Reinforced soil walls: Investigation of the prediction capabilities of the design methods under working stress conditions

S. H. Mirmoradi & M. Ehrlich
Federal University of Rio de Janeiro, Brazil

ABSTRACT: The prediction capabilities of the AASHTO simplified (AASHTO 2014), K-stiffness (Bathurst et al., 2008; Allen et al., 2004), simplified stiffness (Allen and Bathurst, 2015), and EM (Ehrlich and Mirmoradi, 2016) methods are investigated under working stress conditions using data obtained from experimental and numerical studies. For the experimental evaluation, the reinforcement loads measured from two physical model studies constructed at the Geotechnical Laboratory of COPPE/UFRJ were used. Numerical analyses were carried out to evaluate the combined effects of facing stiffness, reinforcement stiffness, toe fixity, and wall height on reinforcement loads. The analyses' results are compared with the reinforcement requirements determined using the design methods. This comparison shows that the methods may overestimate or underestimate the magnitude of the maximum reinforcement load T_{max}, in reinforced soil walls, depending on the reinforcement stiffness, wall height, facing stiffness, and toe restraint.

Keywords: Reinforced soil walls; Design method; Working stress conditions.

1 INTRODUCTION

Prediction of the maximum reinforcement load T_{max}, is a major objective in the design of reinforced soil (RS) structures. There are several methods that can be used for the calculation of T_{max}. They include limit equilibrium methods, those based on the Rankine method [e.g., AASHTO 2014, Federal Highway Administration (FHWA) 2008], or working stress design methods [e.g., Ehrlich and Mitchell (1994), the K-stiffness (Allen et al., 2004; Bathurst et al., 2008), simplified stiffness (Allen and Bathurst, 2015), Ehrlich and Mirmoradi, (2016)].

The AASHTO simplified method is a limit equilibrium procedure that takes into consideration the influence of facing inclination and reinforcement stiffness for geosynthetics and steel. This method, however, does not consider the effects of the facing and apparent cohesion on the calculation. For calculations using this method, use of a direct shear and triaxial soil friction angle is recommended.

Ehrlich and Mirmoradi (2016) presented an analytical procedure for the internal design of reinforced soil walls under working stress conditions, which explicitly takes into account the effect of compaction-induced stress, reinforcement and soil properties, and facing inclination. The proposed method is based on Ehrlich and Mitchell's (1994) procedure. There are three key differences between the proposed new method and Ehrlich and Mitchell's (1994) procedure: (1) the effect of the facing inclination is considered by the new method, while the original method was developed for vertical walls; (2) the calculation of T_{max} using the proposed new method does not need iteration, which was required by the original method; and (3) the equations are simpler to use. Depending on the wall's geometric condition, calculation using the Ehrlich and Mirmoradi (2016) design method can be performed using the plane strain or triaxial compression friction angles. Note that plane strain conditions are typically found in the field.

The K-stiffness (Allen et al., 2004; Bathurst et al., 2008) and simplified stiffness (Allen and Bathurst 2015) methods are empirical methods for RS wall design under working stress conditions. These methods explicitly take into account the effect of reinforcement stiffness, facing stiffness and inclination, and soil strength. The K-stiffness method assumes plane strain conditions in the formulation of the design approach. For the calculation using the simplified stiffness method, it is recommended that the direct shear or triaxial compression friction angle is used.

In this study, based on the results of the physical and numerical model studies, the prediction capabilities of the AASHTO simplified (2014), Ehrlich and Mirmoradi (2016), K-stiffness (Bathurst et al., 2008; Allen et al., 2004), and simplified stiffness (Allen and Bathurst, 2015) design methods are evaluated.

2 EXPERIMENTAL STUDY

Instrument data and measurements from two physical model walls constructed at the Geotechnical Laboratory of COPPE/UFRJ are used. These walls are herein identified as Wall 1 and Wall 2. Figure 1 shows a cross-section and front view of Wall 1. The walls were constructed in a U-shaped concrete model box that is 1.5-m-high, 3.0-m-long, and 2.0-m-wide. The vertical spacing of reinforcements and the facing inclination were 0.4 m and 6° to the vertical, respectively.

The height of the wall was 1.2 m. Three layers of geogrid were installed at 0.2 m, 0.6 m, and 1.0 m above the wall bottom. The reinforcements had a length of 2.12 m measured from the front of the wall face. The backfill soil was compacted using a light vibrating plate (8 kPa). Precast concrete blocks were used for the faces of the walls. The soil unit weight after compaction was 21 kN/m^3. The soil friction angles, determined by triaxial and plane strain tests on samples compacted to this unit weight, were 42° and 50°, respectively.

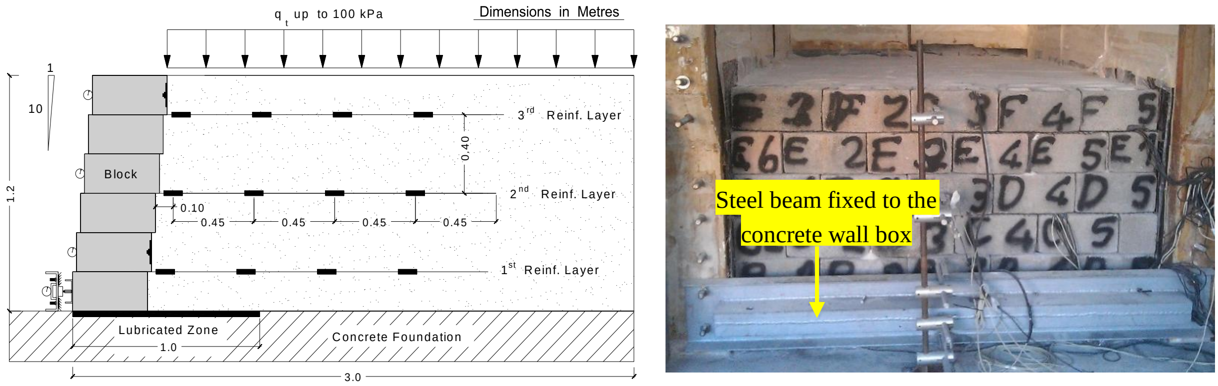

Figure 1. A cross-sectional view of a block-face wall and a front view of Wall 1 at the end of construction.

A 1-m-wide zone at the bottom of the walls, including the base of the block facing, was lubricated through a sandwich of rubber sheets and silicon grease, in order to allow for movement of the potential failure surface, keeping it away from the wall face. After construction, a surcharge loading of up to 100 kPa was applied over the entire surface of the backfill soil using an air bag.

The difference between Walls 1 and 2 were related to the toe conditions. In Wall 1, lateral movement of the toe was restricted by a steel beam fixed to the concrete U-shaped wall box, as shown in Figure 1. After the construction and application of the surcharge, the load was kept constant at 100 kPa. With the surcharge in place, the toe of Wall 1 was released step-by-step (0.5 mm horizontal movement allowed in each step). Details of these two walls can be found in Mirmoradi and Ehrlich (2016) and Mirmoradi et al. (2016).

The reinforcement loads were measured using load cells installed at four points along each reinforcement layer (two load cells at each point). The load cells were attached to the geogrid and measured the mobilised tension along the reinforcements. They allowed tension monitoring without the need to determine the reinforcement stress-strain curves, which are time dependent. The load cells were also capable of counterbalancing the temperature effects and the bending moments, and were strong enough to resist the stress induced during the operation of the compaction equipment (Ehrlich et al., 2012; Ehrlich and Mirmoradi, 2013; Mirmoradi et al., 2016).

2.1 *Test Results*

Figure 2 compares the measured and calculated values of the summation of the maximum reinforcement loads ΣT_{max}, using the AASHTO simplified, K-stiffness, simplified stiffness, and EM design methods. Regarding the AASHTO and simplified stiffness methods, the calculations were performed employing the triaxial and plane strain soil friction angles. The results are shown during the surcharge application and toe release.

Figure 2 shows that, for Wall 1, the AASHTO simplified and the EM methods overestimate the reinforcement loads during the surcharge application. Considering the triaxial friction angle in the calculation, significant overestimation of the AASHTO method is observed compared to the measured values in Walls 1 (by over a factor of three). However, this overestimation decreases during toe release. Of note for Wall 1, is that the combined effect of facing stiffness and toe restraint leads to lower reinforcement loads. This effect is not considered in the AASHTO and EM methods. The figure indicates that, during the toe release, the influence of the block facing on the reinforcement load would vanish, and the accuracy of the AASHTO simplified and EM methods would increase. In Wall 2, as no toe restraint was applied, facing effects do not occur. Therefore, as shown in Figure 2, the reinforcement load values measured in Wall 2 closely match those obtained by the AASHTO and EM methods.

Figure 2 also shows that considering the plane strain friction angle in the calculation, the simplified stiffness method properly captures the measured reinforcement loads in Wall 1, but it under-predicts the values at the end of the toe release. This method also underestimates the measured values in Wall 2, in which the toe of the wall was not restricted. On the other hand, this method properly represents the reinforcement loads measured for Wall 2, in which the triaxial friction angle was employed in the calculation. For Wall 1, this method overestimates the measured values during the surcharge application if the triaxial friction angle is considered in the calculation. Nevertheless, this overestimation vanishes at the end of toe release. Comparison of the measured and calculated values using the K-stiffness design method shows that this method underestimates the reinforcement load values measured in Walls 1 and 2, during both surcharge application and toe release.

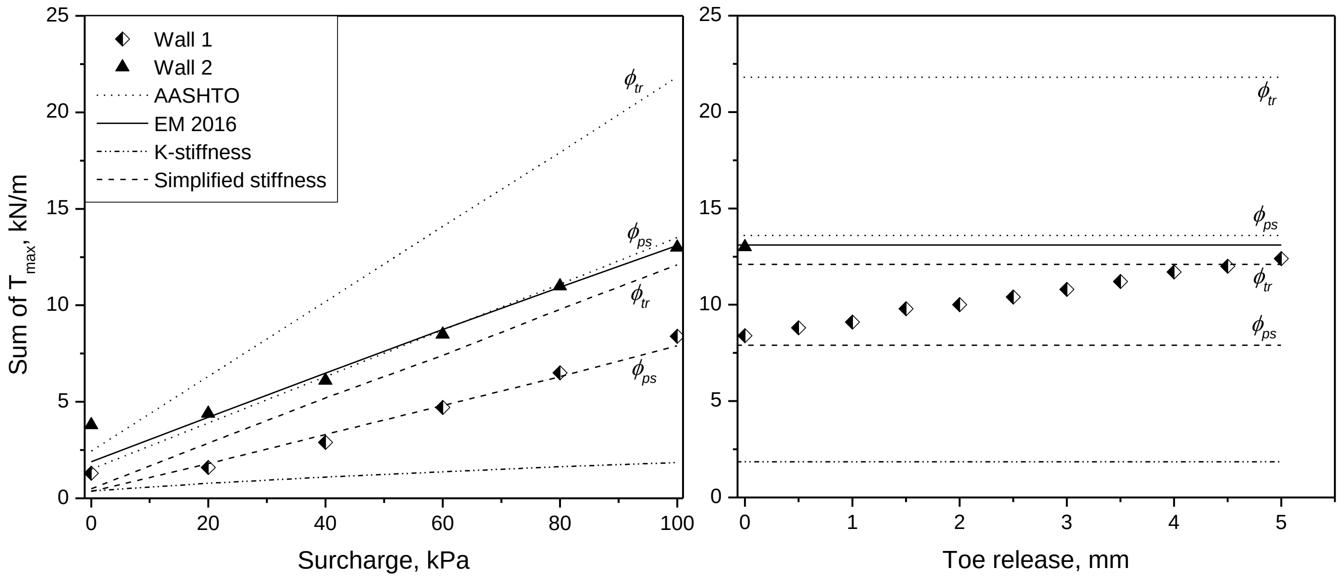

Figure 2. Comparison of the measured and calculated values of ΣT_{max} versus surcharge and toe release.

Figure 3 compares the individual values of the maximum reinforcement loads T_{max}, measured for Walls 1 and 2 at the end of construction (EOC), under 100 kPa surcharge loading (EOL), and at the end of the toe release (EOR) with values calculated using the AASHTO, Ehrlich and Mirmoradi (2016), K-stiffness, and simplified stiffness methods. Results show that at EOC and EOL, the distribution of T_{max} with depth is more uniform for Wall 1. After toe release, however, the shape of the distribution of T_{max} with depth becomes triangular. For Wall 2, where there was no toe restraint from the beginning of the test, a similar triangular shape is observed.

Furthermore, this figure illustrates that the AASHTO and EM methods, in general, properly predict the values of T_{max} at EOC. At EOL, those methods overestimate the values of T_{max} for Wall 1. This discrepancy, however, decreases during toe release. The simplified stiffness method under-predicts T_{max} at EOC. Under 100 kPa, this method, in general, properly captures T_{max} in Walls 1 and 2. In Wall 2, a redistribution of load between the reinforcement layers occurred due to the combined effects of facing stiffness and toe condition, and there was a decrease and increase of load in the reinforcement layers placed at the top and bottom of the wall, respectively. Note that in this wall, the toe of the block facing was not laterally restricted, and therefore a high reinforcement load was expected. The K-stiffness method underestimates the measured values of T_{max}, which are more pronounced at EOL and EOR.

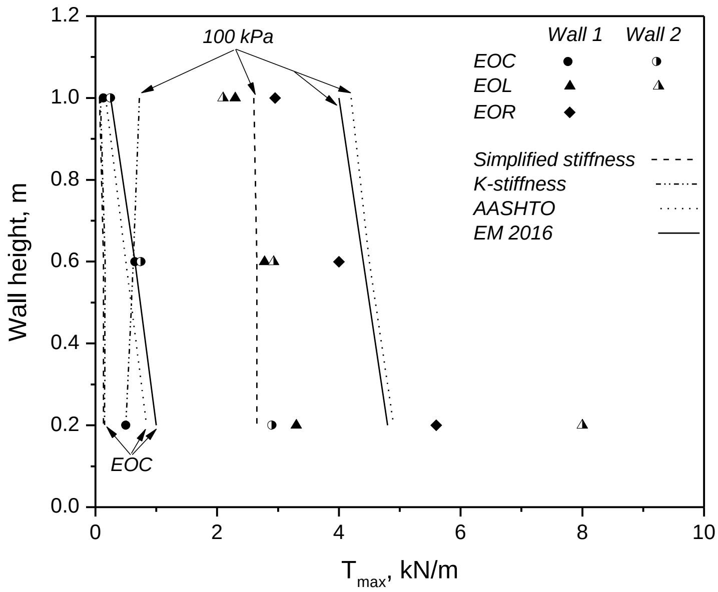

Figure 3. Comparison of measured and calculated values of T_{max} versus depth for two walls.

3 NUMERICAL STUDY

Numerical modelling was performed for block-face MSE walls using the 2D finite element program PLAXIS (Brinkgreve and Vermeer, 2002). Data from a full-scale reinforced soil wall built at the Royal Military College of Canada (RMC) was used to validate the numerical modelling for a block-face wall. Details about the validation of the models can be found in Mirmoradi and Ehrlich (2014, 2015, 2017).

Parametric studies were carried out to evaluate the combined effects of facing stiffness, reinforcement stiffness, toe fixity, and wall height. Three different wall heights were considered: 4, 8, and 16 m. The length and the vertical spacing of reinforcements were 0.7 H and 0.4 m, respectively. Block facing with vertical facing inclination was considered. Two different toe fixity conditions were considered (free and fixed base conditions), and a hardening soil model was applied. A fixed boundary condition in the horizontal direction was employed on the right lateral border. At the bottom of the model, a fixed boundary condition in both the horizontal and vertical directions was applied. For the models with free-base conditions, a fixed boundary condition in the vertical direction was employed at the bottom of the block facing.

Reinforcement was modelled as a linear elastic material with perfect interface adherence to the adjacent soil. Three values of the tensile stiffness modulus of reinforcement J_r, equal to 600, 6000, and 60000 kN/m, were employed. Assuming these values, the relative soil-reinforcement stiffness index S_i, equal to 0.025, 0.25, and 2.5, was calculated. The parameter S_i was developed by Ehrlich and Mitchell (1994) and can be calculated as follows:

$$S_i = \frac{J_r}{kP_a S_v} \tag{1}$$

where J_r is the tensile stiffness modulus of reinforcement, k is the modulus number (hyperbolic stress-strain curve model), P_a is the atmospheric pressure, and S_v is the vertical reinforcement spacing. The interface parameters defined by Hatami and Bathurst (2005) were used for the block facing. Figure 4 shows the geometry of a wall with a height of 4 m. Table 1 lists the input parameters used in the analysis.

3.1 *Results of the Numerical Study*

Figures 5–7 compare the normalised values of the sum of the maximum load in the reinforcements ($\Sigma T_{max}/\gamma H^2$) versus the normalised facing stiffness ($EI/\gamma H^5$) using the values calculated by the PLAXIS, AASHTO, EM, K-stiffness, and simplified stiffness design methods. In these figures, the results presented for the K-stiffness and EM methods are related to the plane strain soil friction angle. For the simplified stiffness method, a triaxial soil friction angle was used. The results are for the three different wall heights (4, 8, and 16 m) and S_i values (0.025, 0.25, and 2.5) for free- and fixed-base conditions.

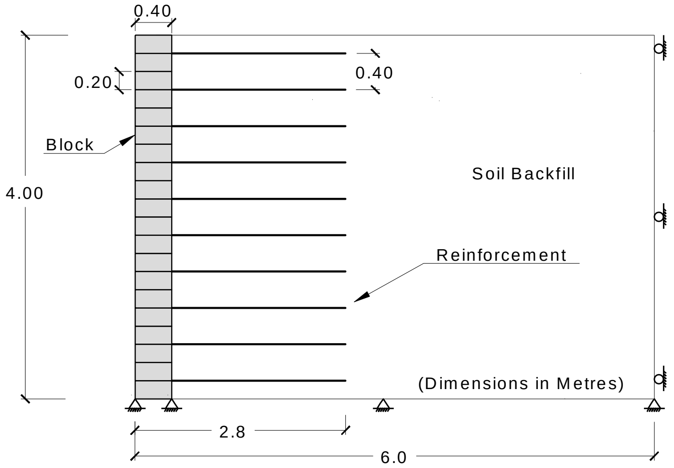

Figure 4. Typical numerical model.

Table 1. Input parameters for the parametric study.

Property	Value
Soil property	
Model	HS
Peak plane strain friction angle, ϕ, (^{o})	50
Cohesion, c, (kPa)	1.0
Dilation angle, Ψ, (^{o})	0.0
Unit weight, γ, (kN/m^3)	20
E^{ref}_{50}, (kPa)	42500
E^{ref}_{oed}, (kPa)	31800
E^{ref}_{ur}, (kPa)	127500
Stress dependence exponent, m	0.5
Failure ratio, R_f	0.9
Poisson's ratio, υ	0.2
Modular block properties	
Model	Linear elastic
Size, (m×m)	0.4×0.2 (length×height)
Unit weight, γ, (kN/m3)	22
Stiffness modulus, (kPa)	1×10^6, 5×10^6, 1×10^7, 5×10^7
Poisson's ratio, υ	0.15
Block-block interface	
Friction angle, ϕ, (^{o})	57
Cohesion, (kPa)	46
Soil-block interface	
Friction angle, ϕ, (^{o})	44
Cohesion, (kPa)	1
Dilation angle, Ψ, (^{o})	11

Figure 5 shows that, for the free-base condition and a given S_i, the $\Sigma T_{max}/\gamma H^2$ values determined using PLAXIS were similar regardless of the normalised facing stiffness, and were well represented by the EM method. Additionally, the results that correspond to S_i equal to 0.025 were similar to those obtained for the active Rankine condition K_a, which also represents the AASHTO simplified method. Nevertheless, the AASHTO simplified method may underestimate reinforcement loads for S_i equal to 0.25, which is assumed to be the upper theoretical limit for the polymeric reinforcement. Therefore, as the lateral earth pressure coefficient for the AASHTO simplified method was modified for walls with steel reinforcement, this modification may also be required for the polymeric reinforcement with high stiffness.

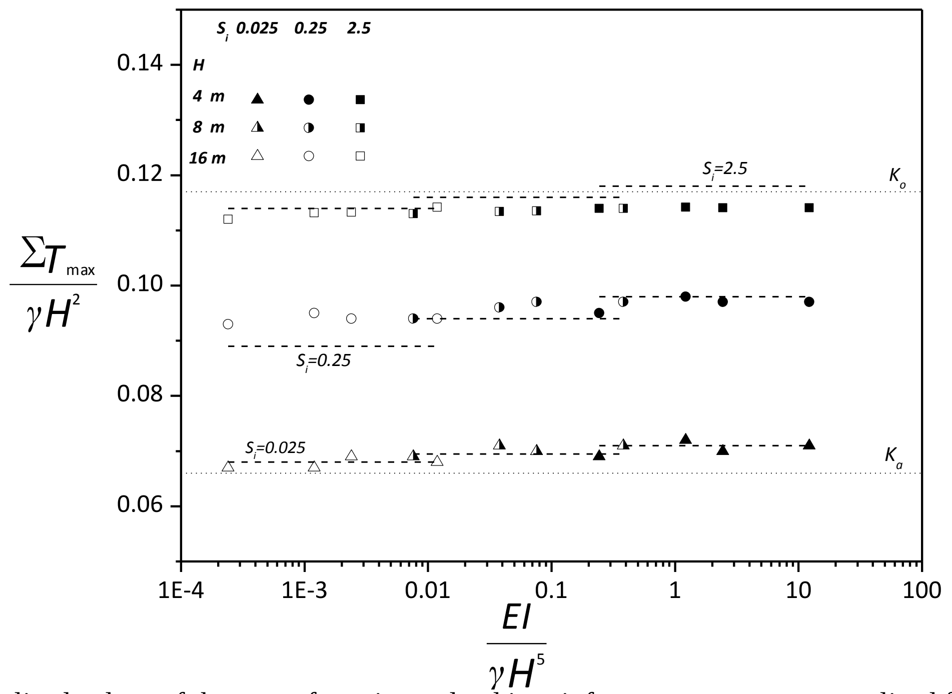

Figure 5. Normalised values of the sum of maximum load in reinforcements versus normalised facing stiffness for the free-base toe condition and different values of wall height. Dashed lines: EM method.

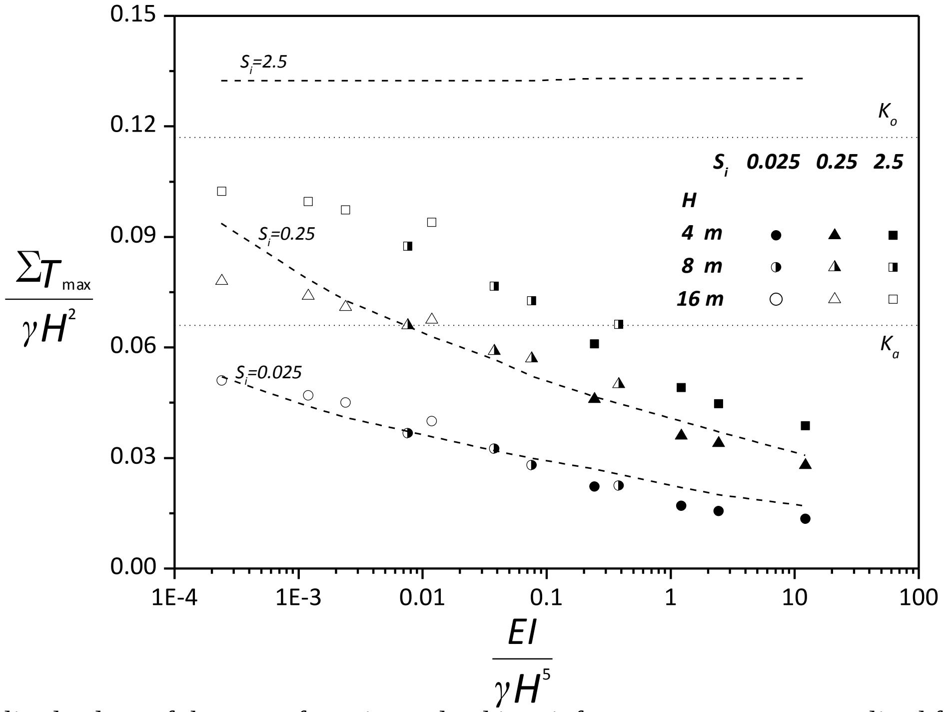

Figure 6. Normalised values of the sum of maximum load in reinforcements versus normalised facing stiffness for the fixed-base toe condition and different values of wall height. Dashed lines: K-stiffness method.

For a given value of $EI/\gamma H^5$, Figure 6 shows generally good agreement between the values of $\Sigma T_{max}/\gamma H^2$ determined by PLAXIS for the fixed-base condition, and the K-stiffness method for polymeric reinforcements (i.e. S_i values of 0.025 and 0.25). Figure 6 also illustrates that, for the steel reinforcement, the K-stiffness method overpredicts the determined ΣT_{max} using PLAXIS.

Figure 7 shows that the simplified stiffness method overestimates the $\Sigma T_{max}/\gamma H^2$ values calculated by PLAXIS for 4-m and 8-m-high walls, and for polymeric reinforcement (i.e., S_i values of 0.025 and 0.25). Note that the results presented for PLAXIS are related to the fixed-base conditions. The simplified stiffness method may properly represent the determined values of ΣT_{max} by PLAXIS for fixed-base conditions and greater heights of the wall, and/or low facing stiffness values (low $EI/\gamma H^5$ values). Moreover, considering the steel reinforcement (S_i = 2.5), the simplified stiffness method significantly overestimates the ΣT_{max} for low wall heights. This overestimation also decreases for greater heights of the wall and/or low facing stiffness values.

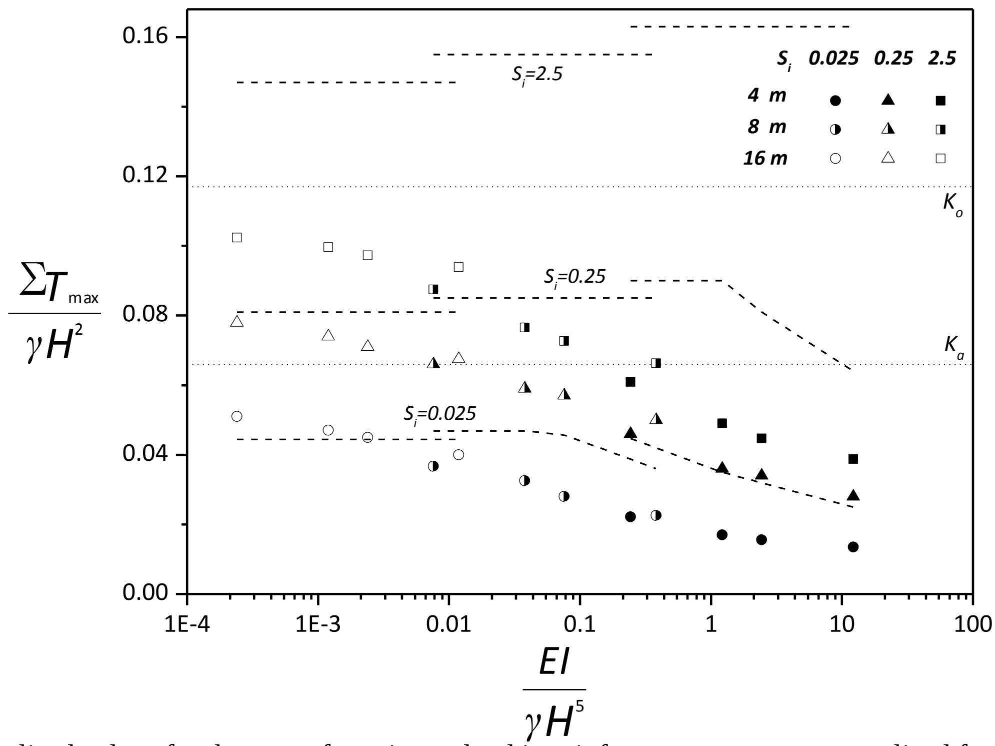

Figure 7. Normalised values for the sum of maximum load in reinforcements versus normalised facing stiffness for the fixed-base toe condition and different values of wall height. Dashed lines: simplified stiffness method.

4 CONCLUSIONS

The prediction capabilities of the AASHTO simplified, EM (2016), K-stiffness, and simplified stiffness design methods have been investigated using data from physical and numerical model studies. The results indicate that these capabilities may be significantly influenced by the combined effect of the facing stiffness, toe resistance, wall height, and reinforcement stiffness.

The AASHTO simplified method may overpredict or under-predict the reinforcement loads depending on S_i values, toe restraint, and friction angle of the backfill soil. Furthermore, as the lateral earth pressure coefficient for the AASHTO simplified method was modified for the walls with steel reinforcement, this modification should also be adopted for a polymeric reinforcement with high stiffness.

For a short wall with fixed-base conditions, the K-stiffness method may properly represent the load in the geosynthetic reinforcement. However, this method may under-predict the mobilised reinforcement loads with increasing wall height and reinforcement stiffness, as well as a reduction in the toe restraint. On the other hand, for the steel-reinforced soil walls the reinforcement loads may be overpredicted by this method.

Comparison of the calculated values of the reinforcement load for the walls with fixed based conditions and the simplified stiffness method indicated that this method may properly predict the reinforcement loads determined by PLAXIS for polymeric reinforcement (i.e., S_i values of 0.025 and 0.25), greater heights of the wall (16-m-high walls), and/or low facing stiffness values (low $EI/\gamma H^5$ values). Nevertheless, the simplified stiffness method significantly overestimates the reinforcement loads for low heights of the wall (4-m-high walls), which is more pronounced considering the steel reinforcement (S_i = 2.5).

The EM design method represented the reinforcement loads for the walls with free-base conditions very well. This method, therefore, may overpredict the reinforcement loads for a short wall with fixed-base conditions due to the mobilised load at the base of the wall facing. Nevertheless, even for fixed-base conditions, the accuracy of this method significantly increases with a decrease in the facing effect, which could occur with an increase in the wall height and reinforcement stiffness.

It is important to notice that for consideration of the effect of toe restraint, two extreme conditions (i.e. fixed- and free-base conditions) were numerically investigated. In real field conditions, the toe of the block facing with no lateral restriction would not occur. On the other hand, it may not be possible to restrict the lateral displacement fully at the toe of the block face walls (Mirmoradi and Ehrlich, 2017). Thus, the practice of ignoring the toe restraint produced by a 0.3–0.5m-deep block may be justified in the design to increase the margin of safety against reinforcement overstressing, in the case where lateral movements of the toe blocks occur. This is supported by Leshchinsky and Tatsuoka (2013), and Tatsuoka et al. (2014) who recommended ignoring toe resistance in the design.

ACKNOWLEDGMENTS

The authors greatly appreciate the funding of this study by the Brazilian Research Council, CNPq, and the Brazilian Federal Agency for Support and Evaluation of Graduate Education, CAPES. We also thank Flavio Montez and Andre Estevao Ferreira da Silva from the Huesker Company for their support, as well as Cid Almeida Dieguez for his support in performing the experiments.

REFERENCES

AASHTO (American Association of State Highway and Transportation Officials) 2014. LRFD bridge design specifications, 7th edn. Washington, DC, USA.

Allen, T., Bathurst, R., Holtz, R., Lee, W., and Walters, D. 2004. New method for prediction of loads in steel reinforced soil walls. J. Geotech. Geoenviron. Eng., 130(11), 1109–1120.

Allen, T.M., Bathurst, R.J. 2015. An improved simplified method for prediction of loads in reinforced soil walls. ASCE J. Geotech. Geoenviron. Eng., 141(11), 04015049.

Bathurst, R.J., Miyata, Y., Nernheim, A., and Allen, A.M., 2008. Refinement of K-stiffness method for geosynthetic-reinforced soil walls. Geosynth. Int., 15(4), 269–295.

Brinkgreve, R.B.J., Vermeer, P.A. 2002. PLAXIS: Finite element code for soil and rock analyses, version 8, CRC Press/Balkema, Leiden, Netherlands.

Ehrlich, M., Mirmoradi, S.H. 2013. Evaluation of the effects of facing stiffness and toe resistance on the behavior of GRS walls. J. Geotextile Geomembr., 40, 28–36.

Ehrlich, M., Mirmoradi, S.H., Saramago, R.P. 2012. Evaluation of the effect of compaction on the behavior of geosynthetic-reinforced soil walls. J. Geotextile Geomembr., 34, 108–115.

Ehrlich, M., Mirmoradi, S.H. 2016. A simplified working stress design method for reinforced soil walls. Géotechnique, 66(10), 854–863.

Ehrlich, M., Mitchell, J.K., 1994. Working stress design method for reinforced soil walls. J. Geot. Eng. ASCE, 120(4), 625–645.

Federal Highway Administration (FHWA). 2008. Geosynthetic design and construction guidelines. FHWA NHI-07-092, Washington, DC.

Hatami, K., Bathurst, R.J., 2005. Development and verification of a numerical model for the analysis of geosynthetic reinforced-soil segmental walls. Can. Geotech. J., 42(4), 1066–1085.

Leshchinsky, D., Tatsuoka, F. 2013. Geosynthetic reinforced walls in the public sector: performance, design, and redundancy. Geosynth. Mag., 31(3), 12–21.

Mirmoradi, S.H., Ehrlich, M. 2014. Geosynthetic reinforced soil walls: experimental and numerical evaluation of the combined effects of facing stiffness and toe resistance on performance. Proc., 10th Int. Conf. on Geosynthetics, International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE), London.

Mirmoradi, S.H., Ehrlich, M. 2015. Numerical evaluation of the behavior of GRS walls with segmental block facing under working stress conditions. ASCE J. Geotech. Geoenviron. Eng., 141(3), 04014109.

Mirmoradi, S.H., Ehrlich, M., and Dieguez, C. 2016. Evaluation of the combined effect of toe resistance and facing inclination on the behavior of GRS walls. J. Geotextile Geomembr., 44(3), pp 287–294.

Mirmoradi, S.H., Ehrlich, M. 2016. Evaluation of the effect of toe restraint on GRS walls. Transportation Geotechnics, SI: Geosynthetics in Tpt., 8, pp. 35–44.

Mirmoradi, S.H., Ehrlich, M. 2017. Effects of facing, reinforcement stiffness, toe resistance, and height on reinforced walls. J. Geotextile Geomembr., 45(1), 67–76.

Tatsuoka, F., Koseki, J., and Kuwano, J., 2014. Natural disasters mitigation by using construction methods with geosynthetics (earthquakes). Keynote Lecture, 10th international conference on geosynthetics, Berlin, ISSMGE, 53p.