Steel Strip Reinforced Soil Walls at Working Stress Conditions

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ABSTRACT
For the purpose of validating design methods for Mechanically Stabilized Earth (MSE) retaining walls, it is customary to predict the maximum reinforcement loads from full-scale instrumented walls under working stress conditions. This paper presents such estimations using the design approach called the Soil Reinforcement Interaction (SRI) method for ten steel strip reinforced walls to demonstrate its prediction capabilities under the working stress conditions. The method was developed for estimating the internal stability of walls reinforced with both extensible and inextensible reinforcements. The method accounts for the soil-reinforcement interaction using a non-empirical analytical model. Apart from estimating the maximum reinforcement load, the method is capable of estimating the reinforcement load distribution and load transmitted to the facing connection. The method is also capable of accounting for the toe resistance and explaining its influence on the reinforcement load distribution unlike the existing design approaches.

RÉSUMÉ
Dans le but de valider les méthodes de calcul pour les murs de soutènement MSE, il est courant de prédire les charges de renforcement maximales des murs instrumentés à pleine échelle dans des conditions de tension de travail. Cet article présente de telles estimations en utilisant l'approche de conception appelée Interaction de renforcement de sol (SRI) pour dix murs renforcés de bandes d'acier pour démontrer ses capacités de prédiction dans les conditions de contrainte de travail. La méthode a été développée pour estimer la stabilité interne des murs renforcés avec des renforts extensibles et inextensibles. La méthode tient compte de l'interaction sol-renforcement en utilisant un modèle analytique non empirique. Outre l'estimation de la charge maximale de renforcement, la méthode permet d'estimer la répartition de la charge de renforcement et la charge transmise à la connexion en regard. La méthode est également capable de tenir compte de la résistance aux orteils et d'expliquer son influence sur la répartition de la charge de renforcement, contrairement aux.

1 INTRODUCTION

In general, the current approaches for designing Mechanically Stabilized Earth (MSE) wall can be broadly categorized as working stress based methods such as the Simplified Stiffness method (Allen and Bathurst 2015) and Ehrlich and Mitchell (1994) or limit state based approaches such as the AASHTO Simplified method and Leshchinsky and Boedeker (1989) method. For validating the working stress based design methods, the most common approach is to compare the maximum reinforcement loads estimated from the design method against measurements obtained from full-scale instrumented walls (e.g., Allen and Bathurst 2015). The validations are limited to the maximum reinforcement load and does not consider the reinforcement distribution which controls the other potential failure modes such as pullout failure or the loads developed at the facing. Despite the reasonably accurate estimations for the maximum reinforcement load under working stress conditions, this approach of validation does not confirm their ability to predict the ultimate failure state or load. This can be partly attributed to the limited number of walls loaded to failure, whereas a relatively large database of instrumented walls are available to confirm behaviour under working stress conditions. The ability to predict the ultimate state is an important consideration in particular for wall reinforced with inextensible reinforcements – more important than confirming the prediction accuracy of the maximum reinforcement load at working stress conditions. It should also be recognized that computed factor of safety (or the safety margin) depends on both working stress and ultimate conditions. The limitations of the current methods in estimating the ultimate state is discussed in the companion paper Weerasekara (2018) and in Weerasekara et al (2017).

Notwithstanding the above limitation, this paper discusses the use of Soil-Reinforcement Interaction (SRI) method for modeling steel strip reinforced walls under working stress conditions. As discussed above, full validation of a design method should include range of loading conditions that span from working stress conditions to ultimate failure. The SRI method is neither categorized as a working stress nor a limit state based method, as this method is capable of estimating the wall performances at all stage of loading. For further details related to validation of the SRI method under ultimate state, readers are referred to Weerasekara et al (2017). This paper discusses the model validation under working stress conditions for walls reinforced with smooth and ribbed steel strips. Due to the space limitations, walls reinforced with extensible reinforcements, welded wire meshes and bar mats are not discussed.

2 SOIL-REINFORCEMENT INTERACTION METHOD

The SRI method consists of the following three sub-models:

(a) SRI Friction Model – Accounts for the soil-reinforcement interface friction;
(b) SRI Local Model – Accounts for the soil-reinforcement interaction in each reinforcement
(c) SRI Global Model – Accounts for the equilibrium and interaction of multiple reinforcements in a reinforced soil mass.

A schematic representation of the SRI Friction model is shown in Figure 1.

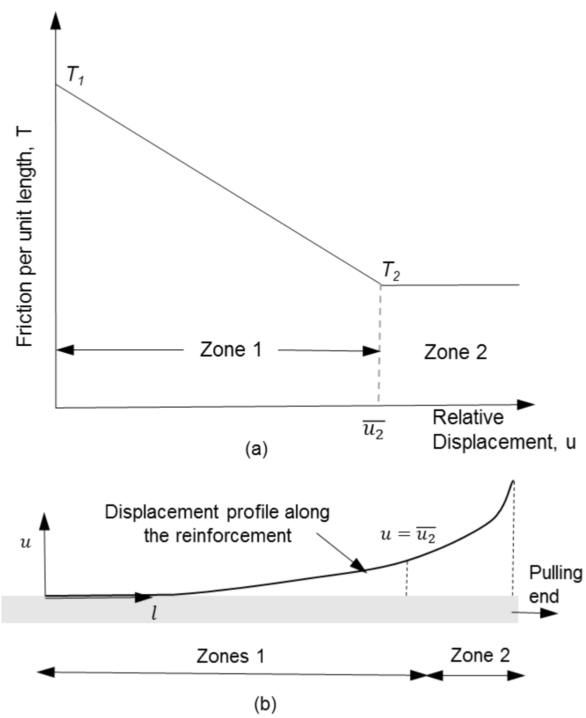


Figure 1. Schematic representation of the SRI Friction model

The maximum friction acting on the reinforcement per unit length (T_1) can be derived using the following equation.

$$T_1 = \frac{2bH\gamma \tan\varphi'_g}{1 - [2(1+\upsilon)/((1-2\upsilon)(1+2K_0))] \tan\varphi'_g \tan\psi_{max}} \quad [1]$$

$$\psi_{max} = 6.25(I_D(Q - ln\sigma') - R) \quad [2]$$

where b is the width of the reinforcement, H is the burial depth, γ is the unit weight of the soil overburden, φ'_g is the soil-reinforcement interface friction angle, υ is the Poisson's ratio of soil, ψ_{max} is the peak angle of dilation and K_0 is the "at-rest" lateral earth pressure coefficient estimated from the Jaky (1944) equation. As the magnitude of soil dilation also depends on the mean effective stress, Bolton (1986) classical stress-dilatancy framework was introduced to express the dilation in terms of relative density (I_D) and mean effective stress σ'. In all tests discussed in this paper, Q and R parameters were selected as 9 and 1, in the absence of experimental results to determine these parameters for each backfill type. The SRI method considers the increase in friction from constrained soil dilation at small displacements. According to the model, this additional friction from soil dilation is expected to decrease gradually and becomes negligible at a displacement of $\overline{u_2}$. At this displacement, the interface friction per unit length (T_2) is given by the following:

$$T_2 = 2bH\gamma \tan\varphi'_g \quad [3]$$

The value of $\overline{u_2}$ is typically obtained from experimental observations and generally considered to be about 150 times d_{50}. In the absence of d_{50} values, a value that ranges from 50 mm to 150 mm was selected based on the description of the backfill type. Typically, the estimated wall performance is relatively insensitive to the value selected for $\overline{u_2}$.

It is important to recognize that Equation (1) and (2) were derived using known soil mechanic theories, and all input parameters can be independently determined from pullout tests or direct shear tests. An interface friction angle of 22 degrees was considered for full-scale instrumented walls reinforced with ribbed steel discussed in this paper. However, the actual interface friction angle is expected to vary based on the backfill soil type. Unless reported, the interface friction angle of smooth steel strips was taken as 0.4 times the soil friction angle.

Once the interface friction behavior is known, the SRI Local model is used to determine the interaction between reinforcement and soil. This is achieved by considering the equilibrium at element level, which results in an analytical model that relates the relative displacement, reinforcement strain and mobilized frictional length along the reinforcement. The SRI Local method was validated by modeling a number of pullout test (Weerasekara et al., 2017).

Once the soil-reinforcement interaction occurring at each reinforcement level is known from the SRI Local model, the SRI Global model is used to assess the stability of the entire MSE wall. The model considers the resisting forces/moment from the reinforcements and driving forces/moments from earth pressures, surcharge and compaction, etc. As the wall displacement increases from the at-rest condition, the driving moment about the wall base is expected to decrease. Concurrently, the resisting moment is expected to increase with the wall displacement/rotation until tensile rupture or pullout of reinforcement occurs. The MSE wall will be in equilibrium when the resisting moment is equal to the driving moment. The difference between horizontal driving forces and summation of reinforcement loads is the load developed at the toe (i.e., toe resistance).

2.1 Wall Deformation Pattern

As stated above, the method requires the wall displacement to be incrementally increased until equilibrium of forces/moments is achieved. In other words, the wall displacement pattern should be known. For wall with a full-height facing panel and sufficient toe embedment, as an initial assumption, it is reasonable to consider the wall to rotate about its base. If the SRI model indicate that a large toe resistance is required to yield this form of wall deformation and if such magnitude of resistance cannot be sustained by the wall embedment, a translational displacement should be added to the rotational movement. This is equivalent to replacing the fixity at the toe with a compliant soil spring. For incremental panel facings or modular block facings, the wall deformation pattern depends on the capacity of the facing panel to transmit shear forces to panel/blocks immediately below or above. The optimum height for transmitting shear forces can be obtained using the hinge height concept introduced in the National Concrete Masonry Association (NCMA 1997) guidelines. The hinge height is the equivalent height of an unjointed facing that is fully efficient in transmitting moment through the height of the facing. For an incremental panel walls, hinge height is generally considered equal to the panel height. Due to the toe resistance, the wall panels are expected to rotate about its base up to the hinge height. The influence of wall embedment vanishes above the hinge height panel, and the wall facing is considered to undergo translational movement above the hinge height. This pattern of wall deformation is generally consistent with wall deformation patterns observed in full-scale instrumented walls with incremental panel facings. Nonetheless, it should be recognized that the hinge height depend on the interlocking characteristics between panels, and it is difficult to generalize a unique value for all types of facing panels. For the incremental panel walls with a nominal toe embedment (partial embedment of the bottom panel), a hinge height of 3 m was considered. For walls with significantly smaller or larger wall embedments, appropriate hinge heights were selected based on the information provided in the source documents.

3 FULL-SCALE INSTRUMENTED WALLS

The following presents the maximum reinforcement load estimations using the SRI method for ten full-scale instrumented walls with smooth and ribbed steel strips. For comparison purposes, the maximum reinforcement loads predicted using the AASHTO Simplified method (AASHTO 2002) are also shown. Although only the maximum reinforcement load is considered, the SRI method is capable of predicting the reinforcement load distributions including the load transmitted to the connection. The input parameters used in the SRI model are given in Table 1. The soil input parameters such as the soil unit weight and plane strain friction angle were adopted from the original publications or from details included in Allen and Bathurst (2003). For the AASHTO method, the maximum reinforcement load was estimated using the triaxial friction angle while the SRI method relies on the plane strain friction angle. For all steel reinforcements, a tensile modulus of 200,000 MPa was considered, unless this value is reported in the original publication.

3.1 Minnow Creek Wall (2001)

At the time of construction, 16.9 m high steel strip reinforced wall was the tallest MSE retaining wall constructed for the Indiana Department of Transportation. The details of the wall were given in Runser (1999) and Runser et al. (2001). The wall was subject to several studies including numerical modeling by Damien et al. (2015). The instrumented section of the south abutment wall contained 11 facing panels and 22 rows of reinforcement with a vertical spacing of 0.75 m. The horizontal spacing varied from 0.3 m at the bottom to 1.0 m at the top of the wall. Excluding the bottom five reinforcement layers, the reinforcement length was 11.9 m which was equivalent to 0.7 times the wall height (see Figure 2). The reinforcement length of the bottom five layers were increased to 15.5 m for reducing the bearing pressure in the foundation soil. The reinforcements consisted of ribbed steel strips of 50 mm wide and 4 mm thick.

Table 1. Key input parameters for the SRI model

Wall	Type	φ'_g (deg)	φ'_s (deg)	Dr (-)
Minnow Creek Wall	Ribbed	22	40	0.70
Lille Wall, France	Smooth	16	49	0.80
Asahigaoka, Japan	Smooth	16	40	0.65
Fremersdorf, Germany	Ribbed	22	40	0.75
WES Wall	Smooth	18	41	0.40
Gjovik, Norway	Ribbed	22	41	0.80
Bourron Marlotte Wall	Ribbed	22	40	0.75
UCLA Wall	Smooth	16	40	0.65
Algonquin Wall	Ribbed	22	43	0.80

φ'_g – Interface friction angle φ'

φ'_s – Soil friction angle (plane strain)

D_r - Relative density

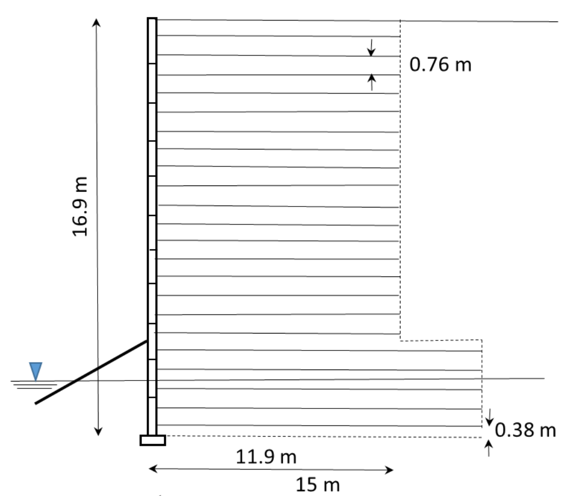


Figure 2. Wall geometry– Minnow Creek Wall

The backfill soil within the reinforced zone consisted of poorly graded sand with gravel with an average dry unit weight of 20.8 kN/m^3 and an average moisture content of 4.8%. The peak friction angle of 38° was measured from six large-scale consolidated-drained triaxial compression tests. As per Allen and Bathrust (2003), the estimated plane strain friction angle was 40 deg. Piezometers indicated that water level was at approximately 2 m above the toe of the wall.

The maximum reinforcement loads estimated using the SRI and AASHTO methods are shown in Figure 3. The plot also indicates the connection loads estimated using the SRI method. The SRI method does not predict any loads be transmitted to the facing about 5 m from the wall base, although the actual measurements indicate loads in the range 10 to 25 kN/m above this height. As pointed out by Damien et al (2015), some of the connection loads are due to the downdrag forces acting between the panels and soils. The SRI method is not able to estimate the connection loads generated due to downdrag forces.

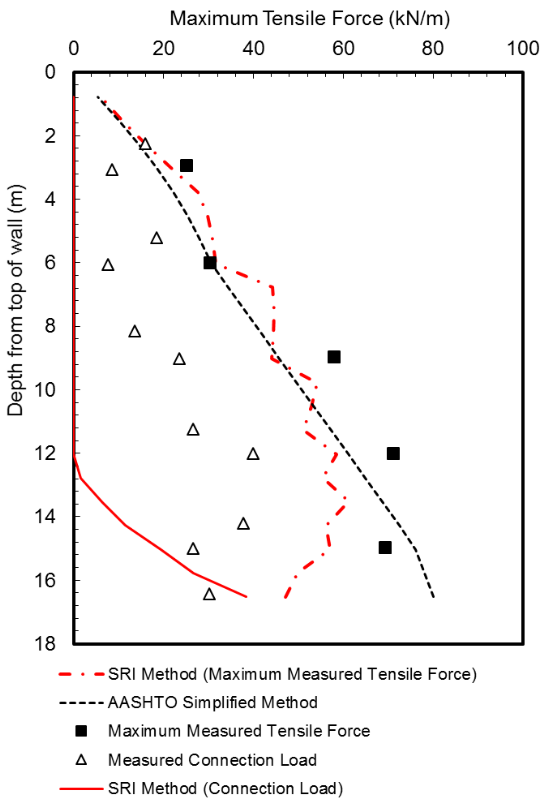


Figure 3. Measured and estimated maximum reinforcement and connection loads from the SRI and AASHTO methods - Minnow Creek Wall.

3.2 Lille, France (1972)

This reinforced earth bridge abutment wall was constructed in 1972 near Lille, France and the details were given in Bastick (1984) and Allen and Bathrust (2003). The wall was 6 m in height constructed using smooth steel strips and with precast concrete facing panels. The smooth steel strips were 1.5 mm x 80 mm. The wall geometry is shown in Figure 4. The wall backfill consisted of gravelly sand, and Allen and Bathrust (2003) reported an equivalent peak plane strain friction angle of 49 deg, whereas the measured triaxial friction angle was 44 deg. Although the compaction details were not provided, the relatively high friction angle implies that backfill was properly compacted. The measured loads and predictions using the SRI and AASHTO methods are shown in Figure 5.

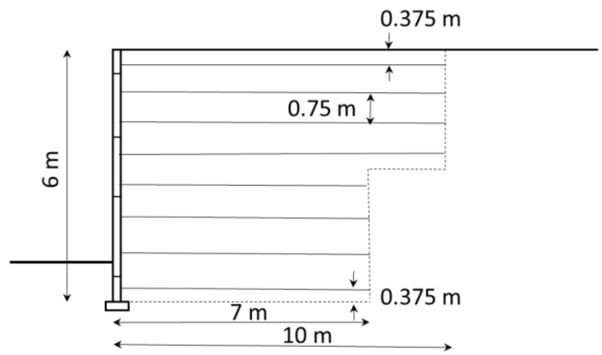


Figure 4. Wall geometry– Lille Wall

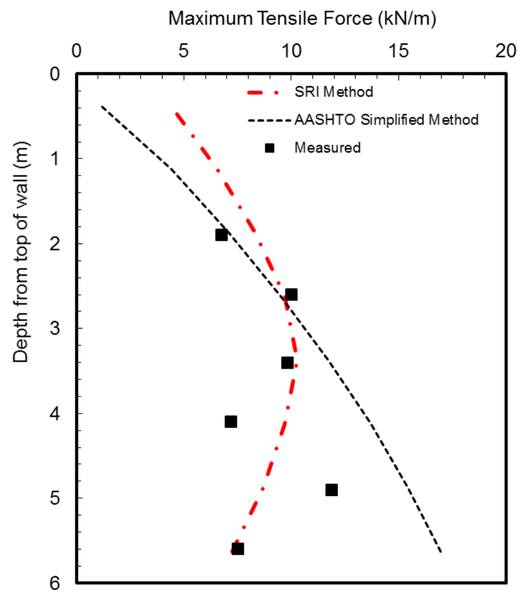


Figure 5. Measured and estimated maximum reinforcement loads from the SRI and AASHTO methods - Lille Wall.

3.3 Asahigaoka, Japan (1982)

The 12 m high smooth steel strip reinforced wall was constructed in Asahigaoka, Japan. The wall consisted of concrete panel facings and 1 m of soil surcharge. The details of the walls were reported by Bastick (1984). The wall consisted of precast concrete facing panels either 180 mm or 220 mm thick. The backfill material was reported as granular with a friction angle of 36 deg and a cohesion of 18.6 kPa, however the details of testing were not reported. Assuming these tests were either direct shear or triaxial tests, an equivalent plane strain friction angle of 40 deg was considered. The soil unit weight was reported as 17.7 kN/m^3. The tensile strength of smooth steel strips were reported as 440 MPa. The geometry of the wall is shown in Figure 6, and the predictions using the SRI and AASHTO methods are shown in Figure 7.

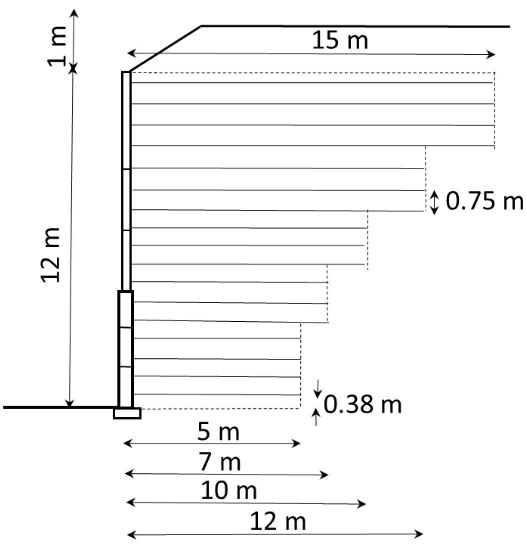


Figure 6. Wall geometry– Asahigaoka Wall

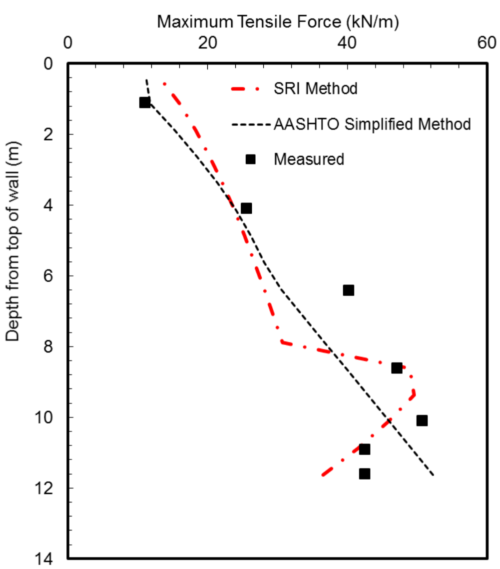


Figure 7. Measured and estimated maximum reinforcement loads from the SRI and AASHTO methods – Asahigaoka Wall

3.4 Fremersdorf, Germany (1980)

The 7.3 m high wall was constructed in 1980 at Fremersdorf, Germany, and the details were provide in Thamm (1981) and Allen and Bathurst (2003). The wall was reinforced with ribbed steel strips of 60 mm x 5 mm, and incremental concrete panel facings. The wall backfill consisted of peaty sand and a 2 m wide free draining medium gravel zone immediately behind the facing. Allen and Bathurst (2003) have estimated a plane strain friction of 40 deg for the soil backfill. The soil unit weight of 19.6 kN/m^3 was measured in situ after compacting 0.375 m thick soil layers using a 90 kN vibratory roller. The geometry of the wall is shown in Figure 8, and the predictions are in Figure 9.

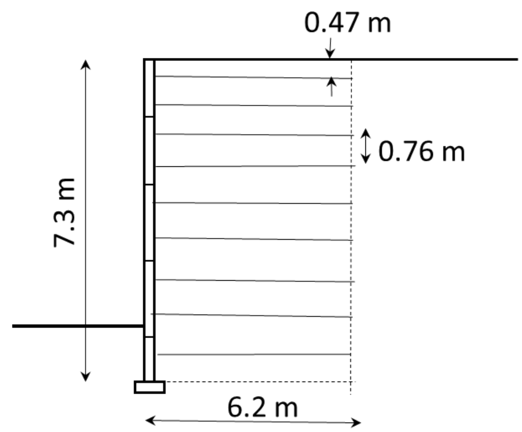


Figure 8. Wall geometry– Fremersdorf Wall

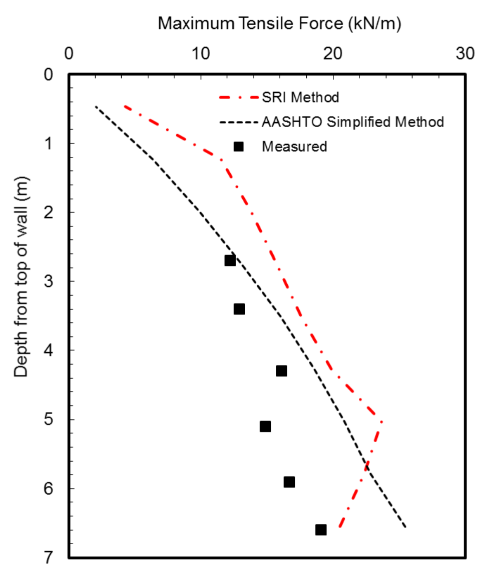


Figure 9. Measured and estimated maximum reinforcement loads from the SRI and AASHTO methods - Fremersdorf Wall.

3.5 WES Steel Strip Wall (1976)

A full-scale test wall of 3.66 m high was constructed in 1976 in a three-sided pit excavated at the US Army Waterways Experiment Station and the details are presented in Al-Hussaini and Perry (1978). The wall was reinforced with six layers of smooth galvanized steel strips 0.635 mm thick, 102 mm wide, 3.1 m long and with spacings of 0.77 m (horizontal) and 0.6 m (vertical). A yield strength of 352 MPa and an initial modulus of 214.6 GPa were measured from uniaxial tensile tests conducted on steel strips. The wall utilized high-strength aluminum facing panels which were connected together with hinge-type connections. The wall geometry is shown in Figure 10.

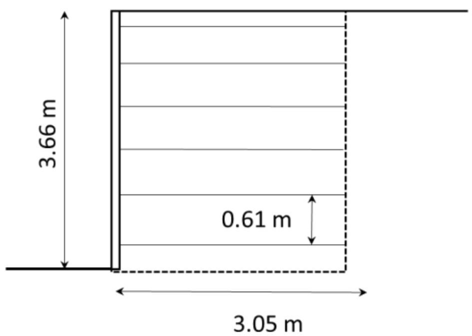


Figure 10. Wall geometry– Waterways Experimental Station Wall

The wall was backfilled with clean sub-angular to angular concrete sand with a d_{50} of 0.48 mm. Direct shear tests conducted on the backfill soil yield a peak friction angle of 36 degrees, which is an equivalent plane strain friction angle of 41.1 degrees based on the empirical relationship proposed by Bolton (1986). Using a modified shear box, the interface friction angle between sand and smooth steel strips was measured as 18 degrees. During the construction of the wall, the backfill was placed in 0.31 m lifts but was not compacted, therefore the additional horizontal stresses from compaction were not considered. Once the wall reached its design height, wall was surcharged to failure in increments of 12 kPa. The wall collapsed when the loading was in progress with an estimated surcharge load of approximately 90.4 kPa. A detailed discussion of this wall is presented in Weerasekara et al. (2017) which includes predictions for its failure, reinforcement load distribution and maximum reinforcement loads under all stages of loading. In this paper, Figure 11 shows only the maximum reinforcement loads predicted prior to surcharging.

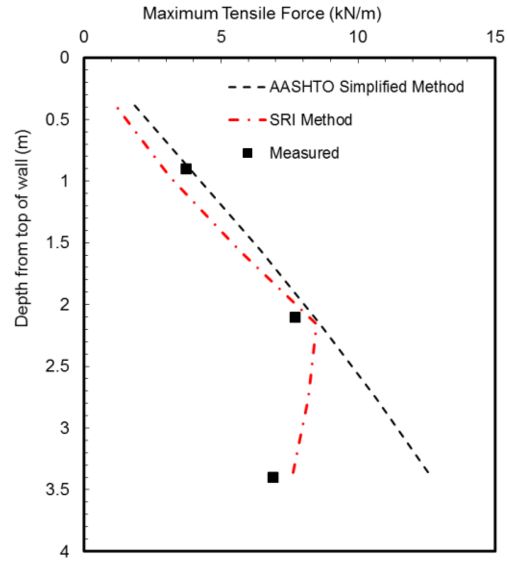


Figure 11. Measured and estimated maximum reinforcement loads from the SRI and AASHTO methods - Waterways Experimental Station Wall.

3.6 Gjovik, Norway (1990)

A 12 m high instrumented steel strip reinforced wall was constructed to support a roadway near Gjovik, Norway. The details of the wall were given in Vaslestad (1993) which was not available to the author, therefore the design input parameters were obtained from Allen and Bathurst (2003). The wall facing consisted of precast concrete facing panels. According to Allen and Bathurst (2003), the backfill was granular and was compacted to 97% of Standard Proctor Maximum Dry Density (ASTM D698) by using full-sized vibratory rollers. Allen and Bathurst (2003) have estimated a plane strain friction angle of 41 degrees. The wall geometry is shown in Figure 12, and predictions are shown in Figure 13.

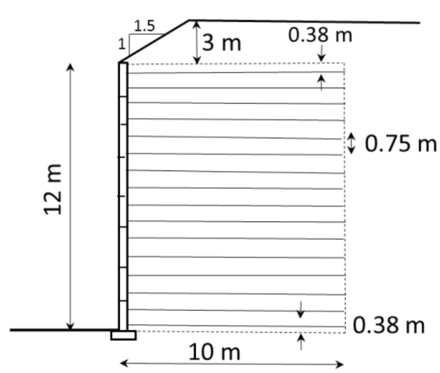


Figure 12. Wall geometry– Gjovik Wall

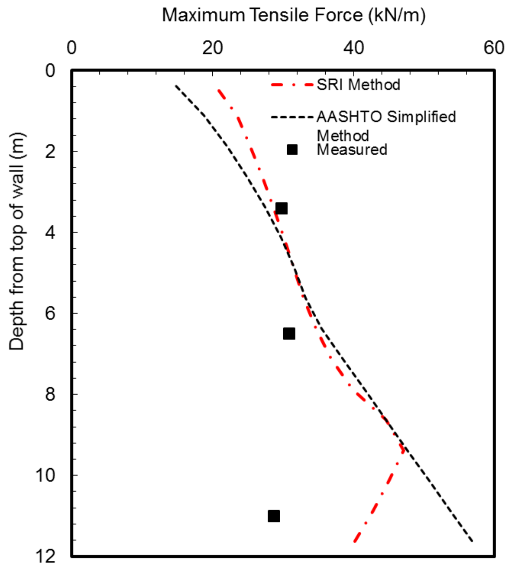
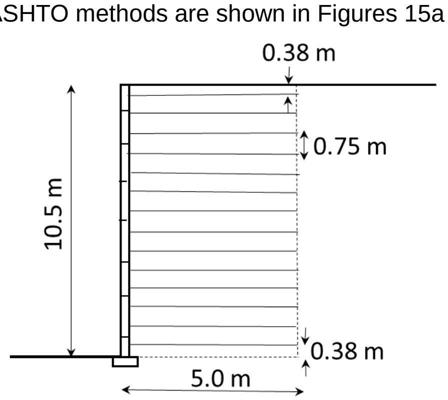


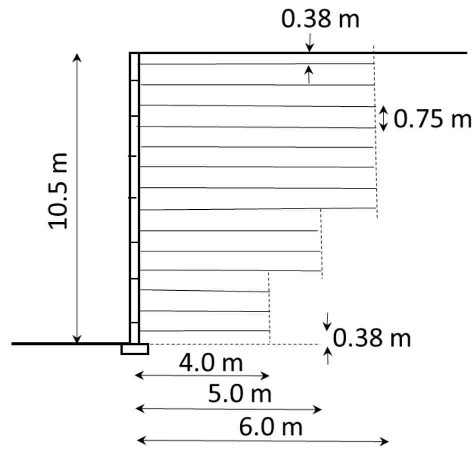
Figure 13. Measured and estimated maximum reinforcement loads from the SRI and AASHTO methods - Gjovik Wall.

3.7 Bourron Marlotte (1993)

Rectangular and trapezoidal shaped 10.5 m high walls were constructed near Bourron, Marlotte in the Fontainbleau Forest. The details of the wall are provided in Bastick et al. (1993). However, the model input parameters were obtained from Allen and Bathurst (2003) as the original report was not available to the author. As shown in Figures 14a and 14b, the rectangular wall had reinforcement of 5 m in length (H/D of 0.47), while the reinforcement length in the trapezoidal wall varied from 4 m at the bottom to 6 m at the top (H/D ratio of 0.38 at the bottom to 0.57 at the top). The walls were reinforced with ribbed steel strips of 60 mm x 5 mm, which were attached to the concrete facing panels. Uniformly graded Fontainbleau sand was used as the backfill with a d_{50} of approximately 0.27 mm. A plane strain friction angle of 40 degrees was reported by Allen and Bathurst (2003). The backfill was lightly compacted and the in situ density was reported as 16.8 kN/m^3. The estimations using the SRI and AASHTO methods are shown in Figures 15a and 15b.

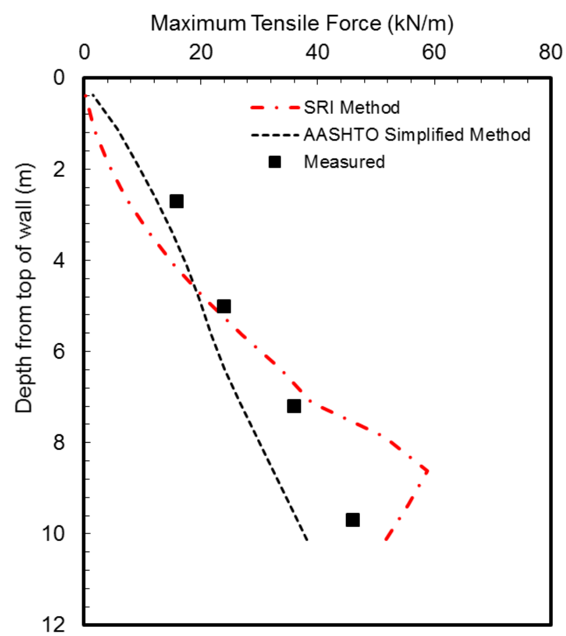


(a)

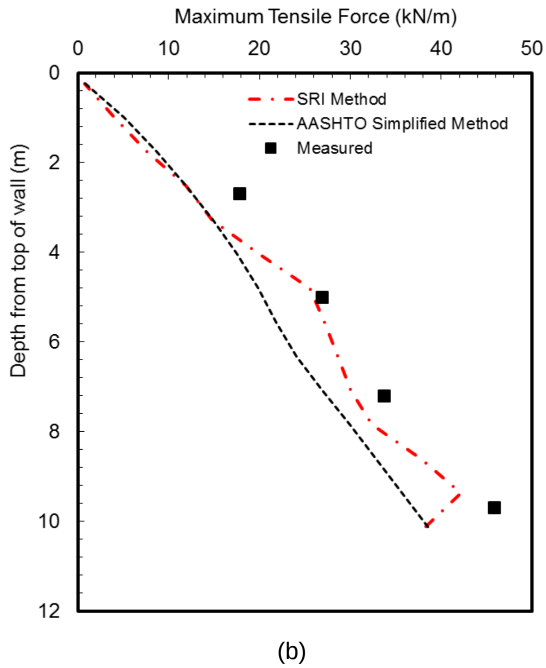


(b)

Figure 14. Wall geometry– (a) Rectangular and (b) trapezoidal sections of Bourron Marlotte Wall.



(a)



(b)

Figure 15. Measured and estimated maximum reinforcement loads from the SRI and AASHTO methods for (a) Rectangular and (b) trapezoidal sections of Bourron Marlotte Wall.

3.8 UCLA, California (1974)

A full-scale instrumented test wall was constructed in 1974 at the UCLA engineering Field Station in Saugus, California to study the static and dynamic behaviors of steel strip reinforced wall, and the details were given in Richardson et al. (1977). The wall was 6.1 m high and 34 m long and reinforced with 80 mm x 3 mm galvanized smooth steel strips. The steel strips were spaced 0.75 m horizontally and vertically, except for the bottom two layers where the vertical spacing was 0.38 m. The steel strips were 4.9 m in length. The backfill was a dusty sandy gravel sourced from a dry stream bed near the site. The d_{60} and d_{10} values were reported as 1.0 mm and 0.15 mm, respectively. A friction angle of 38 degrees was measured from triaxial testing, which corresponds to a plane strain friction angle of about 40 degrees. Backfill was placed in 0.46 m lift thicknesses (actual lift thickness varied from 0.3 m to 0.75 m), and no special effort was made to compact the backfill, besides driving trucks and other hauling equipment over the backfill. The measured soil unit weight was 19.8 kN/m^3 with about 1% of water content. This density corresponded to approximately 85% of Modified Proctor Dry Density and a relative density of about 65%. The wall geometry is shown in Figure 16 and estimations are shown in Figure 17.

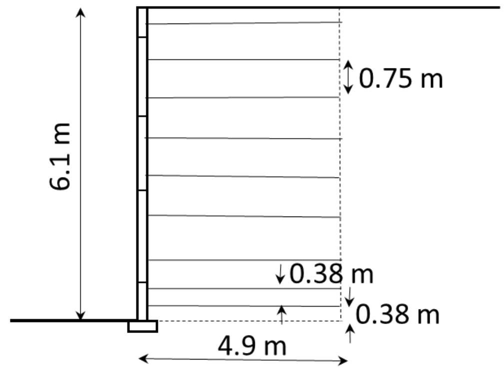


Figure 16. Wall geometry– UCLA Wall

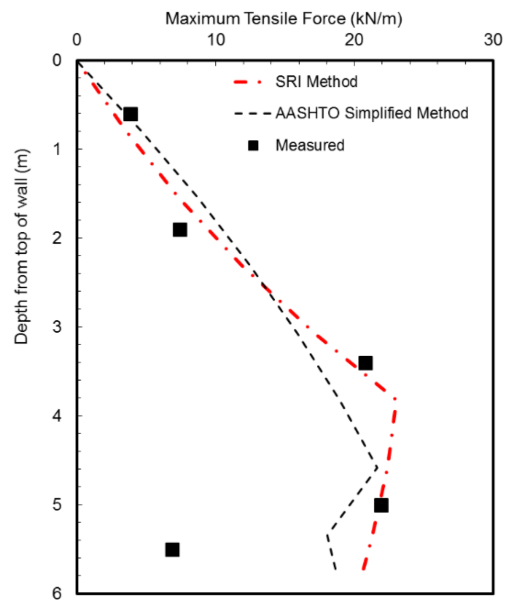


Figure 17. Measured and estimated maximum reinforcement loads from the SRI and AASHTO methods (UCLA Wall 1977).

3.9 Algonquin Wall (1993)

This 6 m high wall was constructed in Algonquin, Illinois, as part of a test program conducted by Federal Highway Administration and the details are provided in Christopher (1993). The wall geometry in shown in Figure 18. Wall 1 was reinforced with ribbed steel strips and backfilled with a gravelly sand backfill with a maximum particle size of 50 mm and d_{50} of 4 mm. The peak friction of this backfill was measured as 40 degrees from triaxial testing which is equivalent to a plane strain friction angle of about 43 degrees. Christopher (1993) reported that the backfill was placed using lift thickness of 200 mm and compacted using four to five passes using a vibrating drum type compactor to achieve a minimum compaction of 95% of Standard Proctor Maximum Dry Density (ASTM D698). The maximum reinforcement loads estimated using the SRI and AASHTO methods are shown in Figure 19.

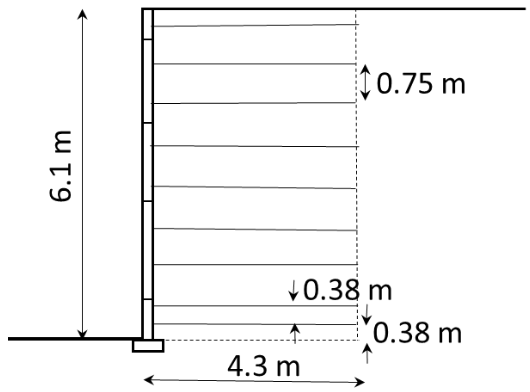


Figure 18. Wall geometry– Algonquin Wall

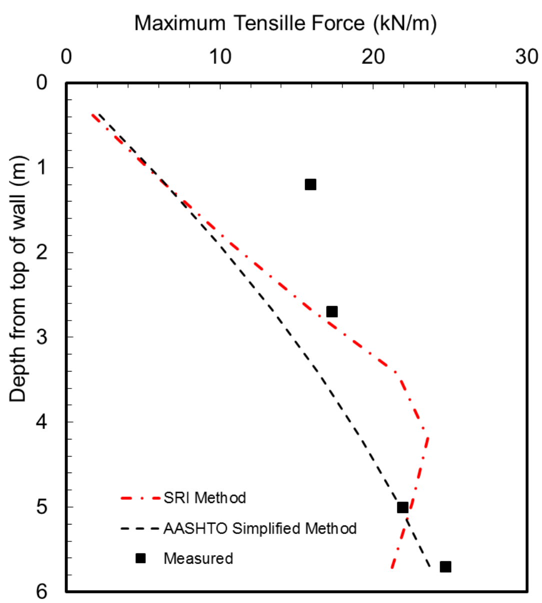


Figure 19. Measured and estimated maximum reinforcement loads from the SRI and AASHTO methods - Algonquin Wall.

4 DISCUSSIONS

The above case histories indicate that the SRI method is capable of estimating the maximum reinforcement load of steel strip reinforced walls with reasonable accuracy. Unlike empirical methods, each input parameter used in the SRI model can be verified by using independent testing and has a physical meaning with the exception of Q and R parameters in the classical Bolton's equation. In most instrumented walls, the measured values at the lowest levels of reinforcement layers is less than the triangular load distribution predicted by the AASHTO and similar methods. This is a phenomenon commonly observed in instrumented walls and in numerical modeling (Christopher, 1993). The reduced reinforcement load near the base can be attributed to the toe resistance generated from the wall embedment and friction. With the exception of the Simplified Stiffness method, current methods are not capable of accounting for the toe resistance. The magnitude of toe resistance incorporated in the Simplified Stiffness method is not known due its empirical formulation. For walls with a smaller toe resistance (or toe embedment), this could lead to an overestimation of the toe resistance. The SRI method allows to quantify the toe resistance and also allows the designer to limit the toe resistances if large values cannot be relied upon.

Besides validating a design method under working stress conditions, Weerasekara et al (2017) highlighted the importance of validating its ability to predict ultimate state. A good prediction under working stress conditions would not necessarily guarantee a good estimation of the safety factor if the design method fails to predict the ultimate state.

5. REFERENCES

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