Simple Reliability Analysis of Mechanically Stabilized Earth Walls

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ABSTRACT

MSE structures are retaining walls that are reinforced with inclusions placed horizontally in the soil. The design method of MSE structures has progressed from using the Allowable Stress Design (ASD) to using the Load and Resistance Factored Design (LRFD) using the- "calibration to fit" method. The natural progression beyond the "calibration to fit" method, is to use reliability theory in calibration of load and resistance factors to satisfy a target reliability index. In this paper, consistent with literature, simple reliability analysis will be applied to MSE walls designed using AASHTO LRFD. It will be demonstrated that reliability analysis can be used in a simplified format by practitioners for MSE wall design. In addition, the results will serve as an indication on the appropriateness of load and resistance factors in AASHTO LRFD for MSE walls.

RÉSUMÉ (TO FOLLOW)

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1 INTRODUCTION

Mechanically Stabilized Earth (MSE) structures are retaining walls that are reinforced with inclusions that consist of horizontally placed steel elements. Steel elements are classified as inextensible soil reinforcing due to the difference in the strain of the steel in comparison to the strain of the soil they reinforce. The steel elements are routinely connected to a facing element. The type of soil reinforcing and the type of facing will depend on the structure application. Inextensible steel soil reinforcing has been used in the current form since the early 1970s and MSE is now a well-established technology worldwide (Yu and Bathurst, 2015).

In this paper, simple reliability analysis is applied to MSE walls designed using AASHTO LRFD. It is demonstrated that reliability analysis can be used in a simplified format by practitioners for MSE wall design. In addition, the results will serve as an indication on the appropriateness of load and resistance factors in AASHTO and CHBDC LRFD for MSE walls.

2 MSE WALLS DESIGN IN AASHTO

In geotechnical engineering the use of a global factor of safety (FS or FOS), often described as the allowable stress design (ASD) method, remains widely utilized. The choice of the FS relies largely on experience (Duncan, 2000). However, this conventional approach suffers several well-known weaknesses (Kulhawy et. al, 2006). In the FS method, the same FS value is commonly used without regard to several uncertainties, such as the method of analysis, load magnitude and frequency, material uncertainties and the method of investigation (Kulhawy and Phoon, 1996).

The uncertainty of MSE structures is less than most geotechnical ground improvement systems. This is because MSE structures utilize a backfill consisting of select granular material that has a lower degree of uncertainty when compared to the uncertainty associated with in-situ soils. In addition, the uncertainties with inextensible steel soil reinforcing is also low because of the stringent ASTM specifications that they are manufactured in accordance with. Further in MSE structures the pullout friction factors are determined following ASTM specifications. Because of these requirements there is a low failure rate for MSE structures that utilize inextensible soil reinforcing.

The design of MSE structures has progressed from designing using the ASD method to the Load and Resistance Factored Design (LRFD) method. The ASD method uses service loads and applies a factor of safety to a design case. In other words, the designer estimates the working or service load and then proportions the member to some allowable stress value.

The LRFD combines the calculation of the limit state for strength and serviceability with a probability approach applied to safety. The uncertainty is applied to both the load factor and to the resistance factor. It uses a procedure where the predictability of the load is modified using load factors and the predictability of the material strength is reduced using resistance factors. AASHTO LRFD manuals state that the resistance factor is a function of the method used to estimate the resistance and thus the model uncertainty is also included in the design process (Allen et al., 2009). The two design methods, ASD and LRFD can be expressed as shown in Equation [1] and Equation [2], respectively.

ASD: $$\Sigma Q_i \le \Sigma \frac{R_{ni}}{FS_i} \quad [1]$$

LRFD: $$\Sigma \gamma_i Q_i \le \Sigma \phi_i R_{ni} \quad [2]$$

Where Q is the load, R_{ni} is the nominal resistance, FS is the safety factor, γ_i is the load factor, ϕ_i is the resistance factor and i is associated with a loading or resistance.

The intent of the LRFD is to provide a more reliable level of safety through appropriate calibration of the resistance factor. A secondary effect of the LRFD would be to produce more economical designs than the designs that used the ASD. Because of the difficulty of determining resistance factors for geotechnical structures the LRFD has been calibrated to fit the ASD method. Theoretically this is supposed to provide structures that are similar no matter if the ASD or LRFD platform is used in the design. The AASHTO LRFD design method has been used to design MSE structures with proven success.

MSE structures are designed to assure that global, external, compound, and internal, stability is satisfied as shown in Figure 1. Global stability is sometimes referenced as overall stability. The failure surface for global stability passes outside the structural components of the retaining structure including the reinforced soil mass. MSE structures are designed considering the reinforced soil mass to be a rigid body. The rigid body is defined by a rectangular zone that extends from the top of the leveling pad to the top of the coping element and from the facing to the terminal end of the soil reinforcing. External stability includes sliding, overturning, and bearing resistance of the rigid body. Compound stability considers failure surfaces that pass through the reinforced soil mass and exit at the facing element. Internal stability considers failure of the soil reinforcement including rupture and pullout. Global and external stability are not a function of the MSE system components. The global and external analyses are the same regardless of the MSE system being used. e.g., small block, large block, segmental concrete panel, geosynthetic or steel soil reinforcing, etc. The analyses are only a function of the rigid body dimensions. Compound and internal stability are MSE system dependent. Compound stability is typically not considered for segmental concrete panel (SCP) systems because of the facing configuration and the connection that is employed to attach the soil reinforcing to the facing. Compound stability is considered in small and large block systems. Internal stability for an inextensible, steel, MSE system will be the focus of the example presented in this paper.

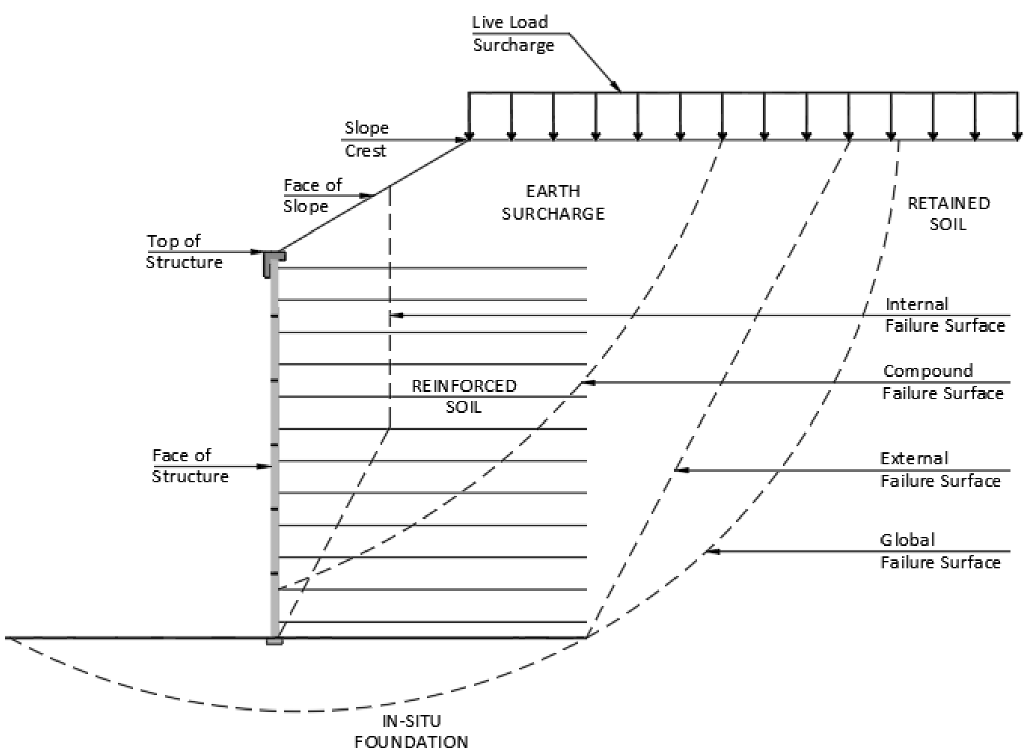

Figure 1. MSE Failure Surfaces for Stability Analyses

To provide a direct comparison between the ASD and the LRFD, the newly released TAC document, "Design, Construction, Maintenance, and Inspection Guide for Mechanically Stabilized Earth Walls" (TAC, 2017) provides a clear description that can be used for this paper. The design guide states that each LRFD limit state must satisfy the relationship shown in Equation 3.

$$\phi_g \cdot R_n \geq \Sigma I_i \cdot \eta_i \cdot \alpha_i \cdot Q_{ni} \quad [3]$$

Where ϕ_g is the geotechnical resistance factor, R_n is the nominal geotechnical resistance, I_i is the corresponding importance factor, η_i is the load combination factor, α_i is the load factor and Q_{ni} is the i^{th} nominal load. Where the factors I_i, η_i, and α_i are combined into an overall load factor denoted as γ_i. The link from ASD to LRFD is demonstrated in the flow chart shown in Figure 2.

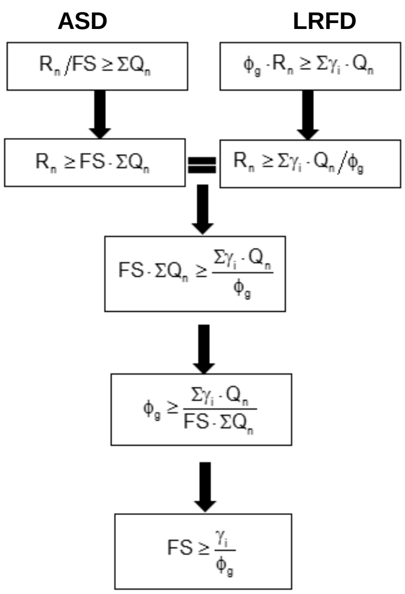

Figure 2 Relationship between ASD and LRFD (TAC, 2017)

Figure 2 demonstrates how the final choice of load and resistance factors is checked during LRFD calibration to match design outcomes based on past practice using ASD. In general terms, the ASD and LRFD are solved in terms of R_n and then set them equal. Based on a single load factor, the factor of safety can be checked to verify that there is a match. Due to rounding that is attributed with the LRFD (i.e., round up to the nearest 0.05) there will be slight discrepancies and an exact match may not occur. The method shown in Figure 2 can be used to calibrate unknown resistance factors with known load factors and known factors of safety.

As previously described to calculate the overall factor of safety for an LRFD design, the method shown in Figure 2 is used. The ASD and the LRFD equations are solved for the resistance factors and the inequalities are set equal. The required resistance factor can be solved for based on a known load factor, and a known factor of safety.

The resistance factors and the load factors from the AASHTO LRFD (2014) are shown in Table 1 and Table 2, respectively. Table 3 demonstrates a direct comparison of the factor of safety from the ASD platform using the method shown in Figure 2.

Table 1. Resistance factors related to MSE wall design [Table 11.5.7-1, AASHTO (2014)]

MSE Walls	Resistance Factor
Sliding	1.0
Tensile Resistance	0.75
Pull-Out Resistance	0.90

Table 2. Load factors related to MSE wall design [Table 3.4.4-1, AASHTO (2014)]

Load	Max.	Min.
Sliding	1.00	1.00
Tensile Resistance Strip Reinforcement – Static Loading	0.75	1.00
Pull-Out Resistance – Static Loading	0.90	0.90

Table 3. Load factors related to MSE wall design [Table 3.4.4-1, AASHTO (2014)]

Design Case	ASD Factor of Safety	LFRD Load Factor	LRFD Resistance Factor	Back Calculated FS	Relationship $FS = \gamma_1 / \phi_g$
Sliding	1.50	1.50	1.00	1.50	Yes
Tensile Resistance Strip Reinforcement – Static Loading	1.80	1.35	0.75	1.80	Yes
Pull-Out Resistance – Static Loading	1.50	1.35	0.90	1.50	Yes

The results shown in Table 3 demonstrate that the LRFD platform was calibrated to provide the same overall factor of safety that has been used in the ASD platform. Based on the AASHTO model, the designs from the ASD platform should equally compare to designs using an LRFD platform. It should be noted that the AASHTO LRFD edition that is in use today is different from the original AASHTO LRFD. AASHTO has refined the resistance values through interim specifications. The resistance values have been refined as more information and correlations have become available. In addition, AASHTO has updated the methodology to better fit the state of past practice. AASHTO has demonstrated the understanding that the implementation of the LRFD should not take away from the successful use of the ASD platform. The transformation of AASHTO from one interim to another has been completed with open collaboration with practicing engineers and MSE industry representatives in cooperation with the AASHTO T-15 committee and the FHWA.

3 RELIABILITY BASED METHODS AND PRESENT CHBDC 2014 CODE CHANGES

Reliability based design methods have been used in geotechnical applications (Christian et al., 1994; Tang et al., 1999; Duncan, 2000; Kulhawy et al. 2006). They provide a method for accounting for the effects of uncertainties, that will yield a consistent design risk, or probability of failure, when calibrated resistance and load factors are used.

Applying reliability theories in determining load and resistance factors of MSE walls has been advanced by several researchers including Javankhoshdel et al. (2018); Bathurst and Javankhoshdel (2016) and Bathurst et al. (2017). The general method involves formulating the limit state function, determining all the variables affecting the limit state function, assuming these variables are randomly distributed, and carrying out a Monte Carlo simulation. The probability of failure using Monte Carlo simulation is computationally expensive as it requires many samples. Moreover, it assumes perfect load and resistance models, and no variability in the contribution to bias.

In 2014, CAN/CSA-S6-14 introduced resistance factors for internal and external stability of MSE walls. The resistance factors are presented in Table 4. The values, as discussed by Fenton et al. (2015), were developed based on the random finite-element method, a method that combines non-linear finite element-reliability analysis with random field generation techniques. Load Factors shown in the CHBDC for MSE walls have not changed in the new code release.

Contrary to AASHTO, CHBDC (2014) does not offer a detailed method for the load evaluation for internal and external stability for various MSE retaining wall systems. This can lead to an inconsistent estimation of applied load. This will yield inconsistent wall designs depending on method of analysis, and inherently inconsistent probability of failure. El-Sharnouby et al. (2017) argued that using CHBDC (2014) in design of MSE walls is challenging due to the lack of a design method associated with the load and resistance factors. Further, that the cost comparison of MSE wall supply and construction, demonstrated that using the CHBDC (2017) yields up to 11% increase in cost when compared directly to AASHTO LRFD.

4 ESTIMATE OF STANDARD DEVIATION

Duncan (2000) presents methods for calculating the standard deviation, from sufficient data, published values, including the "three-sigma rule", and the graphical three-sigma rule. The three-sigma rule was described by Dai and Wang (1992) and was used in Duncan's study. The rule incorporates the estimation of the highest and lowest value of the parameter divided by a factor of six to determine the standard deviation. The three-sigma rule is shown in equation [4] and will be used in the example in this paper.

$$\sigma = \frac{HCV - LCV}{6} \qquad [4]$$

Where HCV is the highest conceivable value and LCV is the lowest conceivable value, in other words, the maximum and minimum values. The estimated maximum and minimum values used in this paper are based on the Author's experience and are shown in Table 6.

5 BETA FACTOR

Duncan (1999) used the lognormal reliability index, known as the beta factor (β_{LN}) to compute an approximate value of the Probability of Failure (P_f). This is based on the coefficient of variation (V) of the factor of safety determined using a plus one and minus one standard deviation between the HCV and LCV shown in equation [4]. The equations that are required to calculate the reliability index are shown in equations [5], [6], [7], and [8].

$$\sigma_f = \sqrt{\sum \left(\frac{\Delta F_i}{2} \right)^2} \qquad [5]$$

$$\Delta F_i = F_{max_i} - F_{min} \qquad [6]$$

$$V = \frac{\sigma_f}{F_{MLV}} \qquad [7]$$

$$\beta_{LN} = \frac{\ln\left(\frac{F_{MLV}}{\sqrt{1+V^2}} \right)}{\ln\sqrt{1+V^2}} \qquad [8]$$

Where σ_f is the standard deviation using a Taylor approximation. ΔF_i is the difference of the factor of safety of the maximum (F_{maxi}) and minimum (F_{mini}) expected values. Where, i, is equal to the number of variables. This is summed for each variable and is given by i. F_{MLV} is the expected likely factor of safety that is calculated using the most-likely values of each variable. The probability of failure is given by the normal distribution of Beta as shown in equation [9].

$$P_f = 1 - NORMSDIST(\beta_{LN}) \qquad [9]$$

The Army Corp of Engineers replaced the nomenclature of the "probability of failure" with the "probability of unsatisfactory performance". They also developed a chart that could be used to classify the probability of unsatisfactory performance based on the magnitude of the beta factor. As such, the beta factors can be used to compare the results between two different methods. Typically, one method is designated as the target index. If the beta factors are equal, then there is no difference between the two methods. If the beta factor is higher than the indexed beta factor it signifies that it is more conservative. If the beta factor is lower than the indexed beta factor it signifies that it is less conservative.

6 EXAMPLE: RELIABILITY INDEX FOR TYPICAL MSE WALLS

In the following example the internal stability of a simple MSE structure will be analyzed using a reliability approach and the ASD simplified design methodology. The simple MSE will consist of a structure with a level surcharge, no live load and inextensible, discrete, linear 2-Wire grid strips. A segmental concrete panel (SCP) with the dimensions of 1.524 m x 1.524 m will be assumed in the analysis. Typically, the SCP has a minimum of two rows of two columns of soil reinforcing. In other words, there are a minimum of 2 discrete, 2-wire elements in each row. In one of the examples the required area of steel will be determined. This may mean that the required area of steel is less than the combined area of the 2, 2-Wire elements. Internal stability will consider both rupture and pullout. The load and resistance factors used in the example for the AASHTO and CHBDC are given in Table 4.

Table 4. Load and Resistance Factors

Variable	AASHTO	CHBDC
Vertical Earth Pressure (EV)	1.35	1.25
Rupture Resistance Factor (ϕ_r)	0.75	0.90
Pullout Resistance Factor (ϕ_p)	0.90	0.60

Based on the values stipulated in Table 4, and using the relationship between the ASD and LRFD as previously discussed, and shown in Figure 2, the equivalent factors of safety for both the AASHTO and CHBDC can be determined. The equivalent factors of safety are shown in Table 5.

Table 5. Calculated Factor of Safety

Variable	AASHTO	CHBDC
Rupture	1.8	1.4
Pullout	1.5	2.1

It is obvious from Table 5 that there is a discrepancy between the AASHTO and CHBDC methods. The AASHTO factor of safety for rupture is more conservative than the CHBDC factor of safety. In contrast, the CHBDC factor of safety for pullout is more conservative than the AASHTO factor of safety. It is the opinions of the Author's that they should be equal. The premise for this opinion is based on the success of, and the low occurrence of unsatisfactory performance of structures that have been designed using the AASHTO design method. Therefore, to increase or decrease the conservatism, when there has been a lack of unsatisfactory performing structures would require a high degree of justification.

6.1 AASHTO Design Methodology

In the AASHTO internal stability design methodology, at the local level of each soil reinforcement, equilibrium must be satisfied using a working stress condition. In the working stress condition, it is assumed that the soil peak strength and tensile strength of the soil reinforcement are not exceeded and occur a very low strains (Allen et al., 2003). Based on this, the maximum unfactored load (T_{max}) at the local level is calculated using the AASHTO ASD as given by equation [10]. For the AASHTO LRFD the load factor for vertical earth pressure (γ_{EV}) is added to the equation as shown in equation [11].

$$T_{max} = K_r \cdot \sigma_v \cdot S_v = K_r \cdot (\gamma_f \cdot z) \cdot S_v \quad [10]$$

$$T_{max} = K_r \cdot \gamma_{EV} \cdot \sigma_v \cdot S_v = K_r \cdot \gamma_{EV} \cdot (\gamma_f \cdot z) \cdot S_v \quad [11]$$

Where S_v is equal to the tributary spacing of the reinforcement, σ_v is equal to the vertical pressure due to the soil self-weight immediately above the soil reinforcing, γ_f is equal to the unit weight of the backfill, z is equal to depth below the top of structure as measured at the back of the facing, and K_r is equal to the lateral earth pressure coefficient and is a function of the stiffness of the soil reinforcing, the depth below the top of the structure and the internal earth pressure coefficient, K_a. For the example in this paper, K_r will be calculated as shown in equation [12] and equation [13].

$$K_r = \left[1.7 - \left(\frac{1.7 - 1.2}{6 \cdot m}\right) \cdot z\right] \cdot K_a \rightarrow z \leq 6 \cdot m \quad [12]$$

$$K_r = 1.2 \cdot K_a \rightarrow z \geq 6 \cdot m \quad [13]$$

It should be noted that when using the simplified local method, the maximum force calculated at comparable elevations for structures of different heights is equal. This is not the true for structures that use the Coherent Gravity design method where the maximum force (T_{max}) is a function of the length of the soil reinforcing and eccentricity.

The factors of safety for rupture (FS_r) and pullout (FS_{po}) are determined using equation [14] an equation [15] respectively. In the LRFD method the factor of safety is replaced by the capacity demand ratio (CDR) as shown in equation [16] and equation [17]. The CDR is required to be greater than or equal to 1.0.

$$FS_r = \frac{T_{al}}{T_{max}} \quad [14]$$

$$FS_{po} = \frac{P_r}{T_{max}} \quad [15]$$

$$CDR_r = \frac{T_{al}}{T_{max}} \quad [16]$$

$$CDR_{po} = \frac{P_r}{T_{max}} \quad [17]$$

Where T_{al} is equal to the soil reinforcing long-term design strength and considers the effects of corrosion. Equation

[18] is for calculating T_{al} using the ASD and equation [19] is for calculating T_{al} using the LRFD where the resistance factor for rupture, ϕ_r, is added.

$$T_{al} = \frac{A_c \cdot F_y}{b} \cdot R_c \quad [18]$$

$$T_{al} = \phi_r \frac{A_c \cdot F_y}{b} \cdot R_c \quad [19]$$

Where, A_c is equal to area of reinforcement corrected for corrosion, F_y is equal to the minimum yield strength of steel, b is the unit width of the soil reinforcing and R_c is the coverage ratio.

The required area of steel to satisfy the factor of safety can be determined using equation [20] and [21] for the ASD and LRFD methodology respectively.

$$A_c = \frac{FS_r \cdot T_{max}}{R_c \cdot F_y} \quad [20]$$

$$A_c = \frac{\gamma_{EV} \cdot T_{max}}{\phi_r \cdot R_c \cdot F_y} \quad [21]$$

The pullout resistance of the soil reinforcing (P_r) in the ASD is given by equation

[18] and for the LRFD a resistance factor (ϕ_p) is added as shown in equation [19].

$$P_r = C \cdot \alpha \cdot F^* \cdot \sigma_v \cdot L_e \cdot R_c \quad [22]$$

$$P_r = \phi_p \cdot C \cdot \alpha \cdot F^* \cdot \sigma_v \cdot L_e \cdot R_c \quad [23]$$

Where C is the overall surface area geometry factor equal to 2.0, α is a scale correction factor equal to 1.0 for inextensible soil reinforcing, F* is the pullout friction factor, σ_v is equal to the vertical stress, L_e is equal to the length of the soil reinforcing in the resisting zone and R_c is the soil reinforcing coverage ratio. The equations can be rearranged to calculate the minimum required length of soil reinforcing in the passive zone to satisfy the pullout factor of safety as shown in equation [24] and [25] for both the ASD and LRFD methodologies, respectively.

$$L_e = \frac{T_{max} \cdot FS_{po}}{C \cdot \alpha \cdot F^* \cdot \sigma_v \cdot R_c} \quad [24]$$

$$L_e = \frac{T_{max} \cdot \gamma_{EV}}{\phi_p \cdot C \cdot \alpha \cdot F^* \cdot \sigma_v \cdot R_c} \quad [25]$$

6.2 Design Example Procedure

The goal of the design example in this paper is to calculate and compare the beta values between the AASHTO and CHBDC methods as described in Figure 2. The calculations in this example will be performed in conformance with the AASHTO ASD simplified design methodology. The calculated AASHTO beta values will be used as the index and compared to the calculated CHBDC beta factors. The following steps will be used to calculate the beta factor.

1. Determine most-likely, highest, and lowest values for the defined variables.
2. Calculate the minimum required density of steel to satisfy the factor of safety for rupture using the most-likely, maximum, and minimum values and the AASHTO factor of safety at each elevation of soil reinforcing.
3. Calculate the minimum required length of soil reinforcing to satisfy the factor of safety for pullout using the most-likely values and the AASHTO factor of safety at each elevation of soil reinforcing.
4. Using the length of soil reinforcing determined in Step 3, calculate the factor of safety for pullout using the most-likely, highest, and lowest values and the AASHTO factors of safety at each elevation of soil reinforcing.
5. Calculate the minimum required density of steel to satisfy the factor of safety for rupture using the most-likely values and the CHBDC factor of safety at each elevation of soil reinforcing.
6. Calculate the minimum required length of soil reinforcing to satisfy the factor of safety for pullout using the most-likely values and the CHBDC factor of safety at each elevation of soil reinforcing.
7. Calculate the Beta factor using the CHBDC model as the most-likely value and the AASHTO model maximum and minimum values at each elevation of soil reinforcing.
8. Compare beta values at each elevation of soil reinforcing.

6.3 Design Parameters

From equation [10], the magnitude of the factor of safety given in equation [14] is a function of the strength parameters of the soil backfill, i.e., unit weight (γ_f) and friction angle (ϕ_r). From equation

[18], the magnitude of the factor of safety given in equation [15] is a function of the friction angle (ϕ_r) of the soil backfill and the pullout friction factor F*. Note that the factor of safety for pullout is not a function of the unit weight of the backfill (γ_f) because it is in both the numerator and denominator of equation

[18].

Based on the reliability analysis described by Duncan and the experience of the Author's, the most-likely, maximum, and minimum values for the variables defined for rupture and pullout described above and that are used in this example are given in Table 6.

Table 6. Variables for Reliability Analysis

Variable	Most Likely	Maximum	Minimum
Unit Weight (kN/m^3)	19	25	16
Internal Friction Angle (deg)	34	45	30
Friction Factor F^*_0	2.50	4.00	1.50

(dim)	F^*_{20}	1.25	2.00	0.58

Using the Duncan three-sigma rule the values that will be used in the example, based on a plus one and minus one standard deviation, are given in Table 7.

Table 7. Variables - Three-Sigma Rule

Variable		σ	σ^{+1}	σ^{-1}
Unit Weight (kN/m^3)		1.5	20.5	17.5
Internal Friction Angle (deg)		2.5	36.5	31.5
Friction Factor (dim)	F^*_0	0.42	2.92	2.08
	F^*_{20}	0.24	1.49	1.01

6.4 Presentation of Results

The results of the AASHTO ASD analysis will be presented for both rupture and pullout for the simple MSE structure defined herein. The results for rupture will be presented based on the required area of steel that satisfies the appropriate factor of safety. The required area of steel for the CHBDC ASD and the most-likely value will be presented with the AASHTO ASD values. The CHBDC steel areas will be determined based on the CHBDC safety factor while the AASHTO steel areas will be determined using the AASHTO safety factor. The results for pullout will be presented based on a calculated minimum length of soil reinforcing that satisfies the appropriate factor of safety. The minimum length of soil reinforcing will be calculated at each elevation i.e., row, and will be based on the most likely values in conjunction with the respective factor of safety for each design code, i.e., AASHTO or CHBDC.

The minimum length of soil reinforcing will be based on the maximum calculated length that was determined at each row of soil reinforcing. This maximum length will then be applied to each soil reinforcing row. The factor of safety of pullout for each row will then recalculated. The length of soil reinforcing is the minimum that satisfies the factor of safety for pullout. It may not follow the typical requirements specified in AASHTO that the length of the soil reinforcing be equal to 70% of the height of the structure.

The required areas of steel will be presented for each elevation of soil reinforcing in Table 8. Results will be presented using the most-likely value (MLV), the plus 1 standard deviation value, and the minus 1 standard deviation value for each variable. In addition, the CHBDC most-likely values will be presented. The unit of measurement for the results are given in mm^2. At the far right of the table the AASHTO and CHBDC calculated lognormal beta values will be presented and are dimensionless.

Based on the results shown in Table 8 the CHBDC will underestimate the required area of steel when compared to the AASHTO because the resistance factor for rupture in the CHBDC is higher than the AASHTO resistance factor. To further demonstrate the effect of the lower area of steel the probability of unsatisfactory performance will be calculated for the simple MSE structure defined in this paper. The number of discrete 2-Wire elements for each row will be determined and the factor of safety and beta factors will be calculated. The minimum number of discrete elements in this method will be set equal to 2. The probability of failure is shown in Table 9.

Table 8. ASD Simplified Reliability Analysis for Rupture – Area of Steel (mm^2)

	AASHTO	Friction Angle		Unit Weight		CHBDC	AASHTO	CHBDC
Depth (m)	MLV	Max	Min	Max	Min	MLV	β_{LN} (dim)	β_{LN} (dim)
0.381	16.3	14.6	18.1	17.6	15.0	12.5	20.9	14.5
1.143	47.0	42.2	52.1	50.7	43.3	36.2	29.3	21.1
1.905	75.2	67.6	83.4	81.1	69.2	57.9	32.9	23.8
2.667	100.9	90.7	112.0	108.9	93.0	77.7	35.1	25.5
3.429	124.2	111.6	137.8	134.0	114.4	95.6	36.7	26.8
4.191	145.0	130.3	160.8	156.4	133.5	111.6	37.9	27.7
4.953	163.3	146.7	181.1	176.1	150.4	125.7	38.9	28.4
5.715	179.1	160.9	198.7	193.2	164.9	137.9	39.5	28.9
6.477	192.4	172.9	213.5	207.6	177.2	148.2	40.0	29.3
7.239	203.3	182.7	225.5	219.3	187.2	156.5	40.5	29.7
8.001	211.7	190.2	234.8	228.4	195.0	163.0	40.8	29.9
8.763	217.6	195.5	241.4	234.8	200.4	167.5	40.9	30.0

Table 9. ASD Simplified Reliability Analysis for Rupture – Factor of Safety

	SR elements		CHBDC	Friction Angle		Unit Weight		AASHTO	
Depth (m)	AASHTO	CHBDC	MLV	Max	Min	Max	Min	β_{LN} (dim)	P_f
0.381	2	2	19.84	22.08	17.89	18.39	21.54	1.000	0.0%

1.143	2	2	6.88	7.65	6.20	6.37	7.47	1.000	0.0%
1.905	2	2	4.30	4.78	3.87	3.98	4.66	1.000	0.0%
2.667	2	2	3.20	3.56	2.88	2.97	3.47	1.000	0.0%
3.429	2	2	2.60	2.89	2.34	2.41	2.82	1.000	0.0%
4.191	2	2	2.23	2.48	2.01	2.06	2.42	1.000	0.0%
4.953	2	2	1.98	2.20	1.78	1.83	2.15	1.000	0.0%
5.715	3	2	1.80	3.01	2.44	2.51	2.94	0.998	0.2%
6.477	3	2	1.68	2.80	2.27	2.33	2.73	0.994	0.6%
7.239	3	2	1.59	2.65	2.15	2.21	2.59	0.988	1.2%
8.001	3	2	1.53	2.55	2.06	2.12	2.48	0.980	2.0%
8.763	3	2	1.48	2.48	2.01	2.06	2.42	0.972	2.8%

The required length of soil reinforcing and factors of safety for pullout will be presented for each elevation of soil reinforcing given in Table 10. Results will be presented using the most likely value, the plus 1 standard deviation value, and the minus 1 standard deviation value for each variable. Further, the CHBDC most-likely values will be presented. The unit of measurement for the length of soil reinforcing is given in meters and the factors of safety are dimensionless. At the far right of the table the AASHTO and CHBDC calculated lognormal beta values will be presented and are dimensionless.

Table 10. ASD Simplified Reliability Analysis for Pullout – Length of Soil Reinforcing (m) and Factor of Safety (dim)

Depth (m)	AASHTO Length (m)	CHBDC Length (m)	AASHTO MLV	Friction Angle Max	Friction Angle Min	Friction Factor Max	Friction Factor Min	CHBDC MLV	AASHTO β_{LN} (dim)	CHBDC β_{LN} (dim)
0.381	4.877	5.791	1.86	2.07	1.68	2.18	1.55	2.66	3.08	7.03
1.143	4.877	5.791	1.81	2.01	1.63	2.12	1.50	2.59	2.91	6.77
1.905	4.877	5.791	1.75	1.95	1.58	2.05	1.45	2.50	2.72	6.49
2.667	4.877	5.791	1.69	1.88	1.52	1.98	1.40	2.41	2.51	6.17
3.429	4.877	5.791	1.62	1.80	1.46	1.91	1.34	2.32	2.28	5.81
4.191	4.877	5.791	1.55	1.72	1.39	1.82	1.27	2.21	2.02	5.41
4.953	4.877	5.791	1.46	1.63	1.32	1.73	1.19	2.09	2.21	5.29
5.715	4.877	5.791	2.54	2.82	2.29	3.01	2.06	3.42	4.61	7.87
6.477	4.877	5.791	2.80	3.12	2.52	3.34	2.26	3.62	4.84	7.65
7.239	4.877	5.791	2.96	3.30	2.67	3.56	2.37	3.71	4.86	7.28
8.001	4.877	5.791	3.01	3.34	2.71	3.64	2.37	3.67	4.66	6.72
8.763	4.877	5.791	2.91	3.23	2.62	3.57	2.24	3.48	4.20	5.92

7 DISCUSSION OF RESULTS

For the example presented in this paper, the area of steel required for the AASHTO analysis is larger than the area of steel required for the CHBDC analysis. There is 25% less steel required in the CHBDC model when compared to the AASHTO model. It is sometimes difficult to realize this if the designer does not consider all the different standard vendor soil reinforcing requirements, different wall heights, different material values (maximum and minimum) and different design methodologies. In other words, a very detailed parametric study is required. Further, there is an inherent conservatism in most systems that are supplied using steel soil reinforcing. This is due to the configuration of the soil reinforcing system and minimum steel requirements that are a function of corrosion. For instance, to provide stability to the SCP there are a minimum number of soil reinforcing elements that are required to be placed on the SCP. For instance, the requirements of the simple MSE system presented in this paper are that there is a minimum of 2, 2-Wire soil reinforcing strips required in each row of soil reinforcing. This can be seen in the number of soil reinforcing strips that are shown in Table 9. The area of steel for the system in this example, for a row that has 2, 2-Wire strips is equal to 177 mm^2. Based on the results presented in Table 8 the area of steel required for row 1 is 10 times lower than the actual area of steel that would be provided in the AASHTO method and 14 times lower than the area of steel that would be provided in the CHBDC method. Because of the system configuration constraints, the probability of unsatisfactory performance for rupture, using the AASHTO, is nearly equal to 0. As shown in Table 9, when the CHBDC is considered the probability of unsatisfactory performance increase at the bottom of the structure reaching 3%. This demonstrates that the resistance factor in the current edition of the CHBDC yields a safety level that is less than AASHTO.

Based on the example in this paper, coupled with the stringent ASTM requirements that most inextensible steel systems adhere to, there may be justification for an increase in the resistance factor because of the degree of uncertainty is less. The equivalent CHBDC resistance factor to match the same design in AASHTO would require the resistance factor to be set equal to 0.7 while maintaining a load factor equal to 1.25 as shown in equation [26] and [27], respectively.

CHBDC $$\frac{\gamma_{EV}}{\phi_r} = \frac{1.25}{0.70} = 1.80 \qquad [26]$$

AASHTO $$\frac{\gamma_{EV}}{\phi_r} = \frac{1.35}{0.75} = 1.80 \qquad [27]$$

The length of soil reinforcing required to satisfy the AASHTO pullout requirements was equal to 4.877 meters and was equal to 5.791 meters for the CHBDC pullout requirements. The example in this worksheet is for a structure that is relatively tall. Because of this, the length of soil reinforcing is large, and pullout will not control. In fact, as can be seen in Table 10 the factor of safety for the AASHTO is greater than 1.5 and for the CHBDC it is greater than 2.1. Therefore, there is no probability of unsatisfactory performance. As the structure decrease in height, pullout starts to control the design due to a decrease length of soil reinforcing. Based on the Author's experience with forensic analysis of failed MSE structures pullout has never been the contributing mode of failure. As described in this paper, the soil that is used in the reinforced mass is a granular material that is placed to a minimum 95% standard proctor under very stringent quality control specifications. Therefore, pullout should never control an MSE design. The equivalent CHBDC resistance factor to match the same design in AASHTO would require the load resistance factor to be equal to 0.83 while maintaining a load factor equal to 1.25 as shown in equation [28] and [29], respectively.

CHBDC $$\frac{\gamma_{EV}}{\phi_{po}} = \frac{1.25}{0.83} = 1.50 \qquad [28]$$

AASHTO $$\frac{\gamma_{EV}}{\phi_{po}} = \frac{1.35}{0.90} = 1.50 \qquad [29]$$

The consequences of artificially increasing the length of the soil reinforcing because of an incorrectly defined resistance factor will increase the amount of backfill and at the same time transportation cost and the construction time. The increased length of soil reinforcing has a profound influence on the cost of the project. The sustainability of the system decreases as the length of the soil reinforcing increases. Based on the example in this paper the resistance factor that has been proposed by the CHBDC needs to be increased to a value that is equivalent to the AASHTO value.

8 SUMMARY AND CONCLUSIONS

The analysis presented herein demonstrates that the simplified design method coupled with the load and resistance factors specified in AASHTO yields a low and acceptable probability of performance. The Duncan's method showed that the CHBDC method yields an unsatisfactorily performance for the rupture limit state. Based on the successful use of AASHTO in the design and analysis of MSE structures the values that are given in AASHTO should be incorporated into the CHBDC through the modification of the proposed resistance factors. A reduction in resistance factor should be done incrementally with consecutive code provisions.

9 REFERENCES

AASHTO, 1994, *LRFD Bridge Design Specifications*, American Association of State Highway and Transportation Officials, First Edition, Washington, D.C., USA.

AASHTO, 1996, *Standard Specifications for Highway Bridges*, American Association of State Highway and Transportation Officials, Sixteenth Edition, Washington, D.C., USA.

AASHTO, 1998, *LRFD Bridge Design Specifications*, with 2003 Interims, American Association of State Highway and Transportation Officials, Second Edition, Washington, D.C., USA.

AASHTO, 2002, *Standard Specifications for Highway Bridges*, American Association of State Highway and Transportation Officials, Seventeenth Edition, Washington, D.C., USA.

AASHTO, 2014, *LRFD Bridge Design Specifications*, with Interims, American Association of State Highway and Transportation Officials, Seventh Edition, Washington, D.C., USA.

AASHTO-AGC-ARTBA Joint Committee, Subcommittee on New Highway Materials, Task Force 27 Report (1990), "In Situ Soil Improvement Techniques", American Association of State Highway and Transportation Officials, Washington, D. C.Allen, T.M., Bathurst, R.J., Holtz, R.D., Walters, D.L. and Lee, W.F. (2003). "A new working stress method for prediction of reinforcement loads in geosynthetic walls." Can. Geot. J., 40(5), 976-994.

Allen, T., Christopher, B.R., Elias, V.E. and DiMaggio, J. (2001) *Development of the Simplified Method for Internal Stability Design of Mechanically Stabilized Earth (MSE) Walls,* Washington State Department of Transportation Research Report WA-RD-513.1.

Bathurst, R.J, and Javankhoshdel, S. (2017). "Influence of model type, bias and input parameter variability on reliability analysis for simple limit states in soil-structure interaction problems" In GEORISK, 11(1), pp. 42-54.

Bathurst, R.J, Javankhoshdel, S., and Allen, T. M. (2017). "LRFD calibration of simple soil-structure limit states considering method bias and design parameter variability" In Journal of geotechnical and geoenvironmental engineering, 10.1061/(ASCE)GT.1943-5606.0001735.

Canadian Standard Association. 2014. *CAN/CSA-S6-14 Canadian Highway Bridge Code*. Mississauga, ON: Canadian Standard Association.

Christopher, B. R., Gill, S. A., Giroud, J. P., Juran, I., Mitchell, J. K., Schlosser, F., and Dunnicliff, J., (1990). "Design and Construction Guidelines for Reinforced Soil Structure - Volume 1." FHWA-RD-89-043, Federal Highway Administration, Washington, D. C.

Duncan, J.M. 2000. "Factors of safety and reliability in geotechnical engineering." In *Journal of Geotechnical and Geoenvironmental Engineering*, 126(4), pp. 307-316.

Government of Canada (2017). Environment and climate change Canada. Retrieved from https://www.ec.gc.ca/

Fenton, G. F., Naghibi. F, Dundas, D., Bathurst, R. B., and Griffiths, D.V. 2015. "Reliability-based geotechnical design in 2014 Canadian Highway Bridge Design Code." In *Canadian Geotechnical Journal*, 53(2), pp. 236-251.

Javankhoshdel, S., Bathurst., R.J., and Cami, B. (2018). "Influence of model type, bias and input parameter variability on reliability analysis for simple limit states with two load terms." In Computer and Geotechnics , 97, pp. 78-89.

Kulhawy, F.H., Phoonn, K-K., Prakoso, W.A., and Hirany, A. 2006. "Reliability-based design for foundation for transmission line structure." 2006 Electrical Transmission Conference. Birmingham, A : American Society of Civil Engineers.

Kulhawy, F.H., Phoon, K-K. 1996. "Engineering Judgment in the evolution from deterministic to reliability-based foundation design." In *ASCE 1996: Uncertainty in the geological environment-from theory to practice- 1996American Society of Civil Engineers Conference.* NY, NY: American Society of Civil Engineers.

Mitchell, J.K. and Villet, W.C.B., "Reinforcement of Earth Slopes and Embankments", NCHRP Report No. 290, Transportation Research Board, Washington, D.C., 1987

Transportation Association of Canada. 2017. *Design, Construction, Maintenance and Inspection Guide for Mechanically Stabilized Earth Walls*. Ottawa, ON: Transportation Association of Canada.

Yu, Y., Bathurst, J. B. 2015. "Analysis of Soil-Steel Bar Mat Pullout Models in Journal of Geotechnical and geoenvironmental engineering, DOI: http://dx.doi.org/10.1061/(ASCE)GT.1943-5606.0001281.