Comparison between AASHTO and CHBDC Design Methods for MSE Retaining Wall and its Implications on Transportation Agencies

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ABSTRACT

Mechanically Stabilized Earth (MSE) structures have been used in their current form since the early 1970s. MSE structures have become the solution of choice over traditional retaining wall systems due to their reduced material costs, ease of installation, and improved performance. This results in a retaining wall system that has a reduced carbon footprint when compared to other retaining wall systems such as Cast-in-Place wall systems. Design of MSE structures has progressed from using the Allowable Stress Design(ASD) method to the Load and Resistance Factored Design (LRFD) method. The American Association of State Highway and Transportation Official (AASHTO) implemented the LRFD method to design MSE structures in 2002 and has established load and resistance factors through calibration to the ASD method, experience and collaboration with the MSE industry. This paper will compare the design of an inextensible reinforced MSE wall system using the latest edition of Canadian Highway Bridge Code (CHBDC, CAN/CSA-S6-14) to the AASHTO (2014) LRFD Bridge Design Specification. This paper will demonstrate how the CHBDC new changes increase the cost of a typical MSE structure. Indirectly, it will demonstrate the present sustainability issues being faced with the current CHBDC design method including, an increase in the steel reinforcement required to be manufacture and the additional select MSE fill that will be required to be processed and shipped to site, resulting in an increase in the carbon footprint for the structure.

Keyword: Mechanically Stabilized Earth, Retaining Wall, sustainability, design, code, CHBDC, AASHTO, MSE, LRFD

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INTRODUCTION

It has been offered that climate change is causing an increase in water levels, melting of the arctic ice, degradation of permafrost and severe droughts. In 2017, The United Nation Food and Agriculture Organization issued in a report that approximately 12 million people are at risk of hunger due to recurring droughts. As part of the government effort to dampen carbon emissions, in 2016, Canada officially ratified the historic Paris Climate Change Agreement. Additionally, all Provinces and Territories will have to implement a carbon tax, or a cap-and-trade system by 2017 (Government of Canada, 2017).

Infrastructure projects inadvertently include processes that contribute to greenhouse gases. For example, according to the government of Canada, the Transportation sector accounts for 24% of greenhouse gas emissions making it the second largest contributing sector after the oil and gas sector (Government of Canada, 2017). It is reported that the emission from freight trucks has tripled since 1990. Despite the climate impact, public works projects remain focused on the lowest cost solution without considering the carbon footprint. Moreover, when changes are made to national codes, carbon footprint is seldom a consideration.

Inextensible Mechanically Stabilized Earth (MSE) structures are retaining walls that are reinforced with inclusions that consist of horizontal placed steel elements. 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 world wide (Yu and Bathurst, 2015). MSE structures have become the solution of choice over traditional retaining wall systems due to their reduced material costs, ease of installation, and improved performance. MSE structures have a reduced carbon footprint when compared to other retaining wall systems such as Cast-in-Place wall systems.

This paper will compare the design of an inextensible reinforced MSE wall system using the current edition of the Canadian Highway Bridge Code (CHBDC, CAN/CSA-S6-14) to the current American Association of State Highway and Transportation Official (AASHTO) Load and Resistance Factored Design (LRFD) Bridge Design Specification. The design philosophy that has been used to develop the new resistance factors and consequence factors for the CHBDC will be explained. In addition, it will demonstrate how the changes effect a standard MSE design and the resulting cost implications.

The intent of this paper is to demonstrate how the new CHBDC design methodology affects the MSE design. It will assist in understanding how the code changes increase the cost of a system that has been successfully used under the AASHTO design code for more than 40 years. In addition, this paper will provide information that will help with the calibration of the new CHBDC code. Indirectly, it will convey the present sustainability issues being faced with the current CHBDC design method.

EVOLUTION OF MSE WALLS DESIGN IN AASHTO

In geotechnical engineering the use of a global factor of safety (FS or FOS), often described as 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 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). It is worth noting, however, 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 soil. This manifest itself in a very low failure rate due to internal stability issues.

MSE wall design has transitioned from vendor developed design methodologies to designs that follow a recognized governing specification such as AASHTO design code. In June of 1987, the United States National Cooperative Highway Research Program (NCHRP) published report 290, Reinforcement of Earth Slopes and Embankments. The 290-Report provided a comprehensive compilation of various earth reinforcement technologies. The 290-Report demonstrated that there was not a uniform design methodology associated with MSE technology. In August of 1990, the In-Situ Soil Improvement Techniques under Task Force 27 Report was published. This report was developed in combination with the Joint Committee of AASHTO, the Associated General Contractors (AGC), and the American Road and Transportation Builders Association (ARTBA). Task Force 27 committee was formed to address the need for guidance on the use of MSE including the application, specifications, design, and construction. The Task Force 27 Guidelines were adopted by AASHTO and presented in the 1994 Interim of the Specifications for Highway Bridges. In November of 1990, the Federal Highway Administration (FHWA) released RD-89-043, Design and Construction Guidelines for Reinforced Soil Structures (Christopher et., al, 1989). This document contained a new approach to designing the internal stability of MSE walls, utilizing the global stiffness of the soil reinforcements to estimate the reinforcement loads. The design methodology for MSE that was presented in this document was called the “Stiffness Method.” This method was included in the 1994 interim AASHTO specification as an acceptable alternative design method. Until 1994, the AASHTO specifications used the tieback wedge or coherent gravity approaches to estimate stresses in MSE structures. There was some variance in the methodologies to account for different reinforcement types as specified in report 290 (Mitchell and Villet, 1987; Berg et al., 1998). The AASHTO specification was not considered a unification design specification but a general design specification that could be used with both inextensible and extensible soil reinforcing (Allen et al., 2001). There was consensus that the code still lacked a method that considered the variance in the internal earth pressure coefficient between different MSE technologies using an analogous method. In 1999, AASHTO adopted the Stiffness Method, changing the name to the Simplified Method, and noted that other widely accepted methods could be used to determine the internal stress in the MSE structure. The transition from a vendor specific to AASHTO Simplified Method has resulted in consistent designs by practitioners.

The Allowable Stress Design platform was used before the LRFD platform. The AASHTO LRFD Bridge Design Specifications was implemented in 1994. In 1996 AASHTO had a major update to Section 3, Loads and Factors, especially Article 3.11. Section 3 contained updated definitions and required load factors that affected retaining wall design. Section 3.11 had major updates to the requirements for the lateral earth pressure in retaining wall design. Up to 1996, the LRFD provisions for retaining walls were based on the Allowable Stress Design methods. This method was practiced in the 1996 AASHTO Standard Specifications for Highway Bridges, 16th Edition.

As discussed above, the design of MSE structures has progressed from designing using the Allowable Stress Design (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 then proportions the member to some allowable stress value. In the ASD, the Factor of Safety is independent of the method that is used to determine the resistance factor. In contrast 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 loads 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.

ASD: $\Sigma Q_i \leq \Sigma \frac{R_{ni}}{FS_i}$ **Equation 1**

LRFD: $\Sigma \gamma_i Q_i \leq \Sigma \phi_i R_{ni}$ **Equation 2**

Where Q is the load, R_{ni} is the nominal resistance, FS is the safety factor, γ is the load factor, ϕ is the resistance factor and i is associated loading or resistance.

The intent of the LRFD was to provide a more consistent 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. Using the, "calibration to fit", method 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. MSE structures consider the system to be a ridged body. The rigid body can be considered part of the structural component. 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 the facing element. Internal stability considers failure of the reinforcement including rupture and pullout. Global and external stability are not a function of the MSE system. The analyses are the same regardless of the system being used. e.g., small block, large block, segmental concrete panel, geosynthetic or steel soil reinforcing, etc. It is only a function of the rigid body dimensions. Compound and internal stability are system dependent. Compound stability is typically not considered for segmental concrete panel 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 is the focus of the parametric study in this paper.

AASHTO provides detailed procedure to calculate external stability for sliding condition, eccentricity, overturning and applied bearing pressure. For internal stability, AASHTO provides detailed method to calculate the nominal load for pullout and rupture conditions, and nominal resistance for pullout condition. Success of AASHTO can be attributed to practitioners using the same method for load evaluation along with same load and resistance factors for various wall products.

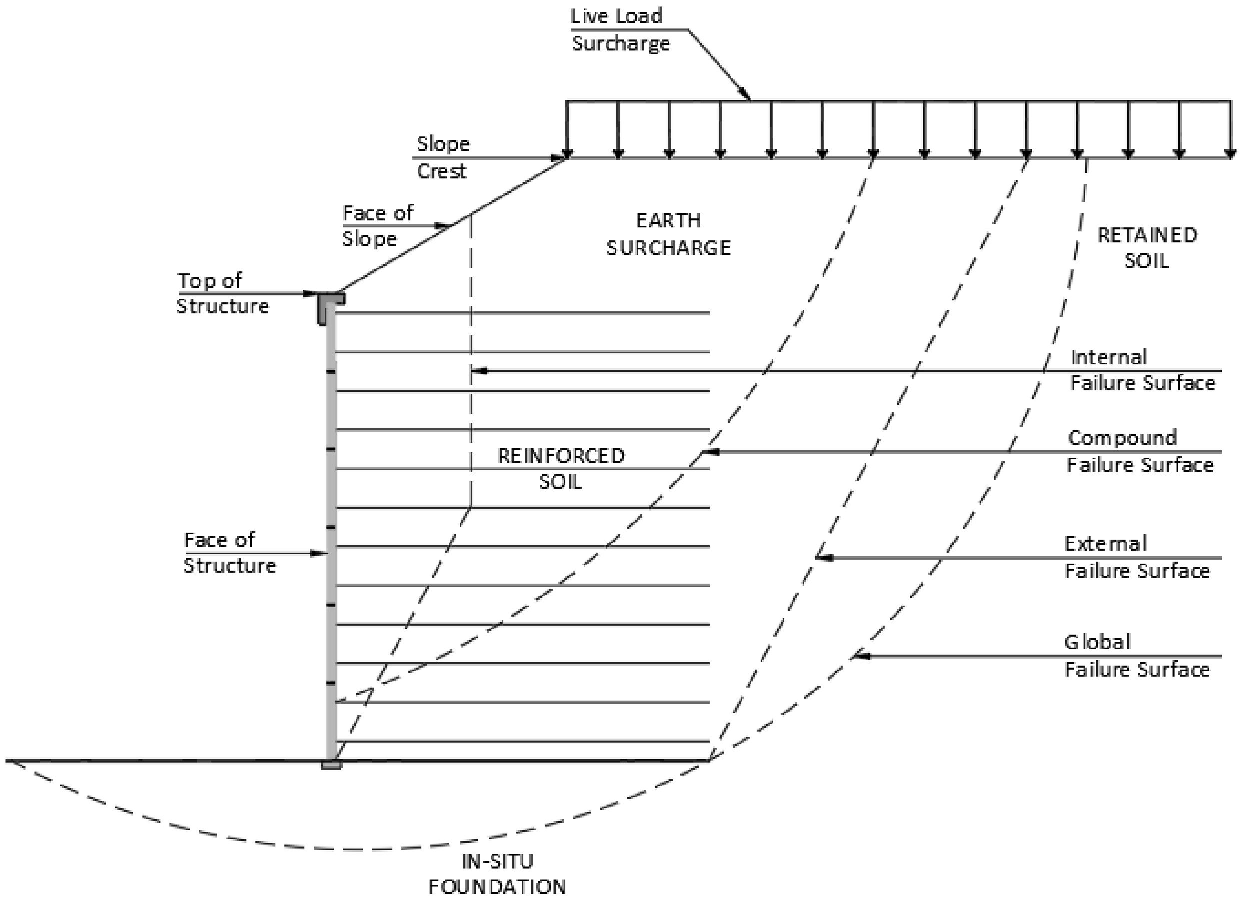

Fig 1. MSE Failure Surfaces for Stability Analyses

Table 1 shows the typical factors of safety for external and internal stability according to ASD in AASHTO, 1996. These values are consistent with the values shown in the literature and in the state of practice, and have been developed based on engineering judgement, experience, and to a lesser extent, on consequence of failure.

Table 1. Table 3.4.4-1 AASHTO (2002) Factors of Safety Related to MSE Wall Design

Load	Factor of Safety
Overturning	2.0
Sliding	1.5
Steel Rupture (0.55Fy)	1.8 (From ASD for Steel Design)
Pull-Out	1.5

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 Equation 3.

$$\phi_g \cdot R_n \geq \Sigma I_i \cdot \eta_i \cdot \alpha_i \cdot Q_{ni}$$ **Equation 3**

Where ϕ_g is the geotechnical resistance factor, R_n is the nominal geotechnical resistance, Ii 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.

In this guide, 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 Figure 2. Figure 2 shows 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, you solve the ASD and LRFD 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 that can be used to calibrate unknown resistance factors with known load factors and known factors of safety is demonstrated in Figure 2.

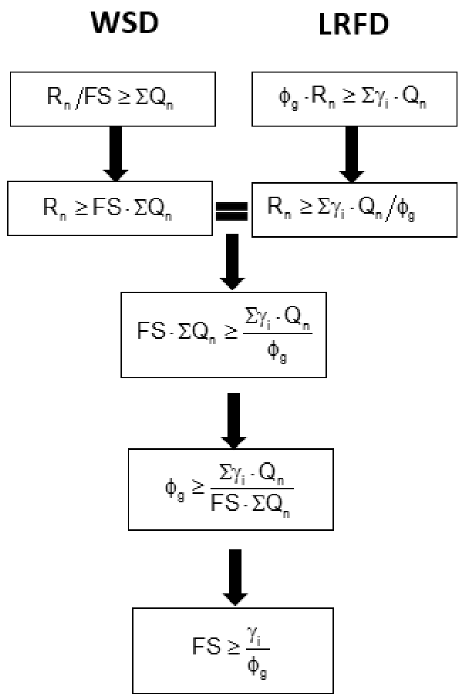

Figure 2. Relationship between ASD and LRFD (after TAC, 2017)

To calculate the overall factor of safety for an LRFD design, the method shown in Figure 2 is used. The ASD and the LRFD equation is solved for the resistance factors and the inequalities are set equal. The required resistance factor is solved for 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 Tables 2 and 3, respectively. Table 4 demonstrates a direct comparison of the factor of safety from the ASD platform using the method shown in Figure 2.

Table 2. Resistance factors related to MSE wall Design [Table 11.5.7-1, AASHTO (2014)]

MSE Walls	Resistance Factor
Sliding	1.00
Tensile Resistance Strip Reinforcement – Static Loading	0.75
Pull-Out Resistance - Static Loading	0.90

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

Load	Max.	Min.
Vertical Earth Load (EV)	1.35	1.00
Live Load (LL)	1.75	1.00
Horizontal Earth Pressure (Active) (EH)	1.50	0.90

Table 4. Relationship between AASHTO ASD (2002) and AASHTO LRFD (2014)

Design Case	ASD	LRFD Load Factor	LRFD Resistance Factor	Back-Calculated FS	Relationship FS = γ_1 / $Ø_g$
Sliding	1.5	1.50	1.00	1.5	Yes
Rupture	1.8	1.35	0.75	1.8	Yes
Pull-Out	1.5	1.35	0.90	1.5	Yes

The results 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 though 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 done by open collaboration with practicing engineers and MSE industry representatives with the AASHTO T-15 committee and collaborative with the FHWA.

PRESENT CHBDC 2014 CODE CHANGES

Reliability based design methods have been gaining acceptance in geotechnical applications. They provide a method for accounting for the effects of uncertainties (Duncan, 2000), that will yield a consistent design risk, or probability of failure, when calibrated resistance and load factors are used (Kulhawy et al. 2006). In 2014, CAN/CSA-S6-14 introduced resistance factors for internal and external stability of MSE walls. The resistance factors are presented in Table 5. The values, as explained 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.

Table 5. Resistance factors related to MSE wall design (Table 6.2, CHBDC)

Application	Limit State	Test Method/Model	Degrees of Understanding		
			Low	Typical	High
Internal MSE Reinforcement	Rupture	Analysis	0.75	0.80	0.85
		Test	0.85	0.90	0.95*
	Pull-Out	Analysis	0.35	0.40	0.50
		Test	0.55	0.60	0.65*
Retaining Systems	Base Sliding	Analysis	0.70	0.80*	0.90
	Overturning	Analysis	0.45	0.50	0.55*
	Connections	Test	0.65	0.70	0.75*

*values used in parametric study.

Table 6. Load factors related to MSE wall design (Table 3.1, CHBDC)

Load	Max.	Min.
Dead Load	1.25	0.80
Live Load	1.70	0.90

In the AASHTO LRFD, the load factors for the unit weight of the soil are dependent on the location of the soil. For an MSE structure the load factor for the reinforced backfill is equal to 1.35 and the load factor for the in-situ retained soil is equal to 1.50. This demonstrates the uncertainty of the in-situ retained backfill when compared with the reinforced backfill. In the CHBDC, the load factor is 1.25 for both the reinforced backfill and the retained backfill. Therefore, the factored load in the AASHTO is higher than the factored load in the CHBDC.

The relationship between the ASD to the CHBDC is shown in Table 7. For completeness, the AASHTO load and resistance values are also shown. Using the method demonstrated in Figure 2, the Factor of Safety (FS) can be back-calculated and the percent increase or decrease determined. As can be seen in Table 7, there is a definite inconsistency between the AASHTO LRFD and the CHBDC LRFD methods. For external stability, CHBDC produces a 13% and 4% higher FS for overturning and sliding, respectively, compared to AASHTO. For internal stability, CHBDC produces 27% lower FS for rupture and 28% higher FS for pullout. To produce equivalent results between AASHTO and CHBDC, resistance factors are back calculated as per Figure. 2 and presented in Table 7. It should be noted that it is customary to round the resistance factor up to the nearest 0.05. It is worth noting, from an internal stability perspective and failure of MSE structures, pullout is commonly not a mode of failure while rupture of the reinforcing is.

Table 7. Relationship between ASD and LRFD from the AASHTO design codes*

Design Case	AASHTO			CHBDC			± FS	CHBDC
	ASD FS	Load Factor	Resistance Factor	Load Factor	Resistance Factor	Back-Calculated FS		Suggests values**
Overturning	2.0	1.50	0.75	1.25	0.55	2.27	+13%	0.75
Sliding	1.5	1.50	1.00	1.25	0.80	1.56	+4%	0.95
Rupture	1.8	1.35	0.75	1.25	0.95	1.31	-27%	0.70
Pullout	1.5	1.35	0.90	1.25	0.65	1.92	+28%	0.95

*Unit weight of backfill in AASHTO and CHBDC are 19 kN/m³ and 22 kN/m³, respectively.
**suggested values to yield FS consistent with AASHTO LRFD (2014).

DESIGN COMPARISON BETWEEN AASHTO AND CHBDC

The commercial software program MSEW (3.0) developed by Adama Engineering, Inc., was used for the parametric study. MSEW is an interactive software program for the design and analysis of mechanically stabilized earth walls. It was developed to follow FHWA and AASHTO guidelines but can be manipulated by the user to follow any design code. The parametric study will analyze two typical MSE structure configurations at four different design cases, as shown in Figures 3 and 4, and Table 8. The MSE structure configurations consist of a level back-slope (Flat-Top) with a traffic live load and an MSE structure with a 2:1 infinite back-slope. These two configurations can be considered the general case for most transportation related MSE structures. The design wall heights that will be used in parametric study are equal to 3, 6, 9, and 12 m. The wall height is defined as the distance from the top of the leveling pad to the top of the coping element. In the absence of project specific backfill strength parameters both the CHBDC and the AASHTO provide typical unit weights and internal friction angles to be used in the design. The soil reinforcing lengths is determined in the parametric study and is based on satisfying external and internal stability capacity demand ratios equal to, or greater than 1.0. The lengths of the soil reinforcing will be specified in increments of 305 mm lengths. The parametric study design cases parameters are defined in Table 9.

Table 8. Wall Design Cases

Case	Wall Height (m)	Slope Condition
Case A	3	Flat
Case B	6	Flat
Case C	9	Flat
Case D	12	Flat
Case E	3	2:1
Case F	6	2:1
Case G	9	2:1
Case H	12	2:1

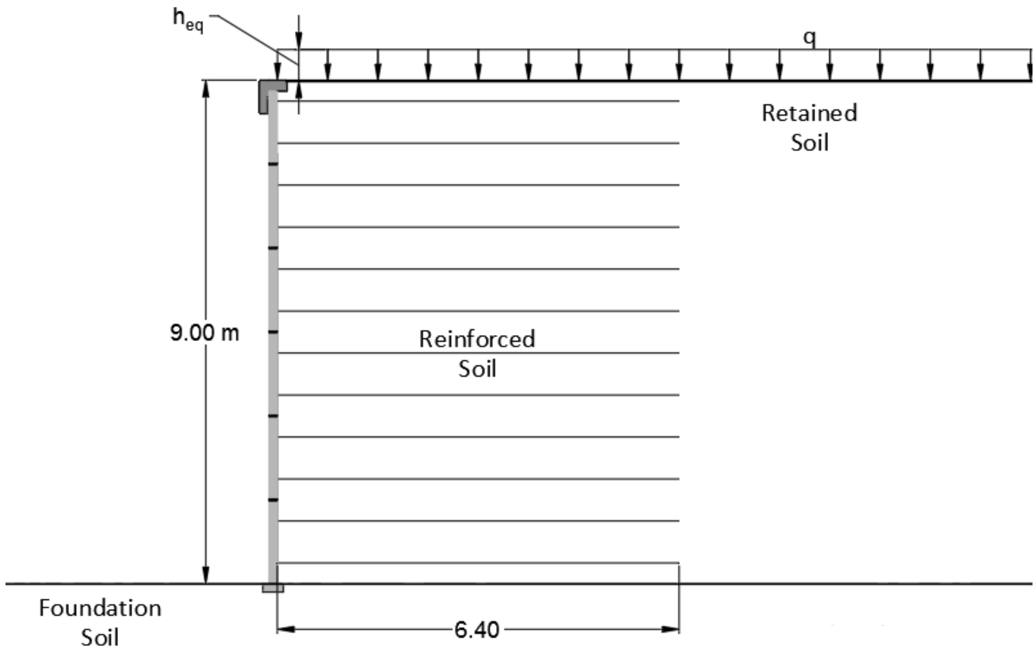

Figure 3. Typical Flat-Top Wall Section.

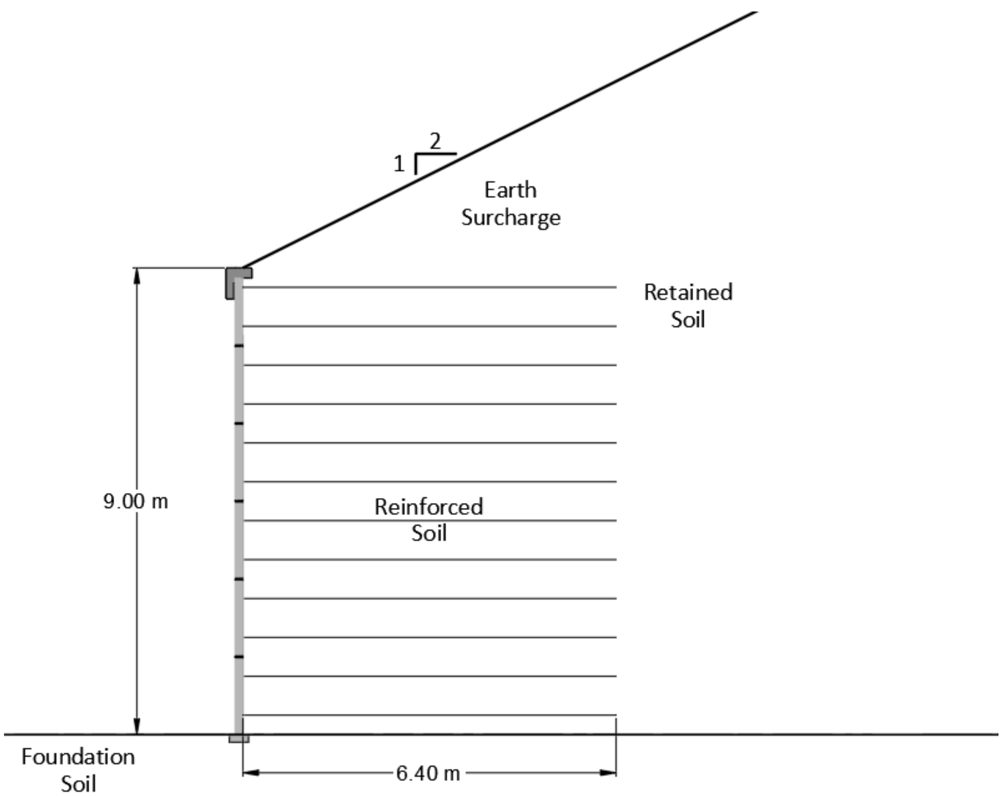

Figure 4. Typical 2:1 wall section.

Table 9– Design inputs for AASHTO and CHBDC MSE design runs

Design Input	AASHTO	CHBDC	Comment
Height (m)	3,6,9,12	3,6,9,12	Multiple heights considered
Wall area (m^2)	250	250	-
Flat Top	Yes	Yes	Flat and Sloping considered to see the difference between the two scenarios
Sloping Top (Infinite) (m)	Yes	Yes	Flat and Sloping considered to see the difference between the two scenarios
Select PHI Angle (deg)*	34	35	Differences Between the code when soil properties are not known
Random Phi Angle (deg)	30	30	-
Foundation Phi Angle (deg)	30	30	-
Select Fill Unit Weight kN/m³ *	19	22	Differences Between the code when soil properties are not known
Random Fill Unit Weight kN/m³	19	22	Differences Between the code when soil properties are not known
Seismic (a/g)	0.00	0.00	Not considered
Design Life (years)	100	100	
Live Load (kPa) *	11.4	17.6	Differences Between the code

*Default values for each code

Wall Comparison Summary

1) Mat length increase of 9% on average with CHBDC design for flat top walls
2) Mat length increase of 15% on average with CHBDC design for sloping top walls
3) Pull-Out design case with CHBDC design:
    a) Decreased in density by 12% for 3m tall flat top wall
    b) 19% Increase for 6m tall flat top wall
    c) 33% Increase for 9m tall flat top wall
    d) 15% Increase for 12m tall flat top wall
    e) 20% Increase for 3m tall Sloping top wall
    f) no change in 6m, 9m and 12m tall Sloping top wall
4) Increase in kilograms per square meters of steel with CHBDC design
    a) 4% for 3m tall flat top wall
    b) 27% for 6m tall flat top wall
    c) 17% for 9m tall flat top wall
    d) -1% for 12m tall flat top wall
    e) 9% for 3m tall sloping top wall
    f) -13% for 6m tall sloping top wall
    g) 0% for 9m tall sloping top wall
    h) 2% for 12m tall sloping top wall

CARBON FOOTPRINT AND COST COMPARISON

To understand the effect of the design on the carbon footprint a generic 250 m² wall is analyzed for each of the 3, 6, 9, and 12 m wall cases. It is assumed that the unit selling price for the wall is equal to \$250/m² and the generic installation cost of the gravel is equal to \$50/m³. Based on the parametric study the CHBDC design is compared to the AASHTO design.

A comparison of backfill quantity and CO_2 emissions from processing and haulage of materials is presented in Table 10. The emissions are calculated as 0.0104 kg of CO_2 per one kg of gravel, 0.197 kg of CO_2 per tonne-km hauling of transport truck (Earthshift 2013), and 1633 kg of CO_2 per tonne of steel manufacturing (Kundak et al. 2009). Haul distance is assumed 200 km. As can be seen, design using CHBDC results in cost increase up to 11% and additional CO_2 emissions of 42,168 kg.

Table 10. CO_2 emissions difference and cost using CHBDC (AASHTO reference design)

Emissions	Case A	Case B	Case C	Case D	Case E	Case F	Case G	Case H
CO_2 Emission (Kg) variance-backfill only	8432	17031	8516	8348	0	25491	34035	42160
CO_2 Emission (Kg) variance-steel manufacturing only	16.4	110.3	69.45	-4.1	36.8	-53.1	0	8.2
Total CO_2 variance	8448	17142	8585	8344	37	25438	34035	42168
Install cost Variance	+5%	11%	+5%	+2%	+2%	+7%	+10%	+10%

SUMMARY AND CONCLUSIONS

Climate change is having an adverse effect on communities around the world. Cost in infra-structure projects, however, remain the major influencer in decision making. Moreover, design approach developed in national codes seldom considers the environmental impact of code changes. This paper provided an overview of the AASHTO code evolution from a global factor of safety method to a LRFD method, both of which are based on deterministic methods and stem mostly from experience. It was demonstrated that using either AASHTO methods yields the same design output. The latest edition of CHBDC introduced resistance factors that are based on reliability methods. The paper demonstrated using a parametric analysis that CHBDC and AASHTO yield different design solutions. Design using the CHBDC increases the cost by up to11% and increases the carbon footprint to approximately 42,168 kg CO_2 per 250m^2 wall case. MSE structures have performed well using AASHTO state-of-practice since 1970s. To yield a structure that is consistent with the past state-of-practice and with AASHTO, the resistance factors in CHBDC can be calibrated to achieve the same over-all factor of safety, as demonstrated in this paper.

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