Technical Report Documentation Page

1. Report No. FHWA/TX-05/0-4485-1	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle An Examination of Design Procedures for Single- and Multi-Tier Mechanically Stabilized Earth Walls		5. Report Date October 2004
		6. Performing Organization Code
7. Author(s) Wade N. Osborne and Stephen G. Wright		8. Performing Organization Report No. 0-4485-1
9. Performing Organization Name and Address Center for Transportation Research The University of Texas at Austin 3208 Red River, Suite 200 Austin, TX 78705-2650		10. Work Unit No. (TRAIS)
		11. Contract or Grant No. 0-4485
12. Sponsoring Agency Name and Address Texas Department of Transportation Research and Technology Implementation Office P.O. Box 5080 Austin, TX 78763-5080		13. Type of Report and Period Covered Technical Report 8/02 - 8/04
		14. Sponsoring Agency Code

15. Supplementary Notes
Project performed in cooperation with the Texas Department of Transportation and the Federal Highway Administration.

16. Abstract

Current AASHTO and FHWA design guidelines for MSE walls are with particular emphasis on multi-tiered wall systems. Current software including the MSEW 2.0 design software and the general-purpose slope stability software known as UTEXAS4 are also examined with emphasis placed on multi-tiered wall systems. Assumptions, differences, and issues with the present guidelines and software are identified. The impact of the assumptions and differences is examined through analyses of a number of hypothetical and actual single and multi-tiered MSE walls. Deficiencies and ambiguities in the current procedures for design of multi-tiered walls are identified and recommendations are provided.

17. Key Words	18. Distribution Statement
Mechanically stabilized earth walls, Retaining walls, Slope stability, Geotechnical engineering, Soil reinforcement	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161; www.ntis.gov.

19. Security Classif. (of report)	20. Security Classif. (of this page)	21. No. of pages	22. Price
Unclassified	Unclassified	202	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

Center for Transportation Research
The University of Texas at Austin

An Examination of Design Procedures for Single- and Multi-Tier Mechanically Stabilized Earth Walls

Wade N. Osborne
Stephen G. Wright

CTR Research Report:	0-4485-1
Report Date:	October 2004
Research Project:	0-4485
Research Project Title	Design Methodology for Tiered MSE or Concrete Block Retaining Walls
Sponsoring Agency:	Texas Department of Transportation
Performing Agency:	Center for Transportation Research at The University of Texas at Austin

Project performed in cooperation with the Federal Highway Administration and the Texas Department of Transportation.

Center for Transportation Research
The University of Texas at Austin
3208 Red River
Austin, TX 78705

www.utexas.edu/research/ctr

Disclaimers

Author's Disclaimer: The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the Federal Highway Administration or the Texas Department of Transportation (TxDOT). This report does not constitute a standard, specification, or regulation.

Patent Disclaimer: There was no invention or discovery conceived or first actually reduced to practice in the course of or under this contract, including any art, method, process, machine manufacture, design or composition of matter, or any new useful improvement thereof, or any variety of plant, which is or may be patentable under the patent laws of the United States of America or any foreign country.

Notice: The United States Government and the State of Texas do not endorse products or manufacturers. If trade or manufacturers' names appear herein, it is solely because they are considered essential to the object of this report.

Engineering Disclaimer

NOT INTENDED FOR CONSTRUCTION, BIDDING, OR PERMIT PURPOSES.

Project Engineer: Mark McClelland

Stephen G. Wright, P.E. (Texas No. 49007)

Acknowledgments

The authors express appreciation to TxDOT for their support of this research study. We are particularly grateful for the vision, guidance, constructive criticism, and continued encouragement of Mr. Mark McClelland and Mr. John Delphia of TxDOT. Mr. Marcus Galvin was also helpful in procuring the design information for a number of the MSE walls that TxDOT has constructed.

Table of Contents

List of Figures

List of Tables

1 Introduction

Mechanically stabilized earth (MSE) walls have been used successfully in the United States for thirty years and even longer overseas (Figure 1-1). As a result, the design procedures for simple single-tier MSE walls are well established. However, for more complex wall geometry (e.g., two or more tiers), the design procedures are not as well defined. In particular, the Texas Department of Transportation (TxDOT) found that the stability of multi-tier walls for highway embankments is difficult to evaluate on the basis of current design criteria. Design documents for multi-tier walls, which have been provided to TxDOT by private engineering firms, have reported factors of safety that are often difficult to reproduce or verify.

In the following chapters a review of the current design guidelines and how they are currently used by designers is presented. Current design practices for evaluating the "external," "internal," and "global" stability according to Federal Highway Administration (FHWA) and American Association of State Highway and Transportation Officials (AASHTO) guidelines are presented in Chapter 2. A number of issues were identified in the course of this study as problematic to the design of MSE walls. These issues and the various assumptions used by designers are presented in Chapter 3. In Chapter 4, two single-tier walls are analyzed using current design guidelines, and the results are compared with the original designer's analyses to identify and illustrate differences. Similar analyses performed on multi-tier walls are presented in Chapter 5 and are compared with the original designer's analyses. A summary, conclusions, and recommendations are presented in Chapter 6.

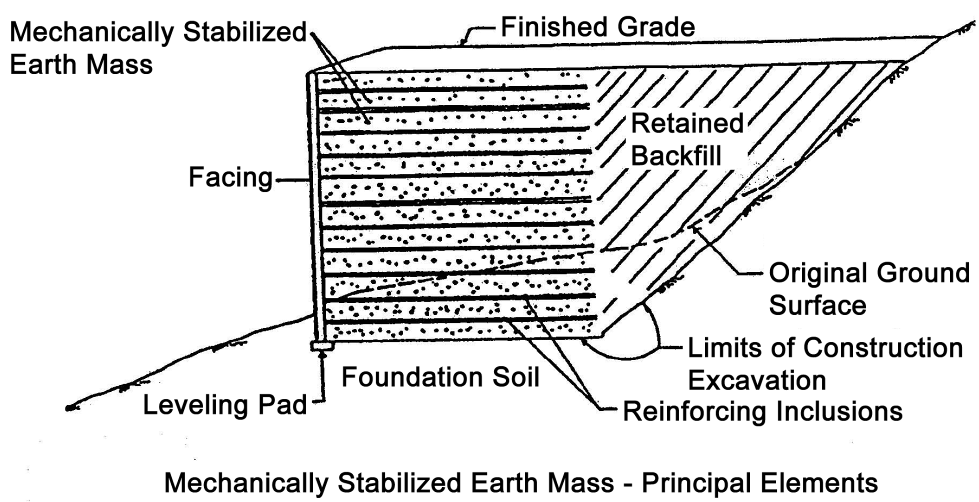

Figure 1-1: Typical cross-section of a mechanically stabilized earth wall (after FHWA 2001).

2 Current Design Practices for MSE Walls

2.1 Introduction

Design of mechanically stabilized earth (MSE)walls is based on evaluating safety for three idealized "modes" of instability: "external," "internal," and "global." For external stability, the MSE wall is treated as a rigid body, much like a concrete retaining wall, and factors of safety for sliding, overturning, and bearing capacity are calculated. For internal stability, factors of safety against rupture of the reinforcing elements and pullout of the reinforcing elements from the soil are calculated. The term "reinforcing element" refers to metallic strips and grids as well as geosynthetic reinforcement such as geogrid or geotextiles.

"Global" stability and the procedures for its evaluation are much less well defined. Global stability is typically evaluated using the same types of procedures used to evaluate slope stability. Global stability may involve consideration of slip surfaces entirely within, entirely outside of, or partially within and partially outside of the reinforced soil mass. Global stability analyses typically seek the slip surface with the lowest factor of safety, regardless of where the slip surface is located; however, this may not always be the case in practice. In some instances it appears that "global" stability is interpreted as applying only to slip surfaces entirely outside the reinforced soil mass. The various modes of stability and the procedures commonly used to evaluate them are described in this chapter.

2.2 Design Guidelines

On the basis of a review of design documents for three single-tier and five multi-tier MSE walls designed for the Texas Department of Transportation (TxDOT), two design guidelines have been used: the Federal Highway Administration (FHWA 2001) guidelines (Elias, Christopher and Berg, 2001) and the American Association of State Highway and Transportation Officials (AASHTO, 1998) guidelines.

The most recent versions of the FHWA and AASHTO guidelines, 2001 and 1998, respectively, were used in the course of this study to review the calculations for existing wall designs. Other versions of these guidelines may also be referenced, but these versions will be followed by the date of the version referenced—for example, FHWA 1996 or AASHTO 1996. Although AASHTO 1998 is the most recent version, several yearly updates or "interims" were available at the time of this document for the years 1999, 2000, 2001, 2002, and 2003, which contain significant changes for MSE wall design. When one of these dates (1999 through 2003) is referenced, it indicates that the information supercedes previous versions or interims available for the AASHTO 1998 guidelines.

Designers of existing wall designs reported using the versions of the following guidelines: FHWA 1997 (Elias and Christopher, 1997) and FHWA 2001, as well as the 1994, 1996, and 1998 versions of the AASHTO. Other designers did not specify which guideline was used. When known, the design guidelines used by the original designer are identified for the particular wall designs in Chapters 4 and 5.

FHWA guidelines for MSE wall design follow an allowable stress design (ASD) approach to determine internal and external stability. FHWA guidelines specify that actual loads and resistances be calculated and an appropriate factor of safety be computed. Several limit equilibrium procedures may be used to evaluate global stability according to the FHWA guidelines.

Past editions of AASHTO's Bridge Design Specifications detailed an ASD until replaced by Load and Resistance Factor Design (LRFD) in 1998 (AASHTO, 1998). Current AASHTO 1998 guidelines for designing MSE walls follow a load and resistance factor design (LRFD) approach. LRFD approaches use load and resistance factors to factor the respective forces. The goal of these factors is to achieve a sufficiently low probability of failure. However, the AASHTO guidelines indicate that little research has been conducted to determine appropriate load and resistance factors based on probabilities. Instead, the current load and resistance factors are based on past design procedures and successful wall performance.

2.3 External Stability

Procedures for evaluating the external stability of MSE walls have been derived largely from the ones used to evaluate stability of conventional, concrete gravity retaining walls. In these procedures the MSE wall, consisting of the reinforced soil mass and wall facing, is considered to be a rigid body. Factors of safety are then calculated for sliding, bearing capacity, and overturning (Figure 2-1).

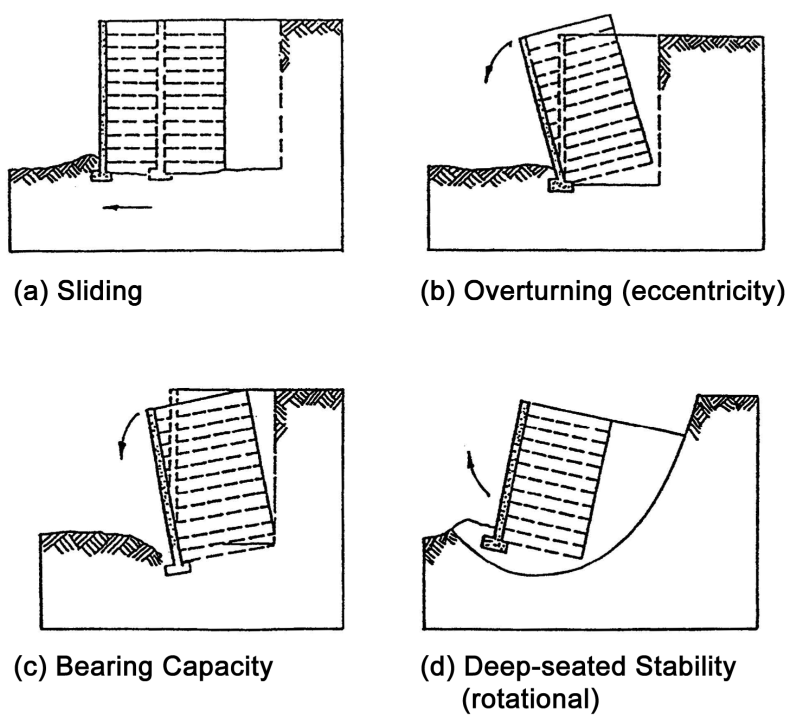

Figure 2-1: Modes of failure for external stability (after FHWA 2001).

2.3.1 Sliding Stability

FHWA and AASHTO guidelines provide different methods for evaluating the stability of a MSE wall against sliding. Many of the forces used to determine sliding stability are calculated identically in each method, but the requirements are different. The FHWA and AASHTO guidelines are discussed further below.

2.3.1.1 FHWA Method to Evaluate Sliding

FHWA guidelines define the factor of safety against sliding (FS_{SL}) as the ratio of horizontal resisting forces to the horizontal driving forces (Equation 2-1).

$$FS_{SL} = \frac{\Sigma \text{ horizontal resisting forces } (P_R)}{\Sigma \text{ horizontal driving forces } (P_d)} \tag{2-1}$$

The factor of safety is calculated at the base of the MSE wall, which is at the bottom of the reinforced soil mass.

Resisting Forces

The resistance to sliding is due to the frictional force developed within the foundation soil, the reinforced soil, or between the reinforcement and the reinforced soil. The horizontal resisting force is calculated by multiplying the vertical forces on the base of the wall by a coefficient of friction (μ). The vertical forces on the base of the wall include the weight of the reinforced soil mass, the weight of any soil above the wall including any sloping backfill, and the vertical component of the earth pressure force on the back of the reinforced soil block. Only permanent loads, not temporary or live loads, are used in computing the resisting forces. Loads such as traffic surcharge are excluded. The coefficient of friction (μ) depends on whether the reinforcement is continuous such as geogrid or noncontinuous such as steel strips or mats. If the reinforcement is continuous, (e.g., geogrid), the coefficient of friction (μ) is determined by the lowest value of the following:

- the tangent of the angle of internal friction of the foundation soil (tan φ_f),
- the tangent of the angle of internal friction of the reinforced soil (tan φ_r), or
- the tangent of the soil-reinforcement friction angle (tan ρ).

If the reinforcement is noncontinuous (e.g., steel strips), the coefficient of friction is calculated using the lowest value of the following:

- the tangent of the angle of internal friction of the foundation soil (tan φ_f) or
- the tangent of the angle of internal friction of the reinforced soil (tan φ_r).

The passive resistance at the toe of the wall due to embedment of the wall is neglected. Any contribution due to shear resistance of the facing system is also neglected (FHWA, 2001).

Calculation of Driving Force

The *horizontal driving forces*, F_D, are the sum of all horizontal forces in the direction of sliding applied to the back of the MSE wall, expressed as follows:

$$F_D = F_1 + F_2 \tag{2-2}$$

where

F_1 = the horizontal force due to the weight of retained soil, and
F_2 = the horizontal force caused by surcharge load(s).

F_D is the horizontal earth pressure force produced on the back of the reinforced soil mass by the weight of soil and surcharge behind the back of the MSE wall.

The horizontal force, F_1, on the back of the wall produced by the weight of the retained soil is calculated as

$$F_1 = \tfrac{1}{2} \cdot \gamma_f \cdot H^2 \cdot K_{a_f} \tag{2-3}$$

where

γ_f = the unit weight of retained soil,
H = height of the wall, and
K_{af} = active earth pressure coefficient of the retained soil.

The live load is typically modeled as a vertical surcharge load (q). The contribution (F_2) of the surcharge to the horizontal driving force is calculated from

$$F_2 = q \cdot H \cdot K_{a_f} \tag{2-4}$$

where

q = live load surcharge.

Other forces such as bridge abutment loads and a sloping backfill that may contribute to sliding are also included in the calculation of the horizontal driving force.

2.3.1.2 AASHTO Method to Evaluate Sliding

AASHTO guidelines specify that a MSE wall is stable against sliding when the following condition is satisfied:

$$Q_R \geq R_D \tag{2-5}$$

where

Q_R = the factored resistance against failure by sliding and
R_D = the ultimate horizontal driving force for sliding.

The condition for sliding stability is evaluated at the base of the MSE wall, which is at the bottom of the reinforced soil mass.

Resisting Force

AASHTO guidelines specify calculating the nominal resisting force in a fashion similar to the FHWA method, but with the inclusion of a passive resistance produced by any soil in front of the wall. The resistance to sliding (Q_R) is calculated as

$$Q_R = \varphi_T \cdot Q_T + \varphi_{ep} \cdot Q_{ep} \qquad (2\text{-}6)$$

where

φ_T = resistance factor for shear resistance between soil and foundation,
Q_T = nominal, or unfactored, shear resistance between soil and foundation,
φ_{ep} = resistance factor for passive resistance, and
Q_{ep} = nominal, or unfactored, passive resistance of soil in front of wall available throughout the design life of the structure.

The nominal shear resistance (Q_T) is calculated by the same procedure described for the FHWA guidelines, and the resistance factor (φ_T) is 1.0 for the "soil on soil" interaction at the base of MSE walls. The passive resistance (Q_{ep}) is calculated as

$$Q_{ep} = \frac{1}{2} \cdot k_p \cdot \gamma_s \cdot (z_e)^2$$

where

k_p = passive earth pressure coefficient,
γ_s = the unit weight of foundation soil, and
z_e = depth of wall embedment.

A resistance factor (φ_{ep}) of 0.5 is applied to the passive resistance.

Calculation of Driving Force

AASTHO guidelines stipulate computing the horizontal driving force (R_D) using the forces computed from Equations 2-3 and 2-4. The horizontal driving force is expressed as

$$R_D = 1.5 \cdot F_1 + 1.75 \cdot F_2 \qquad (2\text{-}7)$$

where

$$F_1 = \tfrac{1}{2} \cdot \gamma_f \cdot H^2 \cdot K_{a_f} \qquad (2\text{-}3)$$

$$F_2 = q \cdot H \cdot K_{a_f}. \qquad (2\text{-}4)$$

The nominal loads F_1 and F_2 are defined and calculated according to the AASHTO guidelines in exactly the same way as presented in the FHWA guidelines. Load factors 1.5 and 1.75 are included in Equation 2-7, which are specified by the AASHTO guidelines, to factor the horizontal earth pressure (F_1) and the surcharge load (q), respectively.

2.3.2 Factor of Safety Against Bearing Capacity Failure

MSE walls are evaluated for bearing capacity in both the FHWA and AASHTO guidelines. The methods in the two guidelines are described in the following sections.

FHWA Method

The FHWA guidelines specify that bearing capacity be evaluated by computing the vertical stress (σ_v) on the base of the wall and comparing this with the allowable bearing capacity (q_a) of the foundation soil. MSE walls are evaluated using a factor of safety for bearing capacity (FS_{BC}) of 2.5.

$$q_a = \frac{q_{ult}}{FS_{BC}} \tag{2-8}$$

The ultimate bearing capacity (q_{ult}) is calculated using classical soil mechanics methods and is represented in the FHWA specifications by Equation 2-9, which is specific to a level grade and no influence of groundwater.

$$q_{ult} = c_f \cdot N_c + 0.5 \cdot (L - 2e) \cdot \gamma_f \cdot N_\gamma \tag{2-9}$$

where

c_f = cohesion of the foundation soil,
L = length of the reinforcement,
e = eccentricity of the resultant of the vertical forces from the centerline of the wall,
γ_f = the unit weight of foundation soil, and
N_c, N_f = bearing capacity factors.

The FHWA guidelines provide values for the bearing capacity factors N_c, N_q, and N_γ, which are the same as those in Das (1994) based on Vesic (1973). However, a more recent version of the text by Das (2002) presents values for N_γ based on Meyerhof (1963), which are 20 to 80 percent smaller.

The vertical stress is calculated as an "adjusted" vertical stress by summing the vertical forces acting on the base of the MSE wall and dividing by a reduced length, which is the actual length of reinforcement (L) minus two times the eccentricity (*e*):

$$\sigma_v = \frac{\Sigma \ Vertical \ Forces}{L - 2e} . \tag{2-10}$$

The vertical forces include the weight of the reinforced soil mass, the weight of any soil above the reinforced zone, any live or dead loads, as well as any loads from bridge abutments.

AASHTO Method

AASHTO guidelines specify that bearing capacity be evaluated by verifying that the factored bearing resistance (q_R) of the foundation soil is greater than the factored vertical pressure (σ_{vf}):

$$q_R \geq \sigma_{vf} . \tag{2-11}$$

The factored bearing capacity (q_R) is defined as the ultimate bearing capacity (q_{ult}) multiplied by a resistance factor (φ_{BC}). The bearing capacity resistance factor (φ_{BC}) ranges from 0.35 to 0.60, depending on the type of soil and the method used to measure shear strength of the foundation soil (Table 2-1).

Table 2-1: Bearing capacity resistance factors (φ_{BC}) for type of soil and method used to measure foundation soil shear strength provided by AASHTO (after Table 10.5.5-1, AASHTO 1999).

Soil	**Method**		**Resistance Factor (φ_{BC})**
Sand	Semiempirical procedure	using SPT data	0.45
		using CPT data	0.55
	Rational method	using φ_f estimated from SPT data	0.35
		using φ_f estimated from CPT data	0.45
Clay	Semiempirical procedure	using CPT data	0.50
	Rational method	using shear resistance measured in lab tests	0.60
		using shear resistance measured in field vane tests	0.60
		using shear resistance estimated from CPT data	0.50
Rock	Semiempirical procedure, Carter and Kulhawy (1988)		0.60
All soils	Plate load test		0.55

Ultimate bearing capacity is calculated using classical soil mechanics methods and is represented in AASHTO guidelines by Equation 2-12, which is specific to cohesionless soils, a level grade, and no groundwater.

$$q_{ult} = 0.5 \cdot \gamma_f \cdot L \cdot C_{w1} \cdot N_{\gamma m} + \gamma_f \cdot C_{w2} \cdot D_f \cdot N_{qm} \qquad (2\text{-}12)$$

where

γ_f = total unit weight of foundation soil,
L = length of reinforcement,
C_{w1}, C_{w2} = coefficients related to the depth to the ground water table below the ground surface,
N_{ym}, N_{qm} = modified bearing capacity factors, modified for shape of the footing, the compressibility of the foundation soil, the inclination of loads from the footing, and the depth of the footing into the foundation soil, and
D_f = depth of wall embedment.

The AASHTO guidelines provide values for the bearing capacity factors N_c, N_q, and N_γ from Barker et al. (1991) based on Vesic (1973).

The vertical stresses are calculated using "factored" vertical forces. The "factored" vertical forces are computed by multiplying the vertical forces by load factors based on the type of load (Table 2-2).

Table 2-2: Load factors used to factor vertical forces used to calculate the vertical stress used in bearing capacity computations (AASHTO, 2002).

Type of Load	Load Factor
Weight of reinforced soil mass	1.35
Weight of soil above reinforced soil mass	1.35
Vertical forces caused by live surcharge load	1.75
Vertical forces caused by horizontal earth pressure load (moments) applied to back of the wall	1.50

Thus, the factored vertical stress (σ_{vf}) is calculated by summing the factored vertical forces acting on the base of the MSE wall and dividing by the length of reinforcement (L) minus two times the eccentricity (*e*):

$$\sigma_{vf} = \frac{\Sigma \; Factored \;\; Vertical \;\; Forces}{L - 2e}. \tag{2-13}$$

The Effect of Bearing Capacity Factors on the Ultimate Bearing Capacity

It is likely that wide variations in factors of safety for bearing capacity may be computed and reported depending on which bearing capacity theory and equations are used. To illustrate how the factor of safety may vary, consider the ultimate bearing capacity (q_{ult}) computed using the bearing capacity factors provided in FHWA 2001 and Das (2002). Assume the foundation soil has a cohesion (c_f) of 100 psf, a unit weight (γ_f) of 125 pcf, and an angle of internal friction (φ_f) of 30°. Also assume the length of the reinforcement (L) is 15.0 ft. and the vertical force on the base of the wall is offset 1.0 ft. from the centerline of the wall. Using the bearing capacity factors from FHWA 2001 and Equation 2-9, the ultimate bearing capacity (q_{ult}) is calculated as

- $q_{ult} = c_f \cdot N_c + 0.5 \cdot (L - 2e) \cdot \gamma_f \cdot N_\gamma$ (2-9)
- $q_{ult} = 100 psf \cdot 30.14 + 0.5 \cdot (15.0 ft - 2 \cdot 1.0 ft) \cdot 125 pcf \cdot 22.40 = 21{,}214 psf$.

For comparison, the ultimate bearing capacity (q_{ult}) calculated using the bearing capacity factors from Das (2002) is

- $q_{ult} = 100 psf \cdot 30.14 + 0.5 \cdot (15.0 ft - 2 \cdot 1.0 ft) \cdot 125 pcf \cdot 15.668 = 15{,}745 psf$.

The ultimate bearing capacity is approximately 35 percent larger when calculated with the bearing capacity factors using the FHWA 2001 guidelines than by using the more recent Das (2002).

2.3.3 Safety Against Overturning

FHWA and AASHTO guidelines suggest evaluating stability against overturning by calculating the eccentricity of the resultant of the vertical forces acting on the base of the wall. Eccentricity (*e*) is defined as the horizontal distance between the location of the resultant force on the base of the wall and the centerline of the wall. FHWA guidelines require that eccentricity be less than one sixth of the reinforcement length at the base of the wall. AASHTO guidelines establish a limit for eccentricity of one fourth the reinforcement length.

Calculating eccentricity and ensuring the eccentricity is less than the specified limits serves the same purpose as computing a factor of safety for overturning by evaluating the ratio of the stabilizing moment to the overturning moment. MSE walls are not likely to overturn owing to their ductile nature, but they can deform until they are unserviceable. Satisfying the requirements for overturning (i.e. eccentricity) probably helps to prevent excessive deformation and distortion of the MSE wall.

2.4 Internal Stability

Internal stability is evaluated by calculating factors of safety for rupture of the reinforcement and for pullout of the reinforcement from the soil. Rupture of the reinforcement occurs when the stresses in the reinforcement exceed the ultimate stress for the reinforcement. Pullout occurs when the forces in the reinforcement due to the horizontal earth pressure are greater than the resisting force developed by friction between the soil and the reinforcement over its embedded length. Factors of safety against rupture and pullout are calculated for each layer. Both factors of safety must be adequate to satisfy internal stability requirements. The following sections discuss the current requirements specified by FHWA and AASHTO guidelines for rupture and pullout of the reinforcement.

2.4.1 Rupture of Reinforcement

Evaluating the safety against rupture of the reinforcement involves evaluating the strength of the reinforcement and the forces in the reinforcement. Different methods are used for this purpose in the FHWA and AASHTO guidelines.

2.4.1.1 Reinforcement Rupture Strength

Several quantities are used to characterize the strength of reinforcement for internal stability against rupture of the reinforcement. The various "measures" of reinforcement strength that are used are described below.

Ultimate (or yield) tensile strength (T_{ult})

Ultimate tensile strength is defined as the load at which reinforcement breaks or ruptures. T_{ult} has dimensions of force per unit width of reinforcement. For metallic reinforcement, the width (b) depends on the type of metal reinforcement. For solid strips, the width is defined as the width of an individual strip. For metal grids, the width is defined as the distance between the two outermost longitudinal bars of the grid. FHWA and AASHTO guidelines stipulate T_{ult} be calculated for metallic reinforcement based on the yield stress (F_y), the cross-sectional area of reinforcement (A_c) corrected for corrosion loss, and the width of the reinforcement (b). The ultimate tensile strength is calculated from Equation 2-14:

$$T_{ult} = \frac{F_y \cdot A_c}{b} . \quad (2\text{-}14)$$

For geosynthetic reinforcement, the ultimate tensile strength based on test data is generally provided by the manufacturer. The ultimate strength is measured in laboratory tests according to the ASTM D4595-03 standard and reported as a minimum average roll value (MARV) in manufacturer's literature. FHWA and AASHTO guidelines do not provide a method for calculating T_{ult} for geosynthetic reinforcement directly from yield stress values as is done for metallic reinforcement.

Long-Term Design Strength (T_{al})

The *long-term design strength* of reinforcement is considered to be the strength accounting for the effects of creep, durability, and installation damage. FHWA and AASHTO guidelines are the same for evaluating the long-term design strength. However, the guidelines determine the long-term design strength differently for geosynthetic and for metallic reinforcement.

According to FHWA and AASHTO guidelines, the long-term design strength (T_{al}) for geosynthetic reinforcement is calculated by dividing the ultimate tensile strength (T_{ult}) by an appropriate reduction factor (RF).

$$T_{al} = \frac{T_{ult}}{RF} \quad (2\text{-}15)$$

where

$$RF = RF_{CR} \cdot RF_D \cdot RF_{ID} . \quad (2\text{-}16)$$

The reduction factor (RF) combines strength reduction factors accounting for potential long-term degradation due to creep (RF_{CR}), chemical and biological degradation (RF_D), and installation damage (RF_{ID}). Reduction factors and/or long-term design strengths are generally provided by manufacturers and may vary depending on the type of backfill soil. The FHWA and AASHTO guidelines recommend using the reduction factors provided by the manufacturer. If a manufacturer does not provide reduction factors, the FHWA and AASHTO guidelines suggest the values shown in Table 2-3.

Table 2-3: Default values for the reduction factors provided by FHWA and AASHTO guidelines.

Design Guidelines	**Reduction Factors**			
	RF	**RF_{ID}**	**RF_{CR}**	**RF_D**
FHWA	≥ 7.0	1.1–3.0	1.6–5.0	1.1–2.0
AASHTO	≥ 7.0	≥ 1.1	–	≥ 1.1

For metallic reinforcement, FHWA and AASHTO guidelines recommend that the long-term design strength (T_{al}) be the same as the ultimate tensile strength (T_{ult}). Metallic reinforcement is not expected to creep or sustain significant damage during installation. Effects of corrosion are incorporated into the ultimate strength by using a "corrected" cross-sectional area (A_c), rather than a reduction factor. The cross-sectional area is reduced by an amount expected due to corrosion over the design life of the wall. The cross-section area (A_c) is reduced by calculating the amount of steel expected to corrode the diameter of steel bars or the thickness of steel strips. Typical corrosion rates for a "mildly corrosive backfill" are provided by the FHWA and AASHTO guidelines.

2.4.1.2 Maximum Reinforcement Tension (T_{MAX})

The maximum force in the reinforcement is considered to be the force tending to cause rupture and pullout of the reinforcement. FHWA and AASHTO guidelines specify calculating the maximum force in the reinforcement (T_{MAX}), as follows:

$$T_{MAX} = \sigma_H \cdot S_V \tag{2-17}$$

where

σ_H = horizontal stress caused by lateral earth pressures, and

S_V = the vertical spacing between layers of reinforcement.

The horizontal stress (σ_H) is calculated from the vertical stress (σ_V) acting on the reinforcing elements at each layer. For calculating the horizontal stress, the vertical stress is computed from all the vertical forces acting on the MSE wall, which include the weight of the reinforced soil, any sloping backfill, surcharge loads, and concentrated vertical loads. The vertical stress is multiplied by a lateral earth pressure coefficient (K_r) to compute the horizontal stress. The lateral earth pressure coefficient (K_r) is calculated from the active earth pressure coefficient (K_a) of the soil. The ratio K_r/K_a, which is different for different types of reinforcement and for metallic reinforcement, decreases linearly from the top of the wall to a depth of 6 m where Kr/Ka is constant (Figure 2-2).

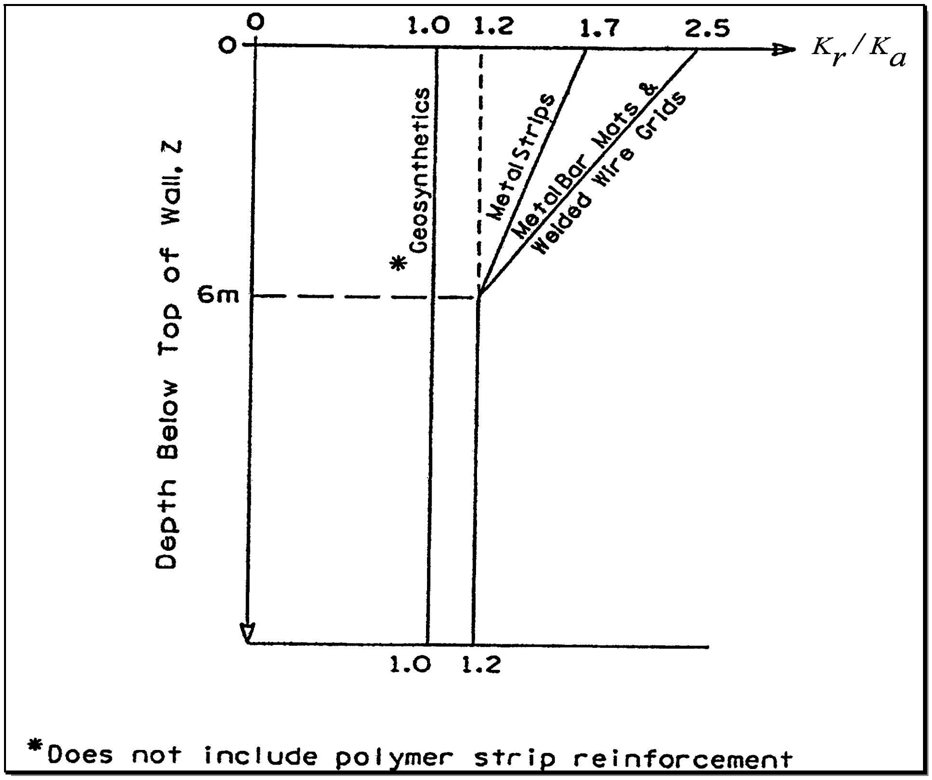

Figure 2-2: Variation of lateral earth pressure coefficient (Kr) with depth in a MSE wall (after FHWA, 2001).

2.4.1.3 Calculation of Safety Against Rupture

FHWA and AASHTO guidelines specify different methods for evaluating the potential for rupture of the reinforcement. The methods are presented in the following sections.

FHWA Method

The FHWA 2001 criterion for rupture of the reinforcement requires the maximum horizontal force (T_{MAX}) be less than the "allowable design strength," T_a. The allowable design strength (T_a) is the long-term design strength (T_{al}) reduced by a suitable factor of safety. The factor of safety is applied to T_{al} to account for uncertainties in structure geometry, fill properties, externally applied loads, the potential for local overstress due to load nonuniformities, and uncertainties in long-term reinforcement strength (FHWA, 2001). FHWA guidelines suggest different methods for calculating the allowable design strength for metallic and for geosynthetic reinforcement.

Allowable design strength (T_a) for geosynthetic reinforcement is computed by dividing the long-term design strength (T_{al}) by a factor of safety (FS_R).

$$T_a = \frac{T_{al}}{FS_R} \qquad (2\text{-}18)$$

A factor of safety of 1.5 is used to ensure the stability of geosynthetic reinforcement against rupture.

The allowable design strength (T_a) for metallic reinforcement is calculated by multiplying the long-term design strength (T_{al}) by what is referred to in this study as a *safety coefficient* (C_R).

$$T_a = C_R \cdot T_{al} = C_R \cdot \frac{F_y \cdot A_c}{b} \qquad (2\text{-}19)$$

It should be noted that the FHWA guidelines refer to the safety coefficient as a *factor of safety* and use the notation "FS" instead of C_R. However, for clarity, in this study the term *factor of safety* (FS) is used to describe quantities greater than 1.0, which are used to reduce strengths or resistance by dividing by them. The term *safety coefficient* (C_R) will be used to describe quantities that are less than 1.0, which are used as multipliers. The safety coefficient is dependent on the type of metallic reinforcement used. FHWA guidelines specify using values of C_R of 0.55 for steel strips and 0.48 for steel grids. The equivalent values of factors of safety for steel strips and grids are 1.82 (=1/0.55) and 2.08 (=1/0.48), respectively.

Reinforcement is considered to have adequate resistance to rupture if the following condition is satisfied:

$$T_a \cdot R_c \geq T_{MAX} \qquad (2\text{-}20)$$

where

R_C = coverage ratio.

The coverage ratio is defined as the width of the reinforcement (b) divided by the horizontal spacing of the reinforcement (S_H).

$$R_C = \frac{b}{S_H} \qquad (2\text{-}21)$$

The coverage ratio is less than 1.0 for discrete reinforcing elements, such as reinforcement where a gap separates the edge of one element to the edge of the next, and equal to 1.0 for continuous reinforcement such as reinforcement where there is no gap between reinforcing elements.

The potential for rupture is evaluated based on the condition expressed by Equation 2-20 for each layer of reinforcement. No additional factor of safety is calculated because a safety coefficient (C_R) or factor of safety (F_{SR}) is applied to calculate T_a.

AASHTO Method

AASHTO 1998 guidelines specify using LRFD load and resistance factors to evaluate safety against rupture of reinforcement. Rupture is evaluated based on the following condition:

$$\varphi_R \cdot T_{al} \geq Y_P \cdot T_{MAX} \qquad (2\text{-}22)$$

where

φ_R = resistance factor for reinforcement tension and
Y_P = the load factor for vertical earth pressure.

The resistance factor (φ_R) is less than 1.0 and is multiplied to the long-term design strength (T_{al}) of the reinforcement. Load factors (Y_P) are greater than 1.0 and are used to increase the load from the maximum horizontal force (T_{MAX}). The values for load and resistance factors for rupture of reinforcement provided by AASHTO guidelines are shown in Table 2-4.

Table 2-4: Load and resistance factors for rupture of reinforcement provided by AASHTO 1998.

Type of Reinforcement	Load Factor (Y_p)	Resistance Factor (φ_R)
Ribbed steel strips	1.35	0.75
Steel grids	1.35	0.65
Geosynthetic	1.35	0.90

Comparison of FHWA and AASHTO Guidelines for Rupture

Although the FHWA and AASHTO procedures for evaluating rupture of the reinforcement are expressed differently, they produce identical results. To illustrate, consider a MSE wall with geogrid reinforcement. The FHWA requirement for rupture is expressed by Equation 2-20, which can be combined with Equation 2-18 and written as follows:

$$\frac{T_{al}}{FS_R} \geq T_{MAX} \qquad (2\text{-}23)$$

or

$$\frac{T_{al}}{T_{MAX}} \geq FS_R\,. \qquad (2\text{-}24)$$

The FHWA guidelines require geogrid reinforcement provide a minimum factor of safety for rupture (FS_R) of 1.5; that is,

$$\frac{T_{al}}{T_{MAX}} \geq 1.5\,. \qquad (2\text{-}25)$$

The AASHTO guideline for rupture of the reinforcement is expressed by Equation 2-22, which can be written as follows:

$$\frac{T_{al}}{T_{MAX}} \geq \frac{Y_P}{\varphi_R}\,. \qquad (2\text{-}26)$$

The load (Y_P) and resistance (φ_R) factors specified by AASHTO guidelines for geogrid reinforcement are 1.35 and 0.9, respectively. Thus,

$$\frac{T_{al}}{T_{MAX}} \geq \frac{1.35}{0.9} = 1.5\,. \qquad (2\text{-}27)$$

Comparison of Equations. 2-25 and 2-27 show that the current FHWA and AASHTO requirements are the same. Table 2-5 illustrates that this condition is true for both metallic and geosynthetic materials.

Table 2-5: Comparison of safety factors for FHWA 2001 and AASHTO 1998 rupture of the reinforcement requirements.

Type of Reinforcement	**AASHTO 1998**			**FHWA 2001 Minimum Factor of Safety against Rupture (FS_R)**
	Load Factor (Y_p)	**Resistance Factor (φ_R)**	**Equivalent Factor of Safety (Y_p/φ_R)**	
Ribbed steel strips	1.35	0.75	1.80	1.82
Steel grids	1.35	0.65	2.08	2.08
Geosynthetic	1.35	0.90	1.50	1.50

2.4.2 Pullout of Reinforcement

Evaluating the safety against pullout of the reinforcement involves evaluating the pullout resistance of the reinforcement and the forces in the reinforcement. Different methods are used for this purpose in the FHWA and AASHTO guidelines.

2.4.2.1 Reinforcement Pullout Resistance (P_r)

The pullout resistance, P_r, of reinforcement represents the load required to cause pullout of the embedded end from the soil. According to FHWA and AASHTO guidelines, pullout resistance is calculated using the following equation for both metallic and geosynthetic reinforcement:

$$P_r = F^* \cdot \alpha \cdot \gamma_r \cdot Z_P \cdot L_e \cdot C \qquad (2\text{-}28)$$

where

L_e = the embedment (adherence) length in the resisting zone behind the failure surface,
C = the effective unit perimeter of the reinforcement; C = 2 for strips, grids, and sheets,
F* = the pullout resistance factor, which is a function of passive and frictional resistance of the reinforcement,
α = a scale effect correction factor to account for a nonlinear stress reduction over the embedded length of highly extensible reinforcements, based on laboratory data (generally 1.0 for metallic reinforcements and 0.6—1.0 for geosynthetic reinforcement),
Z_P = the depth to the layer of reinforcement, and
γ_r = total unit weight of reinforced soil.

The FHWA and AASHTO guidelines specify that values for F* and α should be determined from product-specific pullout tests in the actual backfill material or an equivalent soil. Conservative default values for F* and α are provided by the FHWA and

AASHTO guidelines to be used in the absence of test data (Table 2-6). Pullout resistance has dimensions of force per unit width of reinforcement.

Table 2-6: Default values for F and α provided by FHWA 2001 and AASHTO 1998.*

Type of Reinforcement		F*		α
		Top of Wall	Depth of 20 ft and below	
Metallic	Ribbed strips	$1.2 + \log(C_u) \leq 2.0$	$\tan \varphi$	1.0
	Grid	$20*(t/S_t)$	$10*(t/S_t)$	1.0
Geosynthetic	Geogrid	$2/3*\tan \varphi$		0.8
	Geotextile	$2/3*\tan \varphi$		0.6

C_u = uniformity coefficient of the backfill t = thickness of the transverse bar
S_t = the spacing of the transverse bars

2.4.2.2 Calculation of Safety Against Pullout

FHWA and AASHTO guidelines specify different procedures for evaluating the safety against pullout of the reinforcement from the soil. These are each presented below.

FHWA Method

The FHWA guidelines specify applying a factor of safety (FS_{PO}) of 1.50 to the pullout resistance (P_r) and comparing the resistance with the maximum horizontal force (T_{MAX}); that is,

$$T_{MAX} \leq \frac{P_r \cdot R_C}{FS_{PO}} . \tag{2-29}$$

The maximum horizontal force (T_{MAX}) is calculated in the way described previously in Section 2.3.1.2.

AASHTO Method

AASHTO guidelines employ load and resistance factors to evaluate the pullout of reinforcement. The design for pullout is considered satisfactory if

$$Y_P \cdot T_{MAX} \leq \varphi_{PO} \cdot P_r \cdot R_C \tag{2-30}$$

where

φ_{PO} = resistance factor for reinforcement pullout and
Y_P = the load factor for vertical earth pressure.

The load factor (Y_P) is greater than 1.0, which increases the applied horizontal load (T_{MAX}). The resistance factor (φ_{PO}) is less than 1.0 and reduces the pullout resistance. The values for load and resistance factors for pullout provided by the AASHTO guidelines are shown in Table 2-7.

Table 2-7: Load and resistance factors for pullout of reinforcement provided by AASHTO 1998.

Type of Reinforcement	Load Factor (Y_p)	Resistance Factor (φ_{PO})
Ribbed steel strips	1.35	0.90
Steel grids	1.35	0.90
Geosynthetic	1.35	0.90

Comparison of FHWA and AASHTO for Pullout

Although the FHWA and AASHTO procedures for evaluating safety against pullout are expressed differently, they produce identical results. To illustrate this point, consider a MSE wall with geogrid reinforcement. The FHWA guidelines specify a minimum factor of safety for pullout (FS_{PO}) of 1.5. Thus, Equation 2-29 for pullout can be written as follows:

$$\frac{P_r \cdot R_C}{T_{MAX}} \geq FS_{PO} = 1.5\,. \qquad (2\text{-}31)$$

The AASHTO guidelines (Equation 2-30) can be expressed as

$$\frac{P_r \cdot R_C}{T_{MAX}} \geq \frac{Y_P}{\varphi_{PO}}\,, \qquad (2\text{-}32)$$

where load (Y_P) and resistance (φ_{PO}) factors of 1.35 and 0.90, respectively, are required. Substituting these values for Y_p and φ_{PO} into Equation 2-32 gives the following:

$$\frac{P_r \cdot R_C}{T_{MAX}} \geq \frac{1.35}{0.9} = 1.5\,. \qquad (2\text{-}33)$$

Comparison of Equations. 2-31 and 2-33 confirms that the current FHWA and AASHTO requirements are the same for both metallic and geosynthetic reinforcements.

2.5 Global Stability

Global stability is evaluated using a limit equilibrium procedure of slices to compute a factor of safety. Several definitions of factor of safety are used, leading to a lack of consistency in computed values. Furthermore, current FHWA and AASHTO guidelines provide only minimal guidance on evaluation of global stability. Several items and issues pertaining to global stability are discussed further in the following sections.

The variables and assumptions for two commercially available software programs are discussed. The first software program used and evaluated for this study is MSEW Version 2.0 (Leshchinsky, 2002), which was the most current version available when purchased in 2002. The MSEW 2.0 software performs analyses on MSE walls using the procedures outlined in FHWA 2001. The second software program used is UTEXAS4 (*U*niversity of *TEX*as *A*nalysis of *S*lopes, Version 4), which is a general purpose computer program for slope stability computations (Wright, 1999). The version of the UTEXAS4 software used was last revised on February 14, 2003.

2.5.1 FHWA and AASHTO Requirements

FHWA and AASHTO guidelines specify similar approaches for evaluating global stability. Both recommend conducting a limit equilibrium slope stability analysis to determine critical slip surfaces.

The FHWA guidelines specify that a MSE wall should be analyzed with the reinforced soil mass modeled as a rigid body and that only failure surfaces completely outside the reinforced zone should be considered. The minimum factor of safety for global stability is in this case 1.3.

AASHTO design requirements stipulate that a limit equilibrium analysis should be performed for global stability analysis. The global stability should be evaluated using what is called a *Service 1 Load Combination* and an appropriate resistance factor. For a Service 1 Load Combination, the loads are "factored" by 1.0. The resistance factor (φ) is 0.75 if the geotechnical parameters are well defined and is 0.65 if the geotechnical parameters are based on limited information. AASHTO guidelines specify that the slope stability analysis is a service limit state check, which leads to the belief that a factor of safety greater than or equal to 1.0 is acceptable; however, no factor of safety requirement is provided by the AASHTO guidelines.

2.5.2 Procedure of Slices

Global stability is almost always evaluated using a procedure of slices to compute a factor of safety with the additional forces due to the reinforcement included. Computations may be performed using either circular or noncircular slip surfaces. Several procedures of slices have been used. Some procedures satisfy complete moment and force equilibrium, whereas some satisfy only some of the equations and conditions of static equilibrium.

AASHTO design requirements indicate that a limit equilibrium analysis should be conducted using the Modified Bishop, Simplified Janbu, or Spencer method. FHWA guidelines do not stipulate that a specific limit equilibrium procedure be used, but it appears that the three procedures stipulated by the AASHTO guidelines are the ones most commonly used.

Probably the three most commonly used limit equilibrium procedures are Spencer's procedure, Bishop's Simplified procedure, and the Simplified Janbu procedure. Spencer's procedure is one of the best procedures because it completely satisfies both moment and force equilibrium and can be used to analyze any shape of slip surface. Bishop's Simplified procedure satisfies moment equilibrium for the entire free-body composed of all slices and vertical force equilibrium for individual slices. Bishop's Simplified method is limited to circular slip surfaces, but it is simple to employ and relatively accurate even though it does not satisfy complete static equilibrium. The Simplified Janbu procedure satisfies only equilibrium of forces, not moments, and is probably the least accurate of the three methods. Both circular and noncircular slip surfaces can be analyzed using the Simplified Janbu method. Correction factors have been published for the Simplified Janbu procedure, but the correction factors are empirical and based on only a limited number of cases. Some software programs using the Simplified Janbu procedure have the correction factors embedded in the analysis, and others do not. Software utilizing the Janbu correction factors will likely report different factors of safety than software that does not.

2.5.3 Defining Factors of Safety

The factors of safety for external and internal stability are well defined by the FHWA and AASHTO guidelines, but not clearly defined for global stability. The factors of safety computed in limit equilibrium procedures for reinforced slopes and walls have been defined in the following three ways.

1. The factor of safety is applied to the soil shear strength only.
2. The factor of safety is applied to the soil shear strength and the reinforcement force.
3. The factor of safety is applied to the reinforcement force only.

These definitions are discussed further below.

2.5.3.1 Factor of Safety on Soil Shear Strength

The factor of safety on soil shear strength, F_s, is defined as

$$F_s = \frac{S}{\tau} = \frac{\Sigma\, Available\ shear\ strength}{Shear\ strength\ required\ for\ equilibrium}. \qquad (2\text{-}34)$$

This definition is used in almost all procedures of slices for unreinforced slopes. This approach is also used in most general purpose slope stability software that allows reinforcement to be included. For example, this approach and definition is used in the UTEXAS4 (Wright, 1999) software.

The MSEW 2.0 (Leshchinsky, 2002) software employs as options three different definitions for the factor of safety. One of the three options defines the factor of safety with respect to shear strength, as shown above. This option is referred to as the *Comprehensive Bishop* method in the MSEW 2.0 software and documentation.

2.5.3.2 Balanced Factor of Safety

A “balanced” factor of safety, F_b, is a factor of safety applied equally to both the soil shear strength and the reinforcement force. A balanced factor of safety can be calculated by first assuming a factor of safety and applying it to reduce the reinforcement forces. The factor of safety applied to shear strength is then calculated using the reduced reinforcement forces. This process is repeated until the calculated factor of safety for shear strength is equal to the assumed factor of safety applied to the reinforcement forces. Although general purpose software such as UTEXAS4 does not calculate such a factor of safety automatically, a balanced factor of safety can be calculated by manual trial and error using the procedure just described.

The MSEW 2.0 design software will automatically compute a balanced factor of safety as one of the available options. The option of computing a balanced factor of safety is referred to as the *Demo 82 Approach* in the MSEW 2.0 documentation. When the Demo 82 approach is used, a balanced factor of safety is calculated for slip surfaces that intersect reinforcing elements, but when the slip surface passes entirely outside of the reinforced zone, the MSEW 2.0 software computes the factor of safety applied to shear strength only (Equation 2-34).

2.5.3.3 Factor of Safety on Reinforcement Force

The third approach to defining a factor of safety is to apply the factor of safety to the reinforcing forces only. In this approach, the soil shear strength is assumed to be fully mobilized, and the reinforcement force is factored until equilibrium is satisfied—in other words, the factor of safety is defined as the ratio of the available reinforcing force to the reinforcement force required to produce equilibrium. This definition for the factor of safety has no meaning for slip surfaces outside of the reinforced soil zone.

The method of applying the factor of safety to the reinforcement force is implemented in the MSEW 2.0 software as an option referred to as the *L* (Leshchinsky) *modification of Demo 82 Approach*. When this definition is used in the MSEW 2.0 software, it is used only for slip surfaces that intersect reinforcing elements; when slip surfaces do not intersect, the reinforcement the factor of safety is applied to the soil shear strength (Equation 2-34).

2.5.4 Discussion

The actual factor of safety on soil shear strength for many conventional retaining walls, particularly those with extensible reinforcement, is probably very close to 1.0, because the shear strength is fully mobilized (Arriaga, 2003). However, experience has shown that the wall remains stable despite the shear strength being fully mobilized (F_s = 1.0). On the basis of this experience, one might consider a reasonable approach to designing MSE walls would be to set the factor of safety to 1.0 on soil shear strength and calculate a factor of safety with respect to the reinforcement force. This approach, assuming $F_{soil\ strength}$ = 1.0 and calculating $F_{reinforcement}$, is used for internal stability analysis. However, a major limitation of this approach is that for slip surfaces outside of the reinforced soil mass the factor of safety for reinforcement has no meaning; in other words, the factor of safety is undefined. Factors of safety may also assume odd values when the slip surface intersects only a very small portion of the reinforcement. Finally, this definition is not what is used for unreinforced slopes, and thus, defining the factor of safety in this way represents a departure from normal practice for slope stability analyses.

In almost all conventional practice, the stability of unreinforced earth slopes is evaluated by a factor of safety applied to soil shear strength only. For reinforced slopes and MSE walls, one probably needs to apply factors of safety to both the soil shear strength and the reinforcement force. However, the factor of safety applied to soil shear strength should probably be different from the factor of safety applied to the reinforcement force.

2.5.5 Recommendations

On the basis of the above discussion, separate but different factors of safety should probably be defined and applied to soil shear strength and reinforcement forces to evaluate the global stability of MSE walls. This can be done with almost any limit equilibrium method and stability analysis software. To do so, the reinforcement forces are first factored (reduced) using an appropriate factor of safety for the reinforcement. The factored reinforcement forces are then used as input to compute a factor of safety with respect to the soil shear strength. The factor of safety applied to the reinforcement forces can probably be based on current recommendations provided by FHWA guidelines. The factor of safety on soil shear strength should depend on conventional practice for slopes as well as adjustments to account for the nature of MSE walls.

2.6 Summary

This chapter discussed the procedures for the various stability analyses and identified differences between the FHWA and AASHTO guidelines. Although the factors of safety are clearly defined for external and internal stability analyses by the FHWA and AASHTO guidelines, some assumptions used to calculate those factors of safety are not well defined. Little information is provided by the FHWA and AASHTO guidelines on conducting global stability analyses of MSE walls, which leads to assumptions being made by the designer. The following chapter will identify and discuss the assumptions that are well defined by the FHWA and AASHTO guidelines and the assumptions that are made by individual designers or software programs used to conduct analyses.

3 Issues with Design of MSE Walls

3.1 Introduction

Analyses of mechanically stabilized earth (MSE)walls involve different assumptions and variables that affect the computed factor of safety. In the process of reviewing Federal Highway Administration (FHWA) and American Association of State Highway and Transportation Officials (AASHTO) guidelines and performing analyses for a number of walls during this study, many of these variables and assumptions were identified. The important variables and assumptions are described and discussed in this chapter. Emphasis is placed on differences in the approaches used in various guidelines and by various designers.

The design procedures addressed in this chapter are the current FHWA 2001 and AASHTO 1998 guidelines as identified in the previous chapter. The MSEW 2.0 and UTEXAS4 software are also evaluated and discussed. The MSEW 2.0 software is a design software program for MSE walls written for, and specifically referenced by, the FHWA 2001 guidelines. The MSEW 2.0 software is evaluated in this study on the basis of its endorsement by the FHWA 2001 guidelines. The UTEXAS4 software is a general purpose slope stability program used and evaluated in this study for comparison with the MSEW 2.0 software.

3.2 Uniform Surcharge Loads

Uniform surcharge loads (q) are generally stipulated to model the effects of various live or dead loads acting on the surface of the backfill and natural ground behind a wall. Surcharge loads may be project specific or specified by FHWA or AASHTO guidelines. For example, FHWA and AASHTO guidelines require that a uniform surcharge load be applied to represent traffic loads. The uniform surcharge is equivalent to the vertical stress produced by 2.0 ft. (0.6 m) of reinforced soil.

FHWA and AASHTO guidelines identify only a *traffic surcharge* load in any detail. However, the guidelines state that additional surcharge loads may include *live* and *dead* surcharge loads, but do not identify what these might be. For the purposes of this report and in the MSEW 2.0 software, a *live* surcharge load is considered to be any transient load, such as traffic or construction loads. A *dead* surcharge load is any permanent, evenly distributed load, such as a concrete roadway surface. The final distinction of what is a live or a dead load is actually left to the designer of a MSE wall, but it is important because in some analyses the two types of surcharge (“live” vs. “dead”) are treated differently.

The way in which surcharge loads are treated in design sometimes depends on whether internal, external, or global stability is being evaluated. Surcharge may be treated different for these different stability conditions. Treatment of surcharges is covered separately for internal, external, and global stability in the following three sections.

3.2.1 Effect of Uniform Surcharge Loads on the External Stability

FHWA and AASHTO guidelines specify that a surcharge load should be used for the analysis of bearing capacity, sliding, and overturning.

For bearing capacity analyses, a uniform live surcharge load, representing the traffic load, is applied to the top of the wall beginning at the face and extending beyond the reinforced soil zone. The surcharge load contributes to the vertical stress (σ_v) on the base of the MSE wall in two ways: (1) it increases the normal stress on the top of the wall above the reinforced soil zone, and (2) it increases the horizontal earth pressure on the back of the reinforced soil mass, which in turn creates a moment on the wall and increases the vertical stress at the toe of the wall. Both of these effects are accounted for in the bearing capacity calculations.

Sliding and overturning stability of a MSE wall is calculated using a live load surcharge applied to the ground surface behind the reinforced soil mass, but *not* to the reinforced soil mass itself. Thus, the live surcharge load increases the horizontal earth pressures on the back of the wall, which acts to induce sliding or overturning, but it does not contribute to the resistance of the wall to sliding or overturning.

3.2.2 Effect of Uniform Surcharge Loads on the Internal Stability

For internal stability calculations, a surcharge is treated differently depending on whether the surcharge is a live or dead load. A live surcharge load is used to calculate the maximum tensile force (T_{MAX}) that might cause rupture and pullout of the reinforcement. However, the live load is not included in the vertical stress on the reinforcement that is used to calculate the pullout resistance (P_r). It is conservative to apply the surcharge to the calculation of maximum tensile force but not to that of pullout resistance.

FHWA and AASHTO guidelines provide no information on how to treat dead surcharge loads; instead, they deal mostly with the treatment of live loads. In the case of dead loads, a logical assumption is that the surcharge is used to calculate both the maximum tensile force (T_{MAX}) and pullout resistance (P_r) of the reinforcement. If the dead load is permanent, its contribution to both T_{MAX} and P_r seems reasonable.

3.2.3 Effect of Uniform Surcharge Loads on the Global Stability

For global stability, FHWA and AASHTO guidelines stipulate that a traffic surcharge load be applied to the top of the wall and extend indefinitely beyond the reinforced soil zone. Additional surcharge loads that exist on the wall may also be included at the discretion of the designer.

The MSEW 2.0 software allows the user to stipulate whether a surcharge load is a live or dead load. The way in which live or dead loads are treated in the MSEW 2.0 software is not clearly stated in the user's guide, but rather it was discovered in the course of analyses with the software. In the MSEW 2.0 software, a *live* surcharge load has no effect on the calculated factor of safety for global stability. The additional load on top of the MSE wall is not used in calculating the driving forces on the soil mass and does not increase the pullout resistance. In the MSEW 2.0 software, a *dead* surcharge load is assumed to increase the pullout resistance of the reinforcement, but it does not contribute to driving forces. Consequently, including a dead surcharge load in analyses with the MSEW

2.0 software was found either to not affect or to increase the factor of safety. The increase in factor of safety is due to the increased pullout resistance of the reinforcement caused by the additional vertical stress from the dead surcharge load.

Most general purpose slope stability software probably does not distinguish between live or dead surcharge loads for global stability analyses. For example, in UTEXAS4 the surcharge loads may be applied as external distributed loads and/or to calculate pullout resistances independently of each other. The choice as to how surcharge loads are used is up to the user and reflected in the appropriate input data. If surcharge loads acting on the MSE wall are to increase the pullout resistance, they must be explicitly included in the reinforcement resistance stipulated as input by the user. When either a live or dead surcharge load is used in the analysis, the user has the option of including the effects both as external loads and as contributing to the pullout resistance for reinforcement. If the user determines that a surcharge load should be applied as an external load but neglected in determining the resisting loads in the reinforcement, the factor of safety computed will typically be lower than if the surcharge is considered in calculating the pullout resistance.

3.3 Embedment of the MSE Wall

FHWA and AASHTO guidelines require that MSE walls be embedded into the foundation soil. On the basis of FHWA and AASHTO criteria, the wall should be embedded between 5 and 20 percent of the wall height with a minimum embedment of 1.67 ft. (0.6 m).

For both external and internal stability calculations, the FHWA guidelines specify that the effects of passive resistance of soil in front of the wall are to be ignored. For external stability, AASHTO 2002 allows the passive resistance of the soil to be accounted for in bearing capacity and sliding stability calculations, but it ignores embedment for overturning calculations. For internal stability, AASHTO 2002 neglects effects of embedment.

For global stability, neither FHWA nor AASHTO guidelines provide information on how wall embedment should be modeled. When global stability analyses were conducted using the MSEW 2.0 software, the way the soil in front of the wall was treated was found to depend on the location of the slip surface. When the slip surfaces exit through the face or at the toe of the wall, MSEW 2.0 apparently ignores the soil in front of the wall. However, when slip surfaces pass completely below the toe of the wall, MSEW 2.0 includes the effect of the soil in front of the wall.

When general purpose slope stability software is used to perform global stability analyses, the effects of embedment are left to the discretion of the user. The soil in front of the wall can be either modeled to contribute to the stability or neglected. In fact, most software is very general and, for example, could even allow the user to include the stabilizing effect of the weight of the soil in front of the wall but ignore its shear strength.

3.4 Force Distribution in Reinforcement

Characterizations of the forces in the reinforcement are different depending on whether internal or global stability is being evaluated. External stability analyses are performed by treating the reinforced soil mass as a rigid body, and thus, the characterization of forces in individual reinforcement layers does not affect external stability calculations. The following sections discuss how the force is assumed to be distributed for internal and global stability analyses.

3.4.1 Internal Stability Analyses

Internal stability analyses deal with the force in the reinforcement at only one location, the point of maximum stress where T_{MAX}, T_a, and P_r are calculated. FHWA and AASHTO guidelines stipulate only that at the point of maximum stress the allowable design strength (T_a) and pullout resistance (P_r) of the reinforcement be greater than the maximum tensile force (T_{MAX}). Therefore, the distribution of force in the reinforcement is not required to perform internal stability analyses.

3.4.2 Global Stability Analyses

For global stability analyses, the variation in the force along the reinforcement can be important. The force applied to the slip surface depends on the force at the location where the slip surface intersects the reinforcement. Although the actual distribution of force along the length of the reinforcement is not known, the distribution is assumed on the basis of ultimate or allowable strengths.

3.4.2.1 Generalized Distribution of Force in the Reinforcement

The general shape of the assumed distribution of force in the reinforcement is shown in Figure 3-1.

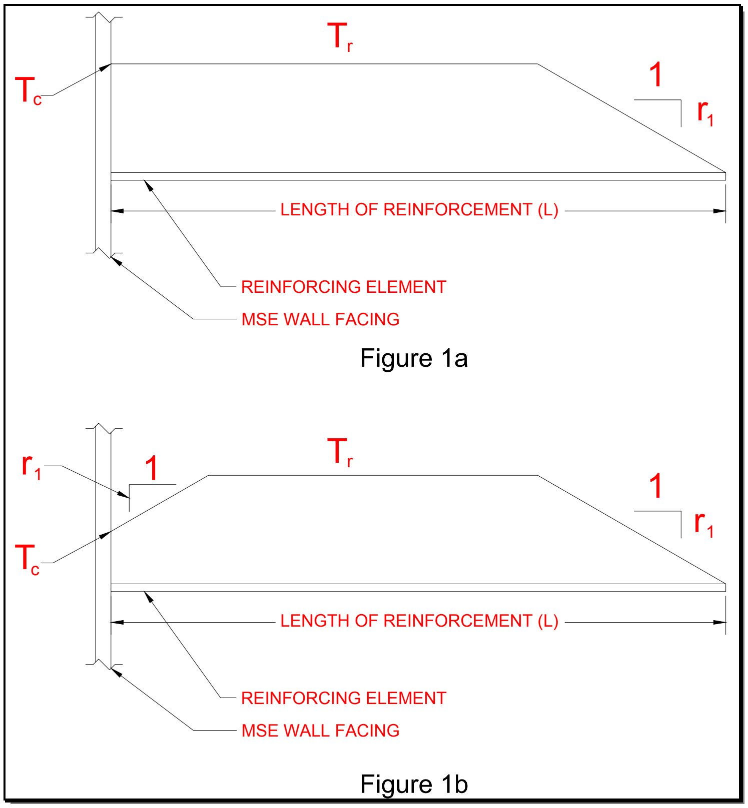

Figure 3-1: Distribution of force in the reinforcement when (a) the connection strength (T_c) is the same as the maximum potential force (T_r), and (b) the connection strength (T_c) is less than the maximum potential force (T_r).

The force near the embedded end of the reinforcement varies at the rate (r_1), which is calculated on the basis of the pullout resistance (P_r). From Section 2.3.2, the rate (r_1) is expressed as

$$r_1 = F^* \cdot \alpha \cdot \sigma_V \cdot C \cdot R_c . \tag{3-1}$$

The factors F*, α, σ_v, C, and R_c are defined in Chapter 2 (cf. Section 2.3.1.2.1). The rate is a function of the vertical stress (σ_v) and, thus, at large depths in the wall (r_1) can be very large and at shallow depths the rate (r_1) can be small. Pullout resistance increases at the rate (r_1) until the rupture strength of the reinforcement is reached.

The force along the middle section of the strip is characterized by the rupture strength, or the maximum potential force (T_r), that can be developed by the reinforcement. Typically, the magnitude for the maximum potential force (T_r) is assumed to be the long-term design strength (T_{al}) of the reinforcement. When a factor of safety is applied to the reinforcement force, the maximum potential force (T_r) is assumed to be the allowable design strength (T_a).

Near the wall face, the distribution of force begins to decrease at the rate (r_2), which is the same as the rate (r_1). The force in the reinforcement decreases until the force is equal to the connection strength (T_c) at the wall face. The connection strength (T_c) is generally provided by manufacturers and may vary depending on the type of reinforcement or facing used.

3.4.2.2 General Purpose Slope Stability Software

In general purpose slope stability software, the distribution of forces along the length of the reinforcement is usually assumed to be constant or, in software such as UTEXAS4, can be defined in a general way following a pattern like the one described in the previous section. For example in UTEXAS4, the designer can select the magnitude of forces T_r and T_c, as well as the rates (r_1) and (r_2) at which these forces develop. The global stability analyses conducted for this study using UTEXAS4 used the general distribution of forces shown in Figure 3-1.

3.4.2.3 The MSEW 2.0 Software

In the MSEW 2.0 software, the distribution of force in the reinforcement is different from the one shown in Figure 3-1. Instead, the distribution of force in the reinforcement is assumed to follow the shapes illustrated in Figure 3-2. The force designated as T_r shown in this figure is taken as the long-term design strength (T_{al}). The long-term design strength is calculated using the methods described in Chapter 2. When the strength of the connection (T_c) is at least equal to the rupture strength, the distribution of force is as illustrated in Figure 3-2a. When the strength of the connection (T_c) is less than the maximum potential force (T_r) in the reinforcement, the distribution is represented by Figure 3-2b. In this case, the force in the reinforcement increases at the rate (r_1) beginning at the embedded end of the reinforcement until the rupture strength (T_{al}) of the reinforcement is reached. From this point at which the rupture strength of the reinforcement is reached, the force in the reinforcement decreases linearly to the connection strength (T_c) at the wall face (Figure 3-2b).

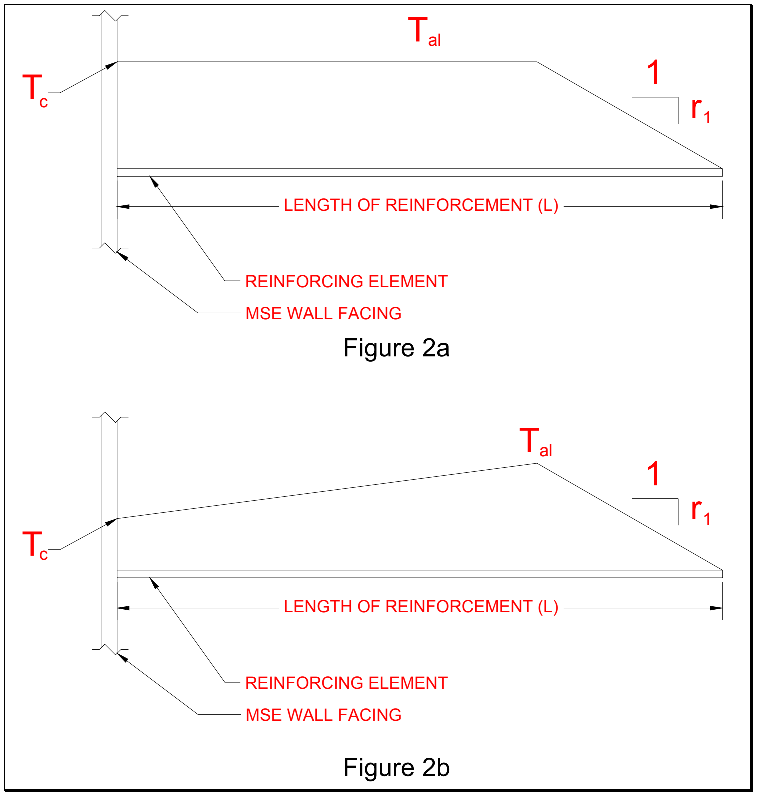

Figure 3-2: Distribution of force in the reinforcement modeled by MSEW 2.0 when (a) the connection strength (T_c) is the same as the maximum potential force (T_r), and (b) the connection strength (T_c) is less than the maximum potential force (T_r).

3.5 Contribution of Reinforcement when Slip Surface Exits Wall Face at a Reinforcement Layer

When a slip surface exits the wall face at precisely the same level as a layer of reinforcement, the way that the reinforcement force is treated can have a significant effect on the computed factor of safety. The force in the layer of reinforcement where the slip surface exits can be either included in the stability calculations or neglected.

In the MSEW 2.0 software, factors of safety are calculated for slip surfaces exiting through the wall face exactly at the level of each layer of reinforcement. For these slip surfaces, the force in the reinforcement layer at the exit point is included in the stability calculations. The MSEW 2.0 software actually calculates the force as though the slip surface intersects the exit level reinforcement at a distance of 0.02 ft. behind the face of the wall.

For most general purpose software, the contribution of the reinforcement force when the slip surfaces exits the face of the wall probably varies from software program to software program and in general is unknown. However, as an example, the UTEXAS4 software includes only the contribution of a layer of reinforcement if the slip surface actually intersects that layer. If a slip surface exits exactly at the elevation of a reinforcement layer without intersecting the layer, the force in the layer is not included in the calculation of factor of safety.

3.6 Shape of the Slip Surfaces

When global stability analyses are performed, a choice can be made between using circular or noncircular slip surfaces. FHWA and AASHTO guidelines provide little information on the shape of slip surfaces that should be used for analyses for global stability: FHWA 2001 specifies only that the global stability be determined using "rotational" or "wedge" analyses, as appropriate. AASHTO 2002 does not state the shape of slip surfaces that should be assumed; however, all illustrations relating to global stability show circular slip surfaces.

The shape of the slip surface producing the minimum factor of safety is expected to depend on the particular soil conditions and the type and layout of reinforcement. If relatively inhomogeneous soil conditions exist, noncircular slip surfaces may be the most critical. However, for relatively homogeneous soil conditions, circular slip surfaces are probably adequate. The MSEW 2.0 software is restricted to the use of circular slip surfaces for global stability analyses, whereas many general purpose computer programs, including UTEXAS4, allow both circular and noncircular slip surfaces to be used. For the relatively homogeneous soil conditions considered in this study, circular slip surfaces were used for all analyses.

3.7 Subdivision of Slices

Most slope stability software uses a "procedure of slices" that requires the soil mass be subdivided into a finite number of vertical slices. If an insufficient number of slices is used, it can affect the computed factor of safety and even the location of the critical slip surface.

The MSEW 2.0 software initially divides the sliding mass into fifty slices of equal width. Then any slice with its base extending through two or more different soils is subdivided so that the base of each slice is in only one soil. Consequently, the actual number of slices may be slightly more than fifty.

The UTEXAS4 software subdivides the soil mass bounded by the circular slip surface and the surface of the MSE wall so that the angle that is subtended by the two radii extended to each side of the base of the slice (slip surface) does not exceed a given value (Figure 3-3). The default value for the subtended angle (θ) is 3° in the UTEXAS4 software, but the user can specify a maximum. Studies conducted by Wang (2004) concluded that both the MSEW 2.0 and UTEXAS4 software programs use a sufficient number of slices to not significantly affect the factor of safety.

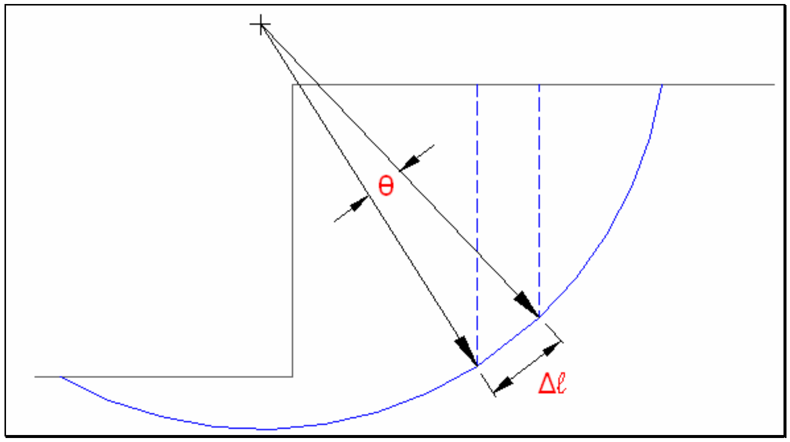

Figure 3-3: Illustration of subtended angle (θ) and arc length (Δℓ) used as criteria for subdivision of circular shear surfaces into slices.

3.8 Lower Limit on Centers of Circles

Experience with analyses of a number of reinforced steep slopes and walls has shown that the center of a critical circle can tend to fall below the top of the slope such that the circle tends to curve back on itself (Figure 3-4). This situation can be dealt with either by introducing a vertical crack where the slip surface first becomes vertical (Figure. 3-5) or by rejecting any circle that tends to curve back on itself.

Introducing a vertical crack (Figure 3-5) may result in the reinforcement forces being neglected where the reinforcement crosses the crack. When this occurs, the slope stability software that is searching for the lowest factor of safety may seek progressively lower and lower center points for the circle because more and more of the reinforcement forces are neglected and the factor of safety becomes progressively lower. Eventually, the centers of circles may approach the level of the toe of the slope or wall. Consequently, introduction of a vertical tension crack when the center of the circle falls below the top of the slope or wall is not always a viable solution for reinforced structures.

To overcome the condition described above, the MSEW 2.0 software restricts the lowest elevation of the centers of circles to the top of the wall. The UTEXAS4 software permits the user to restrict the centers of circles to be above any selected elevation. For this study, the centers of circles were restricted to being at or above to the top of the wall. The FHWA and AASHTO guidelines provide no information regarding this issue.

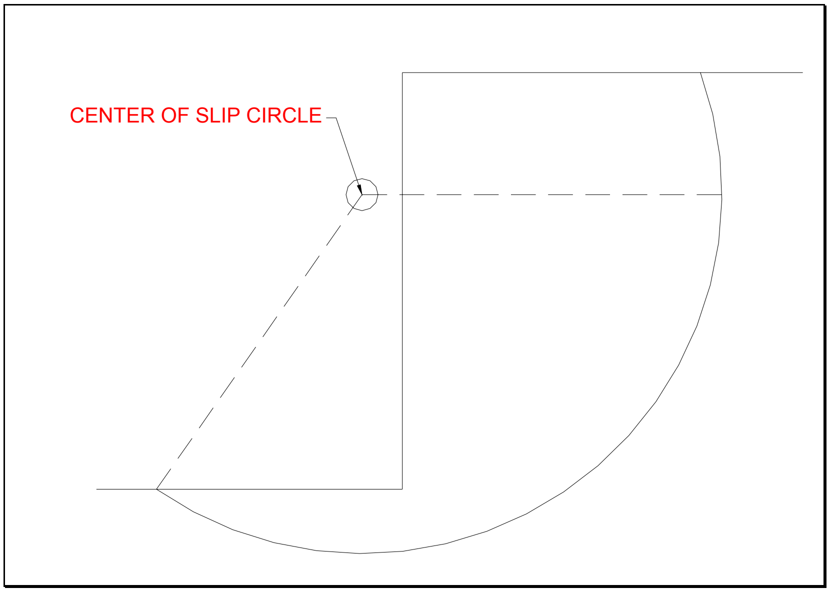

Figure 3-4: Illustration of circular slip surface when the center of the circle falls below the top of the wall.

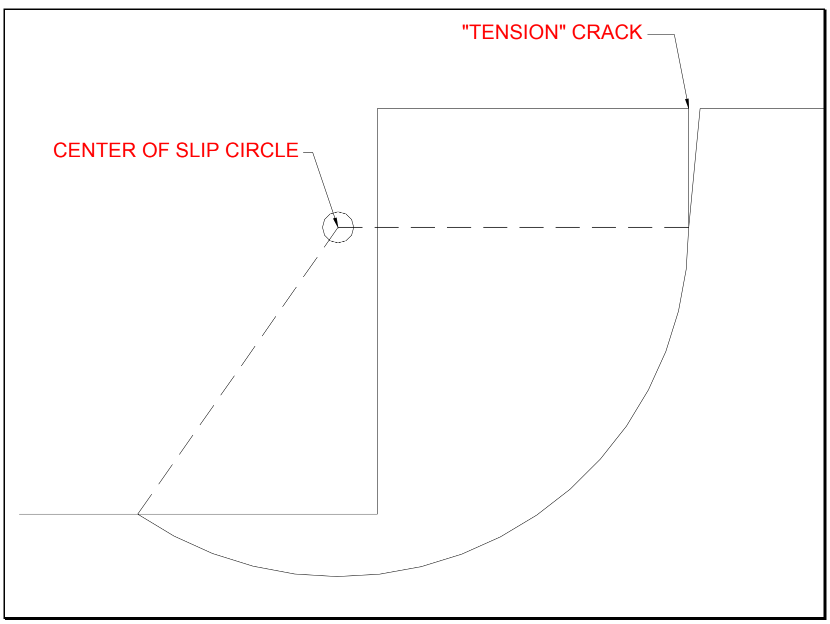

Figure 3-5: Vertical tension crack extending from the top of the wall to the point where the slip circle first becomes vertical.

3.9 Extent of Search for Critical Slip Surfaces

The MSEW 2.0 software performs two distinctly separate searches for a critical slip surface. The first search is referred to by the MSEW 2.0 software as *Compound Stability Analysis*. In this search, slip surfaces exiting the face and toe of the wall are examined. For slip surfaces that exit through the face of the wall, the search is restricted to circles that exit at the exact elevation of layers of reinforcement. The MSEW 2.0 software does not search using slip surfaces that exit the wall face between layers of reinforcement.

The second search performed by the MSEW 2.0 software is referred to as a *Deep-Seated Global Stability Analysis*. In this search, slip surfaces that pass entirely below the toe of the wall are examined. The second search is performed independently of the first and requires the user to enter separate input data. If a slip circle using the Deep-Seated search intersects the toe or the wall face, the circle is automatically rejected.

In the UTEXAS4 slope stability software, slip surfaces can be analyzed that pass anywhere through or below the wall. The search for critical slip surfaces can be either unrestricted or restricted by the user to certain zones in the wall or foundation soil. Probably many other general purpose slope stability programs have similar capabilities.

3.10 Location of Reinforcement Force on Slice

Studies by Wright and Duncan (1991) show that reinforcement forces can be applied to individual slices in a stability analysis in two different ways:

1. Forces can be applied only at the point where the reinforcement intersects the slip surface (Figure 3-6), or
2. forces can be applied to the boundaries between each slice as well as to where the reinforcement intersects the slip surface (Figure 3-7).

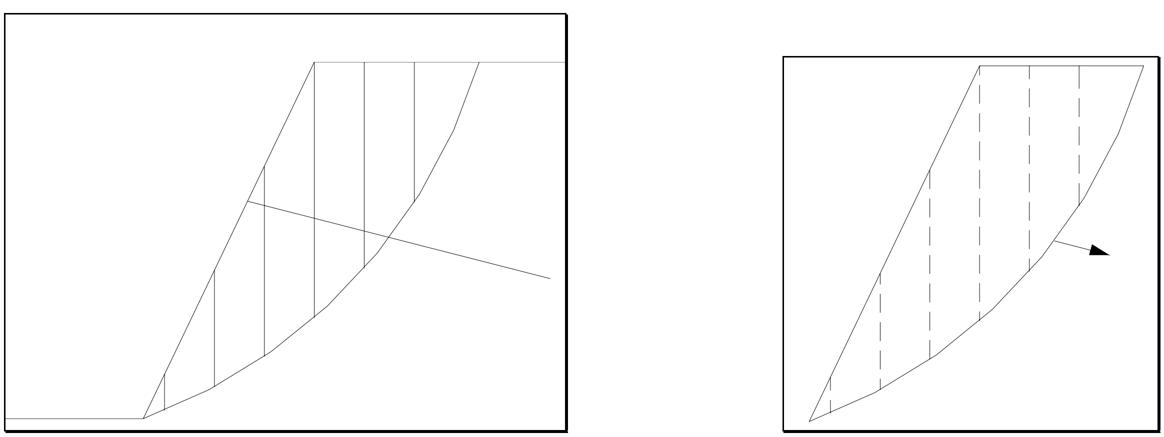

Figure 3-6: Reinforcement force applied only to the base of the slice (after Wright and Duncan, 1991).

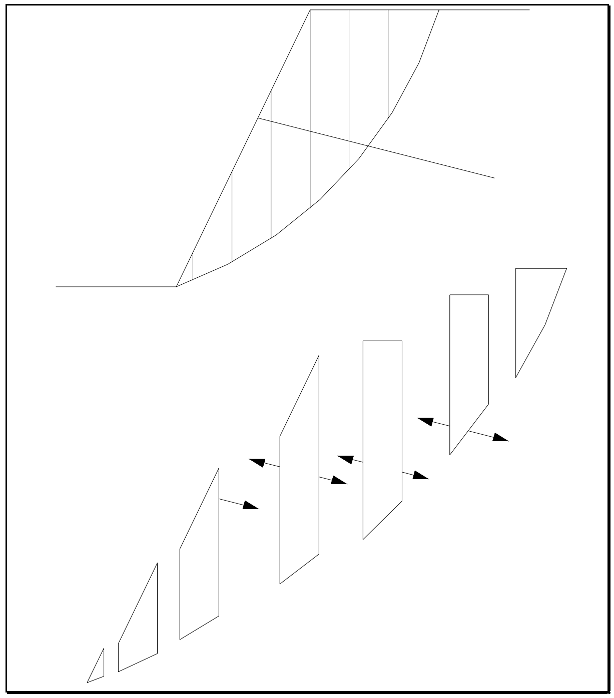

Figure 3-7: Reinforcement forces applied to slice boundaries and base of slice (after Wright and Duncan, 1991).

In the MSEW 2.0 software, reinforcement forces are applied only to the base of the slice. The UTEXAS4 software allows the reinforcement forces to be applied in either of the two ways described above. However, the studies by Wright and Duncan (1991) with the reinforcement forces applied in both ways concluded the difference in the computed factor of safety was very small.

3.11 Rotation of Reinforcement: Orientation of Forces

The tensile forces in the reinforcement can either be assumed to act in the original, generally horizontal, direction of the reinforcement or be rotated by some amount assuming distortion in the reinforcement and soil backfill. In the most extreme case of distortion, reinforcement forces can be assumed to be tangent to the slip surface.

The MSEW 2.0 software allows the user to select the orientation of the reinforcement forces as horizontal, tangential, or some angle between horizontal and tangential. The default assumption for the MSEW 2.0 software is that the reinforcement forces are tangent to the slip surface. The UTEXAS4 default setting assumes the tensile reinforcement forces are parallel to the direction of the reinforcement, and the user can assign other inclinations as an option.

Studies conducted by Wright and Duncan (1991) report that differences in calculated factors of safety using horizontal reinforcement forces as compared with forces tangent to the slip surface were negligible. This conclusion is applicable to limit equilibrium procedures of slices where all components of the reinforcement force are included in the corresponding equations of equilibrium.

The MSEW 2.0 software documentation states that the reinforcement force contributes the greatest amount to the factor of safety when the force acts tangent to the slip surface, and thus this condition should produce *larger* factors of safety. However, in the course of this study, global stability analyses were performed using the MSEW 2.0 software, and factors of safety were found to be *smaller* when forces were tangent to the slip surface, which apparently contradicts the MSEW 2.0 software documentation. To illustrate this, factors of safety were computed using the Comprehensive Bishop procedure in the MSEW 2.0 software for three hypothetical walls. The walls were 10, 30, and 50 ft. tall, and the reinforcement was designed to satisfy FHWA guidelines. The reinforced and foundation soils were assumed to have angles of internal friction of 34° and 40°, respectively. The computed factors of safety are summarized in Table 3-1 for both horizontal and tangent reinforcement forces. For each of the three hypothetical walls, the computed factors of safety were lower when the reinforcement was tangent to the slip surface than when it was horizontal (Table 3-1). However, the differences in factor of safety for horizontal and tangential reinforcement forces are small (less than 4 percent).

Table 3-1: Factors of safety calculated with MSEW 2.0 using reinforcement forces horizontal and tangent to the slip surface.

Wall Height (ft)	Orientation of Reinforcement Force	Coordinates of Critical Circle			Factor of Safety
		Xc (ft)	Yc (ft)	R (ft)	
10	Horizontal	−10.8	36.3	37.9	2.95
	Tangent	−10.3	35.3	36.8	2.90
30	Horizontal	−87.7	136.9	162.6	1.79
	Tangent	−70.9	114.9	135.0	1.74
50	Horizontal	−216.1	327.5	392.4	2.14
	Tangent	−94.4	189.1	211.4	2.12

3.12 Search for Critical Slip Surface

The search for a "critical" slip surface with a minimum factor of safety involves at least three important variables: (1) the shape of the slip surface, (2) the general search regimen or scheme, and (3) the search "grid" spacing.

Common shapes of slip surfaces include circular, noncircular, and a wedge or sliding block. For circular slip surfaces, two common search schemes employed in software are *fixed* grid and *floating* grid searches. A fixed grid provides a specific search area and grid spacing where centers of circles are located and analyzed. The search area has prescribed boundaries and the search is restricted to within those boundaries. A floating grid search employs a moveable grid, and the grid spacing may be reduced as the center of the critical circle is approached. The floating grid often requires far fewer circles to be analyzed and, thus, can be much more efficient. Also, a certain efficiency can be achieved in a floating grid search by automatically reducing the size of the grid as the search progresses. In contrast, the grid spacing for a fixed grid search must be as fine as the finest spacing required.

Global stability analyses performed using the MSEW 2.0 software utilize a fixed grid for the search, whereas with the UTEXAS4 software a floating grid is generally used.[1] Both MSEW 2.0 and UTEXAS4 use a grid spacing for searching that is defined by the user. If suitably small spacings are used, both should give the correct critical circle, although the efficiency (number of circles analyzed) may differ.

Circles were used for the present study; however, if noncircular slip surfaces need to be investigated, general purpose software such as UTEXAS4 must be used. The MSEW 2.0 software cannot search for a critical noncircular slip surface.

3.13 Definition for the Factor of Safety

FHWA and AASHTO guidelines clearly define factors of safety for both external and internal stability. However, little or no guidance is given for the factor of safety for global stability.

For global stability using the MSEW 2.0 software, the user selects one of three options: (1) the Comprehensive Bishop method, (2) the Demo 82 Approach, or (3) the L–Modification to the Demo 82 Approach. Each option corresponds to one of the three definitions of factor of safety discussed in Chapter 2 (cf. Section 2.4.3).

In UTEXAS4 and most other general purpose slope stability software, the factor of safety is defined with respect to soil shear strength. Any factor of safety applied to the reinforcement force must be applied by the user before the reinforcement forces are input into the program.

Factors of safety computed using each of the three definitions of factor of safety can produce significantly different values and critical slip surfaces (Wang, 2004). Thus, it is important to know how the factor of safety is being defined.

3.14 Limit Equilibrium Procedure

AASHTO 2002 stipulates that a limit equilibrium analysis using the Modified Bishop, Simplified Janbu, or Spencer method may be used to evaluate global stability for MSE walls. FHWA guidelines do not provide specifications on which limit equilibrium procedure should be used.

For global stability, MSEW 2.0 employs Bishop's Simplified procedure for each of the three analysis options available: (1) Comprehensive Bishop, (2) the Demo 82 Approach, and (3) the L–Modification to the Demo 82 Approach. In the UTEXAS4 software, the user has the option of using the Spencer's, Bishop's Simplified, or the Simplified Janbu procedures.

3.15 Other Issues

Several issues affecting the calculated factor of safety were discovered when comparing calculations performed by hand, using the MSEW 2.0 software, and by the original designers of actual walls. It was found that factors of safety calculated by hand

[1] The UTEXAS4 software allows the user to perform searches with both fixed and floating grids.

and using the MSEW 2.0 software for internal stability (i.e., rupture and pullout of the reinforcement) did not agree even when the assumptions used to create the model were believed to be the same. The disagreement in the factors of safety is caused by several different assumptions made about how the internal forces in the wall are calculated.

Calculating Maximum Reinforcement Tension (T_{MAX})

Two different approaches are used to calculate the maximum tension (T_{MAX}) acting on the reinforcement. The first approach, suggested by the FHWA and AASHTO guidelines, is to compute the horizontal stress (σ_H) at the depth of the layer of reinforcement. The second approach, employed in the MSEW 2.0 software, is to calculate the horizontal stress as an average value based on a trapezoidal distribution of stress (Figure 5-8). The average horizontal stress is the arithmetic average of the stress ($\sigma_{h\text{-}a}$) at the mid-layer point directly above and the stress ($\sigma_{h\text{-}b}$) at the mid-layer point directly below a given reinforcement layer (Figure 3-9).

In general, the difference between the two assumptions is small for the lower layers of reinforcement in a wall and when the vertical spacing of the reinforcement is constant.

The differences in the maximum reinforcement tension described above can be very large for multi-tier walls. For example, consider the two-tier wall shown in Figure 3-10. The differences between the average of the stresses, sha and shb, and the stress at the depth, Zi, will obviously be quite large. The horizontal stress distribution used to calculate the maximum tensile force (T_{MAX}) on the top layer of reinforcement in the lower tier of a two-tier wall is illustrated in Figure 3-10.

There are also differences in how the horizontal stress is calculated in the FHWA and AASHTO guidelines. Older (1996) versions of the AASHTO guidelines specified that the vertical stress be calculated as

$$\sigma_v = (\gamma_r \cdot Z + \sigma_2 + q + \Delta\sigma_v) \cdot \frac{L}{L-2e} \tag{3-3}$$

where

L = length of the reinforcement and

e = eccentricity of the resultant of the vertical forces acting on the layer of reinforcement.

This equation (3-3) for calculating the vertical stress produces larger stresses on the layer of reinforcement than the equations used in the current FHWA 2001 and AASHTO 1998 guidelines. Also, in the older AASHTO 1996 guidelines the horizontal stress (σ_H) was calculated using different lateral earth pressure coefficients (K_r) than used by current FHWA and AASHTO guidelines. In the AASHTO 1996 guidelines, the lateral earth pressure coefficient (K_r) varies from the at-rest condition (K_0) at the top of the wall to the active condition (K_a) at 20 ft. below the top of the wall. In current FHWA and AASTHO guidelines, the lateral earth pressure coefficient (K_r) is calculated using a multiplier applied to the active earth pressure coefficient (K_a). The difference can be illustrated by examining the calculated lateral earth pressure coefficients (K_r) for three types of reinforcement and soil with an angle of internal friction of 34° (Table 3-2).

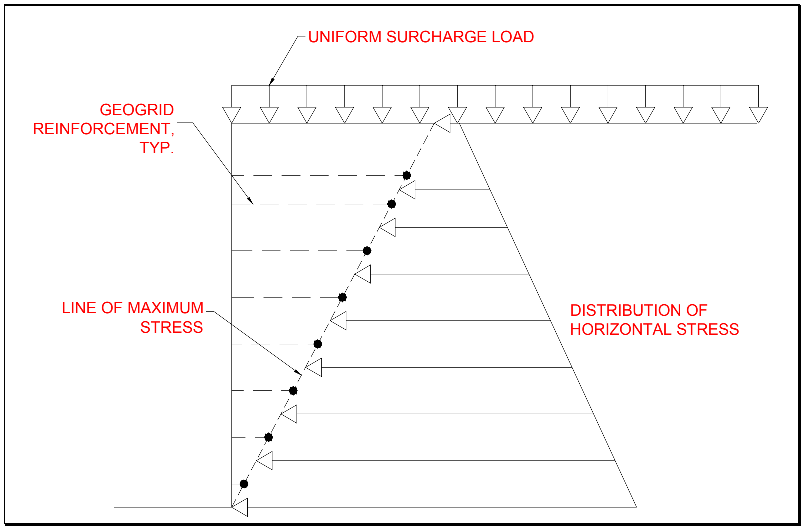

Figure 3-8: Distribution of horizontal stress used to calculate the maximum reinforcement tension (T_{MAX}) using the MSEW 2.0 software.

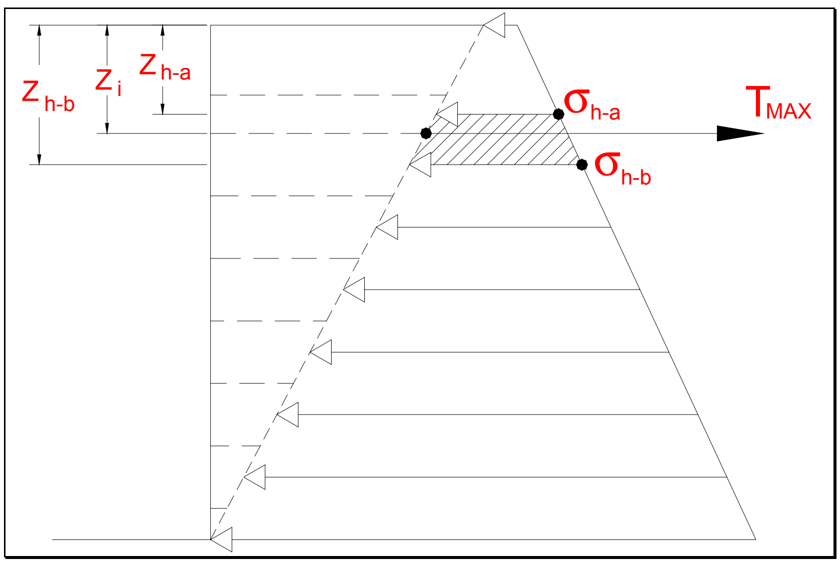

Figure 3-9: Illustration of the horizontal stresses used to calculate T_{MAX} using the MSEW 2.0 software (surcharge load omitted for clarity).

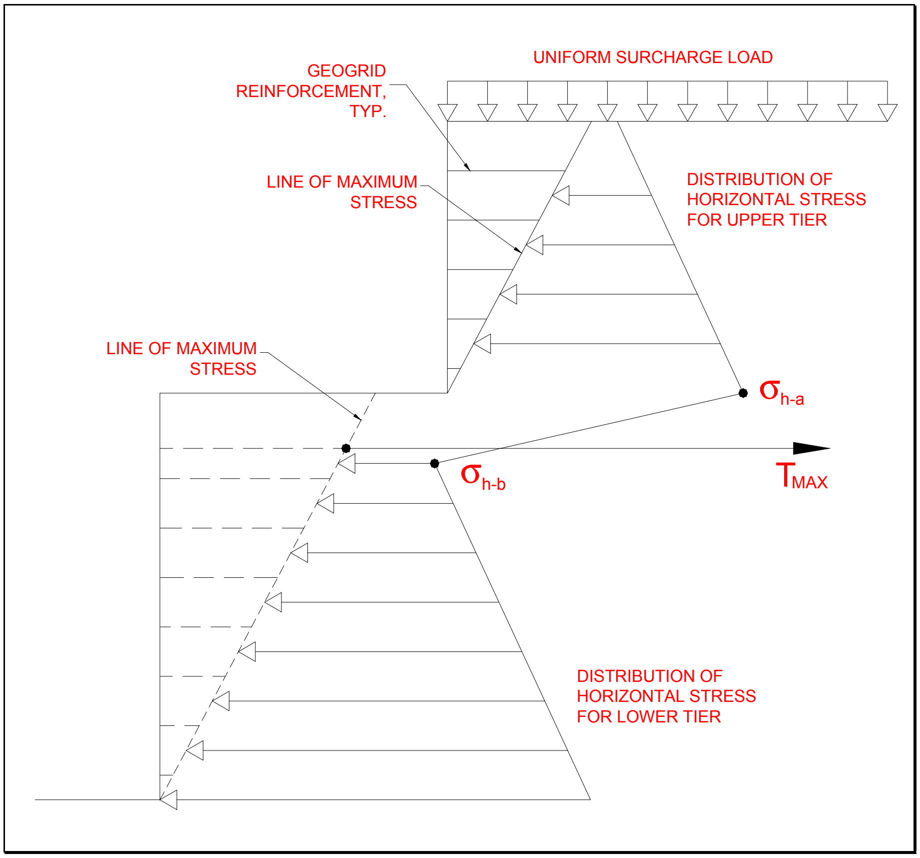

Figure 3-10: Illustration of horizontal stress distribution for multi-tier walls and the stress used in the MSEW 2.0 software to calculate maximum reinforcement tension (T_{MAX}).

Table 3-2: Comparison of the current FHWA and AASHTO guidelines calculation of lateral earth pressure coefficient (K_r) with the method use by the previous version (1996) of AASHTO.

Depth (ft)	FHWA 2001 and AASHTO 1998						AASHTO 1996
	Metallic Strips		Metallic Grid		Geogrid		All Reinforcement
	K_r/K_a	K_r	K_r/K_a	K_r	K_r/K_a	K_r	K_r
0	1.7	0.481	2.5	0.707	1.0	0.283	$K_0 = 0.441$
20	1.2	0.339	1.2	0.339	1.0	0.283	$K_a = 0.283$

Vertical Stress from a Sloping Backfill (σ_2)

The MSEW 2.0 software uses an assumption different from the FHWA and AASHTO guidelines to calculate the vertical stress caused by sloping backfill for single-tier walls. According to FHWA and AASHTO guidelines, the vertical stress from a sloping backfill (σ_2) is defined as

$$\sigma_2 = \frac{1}{2} \cdot L \cdot (\tan \beta) \cdot \gamma_f \tag{3-4}$$

where

L = length of the reinforced soil,

β = the slope of the backfill (Figure 3-11), and

γ_f = total unit weight of retained soil.

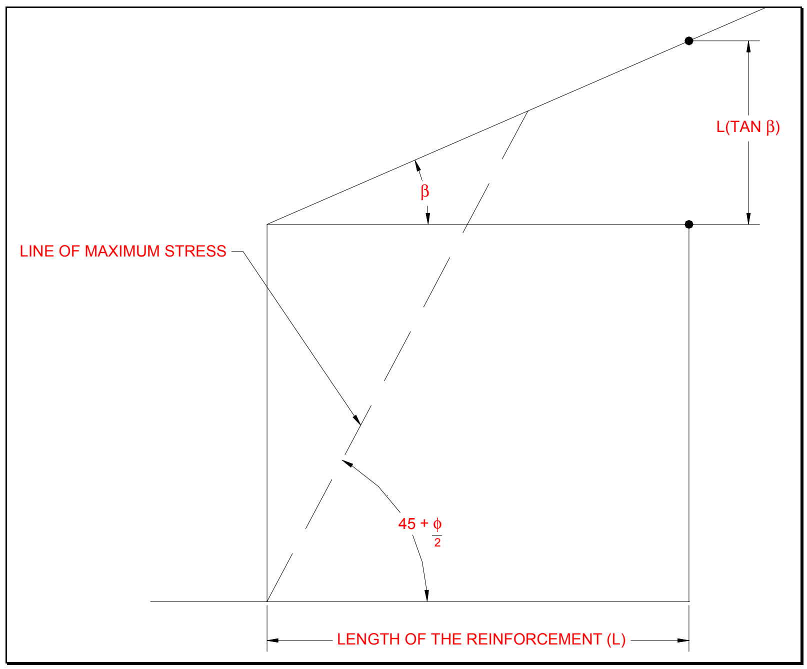

Figure 3-11: Illustration of the dimensions used by FHWA and AASHTO guidelines to calculate the vertical stress caused by the sloping backfill (σ_2).

In contrast, the MSEW 2.0 software defines the vertical stress for a sloping backfill ($\sigma_{2\text{-MSEW}}$) as

$$\sigma_{2-MSEW} = \frac{1}{2} \cdot \gamma_f \cdot h_s \tag{3-5}$$

where

γ_f = total unit weight of retained soil and

h_s = distance from top of reinforced soil zone to the point where the line of maximum tension intersects the top of the ground surface (Figure 3-12)

The vertical stresses calculated according to the FHWA and AASHTO guidelines will be larger than these calculated by the MSEW 2.0 software.

For multi-tier walls with a sloping backfill behind the upper tier, it was found that the MSEW 2.0 software apparently neglects the vertical stress caused by the sloping

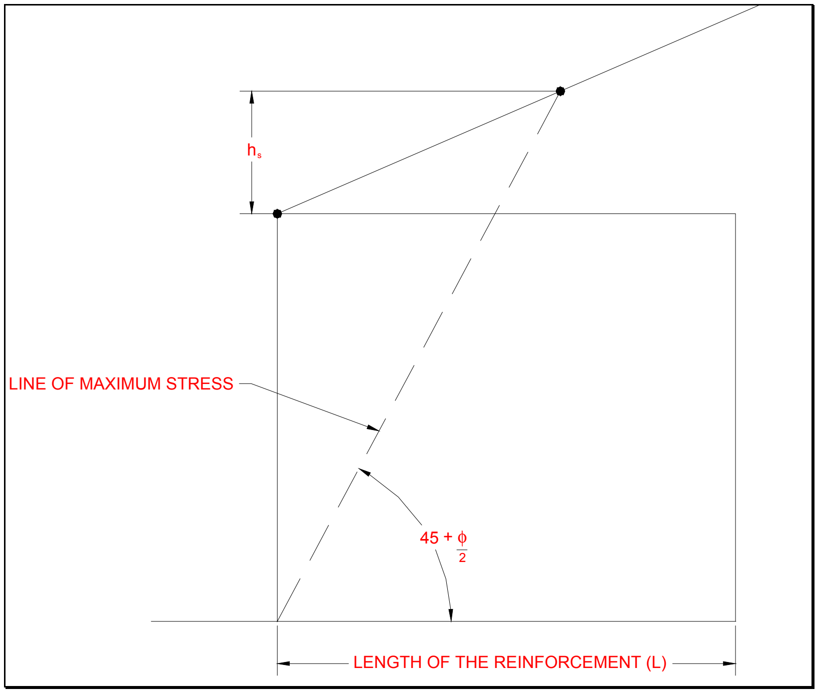

Figure 3-12: Illustration of the height of the sloping backfill (h_s) used by the MSEW 2.0 software to calculate the vertical stress ($\sigma_{2\text{-}MSEW}$).

backfill. Although FHWA and AASHTO guidelines do not directly address this issue, it is logical to assume the effects of a sloping backfill for a multi-tier wall could be calculated using the same method as that used for single-tier walls (Equation 3-4).

Pullout Resistance (P_r) in Multi-Tier Walls

The pullout resistance (P_r) of the reinforcement is defined for single-tier walls by the FHWA and AASHTO guidelines as shown in Chapter 2:

$$P_r = F^* \cdot \alpha \cdot \gamma_r \cdot Z_P \cdot L_e \cdot C \qquad (2\text{-}17)$$

where

the pullout resistance coefficients F*, α, γ_r, L_e and C are defined in Chapter 2 (cf. Section 2.3.1.1) and

Z_P is defined as the depth to the layer of reinforcement.

For multi-tier walls, the FHWA and AASHTO guidelines provide no guidance on how the depth (Z_P) is measured and, thus, different assumptions were made for calculations by hand and by the MSEW 2.0 software. Calculations by hand assumed the depth (Z_P) to the reinforcement was measured as illustrated in Figure 3-13. For layers of reinforcement in the lower tier, $Z_P = Z_{P\text{-}L}$, where the distance measured from the ground surface of the *lower* tier to the reinforcement where it intersects the line of maximum stress. For layers of reinforcement in the upper tier, $Z_P = Z_{P\text{-}U}$, where the distance is measured from the ground surface of the *upper* tier to the reinforcement where it intersects the line of maximum stress. The assumption made by the MSEW 2.0 software for a two-tier wall is that the depth (Z_P) is measured from the top of facing on the *upper* to the layer of reinforcement, regardless of in which tier the reinforcement is located (Figure 3-14).

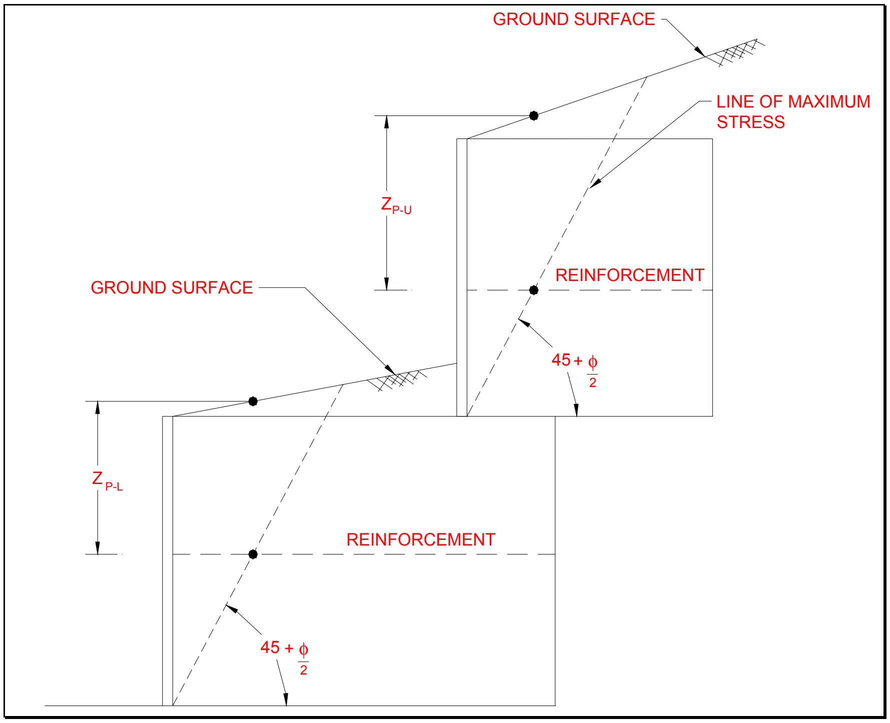

Figure 3-13: The depth (Z_P) used to calculate the pullout resistance of the reinforcement for multi-tier walls using the FHWA and AASHTO guidelines.

Both assumptions discussed for the depth, Z_P (Figures 3-13 and 3-14), are approximate and the actual stress is unknown. The assumption for the calculations by hand was chosen because it produces a smaller pullout resistance and, thus, is more conservative.

3.16 Summary

The procedures and values used for design of MSE walls vary with design guidelines, design software, and the individual engineer. In the course of this study, numerous differences were noted in the procedures and assumptions used. These differences were found to potentially have a significant effect on the computed factors of safety and, thus, the evaluation of the stability of the wall. This chapter was dedicated to the identification and discussion of these issues, because many of the assumptions are either not well documented or not intuitively obvious. The assumptions and how each is treated by various design guidelines and software are summarized in Table 3-3.
In Table 3-3, the way uniform surcharge loads are treated is presented separately for external, internal, and global stability because surcharge loads are sometimes treated differently depending on the stability condition. The effects of the embedment of the wall are presented separately for conventional external stability analyses and general global stability analyses. The procedure for calculation of horizontal stress (σ_H) and pullout resistance (P_r) are outlined for each design guideline and software program for both single-

and multi-tier walls. The table covers contribution of a sloping backfill to the calculation of vertical stress and how the lateral earth pressure coefficient (K_r) is calculated.

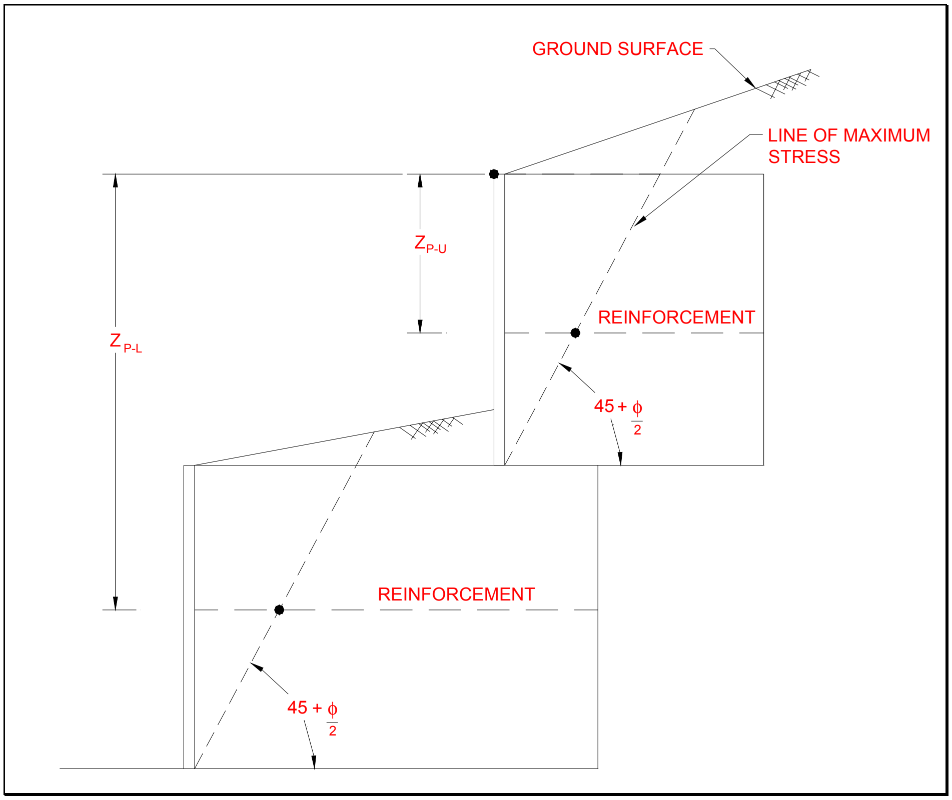

Figure 3-14 The depth (Z_P) used to calculate pullout resistance (P_r) for a multi-tier wall using the MSEW 2.0 software.

Assumptions that pertain solely to global stability analyses include the following.

- Effects of embedment depending on the location of the critical slip surface
- Distribution of forces in the reinforcement
- Contribution of the reinforcement when the slip surface exits the wall face at a reinforcement layer
- Shape of the slip surfaces
- Subdivision of the soil mass into slices
- Imposed lower limit on the centers of circles
- Search for critical slip surfaces, including the search scheme and grid spacing
- Location of the reinforcement force on individual slices
- Rotation of reinforcement forces
- Definition of the factor of safety

The effects of these issues and variables will be explored further in the next two chapters, in which analyses and designs for specific walls are examined. Chapter 4 addresses the issues found during the course of this study for single-tier walls, and Chapter 5 addresses issues found for multi-tier walls.

Table 3-2: Assumptions and variables used by design guidelines and software programs found to effect the design of MSE walls.

Possible Assumptions and Variables		*MSEW 2.0*	*UTEXAS4*	*FHWA*	*AASHTO*
Uniform Surcharge	*External Stability*	Same as FHWA.	User defines surcharge loads.	For bearing capacity a "live" surcharge load is applied to top of wall beginning at the wall face and extending beyond the reinforced soil zone. For sliding and overturning, "live" surcharge is applied to top of the wall beyond the reinforced soil zone.	Same as FHWA.
	Internal stability	Same as FHWA.	User defines rupture strength and pullout resistance of the reinforcement as well as how external loads are applied to wall.	"Dead" loads contribute to the forces causing rupture and pullout, as well as the development of reinforcement resistance. "Live" load contribute to the forces causing rupture and pullout, but not to the development of reinforcement resistance.	Same as FHWA.
	Global stability	"Dead" loads contribute to the development of reinforcement strength, but not to the forces causing failure. "Live" loads do not contribute to the development of reinforcement strength or to the forces causing failure.	User defines rupture strength and pullout resistance of the reinforcement as well as how external loads are applied to wall.	Requirements not specified.	Same as FHWA.
Embedment Soil in Front of Wall	*External Stability (sliding, overturning, or bearing capacity failure)*	Same as FHWA.	User defines the contribution of the embedment soil.	Contribution of soil in front of wall ignored.	Same as FHWA.
	Slip Surface Through Wall	Contribution of the soil in front of the wall is ignored.	User defines the contribution of the embedment soil.	Requirements not specified.	Same as FHWA.
	Slip Surface Through Toe				
	Slip Surface below the toe	For failure surfaces that are below the toe, the soil in front of the wall is included in the stability calculations.			
Force Distribution in Reinforcement		Force increases at rate (r_1) from the embedded end of reinforcement until force is equal to the maximum potential force (T_r), then decreases linearly to the Connection Strength (T_c) of the reinforcement at the wall face.	Force increases at rate (r_1) from the embedded end of reinforcement until force is equal to the maximum potential force (T_r), then remains constant and finally decreases at rate (r_1) to the sonnection strength (T_c) of the reinforcement at the wall face.	None specified, but implied by the calculation of maximum tensile force (T_{MAX}), allowble design strength (T_a) and pullout resistance (P_r) at the line of maximum stress. This implies that each is fully developed at this point.	Same as FHWA.
Contribution of Reinforcement at Point Where Slip Surface Exits Wall Face	*Slip Surface exits at reinforcement*	Reinforcement force contributes to stability.	Reinforcement force does not contribute to the factor of safety calculations.	Requirements not specified.	Same as FHWA.
	Slip Surface Exits between reinforcing layers	This condition does not exist in the MSEW 2.0 software.			
Shape of Slip Surfaces		Circular slip surfaces.	Circular or non-circular slip surfaces.	Rotational or wedge analyses with failure surfaces outside reinforced soil mass	Requirements not specified.
Subdivision of Slices		Divides sliding mass into 50 equal slices, then subdivides further so that base of each slice in only one soil type.	Subdivides soil using a subtended angle (θ) of 3°, then subdivides further so that the base of each slice in only one soil type.	Requirements not specified.	Same as FHWA.
Lower Limit on Centers of Circles		Restricts lowest elevation on the centers of circles to the top of wall.	Software can restrict the lowest elevation on the centers of circles to any elevation or introduce a tension crack where circle first becomes vertical.	Requirements not specified.	Same as FHWA.
Extent of Search for Critical Slip Surfaces		Software searches for critical slip surfaces through the wall, through the toe and below the toe of the wall. Searches through the wall face are restricted to layer at each layer of reinforcement, intermediate elevations cannot be searched.	Slip surfaces anywhere through or beyond the wall can be analyzed.	Requirements not specified.	Same as FHWA.
Location of Reinforcement Force on Slice		Reinforcement forces are applied only at the point where the reinforcement intersects the slip surface.	Can be applied where the slip surface intersects reinforcement, or applied to the boundaries between each slice and where the reinforcement intersects the slip surface.	Requirements not specified.	Same as FHWA.
Rotation of Reinforcement - Orientation of Forces		Default orientation of the reinforcement force is tangent to the slip surface, user can define other orientations.	Default orientation is tangent to the slip surface, user can define other orientations.	Requirements not specified.	Same as FHWA.
Search for Critical Slip Surface	*General Search Scheme*	Employs a fixed grid search scheme.	Employs either a fixed or floating grid search.	Requirements not specified.	Same as FHWA.
	Minimum Grid Spacing	User defined, 1% of wall height was used for this study.	User defined, 1% of wall height was used for this study.	Requirements not specified.	Same as FHWA.
Definition for the Factor of Safety	*Applied to Soil Shear Strength*	Comprehensive Bishop Method (YES) Demo 82 Method (YES) L (Leshchinsky) Modification of Demo 82 Method (NO)	Always applied to soil shear strength.	Definitions for factor of safety not specified.	Same as FHWA.
	Applied to Reinforcement Force	Comprehensive Bishop Method (NO) Demo 82 Method (YES) L (Leshchinsky) Modification of Demo 82 Method (YES)	N.A. - Must be applied to reinforcement force before being input into analysis software.		
Calculation of Horizontal Stress (σ_H)	*Single-tier Walls*	Horizontal stress is calculated by averaging the stress from a trapezoidal distribution.	N.A. - Does not perform purely internal stability calculations and, thus, does not calculate horizontal stress.	Horizontal stress is calculated at the layer of reinforcement.	Same as FHWA.
	Multi-tier Walls			Horizontal stress is calculated at the layer of reinforcement with the contribution of the upper tier calculated as an additional stress.	Does not specify any additional requirements for calculating the horizontal stress for multi-tier walls.
Calculation of Pullout Resistance (P_r) of the Reinforcement for Multi-tier Walls	*Single-tier Walls*	Pullout resistance is calculated using the depth from the top of the of the wall.	N.A. - Does not perform purely internal stability calculations and, thus, does not calculate pullout resistance.	Pullout resistance is calculated using the depth from the top of the wall to the layer of reinforcement.	Same as FHWA.
	Multi-tier Walls	Pullout resistance is calculated using the depth from the top of the upper tier to the layer of reinforcement.		No additional requirements specified for calculating the horizontal stress for multi-tier walls.	Same as FHWA.
Contribution of the Sloped Backfill on the Vertical Stress		Calculates the vertical stress applied to the reinforced soil for a height (h_s), where h_s is the vertical distance from the top of wall face to the point where the line of maximum tension intersects the ground surface at the top of the wall.	N.A. - Does not perform purely internal stability calculations and, thus, a separate factor for the vertical stress.	Vertical stress caused by the sloped backfill calculated using the vertical distance from the top of the wall face to a point on the ground surface directly above the back of the reinforced soil mass.	Same as FHWA.
Lateral Earth Pressure Coefficient (K)		Same as FHWA.	N.A. - Does not perform internal stability calculations.	Calculates (K) using a multiplier applied to K_a, which varies for different types of reinforcement. For example, a metallic strip reinforcement, the lateral earth pressure coefficient decreases linearly from 1.7*(Ka) at the top of the wall to 1.2*(Ka) at a depth of 6 m (19.68 ft) below the top of the wall. The lateral earth pressure coefficient is assumed to be constant at 1.2*(Ka) below 6 m. Reference Figure 2-1.	AASHTO (1998) guidelines require K calculated the same as discussed for the FHWA (2001). AASHTO (1996) assumes the lateral earth pressure coefficient (K) varies from the at-rest condition (K0) at the top of the wall to the active condition (Ka) at 20 ft below the top of the wall.
Depth (Z_P) used to Calculate the Pullout Resistance	*Single-Tier Wall*	Same as FHWA.	N.A. - Does not perform internal stability calculations.	Depth (Z_P) measured for the top of the wall at the ground surface to the layer of reinforcement where it intersects the line of maximum stress (Figure 3-13).	Same as FHWA.
	Multi-Tier Wall — *Layer is Located in Lower Tier*	Depth (Z_P) measured from top of the wall face of the upper tier at the ground surface to the layer of reinforcement (Figure 3-14).		No additional requirements specified for calculating the pullout resistance for multi-tier walls.	Same as FHWA.
	Multi-Tier Wall — *Layer is Located in Upper Tier*				

4 Analyses and Evaluations of Selected Single-Tier MSE Walls

4.1 Introduction

The Texas Department of Transportation (TxDOT) provided design documents for two single-tier mechanically stabilized earth (MSE) walls designed by private engineering firms. The documents were reviewed, and additional design calculations were performed using FHWA and AASHTO guidelines. The results of these calculations were compared with the calculations performed in the original designs. Differences between the analyses performed in this study and the original designs were identified and resolved. This investigation helped to identify several conditions and assumptions that contribute to differences among designs by various individuals and organizations.

4.2 Common Features and Characteristics

The single-tier walls reviewed for this study were designed using the American Association of State Highway and Transportation Officials (AASHTO) 1996 guidelines, which is an older version of the current AASHTO 1998 guidelines discussed in Chapter 2. The AASHTO 1996 guidelines present a "working" or "allowable" stress design approach, which specifies factors of safety for external, internal, and global stability. Unlike the current AASHTO 1998 guidelines, load and resistance factors were not used in the AASHTO 1996 guidelines. Several differences exist between the AASHTO 1996 guidelines and the current versions of Federal Highway Administration (FHWA) and AASHTO guidelines. These differences are discussed for each wall in the following sections.

All soils were assumed to be cohesionless ($c = 0$) with a unit weight of 125 pcf. The reinforced soil was assumed to have an angle of internal friction (φ) of 34°, and the retained and foundation soils were both assumed to have angles of internal friction of 30° (Table 4-1).

Table 4-1: Design properties of the soil used on the US 183 and Brown County walls.

Soil Mass	Total Unit Weight, γ (pcf)	Angle of Internal Friction, φ (deg)	Cohesion, c (psf)
Reinforced soil	125	34	0
Foundation soil	125	30	0
Retained soil	125	30	0

Both walls were analyzed by hand and by using the MSEW 2.0 software for external and internal stability in accordance with the current FHWA 2001 guidelines. Global stability analyses for this study were performed using the MSEW 2.0 and UTEXAS4 software. Analyses with the MSEW 2.0 software were performed using the Comprehensive Bishop procedure to compute the factor of safety. This option defines the factor of safety with respect to the shear strength of the soil only. For the global stability analyses using the MSEW 2.0 and UTEXAS4 software programs, the long-term design strength (T_{al}) was used for the reinforcement forces, which were assumed to act horizontally. The analyses of each wall are presented in the following sections.

4.3 US 183 Wall

The first wall is located on US 183 in Travis County, Texas, and was designed by the Reinforced Earth Company (RECO). The wall is a MSE structure that supports a highway embankment. The tallest section of the wall was selected for analyses.

4.3.1 Wall Geometry and Material Design Properties

The overall length of the wall is approximately 257 ft. The tallest section is approximately 20.25 ft. high, measured from the bottom of the reinforced soil zone to the top of the wall. The wall is embedded into the foundation soil 1.0 ft. The wall is designed with precast concrete panels as facing and steel ribbed strips as reinforcement. A uniform vertical surcharge load of 250 psf is applied to the top of the wall (Figure 4-1) to represent traffic loads.

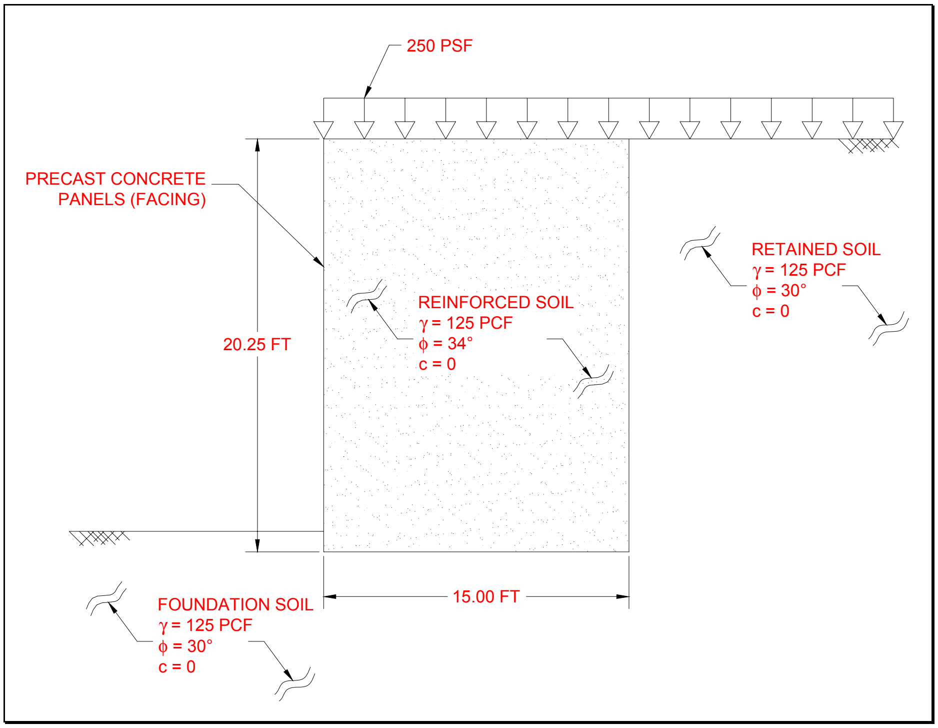

Figure 4-1: Cross-section with typical soil design properties of MSE wall on US 183.

The ribbed steel reinforcement strips used for the wall are 15 ft. long. Each strip has a width of 1.968 in. (50 mm) and a nominal thickness of 0.157 in. (4 mm). The strips are made of galvanized steel with a yield stress of 65 ksi (Table 4-2). The vertical and horizontal spacing of strips varies according to the wall section and elevation. The wall section analyzed for this study was designed to have a horizontal spacing (S_H) of 2.46 ft., which resulted in four strips per wall panel width (9.84 ft.). A cross-section of the MSE wall illustrating the vertical spacing of the reinforcement is shown in Figure 4-2.

4.3.2 External Stability

Analyses were conducted to verify the stability of the wall against sliding, overturning (eccentricity), and bearing capacity. Each calculation was performed both by hand and by using the MSEW 2.0 software. Companion hand calculations are presented in Appendix A.

Table 4-2: Design properties of the ribbed steel strips used on US 183 MSE wall section.

Length, L (ft)	Width, b		Nominal Thickness, E_n		Yield Stress (ksi)	Horizontal Spacing, S_H (ft)	Vertical Spacing, S_V (ft)
	in	mm	in	mm			
15.0	1.968	50.0	0.157	4.0	65	2.46	Varies (cf. Figure 2)

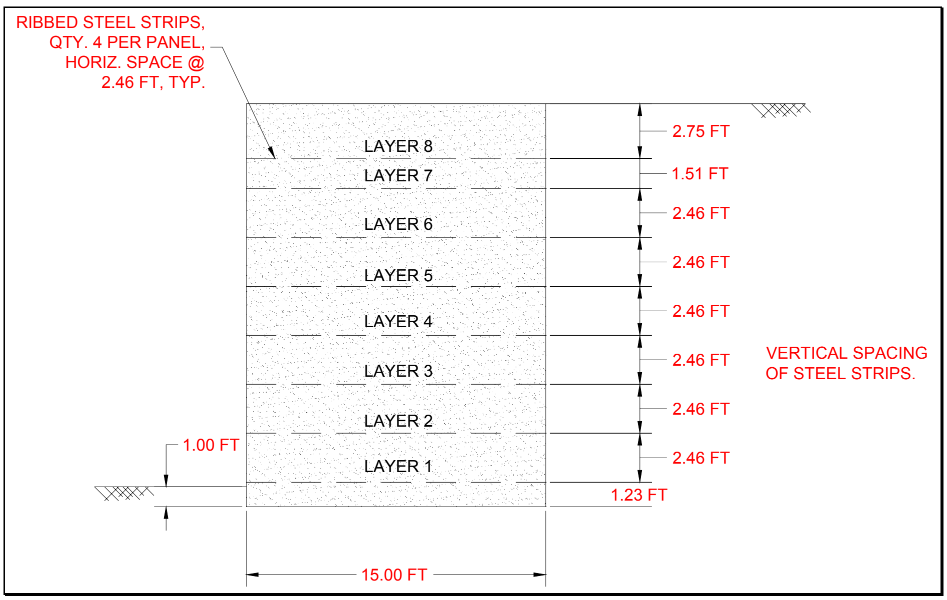

Figure 4-2: Spacing of ribbed steel strips in MSE wall on US 183.

4.3.2.1 Sliding Stability

Hand calculations and the computations using the MSEW 2.0 software revealed a factor of safety against sliding (FS_{SL}) of 2.14. The original designer, RECO, also calculated a factor of safety against sliding of 2.14. This value exceeds the FHWA guideline's minimum requirement of 1.5.

4.3.2.2 Eccentricity Calculation

The eccentricity (*e*) of the vertical force at the base of the wall was computed both by hand and by using the MSEW 2.0 software. The eccentricity was 1.79 ft. The original designer also calculated an eccentricity of 1.79 ft. These values are all within the FHWA 2001 maximum allowed eccentricity of L/6 (15.0 f.t / 6 = 2.5 ft.).

4.3.2.3 Bearing Capacity Failure

Safety against bearing capacity failure was evaluated by comparing the vertical stress (σ_v) at the base of the wall to the allowable bearing pressure (q_a) of the foundation soil. Calculations both by hand and by using the MSEW 2.0 software gave a vertical stress (σ_v) of 3,654 psf and an allowable bearing pressure (q_a) of 6,393 psf following FHWA procedures. Thus, the wall is considered stable in terms of the FHWA requirements for bearing capacity.

The wall designer reported a vertical stress of 3.65 ksf (3,650 psf) and a factor of safety against bearing capacity failure (FS_{BC}) of 4.37. For comparison with these calculations, a factor of safety against bearing capacity failure (FS_{BC}) was computed from the previous hand calculations using the ultimate bearing capacity (15,983 psf) of the foundation soil and the vertical stress reported above. For a vertical stress of 3,654 psf, this produces a factor of safety for bearing capacity of 4.37. Thus, the calculated vertical stress and factor of safety are the same by hand, using the MSEW 2.0 software, and by the original designer. Also, the computed factor of safety (4.37) satisfies the FHWA requirement for a factor of safety of at least 2.5.

4.3.3 Internal Stability

Internal stability against rupture and pullout was evaluated for each layer of reinforcement. Representative calculations are provided in Appendix A for a layer of reinforcement (Layer 6) at an elevation of 13.53 ft. from the bottom of the reinforced soil zone. Calculations for all layers of reinforcement were performed using the methods shown in Appendix A.

4.3.3.1 Factor of Safety Against Rupture of the Reinforcement

The factors of safety against rupture (FS_R) calculated by hand, by using the MSEW 2.0 software, and by the original designer are summarized in Table 4-3. All values meet FHWA minimum requirements with respect to rupture of the reinforcement. However, there are some differences between the factors of safety calculated by different methods or individuals.

Table 4-3: Factors of safety against rupture of the reinforcement.

Layer	Depth (ft)	Factor of Safety Against Rupture (FS_R)			Satisfy FHWA Requirements? ($FS_R \geq 1.82$)
		Hand Calculation	MSE W 2.0	Original Designer	
8	2.75	5.54	7.03	8.56	Yes
7	4.26	7.60	7.38	6.64	Yes
6	6.72	4.58	4.57	4.93	Yes
5	9.18	3.72	3.73	3.94	Yes
4	11.64	3.19	3.22	3.30	Yes
3	14.10	2.83	2.84	2.85	Yes
2	16.56	2.57	2.56	2.50	Yes
1	19.02	2.39	2.37	2.22	Yes

The differences between the factors of safety calculated by hand and by using the MSEW 2.0 software are caused by different assumptions being made to calculate the maximum tension (T_{MAX}) in the reinforcement. For the hand calculation, the maximum tension was calculated at the depth of the reinforcement. In contrast, the MSEW 2.0 software calculated the maximum tension from a trapezoidal distribution of the horizontal stress acting on a plane perpendicular to the reinforcement (Figure 3-9). The differences between the calculations by hand and by using the MSEW 2.0 software are largest near the top of the wall and when the vertical spacing of the reinforcement varies. The differences are small for the lower layers of reinforcement and when the spacing between reinforcement layers is constant.

The differences between the factors of safety calculated by hand and by the original designer result from differences between the AASHTO 1996 and FHWA 2001 guidelines. The AASHTO 1996 guidelines calculate the vertical stress by multiplying the ratio, $\frac{L}{L-2(e)}$, whereas the calculations using current FHWA guidelines do not use this ratio. The differences are also caused by different assumptions for calculating the lateral earth pressure coefficient (K_r), as discussed Chapter 3 (cf. Section 3.15).

4.3.3.2 Factor of Safety Against Pullout of the Reinforcement

The factors of safety against pullout (FS_{PO}) were calculated by hand, by using the MSEW 2.0 software, and by the original designer. The values are summarized in Table 4-4. The factors of safety calculated by hand and by using the MSEW 2.0 software are less than the FHWA minimum requirement of 1.5 for six of the eight layers. However, the original designer's calculations show a factor of safety of at least 1.5 for all layers.

Table 4-4: Factors of safety against pullout of the reinforcement.

Layer	Depth (ft)	Factor of Safety Against Pullout (FS_{PO})			Satisfy FHWA Requirements? ($FS_{PO} \geq 1.5$)
		Hand Calculation	MSEW 2.0	Original Designer	
8	2.75	0.77	0.98	2.07	No
7	4.26	1.55	1.51	2.00	Yes
6	6.72	1.33	1.33	1.86	No
5	9.18	1.32	1.32	1.70	No
4	11.64	1.39	1.41	1.69	No
3	14.10	1.49	1.49	1.71	No
2	16.56	1.51	1.51	1.65	Yes
1	19.02	1.46	1.45	1.51	No

The differences between the factor of safety calculated by hand and by using the MSEW 2.0 software are caused by the differences in the methods used to calculate the maximum tension (T_{MAX}) in the reinforcement, as described earlier (cf. Section 3.15). Differences in the factors of safety calculated by hand in this study and by the original designer are caused by several differences between the AASHTO 1996 and FHWA 2001 guidelines, as discussed in Chapter 3 (cf. Section 3.15). Also, the original designer neglected the contribution of the uniform vertical surcharge load (q) when calculating the horizontal stress (σ_H) acting at the layer of reinforcement. For the hand calculations in this study, the uniform surcharge load (q) was included to calculate the horizontal stress. Differences in the uniform vertical surcharge significantly affect the computed factor of safety in the layers of reinforcement near the top of the wall but become less significant near the bottom of the wall where the surcharge is a smaller fraction of the total vertical stress.

4.3.4 Global Stability

Global stability analyses were performed using the MSEW 2.0 and UTEXAS4 software. The designer of the MSE wall did not report results of global stability analyses. The critical slip surface found by the MSEW 2.0 software exited at the toe of the wall (Figure 4-3). Consequently, the MSEW 2.0 software neglected the effects of wall embedment and the uniform (traffic) surcharge load.[2] The minimum factor of safety found by the MSEW 2.0 software was 1.47. The factor of safety computed using the UTEXAS4 software excluded the effects of wall embedment but included the uniform surcharge load, which is the most critical condition. The minimum factor of safety computed using UTEXAS4 is 1.26 (Figure 4-4).

[2] As discussed in Chapter 3, these assumptions are automatically made in the MSEW 2.0 software for global stability analyses.

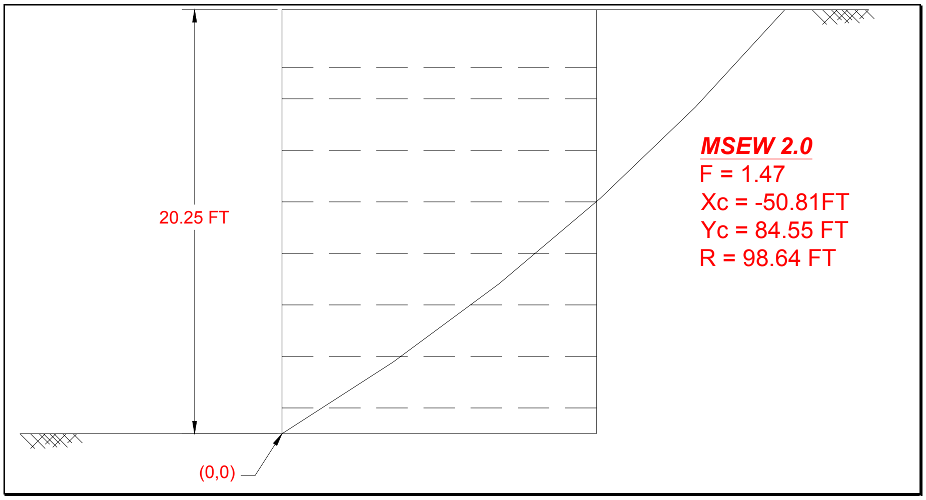

Figure 4-3: Critical slip surface reported by MSEW 2.0 global stability analysis assuming no uniform (traffic) surcharge load and no effects of wall embedment.

The factors of safety calculated using the MSEW 2.0 and UTEXAS4 software (1.47 and 1.26, respectively) are different, owing to how the surcharge was treated in the analyses. In the MSEW 2.0 software, the uniform surcharge was not included in computing the factor of safety. The computations with the UTEXAS4 software were performed with the intent of modeling the most critical condition and, thus, the uniform surcharge was included. An additional analysis was performed using the UTEXAS4 software to compute the factor of safety with the surcharge neglected. The computed factor of safety from this second analysis was 1.47 (Figure 4-5), which is identical to the value computed with the MSEW 2.0 software.

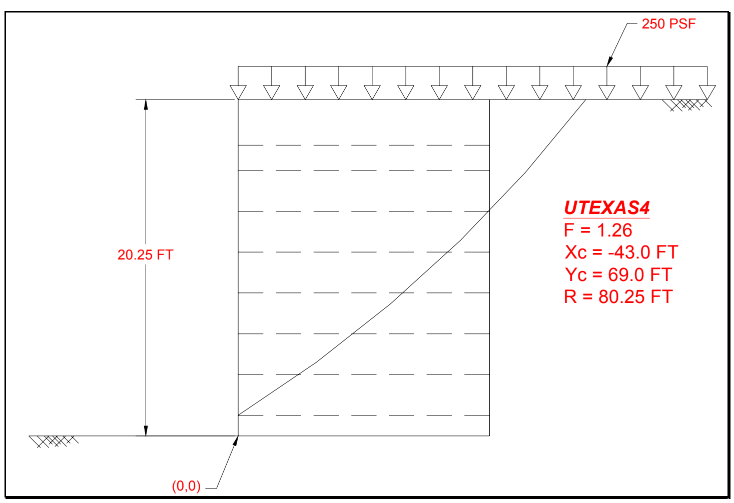

Figure 4-4: Critical slip surface reported by UTEXAS4 global stability analysis assuming a uniform (traffic) surcharge load and no effects of wall embedment.

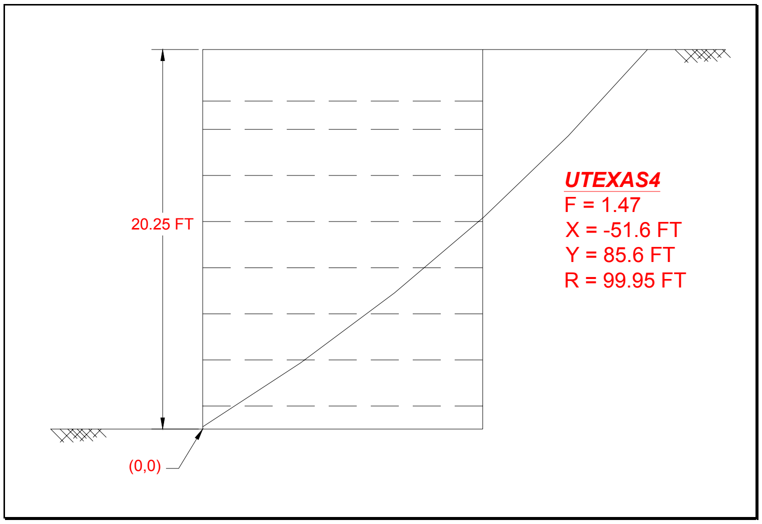

Figure 4-5: Factor of safety calculated with UTEXAS4 for the wall on US 183 neglecting live surcharge load and embedment of the wall.

4.4 Brown County (FM 2524)

The second wall investigated is located in Brown County, Texas. The wall was originally designed by Unintech Consulting Engineers, Inc. The wall is an MSE wall that supports an embankment for highway FM 2524. Calculations by hand and by using the MSEW 2.0 software for this study were performed in accordance with FHWA 2001 design criteria. The original designer performed analyses using the AASHTO 1996 guidelines. The wall section chosen for study is the tallest section of the structure.

4.4.1 Wall Geometry and Material Design Properties

The overall length of the wall is approximately 348 ft. The tallest section is 23.3 ft. high, measured from the bottom of the reinforced soil zone to the top of the wall. The wall is embedded in the foundation soil 2.0 ft. The wall is designed with a precast concrete panel facing and steel grid reinforcement. Traffic loads are represented by a uniform surcharge load of 250 psf on the top of the wall (Figure 4-5). Soil properties used for the wall design are shown in Figure 4-6 and were summarized previously in Table 4-1.

The steel grid reinforcement is composed of longitudinal and transverse bars. The longitudinal bars attach to the concrete panels forming the face of the wall and extend into the reinforced soil mass perpendicular to the face. Transverse bars are welded to the longitudinal bars at a constant spacing in each layer and are oriented parallel to the wall face. The longitudinal and transverse bars both vary in number and size for each layer of reinforcement (Table 4-5).

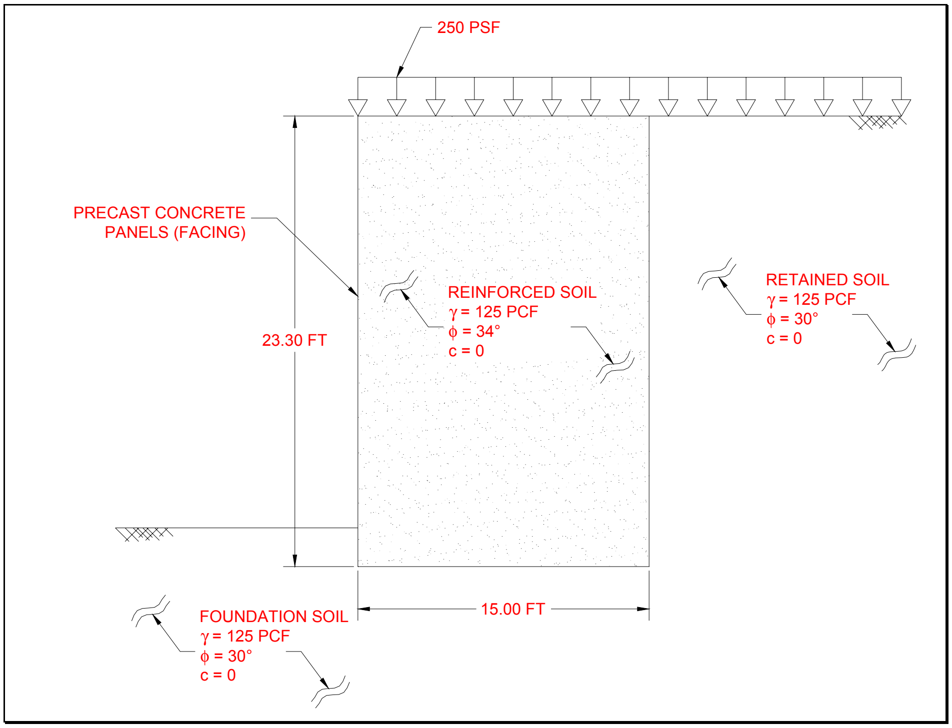

Figure 4-6: Cross-section with typical soil properties of the Brown County wall on FM 2524.

Table 4-5: Specifications for steel grid reinforcement composition at each layer for the Brown County wall.

Layer	Depth (ft)	Longitudinal Bars			Transverse Bars		
		Size (1/100 in^2)	Number of Bars	Horizontal Spacing (in)	Size (1/100 in^2)	Number of Bars	Horizontal Spacing (in)
8	3.11	7	7	6.0	9.5	17	5.7
7	6.17	7	7	6.0	7	12	8.0
6	9.17	7	9	6.0	7	12	8.0
5	12.24	7	10	6.0	7	12	8.4
4	15.24	9.5	9	6.0	9.5	8	15.2
3	18.30	9.5	10	6.0	9.5	9	16.0
2	21.30	9.5	9	6.0	9.5	10	16.6
1	22.80	7	8	6.0	7	21	8.4

The length of the steel grid reinforcement is 15.0 ft. for all layers. The vertical spacing of the steel grid varies with each layer and is shown in Figure 4-7. The spacing of longitudinal bars is 6 in. for all layers. The horizontal spacing of transverse bars varies from layer to layer, as shown in Table 4-5.

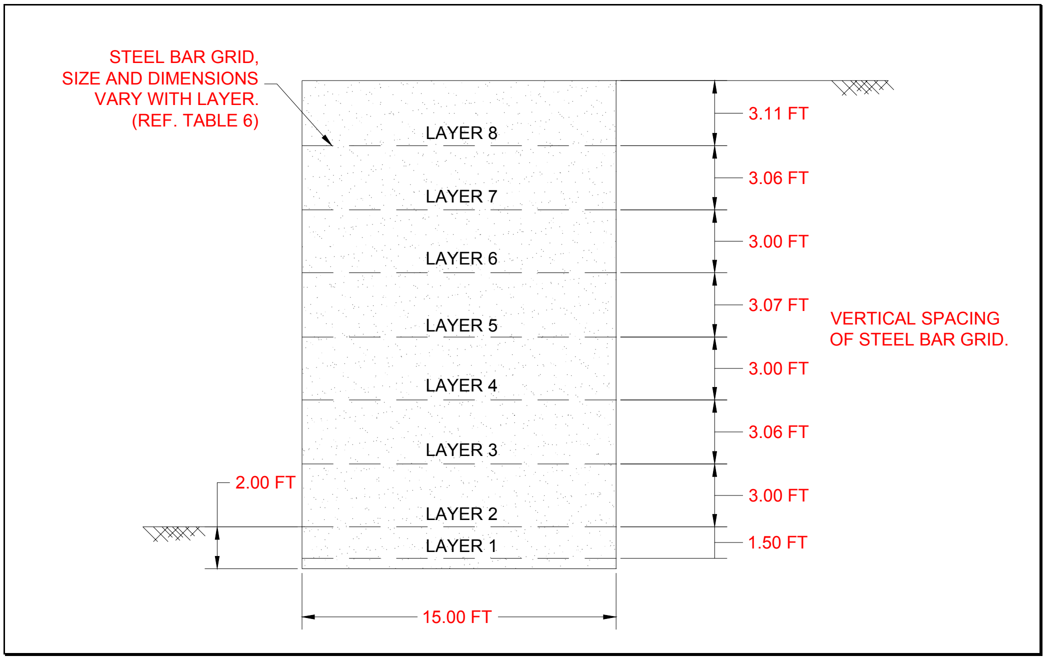

Figure 4-7: Vertical spacing of steel grids in the Brown County wall on FM 2524.

4.4.2 External Stability

Calculations were performed both by hand and by using the MSEW 2.0 software to verify the stability of the wall against sliding, overturning (eccentricity), and bearing capacity. The hand calculations for all the external stability calculations are presented in Appendix B.

4.4.2.1 Sliding Stability

Hand calculations and calculation performed using the MSEW 2.0 software were performed in accordance with the FHWA 2001. Both sets of calculations revealed factors of safety against sliding (FS_{SL}) of 1.90. The original designer calculated a factor of safety against sliding of 1.97, which is in close agreement with the values calculated in this study. All values exceed the FHWA minimum requirement of 1.5.

4.4.2.2 Eccentricity Calculation

The eccentricity (*e*) of the vertical force on the base of the wall was calculated both by hand and by using the MSEW 2.0 software. These calculations and those by the original designer all gave an eccentricity of 2.33 ft. This is within the FHWA maximum allowed eccentricity of L/6 (2.5 ft.).

4.4.2.3 Bearing Capacity Failure

Bearing capacity was evaluated by comparing the vertical stress (σ_v) at the base of the wall with the allowable bearing pressure (q_a) of the foundation soil. Calculations by hand and by using the MSEW 2.0 software both yielded a vertical stress of 4,586 psf and an allowable bearing pressure of 5,792 psf. Thus, the wall is considered stable because the vertical stress (4,586 psf) is less than the allowable bearing pressure (5,752 psf).

The wall designer reported a vertical stress of 4,586 psf and a factor of safety against bearing capacity failure (FS_{BC}) of 4.16. For comparison, a factor of safety was computed from the hand calculations using the ultimate bearing capacity (q_{ult}) of 14,480 psf and the vertical stress of 4,586 psf. This produced a factor of safety for bearing capacity of 3.16. The factors of safety computed by hand in this study (3.16) and by the original designer (4.16) satisfy the FHWA requirement of a factor of safety for bearing capacity failure of 2.5; however, the factors of safety are different.

The difference between factors of safety calculated in this study and by the original designer appears to be caused by different values for the ultimate bearing capacity, because the vertical stresses for the calculations are the same. However, the ultimate bearing capacity was not reported by the designer and, thus, the source of the differences in factor of safety cannot be determined.

4.4.3 Internal Stability

The factors of safety against rupture and pullout were calculated for each layer of reinforcement. Representative calculations for a reinforcing layer (Layer 5) at an elevation of 11.06 ft. from the bottom of the reinforced soil zone are provided in Appendix B.

The Brown County wall was designed with eight different configurations of steel grid. This created a problem when performing internal stability analyses using the MSEW 2.0 software because only five different configurations of reinforcement can be used. In order to calculate factors of safety against rupture and pullout that were appropriate to the original design for each layer of reinforcement, two separate analyses were performed. The first analysis was performed using the steel grid configurations (i.e., bar sizes), longitudinal and transverse bar spacing, and yield stress of steel for Layers 1 through 5 at the elevations specified. In the first analysis, the steel grids in Layers 6 through 8 were input with the same configuration as those in Layer 5. For the second analysis, the steel grid configurations for Layers 6 through 8 were input at the specified elevations, and the grid in Layers 1 through 5 were given the same configuration as those in Layer 6.

4.4.3.1 Factor of Safety against Rupture of the Reinforcement

Factors of safety against rupture (FS_R) of the reinforcement are summarized in Table 4-6. The factors of safety calculated by hand and by using the MSEW 2.0 software are less than the minimum FHWA requirements in several layers of reinforcement. However, the factors of safety reported by the original designer meet or exceed FHWA minimum requirements for all layers.

Table 4-6: Summary of factors of safety against rupture of the reinforcement for the Brown County wall.

Layer	Depth (ft)	Factor of Safety Against Rupture (FS_R)			Satisfy FHWA Requirements? ($FS_R \geq 2.08$)
		Hand Calculation	MSEW 2.0	Original Designer	
8	3.11	1.37	1.64	2.11	No
7	6.17	1.44	1.45	2.09	No
6	9.17	1.49	1.50	2.04	No
5	12.24	1.45	1.46	1.82	No
4	15.24	1.77	1.79	2.02	No
3	18.30	1.94	1.95	1.91	No
2	21.30	2.20	2.24	1.95	Yes
1	22.80	2.28	2.29	1.91	Yes

The factors of safety calculated by hand and by using the MSEW 2.0 software are different for several layers of reinforcement. The differences are caused by different assumptions made for calculating the maximum tension (T_{MAX}) in the reinforcement, as discussed in Chapter 3 (cf. Section 3.15). The maximum tension (T_{MAX}) by hand is computed at the depth to the layer of reinforcement. The MSEW 2.0 software calculates the maximum tension in the reinforcement from a trapezoidal distribution of horizontal stress acting on a plane perpendicular to the reinforcement.

Differences in the factor of safety also exist between the original designer's calculations and those performed in this study. These are caused by differences between the AASHTO 1996 and FHWA 2001 guidelines. As discussed in Chapter 3, these guidelines provide different methods to calculate the vertical stress (σ_v) on the reinforcement *and* the lateral earth pressure coefficient (K_r).

4.4.3.2 Factor of Safety Against Pullout of the Reinforcement

Factors of safety against pullout (FS_{PO}) are summarized in Table 4-7. The factors of safety calculated by hand and by using the MSEW 2.0 software are less than the FHWA recommended minimum value of 1.5. However, the factors of safety reported by the original designer meet or exceed FHWA minimum requirements for all layers.

The factors of safety calculated by hand and by using the MSEW 2.0 software are slightly different for several layers of reinforcement. These differences are again caused by the different methods for calculating the maximum tension (T_{MAX}) in the reinforcement, as described in Chapter 3 (cf. Section 3.15).

The factors of safety calculated by hand and by the original designer are also different. These differences are caused by the differences between the AASHTO 1996 and FHWA 2001 guidelines. These guidelines provide different procedures to calculate the vertical stress on the reinforcement *and* to calculate the lateral earth pressure coefficient (K_r) as described in Chapter 3 (cf. Section 3.15).

Table 4-7: Summary of factors of safety against pullout of the reinforcement on the Brown County wall.

Layer	Depth (ft)	Factor of Safety Against Pullout (FS_{PO})			Satisfy FHWA Requirements? ($FS_{PO} \geq 1.5$)
		Hand Calculation	**MSEW 2.0**	**Original Designer**	
8	3.11	1.15	1.14	1.79	No
7	6.17	1.29	1.31	1.90	No
6	9.17	1.88	1.89	2.59	Yes
5	12.24	2.23	2.22	2.82	Yes
4	15.24	1.65	1.66	1.88	Yes
3	18.30	2.15	2.16	2.15	Yes
2	21.30	2.91	2.94	2.58	Yes
1	22.80	8.05	8.07	6.78	Yes

4.4.4 Global Stability

Although no results of global stability analyses were available from the original designer, global stability analyses were performed for this study using the MSEW 2.0 and UTEXAS4 software. The critical slip surface was found to pass through the foundation soil, below the toe of the wall, and to exit at the ground surface beyond the wall face (Figure 4-8). Consequently, in the analyses with the MSEW 2.0 software, the uniform (traffic) surcharge load was neglected but the effect of the soil in front of the wall was included.[3] The minimum factor of safety found using the MSEW 2.0 software was 1.66. For the analyses with the UTEXAS4 software, the effect of the soil in front of the wall was also excluded, but the uniform surcharge was included. This was done to model the most critical condition for the wall. The minimum factor of safety calculated by UTEXAS4 was 1.42, and the critical slip surface is shown in Figure 4-9.

The factors of safety calculated by the MSEW 2.0 and UTEXAS4 software are 1.66 and 1.42, respectively, and are different due to how the surcharge was treated. An additional analysis was performed with the UTEXAS4 software with the surcharge load neglected. The computed factor of safety for this case was 1.67, which is essentially the same as the value (1.66) computed by the MSEW 2.0 software. The critical slip surface for the second analysis with UTEXAS4 is shown in Figure 4-10.

[3] The assumptions for excluding the uniform surcharge load and including the effect of wall embedment are integral to the MSEW 2.0 software and apparently cannot be changed by the user.

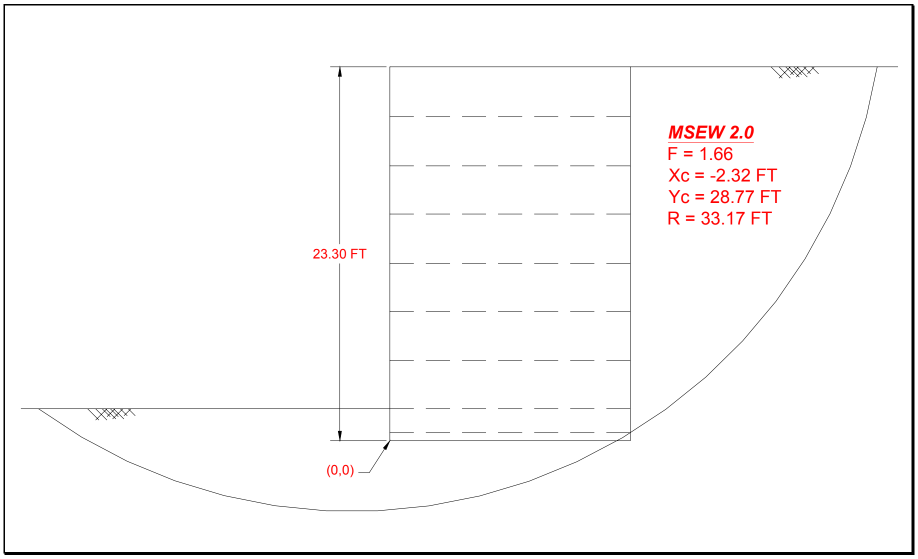

Figure 4-8: Critical slip surface reported by MSEW 2.0 global stability analysis for the Brown County wall assuming no uniform (traffic) surcharge load and no effects of wall embedment.

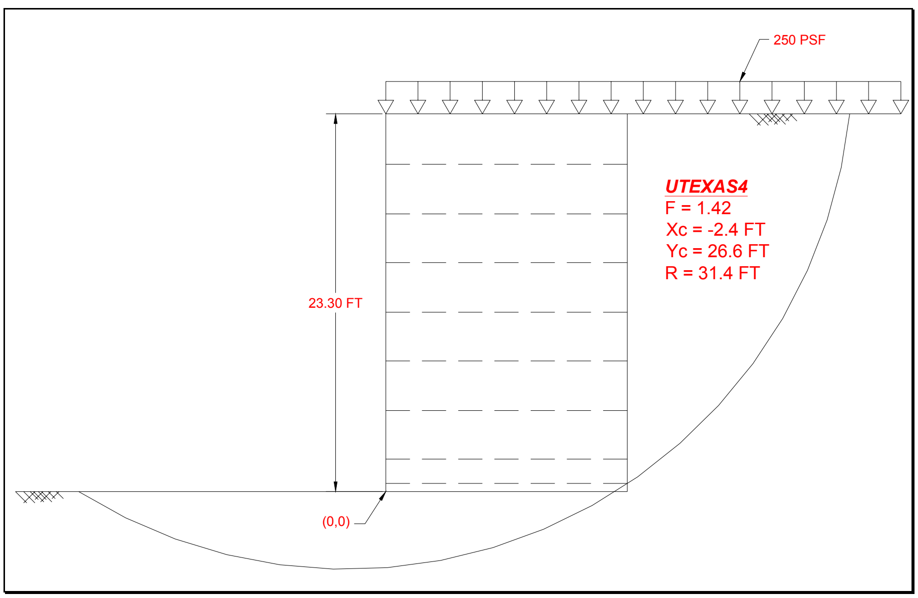

Figure 4-9: Critical slip surface reported by UTEXAS4 global stability analysis for the Brown County wall assuming a uniform (traffic) surcharge load and no effects of wall embedment.

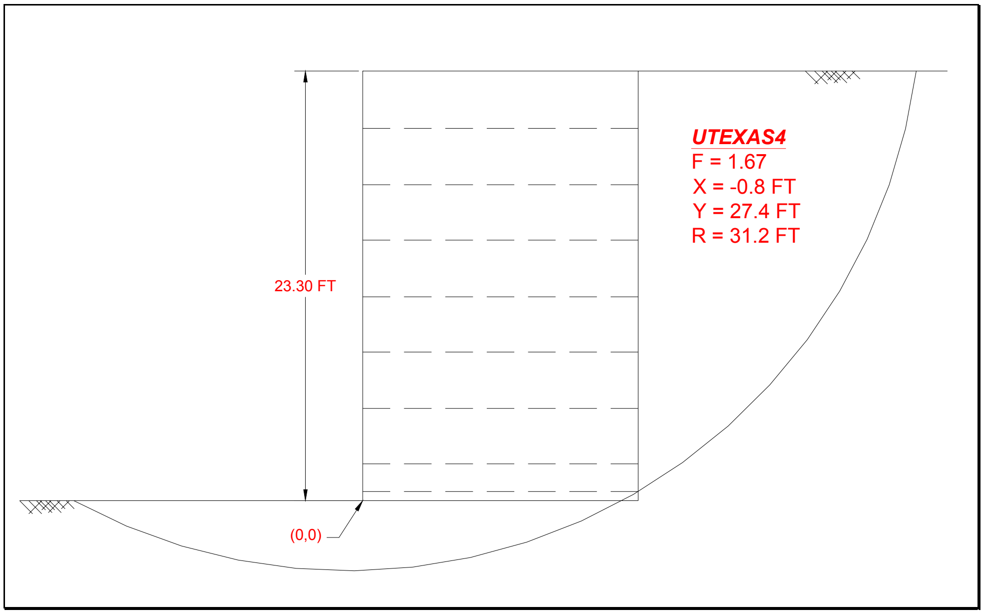

Figure 4-10: Factor of safety calculated with UTEXAS4 for the Brown County wall neglecting live surcharge load and embedment of the wall.

4.5 Discussion

The objective of this chapter was to explore and quantify differences in the methods used to analyze single-tier MSE walls. It was found that the computed factors of safety can be different depending on the design guidelines used and assumptions made either in the software program or by the original designer. Six assumptions were found in the course of this study to cause differences in the computed factors of safety. These are discussed in the following sections.

4.5.1 Assumptions that Effect Internal Stability

Three assumptions were found to cause differences in the computed factors of safety for rupture and pullout of the reinforcement. These assumptions pertain to the vertical stress (σ_v) acting on the reinforcement, the lateral earth pressure coefficient (K_r), and the maximum tension T_{MAX} in the reinforcement.

4.5.1.1 Vertical Stress (σ_v) on the Reinforcement

According to AASHTO 1996 guidelines, the vertical stress (σ_v) is calculated using an additional term of $\frac{L}{L-2(e)}$, which the FHWA 2001 does not include. The vertical stress on each layer of reinforcement for the Brown County wall calculated by hand and that reported by the original designer are summarized in Table 4-8.

Table 4-8: The vertical stress (σ_v) calculated using the FHWA (2001) and AASHTO 1996 procedures for the Brown County wall.

Layer	**Depth (ft)**	**σ_v (psf)**		**Difference (%)**
		Hand Calculation FHWA 2001	**Original Designer AASHTO 1996**	
8	3.11	639	644	0.8
7	6.17	1021	1051	2.8
6	9.17	1396	1479	5.6
5	12.24	1780	1966	9.5
4	15.24	2155	2510	14.1
3	18.30	2538	3161	19.7
2	21.3	2913	3950	26.3
1	22.8	3100	4420	29.9

The vertical stresses shown in Table 4-11 differ by as much as 30 percent depending on whether they were calculated using the FHWA 2001 or AASHTO 1996 guidelines. Also, the differences are largest near the bottom of the wall where the eccentricity is largest and small near the top of the wall. A comparison of vertical stresses similar to the one shown in Table 4-8 could not be made for the US 183 wall, because neither the vertical stress or the eccentricity for each layer of reinforcement was reported by the original designer.

4.5.1.2 Lateral Earth Pressure Coefficient (K_r)

The AASHTO 1996 guidelines specify that the lateral earth pressure coefficient (K_r) varies from the at-rest condition (K_0) at the top of the wall to the active condition (K_a) at a depth of 20 ft. and below. FHWA 2001 guidelines specify that the lateral earth pressure coefficient (K_r) also varies and is expressed by a factor multiplied to the active earth pressure coefficient (K_a). The difference between the FHWA 2001 and AASHTO 1996 guidelines to calculate the lateral earth pressure coefficients (K_r) for three types of reinforcement is illustrated in Figure 4-11.

4.5.1.3 Horizontal Stress for Calculating Maximum Tension (T_{MAX}) in Reinforcement

The MSEW 2.0 software calculates the maximum tension (T_{MAX}) from a trapezoidal distribution of horizontal stress using the procedure discussed in Chapter 3 (cf. Section 3.14). This approach is an apparent departure from the FHWA and AASHTO guidelines, which recommend that T_{MAX} be calculated using the horizontal stress at the level of the reinforcement.

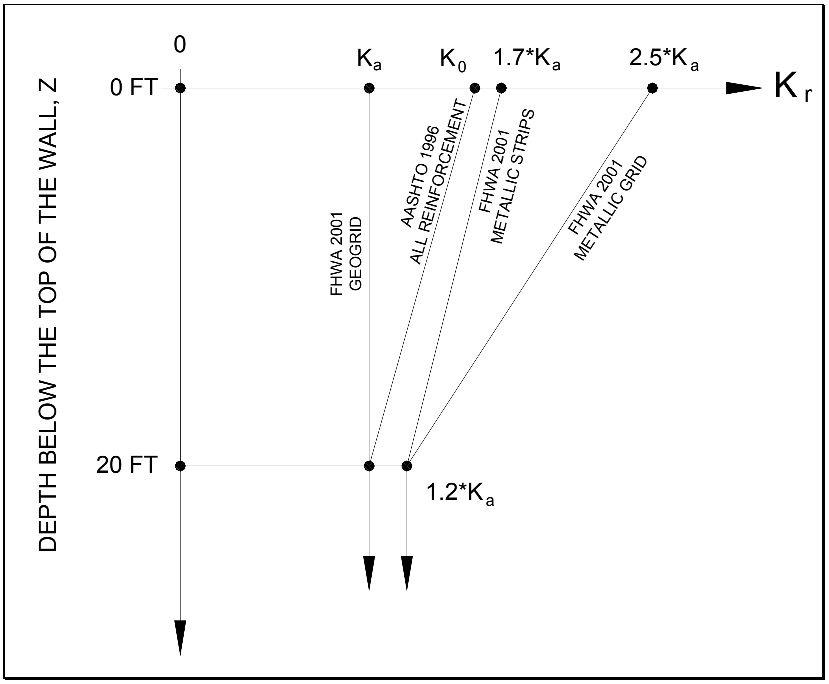

Figure 4-11: The variation of the lateral earth pressure coefficient (K_r) with depth as specified by the FHWA 2001 and AASHTO 1996 guidelines.

Table 4-9: The maximum reinforcement tension (T_{MAX}) calculated by hand and using the MSEW 2.0 software for each layer of reinforcement in the US 183 wall.

Layer	Depth (ft)	T_{MAX} (lb)		Difference (%)
		Hand Calculation	MSEW 2.0	
8	2.75	673	755	10.8
7	4.26	866	720	−20.4
6	6.72	1159	1162	0.2
5	9.18	1426	1425	−0.1
4	11.64	1665	1650	−0.9
3	14.10	1878	1869	−0.5
2	16.56	2064	2074	0.4
1	19.02	2224	2243	0.8

The maximum tension calculated by hand and by using the MSEW 2.0 software for each layer of reinforcement is summarized in Tables 4-10 and 4-11 for the US 183 and Brown County walls, respectively. The values show that the two methods for calculating the maximum tension can produce differences ranging from negligible to as large as 20 percent. The differences are greater near the top of the wall than near the bottom of the wall.

Table 4.10: The maximum reinforcement tension (T_{MAX}) calculated by hand and using the MSEW 2.0 software for each layer of reinforcement in the Brown County wall.

Layer	Depth (ft)	T_{MAX} (lb)		Difference (%)
		Hand Calculation	MSEW 2.0	
8	3.11	1923	1606	−19.7
7	6.17	1831	1807	−1.3
6	9.17	2270	2253	−0.7
5	12.24	2584	2578	−0.2
4	15.24	2757	2735	−0.8
3	18.30	2807	2785	−0.8
2	21.3	2223	2187	−1.6
1	22.8	1315	1308	−0.5

Table 4-11: The horizontal stress (σ_H) calculated reported by the original designer used to calculate the factors of safety against rupture and pullout for each layer of reinforcement on the US 183 wall.

Layer	Depth (ft)	σ_H (psf)		Difference (%) (A) versus (B)
		For Rupture (Surcharge Included) (A)	For Pullout (Surcharge Neglected) (B)	
8	2.75	250	150	−82.4
7	4.26	320	220	−60.1
6	6.72	440	340	−38.6
5	9.18	550	450	−28.8
4	11.64	650	560	−20.9
3	14.10	760	670	−14.0
2	16.56	860	790	−6.2
1	19.02	970	900	−0.4

4.5.1.4 Influence of Surcharge Loads

For the US 183 wall, the designer included the effect of the traffic surcharge in calculating the horizontal stress (σ_H) for rupture but neglected it in calculating the horizontal stress for pullout. In contrast, the FHWA 2001 guidelines specify including the surcharge when calculating the horizontal stresses (σ_H) used to evaluate safety against both rupture and pullout. The difference can be shown by comparing the horizontal stress calculated by the original designer for rupture, including the surcharge and for pullout neglecting the surcharge (Table 4-12). Table 4-12 shows that the horizontal stress calculated for pullout when neglecting the surcharge is as much as 82 percent lower than that calculated with the surcharge included.

4.5.2 Assumptions that Effect Factors of Safety for Global Stability

There are two assumptions used in the MSEW 2.0 software that were found to affect the factors of safety calculated for global stability. The first assumption is whether the surcharge is included or neglected in global stability analyses. The second is whether the effects of the soil in front of the wall due to embedment are included or neglected.

4.5.2.1 Effect of Including or Neglecting the Surcharge Load

Global stability analyses can be performed assuming the uniform surcharge load can be either included or neglected. Calculations using the MSEW 2.0 software automatically neglect the surcharge load. Calculations were performed with the UTEXAS4 software both with and without the surcharge. The results are summarized in Table 4-13. The factors of safety calculated using the MSEW 2.0 and the UTEXAS4 software with the surcharge neglected are in close agreement. Including the surcharge reduced the factor of safety by about 17 percent.

Table 4-12: Minimum factors of safety calculated with the surcharge load included and neglected using the MSEW 2.0 and UTEXAS4 software programs.

Wall	**Minimum Factor of Safety**			**Difference (%)**
	UTEXAS4		**MSEW 2.0 (Surcharge Neglected)**	
	Surcharge Included	Surcharge Neglected		
US 183	1.26	1.47	1.47	16.7
Brown County	1.42	1.67	1.66	17.6

4.5.2.2 Effect of Including or Neglecting the Soil in Front of Wall

Calculations using the MSEW 2.0 software included or excluded the effects of the soil in front of the wall depending on whether the slip surface passed below the toe of the wall or whether the slip surface passed through the toe or through the wall face. For a slip surface that passed below the toe, the soil in front of the wall was included. For slip surfaces that passed through the toe or through the wall face, the soil in front of the wall was neglected. Analyses with the UTEXAS4 software were performed both including and neglecting the soil in front of the wall. The results for the US 183 and Brown County walls are summarized in Table 4-14. For the US 183 and Brown County walls, the factors of safety neglecting the soil in front of the wall were 3.4 and 13.6 percent lower than the factors of safety calculated including the soil in front of the wall.

Table 4-13: Minimum factor of safety calculated neglecting surcharge load and either including or neglecting the soil in front of the wall.

Wall	Minimum Factor of Safety			Difference (%)
	UTEXAS4		MSEW 2.0	
	Soil in Front of Wall			
	Included	Neglected		
US 183	1.52	1.47	1.47 (1)	3.4
Brown County	1.67	1.47	1.66 (2)	13.6

1) Neglects soil in front of the wall.
2) Includes soil in front of wall.

4.6 Summary and Conclusions

The review of designs for two TxDOT MSE walls produced important insight into design procedures and guidelines. The external stability calculations produced factors of safety in close agreement whether performed by hand, by using the MSEW 2.0 software, or by the original designer of each wall. However, several differences were found to have an effect on the factor of safety for internal and global stability. These are caused by the differences summarized below.

Vertical Stress (σv) on the Reinforcement: Differences between the AASHTO 1996 and FHWA 2001 guidelines can produce differences as much as 30 percent in the calculated vertical stress on the reinforcement (Table 4-8).

Lateral Earth Pressure Coefficient (Kr): Differences between the AASHTO 1996 and FHWA 2001 guidelines can produce differences of as much as 60 percent in the calculated lateral earth pressure coefficient (Table 4-9).

Maximum Tension in the Reinforcement (TMAX): The MSEW 2.0 software uses a trapezoidal distribution of horizontal stress to calculate the maximum tension in the reinforcement. Whether the maximum tension in the reinforcement is calculated as an average value assuming a trapezoidal distribution of stress or from the stress at the layer of reinforcement can produce differences of as much as 20 percent in the calculated tension (Tables 4-10 and 4-11).

Treatment of Surcharge—Internal Stability: The original designer of the US 183 wall neglected the surcharge load to calculate the horizontal stress used to evaluate the safety with respect to pullout of the reinforcement. Whether the surcharge load is included or neglected can produce differences as much as 82 percent in the calculated horizontal stress (Table 4-12).

Treatment of Surcharge—Global Stability: Uniform surcharge loads are neglected in the MSEW 2.0 software for global stability analyses. Whether the surcharge is included or neglected can produce differences of as much as 18 percent in the calculated factor of safety (Table 4-13).

Soil in Front of Embedded Walls: The presence of soil in front of the wall is either included or neglected in the MSEW 2.0 software, depending on the location of the slip surface. Whether soil is included or not can produce as much as a 13.6 percent difference in the calculated factor of safety (Table 4-14).

It is important to understand all of the assumptions made in the design of single-tier MSE walls. Numerous assumptions may be embedded into a single design, and they may not be well documented. Consequently, considerable differences may be found in the computed factors of safety. Knowing the assumptions that affect the design of single-tier walls will also provide insight into the assumptions used in the design of multi-tier walls, which will be discussed in the following chapter.

5 Analyses and Evaluations of Selected Multi-Tier MSE Walls

5.1 Introduction

The Texas Department of Transportation (TxDOT) provided design documents for five multi-tier mechanically stabilized earth (MSE) walls. The name, number of tiers, and general location for each wall are summarized in Table 5-1. Additional analyses were performed for each of these walls following the Federal Highway Administration (FHWA) 2001 guidelines. The results of these analyses were then compared with the original design calculations. Differences between the analyses performed in this study and the original designs were identified and resolved.

Table 5-1: Summary of the multi-tier wall designs provided by TxDOT.

Wall	Number of Tiers	Location
Socorro Bridge	2	El Paso County
Redd Road Overpass (IH-10)	3	El Paso County
Town of Anthony (BR 93)	4	El Paso County
US 67 Bypass	4	Johnson County
US 290	4	Travis County

5.2 FHWA Criteria

FHWA guidelines provide the criteria pertaining to analyses of multi-tier walls. According to these criteria, a wall may be analyzed treating each tier as an isolated single-tier wall if one of the following two conditions apply:

$$D \leq \frac{H_L + H_U}{20} \tag{5-1}$$

or

$$D > H_L \cdot \tan(90° - \varphi_r) \tag{5-2}$$

where

D = the horizontal offset between the lower and the upper tiers,
H_L = height of the lower tier,
H_U = height of the upper tier, and
φ_r = angle of internal friction of the reinforced soil.

If the offset of the wall is very small, as indicated by Equation 5-1, it is assumed that the two walls act and can be analyzed as a single wall. When the offset is very large (Equation 5-2), each tier acts and can be analyzed as an independent single-tier wall.

FHWA guidelines do not address multi-tier walls in general, but they do address two-tier walls. According to FHWA guidelines, both tiers of two-tier walls must be analyzed together as a composite wall when the offset distance, D, satisfies the following condition:

$$\frac{H_L + H_U}{20} < D \leq H_L \cdot \tan(90° - \varphi_r). \quad (5\text{-}3)$$

The geometry for the five multi-tier walls analyzed in this study is summarized in Table 5-2. According to the values presented in Table 5-2 and the FHWA criteria, three of the walls—Redd Road Overpass, Town of Anthony, and the US 67 Bypass wall—can be analyzed by treating each tier as an independent wall. The values also indicate that the Socorro Bridge and US 290 wall should be analyzed as multi-tier walls.

Table 5-2: The tier heights, offsets, and FHWA guidelines criteria for single- or two-tier analyses for each multi-tier wall.

Wall	**Tier**	**Height (ft)**	**Offset of Upper Tier (ft)**	**(H_L+H_U)/20 (ft)**	**(H_L)*tan(90-φ_r) (ft)**	**Analyzed as Single- or Two-Tier Wall Using FHWA Guidelines?**
Redd Road Overpass – IH-10	1	8.7	18.8	0.7	12.9	Single-tier
	2	5.8	18.8	0.6	8.7	Single-tier
	3	5.2	–	–	–	–
Town of Anthony	1	5.0	15.9	0.6	7.4	Single-tier
	2	6.0	20.0	0.6	8.9	Single-tier
	3	6.0	28.0	0.6	8.9	Single-tier
	4	6.0	21.5	0.7	8.9	Single-tier
	5	8.7	–	–	–	–
US 67 Bypass	1	3.0	10.0	0.5	4.4	Single-tier
	2	6.4	10.0	0.6	9.4	Single-tier
	3	5.6	10.0	0.6	8.3	Single-tier
	4	5.7	–	–	–	–
Socorro Bridge	1	12.3	9.84	1.0	18.3	Two-tier
	2	7.0	–	–	–	–
US 290	1	7.3	8.0	0.6	10.8	Two-tier
	2	5.5	8.0	0.8	8.2	Two-tier
	3	10.2	8.0	0.8	15.1	Two-tier
	4	5.5	–	–	–	–

5.2.1 Analyses as Single-Tier Walls

The Redd Road Overpass and Town of Anthony walls were analyzed as individual single-tier walls using the procedures in the FHWA guidelines for single-tier walls. The minimum factors of safety and critical slip surfaces were found for the Redd Road Overpass and the Town of Anthony walls using the UTEXAS4 software and are shown in Figures 5-1 and 5-2, respectively. The critical slip surface of each wall encompasses only one tier and, thus, confirms the FHWA criteria. Because these two walls (Redd Road Overpass and Town of Anthony) behave as independent single-tier walls and may be analyzed accordingly, no further consideration of them is given in this chapter.

The global stability analysis for the US 67 Bypass wall suggests that the wall acts more as a multi-tier wall than as a series of isolated single-tier walls in overall stability. The minimum factor of safety and critical slip surface found using the UTEXAS4 software for the US 67 Bypass wall are shown in Figure 5-3. Clearly, the slip surface involves all the tiers of the wall and, thus, the wall cannot be considered as a series of isolated, independent walls.

5.2.2 Analyses as Multi-Tier Walls

Three walls were analyzed as multi-tiered walls: Socorro Bridge, US 290, and US 67 Bypass. The Socorro Bridge and the US 290 walls were analyzed as multi-tier walls because both walls satisfy the FHWA criteria for multi-tier walls. The US 67 Bypass wall was also analyzed as a multi-tier wall on the basis of the analyses discussed in Section 5.2.1. The Socorro Bridge wall is the only wall that has two tiers; however, all three walls were analyzed using the FHWA criteria for two-tier walls.

Although the FHWA guidelines address only two-tier walls, the criteria for two-tier walls were applied to all the multi-tier walls examined in this study. For the US 290 wall, analyses were performed for a series of "equivalent" two-tier walls, where any tiers above the second tier were represented as a surcharge load on the top of the second tier. The US 290 wall was analyzed first by considering Tiers 1 and 2 (bottom two tiers) as an "equivalent" two-tier wall with Tiers 3 and 4 (top two tiers) represented as a surcharge applied to the top of Tier 2. The process is repeated for the remaining two-tier combinations. That is, Tiers 2 and 3 were analyzed with Tier 4 as a surcharge. Finally, Tiers 3 and 4 were analyzed.

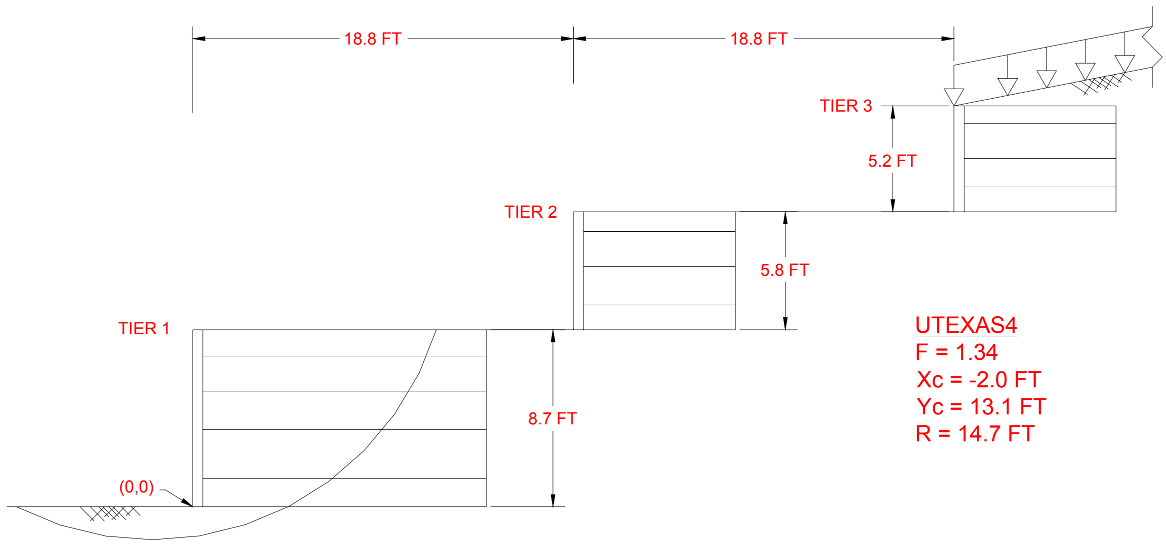

Figure 5-1: The critical slip surface and minimum factor of safety found using the UTEXAS4 software for the Redd Road Overpass wall.

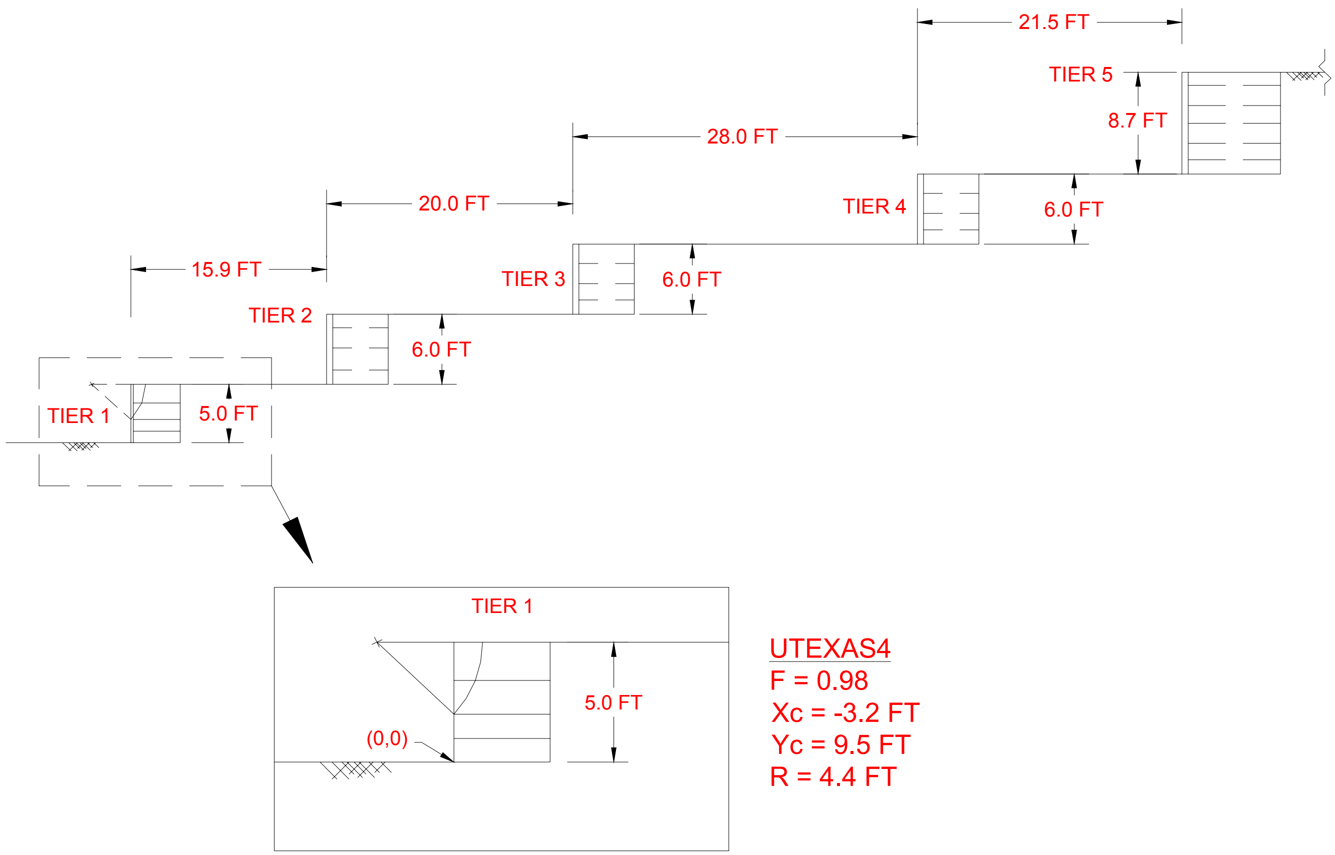

Figure 5-2: The critical slip surface and minimum factor of safety found using the UTEXAS4 software for the Town of Anthony wall.

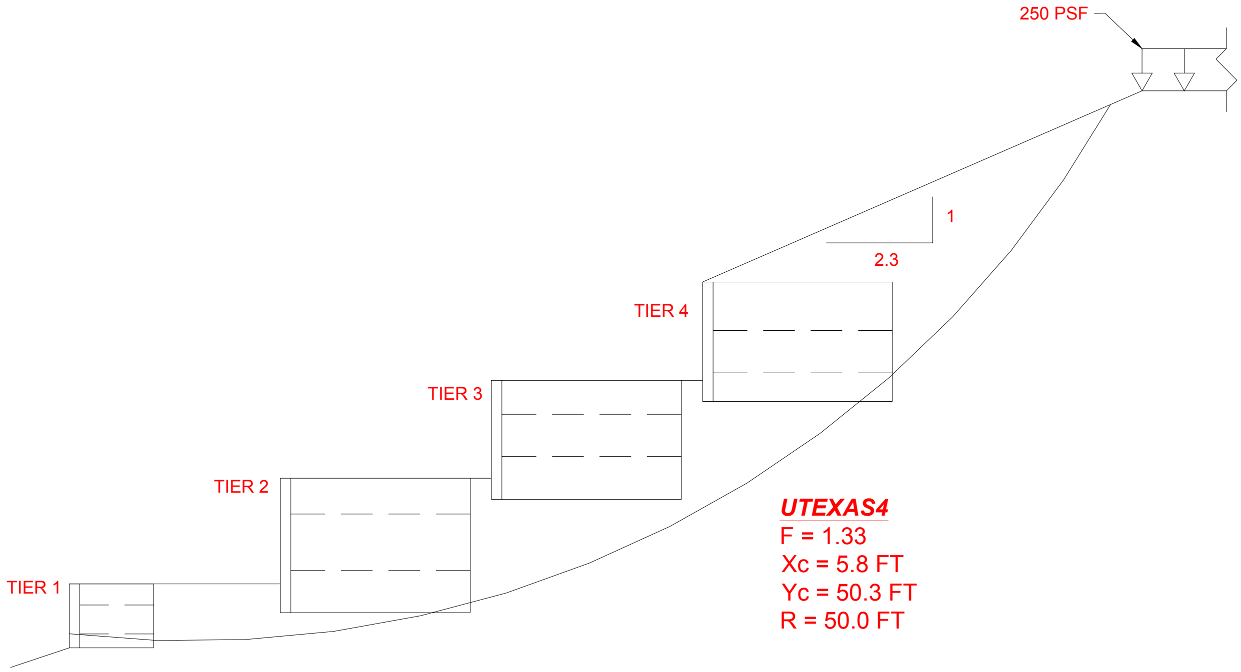

Figure 5-3: Slip surface and the minimum factor calculated using the UTEXAS4 software assuming the surcharge load is included and the soil in front of the wall is neglected for the US 67 Bypass wall.

5.3 Common Features and Characteristics of Analyses

All three of the walls that were analyzed as multi-tier walls were designed using the same strength properties and unit weights. The original designer of each wall specified that the reinforced soil has an angle of internal friction (φ) of 34°, and the retained and foundation soils have an angle of internal friction of 30°. All soils are assumed to be cohesionless ($c = 0$) and have a unit weight (γ) of 120 pcf. The soil properties are summarized in Table 5-3.

Table 5-3: Design properties of the soil used for each of the multi-tier walls presented in this chapter.

Soil Mass	**Total Unit Weight, γ (pcf)**	**Angle of Internal Friction, φ (deg)**	**Cohesion, c (psf)**
Reinforced soil	120	34	0
Foundation soil	120	30	0
Retained soil	120	30	0

Global stability analyses for this study were performed using the MSEW 2.0 and UTEXAS4 software. Analyses with the MSEW 2.0 software were conducted using the Comprehensive Bishop option and horizontal reinforcement forces.[4] For the Comprehensive Bishop option, the factor of safety is applied to the shear strength of the soil only.[5] The analyses conducted using the UTEXAS4 software were performed using the long-term design strength (Tal) to compute the reinforcement forces, and the reinforcement forces were assumed to act horizontally. As with all analyses using the UTEXAS4 software, the factor of safety was applied to the shear strength of the soil only.

5.4 Socorro Bridge Wall

The Socorro Bridge wall is located on Loop 375 in El Paso County, Texas, and was designed by Tensar Earth Technologies, Inc. The wall has two tiers and supports a highway embankment. The original designer (Tensar) conducted analyses using the FHWA 1997 guidelines. The tallest section of the wall was selected for analyses.

5.4.1 Wall Geometry and Material Design Properties

The overall length of the wall is approximately 230 ft. The tallest section is 19.33 ft. high, which is the total height for two tiers. The bottom tier, referred to as Tier 1, is 12.34 ft. tall measured from the bottom of the reinforced soil zone to the top of the tier. The second tier, Tier 2, is 6.99 ft. tall measured from the bottom of the reinforced soil zone of Tier 2 to the top of the tier. The wall is not embedded into the foundation soil. Tier 2 is also not embedded; it rests entirely above the top of Tier 1. The backfill behind the face of

[4] The assumptions used by the MSEW 2.0 software are discussed in detail in Chapter 3.
[5] Definitions for the factors of safety are discussed in detail in Chapter 2.

the upper tier slopes at 3H:1V and supports what is referred to on the construction drawings as *concrete stamped riprap*, which is modeled as a uniform vertical surcharge load of 100 psf (Figure 5-4). The slope of the backfill extends for a horizontal distance of 29.5 ft. from the face of the upper tier, at which point the backfill becomes horizontal. A uniform surcharge of 1,146 psf was applied by the original designer, beginning at the point where the backfill becomes horizontal; however, the purpose of the surcharge was not indicated. The wall was designed with precast concrete modular blocks for the facing and Tensar geogrid for the reinforcement.

The geogrid reinforcement is manufactured by Tensar Earth Technologies and is referred to as *MESA geogrid*. Two geogrid products (UXMESA3 and UXMESA5) were used in the design. The ultimate tensile strengths and reduction factors for these are summarized in Table 5-4.

Table 5-4: Design properties of the Tensar geogrid used in MSE wall section on Loop 375.

Tensar Geogrid	**Ultimate Tensile Strength, T_{ult} (lb/ft)**	**Reduction Factors**		
		Durability (RF_d)	**Installation Damage (RF_{id})**	**Creep (RF_c)**
UXMESA3	4,392	1.10	1.12	2.40
UXMESA5	8,997	1.10	1.20	2.45

The width of the geogrid is 4.30 ft., and the length is 13.65 ft. for both tiers. Both horizontal and vertical spacings were varied depending on the layer. A summary of the type, width, horizontal spacing (S_H), and coverage ratio of the geogrid for each layer of reinforcement are shown in Table 5-5. The vertical spacing of each layer is shown in Figure 5-5.

5.4.2 External Stability

External stability analyses were conducted to verify the safety of the wall against overturning and bearing capacity. FHWA guidelines specify that the sliding stability of two-tier walls be evaluated by performing global stability analyses. Global stability analyses are discussed later in Section 5.4.4. Calculations for safety against overturning and bearing capacity failure were performed both by hand and by using the MSEW 2.0 software. The detailed hand calculations are provided in Appendix C.

FHWA guidelines state only that the weight of the upper tier can be considered as a surcharge in computing bearing pressures for the lower tier. FHWA guidelines provide no guidance pertaining to the location of the vertical plane where the horizontal earth pressure forces should be calculated and applied on the back side of the wall. The MSEW 2.0 software performed external stability analyses using a vertical plane at the back edge of the reinforced soil in the lower tier to calculate the horizontal earth pressure forces F_1 and F_2 (Figure 5-6). The hand calculations in Appendix C were performed using the same vertical plane as that used by the MSEW 2.0 software. This assumption for the vertical plane neglects any contribution of the reinforcement in the upper tier to resist sliding.

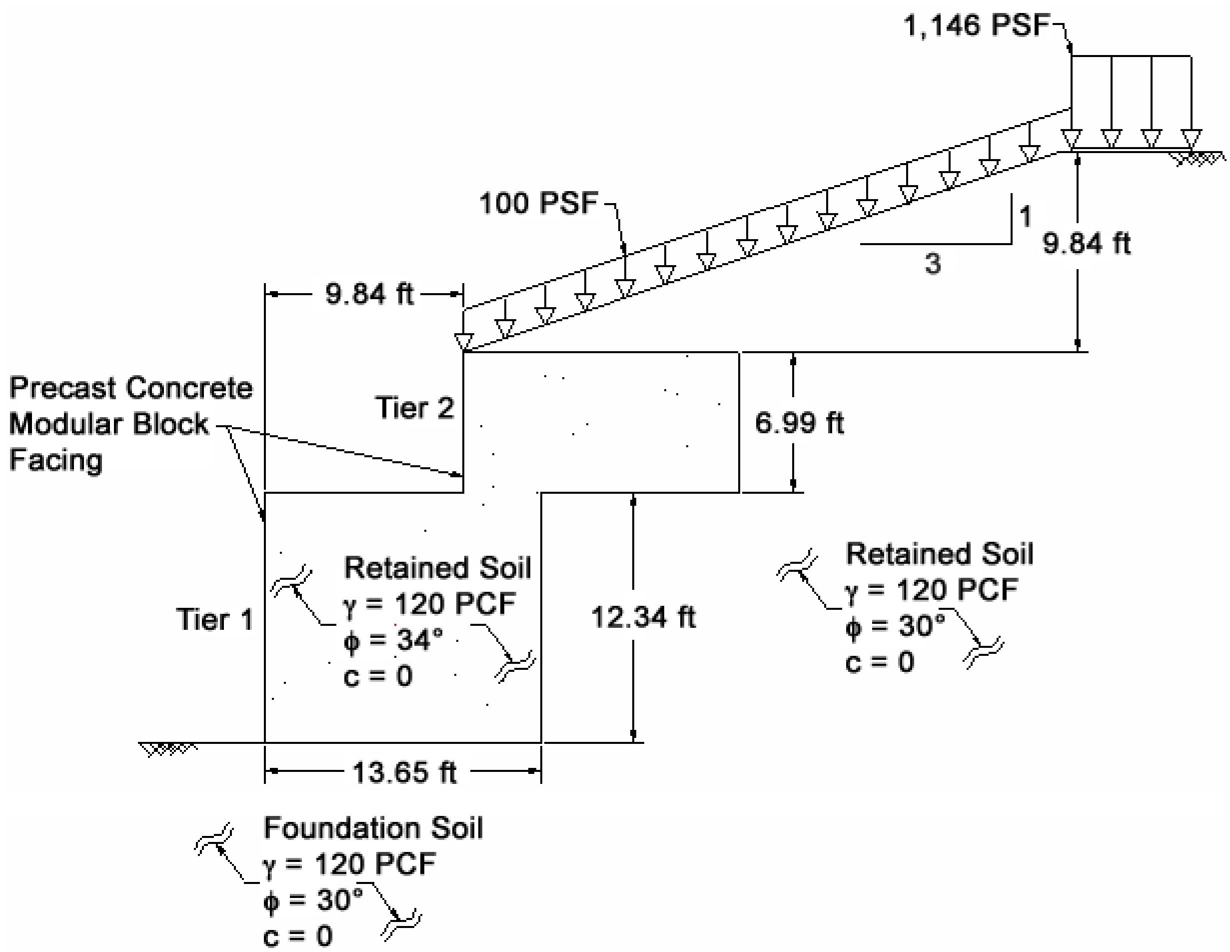

Figure 5-4: Cross-section of the Socorro Bridge wall the with typical soil design properties.

Table 5-5: Detailed summary of the layers of reinforcement for the Socorro Bridge wall.

Tier	Layer	Elevation (ft)	Tensar Geogrid	Width of Geogrid, b (ft)	Horizontal Spacing, S_H (ft)	Coverage Ratio, R_c
2 (top)	10	18.34	UXMESA3	4.30	7.61	0.56
	9	16.31	UXMESA3	4.30	7.61	0.56
	8	14.34	UXMESA5	4.30	7.61	0.56
1 (bottom)	7	11.35	UXMESA3	4.30	7.61	0.56
	6	9.32	UXMESA3	4.30	7.61	0.56
	5	7.35	UXMESA3	4.30	7.61	0.56
	4	5.35	UXMESA3	4.30	7.61	0.56
	3	4.04	UXMESA5	4.30	5.71	0.75
	2	2.69	UXMESA5	4.30	4.30	1.00
	1	1.35	UXMESA5	4.30	4.30	1.00

5.4.2.1 Eccentricity Calculation

The eccentricity (*e*) of the resultant of vertical force on the base of the wall, calculated by hand and by using the MSEW 2.0 software, was 1.27 ft. The original designer also calculated an eccentricity of 1.27 ft. All the calculated eccentricities are within the FHWA 2001 maximum allowed eccentricity of L/6 (= 13.65 ft. ⁄ 6 = 2.27 ft.).

5.4.2.2 Bearing Capacity Failure

The safety against bearing capacity failure was evaluated by comparing the vertical stress (σ_v) at the base of the wall with the allowable bearing pressure (q_a) of the foundation soil. Calculations by hand and by using the MSEW 2.0 software revealed a vertical stress (σ_v) of 2,391 psf and an allowable bearing pressure (q_a) of 5,973 psf. Both sets of calculations show that the vertical stress is less than the allowable bearing pressure, and thus, the wall satisfies the FHWA requirements.

The original designer of the wall did not report either a vertical stress or ultimate bearing pressure but did report a factor of safety against bearing capacity of 5.97. For comparison with the original designer's calculations, the factor of safety against bearing capacity failure (FS_{BC}) computed from the ultimate bearing capacity (q_{ult}) of the foundation soil (14,934 psf) and the vertical stress, σ_v (2,391 psf), is 6.24. The factors of safety calculated by hand, by using the MSEW 2.0 software, and by the original designer all exceed the FHWA requirement that FS_{BC} be greater than 2.5.

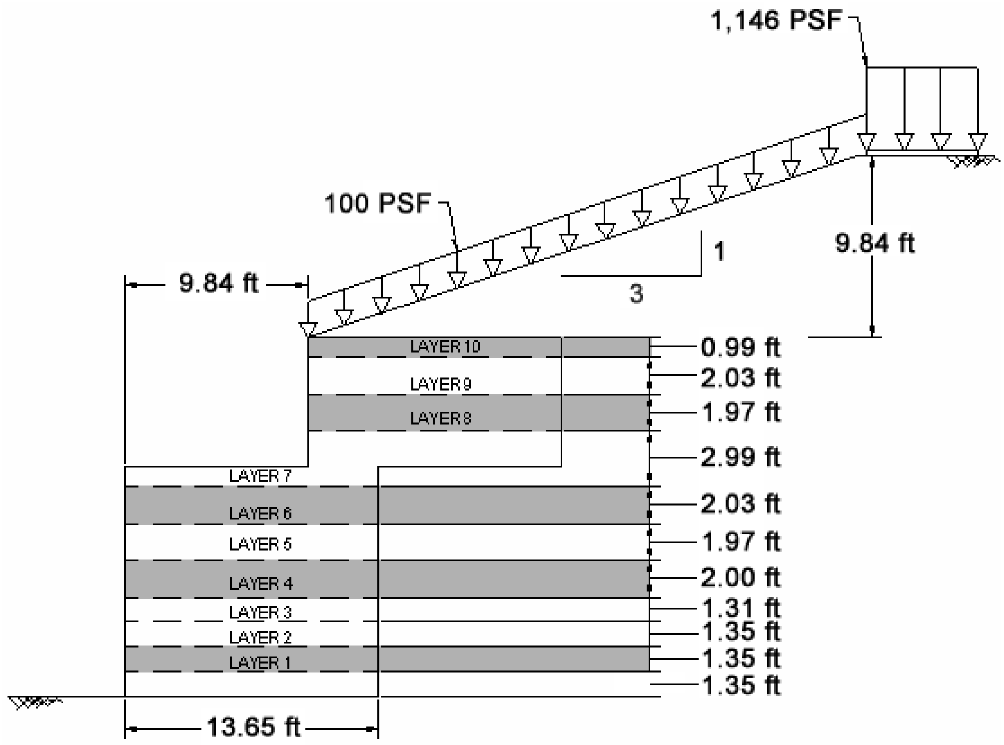

Figure 5-5: Vertical spacing of the geogrid reinforcement in the Socorro Bridge wall.

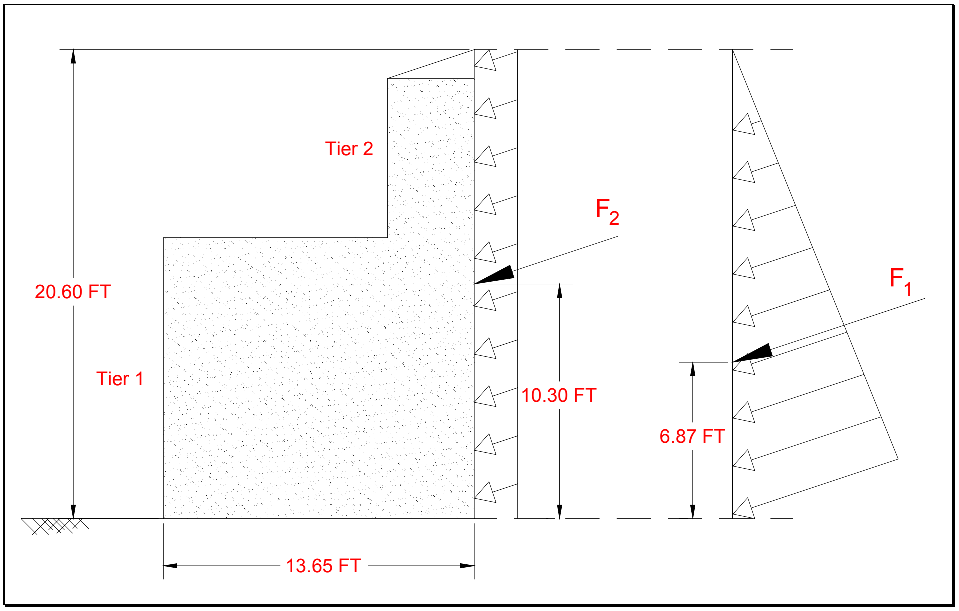

Figure 5-6: Illustration of the earth pressure forces F_1 and F_2 calculated using the MSEW 2.0 for the external stability analyses conducted for the Socorro Bridge wall.

5.4.3 Internal Stability

The factors of safety against rupture and pullout of the reinforcement were evaluated for each layer of reinforcement. Representative calculations are presented in Appendix C to illustrate the procedures used. The calculations in Appendix C are for a reinforcing layer (Layer 3) at an elevation of 4.04 ft. from the bottom of the reinforced soil zone.

5.4.3.1 Factor of Safety Against Rupture of the Reinforcement

The factor of safety against rupture (FS_R) of the reinforcement was calculated for each layer, and the values are summarized in Table 5-6. For comparison, the corresponding factors of safety computed by using the MSEW 2.0 software and by the original designer are also shown in this table. The factors of safety presented in Table 5-6 on the basis of the calculations by hand and by using the MSEW 2.0 software show that two layers of reinforcement do not meet FHWA minimum requirements. However, the factors of safety calculated by the original designer meet the FHWA guidelines requirements.

Table 5-6: Factors of safety against rupture of the reinforcement.

Tier	Layer	Elevation (ft)	Factor of Safety Against Rupture (FS_R)			All Satisfy FHWA Requirements? ($FS_R \geq 1.5$)
			Hand Calculation	MSEW 2.0	Original Designer	
2	10	18.34	2.99	3.00	3.01	Yes
	9	16.31	2.00	2.01	2.02	Yes
	8	14.34	1.90	1.79	1.80	Yes
1	7	11.35	12.35	1.90	2.03	Yes
	6	9.32	4.06	2.02	4.00	Yes
	5	7.35	1.52	1.20	2.16	No
	4	5.35	1.25	1.23	1.62	No
	3	4.04	3.52	3.41	3.77	Yes
	2	2.69	4.20	4.09	4.10	Yes
	1	1.35	2.55	2.45	2.31	Yes

The hand calculations and calculations with the MSEW 2.0 software gave different factors of safety, owing to the assumptions made in the MSEW 2.0 software for calculating the maximum tension (T_{MAX}) in the reinforcement assuming a trapezoidal distribution of stress, as discussed in Chapter 3 (cf. Section 3.15).

The original designer performed stability calculations with design software known as MesaPro 1.0, a version of software specialized for Tensar Earth Technologies, Inc. that is very similar to MSEW 2.0. Consequently, the differences in factors of safety against rupture of the reinforcement were not expected to be found between the calculations performed with MSEW 2.0 software and those performed with the MesaPro 1.0 software. The cause(s) of the differences in factor of safety could not be determined, owing to a lack of information about the original design and MesaPro 1.0 software.

5.4.3.2 Factor of Safety Against Pullout of the Reinforcement

The factor of safety against rupture (FS_{PO}) of the reinforcement was calculated for each layer by hand and by using the MSEW 2.0 software. The values are summarized in Table 5-7. For comparison, the corresponding factors of safety computed by the original designer are also shown in this table. All the factors of safety presented in Table 5-7 meet FHWA minimum requirements with respect to pullout of the reinforcement. However, some differences exist in the factors of safety calculated by different methods or individuals.

Table 5-7: Factor of safety against pullout for each layer of the reinforcement in the US 290 wall.

Tier	**Layer**	**Elevation (ft)**	**Factor of Safety Against Pullout (FS_{PO})**			**All Satisfy FHWA Requirements? ($FS_{PO} \geq 1.5$)**
			Hand Calculation	**MSEW 2.0**	**Original Designer**	
2	10	18.34	2.70	3.54	3.32	Yes
	9	16.31	6.08	6.67	6.25	Yes
	8	14.34	5.56	5.51	5.50	Yes
1	7	11.35	8.12	10.95	10.93	Yes
	6	9.32	9.29	16.34	30.08	Yes
	5	7.35	6.46	12.82	21.44	Yes
	4	5.35	8.24	16.72	20.61	Yes
	3	4.04	15.63	28.78	31.68	Yes
	2	2.69	23.08	39.71	39.73	Yes
	1	1.35	16.96	27.14	25.57	Yes

The differences between the factors of safety calculated by hand and by using the MSEW 2.0 software are caused by different assumptions used to determine the depth (Z_P) for calculating the pullout resistance (P_r) of the reinforcement, as discussed in Chapter 3 (cf. Section 3.15).

The causes of the differences between the factors of safety calculated for this study and those reported by the original designer are not known. However, the factors of safety reported by the original designer are closer to the ones calculated by using the MSEW 2.0 software than to those calculated by hand.

5.4.4 Global Stability

Global stability analyses were conducted using the MSEW 2.0 and UTEXAS4 software. With the MSEW 2.0 software, the critical slip surface passed beneath the toe of the wall (Figure 5-7); thus, the uniform (traffic) surcharge load was automatically neglected. As discussed in Chapter 3, the MSEW 2.0 software automatically neglects the uniform surcharge load when performing global stability analyses. The minimum factor of safety reported by the MSEW 2.0 software was 1.29.

The UTEXAS4 computations were performed with the intent to model the most critical condition, and thus, the uniform surcharge was included. The minimum factor of safety computed using the UTEXAS4 software is 1.18 (Figure 5-8). Analyses were also performed with the UTEXAS4 software neglecting the uniform surcharge load, and the computed factor of safety was 1.30, which is very close to the value of 1.29 from the MSEW 2.0 software (Figure 5-9). The original designer, performing analyses using the MesaPro 1.0 software, calculated a factor of safety of 1.28, which is in close agreement with the values calculated using the MSEW 2.0 and the UTEXAS4 software with the surcharge neglected.

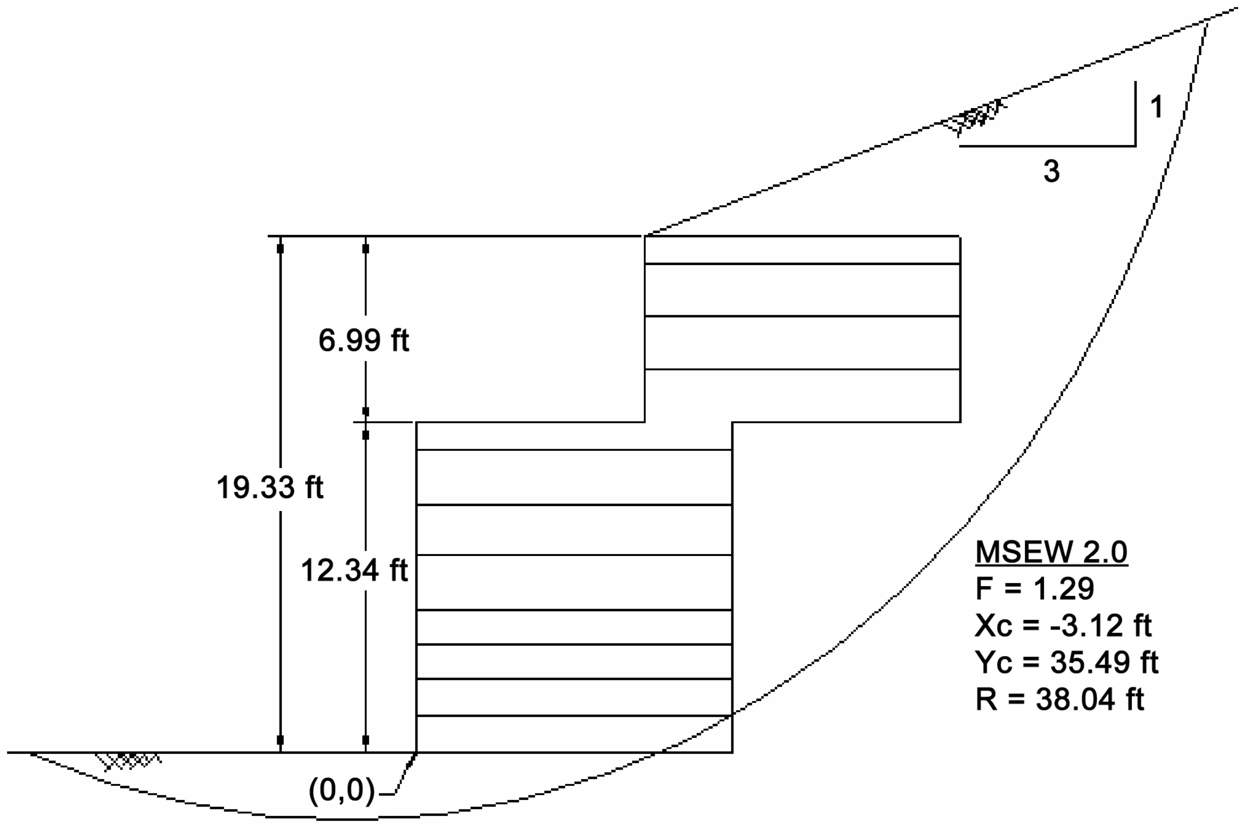

Figure 5-7: Critical slip surface and minimum factor or safety found by the MSEW 2.0 software for the Socorro Bridge wall.

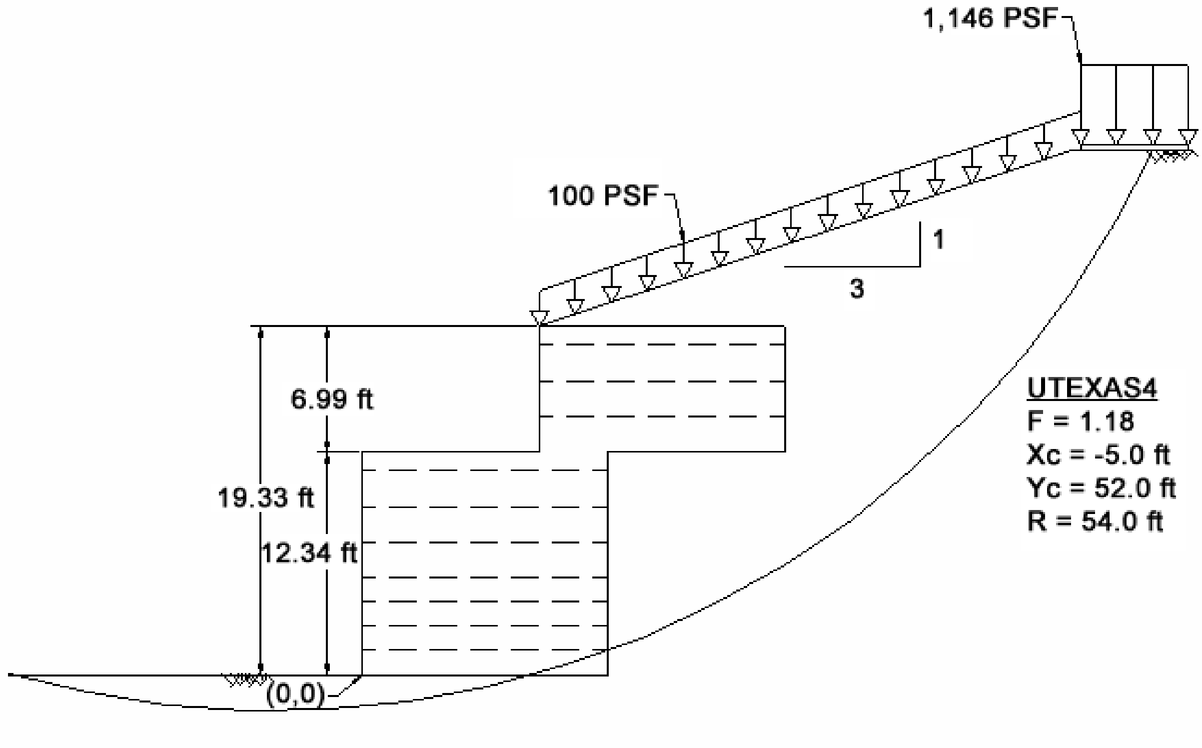

Figure 5-8: Critical slip surface and minimum factor of safety found by the UTEXAS4 software assuming the surcharge load is included and the soil in front of the wall is neglected for the Socorro Bridge wall.

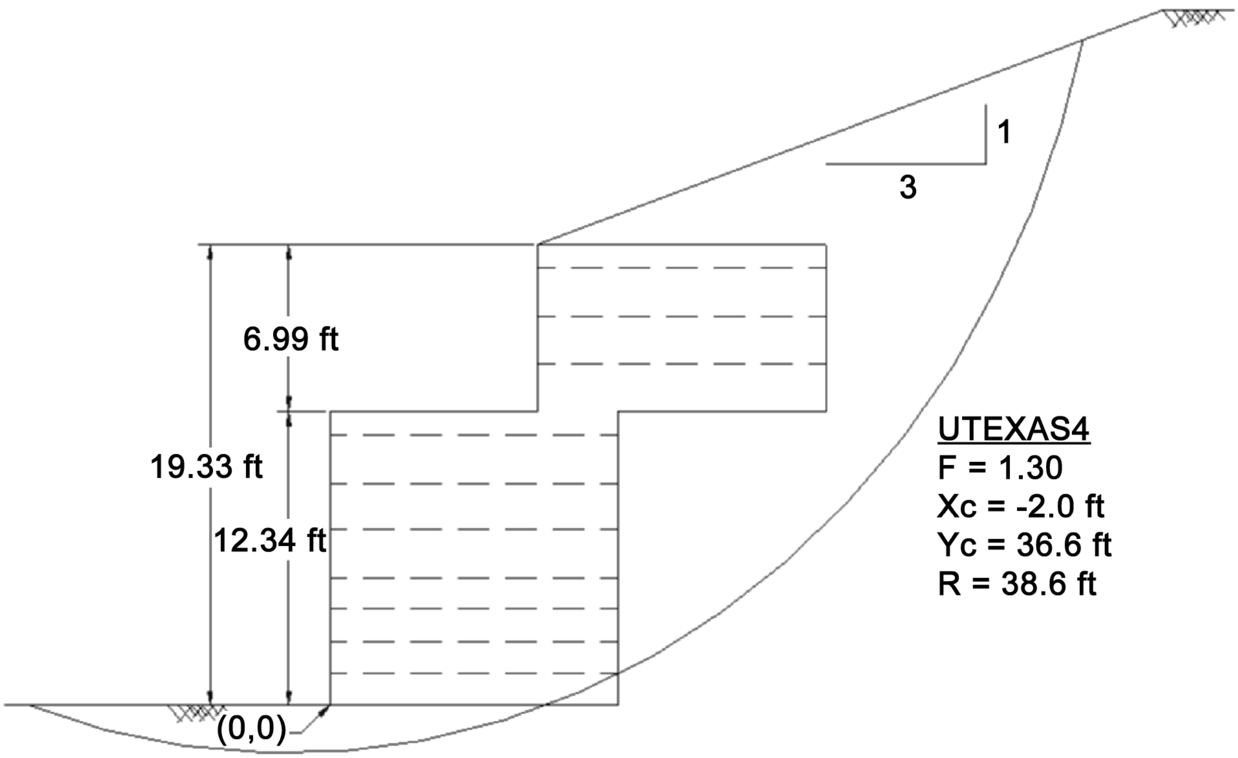

Figure 5-9: Critical slip surface and minimum factor of safety found by the UTEXAS4 software assuming the surcharge load and the soil in front of the wall are neglected for the Socorro Bridge wall.

5.5 US 290 Wall

The second multi-tier wall analyzed is located in Travis County, Texas. This wall was also designed by Tensar Earth Technologies, Inc. The wall supports the US 290 highway embankment. Analyses for this study were conducted in accordance with FHWA 2001 design criteria for two-tier walls. Additional tiers were assumed to act as surcharges applied to the top of the upper tier. The original designer specified that analyses were conducted using "AASHTO Design Guidelines," but did not identify which version of AASHTO guidelines was used.

5.5.1 Wall Geometry and Material Design Properties

The overall length of the wall is approximately 714 ft. The wall section has four tiers, referred to as Tiers 1 through 4, with Tier 1 being at the bottom. The tallest section of the wall was chosen for analyses. The total wall height for this section is 29.5 ft., measured from the bottom of the reinforced soil zone to the top of Tier 4. The soil properties used for design and the traffic loads, which are represented as a uniform surcharge of 250 psf on the top of Tier 4, are shown in Figure 5-10. The height of each tier and the length of the reinforced soil zone vary from tier to tier. The lower tier of the wall is embedded into the foundation soil 1.0 ft. Each overlying tier is embedded into the top of the tier below. The depth of embedment varies from 0.8 to 1.5 ft. The wall is designed with precast concrete modular blocks for facing and geogrid reinforcement.

The reinforcement was manufactured by Tensar Earth Technologies and is known as *MESA geogrid*. Ultimate tensile strengths (T_{ult}) and reduction factors (RF) for the geogrid are provided in Table 5-8.

Table 5-8: Design properties of the Tensar geogrid used in MSE wall section on US 290 wall.

Tensar Geogrid	**Ultimate Tensile Strength, T_{ult} (lb/ft)**	**Reduction Factors**		
		Durability (RF_d)	**Installation Damage (RF_{id})**	**Creep (RF_c)**
UXMESA3	4,720	1.10	1.05	2.65
UXMESA4	7,550	1.10	1.05	2.65

All layers of reinforcement are continuous—that is, there is no space between strips of geogrid in the horizontal direction. The *horizontal spacing* (S_H), which is considered to be the width (b) of the reinforcement and the coverage ratio (R_c), is 1.0. The vertical spacing of the geogrid varies (Figure 5-11). The elevation from the toe of the wall and the length of each layer of reinforcement are summarized in Table 5-9.

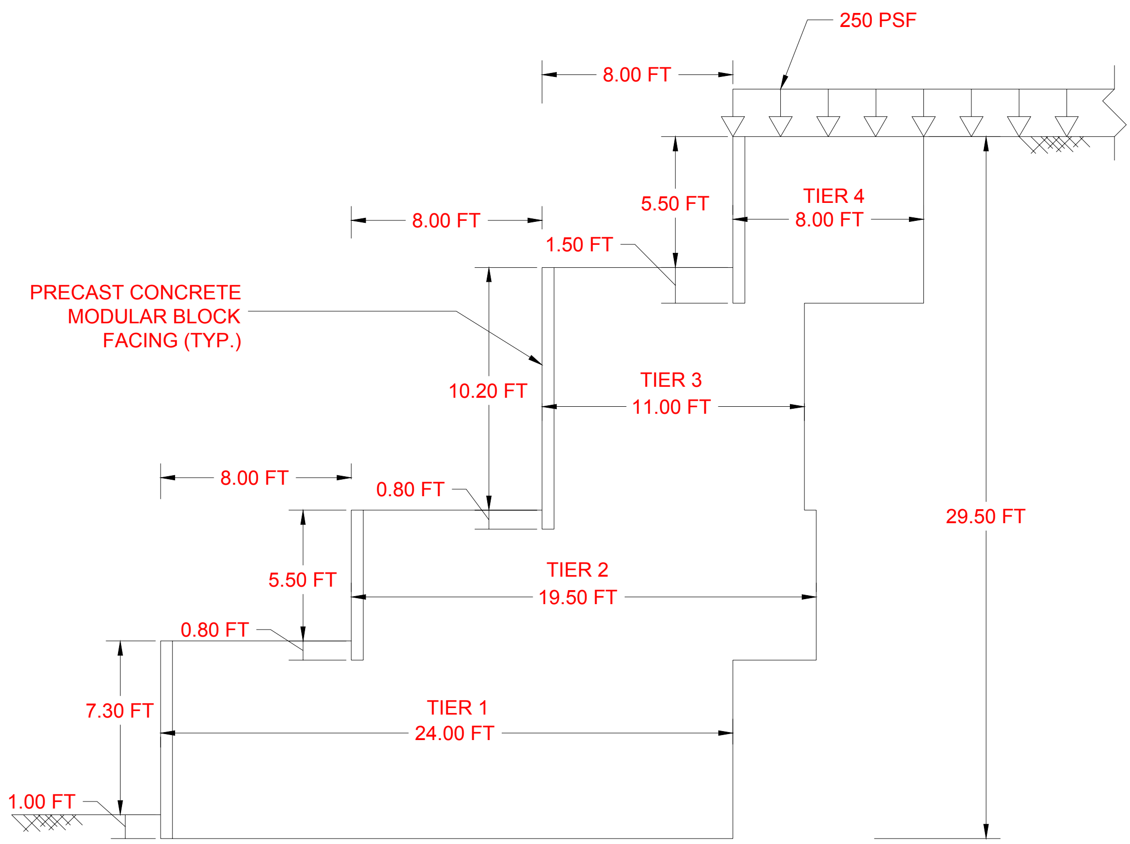

Figure 5-10: Illustration of the height of each tier, the length of reinforcement, and the offset between tiers for the US 290 wall.

Table 5-9: The elevation of reinforcement above the toe of the wall and the type of geogrid used at each layer of reinforcement in the MSE wall on US 290 wall.

Tier	Layer	Elevation (ft)	Tensar Geogrid	Length of the Reinforcement, L (ft)
4	16	27.53	UXMESA3	8.0
	15	25.53	UXMESA3	8.0
	14	23.53	UXMESA3	8.0
3	13	22.63	UXMESA3	11.0
	12	21.33	UXMESA3	11.0
	11	20.03	UXMESA3	11.0
	10	18.03	UXMESA3	11.0
	9	16.03	UXMESA3	11.0
	8	14.03	UXMESA3	11.0
2	7	12.53	UXMESA3	19.5
	6	10.53	UXMESA3	19.5
	5	8.53	UXMESA3	19.5
1	4	6.67	UXMESA3	24.0
	3	4.67	UXMESA4	24.0
	2	2.67	UXMESA4	24.0
	1	0.67	UXMESA4	24.0

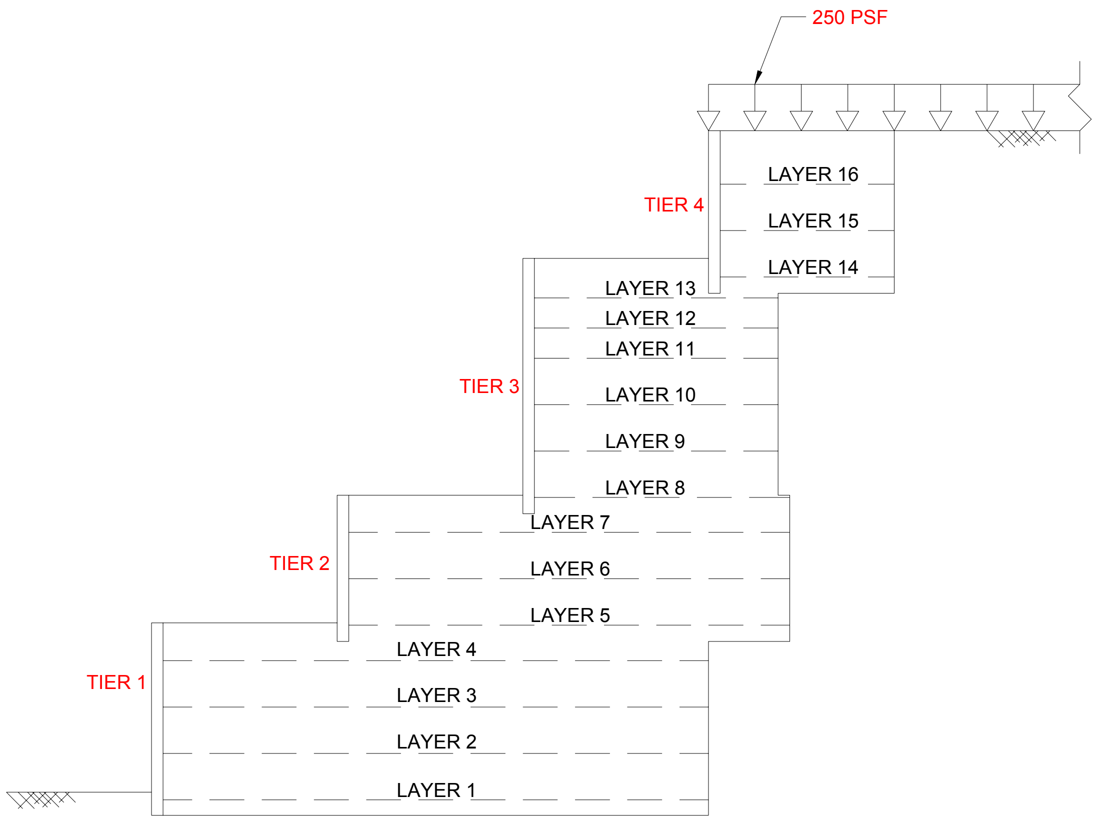

Figure 5-11: Layout of the geogrid reinforcement in the US 290 wall.

5.5.2 External Stability

Analyses were performed to verify external stability for overturning and bearing capacity. Calculations for overturning and bearing capacity failure were performed using the MSEW 2.0 software for the "equivalent" two-tier wall shown in Figure 5-12. Hand calculations were not performed for the US 290 wall, because additional approximations to those made in the MSEW 2.0 software would have been needed. Because the purpose of this study was to identify and not to introduce assumptions used in design, hand calculations were not performed.

5.5.2.1 Eccentricity Calculation

The eccentricity computed using the MSEW 2.0 software was −0.76 ft. A negative value of eccentricity indicates that the resultant of the vertical forces is located toward the retained fill (back of wall) from the centerline. The eccentricity is within the FHWA maximum allowed eccentricity of L/6 (= 24.0 ft. ⁄ 6 = 4.0 ft.). The original designer, using the MesaPro 2.0 software, also calculated this same eccentricity (−0.76 ft.).

5.5.2.2 Bearing Capacity Failure

The safety against bearing capacity failure was evaluated by comparing the vertical stress (σ_v) at the base of the wall with the allowable bearing pressure (q_a) of the foundation soil. The vertical stress computed with the MSEW 2.0 software was 2,071 psf, and the corresponding allowable bearing pressure was 12,085 psf. The original designer calculated the same magnitude of vertical stress and allowable bearing pressure as calculated with the MSEW 2.0 software. The calculations by both methods meet the FHWA requirement that the vertical stress (σ_v) be less than the allowable bearing pressure (q_a).

5.5.3 Internal Stability

Internal stability was assessed by evaluating the factors of safety against rupture and pullout of the reinforcement for each layer. Factors of safety were calculated by hand and by using the MSEW 2.0 software. Representative calculations are provided in Appendix D. The calculations in Appendix D were performed for the reinforcing layer (Layer 2) at an elevation of 2.67 ft. above the bottom of the reinforced soil zone

Calculations for internal stability by hand and by using the MSEW 2.0 software were performed for three separate "equivalent" two-tier wall systems. Consequently, two different factors of safety were computed for the reinforcement in each tier, depending on which pair of tiers was analyzed. For example, different values for the factor of safety for the reinforcement in Tier 2 were computed when Tiers 1 and 2 were modeled and when Tiers 2 and 3 were modeled. The lowest of the two factors of safety was reported.

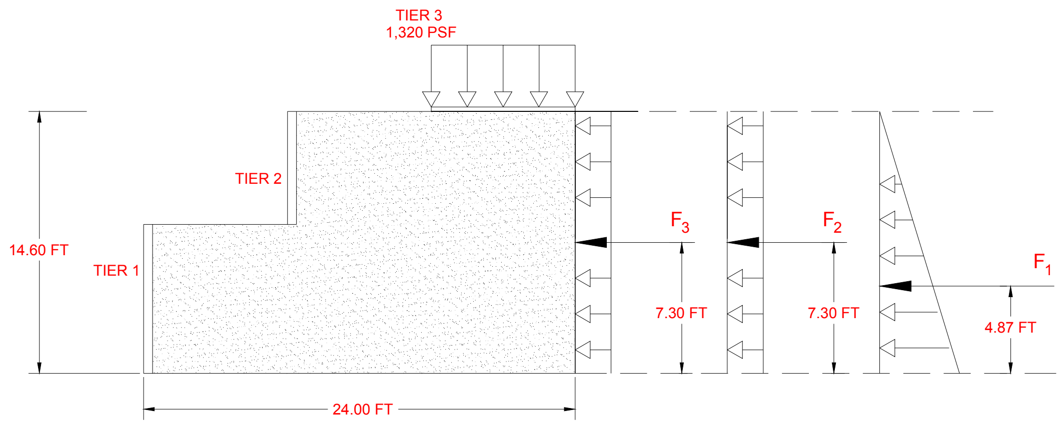

Figure 5-12: Illustration of the earth pressure forces F1, F2, and F3 calculated by the MSEW 2.0 software to perform external stability analyses for the US 290 wall.

5.5.3.1 Factor of Safety Against Rupture of the Reinforcement

The factors of safety against rupture (FS_R) calculated by hand, by using the MSEW 2.0 software, and by the original designer are summarized in Table 5-10. The factors of safety presented in Table 5-10 all indicate that each layer of reinforcement meets FHWA minimum requirements for safety against rupture of the reinforcement. However, some differences exist among the factors of safety calculated by the different methods or individuals.

The differences between the factors of safety calculated by hand and by using the MSEW 2.0 software are caused by assumptions made to calculate the maximum tension (T_{MAX}) in the reinforcement: The MSEW 2.0 software uses a trapezoidal distribution of stress, whereas for hand calculations, stress is calculated at the level of reinforcement as discussed in Chapter 3 (cf. Section 3.15).

Table 5-10: Summary of factors of safety against rupture of the reinforcement for the US 290 wall.

Tier	Layer	Elevation (ft)	Factor of Safety Against Rupture (FS_R)			All Satisfy FHWA Requirements? ($FS_R \geq 1.5$)
			Hand Calculation	MSEW 2.0	Original Designer	
4	16	27.53	5.33	6.13	6.13	Yes
	15	25.53	5.41	5.24	5.24	Yes
	14	23.53	3.86	3.96	3.96	Yes
3	13	22.63	27.30	4.76	5.15	Yes
	12	21.33	10.69	4.39	7.53	Yes
	11	20.03	2.84	2.35	3.85	Yes
	10	18.03	2.10	1.95	2.43	Yes
	9	16.03	1.77	1.66	1.77	Yes
	8	14.03	1.84	1.75	1.73	Yes
2	7	12.53	10.60	2.54	2.53	Yes
	6	10.53	6.26	6.26	4.99	Yes
	5	8.53	4.09	4.48	3.14	Yes
1	4	6.67	10.60	4.98	4.33	Yes
	3	4.67	10.02	3.87	12.05	Yes
	2	2.67	4.83	7.94	5.73	Yes
	1	0.67	4.57	4.83	4.44	Yes

There are also differences between the factors of safety calculated for this study and those calculated by the original designer. The original designer performed stability calculations using the MesaPro 2.0 software. Consequently, the differences in factors of safety against rupture calculated by the MSEW 2.0 software and those calculated by the MesaPro 2.0 software were unexpected. The cause(s) of these differences could not be determined, owing to a lack of information about the MesaPro 2.0 software.

5.5.3.2 Factor of Safety Against Pullout of the Reinforcement

The factors of safety (FS_{PO}) calculated for each layer of reinforcement by hand, by using the MSEW 2.0 software, and by the original designer are summarized in Table 5-11. All the values presented in Table 5-11 meet FHWA minimum requirements, although different values were calculated by the three methods. The differences between the factors of safety calculated by hand and those calculated using the MSEW 2.0 software are caused by the differences in how the maximum tension (T_{MAX}) in the reinforcement is calculated, as described earlier for rupture. Also, differences in the factors of safety for Tier 1 are caused by the depth (Z_P) used by the MSEW 2.0 software for calculating the pullout resistance (P_r) of the reinforcement, as discussed in Chapter 3 (cf. Section 3.15). This affects only the factors of safety reported for Tier 1. The factors of safety reported for Tiers 2, 3, and 4 are minimum values from analyses of equivalent two-tier walls; the minimum values all correspond to the one where each of these tiers (2, 3, and 4) was the upper tier.

Table 5-11: Summary of factors of safety against pullout of the reinforcement on the US 290 wall.

Tier	Layer	Elevation (ft)	Factor of Safety Against Pullout (FS_{PO})			All Satisfy FHWA Requirements? ($FS_{PO} \geq 1.5$)
			Hand Calculation	MSEW 2.0	Original Designer	
4	16	27.53	3.91	3.70	3.57	Yes
	15	25.53	6.64	6.43	6.21	Yes
	14	23.53	9.23	9.44	9.13	Yes
3	13	22.63	9.77	5.34	5.50	Yes
	12	21.33	10.02	7.46	8.84	Yes
	11	20.03	5.77	6.55	10.31	Yes
	10	18.03	7.14	8.08	9.77	Yes
	9	16.03	8.92	9.61	9.89	Yes
	8	14.03	12.73	13.47	12.84	Yes
2	7	12.53	18.52	18.24	17.63	Yes
	6	10.53	25.88	20.87	20.17	Yes
	5	8.53	27.74	27.62	26.70	Yes
1	4	6.67	22.27	64.99	70.21	Yes
	3	4.67	30.80	103.56	151.86	Yes
	2	2.67	24.18	76.33	87.55	Yes
	1	0.67	32.47	87.18	80.41	Yes

5.5.4 Global Stability

Factors of safety for global stability were calculated using the MSEW 2.0 and UTEXAS4 software, and these values were compared with the values reported by the original designer. For analyses performed with the MSEW 2.0 software, the same three "equivalent" two-tier walls that were used for internal stability were again used. For each of the equivalent two-tier walls, the critical slip surface was found to pass beneath the toe of the wall. Thus, the MSEW 2.0 software automatically neglected the uniform (traffic) surcharge load. The minimum factors of safety calculated using the MSEW 2.0 software for each "equivalent" wall are summarized in Table 5-12.

Table 5-12: Factors of safety computed from global stability analyses using the MSEW 2.0 software for "equivalent" two tier walls on US 290.

Tiers Modeled	Factor of Safety from Global Stability Analysis	Reference
1 and 2	1.50	Figure 5-13
2 and 3	1.45	Figure 5-14
3 and 4	1.42	Figure 5-15

The factor of safety was computed with the UTEXAS4 software excluding the effects of wall embedment and including the uniform surcharge. The minimum factor of safety calculated using UTEXAS4 was 1.32 (Figure 5-16).

The global stability analysis conducted by the original designer excluded the effects of wall embedment and included the uniform surcharge load. The long-term design strength (T_{al}) was used for the reinforcement forces; however, the orientation of the reinforcement forces on the slip surface is not known. The original designer reported a factor of safety of 1.30.

The minimum factor of safety calculated using the UTEXAS4 software (1.32) is essentially identical to the factor of safety calculated by the original designer (1.30). Both of these factors of safety were computed in analyses in which the entire four-tier wall was modeled.

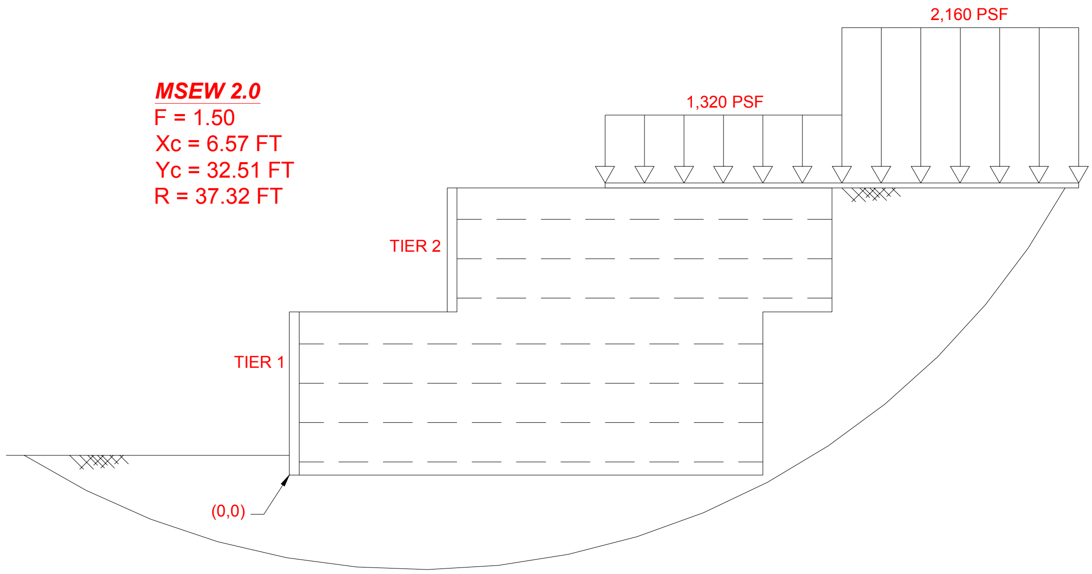

Figure 5-13: Slip surface and the minimum factor of safety for the "equivalent" two-tier wall using Tiers 1 and 2 with the MSEW 2.0 software on the US 290 wall.

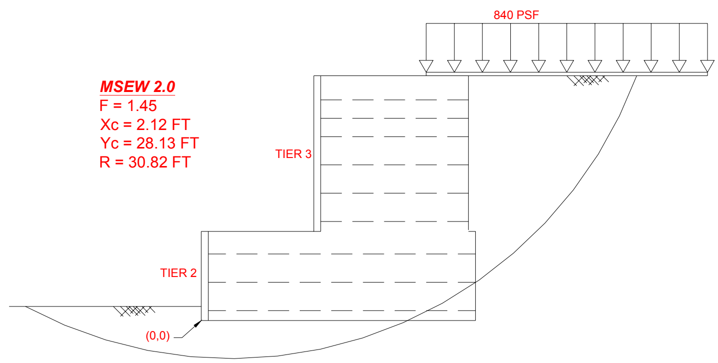

Figure 5-14: Slip surface and minimum factor of safety for the "equivalent" two-tier wall using Tiers 2 and 3 with the MSEW 2.0 software on the US 290 wall.

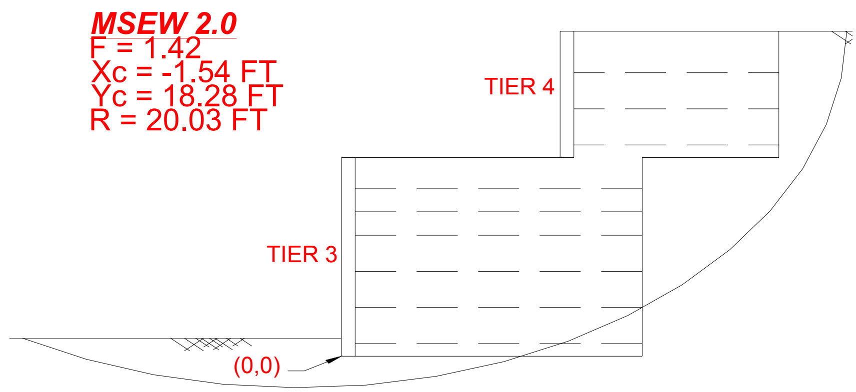

Figure 5-15: Slip surface and minimum factor of safety for the "equivalent" two-tier wall using Tiers 3 and 4 with the MSEW 2.0 software on the US 290 wall.

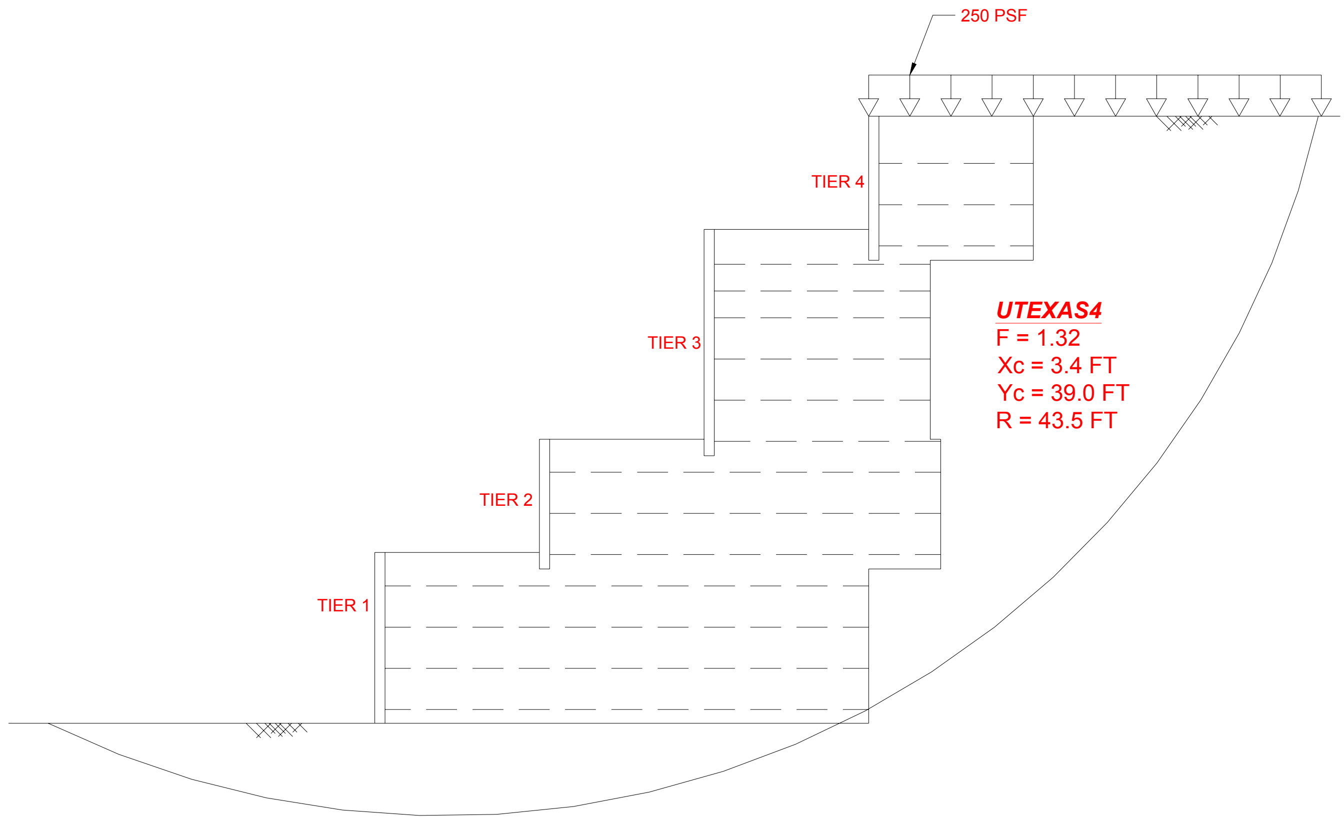

Figure 5-16: Slip surface and the minimum factor of safety calculated using the UTEXAS4 software assuming the surcharge load is included and the soil in front of the wall is neglected for the US 290 wall.

5.6 US 67 Bypass, Cleburne, TX

The third wall is located in the town of Cleburne in Johnson County, Texas, and was designed by Tensar Earth Technologies, Inc. The wall has four tiers and supports an embankment for US Highway 67. The design guidelines used by the original designer were not available and are not known. The only documentation available for this wall was the construction drawings. Consequently, no comparison could be made with analyses by the original designer.

As discussed in Section 5.2.1, the tier heights and offset distances for this wall are configured in such a way that, according to the FHWA guidelines, the wall can be designed as a series of individual single-tier walls. Because the wall satisfies FHWA criteria for analysis as single-tier walls, the wall must be input into the MSEW 2.0 as a single-tier wall. Therefore, four separate analyses are required. Accordingly, the US 67 Bypass wall was analyzed for external and internal stability by hand and by using the MSEW 2.0 software on the basis of the assumption that each tier acted as an individual single-tier wall. These analyses are referred to in the following sections as *single-tier analysis*.

Additional external and internal stability analyses were then performed to explore the validity of treating each tier as a single wall. The additional analyses were performed using the MSEW 2.0 software to model a series of single-tier walls assuming the upper tiers are applied to the top of the wall as surcharge loads. These analyses are referred to as *multi-tier analysis* in the following sections.

5.6.1 Wall Geometry and Reinforcement Properties

The overall length of the wall is approximately 340 ft. The section of the wall analyzed has four tiers, referred to as Tiers 1 through 4, with Tier 1 at the bottom. The total height of the wall at the section chosen for analyses is 17.33 ft., measured from the bottom of the reinforced soil zone to the top of Tier 4. A cross-section of the tiered wall with key dimensions is shown in Figure 5-17. The walls are designed with precast concrete modular blocks and geogrid reinforcement. Traffic loads are represented by a uniform surcharge load of 250 psf on the top of Tier 4. The height of each tier and the length of the reinforced soil zone vary from tier to tier. The wall is embedded into the foundation soil 0.75 ft. Each overlying tier is embedded into the tier below by an amount that varies from 1.0 ft. to 1.36 ft.

The reinforcement is a Tensar geogrid designated UX1000SB. The ultimate tensile strength (T_{ult}) and reduction factors (RF) for the geogrid used in the original design were not available. However, properties of the UX1000SB geogrid were obtained from the manufacturer's current product guide and are shown in Table 5-13. These properties were used for the analyses for this study, but it is not known whether they are the same as those used by the original designer.

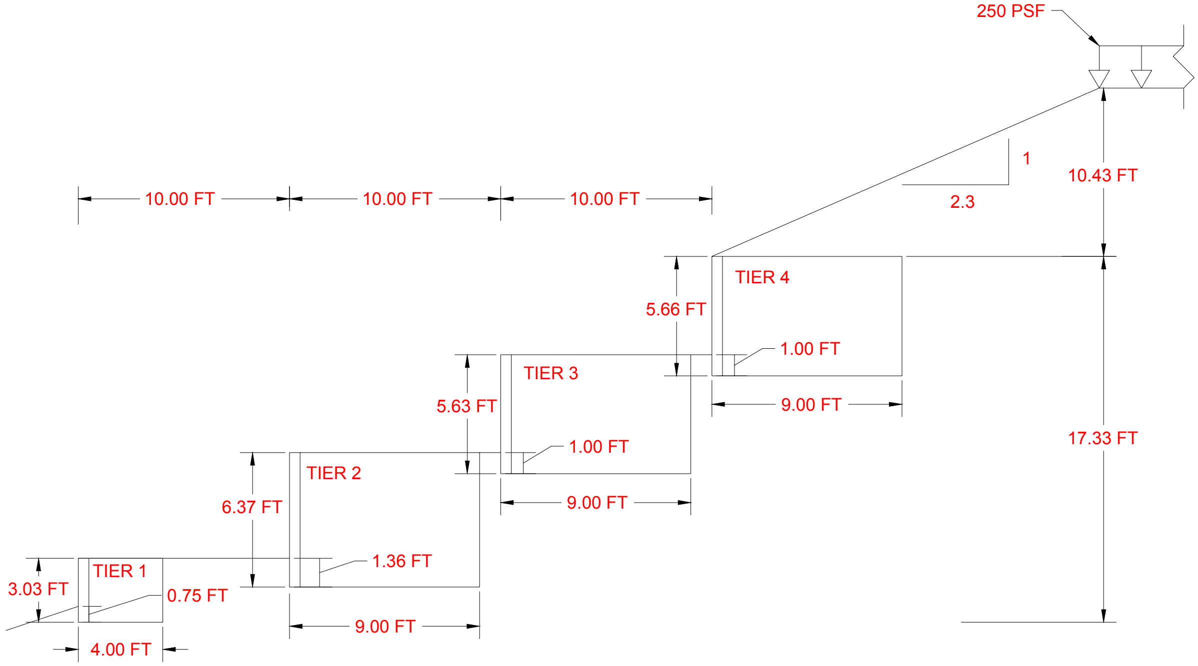

Figure 5-17: Cross-section of the US 67 Bypass wall illustrating the soil properties used in design.

Table 5-13: Design properties of the Tensar geogrid used in MSE wall section on US 290.

Tensar Geogrid		UX1000
Ultimate Tensile Strength, T_{ult} (lb/ft)		3,152
Reduction Factors	Durability (RF_d)	1.10
	Installation damage (RF_{id})	1.10
	Creep (RF_c)	2.60

Each layer of reinforcement is continuous, with no horizontal separation between adjacent strips of geogrid. Accordingly, the horizontal spacing (S_H) is equal to the width (b) of the reinforcement, and the coverage ratio (R_c) is 1.0. The vertical spacing of the geogrid varies from layer to layer. The length and height above the toe of the wall of each layer of reinforcement are shown in Table 5-14. The layout of the geogrid is also shown in Figure 5-18.

Table 5-14: The elevation of reinforcement above the toe of the wall and the type of geogrid used at each layer of reinforcement in the MSE wall on US 67 Bypass.

Wall Tier	**Reinforcement Layer**	**Height above Toe of Wall (ft)**	**Length of the Reinforcement, L (ft)**
4	8	15.03	9.0
	7	13.03	9.0
3	6	11.07	9.0
	5	9.07	9.0
2	4	6.34	9.0
	3	3.67	9.0
1	2	2.03	4.0
	1	0.66	4.0

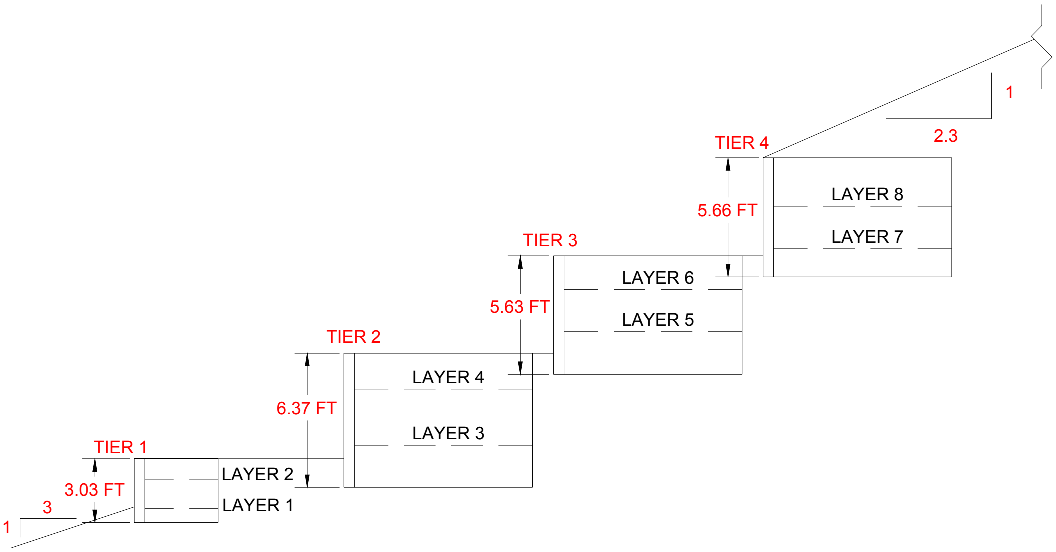

Figure 5-18: Layout of the geogrid reinforcement in the US 67 Bypass wall.

5.6.2 External Stability

External stability analyses were performed to evaluate the safety of the wall against sliding, overturning (eccentricity), and bearing capacity. The four "equivalent" two-tier wall representations of the multi-tier wall analyzed with the MSEW 2.0 software are shown in Figures 5-19, 5-20, 5-21, and 5-22.

5.6.2.1 Sliding Stability

Hand calculations and the calculations performed using the MSEW 2.0 software produced the factors of safety against sliding (FS_{SL}) shown in Table 5-15 for each of the four tiers.

Table 5-15: Factors of safety against sliding calculated by hand and by using the MSEW 2.0 software on the US 67 Bypass wall.

Tier	Factor of Safety Against Sliding (FS_{SL})			Satisfy FHWA Requirements? ($FS_{SL} \geq 1.5$)
	Single-Tier Analysis		Multi-Tier Analysis	
	Hand Calculation	MSEW 2.0	MSEW 2.0	
4	8.27	8.27	8.27	Yes
3	5.54	5.54	3.20	Yes
2	4.89	4.89	2.85	Yes
1	4.57	4.57	4.57	Yes

The values shown in Table 5-15 exceed the FHWA minimum requirement of 1.5; however, there are differences between the factors of safety calculated for the single-tier analysis and those calculated for the multi-tier analysis. The differences are caused by the different methods (i.e., each tier assumed to be independent vs. the upper tiers modeled as surcharge loads) used in each analysis.

5.6.2.2 Evaluation of Safety Against Overturning

Eccentricities computed for each tier by hand and by using the MSEW 2.0 software are summarized in Table 5-16. All the eccentricities are within the FHWA maximum allowed eccentricity of L/6, but differences exist depending on whether the tiers were considered to be independent walls (single-tier analysis) or whether the upper tiers were modeled as a surcharge (multi-tier analysis) for Tiers 2 and 3.

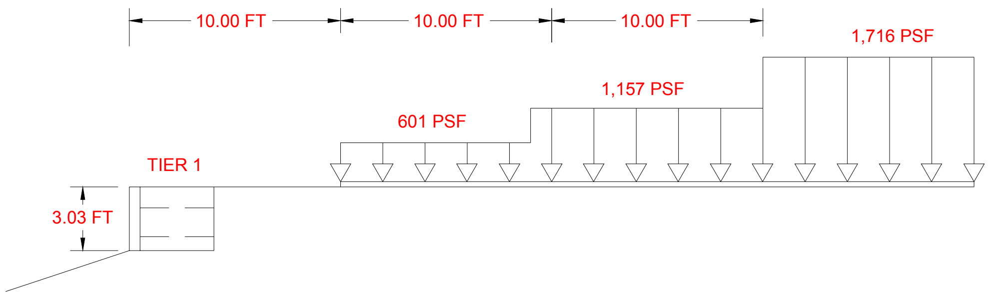

Figure 5-19: Illustration of Tier 1 assuming Tiers 2, 3, and 4 are modeled as surcharge loads for analyses on the US 67 Bypass wall.

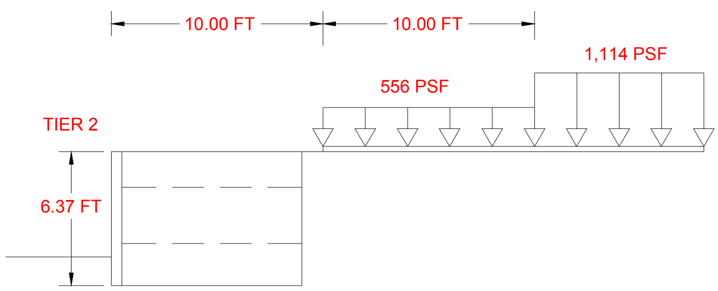

Figure 5-20: Illustration of Tier 2 assuming Tiers 3 and 4 are modeled as surcharge loads for analyses on the US 67 Bypass wall.

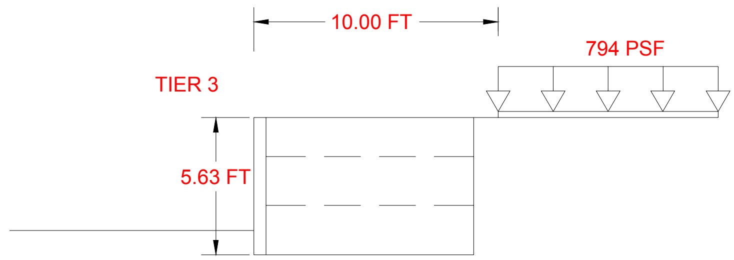

Figure 5-21: Illustration of Tier 3 assuming Tier 4 is modeled as a surcharge load for analyses on the US 67 Bypass wall.

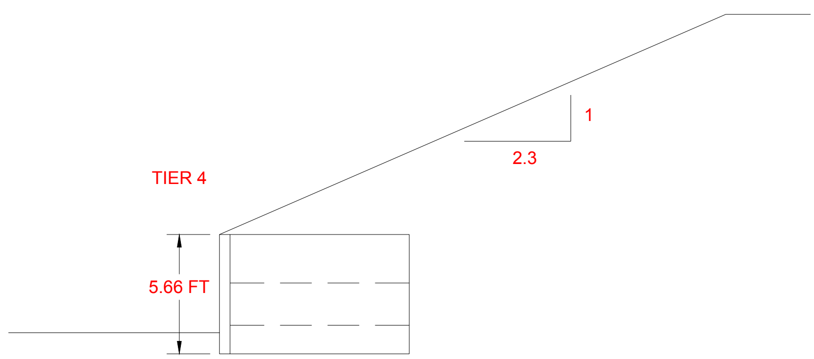

Figure 5-22: Illustration of Tier 4 as modeled in the MSEW 2.0 software for analyses on the US 67 Bypass wall.

Table 5-16: The calculated eccentricity of the resultant of the vertical forces for the US 67 Bypass wall.

Tier	Eccentricity, e (ft)			FHWA Maximum Eccentricity (L/6)	Satisfy FHWA Requirements? ($FS_{SL} \geq 1.5$)
	Single-Tier Analysis		Multi-Tier Analysis		
	Hand Calculation	MSEW 2.0	MSEW 2.0		
4	0.22	0.22	0.22	1.5	Yes
3	0.20	0.20	0.31	1.5	Yes
2	0.25	0.25	0.41	1.5	Yes
1	0.13	0.13	0.13	0.67	Yes

5.6.2.3 Bearing Capacity Failure

The safety against bearing capacity failure was evaluated by comparing the vertical stress (σ_v) at the base of the wall with the allowable bearing pressure (q_a) of the foundation soil. The vertical stress and the allowable bearing pressure were calculated by hand and by using the MSEW 2.0 software; these values are summarized in Table 5-17.

Table 5-17: The vertical stress and allowable bearing capacity for each tier on the US 67 Bypass wall.

Tier	Analysis	Method	Vertical Stress, σ_v (psf)	Allowable Bearing Capacity, q_a (psf)	Satisfy FHWA Requirements? ($\sigma_v \leq q_a$)
4	Single-tier	Hand calculation	1,393	4,606	Yes
		MSEW 2.0	1,393	4,606	Yes
	Multi-tier	MSEW 2.0	1,393	4,606	Yes
3	Single-tier	Hand calculation	706	4,629	Yes
		MSEW 2.0	706	4,629	Yes
	Multi-tier	MSEW 2.0	725	4,508	Yes
2	Single-tier	Hand calculation	809	4,570	Yes
		MSEW 2.0	809	4,570	Yes
	Multi-tier	MSEW 2.0	841	4,397	Yes
1	Single-tier	Hand calculation	388	899	Yes
		MSEW 2.0	388	899	Yes
	Multi-tier	MSEW 2.0	388	899	Yes

Each tier meets the FHWA requirements that the vertical stress (σ_v) be less than the allowable bearing pressure (q_a); however, the values do not agree in all cases. The differences are caused by the different assumptions for how the upper tiers are modeled between single-tier analysis and multi-tier analysis.

5.6.3 Internal Stability

Safety against rupture and pullout of the reinforcement was evaluated for each layer by hand and by using the MSEW 2.0 software.

5.6.3.1 Factor of Safety Against Rupture of the Reinforcement

The factors of safety against rupture (FS_R) are summarized in Table 5-18. The values in Table 5-18 all meet FHWA minimum requirements for the factor of safety being at least 1.5, but some differences exist between the values calculated by hand and those calculated by using the MSEW 2.0 software for single-tier analysis. The differences are caused by the assumption used in the MSEW 2.0 software to calculate the maximum tension (T_{MAX}) in the reinforcement using a trapezoidal distribution of stress. Also, the values are different because of the two different methods used to calculate the contribution of the sloping backfill (σ_2) by hand and by using the MSEW 2.0 software, as discussed in Chapter 3 (cf. Section 3.15).

Table 5-18: Summary of factors of safety against rupture of the reinforcement in the US 67 Bypass wall.

Tier	**Layer**	**Factor of Safety Against Rupture (FS_R)**			**Satisfy FHWA Requirements? ($FS_R \geq 1.5$)**
		Single-Tier Analysis		**Multi-Tier Analysis**	
		Hand Calculation	**MSEW 2.0**	**MSEW 2.0**	
4	8	2.10	3.56	3.56	Yes
	7	2.00	2.34	2.34	Yes
3	6	7.09	8.74	8.74	Yes
	5	2.70	2.37	2.37	Yes
2	4	5.71	6.41	6.41	Yes
	3	2.02	1.88	1.88	Yes
1	2	17.49	20.80	20.80	Yes
	1	9.25	9.31	9.31	Yes

5.6.3.2 Factor of Safety Against Pullout of the Reinforcement

Factors of safety against pullout (FS_{PO}) of the reinforcement are summarized in Table 5-19. All values meet FHWA minimum requirements, although some differences exist depending on the method of calculation. The differences between calculations by hand (single-tier analysis) and those performed using the MSEW 2.0 software (single-tier analysis and 2) are caused by the different methods of calculation for T_{MAX} and the contribution of the sloping backfill (σ_2), as discussed in the previous section. There are also differences between the two MSEW 2.0 analyses, which are caused by different assumptions made in the calculations—that is, whether each tier is considered an independent wall or whether the upper tiers are represented by a surcharge loads.

Table 5-19: Summary of factors of safety against rupture of the reinforcement in the US 67 Bypass wall.

Tier	Layer	Factor of Safety Against Pullout (FS_{PO})			Satisfy FHWA Requirements? ($FS_{PO} \geq 1.5$)
		Single-Tier Analysis		Multi-Tier Analysis	
		Hand Calculation	MSEW 2.0	MSEW 2.0	
4	8	6.03	12.35	12.35	Yes
	7	9.87	12.86	12.86	Yes
3	6	10.07	9.91	9.91	Yes
	5	9.98	6.98	7.26	Yes
2	4	8.20	7.34	7.34	Yes
	3	9.08	6.75	7.29	Yes
1	2	6.62	6.28	6.28	Yes
	1	10.36	8.33	8.33	Yes

5.6.4 Global Stability

Global stability was evaluated using both the MSEW 2.0 and UTEXAS4 software programs. The analyses with the MSEW 2.0 software were performed for four separate single-tier walls and with the weight of upper tiers modeled as strip loads. The minimum factor of safety for each tier is summarized in Table 5-20. The critical slip surface found for Tier 1 using the MSEW 2.0 software passed through the toe of the wall, and thus the effects of the embedment soil are neglected. For the remaining tiers, the critical slip surfaces passed below the toe of the wall, and thus, the effects of the embedment are included. The analyses performed with UTEXAS4 produced a minimum factor of safety for the wall of 1.33 (Figure 5-27). The minimum factor of safety calculated using the MSEW 2.0 and UTEXAS4 software programs are 1.46 and 1.33, respectively. Both calculated factors of safety exceed the minimum FHWA guidelines requirement for global stability of 1.3. The difference in the factors of safety calculated using the MSEW 2.0 and UTEXAS4 software programs is caused by the assumption used for the MSEW 2.0 software to modeled "equivalent" two-tier walls.

Table 5-20: Factors of safety computed from global stability analyses for each tier using the MSEW 2.0 software for US 67 Bypass.

Tier Modeled	Factor of Safety from Global Stability Analysis	Reference
1	2.44	Figure 5-23
2	1.75	Figure 5-24
3	1.86	Figure 5-25
4	1.46	Figure 5-26

5.7 Discussion

The purpose of the analyses presented in this chapter was to identify and quantify differences in the current methods used to analyze multi-tier MSE walls. It was found that the computed factors of safety can be different depending on the assumptions made, including known or unknown assumptions in software programs and known or unknown assumptions made by the designer. The following section identifies the known assumptions and discusses their effect on the external, internal, and/or global stability.

5.7.1 Horizontal Earth Pressure Forces

Evaluating safety against overturning and bearing capacity failure for multi-tier walls requires the designer or software to select a vertical plane where the horizontal earth pressure forces are calculated. The MSEW 2.0 software calculates the horizontal earth pressure forces on a vertical plane at the back of the reinforcement in the lower tier. The same assumption for the vertical plane was used by the designers of the multi-tier walls considered in this chapter, because each wall was analyzed using a version of the MesaPro software, which is computationally equivalent to the MSEW 2.0 software. This can be an issue if one were to analyze an entire multi-tier wall. As an example, consider the US 290 wall, which was discussed earlier and is shown in Figure 5-28. Applying the assumption used in the MSEW 2.0 software, the horizontal forces would be calculated for the vertical plane taken at the back edge of the reinforcement in Tier 1, as illustrated in Figure 5-29. Another assumption might be to include all the reinforcement in the rigid block, and thus, the vertical plane at the back edge of the reinforcement would be selected for Tier 4, as shown in Figure 5-30. In this case, the different assumptions produce different horizontal earth pressure forces, which would produce different factors of safety.

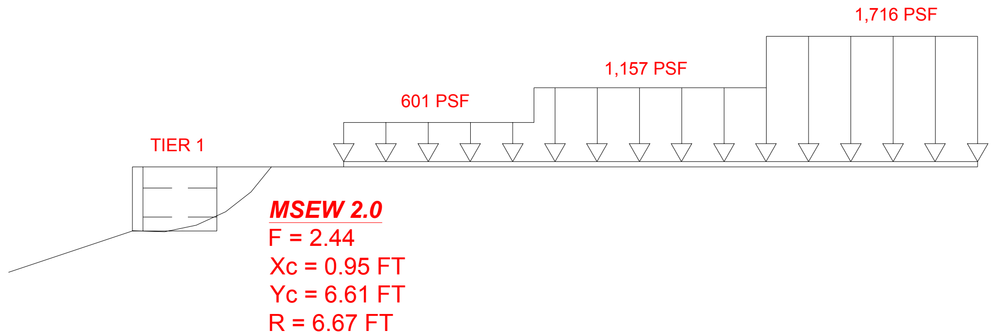

Figure 5-23: Slip surface and the minimum factor for Tier 1 assuming Tiers 2, 3, and 4 are modeled as surcharge loads for the US 67 Bypass wall..

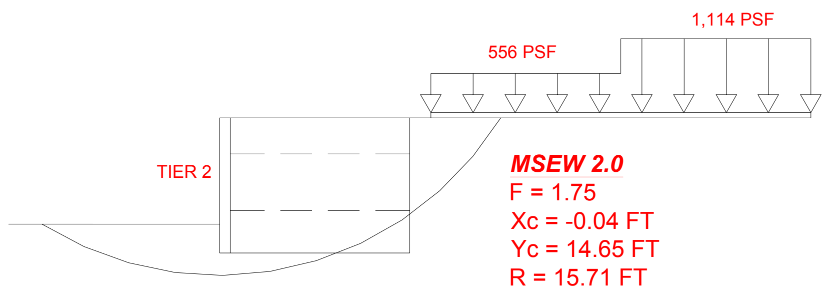

Figure 5-24: Slip surface and the minimum factor for Tier 2 assuming Tiers 3 and 4 are modeled as surcharge loads for the US 67 Bypass wall.

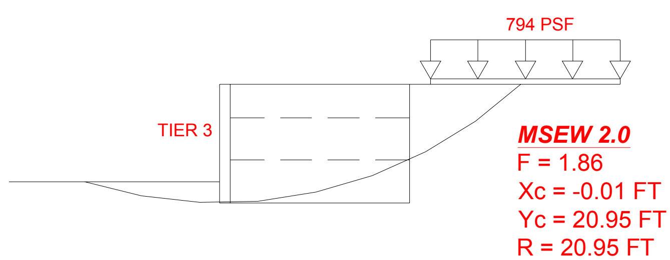

Figure 5-25: Slip surface and the minimum factor for Tier 3 assuming Tier 4 is modeled as a surcharge load for the US 67 Bypass wall.

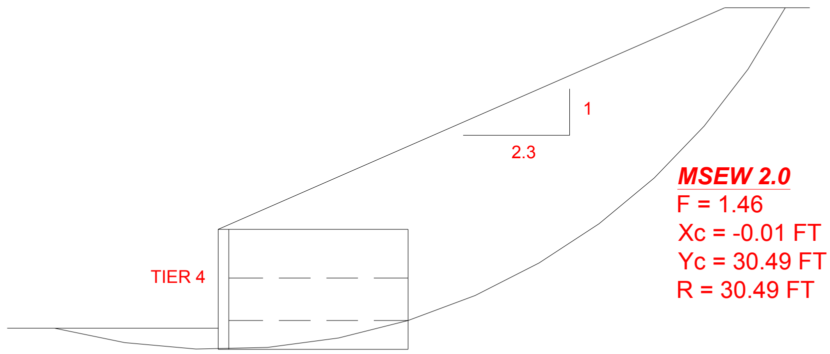

Figure 5-26: Slip surface and the minimum factor for Tier 4 assuming the surcharge load is neglected and the soil in front of the wall is included for the US 67 Bypass wall.

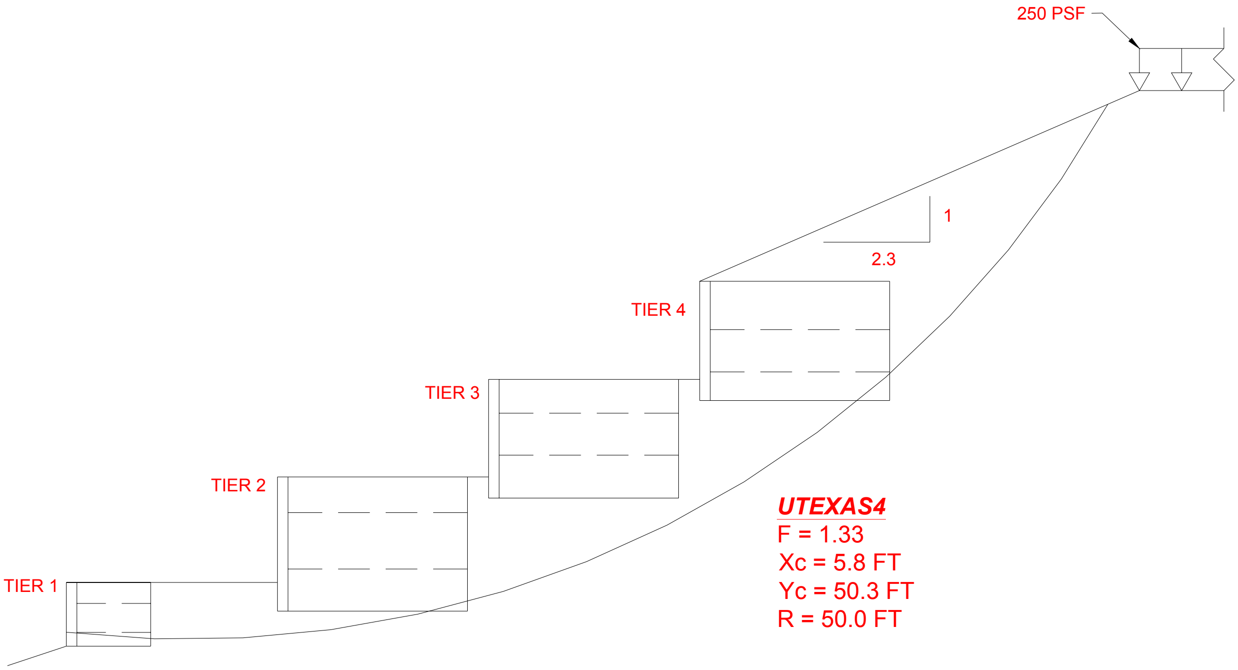

Figure 5-27: Slip surface and the minimum factor calculated using the UTEXAS4 software assuming the surcharge load is included and the soil in front of the wall is neglected for the US 67 Bypass wall.

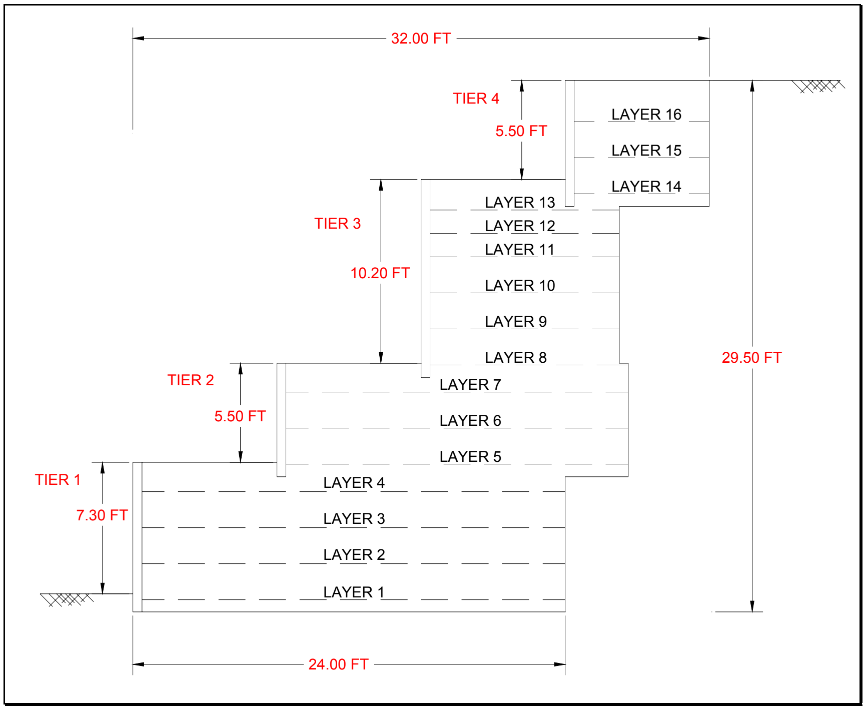

Figure 5-28: Cross-section of the US 290 wall.

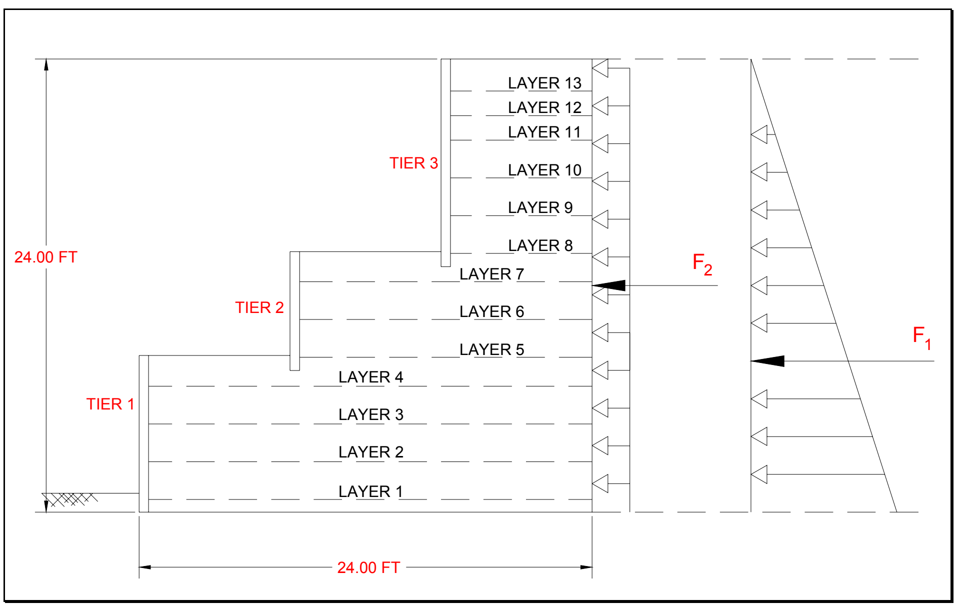

Figure 5-29: A vertical plane from the back edge of the reinforcement in Tier 1 used to calculate the horizontal earth pressure forces.

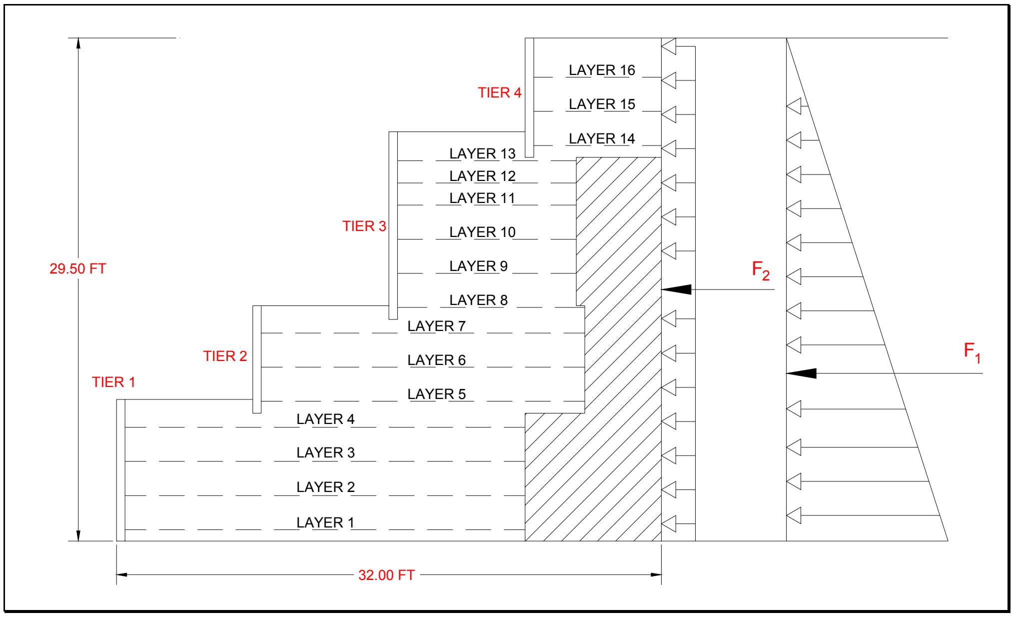

Figure 5-30: A vertical plane from the back edge of the reinforcement in Tier 4 used to calculate the horizontal earth pressure forces.

5.7.2 Criteria for Analyses of Multi-Tier Walls as Isolated Single Tiers

The FHWA 2001 criteria for determining when a multi-tier wall system can be analyzed as a series of single-tier walls does not seem to be valid. These criteria suggest that each tier on the US 67 Bypass wall could be analyzed using the procedures specified for single-tier walls. However, the global stability analyses performed with the UTEXAS4 software found the critical slip surface involves all four tiers of the wall (Figure 5-27). The FHWA guidelines failed to properly identify that the wall should be treated as a multi-tier wall.

5.7.3 Representation of Overlying Tiers as Surcharge Loads

Analyses performed for this study revealed differences in the calculated factors of safety when "isolated" single-tier walls were modeled with the upper tiers included as surcharge loads and when the upper tiers were neglected. To examine this issue further, consider the external and internal analyses performed on the US 67 Bypass wall, which the FHWA 2001 criteria specified could be analyzed as a series of "isolated" single-tier walls. These analyses were performed on a series of single-tier walls using the following two assumptions: (1) The upper tiers were neglected and (2) the upper tiers of the wall were included as surcharge loads.

External Stability

The results from the external stability analyses performed on the US 67 Bypass wall assuming that the upper tiers were represented by surcharge loads and assuming the upper tiers are neglected are summarized in Table 5-21. These values show that for Tiers 2 and 3 the stability against sliding, overturning, and bearing capacity failure is affected by modeling the upper tiers as surcharge loads.

Table 5-21: Results of external stability analyses calculated by hand assuming each tier is independent and by using the MSEW 2.0 software assuming the weight of the uppers act as a surcharge load for the US 67 Bypass wall.

Tier	**Factor of Safety**				**Eccentricity, e (ft)**	
	Sliding (FS_{SL})		**Bearing Capacity Failure (FS_{BC})**			
	Hand Calculation (1)	**MSEW 2.0 (2)**	**Hand Calculation (1)**	**MSEW 2.0 (2)**	**Hand Calculation (1)**	**MSEW 2.0 (2)**
4	8.27	8.27	8.26	8.26	0.22	0.22
3	5.54	3.20	16.40	15.54	0.20	0.31
2	4.89	2.85	14.12	13.07	0.25	0.41
1	4.57	4.57	5.80	5.80	0.13	0.13

(1) – Surcharge is neglected.
(2) – Surcharge is included.

Internal Stability

The factors of safety for pullout of the reinforcement were different depending on whether they were calculated with the upper tier neglected or included as a surcharge. The results of the analyses performed on the US 67 Bypass wall are summarized in Table 5-22. The values in Table 5-22 show that the factors of safety for pullout are affected by modeling the upper tiers as surcharge loads, but they are affected by only a small amount (≤ 8 percent) and only for Layers 3 and 5. The factors of safety against rupture of the reinforcement were not affected and, thus, are not shown.

Table 5-22: The factor of safety against pullout of the reinforcement assuming walls are independent and that upper tiers act as a surcharge on the US 67 Bypass wall.

Tier	Layer	Factor of Safety Against Rupture (FS_R)		Difference (%) (A) vs (B)
		MSEW 2.0 (Surcharge Neglected) (A)	MSEW 2.0 (Surcharge Included) (B)	
4	8	12.35	12.35	0.0
	7	12.86	12.86	0.0
3	6	9.91	9.91	0.0
	5	6.98	7.26	4.0
2	4	7.34	7.34	0.0
	3	6.75	7.29	8.0
1	2	6.28	6.28	0.0
	1	8.33	8.33	0.0

Global Stability

Factors of safety for global stability are different depending on whether they are calculated using the assumption that overlying tiers are included as a surcharge or neglected. The factors of safety calculated using the MSEW 2.0 software for analysis of each tier of the US 67 Bypass wall are summarized in Table 5-23. The differences range from 22 to 25 percent and affect only Tiers 2 and 3.

Table 5-23: Factors of safety for global stability assuming the upper tiers are applied as a surcharge and that the upper tiers are neglected for US 67 Bypass wall.

Tiers Modeled	Factor of Safety		Difference (%) (A) vs (B)
	Surcharge Neglected (A)	Surcharge Included (B)	
1	2.44	2.44	0.0
2	2.26	1.75	22.6
3	2.47	1.86	24.7
4	1.46	1.46	0.0

5.7.4 "Equivalent" Two-Tier Wall Model

"Equivalent" two-tier walls were used to evaluate stability of multi-tier walls for two reasons: (1) The MSEW 2.0 software could model only one- or two-tier walls, and (2) the FHWA guidelines address only one- and two-tier walls. Issues pertaining to modeling multi-tier walls as a series of "equivalent" two-tier walls apply to the US 290 wall only. The Socorro Bridge wall is a two-tier wall and, thus, modeling it as a two-tier wall is appropriate. The US 67 Bypass multi-tier wall was analyzed as a series of single-tier walls with the upper tiers represented as surcharges, rather than using equivalent two-tier walls.

Minimum factors of safety calculated using "equivalent" two-tier walls with surcharges produced different factors of safety and critical slip surfaces than those calculated when the entire wall geometry was modeled. The minimum factors of safety calculated for the US 290 wall using the MSEW 2.0 software to model "equivalent" two-tier walls and those calculated by the UTEXAS software to model the entire four-tier wall are 1.42 and 1.32, respectively, as shown in Table 5-24. Modeling multi-tier walls as "equivalent" two-tier walls produce minimum factors of safety different from those produced when the entire multi-tier wall is modeled. The difference is small (approximately 7 percent), but modeling "equivalent" two-tier walls is not conservative. Also, modeling as "equivalent" two-tier walls did not yield the proper location for the critical slip surface, as shown previously in Figures 5-15 and 5-16. Analyzing a multi-tier wall by modeling it as a series of "equivalent" two-tier walls can provide a rough estimate of the minimum factor of safety but not the critical slip surface.

Table 5-24: The minimum factors of safety calculated by using the MSEW 2.0 software assuming "equivalent" two-tier walls and by using the UTEXAS4 software for the entire four-tier wall.

Software	Tiers Modeled	Factor of Safety	Reference
MSEW 2.0	1 and 2	1.50	Figure 5-13
	2 and 3	1.45	Figure 5-14
	3 and 4	1.42	Figure 5-15
UTEXAS4	Entire wall	1.32	Figure 5-16

5.7.5 Assumptions for Calculating Maximum Tension (T_{MAX}) in the Reinforcement

The analyses performed on multi-tier walls revealed differences in the computed maximum tension (T_{MAX}) in the reinforcement caused by the method used by the MSEW 2.0 software to calculate the maximum tension (T_{MAX}) from a trapezoidal distribution of stress, as discussed in Chapter 3 (c. Section 3.15). The values for maximum tension (T_{MAX}) in the reinforcement calculated by hand and those calculated using the MSEW 2.0 software for the each wall are summarized in Tables 5-25, 5-26, and 5-27. The maximum tension (T_{MAX}) calculated by hand and that calculated using the MSEW 2.0 software differ by as much as 550 percent. The largest differences are found at the top of each tier and decrease with depth into the tier.

Table 5-25: The maximum reinforcement tension (T_{MAX}) calculated by hand and by using the MSEW 2.0 software for each layer of reinforcement in the Socorro Bridge wall.

Tier	Layer	Elevation (ft)	T_{MAX} (lb) Hand Calculation (A)	T_{MAX} (lb) MSEW 2.0 (B)	Difference (%) (A) vs (B)
2 (top)	10	18.34	278	277	0.4
	9	16.31	416	413	0.5
	8	14.34	820	869	6.0
1 (bottom)	7	11.35	67	438	550.8
	6	9.32	205	411	100.6
	5	7.35	546	691	26.6
	4	5.35	665	679	2.2
	3	4.04	593	612	3.2
	2	2.69	662	681	2.9
	1	1.35	1,086	1,136	4.7

Table 5-26: The maximum reinforcement tension (T_{MAX}) calculated by hand and by using the MSEW 2.0 software for each layer of reinforcement in the US 290 wall.

Tier	Layer	Elevation (ft)	T_{MAX} (lb) Hand Calculation (A)	T_{MAX} (lb) MSEW 2.0 (B)	Difference (%) (A) vs (B)
4 (top)	16	27.53	289	252	14.9
	15	25.53	285	294	3.2
	14	23.53	399	390	2.4
3	13	22.63	56	324	82.6
	12	21.33	144	351	58.9
	11	20.03	544	657	17.2
	10	18.03	733	793	7.5
	9	16.03	869	928	6.4
	8	14.03	839	879	4.6
2	7	12.53	145	608	76.1
	6	10.53	246	246	0.0
	5	8.53	377	344	9.7
1 (bottom)	4	6.67	145	398	63.5
	3	4.67	246	311	20.7
	2	2.67	511	511	0.0
	1	0.67	540	530	1.8

Table 5-27: The maximum reinforcement tension (T_{MAX}) calculated by hand and by using the MSEW 2.0 software for each layer of reinforcement in the US 67 Bypass wall.

Tier	**Layer**	**Elevation (ft)**	**T_{MAX} (lb)**		**Difference (%) (A) vs (B)**
			Hand Calculation (A)	**MSEW 2.0 (B)**	
4 (top)	8	15.03	477	281	69.6
	7	13.03	501	428	17.0
3	6	11.07	141	115	23.1
	5	9.07	370	423	12.5
2	4	6.34	175	156	12.0
	3	3.67	494	532	7.1
1 (bottom)	2	2.03	57	48	18.7
	1	0.66	108	108	0.5

5.7.6 Calculating the Pullout Resistance (P_r) of the Reinforcement

Pullout resistances (P_r) calculated by hand and by using the MSEW 2.0 software are summarized in Table 5-28 and 5-29 for the Socorro Bridge and US 290 walls, respectively. The different pullout resistances (P_r) are caused by the different depths (Z_P) used in the calculations by hand and by using the MSEW 2.0 software. The different assumptions used to measure the pullout depth (Zp) are discussed in detail in Chapter 3 and are illustrated in Figures 5-31 and 5-32. The pullout resistances calculated by hand and by using the MSEW 2.0 software differ by as much as 897 percent. The largest differences are found at the top of each tier and decrease for layers of reinforcement deeper in the tier. This issue does not affect the US 67 Bypass wall because it was analyzed as a series of single-tier walls.

Table 5-28: The pullout resistance (P_r) calculated by hand and by using the MSEW 2.0 software for each layer of reinforcement in the Socorro Bridge wall.

Tier	Layer	Elevation (ft)	Pullout Resistance, P_r (lb)		Difference (%) (A) vs (B)
			Hand Calculation (A)	MSEW 2.0 (B)	
2	10	18.34	1,556	981	36.9
	9	16.31	3,115	2,755	11.5
	8	14.34	4,878	4,785	1.9
1	7	11.35	547	4,801	778.0
	6	9.32	1,904	6,714	252.6
	5	7.35	3,526	8,861	151.3
	4	5.35	5,478	11,358	107.4
	3	4.04	9,273	17,613	90.0
	2	2.69	15,271	27,030	77.0
	1	1.35	18,406	30,831	67.5

Table 5-29: The pullout resistance (P_r) calculated by hand and by using the MSEW 2.0 software for each layer of reinforcement in the US 290 wall.

Tier	**Layer**	**Elevation (ft)**	**Pullout Resistance, P_r (lb)**		**Difference (%) (A) vs (B)**
			Hand Calculation (A)	**MSEW 2.0 (B)**	
4	16	27.53	930	930	0.0
	15	25.53	1,892	1,892	0.0
	14	23.53	3,682	3,680	0.1
3	13	22.63	552	1,727	212.9
	12	21.33	1,446	2,620	81.2
	11	20.03	3,135	4,304	37.3
	10	18.03	5,236	6,405	22.3
	9	16.03	7,751	8,921	15.1
	8	14.03	10,679	11,842	10.9
2	7	12.53	2,694	26,860	897.0
	6	10.53	6,375	32,205	405.2
	5	8.53	10,469	37,846	261.5
1	4	6.67	3,238	25,892	699.6
	3	4.67	7,586	32,169	324.0
	2	2.67	12,348	38,976	215.7
	1	0.67	17,522	46,238	163.9

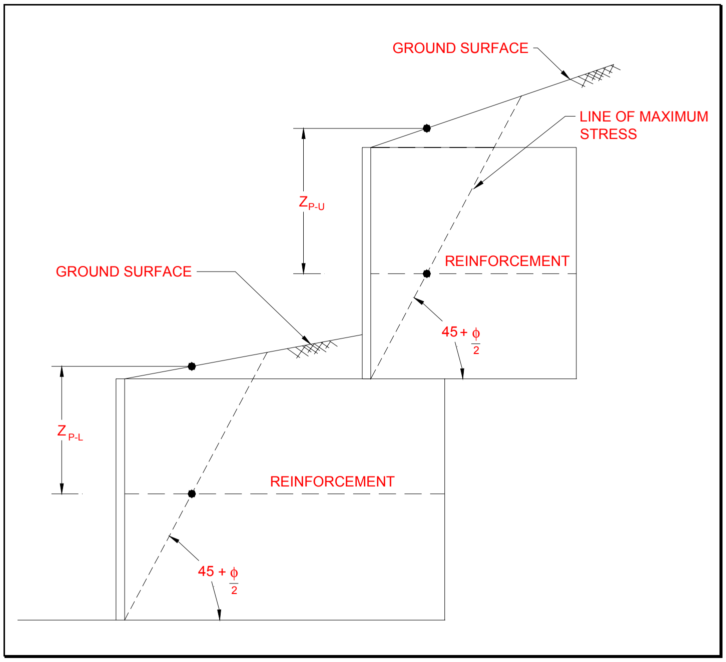

Figure 5-31: The depth (Z_P) used to calculate the pullout resistance of the reinforcement for multi-tier walls using the FHWA guidelines.

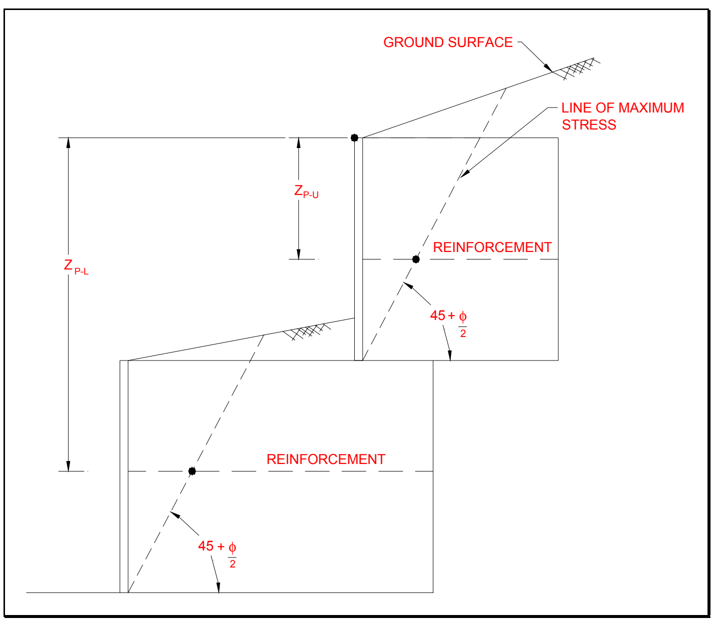

Figure 5-32: The depth (Z_P) used to calculate the pullout resistance of the reinforcement for multi-tier walls using the MSEW 2.0 software.

5.7.7 Influence of the Sloping Backfill

When the backfill behind the uppermost MSE wall is sloping, FHWA 2001 guidelines and the MSEW 2.0 software utilize different methods of treatment that result in different values for the maximum tension (T_{MAX}) in the reinforcement. The contribution from the sloping backfill (σ_2) on the vertical stress of the reinforcement is calculated in the MSEW 2.0 software using a height, h_s (Figure 3-12), that depends on the location of the line of maximum stress, whereas the FHWA 2001 guidelines specify the height should be calculated as the length of the reinforcement multiplied by the tangent of the slope angle, as discussed in Chapter 3 (cf. Section 3.15). The maximum tension (T_{MAX}) in the reinforcement calculated by hand and by using the MSEW 2.0 software for the top tiers of the Socorro Bridge and US 67 Bypass walls are summarized in Table 5-30. Both walls had sloping backfills. The difference in the values for force calculated by the two procedures (FHWA guidelines and MSEW 2.0 software) varies by as much as 70 percent; however, some of this difference is also caused by the method used by the MSEW 2.0 software to calculate T_{MAX} using a trapezoidal distribution of stress, as discussed in Section 5.7.4. The differences are larger for the US 67 Bypass wall because the slope (2.3H:1V) is greater than the slope (3H:1V) for the Socorro Bridge wall.

Table 5-30: The maximum tension (T_{MAX}) in the reinforcement for the Socorro Bridge Tier 2 and the US 67 Bypass Tier 4 calculated by hand and by using the MSEW 2.0 software.

Wall	Tier	Layer	Elevation (ft)	T_{MAX} (lb) Hand Calculation (A)	MSEW 2.0 (B)	Difference (%) (A) vs (B)
Socorro Bridge	2 (top)	10	18.34	278	277	0.4
		9	16.31	416	413	0.5
		8	14.34	820	869	6.0
US 67 Bypass	4 (top)	8	15.03	477	281	69.6
		7	13.03	501	428	17.0

5.8 Summary and Conclusions

The review and analyses of the designs for the Socorro Bridge, US 290, and US 67 Bypass walls yielded several important conclusions.

The FHWA criterion shown in Equation 5-3 for multi-tier walls analyzed as a composite system rather than as isolated single-tier walls is apparently valid.

$$\frac{H_L + H_U}{20} < D \leq H_L \cdot \tan(90° - \varphi_r) \tag{5-3}$$

An example is the Socorro Bridge wall, which satisfies the FHWA criteria to be analyzed as a two-tier wall. The external, internal, and global stability analyses performed indicate that the wall should behave as a two-tier wall.

- The FHWA criterion shown in Equation 5-1, which is used to determine whether a multi-tier wall should be analyzed as a series of independent single-tier walls, is apparently not valid.

$$D > H_L \cdot \tan(90° - \varphi_r) \qquad (5\text{-}1)$$

As an example, the US 67 Bypass wall satisfied the FHWA criteria for being analyzed as a series of single-tier walls, but the external, internal, and global stability analyses showed an influence from upper tiers.

- The MSEW 2.0 software follows the FHWA guidelines to determine whether a multi-tier wall can be analyzed as a series of independent single-tier walls. If a wall satisfies the criteria for analysis as independent single-tier wall (Equation 5-1), the MSEW 2.0 software will allow only one tier to be input at a time. As illustrated in the case of the Socorro Bridge wall, this is probably not a correct model of the wall system.
- FHWA guidelines provide no guidance for multi-tier walls with more than two tiers. Walls with more than two tiers were analyzed in this study as a series of "equivalent" two-tier walls. However, this is an approximation and does not adequately identify the critical slip surface. On the basis of the analyses of the US 290 wall, this approach can overestimate the minimum factor of safety for global stability by about 10 percent and, thus, is not conservative.
- The MSEW 2.0 software requires that walls with more than two tiers be modeled as a series of "equivalent" two-tier walls. Thus, analyses with this software will reflect the errors associated with the "equivalent" two-tier approximation.
- FHWA guidelines provide no guidance on how the depth used for calculating overburden stress for pullout resistance (P_r) is to be determined for multi-tier walls. Hand calculations of pullout resistance were performed using the depth below the top of the wall in which the reinforcement exists, as specified by the FHWA guidelines for single-tier walls and shown in Figure 5-31. The MSEW 2.0 software uses the depth from the top of the facing of the upper tier, as illustrated in Figure 5-32. The differences in the pullout resistance calculated by hand and that calculated by using the MSEW 2.0 software can be as much as 900 percent.
- There are two methods employed to calculate the vertical stress caused by a sloping backfill. The MSEW 2.0 software calculates the contribution of the sloping backfill (σ_2) on the vertical stress acting on the reinforcement using the height, h_s (Figure 3-12), whereas the FHWA 2001 guidelines specify the height should be calculated as the length of the reinforcement multiplied by the tangent of the slope angle, as discussed in Chapter 3 (cf. Section 3.15). The difference in the maximum tension (T_{MAX}) in the reinforcement caused by the different methods can be as much as 70 percent, as found in the analyses performed on the Socorro Bridge and US 67 Bypass walls.
- Two different methods are employed to calculate the maximum tension (T_{MAX}) in the reinforcement. The MSEW 2.0 software calculates the maximum tension (T_{MAX}) in the reinforcement from a trapezoidal distribution of stress. FHWA guidelines specify that the maximum tension should be calculated at the layer of reinforcement. The differences in the calculated maximum tension in the

reinforcement be as much as 550 percent, according to the analyses conducted for this study.

- Global stability analyses can be performed either including or neglecting the uniform (traffic) surcharge. Calculations using the MSEW 2.0 software automatically neglect the uniform (traffic) surcharge load, whereas calculations using the UTEXAS4 software can be performed either with or without the surcharge. The minimum factor of safety computed using the MSEW 2.0 software on the Socorro Bridge wall was 1.29. The minimum factor of safety calculated using the UTEXAS4 software was 1.18 when the uniform (traffic) surcharge was included and 1.30 when the surcharge was neglected. For this case, factors of safety with the surcharge neglected are about 10 percent larger than when the surcharge is included.

The review of existing multi-tier wall designs helped to identify the different assumptions and methods used in design. Several issues found to affect single-tier walls also produced differences in calculated factors of safety for multi-tier walls. One issue is the effect of wall embedment on the factor of safety for global stability. In general, wall embedment is probably an issue for multi-tier walls, although it was not an issue for the specific walls discussed in this chapter. The multi-tier walls reviewed either were not embedded into the foundation soil (Socorro Bridge) or the presence or absence of embedment did not affect the minimum factor of safety (US 290 and Socorro Bridge walls). On the basis of the analyses presented in this chapter, a stability analysis in which the entire multi-tier wall system is modeled is the best way to evaluate global stability.

6 Summary, Conclusions, and Recommendations for Future Work

6.1 Summary and Conclusions

The purpose of this research was to identify assumptions that lead to differences in factors of safety calculated for both single- and multi-tier mechanically stabilized earth (MSE) walls. The research involved a detailed review of current design guidelines and the application of the guidelines to the analyses of single- and multi-tier MSE walls.

In Chapter 2, current Federal Highway Administration (FHWA) and American Association of State Highway and Transportation Officials (AASHTO) guidelines were reviewed, and the current design practices were summarized. The analysis procedures for external, internal, and global stability specified by the FHWA and AASHTO guidelines were evaluated. The procedures for evaluating the external and internal stability of MSE walls are well defined, but the procedures for global stability are not as well defined.

In Chapter 3, the different assumptions and variables that affect the computed factors of safety were identified. Each of the different assumptions and variables were described and discussed. The assumptions pertaining to MSE wall design were found to produce potentially large differences in the computed factors of safety. In some cases, the different assumptions are documented, whereas in other cases they are not. The assumptions used for design were found to vary among the design guidelines, the designer, and the software used.

In Chapter 4, analyses were performed on actual single-tier walls that were designed and built for the Texas Department of Transportation (TxDOT). Calculations were performed by hand and by using the MSEW 2.0 software; these calculations were compared with the calculations reported by the original designer. The investigation of existing walls using current design guidelines helped identify a number of the conditions and assumptions that cause differences among computed results. The external, internal, and global stability analyses of the single-tier walls helped to quantify the effect of each assumption on the calculated factors of safety. Numerous assumptions were found to affect the factors of safety calculated for internal stability, such as the method used by the MSEW 2.0 software to calculate the maximum tension in the reinforcement from a trapezoidal distribution of stress, as compared with the FHWA guidelines that state the maximum tension should be calculated at the level of reinforcement. The different methods used by the FHWA 2001 and AASHTO 1996 guidelines to calculate the lateral earth pressure coefficient (K_r) produced different factors of safety. Neglecting the surcharge load for internal and global stability can produce larger factors of safety. The factors of safety for global stability are affected by whether the soil in front of the wall is included or neglected.

In Chapter 5, analyses were presented for multi-tier walls that were designed by private engineering firms and built for TxDOT. The factors of safety reported by the original designer were compared with those calculated in accordance with current design guidelines by hand and by using the MSEW 2.0 software. On the basis of the investigation of these walls, it was found that the criteria provided by the current guidelines for determining whether the wall should be analyzed as a single- or multi-tier wall do not always work. Also, the design guidelines provide information on only one- and two-tier walls, which means that for walls with more than two tiers the designer is required to make additional assumptions for design. Many of the assumptions found to affect single-tier wall design also affected the design of multi-tier walls. The method used by the MSEW 2.0 software to calculate the effects of a sloping backfill on the vertical stress produces differences in the calculated factors of safety for rupture and pullout. The method used by the MSEW 2.0 software to calculate the maximum tension in the reinforcement from a trapezoidal distribution of stress affects both single- and multi-tier wall design. Other assumptions were unique to multi-tier wall design—for example, the method used by the MSEW 2.0 software to select the depth used in calculating the pullout resistance of the reinforcement is different from the depth used in the FHWA guidelines.

6.2 Recommendations for Future Work

On the basis of the work presented in this study, three recommendations for future work can be made. First, design guidelines for MSE walls should better define the procedures for global stability analysis of single- and multi-tier walls. Second, additional work should be completed to address walls with more than two tiers, like those covered in Chapter 5. Finally, there needs to be careful and thorough documentation of software used for MSE wall design. Hopefully, the work completed for this study will facilitate future work.

APPENDIX A: Representative Design Calculations for the US 183 Wall

The US 183 wall is a single-tier mechanically stabilized earth (MSE) wall located in Travis County, Texas. The following calculations were performed in accordance with the Federal Highway Administration (2001) guidelines to evaluate the external and internal stability of the wall.

A.1 Material Design Properties

The section of the US 183 wall analyzed is shown in Figure A-1 with the soil properties and approximate dimensions. A detailed discussion of the wall geometry and the reinforcement used in the US 183 wall is available in Chapter 4 (cf. Section 4.3.1).

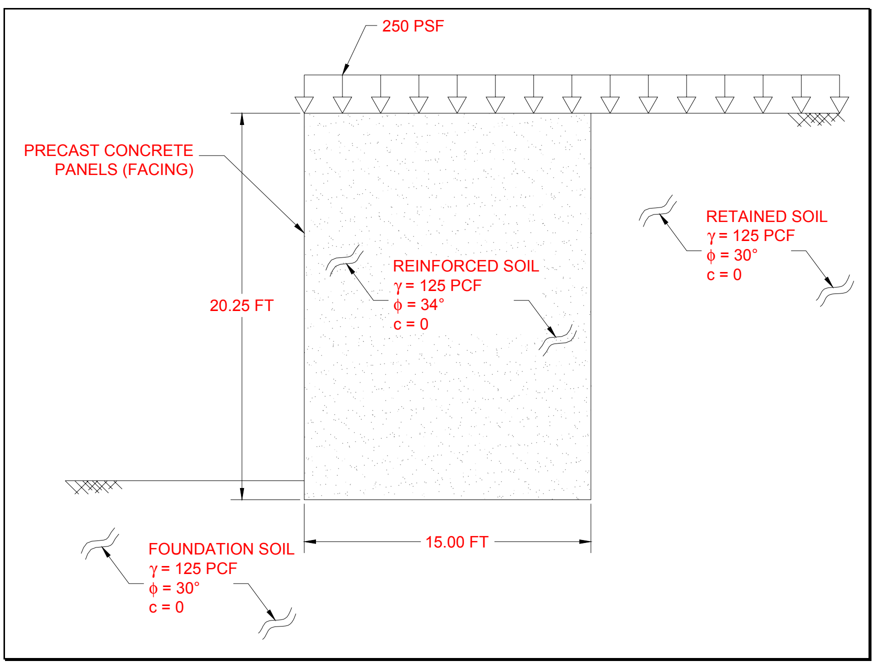

Figure A-1: Cross-section with typical soil design properties of MSE wall on US 183.

A.2 External Stability Calculations

This section presents calculations for sliding, overturning (eccentricity), and bearing capacity.

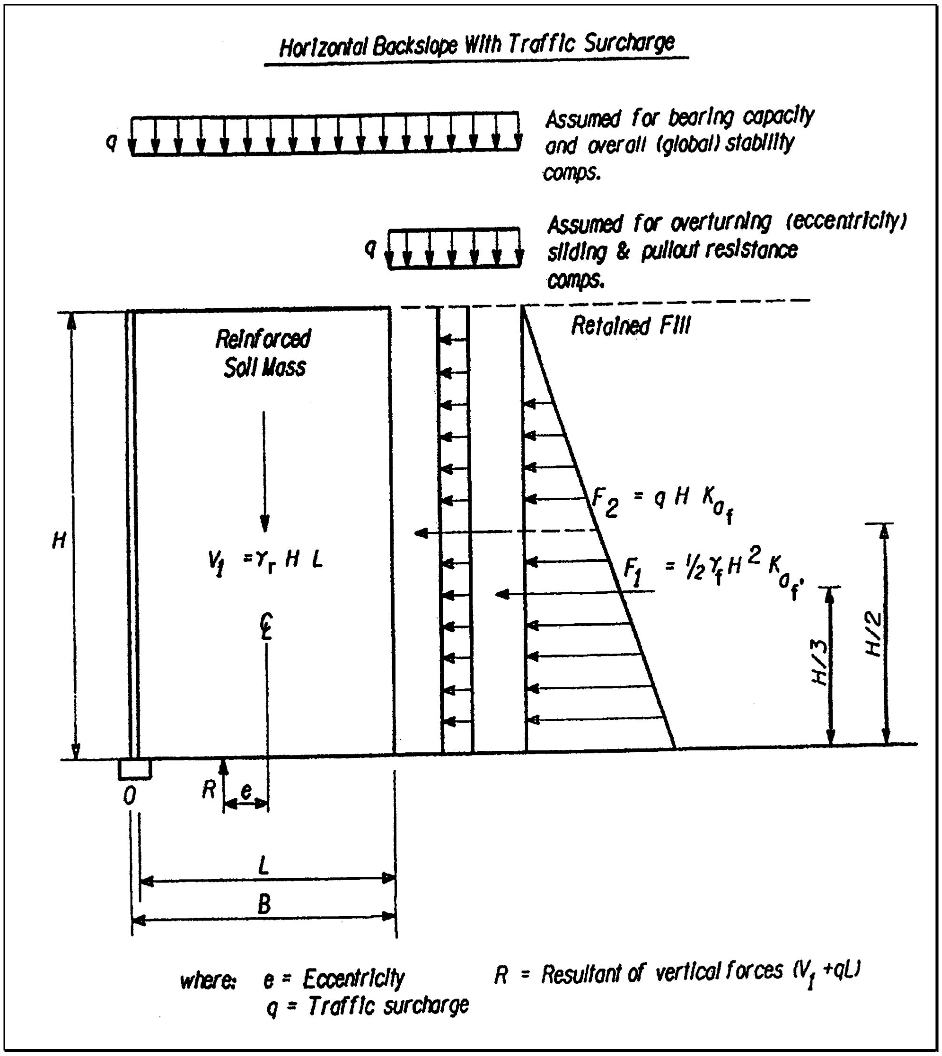

Figure A-2: Design forces used in external stability analysis on single-tier MSE wall with traffic surcharge (after FHWA, 2001).

A.2.1 Sliding Stability Calculations

The following calculations for sliding were performed using the forces illustrated shown in Figure A-2.

Horizontal resisting forces (P_R)

- $P_R = V_1 \cdot \mu = (37{,}968.8\,\tfrac{lb}{ft}) \cdot (0.5774) = 21{,}921.3\,lb$

- $V_1 = \gamma_R \cdot H \cdot L = (125\,pcf) \cdot (20.25\,ft) \cdot (15.0\,ft) = 37{,}968.8\,lb$
- $\mu = \min[\tan\varphi_f, \tan\varphi_r] = \tan\phi_f = \tan 30° = 0.5774$

where

P_R = the sum of the horizontal resisting forces,

V_1 = force caused by the weight of reinforced soil mass (Figure A-2),

μ = coefficient of friction,

γ_r = unit weight of reinforced soil,

H = total height of the MSE wall (Figure A-2),

L = length of the reinforcement (Figure A-2),

φ_f = angle of internal friction of foundation soil = 30°, and

φ_r = angle of internal friction of reinforced soil = 34°.

Horizontal driving forces (P_d)

- $P_d = F_1 + F_2 = 8{,}543.0\,lb + 1{,}687.5\,lb = 10{,}230.5\,lb$
- $F_1 = \frac{1}{2} \cdot \gamma_f \cdot H^2 \cdot K_{af} = \left(\frac{1}{2}\right) \cdot (125\,pcf) \cdot (20.25\,ft)^2 \cdot (0.333) = 8{,}543.0\,lb$
- $K_{af} = \tan^2\left(45° - \frac{\varphi_f}{2}\right) = \tan^2\left(45° - \frac{30°}{2}\right) = 0.333$
- $F_2 = q \cdot H \cdot K_{af} = (250\,psf) \cdot (20.25\,ft) \cdot (0.333) = 1{,}687.5\,lb$

where

P_d = the sum of the horizontal driving forces,

F_1 = the horizontal force caused by weight of retained soil (Figure A-2),

F_2 = the horizontal force caused by surcharge load(s), (Figure A-2),

γ_f = unit weight of foundation soil,

H = total height of the MSE wall (Figure A-2),

K_{af} = active earth pressure coefficient for the retained soil,

φ_f = angle of internal friction of foundation soil (30°), and

q = uniform surcharge load (traffic load).

Factor of safety against sliding (FS_{SL})

- $FS_{SL} = \dfrac{\sum P_R}{\sum P_d} = \dfrac{21{,}921.3lb}{10{,}230.5lb} = \underline{\underline{2.14}}$

A.2.2 Eccentricity Calculation

The eccentricity (e) of the resultant of the vertical forces on the base of the wall is computed using the following procedure.

Sum moments about centerline of the base of the wall (counterclockwise is positive)

- $\sum M_{CL} = \left(F_1 \cdot \dfrac{H}{3}\right) + \left(F_2 \cdot \dfrac{H}{2}\right) - (R \cdot e) = 0$
- $R = V_1 + q \cdot L = 37{,}968.8lb + 250psf \cdot 15ft = 41{,}718.8lb$
- $e = \dfrac{\left(F_1 \cdot \dfrac{H}{3}\right) + \left(F_2 \cdot \dfrac{H}{2}\right)}{R} = \dfrac{\left(8{,}543.0lb \cdot \dfrac{20.25ft}{3}\right) + \left(1{,}687.5lb \cdot \dfrac{20.25ft}{2}\right)}{41{,}718.8lb}$
- $e = \underline{\underline{1.79ft}}$

where

F_1 = the horizontal force caused by the weight of retained soil (Figure A-2),

F_2 = the horizontal force caused by surcharge load(s) (Figure A-2),

H = total height of the MSE wall (Figure A-2),

R = resultant of vertical forces (Figure A-2), and

e = eccentricity—that is, the distance from the centerline of the wall to the resultant force (R) (Figure A-2).

A.2.3 Bearing Capacity Calculations

The following calculations are for the safety of the wall with respect to bearing capacity failure.

Bearing Pressures

- $q_{ult} = c_f \cdot N_c + 0.5 \cdot (L - 2e) \cdot \gamma_f \cdot N_\gamma$
- $q_a = (0 \cdot 30.14) + (0.5 \cdot (15\,ft - 2 \cdot 1.792\,ft) \cdot 125\,pcf \cdot 22.40) = 15{,}983\,psf$
- $q_a = \dfrac{q_{ult}}{FS_{BC}} = \dfrac{15{,}983\,psf}{2.5} = 6{,}393\,psf$
- $\sigma_v = \dfrac{V_1 + q \cdot L}{L - 2e} = \dfrac{(37{,}968.8\,lb) + (250\,psf \cdot 15\,ft)}{15\,ft - 2 \cdot (1.792\,ft)} = 3{,}654.3\,psf$
- $\underline{\underline{\sigma_v = 3{,}654\,psf \le q_a = 6{,}393\,psf}}$ $\therefore$ *Safe.*

where

σ_v = the vertical stress on the base of the wall,

q_a = allowable bearing pressure of the foundation soil,

FS_{BC} = factor of safety against bearing capacity failure (≥ 2.5),

c_f = cohesion of the foundation soil (0 psf),

N_c = bearing capacity factor (after Vesic 1973), and

N_γ = bearing capacity factor (after Vesic 1973).

Factor of Safety for Bearing Capacity

- $$FS_{BC} = \frac{q_{ult}}{\sigma_v} = \frac{15{,}983\,psf}{3{,}654.3\,psf} = \underline{\underline{4.37}}$$

A.3 Internal Stability Calculations

Calculations for the factor of safety against rupture and against pullout of the reinforcement are presented for Layer 6 of the reinforcement (Figure A-3). This layer is at a height of 13.53 ft. from the bottom of the reinforced soil zone and is 6.72 ft. from the top of the wall.

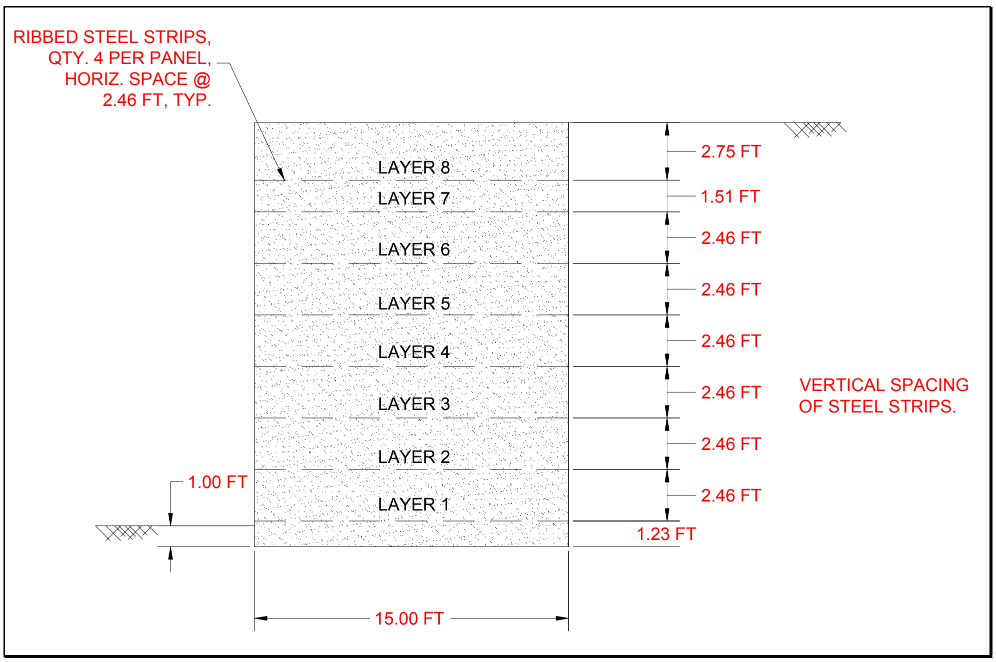

Figure A-3: Layout of the ribbed steel strip reinforcement in the US 183 wall.

A.3.1 Factor of Safety Against Rupture of the Reinforcement

Calculating the factor of safety against rupture (FS_R) of the reinforcement is a three-part process involving calculation of the vertical stress (σ_v) acting on the reinforcement, the horizontal stress (σ_H) acting on a plane perpendicular to the reinforcement, and finally, computing the maximum tension (T_{MAX}) in the reinforcement.

Vertical stress at the elevation of the reinforcement (σ_v)

- $\sigma_v = \gamma_R \cdot Z + \sigma_2 + q + \Delta\sigma_v = (125\,pcf)\cdot(6.72\,ft) + 0 + 250\,psf + 0 = 1{,}090\,psf$

where

γ_R = unit weight of reinforced soil (125 pcf),

Z = depth to the reinforcement layer (6.72 ft.),

σ_2 = vertical stress caused by soil backfill on top of wall (0 psf),

q = uniform surcharge load—that is, traffic load (250 psf), and

$\Delta\sigma_v$ = vertical stress caused by strip loads, footing loads, or point loads acting on the wall (0 psf).

Horizontal stress at level of reinforcement (σ_H)

- $\sigma_H = (K_r \cdot \sigma_v) + \Delta\sigma_h = (0.4322 \cdot 1{,}090\,psf) + 0 = 471.3\,psf$
- $K_r = K(z) = 1.598 \cdot K_a = 1.598 \cdot 0.2827 = 0.4322$
- $K_a = \tan^2\left(45 - \frac{\varphi_R}{2}\right) = \tan^2\left(45 - \frac{34°}{2}\right) = 0.2827$

where

$\Delta\sigma_h$ = horizontal stress caused by strip loads, footing loads, or point loads acting on the wall (0 psf),

K_r = stress variation coefficient (Figure A-4),

K_a = the active earth pressure coefficient for the retained soil, and

Φ_R = angle of internal friction of the reinforced soil (34°).

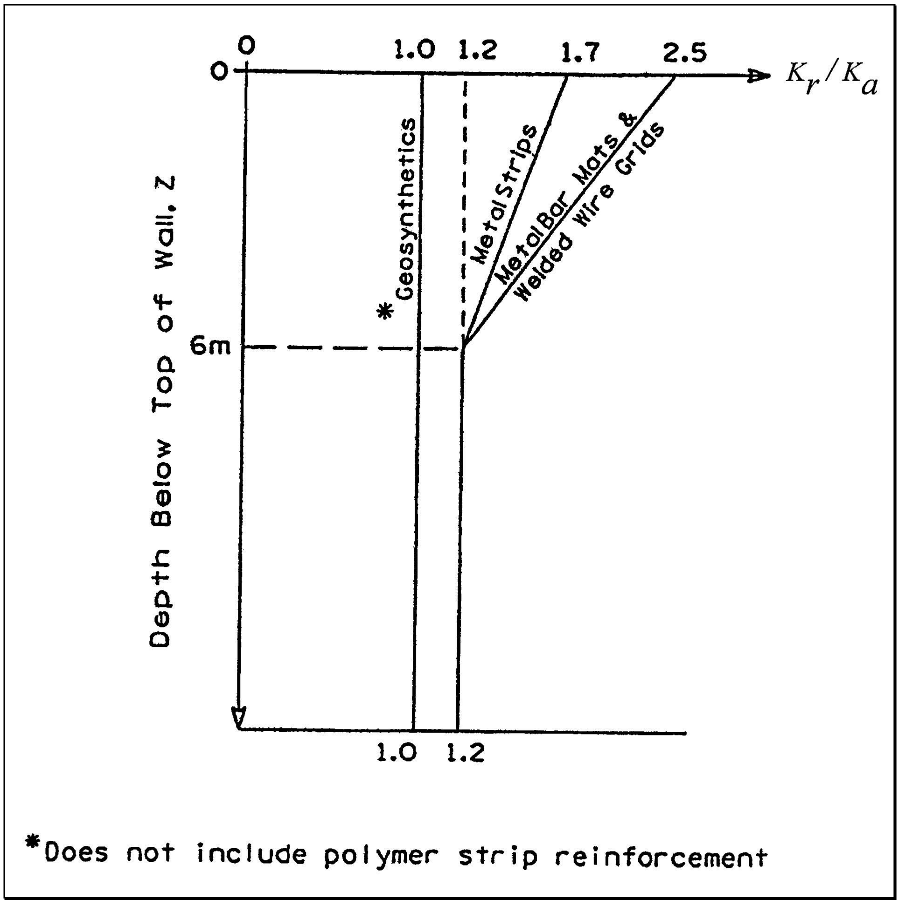

Figure A-4: Variation of stress ratio with depth in an MSE wall (after FHWA, 2001).

Maximum reinforcement tension (T_{MAX})

- $T_{\max} = \sigma_H \cdot S_v = 471.2 psf \cdot 2.46 ft = 1{,}159 lb$

where

S_v = vertical spacing of the reinforcement (2.46 ft.).

The units for maximum reinforcement tension (T_{MAX}) are a force per unit width of wall. Because this example is computed using US standard units, the units for T_{MAX} are pounds per foot width of wall.

Allowable design strength (T_a)

- $T_a = C_R\left(\frac{A_c \cdot F_y}{b}\right) = 0.55\left(\frac{0.20in^2 \cdot 65ksi}{1.968in}\right) \cdot \left(\frac{1000lb}{kip}\right) = 3{,}650lb$
- $A_c = b \cdot E_c = 1.968in \cdot 0.102in = 0.20in^2$
- $E_c = E_n - E_r = 0.157in - 0.055in = 0.102in$

where

C_R = safety coefficient (0.55 for steel strips),

A_c = cross-sectional area of reinforcement corrected for corrosion,

F_y = yield stress of steel (65 ksi),

b = width of the strips (1.968 in.),

E_c = thickness of the strip corrected for corrosion,

E_n = nominal strip thickness, and

E_r = reduction in strip thickness caused by corrosion

Notes

- The galvanization of the strips has provided a zinc coating of 3.4 mils or 86μm. Corrosion rates for the zinc are 0.6 mils/year per side for the first two years and 0.16 mils/year per side after two years. When the zinc is completely corroded, the residual carbon steel will corrode at a rate of 0.5 mils/year per side of the strip. On the basis of these rates, complete corrosion of galvanization with the minimum required thickness of 86 μm (3.4 mils) is estimated to occur during the first sixteen years, and a carbon steel thickness loss of 1.42 mm (0.055 in) would be anticipated over the remaining seventy-five years. Therefore, E_r = 0.055 in.

- The units for allowable design strength (T_a) are a force per unit width of reinforcement. Because this example is computed using US standard units, the units for T_a are pounds per inch width of steel strip.

Safety against rupture of reinforcement

- $T_a \cdot R_c = 3{,}650 lb \cdot \dfrac{1.968 in}{2.46 ft} = 2{,}921 lb$
- $R_c = \dfrac{b}{S_H} = \dfrac{1.968 in}{2.46 ft}$
- $T_a \cdot R_c = 2{,}921 lb \geq T_{MAX} = 1{,}159 lb \therefore Safe$

where

R_c = coverage ratio = (b/S_H),

b = the width of ribbed steel strip (1.968 in.), and

S_H = horizontal spacing between strips (2.46 ft.).

Factor of safety against rupture (FS_R)

- $T_{al} = T_{ult} = \dfrac{F_y \cdot A_c}{b} = \left(\dfrac{65 ksi \cdot 0.20 in^2}{1.968 in}\right) \cdot \left(\dfrac{1000 lb}{kip}\right) = 6{,}636.5 lb$
- $FS_R = \dfrac{T_{al} \cdot R_c}{T_{MAX}} = \dfrac{(6{,}636.5 lb) \cdot \left(\dfrac{1.968 in}{2.46 ft}\right)}{1{,}159.2 lb} = 4.58$

where

T_{al} = long-term design strength.

The factors of safety for each layer of reinforcement are summarized in Table 4-3 (cf. Section 4.3.3.1).

A.3.2 Factor of Safety against Pullout of the Reinforcement

Pullout resistance (P_r)

- $P_r = F^* \cdot \alpha \cdot \sigma_V \cdot L_e \cdot C = (1.548) \cdot (1.0) \cdot (840\,psf) \cdot (8.925\,ft) \cdot (2) = 23{,}210.7\,lb$

- $\dfrac{P_r \cdot R_c}{FS_{PO}} = \dfrac{23{,}210.7\,lb \cdot \left(\dfrac{1\,ft}{12\,in}\right)\left(\dfrac{1.968\,in}{2.46\,ft}\right)}{1.5} = 1{,}031\,lb$

- $T_{MAX} = 1{,}159\,psf \leq \dfrac{P_r \cdot R_c}{FS_{PO}} = 1{,}032\,lb$ $\therefore$ *Not Safe.*

where

P_r = pullout resistance of the reinforcement,

FS_{PO} = factor of safety against pullout (1.5),

L_e = the embedment or adherence length in the resisting zone behind the failure surface,

C = the effective unit perimeter of the reinforcement (C = 2 for strips, grids, and sheets),

F* = the pullout resistance factor, which is a function of passive and frictional resistance of the reinforcement,

α = a scale effect correction factor to account for a nonlinear stress reduction over the embedded length of highly extensible reinforcements, based on laboratory data (1.0 for metallic reinforcements), and

σ_v = the vertical stress at the soil–reinforcement interface.

The units for pullout resistance (P_r) are a force per unit width of reinforcement. Because this example is computed using US standard units, the units for P_r are pounds per inch width of steel strip.

Factor of safety against pullout (FS_{PO})

- $$FS_{PO} = \frac{P_r \cdot R_c}{T_{MAX}} = \frac{23{,}210.7lb \cdot \left(\frac{1ft}{12in}\right)\left(\frac{1.968in}{2.46ft}\right)}{1{,}159.3lb} = \frac{1{,}547.4lb}{1{,}159.3lb} = 1.33$$

The factors of safety for pullout are summarized for each layer of reinforcement in Table 4-4 (cf. Section 4.3.3.2).

APPENDIX B Representative Design Calculations for the Brown County Wall

The Brown County wall is a single-tier mechanically stabilized earth (MSE) wall located in Brown County, Texas. The following calculations were performed in accordance with the Federal Highway Administration (FHWA) 2001 guidelines to evaluate the external and internal stability of the wall.

B.1 Material Design Properties

The section of the Brown County wall analyzed is shown in Figure B-1 with the soil properties and approximate dimensions. A detailed discussion of the wall geometry and the reinforcement used in the Brown County wall is available in Chapter 4 (cf. Section 4.4.1).

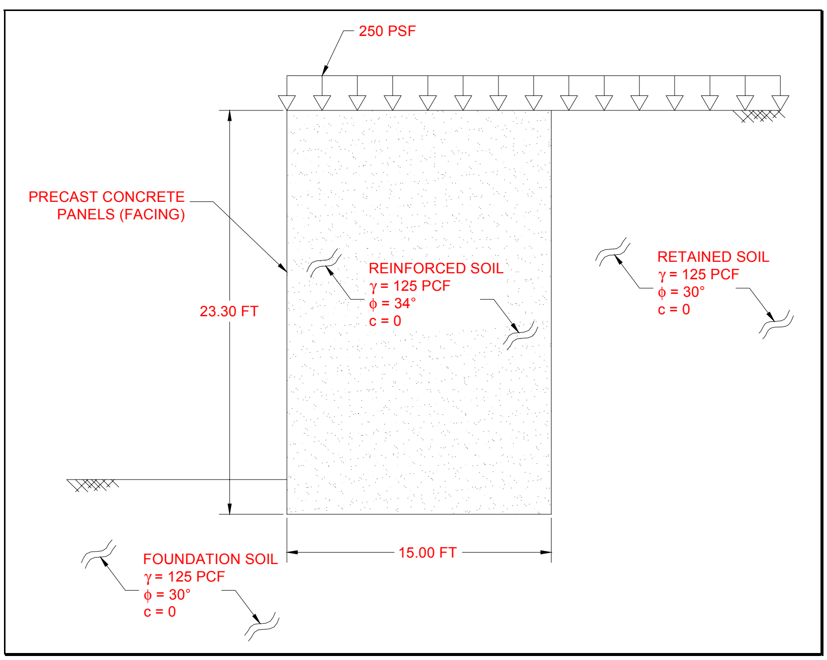

Figure B-1: Cross-section of the tallest section of the Brown County wall.

B.2 External Stability Calculations

This section presents calculations for sliding, overturning (eccentricity), and bearing capacity. FHWA 2001 guidelines specify that the calculations be performed using the forces illustrated in Figure 5-2.

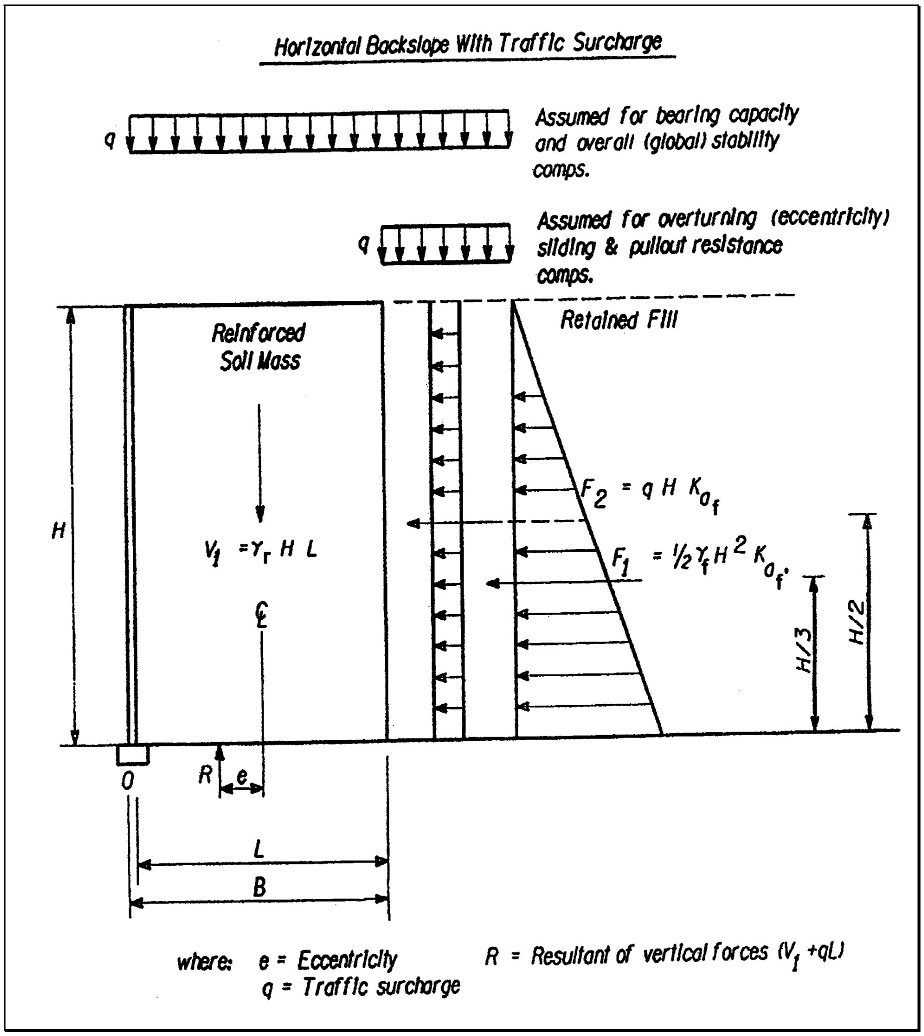

Figure B-2: Design forces used in external stability analysis on single-tier MSE wall with traffic surcharge (after FHWA, 2001).

B.2.1 Sliding Stability Calculations

The following calculations for sliding were performed using the forces illustrated in Figure B-2.

Horizontal resisting forces (P_R)

- $P_R = V_1 \cdot \mu = (43{,}687\,\tfrac{lb}{ft}) \cdot (0.5774) = 25{,}223 lb$

where

- $V_1 = \gamma_R \cdot H \cdot L = (125\,pcf) \cdot (23.3\,ft) \cdot (15.0\,ft) = 43{,}687 lb$
- $\mu = \min[\tan\varphi_f, \tan\varphi_r] = \tan\phi_f = \tan 30° = 0.5774$

P_R = the sum of the horizontal resisting forces,

V_1 = force caused by the weight of reinforced soil mass (Figure B-2),

μ = coefficient of friction,

γ_r = unit weight of reinforced soil,

H = total height of the MSE wall (Figure B-2),

L = length of the reinforcement (Figure B-2),

φ_f = angle of internal friction of foundation soil = 30°, and

φ_r = angle of internal friction of reinforced soil = 34°.

Horizontal driving forces (P_d)

- $P_d = F_1 + F_2 = 11{,}310 lb + 1{,}942 lb = 13{,}252 lb$

where

- $F_1 = \frac{1}{2} \cdot \gamma_f \cdot H^2 \cdot K_{af} = \left(\frac{1}{2}\right) \cdot (125\,pcf) \cdot (23.3\,ft)^2 \cdot (0.333) = 11{,}310 lb$
- $K_{af} = \tan^2\left(45° - \frac{\varphi_f}{2}\right) = \tan^2\left(45° - \frac{30°}{2}\right) = 0.333$

- $F_2 = q \cdot H \cdot K_{af} = (250\,psf) \cdot (23.3\,ft) \cdot (0.333) = 1{,}942\,lb$

P_d = the sum of the horizontal driving forces,

F_1 = the horizontal force caused by weight of retained soil (Figure B-2),

F_2 = the horizontal force caused by surcharge load(s) (Figure B-2),

γ_f = unit weight of foundation soil,

H = total height of the MSE wall (Figure B-2),

K_{af} = active earth pressure coefficient for the retained soil,

φ_f = angle of internal friction of foundation soil (30°), and

q = uniform surcharge load (traffic load) (Figure B-2).

Factor of safety against sliding (FS_{SL})

- $FS_{SL} = \dfrac{\sum P_R}{\sum P_d} = \dfrac{25{,}223\,lb}{13{,}252\,lb} = \underline{\underline{1.90}}$

B.2.2 Eccentricity Calculation

The eccentricity (e) of the resultant of the vertical forces on the base of the wall is computed using the following procedure.

Sum moments about centerline of the base of the wall (counterclockwise is positive)

- $\sum M_{CL} = \left(F_1 \cdot \frac{H}{3}\right) + \left(F_2 \cdot \frac{H}{2}\right) - (R \cdot e) = 0$
- $R = V_1 + q \cdot L = 43{,}687 lb + 250 psf \cdot 15 ft = 47{,}438 lb$
- $e = \frac{\left(F_1 \cdot \frac{H}{3}\right) + \left(F_2 \cdot \frac{H}{2}\right)}{R} = \frac{\left(11{,}310 lb \cdot \frac{23.3 ft}{3}\right) + \left(1{,}942 lb \cdot \frac{23.3 ft}{2}\right)}{47{,}438 lb}$
- $e = \underline{\underline{2.33 ft}}$

where

F$_1$ = the horizontal force caused by the weight of retained soil (Figure B-2),

F$_2$ = the horizontal force caused by surcharge load(s) (Figure B-2),

H = total height of the MSE wall (Figure B-2),

R = resultant of vertical forces (Figure B-2), and

e = eccentricity—that is, the distance from the centerline of the wall to the resultant force (R) (Figure B-2).

B.2.3 Bearing Capacity Calculations

The following calculations are for the safety of the wall with respect to bearing capacity failure

Bearing Pressures

- $q_{ult} = c_f \cdot N_c + 0.5 \cdot (L - 2 \cdot e) \cdot \gamma_f \cdot N_\gamma$
- $q_{ult} = 0.5 \cdot (15.0 ft - 2 \cdot 2.33 ft) \cdot 125 pcf \cdot 22.4 = 14{,}480 psf$
- $q_a = \frac{q_{ult}}{FS_{BC}} = \frac{14{,}480 psf}{2.5} = 5{,}786 psf$

- $\sigma_v = \dfrac{V_1 + q \cdot L}{L - 2e} = \dfrac{(43{,}687\,lb) + (250\,psf \cdot 15\,ft)}{15\,ft - 2 \cdot (2.33\,ft)} = 4{,}587\,psf$

- $\underline{\underline{\sigma_v = 4{,}587\,psf \le q_a = 5{,}786\,psf}}\ \therefore\ Safe.$

where

σv = the vertical stress on the base of the wall,

q_a = allowable bearing pressure of the foundation soil,

FS_{BC} = factor of safety against bearing capacity failure (≥ 2.5),

c_f = cohesion of the foundation soil (0 psf),

N_c = bearing capacity factor (after Vesic 1973), and

N_γ = bearing capacity factor (after Vesic 1973).

Factor of Safety for Bearing Capacity

- $FS_{BC} = \dfrac{q_{ult}}{\sigma_v} = \dfrac{14{,}480\,psf}{4{,}587\,psf} = \underline{\underline{3.16}}$

B.3 Internal Stability Calculations

Calculations for the factor of safety against rupture and against pullout of the reinforcement are presented for Layer 5 of the reinforcement (Figure B-3). This layer is at a height of 11.06 ft. from the bottom of the reinforced soil zone and is 12.24 ft. from the top of the wall.

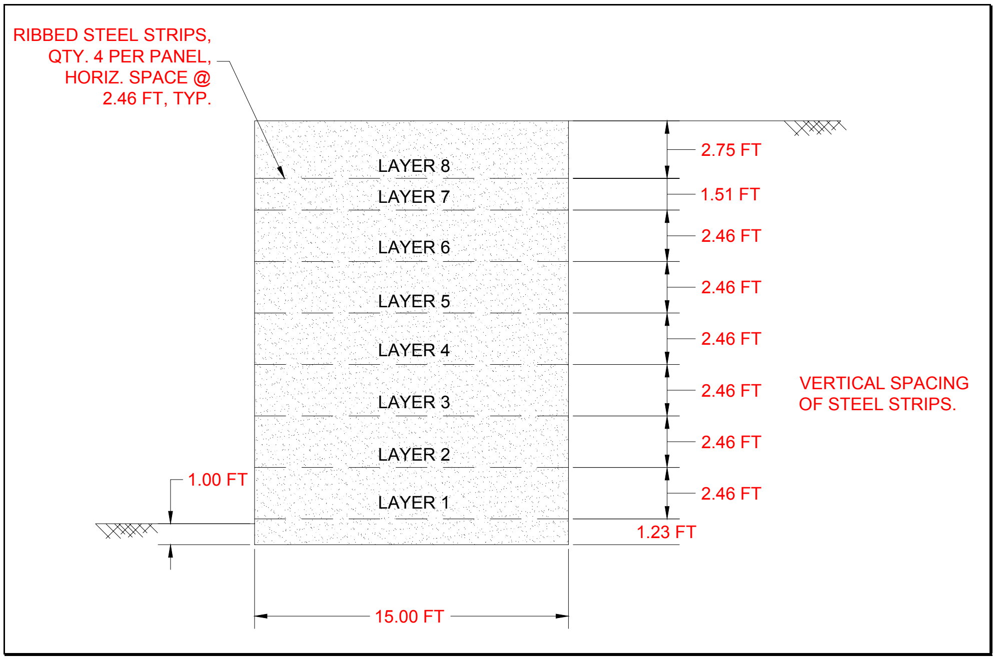

Figure B-3: Vertical spacing of steel grids in the Brown County wall on FM 2524.

B.3.1 Factor of Safety against Rupture of the Reinforcement

Calculating the factor of safety against rupture (FS_R) of the reinforcement is a three-part process involving calculation of the vertical stress (σ_v) acting on the reinforcement, the horizontal stress (σ_H) acting on a plane perpendicular to the reinforcement, and finally, computing the maximum tension (T_{MAX}) in the reinforcement.

Vertical stress at the elevation of the reinforcement (σ_v)

- $\sigma_v = \gamma_R \cdot Z + \sigma_2 + q + \Delta\sigma_v$
- $\sigma_v = (125\,pcf)\cdot(12.24\,ft) + 0 + 250\,psf + 0 = 1{,}780\,psf$

where

γ_R = unit weight of reinforced soil (125 pcf),

Z = depth to the reinforcement layer (12.24 ft.),

σ_2 = vertical stress caused by soil backfill on top of wall (0 psf), q = uniform surcharge load—that is, traffic load (250 psf), and

$\Delta\sigma_v$ = vertical stress caused by strip loads, footing loads, or point loads acting on the wall (0 psf).

Horizontal stress at level of reinforcement (σ_H)

- $\sigma_H = (K_r \cdot \sigma_v) + \Delta\sigma_h = (0.4783 \cdot 1{,}780\,psf) + 0 = 851\,psf$
- $K_r = K(z) = 1.692 \cdot K_a = 1.692 \cdot 0.2827 = 0.4783$
- $K_a = \tan^2\left(45 - \frac{\varphi_R}{2}\right) = \tan^2\left(45 - \frac{34°}{2}\right) = 0.2827$

where

$\Delta\sigma_h$ = horizontal stress caused by strip loads, footing loads, or point loads acting on the wall (0 psf),

K_r = stress variation coefficient (Figure B-4),

K_a = the active earth pressure coefficient for the retained soil, and

Φ_R = angle of internal friction of the reinforced soil (34°).

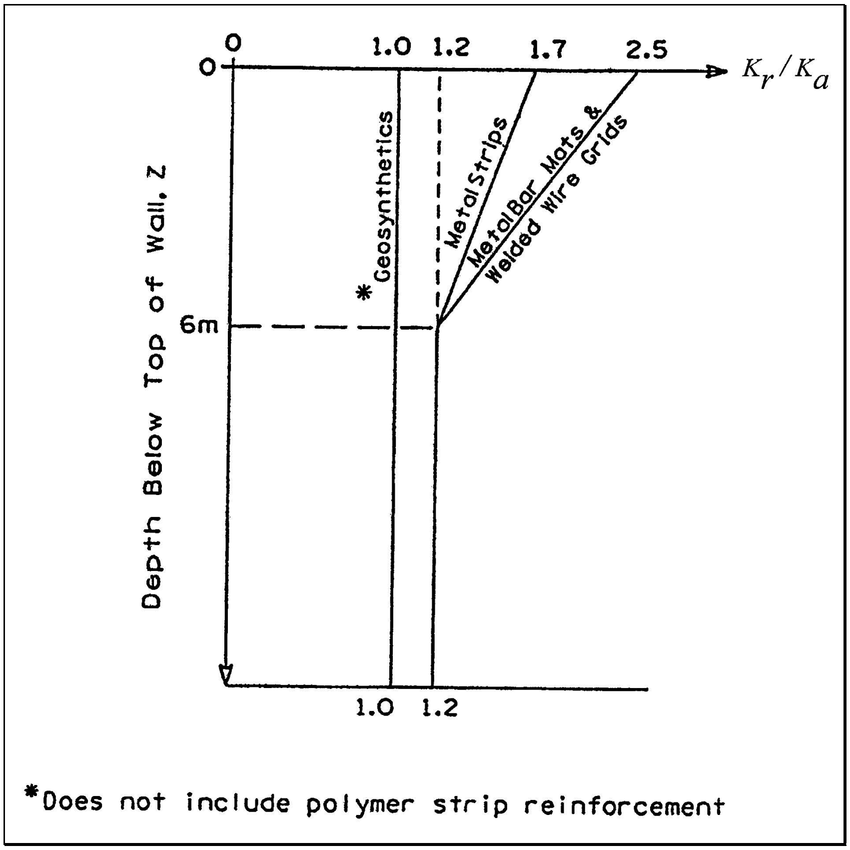

Figure B-4: Variation of stress ratio with depth in a MSE wall (after FHWA, 2001).

Maximum reinforcement tension (T_{MAX})

- $T_{\max} = \sigma_H \cdot S_v = 851 psf \cdot 3.035 ft = 2{,}584 lb$

where

S_v = vertical spacing of the reinforcement (3.035 ft.).

The units for maximum reinforcement tension (T_{MAX}) are a force per unit width of wall. Because this example is computed using US standard units, the units for T_{MAX} are pounds per foot width of wall.

Allowable design strength (T_a)

- $T_a = C_R \cdot \left(\frac{A_c \cdot F_y}{b} \right) = 0.48 \cdot \left(\frac{0.462 in^2 \cdot 65 ksi}{4.5 ft} \right) \cdot \left(\frac{1000 lb}{kip} \right) = 3{,}204 lb$

- $A_c = (\#bars) \cdot \pi \cdot \frac{(D_c)^2}{4} = (10) \cdot \pi \cdot \frac{(0.2425in)^2}{4} = 0.462in^2$

- $D_c = D_n - D_r = 0.2985in - 0.0560in = 0.2425in$

where

C_R = safety coefficient (0.55 for steel grid),

A_c = cross-sectional area of reinforcement corrected for corrosion,

F_y = yield stress of steel (65 ksi),

b = distance between the two outermost longitudinal bars of the steel grids (4.5 ft.),

D_c = thickness of the steel bar corrected for corrosion,

D_n = nominal bar thickness, and

D_r = reduction in bar thickness caused by corrosion.

Notes

- The galvanization of the bars has provided a zinc coating of 3.4 mils or 85μm. Corrosion rates for the zinc are 0.6 mils per year for the first two years and 0.16 mils per year after two years. When the zinc is completely corroded, the residual carbon steel will corrode at a rate of 0.5 mils per year. On the basis of these rates, complete corrosion of galvanization with the minimum required thickness of 85 μm (3.4 mils) is estimated to occur during the first 15.75 years, and a carbon steel thickness loss of 0.056 inches would be anticipated over the remaining seventy-five years. Therefore, E_r = 0.056 in.
- The allowable design strength (T_a) is the total strength of all the longitudinal bars per unit width of the reinforcement. Because this example is computed

using US standard units, the units for T_a are pounds per inch width of steel grid.

Safety against rupture of reinforcement

- $T_a \cdot R_c = 3{,}204 lb \cdot \frac{4.5 ft}{8.0 ft} = 1{,}802 lb$
- $R_c = \frac{b}{S_H} = \frac{4.5 ft}{8.0 ft}$
- $T_a \cdot R_c = 1{,}802 lb \leq T_{MAX} = 2{,}584 lb$ $\therefore$ *Not Safe.*

where

R_c = coverage ratio = (b/S_H),

b = distance between the two outermost longitudinal bars of the steel grids (4.5 ft.), and

S_H = center-to-center spacing of steel grids (8.0 ft.).

Factor of safety against rupture (FS_R)

- $$T_{al} = T_{ult} = \frac{F_y \cdot A_c}{b} = \left(\frac{65ksi \cdot 0.462in^2}{4.5ft}\right) \cdot \left(\frac{1000lb}{kip}\right) = 6{,}674lb$$

- $$FS_R = \frac{T_{al} \cdot R_c}{T_{MAX}} = \frac{(6{,}674lb) \cdot \left(\frac{4.5ft}{8.0ft}\right)}{2584lb} = \underline{\underline{1.45}}$$

where

T_{al} = long-term design strength.

The factors of safety for each layer of reinforcement are summarized in Table 4-6 (cf. Section 4.4.3.1).

B.3.2 Factor of Safety against Pullout of the Reinforcement

Pullout resistance (P_r)

- $$P_r = F^* \cdot \alpha \cdot \gamma_R \cdot Z_P \cdot L_e \cdot C$$

- $$P_r = (0.400) \cdot (1.0) \cdot (125pcf) \cdot (12.24ft) \cdot (8.364ft) \cdot (2) = 10{,}229lb$$

- $$\frac{P_r \cdot R_c}{FS_{PO}} = \frac{10{,}229lb \cdot \left(\frac{4.5ft}{8.0ft}\right)}{1.5} = 3{,}836lb$$

- $$T_{MAX} = 2{,}584lb \le \frac{P_r \cdot R_c}{FS_{PO}} = 3{,}836lb \therefore Safe.$$

where

P_r = pullout resistance of the reinforcement,

FS_{PO} = factor of safety against pullout (1.5),

L_e = the embedment or adherence length in the resisting zone behind the failure surface,

C = the effective unit perimeter of the reinforcement (C = 2 for strips, grids, and sheets),

F* = the pullout resistance factor, which is a function of passive and frictional resistance of the reinforcement,

α = a scale effect correction factor to account for a nonlinear stress reduction over the embedded length of highly extensible reinforcements, based on laboratory data (1.0 for metallic reinforcements),

γ_r = unit weight of reinforced soil (125 pcf), and

Z_P = depth to the reinforcement layer (12.24 ft.).

The units for pullout resistance (P_r) are a force per unit width of reinforcement. Because this example is computed using US standard units, the units for P_r are pounds per inch width of steel strip.

Factor of safety against pullout (FS_{PO})

- $$FS_{PO} = \frac{P_r \cdot R_c}{T_{MAX}} = \frac{10{,}229 lb \cdot \left(\frac{4.5 ft}{8.0 ft}\right)}{2{,}584 lb} = \frac{5{,}754 lb}{2{,}584 lb} = 2.23$$

The factors of safety for pullout are summarized for each layer of reinforcement in Table 4-7 (cf. Section 4.4.3.2).

APPENDIX C Representative Design Calculations for the Socorro Bridge Wall

The Socorro Bridge wall is a multi-tier mechanically stabilized earth (MSE) wall located in El Paso County, Texas. The following calculations were performed in accordance with the Federal Highway Administration (FHWA) 2001 guidelines to evaluate the external and internal stability of the wall.

C.1 Material Design Properties

The section of the Socorro Bridge wall analyzed is shown in Figure C-1. A detailed discussion of the wall geometry and the reinforcement used in the Socorro Bridge wall is available in Chapter 5 (cf. Section 5.4.1).

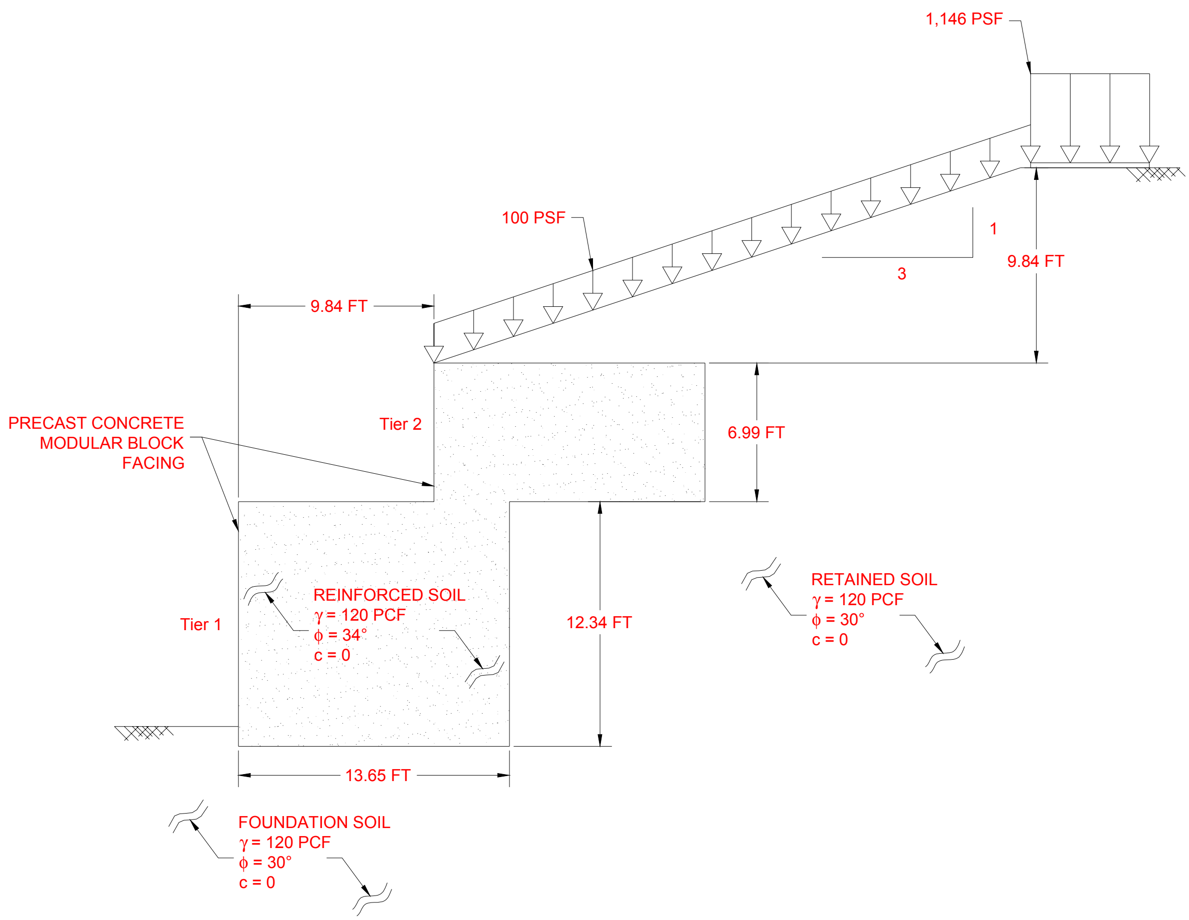

Figure C-1: Cross-section with typical soil properties of the Socorro Bridge wall.

C.2 Single- Versus Multi-Tier Wall Criteria

The FHWA guidelines provide criteria for establishing whether a wall should be considered a single- or multi-tier wall for the external and internal stability calculations. These criteria are based on the wall offset and are evaluated as follows.

1. Walls are designed as independent single-tier walls if $D > H_L \cdot \tan(90° - \varphi_r)$.
2. Walls are designed as one single wall if

   $D \le \left[\dfrac{H_L + H_U}{20}\right]$.

3. Walls are designed as multi-tier walls if $\left[\dfrac{H_L + H_U}{20}\right] < D \le H_L \cdot \tan(90° - \varphi_r)$

where

D = offset distance between the lower tier and upper tier,

H_L = height of the lower tier,

H_U = height of the upper tier, and

φ_r = angle of internal friction of the reinforced soil.

The following calculations were performed to determine whether the Socorro Bridge wall should be analyzed as a single- or two-tier wall.

Tier 1–Tier 2

- $D_{1-2} = 9.84\,ft$
- $\left[\dfrac{H_L + H_U}{20}\right] = \left[\dfrac{12.34\,ft + 6.99\,ft}{20}\right] = 0.97\,ft$
- $H_L \cdot \tan(90° - \varphi_r) = 12.34\,ft \cdot \tan(90° - 34°) = 18.3\,ft$

where

H_L = height of Tier 1 = 12.34 ft.,

H_U = height of Tier 2 = 6.99 ft., and

φ_r = 34°.

This satisfies the criteria

- $\left[\dfrac{H_L + H_U}{20}\right] = 0.97\,ft < D_{1-2} = 9.84\,ft \le H_L \cdot \tan(90° - \varphi_r) = 18.3\,ft$.

The external and internal stability analyses for the wall were conducted using the two-tier MSE wall procedures specified by FHWA guidelines.

C.3 External Stability Calculations

This section presents calculations for overturning (eccentricity) and bearing capacity. FHWA 2001 guidelines specify that the safety against sliding of the wall should be evaluated as part of the global stability analyses.

C.3.1 Eccentricity Calculation

The following calculations were performed using the vertical plane at the back edge of the reinforced soil mass of the lower tier to calculate the horizontal earth pressure forces F_1 and F_2 as used in the MSEW 2.0 software, as shown in Figure C-2.

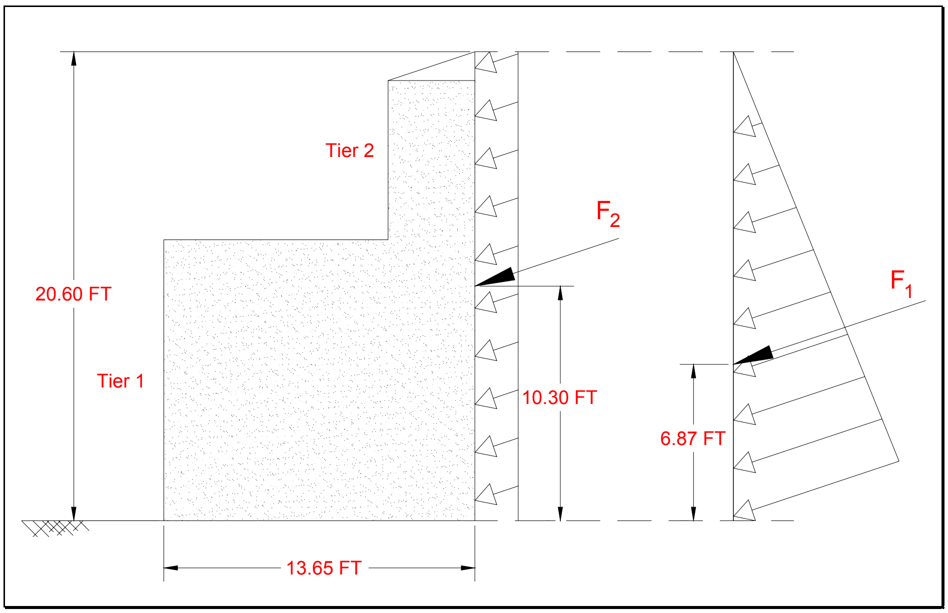

Figure C-2: Illustration of horizontal forces acting on the back of the reinforced soil zone for Socorro Bridge wall.

Horizontal forces

- $F_1 = \frac{1}{2} \cdot \gamma_f \cdot H^2 \cdot K_{af} = \left(\frac{1}{2}\right) \cdot (120\,pcf) \cdot (19.3\,ft)^2 \cdot (0.3688) = 8{,}267\,lb$
- $F_2 = q \cdot H \cdot K_{af} = (100\,psf) \cdot (19.33\,ft) \cdot (0.3688) = 713\,lb$
- $K_{af} = \cos(I) \cdot \dfrac{\cos(I) - \sqrt{\cos^2 I - \cos^2 \varphi_f}}{\cos^2(I) + \sqrt{\cos^2 I - \cos^2 \varphi_f}}$
- $K_{af} = \cos(14.29°) \cdot \dfrac{\cos(14.29°) - \sqrt{\cos^2(14.29°) - \cos^2(30°)}}{\cos^2(14.29°) + \sqrt{\cos^2(14.29°) - \cos^2(30°)}} = 0.3688$
- $I = \arctan\left(\dfrac{Y}{2H}\right) = \arctan\left(\dfrac{9.84\,ft}{2 \cdot 19.33\,ft}\right) = 14.29°$

where

F_1 = the horizontal force caused by the weight of retained soil,

F_2 = the horizontal force caused by surcharge load(s),

γ_f = unit weight of foundation soil,

H = total height of the MSE wall,

K_{af} = active earth pressure coefficient for the retained soil,

φ_f = angle of internal friction of foundation soil (30°),

q = uniform surcharge load,

I = broken back equivalent angle,

Y = vertical rise of slope, and

H = total height of wall.

Vertical forces components

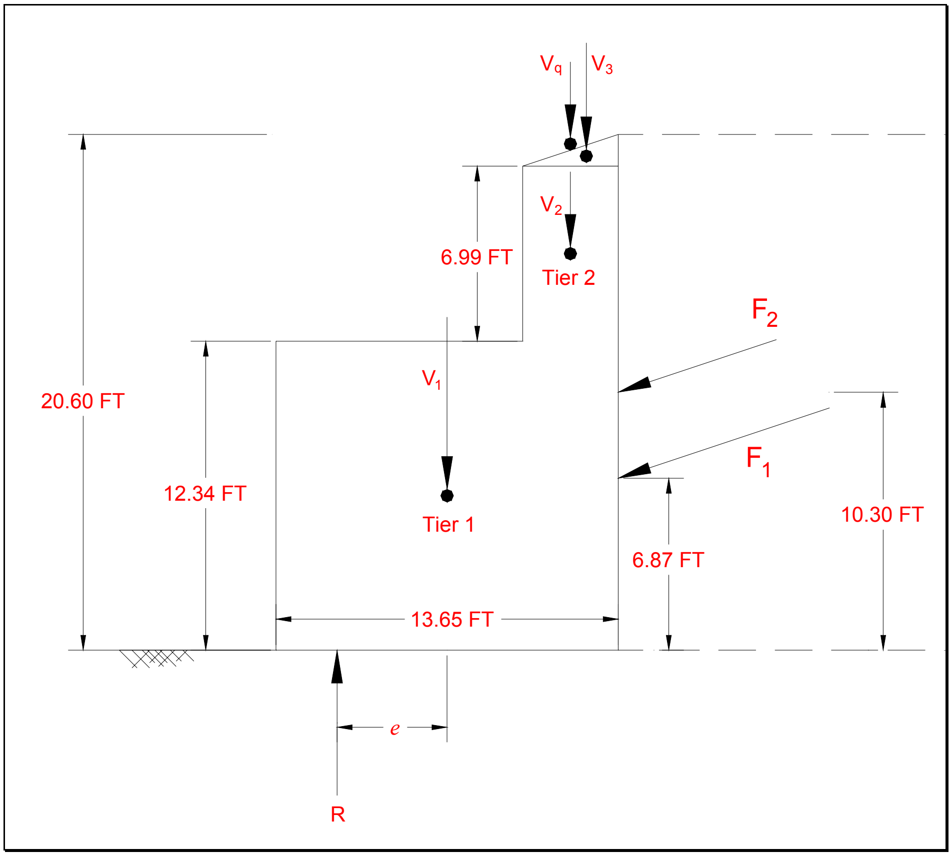

Figure C-3: Forces used for external stability analyses for the Socorro Bridge wall.

- $V_1 = \gamma_r \cdot H_1 \cdot L = (120\,pcf) \cdot (12.34\,ft) \cdot (13.65\,ft) = 20{,}213 lb$
- $V_2 = \gamma_r \cdot H_2 \cdot L_2 = (120\,pcf) \cdot (6.99\,ft) \cdot (3.81\,ft) = 3{,}195 lb$
- $V_3 = \frac{1}{2} \cdot \gamma_r \cdot H_3 \cdot L_3 = \frac{1}{2} \cdot (120\,pcf) \cdot (1.27\,ft) \cdot (3.81\,ft) = 290 lb$
- $V_q = q \cdot L_3 = (100\,psf) \cdot (3.81\,ft) = 381 lb$
- $F_{V1} = F_1 \cdot \sin(I) = 8{,}267 lb \cdot \sin(14.29°) = 2{,}041 lb$
- $F_{V2} = F_2 \cdot \sin(I) = 713 lb \cdot \sin(14.29°) = 176 lb$

where

V_1, V_2, V_3 = force caused by the weight of the reinforced soil mass of Tiers 1, 2, and 3, respectively,

H_1, H_2, H_3 = height of Tiers 1, 2 and 3, respectively,

L = length of the reinforcement of Tier 1,

L_2, L_3 = length of the reinforcement above the reinforced soil zone of Tier 1 for Tiers 2 and 3, respectively, and

γ_r = unit weight of reinforced soil.

Resultant of the vertical forces

- $R = V_1 + V_2 + V_3 + F_{V1}$
- $R = 20{,}213lb + 3{,}195lb + 290lb + 2{,}041lb = 25{,}739lb$

Sum moments about centerline of lower tier (Tier 1)

- $\sum M_{CL} = \left(F_1 \cdot \cos I \cdot \frac{H}{3}\right) - V_2 \cdot x_2 - V_3 \cdot x_3 - (R \cdot e) = 0$
- $e = \dfrac{\left(F_1 \cdot \cos I \cdot \frac{H}{3}\right) - (V_2 \cdot x_2) - (V_3 \cdot x_3)}{R}$
- $e = \dfrac{\left(17{,}405lb \cdot \cos(I) \cdot \frac{19.33ft}{3}\right) - (3{,}195 \cdot 4.92ft) - (290lb \cdot 5.56ft)}{25{,}739lb}$
- $e = \underline{\underline{1.27ft}}$

where

F_1 = the horizontal force caused by the weight of retained soil,

V_2, V_3 = force caused by the weight of the reinforced soil mass of Tiers 1, 2, and 3, respectively,

x_2, x_3 = distance from the centerline of Tier 1 to the forces V_2 and V_3, respectively,

H = total height of the MSE wall,

R = resultant of vertical forces, and

e = eccentricity—that is, the distance from the centerline of the wall to the resultant force (R).

C.3.2 Bearing Capacity Calculations

The following calculations are for the safety of the wall with respect to bearing capacity failure.

Bearing Pressures

- $q_{ult} = c_f \cdot N_c + 0.5 \cdot (L - 2e) \cdot \gamma_f \cdot N_\gamma$
- $q_{ult} = 0.5 \cdot (13.65\,ft - 2 \cdot (1.27\,ft)) \cdot 120\,pcf \cdot 22.4 = 14{,}934\,psf$
- $q_a = \dfrac{q_{ult}}{FS_{BC}} = \dfrac{14{,}932\,psf}{2.5} = 5{,}973\,psf$
- $\sigma_v = \dfrac{V_1 + V_2 + V_3 + V_q + F_{V1}}{L - 2e}$
- $\sigma_v = \dfrac{20{,}213\,lb + 3{,}195\,lb + 290\,lb + 381\,lb + 2{,}041\,lb}{13.65\,ft - 2 \cdot (1.27\,ft)} = 2{,}389\,psf$
- $\underline{\underline{\sigma_v = 2{,}389\,psf \le q_a = 5{,}973\,psf}} \therefore Safe.$

where

σ_v = the vertical stress on the base of the wall,

q_a = allowable bearing pressure of the foundation soil,

FS_{BC} = factor of safety against bearing capacity failure (≥ 2.5),

c_f = cohesion of the foundation soil (0 psf),

N_c = bearing capacity factor (after Vesic 1973), and

N_γ = bearing capacity factor (after Vesic 1973).

Factor of Safety for Bearing Capacity

- $FS_{BC} = \dfrac{q_{ult}}{\sigma_v} = \dfrac{14{,}934\,psf}{2{,}389\,psf} = \underline{\underline{6.25}}$

C.4 Internal Stability Calculations

Calculations for the factor of safety against rupture and pullout of the reinforcement are presented for Layer 3 of the reinforcement (Figure C-4). This layer is at a height of 4.04 ft. from the bottom of the reinforced soil zone and is 8.30 ft. from the top of Tier 1.

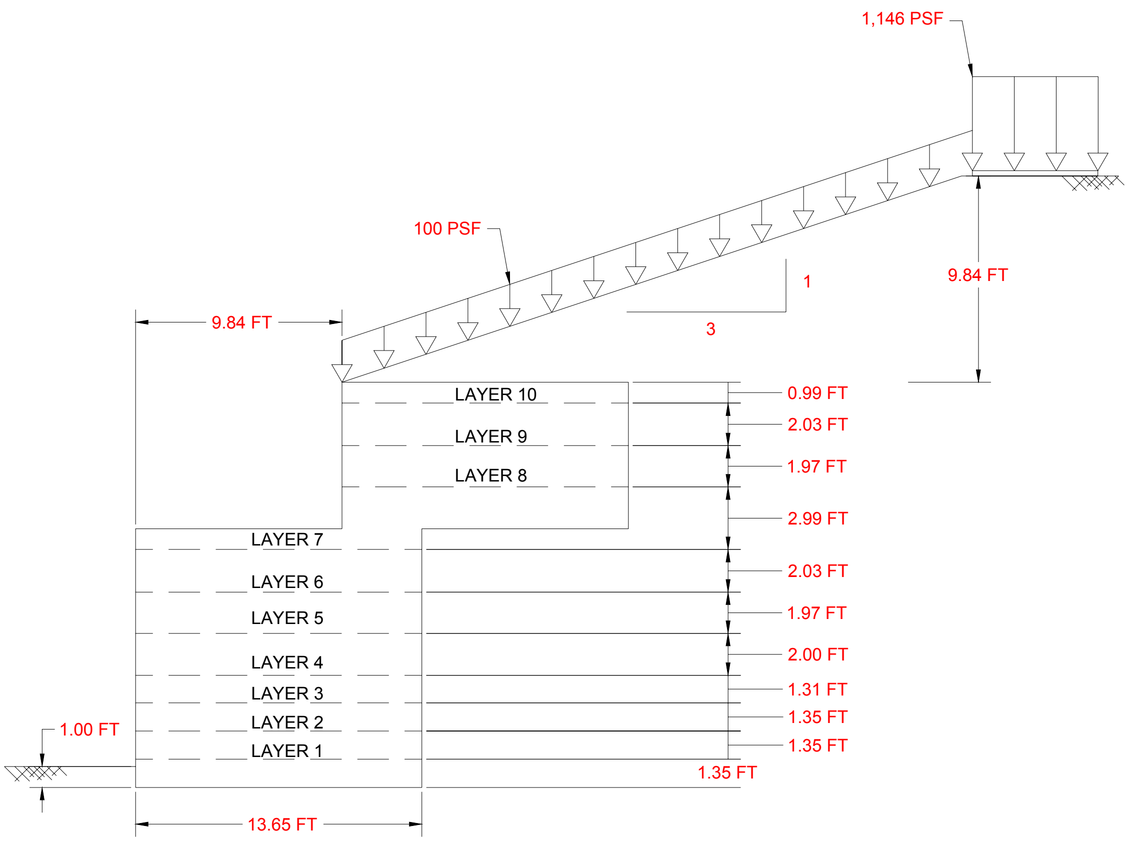

Figure C-4: Cross-section of the Socorro Bridge wall illustrating tier heights, length of reinforcement, and embedment of each tier.

C.4.1 Factor of Safety Against Rupture of the Reinforcement

Calculating the factor of safety against rupture (FS_R) of the reinforcement for multi-tier walls is a four-part process that involves calculating of the additional vertical stress (σ_i) on the reinforcement from the upper tier, the vertical stress (σ_v) on the reinforcement, the horizontal stress (σ_H) acting on a plane perpendicular to the reinforcement, and finally, computing the maximum tension (T_{MAX}) in the reinforcement.

Additional vertical stress (σ_i) on the reinforcement from Tier 2

The FHWA guidelines specify the additional stress from an upper tier acting on a layer of reinforcement in the lower tier can be illustrated as in Figure C-5.

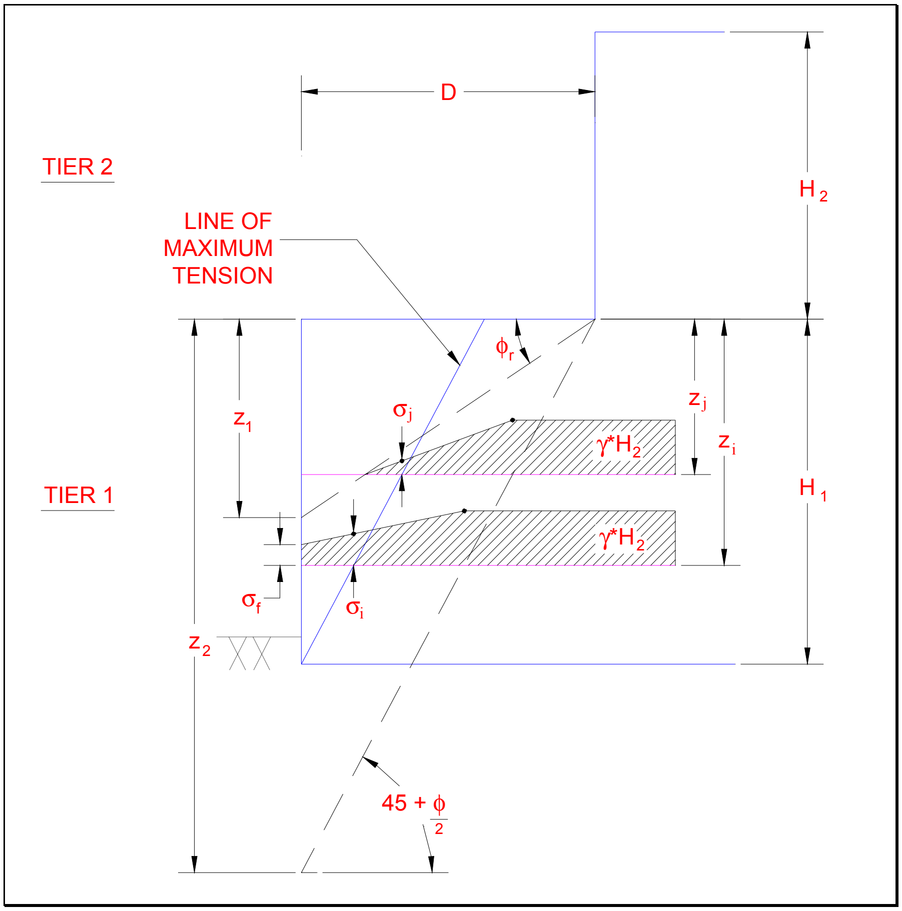

Figure C-5: Illustration of additional stress calculation using the FHWA guidelines (after FHWA, 2001).

- $z_1 = D_{1-2} \cdot (\tan \varphi_r) = 9.84\,ft \cdot (\tan 34°) = 6.637\,ft$

- $z_2 = D_{1-2} \cdot \tan\left(45 + \dfrac{\varphi_r}{2}\right) = 9.84\,ft \cdot \tan\left(45 + \dfrac{34°}{2}\right) = 18.506\,ft$

- $\sigma_2 = \dfrac{\gamma_f \cdot L \cdot \tan(\beta)}{2} = \dfrac{120\,pcf \cdot 13.65\,ft \cdot \tan(18.4°)}{2} = 272.4\,psf$

- $\sigma_f = \dfrac{z_i - z_1}{z_2 - z_1} \cdot (\gamma \cdot H_U + \sigma_2 + q)$

- $\sigma_f = \dfrac{8.30\,ft - 6.637\,ft}{18.506\,ft - 6.637\,ft} \cdot (120\,pcf \cdot 6.99\,ft + 272.4\,psf + 100\,psf) = 170\,psf$

- $$\frac{(\gamma \cdot H_U + \sigma_2 + q) - \sigma_f}{(z_2 - z_i) \cdot \tan\left(45 - \frac{\varphi_r}{2}\right)} = \frac{(\sigma_i - \sigma_f)}{L_a}$$

- $$\sigma_i = \frac{(\gamma \cdot H_U + \sigma_2 + q) - \sigma_f}{(z_2 - z_i) \cdot \tan\left(45 - \frac{\varphi_r}{2}\right)} \cdot L_a + \sigma_f$$

- $$\sigma_i = \left[\frac{(120\,pcf \cdot 6.99\,ft + 272.4\,psf + 100\,psf) - 170\,psf}{(18.506\,ft - 8.30\,ft) \cdot \tan\left(45° - \frac{34°}{2}\right)} \cdot 2.15\,ft\right] + 170\,psf = 582\,psf$$

- $$L_a = (H_L - z_i) \cdot \tan\left(45° - \frac{\varphi_r}{2}\right) = (12.34\,ft - 8.30\,ft) \cdot \tan\left(45° - \frac{34°}{2}\right) = 2.15\,ft$$

Vertical stress (σ_v) on the reinforcement

- $$\sigma_v = \gamma_R \cdot Z + \Delta\sigma_2 + \sigma_i + \Delta\sigma_v$$

- $$\sigma_v = (120\,pcf) \cdot (8.30\,ft) + 0 + 582\,psf + 0 = 1{,}578\,psf$$

where

γ_r = unit weight of reinforced soil (120 pcf),

z_i = depth to the *i*th layer of reinforcement (5.63 ft.),

σ_2 = vertical stress caused by soil backfill on top of wall,

σ_i = additional stress on the *i*th layer of reinforcement caused by Tier 2,

σ_f = additional stress at the wall face on the *i*th layer of reinforcement caused by Tier 2,

$\Delta\sigma_v$ = vertical stress caused by strip loads, footing loads, or point loads acting on the wall (0 psf), and

q = uniform surcharge load (100 psf).

Horizontal stress on the layer of reinforcement (σ_H)

- $\sigma_H = (K_r \cdot \sigma_v) + \Delta\sigma_h = (0.2827 \cdot 1{,}578 psf) + 0 = 446 psf$
- $K_r = K(z) = 1.0 \cdot K_a = 1.0 \cdot 0.2827 = 0.2827$
- $K_a = \tan^2\left(45 - \frac{\varphi_R}{2}\right) = \tan^2\left(45 - \frac{34°}{2}\right) = 0.2827$

where

$\Delta\sigma_h$ = horizontal stress caused by strip loads, footing loads, or point loads acting on the wall (0 psf),

K_r = stress variation coefficient (Figure C-6),

K_a = the active earth pressure coefficient for the retained soil, and

φ_r = angle of internal friction of the reinforced soil (34°).

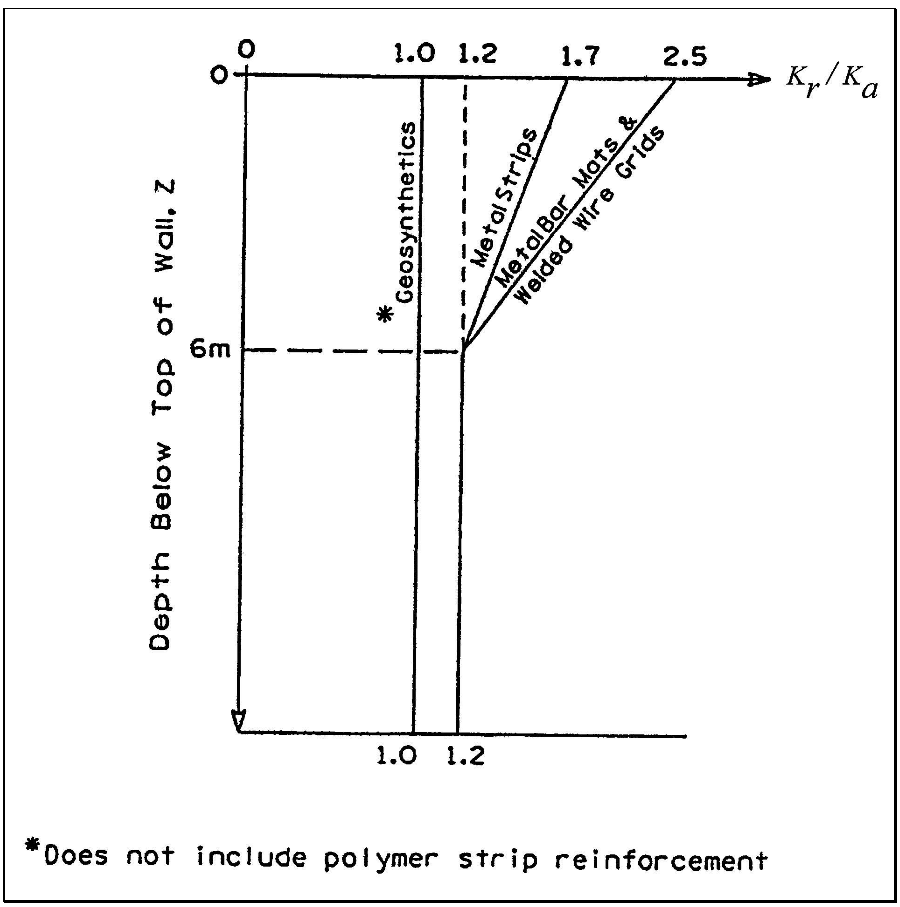

Figure C-6: Variation of stress ratio with depth in a MSE wall (after FHWA, 2001).

Maximum reinforcement tension (T_{MAX})

- $T_{\max} = \sigma_H \cdot S_v = 446 psf \cdot 1.33 ft = 593 lb$

where

S_v = vertical spacing of the reinforcement (2.0 ft.),

The units for maximum reinforcement tension (T_{MAX}) are a force per unit width of wall. Because this example is computed using US standard units, the units for T_{MAX} are pounds per foot width of wall.

Safety against rupture of reinforcement

- $T_{ult} = 8{,}997 lb$
- $T_{al} = \dfrac{T_{ult}}{RF_d \cdot RF_{id} \cdot RF_c} = \dfrac{8{,}997 lb}{1.10 \cdot 1.12 \cdot 2.40} = 2{,}782 lb$
- $T_a = \dfrac{T_{al}}{FS_R} = \dfrac{2{,}782 lb}{1.5} = 1{,}855 lb$
- $R_c = 0.75$
- $T_a \cdot R_c = 1{,}855 lb \cdot 0.75 = 1{,}391 lb \geq T_{MAX} = 593 lb \therefore Safe.$

where

R_c = coverage ratio for continuous reinforcement (1.0).

The allowable design strength (T_a) is the strength of the geogrid per unit width of the reinforcement. Because this example is computed using US standard units, the units for T_a are pounds per foot width of geogrid.

Factor of safety against rupture (FS_R)

- $FS_R = \dfrac{T_{al} \cdot R_c}{T_{MAX}} = \dfrac{(2{,}782 lb) \cdot (0.75)}{593 lb} = 3.52$

where

T_{al} = long-term design strength.

The factors of safety for each layer of reinforcement are summarized in Table 5-6 (cf. Section 5.4.3.1).

C.4.2 Factor of Safety Against Pullout of the Reinforcement

Pullout resistance (P_r)

- $P_r = F^* \cdot \alpha \cdot \gamma_r \cdot Z_P \cdot L_e \cdot C$
- $P_r = (0.54)\cdot(1.0)\cdot(120\,psf)\cdot(8.30\,ft)\cdot(11.50\,ft)\cdot(2) = 12{,}363 lb$
- $\dfrac{P_r \cdot R_c}{FS_{PO}} = \dfrac{12{,}363 lb \cdot 0.75}{1.5} = 6{,}182 lb$
- $T_{MAX} \le \dfrac{P_r \cdot R_c}{FS_{PO}}$

where

P_r = pullout resistance of the reinforcement,

FS_{PO} = factor of safety against pullout (1.5),

L_e = the embedment or adherence length in the resisting zone behind the failure surface,

C = the effective unit perimeter of the reinforcement (C = 2 for strips, grids, and sheets),

F* = the pullout resistance factor, which is a function of passive and frictional resistance of the reinforcement (0.8*tanφ = 0.54),

α = a scale effect correction factor to account for a nonlinear stress reduction over the embedded length of highly extensible reinforcements, based on laboratory data (1.0 for metallic reinforcements),

γ_r = the vertical stress at the soil–reinforcement interface, and

Z_P = depth from the top of Tier 1 to the layer of reinforcement.

The units for pullout resistance (P_r) are a force per unit width of reinforcement. Because this example is computed using US standard units, the units for P_r are pounds per foot width of geogrid.

Factor of safety against pullout (FS_{PO})

- $FS_{PO} = \frac{P_r \cdot R_c}{T_{MAX}} = \frac{12{,}363 lb \cdot (0.75)}{593 lb} = 15.6$

The factors of safety for pullout (FS_{PO}) are summarized for each layer of reinforcement in Table 5-7 (cf. Section 5.4.3.2).

APPENDIX D Representative Design Calculations for the US 290 Wall

The US 290 wall is a multi-tier mechanically stabilized earth (MSE) wall located in Travis County, Texas. The following calculations were performed in accordance with the Federal Highway Administration (FHWA) 2001 guidelines to evaluate the internal stability of the wall.

D.1 Material Design Properties

The section of the US 290 wall analyzed is shown in Figure D-1. A detailed discussion of the wall geometry and the reinforcement used in the US 290 wall is available in Chapter 5 (cf. Section 5.5.1).

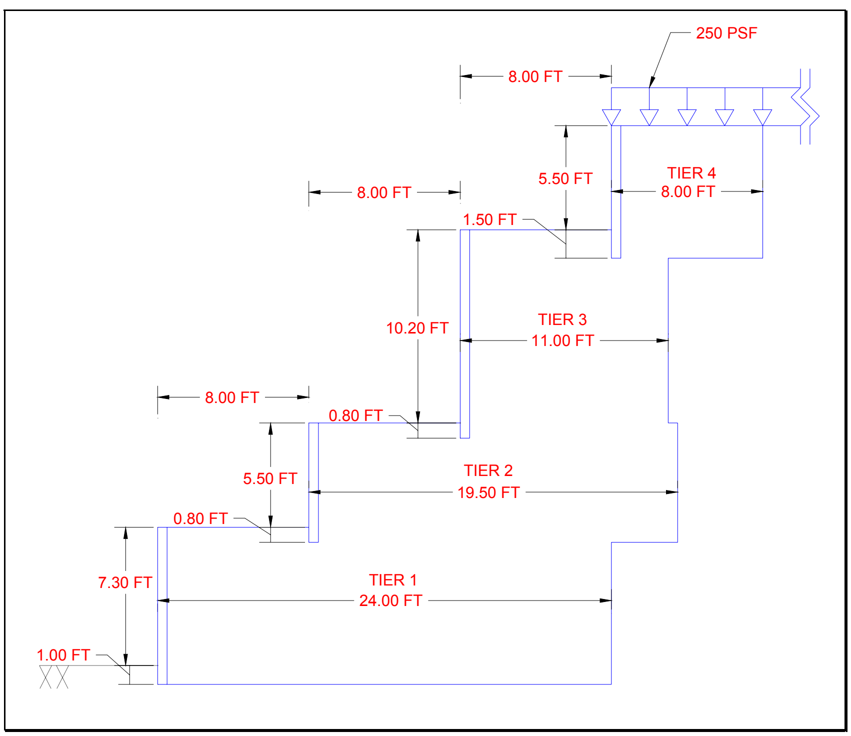

Figure D-1: Cross-section of the US 290 wall illustrating tier heights, length of reinforcement, and embedment of each tier.

D.2 Single- Versus Multi-Tier Wall Criteria

The FHWA guidelines provide criteria for establishing whether a wall should be considered a single- or multi-tier wall for the internal stability calculations. These criteria are based on the wall offset and are evaluated as follows.

4. Walls are designed as independent single-tier walls if $D > H_L \cdot \tan(90° - \varphi_r)$.
5. Walls are designed as one single wall if

   $D \le \left[\dfrac{H_L + H_U}{20}\right]$.

6. Walls are designed as multi-tier walls if $\left[\dfrac{H_L + H_U}{20}\right] < D \le H_L \cdot \tan(90° - \varphi_r)$

   where

   D = offset distance between the lower tier and upper tier,

   H_L = height of the lower tier,

   H_U = height of the upper tier, and

   φ_r = angle of internal friction of the reinforced soil.

The US 290 wall has four tiers. Each pair of two tiers was evaluated as to applicable design criteria. Sample calculations are presented below for the evaluation of the Tier 1–2 combination only.

Tier 1–Tier 2

- $D_{1-2} = 8.0\,ft$
- $\left[\dfrac{H_L + H_U}{20}\right] = \left[\dfrac{8.30\,ft + 6.30\,ft}{20}\right] = 0.73\,ft$
- $H_L \cdot \tan(90° - \varphi_r) = 8.3\,ft \cdot \tan(90° - 34°) = 12.3\,ft$

where

H_L = height of Tier 1 = 8.30 ft.,

H_U = height of Tier 2 = 6.30 ft., and

φ_r = 34°.

This satisfies the criteria

- $\left[\dfrac{H_L + H_U}{20}\right] = 0.73\,ft < D_{1-2} = 8.0\,ft \le H_L \cdot \tan(90° - \varphi_r) = 12.3\,ft$.

Each combination of tiers was checked using the criteria in the FHWA guidelines and are summarized in Table 5-2 (cf. Section 5.2).

D.3 Internal Stability Calculations

Calculations for the factor of safety against rupture and against pullout of the reinforcement are presented for Layer 2 of the reinforcement (Figure D-2). This layer is at a height of 2.67 ft. from the bottom of the reinforced soil zone and is 5.63 ft. from the top of Tier 1.

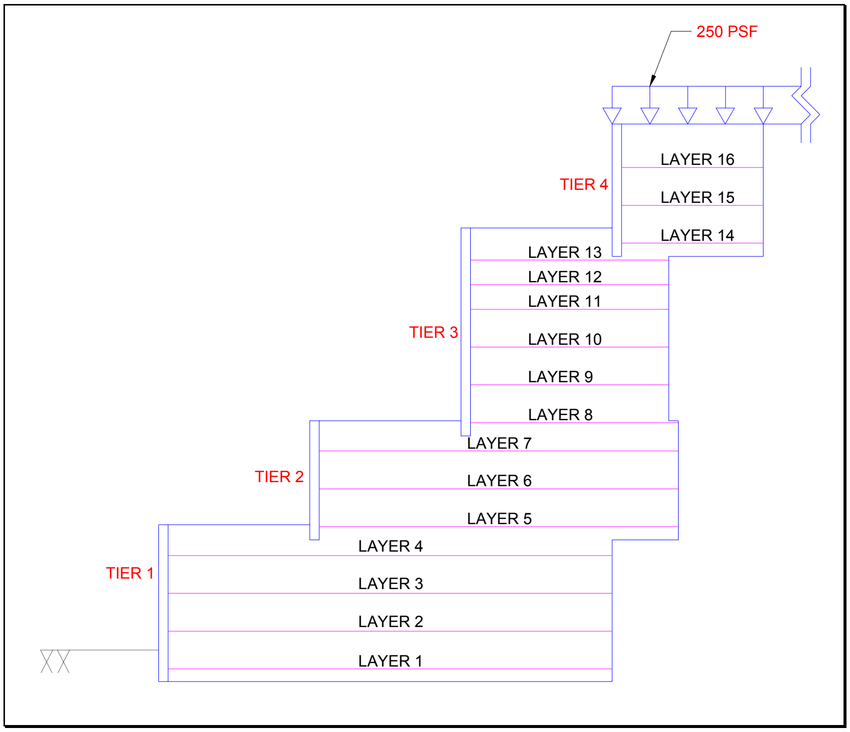

Figure D-2: Layout of geogrid in the multi-tier MSE wall section on US 290.

D.3.1 Factor of Safety Against Rupture of the Reinforcement

Calculating the factor of safety against rupture (FS_R) of the reinforcement for multi-tier walls is a four-part process that involves calculating of the additional vertical stress (σ_i) on the reinforcement from the upper tier, the vertical stress (σ_v) on the reinforcement, the horizontal stress (σ_H) acting on a plane perpendicular to the reinforcement, and finally, computing the maximum tension (T_{MAX}) in the reinforcement.

Additional vertical stress (σ_i) on the reinforcement from Tier 2

The FHWA guidelines specify the additional stress from an upper tier acting on a layer of reinforcement in the lower tier can be illustrated as in Figure D-3.

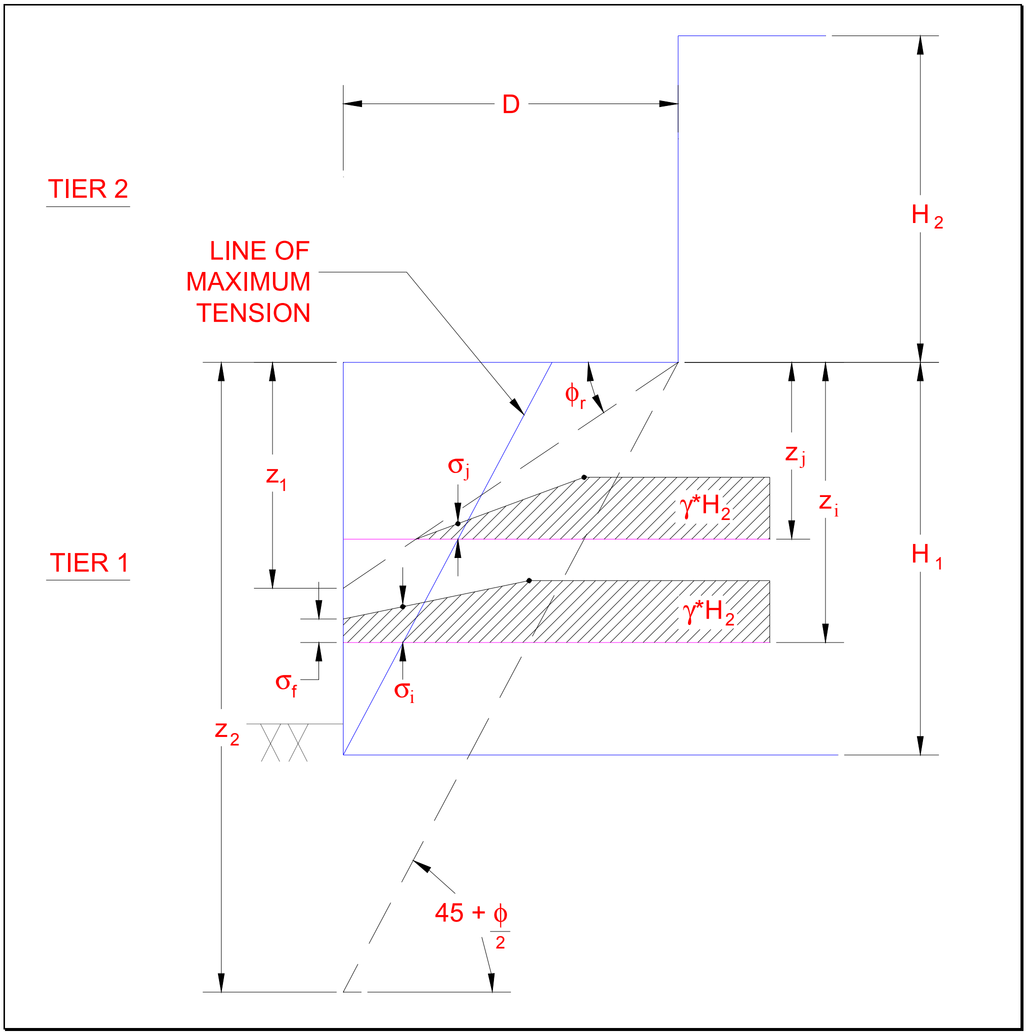

Figure D-3: Illustration of additional stress calculation using the FHWA guidelines (after FHWA, 2001).

- $z_1 = D_{1-2} \cdot (\tan \varphi_r) = 8.0\,ft \cdot (\tan 34°) = 5.40\,ft$

- $z_2 = D_{1-2} \cdot \tan\left(45 - \dfrac{\varphi_r}{2}\right) = 8.0\,ft \cdot \tan\left(45 - \dfrac{34°}{2}\right) = 15.0\,ft$

- $\sigma_f = \dfrac{z_i - z_1}{z_2 - z_1} \cdot (\gamma \cdot H_2) = \dfrac{5.63\,ft - 5.40\,ft}{15.0\,ft - 5.40\,ft} \cdot (120\,pcf \cdot 6.30\,ft) = 18.33\,psf$

- $\dfrac{\gamma \cdot H_2 - \sigma_f}{(z_2 - z_i) \cdot \tan\left(45 - \dfrac{\varphi_r}{2}\right)} = \dfrac{(\sigma_i - \sigma_f)}{L_a}$

- $\sigma_i = \dfrac{\gamma \cdot H_2 - \sigma_f}{(z_2 - z_i) \cdot \tan\left(45 - \dfrac{\varphi_r}{2}\right)} \cdot L_a + \sigma_f$

- $\sigma_i = \left[\dfrac{120\,pcf \cdot 6.30\,ft - 18.33\,psf}{(15.0\,ft - 5.63\,ft) \cdot \tan\left(45° - \dfrac{34°}{2}\right)} \cdot 1.42\,ft \right] + 18.33\,psf = 227.5\,psf$

- $L_a = (H_1 - z_i) \cdot \tan\left(45° - \dfrac{\varphi_r}{2}\right) = (8.30\,ft - 5.63\,ft) \cdot \tan\left(45° - \dfrac{34°}{2}\right) = 1.42\,ft$

Vertical stress (σ_v) on the reinforcement

- $\sigma_v = \gamma_R \cdot Z + \sigma_2 + \sigma_i + \Delta\sigma_v$
- $\sigma_v = (120\,pcf) \cdot (5.63\,ft) + 0 + 227.5\,psf + 0 = 903\,psf$

where

γ_r = unit weight of reinforced soil (120 pcf),

z_i = depth to the *i*th layer of reinforcement (5.63 ft.),

σ_2 = vertical stress caused by soil backfill on top of wall (0 psf),

σ_i = additional stress on the *i*th layer of reinforcement caused by Tier 2,

σ_f = additional stress at the wall face on the *i*th layer of reinforcement caused by Tier 2, and

$\Delta\sigma_v$ = vertical stress caused by strip loads, footing loads, or point loads acting on the wall (0 psf).

Horizontal stress on the layer of reinforcement (σ_H)

- $\sigma_H = (K_r \cdot \sigma_v) + \Delta\sigma_h = (0.2827 \cdot 903\,psf) + 0 = 255\,psf$
- $K_r = K(z) = 1.0 \cdot K_a = 1.0 \cdot 0.2827 = 0.2827$

- $K_a = \tan^2\left(45 - \frac{\varphi_R}{2}\right) = \tan^2\left(45 - \frac{34°}{2}\right) = 0.2827$

where

$\Delta\sigma_h$ = horizontal stress caused by to strip loads, footing loads, or point loads acting on the wall (0 psf),

K_r = stress variation coefficient (Figure D-4),

K_a = the active earth pressure coefficient for the retained soil, and

φ_r = angle of internal friction of the reinforced soil (34°).

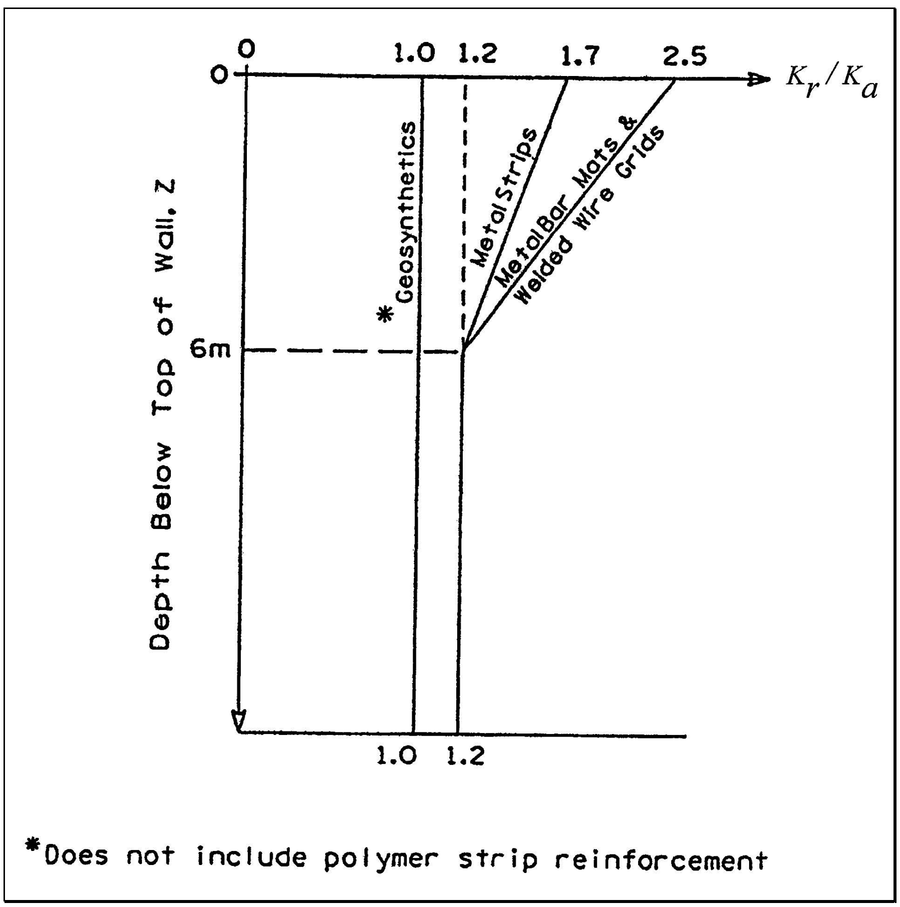

Figure D-4: Variation of stress ratio with depth in a MSE wall (after FHWA, 2001).

Maximum reinforcement tension (T_{MAX})

- $T_{\max} = \sigma_H \cdot S_v = 255\,psf \cdot 2.0\,ft = 510\,lb$

where

S_v = vertical spacing of the reinforcement (2.0 ft.).

The units for maximum reinforcement tension (T_{MAX}) are a force per unit width of wall. Because this example is computed using US standard units, the units for T_{MAX} are pounds per foot width of wall.

Safety against rupture of reinforcement

- $T_{ult} = 7{,}550\,lb$
- $T_{al} = \dfrac{T_{ult}}{RF_d \cdot RF_{id} \cdot RF_c} = \dfrac{7{,}550\,lb}{1.10 \cdot 1.05 \cdot 2.65} = 2{,}467\,lb$
- $T_a = \dfrac{T_{al}}{FS_R} = \dfrac{2{,}467\,lb}{1.5} = 1{,}644\,lb$
- $R_c = 1.0$
- $T_a \cdot R_c = 1{,}644\,lb \geq T_{MAX} = 510\,lb \therefore Safe.$

where

R_c = coverage ratio for continuous reinforcement (1.0).

The allowable design strength (T_a) is the strength of the geogrid per unit width of the reinforcement. Because this example is computed using US standard units, the units for T_a are pounds per foot width of geogrid.

Factor of safety against rupture (FS_R)

- $FS_R = \dfrac{T_{al} \cdot R_c}{T_{MAX}} = \dfrac{(510\,lb) \cdot (1.0)}{2{,}467\,lb} = 4.83$

where

T_{al} = long-term design strength.

The factors of safety for each layer of reinforcement are summarized in Table 5-10 (cf. Section 5.5.3.1).

D.3.2 Factor of Safety Against Pullout of the Reinforcement

Pullout resistance (P_r)

- $P_r = F^* \cdot \alpha \cdot \sigma_V \cdot L_e \cdot C$
- $P_r = (0.405) \cdot (1.0) \cdot (120\,psf) \cdot (5.63\,ft) \cdot (22.58\,ft) \cdot (2) = 12{,}348\,lb$
- $\dfrac{P_r \cdot R_c}{FS_{PO}} = \dfrac{12{,}348\,lb}{1.5} = 8{,}232\,lb$
- $T_{MAX} = 510\,lb \le \dfrac{P_r \cdot R_c}{FS_{PO}} = 8{,}232\,lb$

where

P_r = pullout resistance of the reinforcement,

FS_{PO} = factor of safety against pullout (1.5),

L_e = the embedment or adherence length in the resisting zone behind the failure surface,

C = the effective unit perimeter of the reinforcement(C = 2 for strips, grids, and sheets),

F* = the pullout resistance factor, which is a function of passive and frictional resistance of the reinforcement (0.6*tanφ = 0.405).,

α = a scale effect correction factor to account for a nonlinear stress reduction over the embedded length of highly extensible reinforcements, based on laboratory data (1.0 for metallic reinforcements), and

σ_v = the vertical stress at the soil–reinforcement interface.

The units for pullout resistance (P_r) are a force per unit width of reinforcement. Because this example is computed using US standard units, the units for P_r are pounds per foot width of geogrid.

Factor of safety against pullout (FS_{PO})

- $FS_{PO} = \frac{P_r \cdot R_c}{T_{MAX}} = \frac{12{,}348 lb \cdot (1.0)}{510 lb} = 24.18$

The factors of safety for pullout (FS_{PO}) are summarized for each layer of reinforcement in Table 5-11 (cf. Section 5.5.3.2).

References

AASHTO (1998). LRFD Bridge Design Specifications. American Association of State Highway and Transportation Officials, Washington, DC.

Arriaga, F. (2003). "Response of Geosynthetic-Reinforced Structures Under Working Stress and Failure Conditions." Ph.D. Thesis, Department of Civil, Environmental, and Architectural Engineering, University of Colorado, Boulder.

Barker, R. M., J. M. Duncan, K. B. Rojiani, P. S. K. Ooi, C. K. Tan, S. G. Kim (1991). Manuals for the Design of Bridge Foundations. NCHRP Report 343. TRB, National Research Council, Washington, DC.

Carter, J. P., and F. H. Kulhawy (1988). Analysis and Design of Foundations Socketed into Rock. Report No. EL-5918. Empire State Electric Engineering Research Corporation and Electric Power Research Institute, New York, 158p.

Das, B. M. (1994). Principles of Geotechnical Engineering, 3rd ed. PWS Publishing Co., Boston.

Das, B. M. (2002). Principles of Geotechnical Engineering, 5th ed. Brooks/Cole, Pacific Grove, CA.

Elias, V., Christopher, B. R. (1996). Mechanically Stabilized Earth Walls and Reinforced Soil Slopes: FHWA Demonstration Project 82 Reinforced Soil Structures MSEW and RSS. Federal Highway Administration, Washington, DC, Publication No. FHWA-SA-96-071, 371p.

Elias, V., Christopher, B.R., Berg, R.R. (2001). Mechanically Stabilized Earth Walls and Reinforced Soil Slopes: Design and Construction Guidelines. NHI Course No. 132042, Federal Highway Administration, Washington, DC, Publication No. FHWA-NHI-00-043, 394p.

Leshchinsky, D (2002). "MSEW 2.0." Adama Engineering, Inc., Newark, DE.

Meyerhof, G. G. (1963). "Some Recent Research on the Bearing Capacity of Foundations." Canadian Geotechnical Journal, Vol. 1, pp. 16-26.

Vesic, A. S. (1973). "Analysis of Ultimate Loads on Shallow Foundations." Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 99, No. SM1, pp. 45-73.

Wang, L. (2004). "Global Stability of Mechanically Stabilized Earth Walls." M.S. Thesis, Department of Civil Engineering, The University of Texas at Austin.

Wright, S. G., Duncan, J. M. (1991). "Limit Equilibrium Stability Analyses for Reinforced Slopes." Transportation Research Record No. 1330. TRB, National Research Council, Washington, DC., pp. 40-46.

Wright, S. G. (1999). "UTEXAS4 A Computer Program for Slope Stability Calculations." The University of Texas at Austin, September, 221p.