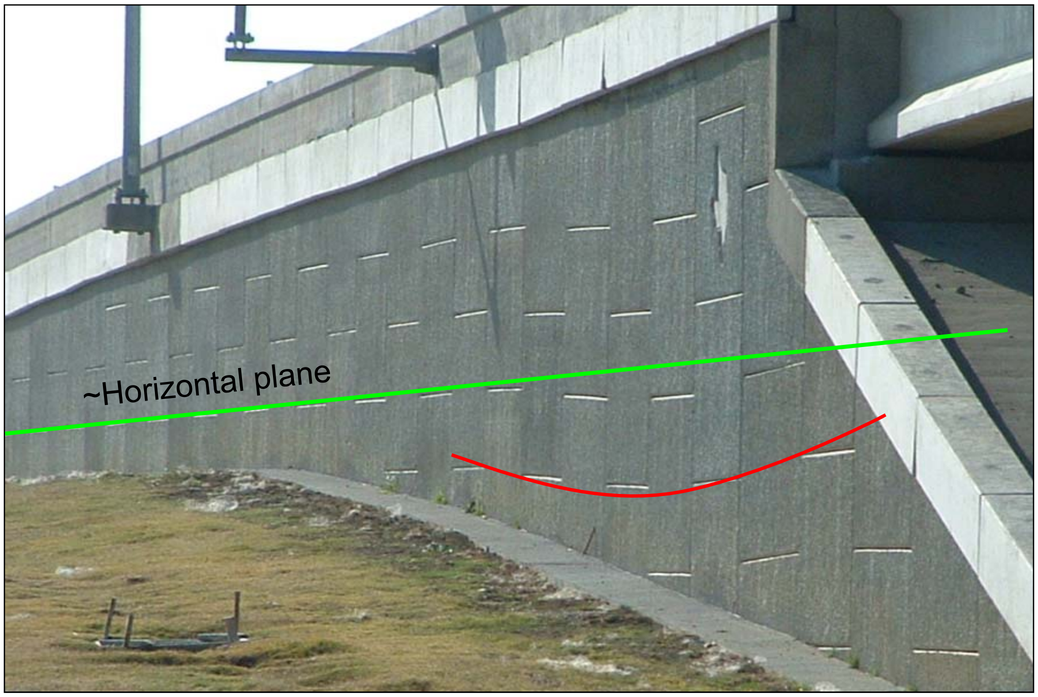


Figure 3-10: Differential Settlement

3.2.1.3 Toe Slope

Distress features in front of the wall face occur in an area ranging from immediately at the face to distances greater than the wall width depending on soil conditions and toe slope angle (after Bowles, 1996). Assessment distances can be refined through preliminary global stability analysis and bearing capacity evaluations. Toe slope distress features include vertical and horizontal deformations of soil, structures, and/or paving/flat work in front of the wall. These deformations commonly manifest as heaving of soil/flatwork, depressions/ponding water, shearing of curbs, and distress to contiguous structures such as traffic rail.

Figure 3-11: Sheared Curb at Wall Base

Figure 3-12: Spalled Concrete Traffic Rail at Wall Base

Assessment of the toe slope should include identification of water seepage. Seepage may or may not be evident from the wall face, potentially surfacing at a low point away from the wall toe or through cracks in the paving.

Figure 3-13: Seepage at Base of Wall (through
~3ft tall traffic rail

Figure 3-14: Seepage Offset from Base of Wall

3.2.2 *Surrounding Slopes, Structures, and Utilities*

Assessments should identify and record information on the geometry of the slopes adjacent to the walls as well as the location and description of structures, utilities, drainage features, landscape, and other structures/activities that could impact the performance of the wall. This data should be utilized in the as-built design analysis in order to more accurately reflect the as-built slope geometry and surcharge loads.

Figure 3-15: Abandoned Utility in the Retained Zone

Special attention should be paid to drainage features that are integrated in the wall reinforced zone (Figure 3-16). Clogging or damage to these drainage structures can introduce unplanned volumes of water flow into the reinforced zone and foundation zone.

(a) (b)

Figure 3-16: Debris in Integrated Wall Inlet: (a) damaged flume at inlet; (b) debris in inlet

Erosion, animal burrowing, mowing, and construction activities are some of the other external influencers that may impact apparent wall distress features (Figure 3-17).

Figure 3-17: Localized Erosion at Toe of Wall (~4ft width)

3.2.3 Assessment Procedures and Checklist

Assessments are performed by teams with multidisciplinary experience in order to better evaluate the wall and its surroundings as an entire system. Teams use a measuring wheel to track location along the wall. A second measuring wheel is useful in measuring offsets from the wall, and a tape measure is used to measure cracks in panels, soil, pavement, etc. A four foot digital level provides information on slope angles, wall batter, and verticality of surrounding structures. Pictures should be taken of the overall wall setting as well as detail photos. Pictures should include non-distressed areas in order to allow for comparison with future assessments. Each pictures location and direction of view should be logged when the picture is taken. Primary wall assessments should be performed during safe, comfortable weather conditions to promote accuracy; however supplementary assessments should be performed during or immediately after a rain event (the Author has identified significant blockages in storm drain systems that were only evident during/after a rain event).

When working on or adjacent to roadways, traffic control is necessary. Adequate assessments require time and access to travel lanes. For the convenience of the Owner, assessments can be performed in conjunction with other maintenance activities that require lane closures.

Assessment teams record observations on the Wall and Slope Distress Visual Indicators checklist (copy provided in the Appendix A). The checklist is separated into the three distress feature zones. There are a total of twenty two questions on the checklist. The checklist wording was designed to be easily interpreted by all parties that may assess the wall. The assessment team may elect to use multiple checklists for a wall in order to separate sections of the wall demonstrating different levels of distress; however adequate observational data should be recorded in either case to overlay wall distress features on the wall layout.

3.2.4 Application of assessment data

Assessment data is reduced to numerical equivalents (factors) through the checklist information by summarizing the general overall distress (score factor), the frequency of distress (frequency factor), and the level of distress (distress level factor) into an overall level of distress for a section and zone of the wall (condition factor). Data from the physical assessment is analyzed to determine separate condition factors for each wall distress zone: top slope (T), wall face (W), and toe slope (B). The condition factor for each zone is a weighted average of three sub-factors: score factor, frequency factor, and distress level factor. Calculation of these sub-factors and the condition factor is described below.

The assessment checklist is organized so that “yes” answers to assessment questions indicate distress. The percentage of “yes” answers for each section of the checklist correlates to the score factor value according to Table 3-1. Distress feature observations are normalized by the length of the wall or length of the wall segment. The resulting ratio is used to determine the frequency factor as shown in Table 3-1.

Table 3-1: Score and Frequency Factor Rating Correlations

Factor Value	Score Factor	Frequency Factor
0	≤ 10%	≤ 0.001
1	11% to ≤30%	>0.001 to ≤0.003
2	31% to ≤60%	>0.003 to ≤0.007
3	>60%	>0.007

Each distress observation is assigned a distress level. Distress levels are:

- 0 = Observations consistent with construction tolerances
- 1 = Observations consistent with the current service life of the wall and within standard maintenance practices
- 2 = Distress feature requires frequent and/or significant maintenance
- 3 = Advanced distress requiring immediate maintenance action, ongoing maintenance, or stabilization

The distress level factor is the average of the distress levels for each distress observation.

Condition factors can be used to identify segments within the wall with similar and elevated levels of distress. These segments should be delineated and further evaluated in the as-built design assessment detailed in Section 3.3

The condition factor for each distress zone is calculated using a weighted average of the score factor (ScF), frequency factor (FF), and distress level factor (DF) as shown in Equation 3-1. Weighting levels were selected to place emphasis on walls with significant distress levels and frequencies of distress without neglecting general overall distress.

$$T, W, B = \frac{ScF + 2*FF + 3*DF}{6}$$ (Janacek and Fraser, 2012)

Equation 3-1

The condition factors will be combined with the as-built design factors as described in Section 3.4.

3.3 As-built design assessment

Current asset management systems reviewed in Section 2.5 provide multiple methods to assess the physical conditions of walls, and the CDOT (Hearn, 2003) study does recommend evaluating walls based on updated codes, though the examples are generally traffic safety related. An assessment of the as-built design will identify the probable failure modes for segments of a wall, and can be combined with the physical assessment to develop a wall rating as described in Section 3.4.

3.3.1 Selection of design sections

Initial sections for as-built design analysis are based on any unique combination of wall geometry and soil design parameters. The tallest section of wall within a unique section should be analyzed. When the wall height exceeds twenty feet, a mid-height analysis should be considered depending on the actual conditions being analyzed. Variables considered in determining unique sections are:

Geometry

- Wall Height
- Top Slope
- Toe Slope
- L/H
- Surcharge Conditions
- Complex Geometry

Soil Parameters (each zone)

- Unit Weight
- Friction Angle
- Cohesion
- Foundation Layering

3.3.1.1 Geometric variable

Geometric variations for as-built design analysis are refined based on the measurements from the physical assessment. If there are significant changes in a given variable over a short length of wall (less generally less than 50 feet), sensitivity of the range should be evaluated (assuming other variables within the range are constant) to determine if an upper/lower bound analysis or mean/standard deviation analysis is appropriate.

Figure 3-18 shows the sensitivity of the FS for sliding to geometric variations for three retained and foundation soil combinations. The calculations for Figure 3-18 used an example wall that has baseline values of H= 20ft, L/H = 70%, and β=0. For Figure 3-18.A, H is varied from 0 feet to 95 feet, while L/H and β are held constant. This variation was performed with the retained and foundation soils having equal values of 15 degrees, 25 degrees, and 30 degrees. Figure 3-18.B and Figure 3-18.C repeat this process for variations in β and L/H respectively.

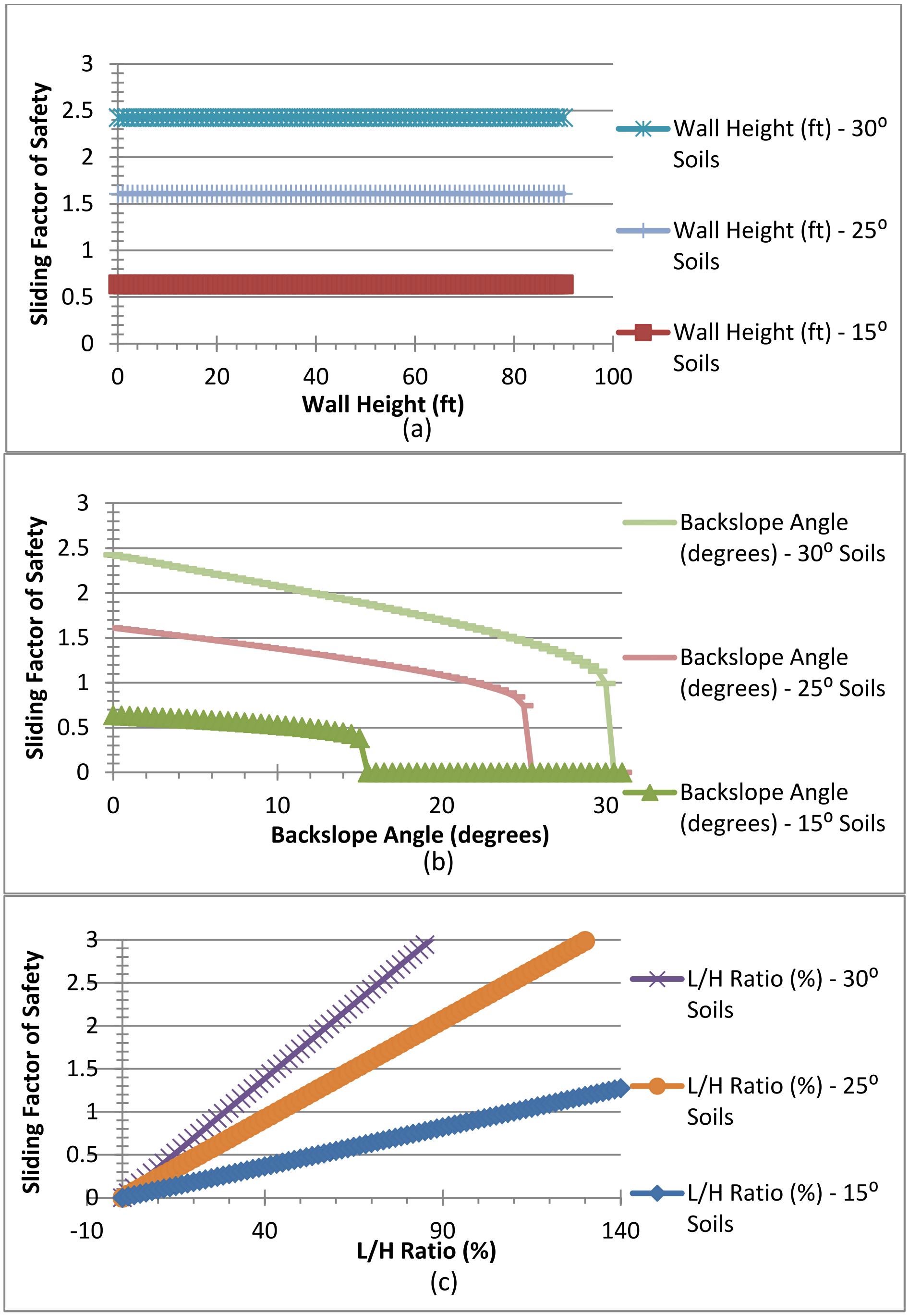


Figure 3-18: Sliding FS vs. Geometric Variations for baseline H= 20ft, L/H = 70%, and β=0: (a) Sliding FS vs Wall Height (ft), (b) Sliding FS vs Backslope Angle (degrees), and (c) Sliding FS vs L/H Ratio (%)

Observations from Figure 3-18 are:

- FS for sliding is not affected by wall height
- FS for sliding is more sensitive to changes in L/H and β at higher shear strengths for the foundation and retained zones

Additional design sections are added as needed to analyze segments of distress delineated in the condition factor analysis from Section 3.2.4.

3.3.1.2 Soil variables

As-built soil parameters must be developed independent of parameters used in the original calculations, or shown on the plans. Case studies of failures have documented that different materials were used in construction of a wall than used in the design analysis (Harpstead, Schmidt, & Christopher, 2010). For foundation soils and walls in cut conditions, the original geotechnical report may contain appropriate information to estimate the native soil properties. Review data from shear tests and logs of borings in the geotechnical report directly and identify the location of borings along the wall. Quality control and quality assurance documents from the original construction of the wall may contain proctor results, material gradations, and classification testing for fill used in the foundation, reinforced and retained zones. Nuclear density gauge test results may also be available.

Many construction testing records are held for a minimum of seven years; however they may be discarded after that; therefore field exploration may be required to obtain as-built soil information.

(1) Field and laboratory program

The field investigation should be designed to identify the required design parameters in both distressed and adequately performing areas. Specifics of a field investigation and laboratory testing must be developed in conjunction with a local geotechnical engineer familiar with the geologic unit(s) the walls are constructed in. The following paragraphs provide baseline recommendations for a field and laboratory investigation based on the Authors experience.

Continuous sampling is recommended to identify lenses of material that could impact stability of the wall. Access for drilling equipment will likely be limited and difficult, and considering that construction techniques and materials are generally consistent along a wall, only a few borings are typically needed (approximately one per 800 linear foot of wall, with a minimum of two per wall). Continuous sampling with a shelby tube is recommended for clayey soil, and continuous standard penetration test sampling with a split spoon is recommended for granular soils. Tests performed on the samples include:

- Atterberg limits (liquid limit and plastic limit)
- Unit weight
- Moisture content

- Gradation (generally the #4, #40, and #200 sieves)
- Hygrometer
- CU - Consolidated Undrained triaxial tests with pore pressure measurements (clayey soils)
- DS - Direct shear test (granular soils, anisotropic bedrock)
- One-dimensional consolidation test (foundation soils)

Test frequency is dependent on the geology, construction techniques, and as-built data available. Generally each boring should have multiple Atterberg limit, unit weight, moisture content, and gradation tests. Hygrometer testing should be performed with 10% to 25% of the gradation tests in clays with naturally low silt contents, and in 25% to 50% of clays from or near alluvial deposits. CU tests are assigned on a project wide basis to evaluate the geologic units and fill soils encountered. Typically highway related MSE walls have adequate shear strength tests for the native foundation soils in the original geotechnical report. This data should be reviewed to determine if additional, undisturbed CU tests are required. CU tests for fill soils are performed on samples remolded to the average unit weight and moisture content for the soil type being tested. Testing remolded samples can reduce impacts of sample quality and is consistent with the fully softened shear strength approach described in Section 2.3.4.1(2). For clay soils, the fully softened shear strength is utilized in the design assessment calculations. DS tests should also be performed at average field density at overburden pressure ranges for remolded granular soils. Undisturbed samples of anisotropic bedrock such as shale should be tested using DS as well. If cement stabilized sand is encountered in the reinforced zone or if the MSE wall is founded on or near bedrock, interface shear tests should be considered. One-dimensional consolidation tests from samples beneath the wall can be compared to one-dimensional consolidation tests from an unmodified area outside of the wall (if available) to estimate settlement characteristics of the structure. In-situ testing such as cone penetrometer, flat plate dilatometer, and pressure meter testing can be used to develop additional information on the soils and provide more data points for statistical analysis. In-situ tests can be especially beneficial in alluvial deposits where undisturbed sampling has a lower chance of success.

(2) Soil property selection

Figure 3-19 illustrates the impact of variations of soil properties on the sliding factor of safety for a wall with H= 20ft, L/H = 70%, and β=0. Baseline foundation and retained zone soils are held at γ = 125 pcf, ϕ = 25^0, and c = 0. MSE reinforced fill is modeled at γ = 125 pcf. Unit weight and friction angle variations are analyzed with all other variables held constant. Note that when cohesion is varied for the foundation zone only in this figure and the foundation friction angle is held at 25 degrees. Each soil property shown in Figure 3-19 is varied with the remaining variables held constant at the values described above.

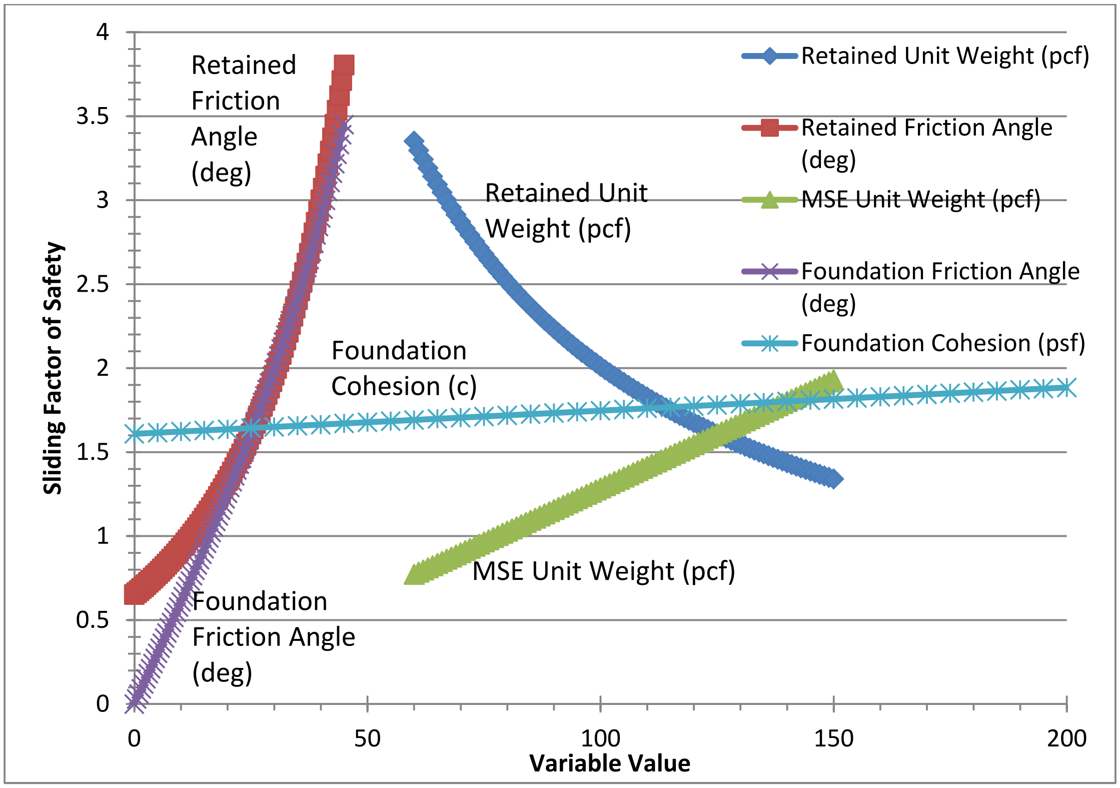


Figure 3-19: Variations of Soil Parameters for Fixed Wall Geometry

Figure 3-20 illustrates variations considering changes in c and ϕ in both the foundation and retained zones, and the impact of tension crack development.

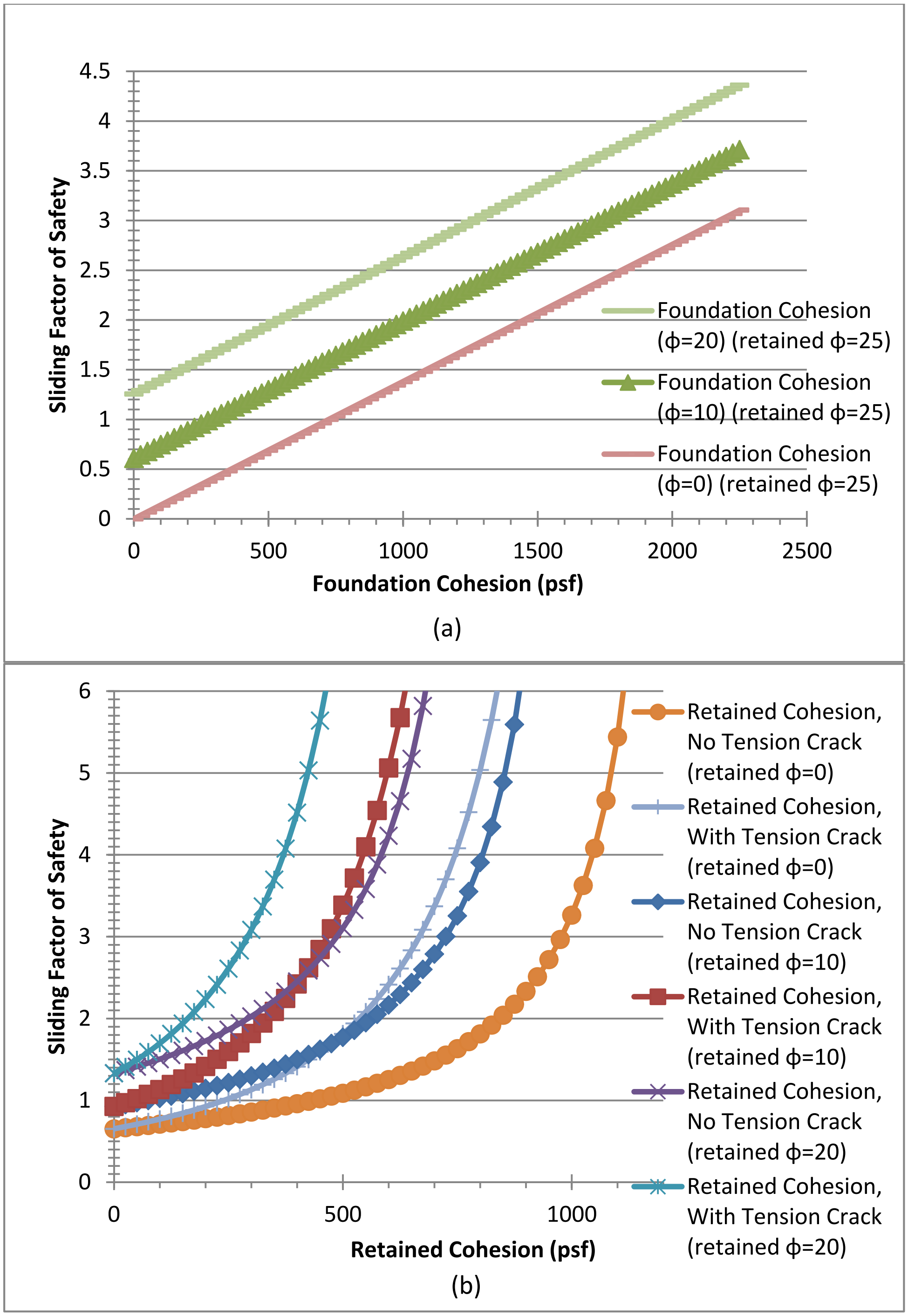


Figure 3-20 - Variations of Cohesion & Friction Angle for Fixed Wall Geometry: (a) Sliding FS vs Foundation Cohesion; (b) Sliding FS vs Retained Cohesion

Observations from Figure 3-19 and Figure 3-20 are:

- Sliding FS is the most sensitive to retained zone and foundation zone friction angles
- Foundation cohesion has a linear influence on sliding FS
- Sliding FS is more sensitive to tension crack formation in cohesive retained zone soils at higher levels of cohesion and/or lower friction angle components
- The increase in FS for sliding with cohesion increase is asymptotic within the applicable range of analysis (see discussion below)
- As the friction angle component increases, the asymptotic increase in FS for sliding occurs at lower values of cohesion
- Sliding FS is less sensitive to lower values of retained zone cohesion

Figure 3-18, Figure 3-19, and Figure 3-20 illustrate the rate of change in sliding factor of safety for many of the wall variables. Similar evaluations of bearing capacity and eccentricity indicate that variations in shear strength of the retained zone and foundation zone, the L/H ratio, unit weight of the retained/reinforced/foundation soils, and backslope have a higher rate of change in factor of safety than other variables. A high rate of change to the factor of safety indicates that the failure mode is sensitive to variations in a given variable. This analysis is consistent with parametric studies by

Chalermyanont and Benson (Chalermyanont & Benson, 2005); with the exceptions that calculations for the figures above separate reinforced zone and retained zone unit weights, and lower values of friction angle were incorporated.

Soil property selection considers available original project information, physical assessment data, and field/laboratory investigation data. Field measurements improve the certainty of backslope geometry. Review of the original wall manufacturers/designers drawings will generally increase the certainty of L/H. Field testing reports, materials submittals, and inspectors reports are some of the applicable sources to identify the reinforced zone fill placed density (if in-situ density reports are available) or material type (allowing for a more accurate estimate of the placed unit weight). The project geotechnical report, original soil testing reports, field/laboratory investigation, and published correlations are used to develop a range of values for the retained and foundation soil properties due to their sensitivity and variability. The lower limit of this range is termed the low range value (LRV) and the upper limit is the high range values (HRV).

3.3.2 Analyses of design sections

Assuming that values for the backslope, L/H, and reinforced zone unit weight have been narrowed as described above for a specific design section, analysis is performed holding geometric and reinforced zone parameters constant, and applying the LRV and HRV to the stability equations developed in

FHWA's ASD manual (Elias, Christopher, & Berg, 2001) in accordance with the boundary cases shown in Table 3-2.

Table 3-2: Boundary Condition Combinations

Analytical Case	Retained Zone		Foundation Zone	
	γ_f	ϕ_f	γ_b	ϕ_b
Lower Bound	HRV	LRV	LRV	LRV
Upper Bound	LRV	HRV	HRV	HRV

The example below follows this procedure for a wall with H= 20ft, L/H =0.7, β=0, and γ_r = 125 pcf. The foundation and retained zone LRV and HRV values are:

- Foundation Zone
  - LRV: γ_b = 120 pcf, ϕ_b = 22^0
  - HRV: γ_b = 130 pcf, ϕ_b = 28^0
- Retained Zone
  - LRV: γ_f = 115 pcf, ϕ_f = 20^0
  - HRV: γ_f = 125 pcf, ϕ_f = 25^0

Lower bound case:

Equation 2-3

$$k_{af} = tan^2\left(45 - \frac{\phi_f}{2}\right) = tan^2\left(45 - \frac{20}{2}\right) = 0.4903$$

Equation 2-5

$$F_T = \frac{1}{2}\gamma_f h^2 k_{af} = \frac{1}{2}(125pcf)(20ft)^2(0.4903) = 12{,}257.26\ lbs/ft$$

Equation 2-7

$$V_1 = \gamma_r HL = (125pcf)(20ft)(14ft) = 35{,}000\ lbs/ft$$

Equation 2-9

$$\mathrm{e} = \frac{F_T(cos\beta)\frac{h}{3}+F_S(cos\beta)\frac{h}{2}-F_T(sin\beta)\frac{L}{2}-F_S^*(sin\beta)\frac{L}{2}-V_2\frac{L}{6}}{V_1+V_2+F_T sin\beta} = \frac{\left(12{,}257.26\frac{lbs}{ft}\right)\left(\frac{20\ ft}{3}\right)}{35{,}000\frac{lbs}{ft}} = 2.33$$

Equation 2-10

$$\sigma_v = \frac{qL+V_1+V_2+F_T sin\beta}{L-2e} = \frac{12{,}257.26\ lbs/ft}{14ft-2(2.33ft)} = 3751.11\ psf$$

Equation 2-11

$$q_u = c_b N_c + 0.5L'\gamma_b N_\gamma = 0.5(14ft - 2 * 2.33ft)(120pcf)(7.13) =$$

$$3{,}995.65\ psf$$

Equation 2-13

$$P_R = (V_1 + V_2 + F_T sin\beta)\mu = 14{,}140.92\ lbs$$

Resulting in:

$$FS_{Sliding} = \frac{F_T}{P_R} = 1.15$$

$$FS_{Bearing} = \frac{\sigma_V}{q_u} = 1.07$$

$$\frac{L}{6} = 2.33ft = e; eccentricity\ is\ OK$$

$$\mathrm{FS}_{OT} = \frac{(F_T + F_S^*)(sin\beta)L + V_2\frac{2L}{3} + V_1\frac{L}{2}}{(F_T\frac{h}{3} + F_S\frac{h}{2})(cos\beta)} = \frac{\left(35{,}000\frac{lbs}{ft}\right)\left(\frac{14\,ft}{2}\right)}{\left(12{,}257.26\frac{lbs}{ft}\right)\left(\frac{20\,ft}{3}\right)} = 3.0$$

The same calculations are performed using the upper bound case values. The results of both cases are summarized in Table 3-3 along with the factors of safety calculated using average soils parameters (mid-point of each range of values).

Table 3-3: Example Wall External Stability Safety Factors

Analytical Case	FS Sliding	FS Bearing	FS Overturning	e (ft) (e < L/6)
Lower Bound	1.15	1.07	3	2.33 (OK)
Upper Bound	1.99	3.39	3.94	1.78 (OK)
Average Input Values	1.52	1.91	3.43	2.04 (OK)

For use in the rating system described in Section 3.4, a weighted average of the upper and lower bound cases is used to calculate the assessment value factor of safety (FS_{AV}) for each failure mode.

$$FS_{AV} = \frac{F_{upper} + 2 \times F_{lower}}{3} \qquad \text{Equation 3-2}$$

The FS_{AV} is used to develop the design factor values described in Section 3.3.3. For this example, the calculated FS_{AV} for each failure mode is:

Table 3-4: Example Wall External Stability Assessment Value Factor of Safety

Analytical Case	FS Sliding	FS Bearing	FS Overturning	e (ft) (e < L/6)
Assessment Value	1.43	1.84	3.31	2.15 (OK)

3.3.3 Application of as-built design assessment

Similar to the condition factors, the as-built design assessment is reduced to design factors for each failure mode. Design factor values range from 1 to 3 are based on comparison FS_{AV} to factor of safety values that correlates to a predetermined probability of failure. To determine the appropriate boundaries for each rating, a probability of failure (P_f) analysis was performed on an MSE wall system using the methods outlined by Duncan (Duncan, 2000) in Section 2.4 Published coefficients of variation from Table 3-5 were used to estimate the standard deviation for each parameter in accordance with Equation 2-30. Eleven variables were considered in the probability of failure analysis as shown in Table 3-5 and Table 3-6.

Table 3-5: Condensed* Summary of Coefficient of Variation for External Stability Analysis (Duncan, 2000 and USACE, 2006)

	Reinforced Zone	Retained Zone	Foundation Zone
	Coefficient of Variation		
Cohesion (psf)	N/A	0.40	0.40
Friction Angle (deg)	N/A	0.13	0.13
Unit Weight (pcf)	0.07	0.07	0.07
Surcharge (psf)	0.25		

*see Table 2-7 for detailed references.

Proposed standard deviations for additional variables are provided in Table 3-6. These proposed values are based on the Authors experience in construction and assessment of retaining walls.

Table 3-6: Proposed Standard Deviations

Variable	Standard Deviation	Unit
L/H	5	%
Back Slope, β	2	Degrees
Broken Back Distance	10	Feet

Table 3-7 provides an example set of most likely values for wall design parameters (note that all of these parameters except for reinforced zone friction angle and cohesion are variables in the Pf calculations).

Table 3-7: Example Pf Most Likely Values

	Reinforced Zone	Retained Zone	Foundation Zone
Cohesion (psf)	0	40	200
Friction Angle	34	22	22
Unit Weight (pcf)	115	125	125
L/H (%)	70		
Back slope angle (β)	14.04	Surcharge (psf)	250
Broken Back Slope Distance (ft)	20		

Since 22 iterations of factor of safety calculations had to be run for each of the four failure modes analyzed, a Microsoft Excel spreadsheet was created to perform these calculations. Tables 3-8 through 3-11 provide example calculations for Duncan's Pf methodology using the values from Tables 3-5 through 3-7.

Table 3-8: Example Sliding Factor of Safety Pf Analysis

Variable	Sliding Factor of Safety (FS)			Δ FS
	MLV	+1σ	-1σ	
γ_r	1.010	1.062	0.958	0.103
c_f	1.010	1.037	0.984	0.054
ϕ_f	1.010	1.131	0.903	0.228
γ_f	1.010	0.952	1.075	-0.123
c_b	1.010	1.073	0.946	0.127
ϕ_b	1.010	1.135	0.890	0.245
γ_b	1.010	1.010	1.010	0.000
L/H	1.010	1.064	0.955	0.109
B	1.010	1.217	1.042	0.175
Broken back distance	1.010	0.995	1.089	-0.093
Surcharge	1.010	0.972	1.051	-0.079
Standard Deviation for Sliding Factor of Safety				0.231
Coefficient of Variation for Sliding				0.229
Most Likely Value for Sliding Factor of Safety				1.010
Reliability Index, β_{LN}				-0.069
Probability of Failure, P_F				52.8%

Table 3-9: Example Bearing Capacity Factor of Safety Pf Analysis

Variable	Bearing Capacity Factor of Safety (FS)			Δ FS
	MLV	+1σ	-1σ	
γ_r	1.145	1.180	1.100	0.078
c_f	1.145	1.190	1.100	0.087
ϕ_f	1.145	1.350	0.940	0.407
γ_f	1.145	1.050	1.250	-0.199
c_b	1.145	1.390	0.900	0.481
ϕ_b	1.145	1.410	0.850	0.558
γ_b	1.145	1.180	1.110	0.076
L/H	1.145	1.400	0.880	0.518
B	1.145	1.600	1.220	0.381
Broken back distance	1.145	1.180	1.290	-0.112
Surcharge	1.145	1.020	1.280	-0.264
Standard Deviation for Bearing Capacity Factor of Safety				0.562
Coefficient of Variation for Bearing Capacity				0.491
Most Likely Value for Bearing Capacity Factor of Safety				1.145
Reliability Index, β_{LN}				0.059519
Probability of Failure, P_F				47.6%

Table 3-10: Example Overturning Factor of Safety Pf Analysis

Variable	Overturning Factor of Safety (FS)			Δ FS
	MLV	+1σ	-1σ	
γ_r	1.882	1.990	1.780	0.212
c_f	1.882	1.930	1.840	0.086
ϕ_f	1.882	2.090	1.690	0.400
γ_f	1.882	1.790	1.990	-0.198
c_b	1.882	1.880	1.880	0.000
ϕ_b	1.882	1.880	1.880	0.000
γ_b	1.882	1.880	1.880	0.000
L/H	1.882	2.100	1.670	0.434
B	1.882	1.810	1.950	-0.141
Broken back distance	1.882	1.910	2.040	-0.133
Surcharge	1.882	1.780	1.990	-0.205
Standard Deviation for Overturning Factor of Safety				0.360
Coefficient of Variation for Overturning				0.191
Most Likely Value for Overturning Factor of Safety				1.882
Reliability Index, β_{LN}				3.237
Probability of Failure, P_F				0.1%

Table 3-11: Example Limiting Eccentricity Probability Analysis

Variable	Eccentricity Distance (ft)			Δ e
	MLV	+1σ	-1σ	
γ_r	3.465	3.270	3.690	-0.422
c_f	3.465	3.380	3.550	-0.173
ϕ_f	3.465	3.080	3.890	-0.816
γ_f	3.465	3.660	3.270	0.396
c_b	3.465	3.470	3.470	0.000
ϕ_b	3.465	3.470	3.470	0.000
γ_b	3.465	3.470	3.470	0.000
L/H	3.465	3.260	3.690	-0.426
β	3.465	3.580	3.360	0.219
Broken back distance	3.465	3.330	3.270	0.062
Surcharge	3.465	3.680	3.250	0.435
Standard Deviation for Eccentricity				0.603
Coefficient of Variation for Eccentricity				0.174
Most Likely Value for Eccentricity				3.465
Eccentricity Ratio [(L/6)/MLV]				0.673
Reliability Index, β_{LN}				-2.205
Probability of Exceeding, P_E				98.6%

To further evaluate the coefficients of variation for each failure mode, a partial parametric study was performed by evaluating changes in parameters identified to have a greater impact on the factors of safety. Two back slope conditions, two L/H ratios, surcharge/no surcharge, high friction angle soils, cohesive foundation soils, and low friction angle soils were used as analytical cases as shown in Table 3-12.

Table 3-12: Partial Parametric Study of Calculated Coefficients of Variations (V) for Wall Failure Modes

Geometric Variations			Sliding V			Bearing V		
Back Slope	L/H (%)	q	ϕ_f=30, ϕ_b=30	ϕ_f=17, ϕ_b=0, c_b=750 psf	ϕ_f=15, ϕ_b=15	ϕ_f=30, ϕ_b=30	ϕ_f=17, ϕ_b=0, c_b=750 psf	ϕ_f=15, ϕ_b=15
Flat	70	N	0.255	0.421	0.197	0.542	0.416	0.360
Flat	70	Y	0.252	0.419	0.192	0.560	0.434	0.480
Flat	100	N	0.251	0.418	0.191	0.520	0.405	0.244
Flat	100	Y	0.247	0.417	0.186	0.521	0.405	0.253
13º,BB@ 30 ft	70	N	0.259	0.455	0.214	0.556	0.438	0.461
13º,BB@ 30 ft	70	Y	0.254	0.449	0.207	0.586	0.486	0.715
13º,BB@ 30 ft	100	N	0.272	0.4779	0.237	0.524	0.414	0.278
13º,BB@ 30 ft	100	Y	0.264	0.469	0.226	0.529	0.418	0.311
Geometric Variations			Overturning V			Eccentricity V		
Back Slope	L/H (%)	q	ϕ_f=30, ϕ_b=30	ϕ_f=17, ϕ_b=0, c_b=750 psf	ϕ_f=15, ϕ_b=15	ϕ_f=30, ϕ_b=30	ϕ_f=17, ϕ_b=0, c_b=750 psf	ϕ_f=15, ϕ_b=15
Flat	70	N	0.236	0.192	0.188	0.200	0.147	0.141
Flat	70	Y	0.232	0.187	0.183	0.194	0.140	0.134
Flat	100	N	0.215	0.165	0.160	0.193	0.137	0.132
Flat	100	Y	0.209	0.158	0.153	0.188	0.130	0.124
13º,BB@ 30 ft	70	N	0.212	0.167	0.166	0.221	0.154	0.148
13º,BB@ 30 ft	70	Y	0.209	0.163	0.161	0.207	0.143	0.139
13º,BB@ 30 ft	100	N	0.203	0.158	0.159	0.266	0.170	0.160
13º,BB@ 30 ft	100	Y	0.197	0.151	0.152	0.234	0.151	0.145

The coefficients of variation from Table 3-12 were then grouped by the different baseline variables to identify categories of analysis that would result in standard deviations for coefficients of variation of approximately 10% for all failure modes. This analysis resulted in separating coefficients of variation into friction angle only soils (effective stress and fully softened conditions) and combined cohesion/friction (total stress and intermediate conditions). These results are summarized in Table 3-13.

Table 3-13: Summary of Coefficient of Variation (V) Categories

Failure Mode	Friction Angle Only		Cohesion/Friction Angle	
	Average V	Standard Deviation of V	Average V	Standard Deviation of V
Sliding	0.230	0.030	0.440	0.030
Bearing	0.465	0.136	0.427	0.027
Overturning	0.190	0.028	0.168	0.014
Eccentricity	0.177	0.042	0.146	0.012

The average coefficients of variation from Table 3-13 were then used to calculate factors of safety for each mode that correspond to a target probability of failure. For the purpose of this program, probabilities of failure of 10%, 25% and 40% were selected based on the descriptions provided in Table 2-9. The limiting eccentricity was evaluated using the probability of exceeding L/4. The resulting design factors and associated factors of safety (based probabilities of failure development described in this section and rounded up to the nearest 0.05) are summarized in Table 3-14. These values are dependent on the coefficients of variations and standard deviations selected for the individual parameters (Tables 3-5 and 3-6). Changes to the coefficients of variation and/or standard deviations will change the equivalent factors of safety shown in Table 3-14.

Table 3-14: Design Factor

Factor	Global* FS	Pf	Equivalent Sliding FS	Equivalent Bearing FS	Equivalent Overturning FS	Equivalent Eccentricity (e/L)
Friction Angle Only Soils						
0	1.5	10%	1.45	2.40	1.35	0.194
1	1.3	25%	1.25	1.70	1.20	0.221
2	1.1	40%	1.15	1.35	1.10	0.242
3	<1.1	>40%	<1.15	<1.35	<1.10	>0.242
Cohesion/Friction Angle Soils						
0	1.5	10%	1.95	1.95	1.30	0.207
1	1.3	25%	1.5	1.50	1.15	0.228
2	1.1	40%	1.25	1.25	1.1	0.243
3	<1.1	>40%	<1.25	<1.25	<1.15	>0.243

*Global FS limits based on AASHTO/TxDOT recommended values (1.5 and 1.3 respectively), and a selected value of 1.1, not a probability of failure limit.

3.4 Rating System

Figure 3-21 details the wall rating development. Wall ratings are calculated based on combinations of the physical assessment and the as-built design assessment, general time rate of the most likely failure mode, and a consequences of failure factor. Additional factors can be linearly added to the rating based on owner driven factors to aid in prioritizing wall maintenance expenditures.

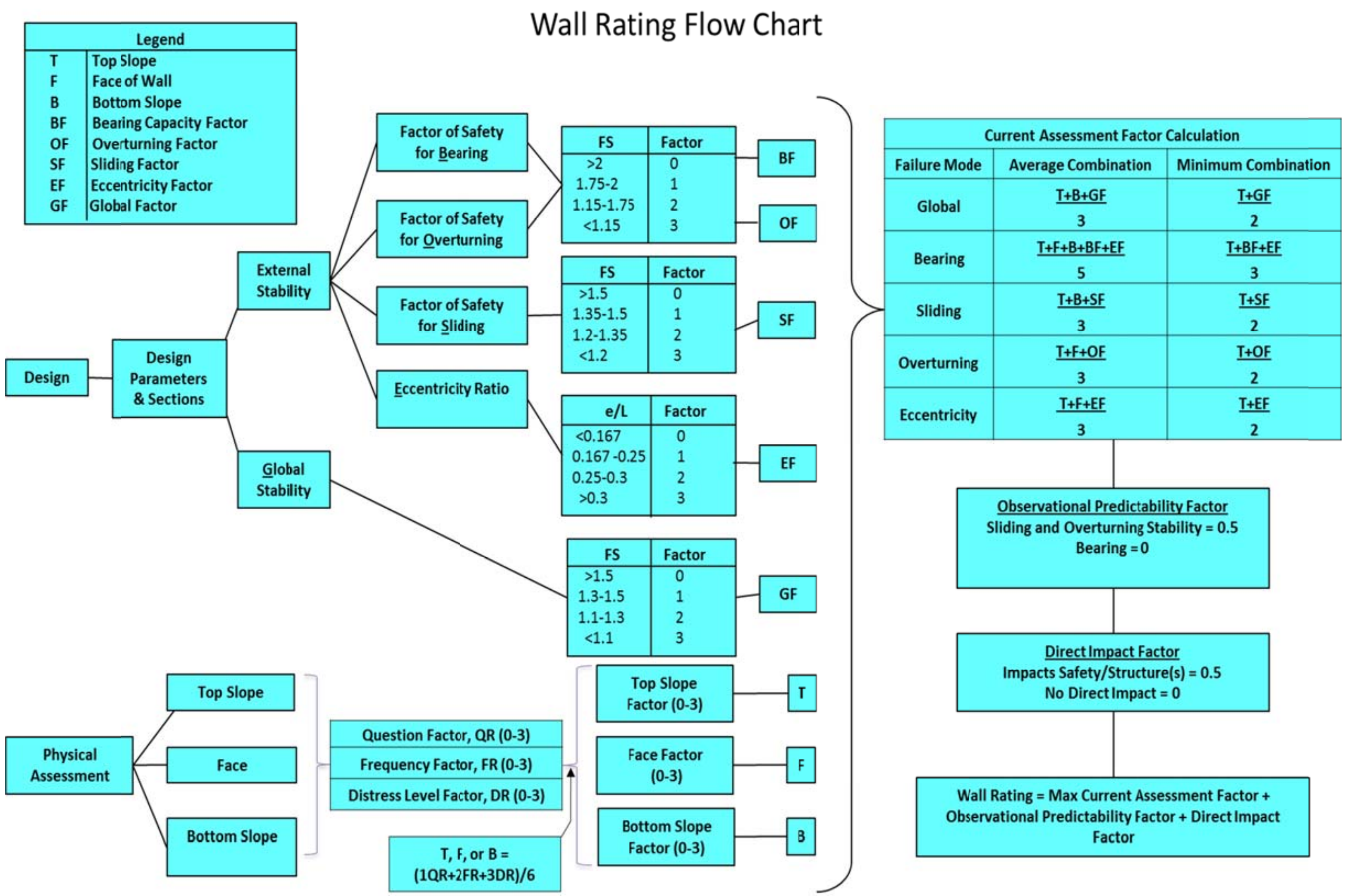


Figure 3-21: Wall Rating Flow Chart (Janacek & Fraser, 2012)

The current assessment factor (CAF) is systematic approach to combining the condition factors and design factors to compare predicted performance with current performance. Conceptually, deformations/distress occurring in the field is related to the stress-strain behavior of soil as well as the development of active earth pressure. Due to the nature of many of the soils (specifically clays) encountered in the foundation and retained zones of the wall; the deformation is not only a function of time (pore pressure dissipation) but of cyclical wetting and drying. These time/environment dependent factors help explain why calculations that use long term strength estimations may indicate a factor of safety less than one for a given failure mode, implying the wall has failed, when physical inspection of the wall indicates it has not failed. Secondary distress features may develop due to or in conjunction with primary deformation distress, and additional loads imparted due to new construction will start the stress-strain response movements anew. Once established, unless there is a geometric change to the wall surroundings, or additional information that modifies the soil properties used in the as-built design, the design factor should remain constant. Condition factors represent the current distress locations and magnitudes. The condition factors and design factors are combined as shown in Table 3-15. The combination(s) with the highest CAF are considered the likely failure mode(s) at the time of that physical assessment.

Table 3-15: Current Assessment Factor Calculation (after Janacek & Fraser, 2012)

Failure Mode	Global	Sliding	Bearing	Overturning	Eccentricity
Avg.	$\frac{T+B+GF}{3}$	$\frac{T+B+SF}{3}$	$\frac{T+W+B+BF+EF}{5}$	$\frac{T+W+OF}{3}$	$\frac{T+W+EF}{3}$
Min.	$\frac{T+GF}{2}$	$\frac{T+SF}{2}$	$\frac{T+BF+EF}{3}$	$\frac{T+OF}{2}$	$\frac{T+EF}{2}$

Where:

T: Top Slope Factor W: Wall Face Factor

B: Bottom Slope Factor GF: Global Stability Factor

SF: Sliding Factor BF: Bearing Factor

OF: Overturning Factor EF: Eccentricity Factor

The average combination shown in Table 3-15 was selected based on the theoretical failure angle calculations and common failure surface diagrams shown in Section 2. The minimum combination accounts for distress features that may not be apparent due to physical characteristics such as cement stabilized sand (CSS) reinforced fill (which may limit wall face distress due to the rigidity of the CSS) and behavioral features such as in local shear when no heave at the toe is typically visible. The CAF value is the highest value resulting from the combinations in Table 3-15. CAF values range from the best case of 0 to the highest probability of failure and visual distress of a 3.

Observational Predictability Factor (OPF) accounts for the relative time frame of the failure mode. Assuming that a retaining wall has been in service for several years, it is likely that global stability failures, bearing capacity failures, and eccentricity related problems will develop over a relatively longer period of time (with the exception of catastrophic events such as waterline breaks, or similar), allowing for distress to be monitored and prioritization to be based on monitoring measurements. The above failure modes are given an OPF of 0 until monitoring indicates that they should be the maximum value of 0.5. Sliding failures, overturning failures, and combined failures have been observed to occur within a limited timeframe; therefore these failure modes are given an OPF of 0 when the design factor is less than 2 and a 0.5 when the design factor is greater than or equal to 2.

Direct Impact Factor (DIF) takes into account if the failure has an immediate consequence to safety or surrounding structures. DIF will apply to walls that have a design factor greater than or equal to two, and that have a travel lane (vehicular or pedestrian) or structure within two wall heights behind the face of wall or two wall heights in front of the wall, are at a bridge abutment, or if failure of the wall would damage critical utilities.

The wall rating is the sum of the CAF, OPF, and DIF. Owners can input additional components with values from 0 to1 to meet the specific needs of their system. Table 3-16 provides a summary of recommended actions based on the wall rating.

Table 3-16: Wall Rating Recommended Actions (after Janacek and Fraser, 2012)

Rating	Recommended Action	Recommended Frequency for Assessment	Reevaluate Frequency of Assessment
0	Standard maintenance	Yearly	5 years
1	Standard maintenance	Yearly	5 years
2	Targeted maintenance program	6 to 9 months, accounting for seasonal changes	5 years
3	Frequent preventative maintenance program. Install monitoring devices. Consider a detailed wall investigation	Quarterly, accounting for seasonal changes	Post Stabilization or at 5 years
4	Immediate preventative maintenance. Install monitoring devices. Perform detailed wall investigation and evaluation of stabilization	Monthly until stabilized	Post Stabilization

Chapter 4

Conclusions and Recommendations

MSE walls are a large and growing asset within our nation's infrastructure; however MSE wall failures are becoming more common (Bachus & Griffin, 2010). Asset management of walls can aid in planning and budgeting for wall repairs (Anderson, Alzamora, & DeMarco, 2008) and potentially extend the service life of a wall, which represents additional return on investment for the owner (Hearn, 2003). Existing wall assessment programs rate walls relative to their physical distress and separately for their consequences of failure. ASD and LRFD design methods do not adequately account for the up to 40% variability that can occur in the over a dozen factors influencing wall performance. To refine the rating of a wall, this thesis set forth a methodology that uses reliability based limits of factor of safety and zoned field assessments that target key areas of distress. This methodology accounts for soils that have time dependent and environmentally driven variations. The various "factors" within the system inherently identify maintenance action items and can assist in the development of conceptual stabilization plans.

Some observations of note:

- Soil variables have the greatest impact on wall performance.

- Peak friction angle values are often used for design; however these values may overestimate the actual field resistance (Kim & Salgado, Load and Resistance Factors for External Stability Checks of Mechanically Stabilized Earth Walls, 2012).
- Stress path, significantly effects soil strength (Lambe, 1997).
- Cyclical wetting and drying of compacted clays and stiff, fissured clays impacts the long term shear strength of these soils (Wright, Zornberg, & Aguettant, 2007).
- Probabilistic analysis for geotechnical structures commonly target probabilities of failure of 0.13% to 0.023% (Kim & Salgado, Load and Resistance Factors for External Stability Checks of Mechanically Stabilized Earth Walls, 2012); however based on the probability of failure analysis and correlation to factor of safety performed in this thesis, the target probabilities of failure would require factors of safety above the minimum recommended values (Elias, Christopher, & Berg, 2001).

Recommendations for future research are:

- Monitoring of long term stress and strain development
    - Several studies have evaluated actual pressures on retaining walls; however a long term study monitoring

pressure and strain/displacement of the wall system would provide a better understanding of time dependent effects on the wall system. To shorten the actual duration of this program, multiple walls from different time periods could be instrumented for one to two years and their relative movements compared. This would require finding multiple walls in the same geology, constructed in similar fashions in order to provide an unbiased comparison, or a large enough sampling of walls that the time dependent impacts could be isolated.

- Incorporation of internal assessment
    - Internal stability and performance of the internal wall structure is more difficult to evaluate post-construction due to access. Areas of interest include corrosion of metallic reinforcement, degradation of reinforced zone fill, infiltration of fines, and performance of drainage components.
- Evaluation of low frequency cyclical loading on foundation soils
    - Wetting and drying cycles in the retained zone change the moisture content of clay soils, likely resulting in changes applied pressure on the wall. While this is not a high

frequency change in loading, the impact on interface shear should be evaluated. This loading can be studied using direct shear methods and cycling the shearing and normal forces (to account for sloping backfill) based on the magnitude of load change for shrink-swell cycles.

- Incorporation of stress-path considerations
  - Construction sequencing of MSE walls has been observed to vary from construction of the reinforced zone ahead of the retained zone to construction of the retained zone that is then cut near vertical for installation of the reinforced zone. This construction sequence impacts MSE wall deformation due to the timing of wall construction relative to active pressure development. Development of the soil loads and resistances can be modeled using stress path testing and analysis.
- Incorporation of swell pressure variations
  - The probability of failure analysis undertaken in this thesis can be modified to incorporate variations due to lateral swelling of expansive clays.

- Incorporation of preventative maintenance costs and costs for failure to develop cost-benefit analysis for various maintenance/stabilization activities
- Effect of eccentricity greater than L/6 on sliding resistance due to asymmetric stress distribution
    - o High eccentricity values can create significant variations in applied pressure across the width of the foundation. The resulting difference in normal force reacting on the foundation creates variable friction resistance along the foundation and the potential for stress concentrations leading to progressive failure. Laboratory testing using direct shear methods with varying normal force can be used to develop variations of resistance for equal strains at different normal forces. A computer model could then be created to evaluate the impact of friction resistance strain compatibility based on eccentricity levels.

Appendix A

Visual Assessment Checklist

Wall and Slope Distress Visual Indicators (after Janacek & Fraser, 2012)

Slopes or Slope / Paving Above Wall			
Observed			**CHECKS (characteristics)**
Yes	No	NA	
☐	☐	☐	1. Cracks in slope/pavement (circle size: marble, golf ball, tennis ball, football, _________)
☐	☐	☐	2. For cracks, circle: new/previous, sealed/unsealed, previously treated Approximate distance from wall face:________________ ft.)
☐	☐	☐	3. Formation of shallow surface slides, depressions near top, bulging near bottom?
☐	☐	☐	4. New or increased dip in paving, curb, gutter line or slope?
☐	☐	☐	5. Landscape areas (Yes/No) – Working irrigation (Yes/No)
☐	☐	☐	6. Ponding water?
☐	☐	☐	7. Joint separation in pavement and shoulder joint pavement?
☐	☐	☐	8. Erosion of soil on slope or around flatwork, flumes, guardrails, light fixture base, electrical communication bores?
☐	☐	☐	9. Soft and / or wet soil (greener grass, ruts)?
☐	☐	☐	10. Washout areas, silt buildup?
☐	☐	☐	11. Signs, structures or rails next to wall (plumb, leaning to/away from wall face ___inches over ___ft.)
Wall Face			
Observed			CHECKS (characteristics)
Yes	No	NA	
☐	☐	☐	12. Bulging of panels?
☐	☐	☐	13. New or increased cracks in panels?

☐	☐	☐	14.	Wall leaning? Direction?
☐	☐	☐	15.	Water runoff through wall face (circle: contains sand/gritty, or muddy)
☐	☐	☐	16.	New openings or changes in the gaps between the panels (either closer to farther apart?)
☐	☐	☐	17.	Contact and/or spalling of the panels?
In Front of Wall or Slope				
Observed				CHECKS (characteristics)
Yes	No	NA		
☐	☐	☐	18.	Heaving of flatwork / pavement or soil?
☐	☐	☐	19.	Crushing / cracking / spalling of pavement / sidewalk (or panel where it ties into the pavement?)
☐	☐	☐	20.	Cracking / lifting of nearby curbs?
☐	☐	☐	21.	Ponding water?
☐	☐	☐	22.	Presence of soil /silt, particularly along panel joints? (Height: Width:)

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Biographical Information

Noel Janacek.is a Registered Professional Engineer in the State of Texas. He was born in Houston, Texas in 1976. He received his Bachelors of Science degree from Texas A&M University in 1999. From 2000 to 2008 Mr. Janacek worked in the design-build construction industry, primarily for Hayward Baker, Inc, a specialty geotechnical contractor, where he specialized in earth retention, structural underpinning, and ground modification. In 2008 Mr. Janacek became and engineering consultant with Kleinfelder where he has worked extensively with forensic evaluation, remediation, and new construction of MSE walls. He enrolled as a distance education student in the civil engineering Master of Science program at the University of Texas at Arlington in the fall of 2008, and with the support of his family and the guidance of Dr. Anand Puppala, P.E. he graduated in December 2012.