LRFD pullout resistance calibration of coherent gravity method for steel reinforced MSE walls

Le calibrage de résistance de dégagement de LRFD de la méthode logique de pesanteur pour l'acier de renforcement des murs de MSE

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ABSTRACT

The current AASHTO LRFD Specifications require that the maximum load in reinforcements be calculated using the simplified method. Although alternative methods are permitted, they may be used only if calibrated resistance factors for the method developed. This paper describes the calibration of resistance factors for the internal stability design of metallically reinforced mechanically stabilized earth walls using the Coherent Gravity Method (CGM). The CGM is a common method used for allowable stress design, but it is not included in the current AASHTO LRFD specifications. Calibration of the CGM is conducted using Monte Carlo simulation of selected worldwide, well-documented case studies of full-scale, metallically reinforced earth (MSE) walls. Calibration was performed for the pullout resistance of steel reinforcements. Calibration was performed for conditions at the end of wall construction, and at the end of the design service life of the wall considering corrosion. Reliability indices (β) of 2.38 and 2.60 were calculated for pullout resistance of steel strips and steel grids, respectively.

RÉSUMÉ

Les spécifications courantes d'AASHTO LRFD exigent que la charge maximum dans les renforts soit calculée suivant la méthode simplifiée. Bien que des méthodes alternatives sont développés, elles peuvent être employées seulement si les facteurs de résistance calibrés pour la méthode se développaient. Cet article décrit le calibrage des facteurs de résistance pour la conception de stabilité interne des murs mécaniquement stabilisés par renforcements de la terre suivre la Méthode Logique de Pesanteur (MLP). La MLP est une méthode commune employée pour la conception d'effort permis, mais elle n'est pas incluse dans les caractéristiques courantes d'AASHTO LRFD. Le calibrage de la MLP est conduit utilisant la simulation de Monte Carlo d'études choisies de cas internationales et bien documentées des murs complets et métallique renforcés de la terre. Le calibrage a été effectué pour la résistance de dégagement des renforts en acier. Le calibrage a été effectué pour des conditions à la fin de la construction de mur, et à la fin de la vie de service d'esthétique industrielle du mur considérant la corrosion. Des index de fiabilité de 2.38 et de 2.60 ont été calculés pour la résistance de dégagement des bandes en acier et des grilles en acier, respectivement.

Keywords : LRFD, Monte Carlo calibration, MSE walls, coherent gravity design method resistance factor, strips, grids

1 INTRODUCTION

Since their introduction almost 40 years ago, MSE walls have been proportioned to satisfy safety criteria against (1) soil failure by sliding, bearing and overturning (i.e., external stability), (2) structural failure by pullout or rupture of the reinforcements in the reinforced soil mass (i.e., internal stability), and (3) structural failure at the connection between the reinforcement and the wall facing. In the LRFD Specifications (AASHTO 2007), these safety criteria are met by ensuring that the factored resistance exceeds the sum of the factored load effects. Analytically this is demonstrated in LRFD by:

$$\Sigma\gamma_i Q_{ni} \leq \phi R_n \tag{1}$$

where γ_i = load factor applicable to a specific load component i; Q_{ni} = a specific nominal load component type (i.e., vertical earth load, lateral earth load, surcharge and vehicle live load); ϕ = resistance factor for a particular failure mode and limit state; and R_n = nominal or ultimate resistance. A limit state is a condition in which a load or a combination of loads just equals the available resistance, so that the structure is at incipient failure as defined by the prescribed failure criterion. Each failure criterion is represented by an equation having the general form of Equation 1.

The load and resistance factors in Equation 1 are used to account for material variability, uncertainty in magnitude and application of the applied loads, design model uncertainty, and other sources of uncertainty. The objective in LRFD is to ensure that for each limit state, the available factored resistance is at least as large as the total factored load effects.

The LRFD Specifications (AASHTO 2007) require that the maximum load in reinforcements be calculated using the Simplified Method approach along with the prescribed resistance factors calibrated for the method. So while the LRFD Specifications allow "other widely accepted and published design methods" to be used "for calculation of reinforcement loads", alternative methods (e.g., Coherent Gravity) are permitted only "if the wall designer develops method-specific resistance factors for the method employed". Therefore, the objective of the study is to develop resistance factors for the internal stability design of metallically reinforced MSE walls using the Coherent Gravity Method.

2 DATABASE

A worldwide database of well documented MSE wall case studies was compiled from published sources and through personal communications. The database includes 17, full-scale metallically reinforced MSE walls (22 wall sections) as

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summarized in Table 1. Five of the walls are reinforced using steel grids and 12 of the walls are reinforced using steel strips. The walls were constructed between 1972 and 1998. The cross sections of the reinforcements for the wall sections are rectangular except for four sections (Waltham wall, Asahigaoka wall, Ngauranga wall and Bourron Marlotte wall) which are trapezoidal. Except for the Guildford Bypass Wall (Profiles A and B), all of the walls using steel strip reinforcements were constructed by the Reinforced Earth Company (RECO). All of the walls using bar mat reinforcements were constructed by VSL. All of the walls using welded wire reinforcements were constructed by Hilfiker Retaining Walls.

The wall sections were instrumented to evaluate their field performance in terms of wall movements and earth pressures, and to measure in-situ stresses and deformations of the steel reinforcements due to static earth loads during and for short-term periods after construction. For this calibration study, our interest focused on deformations and tensile stresses measured on the steel reinforcements.

3 DATABASE STATISTICS

The statistical characteristics in the database include information regarding wall geometries, the engineering properties of the reinforced backfill soil, and reinforcement type, dimensions and spacing. Tables 2 through 4 summarize the maximum, average and minimum of various parameters of the database. Table 2 indicates that wall heights in the database vary between 3.66 m and 18.90 m. For these walls, the ratio of the reinforcement length to the wall height (L/H) varies between 0.38 and 1.79. AASHTO (2007) recommends L/H ≥ 0.7, but the code permits L/H as low as 0.63 for high structures where H ≥ 12 m. The angle of friction of the backfill soils (ϕ_s) for walls in the database varies between 30^o and 44^o based on the results of laboratory tests. For the Simplified Method, AASHTO (2007) allows a maximum ϕ_s = 34^o unless the backfill is tested for frictional strength using triaxial or direct shear laboratory methods. Tables 3 and 4 summarize the range and the average of cross section and spacing of steel strips and grids. The two types of steel reinforcement have similar average vertical spacing of about 0.67 m.

4 LOAD AND RESISTANCE CALCULATIONS

The maximum tension load, and pullout and rupture resistance on a steel reinforcement using the CGM are calculated using equations defined by the AASHTO Standard Specifications (1996). The internal forces on a reinforcement are a function of the position and strength of the steel within the reinforced soil mass. The nominal tension load ($Q_n = T_{max}$) at each reinforcement level is calculated using:

$$Q_n = T_{max} = \sigma_h' A = K \sigma_v' A \quad (2)$$

where σ_h' = horizontal effective stress; A= tributary area of a steel strip or grid; K= coefficient of lateral earth pressure; and σ_v' = vertical effective stress. The CGM assumes an at rest lateral earth pressure coefficient at the top of a wall ($K = K_o$) that transitions to a Rankine active earth pressure coefficient (K_a) at a depth of 6 m and lower depths. The nominal pullout resistance ($T_{po} = P_r$) of a steel reinforcement is given by:

$$P_r = T_{po} = F^* \alpha \sigma_v' L_e C R_c S_h \quad (3)$$

where F* = pullout resistance factor, which is a function of passive and frictional resistance of the reinforcement (see Table 5); α = scale effect correction factor = 1; L_e = embedment length in the resisting zone; C = effective unit perimeter of reinforcement = 2; R_c = reinforcement coverage ratio; and S_h = horizontal spacing of reinforcement.

Table 1. Summary of wall sections

Wall	Steel type	Location	Reference
1974 UCLA Wall, CA, USA	Smooth steel strips	CA, USA	Richardson, et al. 1977
1976 Vicksburg WES Wall, MS, USA	Smooth steel strips	MS, USA	Al-Hussaini & Perry 1978
1981 Guildford Bypass Wall, Profile A, UK	Smooth steel strips	UK	Murray & Hollinghurst
1981 Guildford Bypass Wall, Profile B, UK	Smooth steel strips	UK	(1986)
1988 Algonquin Wall 1, IL, USA	Ribbed steel strips	IL, USA	Christopher (1993)
1981 Waltham Wall, UK	Mild ribbed steel strips	UK	Murray & Farra (1990)
1982 Asahigaoka Wall, Japan	Smooth steel strips	Japan	Bastick (1984)
1972 Lille Wall, France	Smooth stainless steel strips	France	Bastick (1984)
1980 Fremersdorf Wall, Germany	Ribbed steel strips	Germany	Bastick (1984)
1988 Minnow Creek Wall, IN, USA	Ribbed steel strips	IN, USA	Runser et al. (2001)
1990 Gjovik Wall, Norway	Ribbed steel strips	Norway	Allen et al. (2001)
1985 Ngauranga Wall, New Zealand	Ribbed steel strips	New Zealand	Boyd (1993)
1993 Bourron Marlotte Wall, France	Ribbed steel strips	France	Bastick et al. (1993)
1993 Bourron Marlotte Wall, France	Ribbed steel strips	France	Bastick et al. (1993)
1981 Hayward Wall, Section 1, CA, USA	Bar mat	CA, USA	Neely (1993)
1981 Hayward Wall, Section 2, CA, USA	Bar mat	CA, USA	Neely (1993)
1988 Cloverdale Wall, CA, USA	Bar mat	CA, USA	Jakura (1988)
1988 Algonquin Wall 3, IL, USA	Bar mat	IL, USA	Christopher (1993)
1988 Algonquin Wall 4, IL, USA	Bar mat	IL, USA	Christopher (1993)
1988 Algonquin Wall 5, IL, USA	Bar mat	IL, USA	Christopher (1993)
1985 Rainier Seattle Wall, WA, USA	Welded wire	WA, USA	Anderson, et al. (1987)
Houston Wall 17, TX, USA	Welded wire	TX, USA	Sampaco (1995)

Table 2. Statistical summary of wall geometries and backfill properties

Parameter	Wall height (m)	Reinforcement length (m)	Total soil unit weight (kN/m^3)	Mobilized angle of friction (degrees)
Min	3.66	3.05	16.0	30
Max	18.90	15.00	23.1	44
Average	9.39	7.32	19.9	36.8

The nominal tension loads and pullout resistances of steel reinforcing wall sections in the database were calculated using Equations 2 and 3. The loads and resistances were calculated at the end of a 75-year design service life considering the effects of corrosion using the AASHTO (2007) corrosion model as the basis for loss of reinforcement cross section. The AASHTO metal-loss model defines the rates at which first zinc, then steel, will be lost from a steel reinforcement section. Using the AASHTO (2007) specified minimum 86 μm thickness of zinc, the expected life of the zinc is 16 years, followed by 59 years of steel loss of strength before the 75 year design life is reached.

Table 3. Statistical Summary of steel strip reinforcement cross section and spacing

Parameter	Vertical spacing (m)	Horizontal spacing (m)	Width (mm)	Thickness (mm)
Min	0.30	0.31	39.9	0.6
Max	0.76	1.00	101.6	15.0
Average	0.67	0.69	68.8	5.4

Table 4. Statistical summary of steel grid reinforcement cross section and spacing

Parameter	Longitudinal steel				Transverse steel	
	S_v (m)	S_h (m)	Diameter (mm)	b (m)	Diameter (mm)	S_t (m)
Min	0.46	0.15	6.1	0.15	5.4	0.23
Max	0.76	2.05	12.9	1.22	9.5	0.61
Average	0.68	1.25	9.6	0.61	8.5	0.53

Table 5. Design values for F* (AASHTO, 2007)

Reinforcement type	Top of wall	Depth of 6 m and below
Smooth strips	0.4	0.4
Ribbed strips	$1.2 + \log(C_u) \leq 2.0$	tan (angle of friction)
Grids	$20\ (t/S_t)$	$10\ (t/S_t)$

Notes: C_u = uniformity coefficient of backfill; t = thickness of transverse bar; S_t = spacing of transverse bars

Table 6. Statistical properties of steel strip data sets

Variable	Mean	COV	Source
Predicted load	13.1 kN	0.72	This study
Measured load	15.9 kN	0.71	This study
Load bias	1.29	0.39	This study
Predicted pullout resistance	47.8 kN	0.94	This study
Pullout bias	2.28	0.60	D'Appolonia (1999)

Table 7. Statistical properties of steel grid data sets

Variable	Mean	COV	Source
Predicted load	26.8 kN	0.97	This study
Measured load	25.3 kN	0.91	This study
Load bias	1.08	0.68	This study
Predicted pullout resistance	160 kN	1.60	This study
Pullout bias	1.318	0.39	D'Appolonia (1999)

5 CALIBRATION USING MONTE CARLO ANALYSIS

Calibration of the CGM was conducted using the Level II probabilistic analysis method described in Allen et al. (2005). To accomplish the analysis, a limit state equation is developed that incorporates and relates all of the variables that affect the potential for failure of the reinforcements. The parameters of load and resistance are considered to be random variables, with the variation modeled using available statistical data. The required steps needed to conduct the calibrations are presented in the following sections.

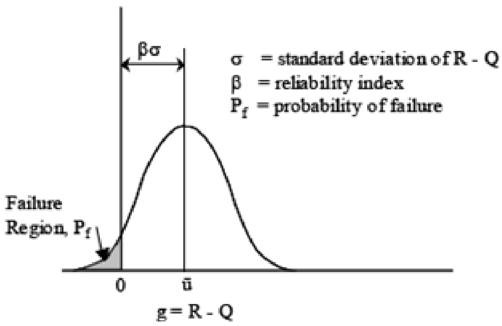

Figure 1. Probability of failure and probability index.

Step 1 – Develop Limit State Equations: The limit state equations include all the parameters that describe the failure mechanism and that are used for a deterministic design of metallic MSE wall reinforcements. The limit state equation is:

$$g = R - Q \geq 0 \tag{4}$$

where g = the random variable representing the safety margin; R = the random variable representing resistance; and Q = the random variable representing load. The magnitude of the load and resistance factors, and the difference between R and Q, are determined such that the probability of failure (P_f) that Q > R is acceptably small. Figure 1 illustrates this principle, in this case for normal distributions for load and resistance. P_f is typically represented by the reliability index (β) which represents the number of standard deviations of the mean of R – Q to the right of the origin. Using Equations 2, 3 and 4, the limit state equation for pullout of steel reinforcements using the CGM is:

$$g = R - Q = \phi F^* \alpha \sigma_v' L_e CR_c S_h - \gamma K \sigma_v' A \geq 0 \tag{5}$$

Step 2 – Statistically Characterize the Performance Data: The variable statistical parameters needed to perform the Monte Carlo analysis include the mean, standard deviation, coefficient of variation, and bias factor (measured value/predicted value). Statistical parameters of measured and predicted loads, and predicted pullout resistances of steel strips and steel grids are summarized in Tables 6 and 7, respectively. Figure 2 compares the maximum measured and calculated tension loads for steel strips. Figure 2 shows generally good agreement between the measured and predicted loads up to a load envelope of about 20 kN. Figure 3 compares the maximum measured and calculated tension loads for steel grids. Figure 3 shows generally good agreement between measured and predicted loads except for a few points. The bias of pullout resistance is determined using a database reported by D'Appolonia (1999).

Step 3 – Select Target β: Previous studies to assess the reliability inherent in ASD foundation and retaining wall design practice (e.g., D'Appolonia 1999; Allen et al., 2005) have reported that β ranges from about 3.5 to 2.3 depending on the type of structure. For the example calibration for pullout of steel grid presented in Allen et al. (2005), β_T = 2.3 was used for the Simplified Method based on recommendations presented in D'Appolonia (1999) to reflect the redundancy of structural elements in the MSE wall reinforced soil mass used for the analyses. For the calibration analysis herein, β represents the output of the Monte Carlo analysis and is compared to these ranges for acceptability.

Step 4 – Determine Resistance Factors: Using the limit state equation from Step 1 and the statistical properties compiled from Step 2, the resistance factor can be estimated through iteration to produce the desired magnitude for β (Step 3). For the calibrations described here, the Monte Carlo simulation method is used because the approach is adaptable and sufficiently rigorous to deal with the variability of the available data. It has also become the preferred approach for calibrating load and resistance factors for the LRFD Specifications. The analyses were conducted using the Lumenaut (2007) software.

6 MONTE CARLO SIMULATION

A Monte Carlo analysis to determine β for pullout of steel strips is discussed. The mean and standard deviation of the load bias (Table 6) are the input for the load. The mean and standard deviation of the pullout resistance bias (Table 6) are the input for the resistance. The distribution of the biases for both load and pullout resistance are lognormal. The random numbers for g (Equation 4) were generated using the statistical properties of the inputs from Table 6 for load and resistance, a load factor, γ = 1.35 (AASHTO, 2007 for static vertical earth pressure loads), and a resistance factor, ϕ = 0.9. For each Monte Carlo run, including the one discussed here, 10,000 iterations (i.e., g's) are generated. The iteration results are summarized in the form of a histogram and corresponding intervals are summarized in a table. A spread sheet, including the actual 10,000 iterations, is also generated as an output. The g values are then sorted and ranked in an ascending order. The cumulative probability at each g, P(g), is calculated as (g rank/(10,000+1)). Then the standardized normal value (z) of a P(g) is calculated using the NORMSINV function in Microsoft Excel where z = NORMSINV [P(g)]. The reliability index, β, is equal to (-z) at g = 0. Figure 4 shows the standardized normal variable (z) versus randomly generated g for the above input, and β = 2.38 corresponding to P_f = 8.7E-03.

Similar Monte Carlo analyses were performed to calibrate the pullout and rupture resistances of steel grids using the statistical properties summarized in Table 7. For ϕ = 0.9 at the end of a structure lifetime, the Monte Carlo simulation resulted in a P_f = 4.6E-03 and β = 2.60 as shown in Figure 4.

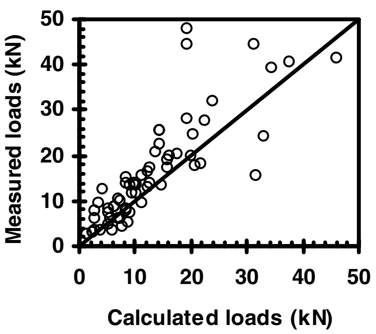

Figure 2. Comparing measured and calculated loads for steel strips.

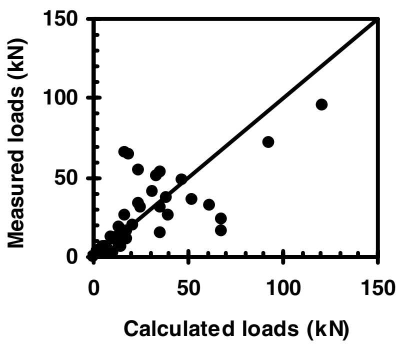

Figure 3. Comparing measured and calculated loads for steel grids.

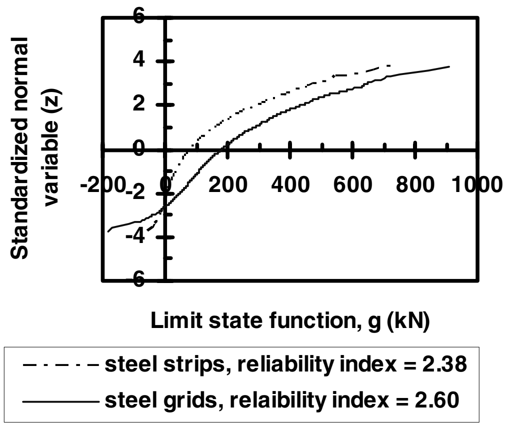

Figure 4. Monte Carlo analysis results for pullout resistance of steel reinforcements (γ = 1.35 and ϕ = 0.9).

7 CONCLUSIONS

The Monte Carlo calibration analyses result in β values consistent with the range for the pullout resistance of steel strips and steel grids from previous LRFD calibrations. The analysis shows that for ϕ = 0.9, reasonable β values of 2.38 and 2.60 are computed for pullout resistance of steel strips and steel grids, respectively. Based on the results of these analyses, ϕ = 0.9 is appropriate for the pullout resistance of steel strips and steel grids using the Coherent Gravity Method, which is the same value prescribed in the LRFD Specifications (AASHTO, 2007) for pullout resistance using the Simplified Method.

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