Influence of geosynthetic stiffness on reinforced soil walls

L'influence de la rigidité des géosynthétiques sur les murs en sols renforcés

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ABSTRACT

The paper describes a series of four full-scale modular block walls that were constructed with reinforcement layers having different stiffness. The walls were 3.6 m in height and were reinforced with two different polypropylene geogrid reinforcement materials, a polyester geogrid and a welded wire mesh. Each was constructed with the same modular block facing and reinforcement spacing of 0.6 m. Maximum reinforcement loads are compared to values predicted using the current AASHTO Simplified Method and the K-stiffness Method. The predicted magnitude and distribution of reinforcement loads are shown to be more accurate using the K-stiffness Method.

RÉSUMÉ

L'article décrit une série de quatre murs grande échelle avec parement de blocs modulaires, construits avec des couches de renforcements ayant différentes rigidités. Les murs avaient 3.6 m de hauteur et étaient renforcés de deux différentes géogrilles de polypropylène, une géogrille de polyester et une grille de fils soudés. Chacun a été construit avec le même parement de blocs modulaires et le même espacement des couches de renforcement de 0.6 m. Les charges maximums dans les renforcements sont comparées à celles prédites par la méthode simplifiée courante de l'AASHTO et la méthode de la rigidité K. Les meilleurs prédictions de l'intensité et la distribution des charges dans le renforcement sont révélées être celles issues de la méthode de la rigidité K.

Keywords : geosynthetics, walls, reinforced soil, reinforcement stiffness, welded wire mesh, physical modeling

1 INTRODUCTION

A series of 11 full-scale reinforced soil walls has been recently completed at the Royal Military College of Canada (RMC). The walls were 3.6 m in height and were constructed with the same sand backfill material. Following construction, the walls were loaded with a uniform surcharge in stages to load levels well beyond working stress levels. The test program was designed to investigate the influence of a range of different wall components and geometry on wall performance. The parameters varied were: wall facing type, wall batter angle, reinforcement type, stiffness and spacing. The general experimental design and selected results from several walls in this series have been reported by Bathurst et al. (2000, 2006) and Hatami and Bathurst (2005, 2006). This paper is focused on four full-scale modular block walls in the RMC research program (Wall 1 (control), 2, 5 and 6). The walls were 3.6 m in height (H) and were reinforced with two polypropylene geogrid reinforcement materials with different stiffness, a polyester geogrid and a welded wire mesh. Each was constructed with the same modular block facing and reinforcement spacing $S_v = 0.6$ m. The target variable between the otherwise nominally identical four walls was the reinforcement stiffness. Maximum reinforcement loads are compared to values predicted using the current AASHTO (2002) Simplified Method and the K-stiffness Method (Bathurst et al. 2008). The predicted magnitude and distribution of reinforcement loads are shown to be more accurate using the K-stiffness Method. An expanded version of this paper is reported by Bathurst et al. (2009).

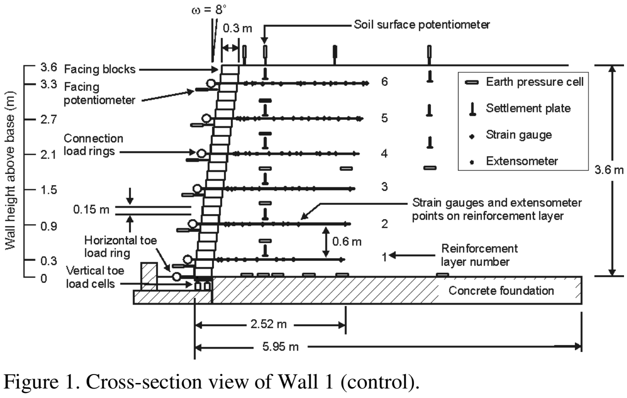

Figure 1. Cross-section view of Wall 1 (control).

2 EXPERIMENTAL PROGRAM

2.1 *General*

Figure 1 shows a cross-section of Wall 1 (control) constructed with six layers of a polypropylene (PP) geogrid at a spacing of $S_v = 0.6$ m and a target facing batter $\omega = 8°$ from the vertical. The wall was designed to satisfy current National Concrete Masonry Association guidelines (NCMA 1997). An additional design constraint was that the reinforcement layer spacing should not exceed a distance equal to twice the modular block toe to heel dimension (AASHTO 2002).

The wall facing was built with three discontinuous vertical sections with separate reinforcement layers in plan view (i.e. each reinforcement layer was discontinuous in the cross-plane

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strain direction). The base of each model was seated on a rigid concrete floor. The toe of the wall facing was seated on an instrumented footing with a stiff horizontal boundary condition. These idealized boundary conditions were selected to make construction of the model walls as easy as possible and to simplify the interpretation of wall performance. More than 300 instruments were installed during construction and monitored for the duration of uniform surcharging (Figure 1).

2.2 *Reinforcement*

The geosynthetic reinforcement product used in Wall 1 (control) was a relatively weak, biaxial integral drawn polypropylene (PP) geogrid that was oriented in the weak direction to encourage detectable strains in the reinforcement. Each layer of geogrid was 2.52 m in length measured from the front of the facing column (Figure 1). The aperture size for the PP reinforcement was 25 mm between longitudinal members and 33 mm between transverse members. Wall 2 was nominally identical to Wall 1 except that every second longitudinal geogrid member was removed to give a material with half the stiffness and strength of the reinforcement in the control wall. Wall 5 was constructed with less stiff polyester (PET) geogrid reinforcement, and Wall 6 was constructed with stiff welded wire mesh (WWM) reinforcement. The focus of this investigation is on geosynthetic reinforced soil walls. The WWM material was selected to represent an idealized "geogrid" with similar strength to the polymeric products used in the other walls but with very high stiffness. Reinforcement properties are summarized in Table 1. The combined influence of stiffness and number of reinforcement layers is a key factor that determines the magnitude of tensile load in geosynthetic reinforced walls. For example, as reinforcement stiffness increases, reinforcement loads have been observed to increase when all other factors are the same (Allen et al. 2003). This effect can be quantified using a global reinforcement stiffness value computed as:

$$S_{global} = \frac{1}{H}\sum_{i=1}^{n} J_i \tag{1}$$

Where: J_i = secant tensile stiffness of an individual reinforcement layer; n = number of reinforcement layers, and; H = wall height. The secant stiffness value has been computed in two ways: 1) using the results of conventional in-air (index) tensile constant rate of strain tests, and; 2) stiffness values computed at 2% strain after 1000 hours from conventional constant-load (creep) tests. The data show that creep-based values give similar global stiffness values for Walls 2 and 5. Global stiffness values interpreted from creep tests are judged here to better represent the in-situ stiffness of the reinforcement at the end of construction (Walters et al. 2002).

2.3 *Soil*

The backfill was a poorly graded, naturally deposited rounded beach sand (SP according to the Unified Soil Classification System) with D_{50} = 0.34 mm, coefficient of curvature C_c = 2.25 and coefficient of uniformity C_u = 1.09. Hatami and Bathurst (2005) reported the results of direct shear tests conducted under confining pressures representative of vertical stress levels in the walls. They reported the peak direct shear friction angle as ϕ_{ds} = 41° and constant volume friction angle as ϕ_{cv} = 35°. The (secant) peak plane strain friction angle was also determined directly from plane strain (bi-axial) compression tests carried out under similar load levels and reported as ϕ_{ps} = 44°. The fines content (particle sizes < 0.075 mm) was less than 1%. An unavoidable change in compaction plant occurred between Walls 1 and 2, and Walls 5 and 6. The result was that the compaction energy was greater for the last two walls (electrical powered vibrating rammer) than for the first two walls (vibrating plate). However, because of the flat compaction curve for the uniform size sand soil the as-compacted densities between walls were very close (bulk unit density γ = 17 kg/m^3). There were detectable differences in wall performance that were ascribed to compaction method – specifically, end of construction wall deformations using the heavier compaction were 60% to 80% greater and horizontal toe loads were 40% to 75% greater. However, the influence of compaction plant on reinforcement loads and strains was not detectable between walls at end of construction and during subsequent surcharging. A complete discussion on the effects of compaction on wall response can be found in the paper by Bathurst et al. (2009).

Table 1. Mechanical properties of reinforcement products.

Wall number	Reinforcement type	Machine direction tensile strength using method of test ASTM D 6637 or D 4595: Strength at 5% strain (kN/m)	Machine direction tensile strength using method of test ASTM D 6637 or D 4595: Ultimate strength (T_{ult}) (kN/m)	Index global reinforcement stiffness [a] (kN/m^2)	Global reinforcement stiffness [b] (kN/m^2)
1	PP integral drawn geogrid	9.0	12.6	477	154
2	Modified PP geogrid	4.5	6.3	238	77
5	PET geogrid PVC coated	5.3	16.2	153	80
6	14 gauge welded wire mesh (WWM)	-	7.1 (yield strain = 0.19%)	5170	

[a] secant stiffness J_i taken at 2% strain from in-isolation constant rate of strain (CRS) tensile tests carried out at 10% strain/minute (ASTM D 6637 or D 4595); [b] secant stiffness J_i taken at 2% strain and t = 1000 hours from in-isolation constant load (creep) tests

3 COMPARISON OF MEASURED AND PREDICTED MAXIMUM REINFORCEMENT LOADS

The "measured" maximum reinforcement loads can be compared to predicted loads using the AASHTO (2002) Simplified Method (tie back wedge method for geosynthetic reinforced soil walls) and the most recent version of the K-stiffness Method (Bathurst et al. 2008) which is an empirical-based working stress method. The AASHTO calculation for maximum reinforcement load (T_{max}) is expressed as:

$$T_{max} = K(\gamma z + q)S_v \tag{2}$$

Here: z = depth of reinforcement layer below the crest of the wall; q = uniform surcharge pressure, and; K = active earth pressure coefficient calculated as:

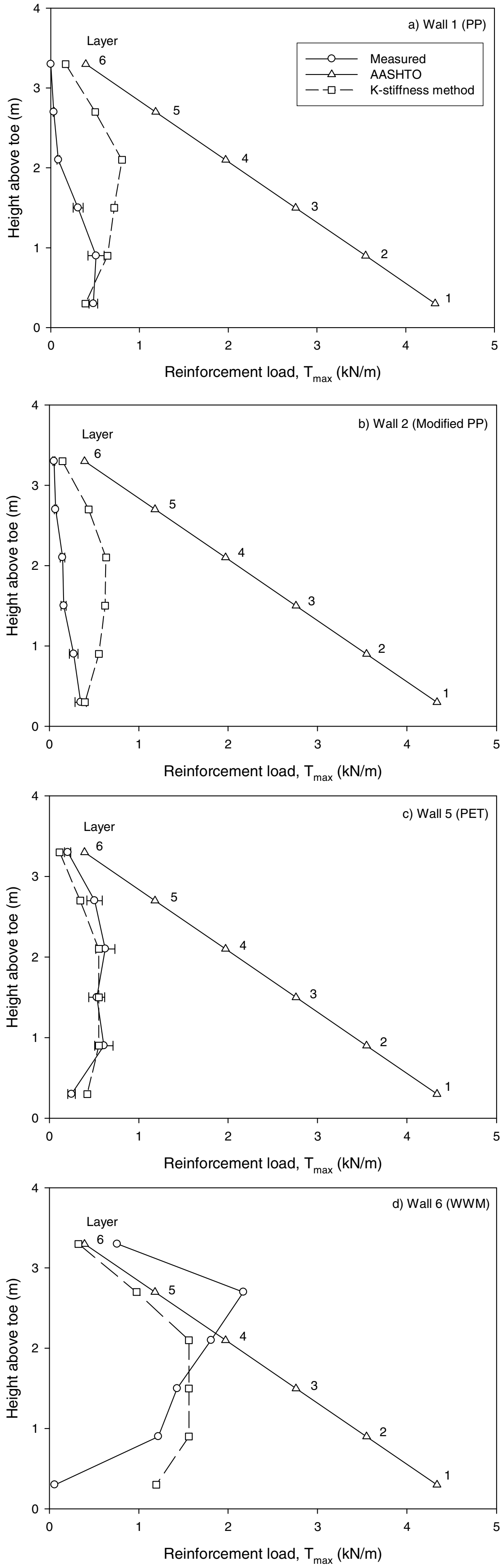

Figure 2. Predicted versus measured maximum reinforcement load in instrumented layers at end of construction.

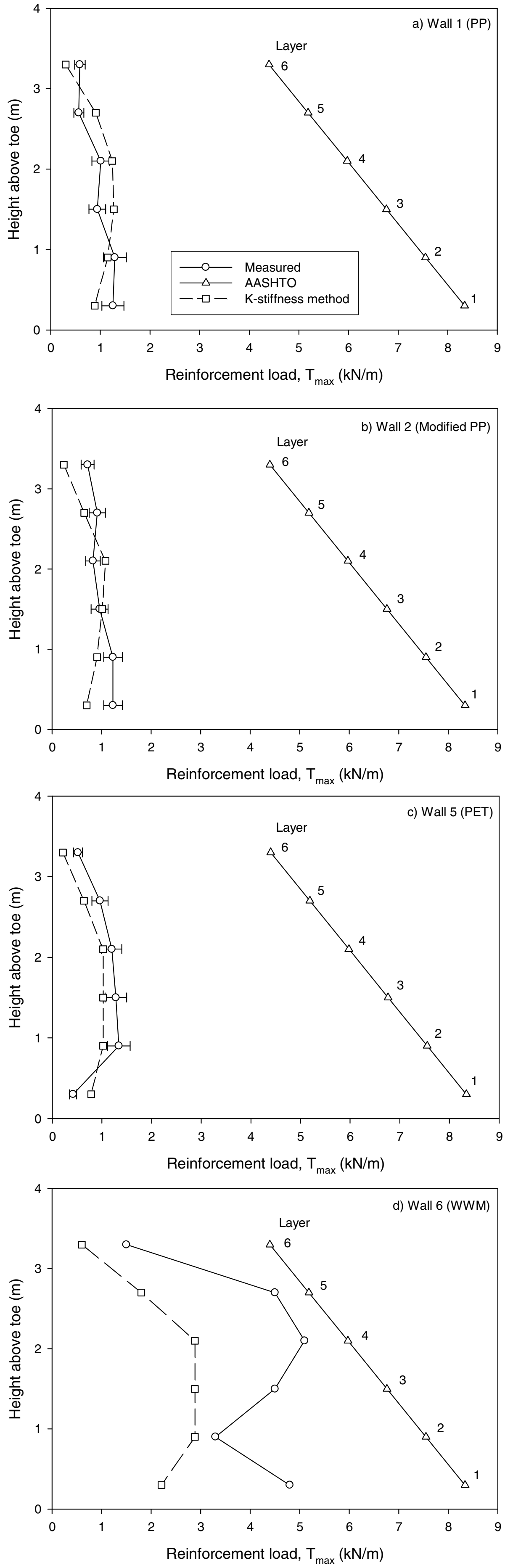

Figure 3. Predicted versus measured maximum reinforcement load in instrumented layers at q = 50 kPa.

$$K = \cos^2(\phi+\omega)/\cos^2\omega\left(1+\frac{\sin\phi}{\cos\omega}\right)^2 \quad (3)$$

All other parameters have been defined previously. The equivalent expression using the K-stiffness Method is:

$$T_{max} = \frac{1}{2}K\gamma(H+S)\,S_v\;D_{tmax}\Phi_g\Phi_{local}\Phi_{fs}\Phi_{fb}\Phi_c \quad (4)$$

Here: S = equivalent height of uniform surcharge pressure q (i.e. $S = q/\gamma$); D_{tmax} = load distribution factor that modifies the reinforcement load based on layer location. The remaining terms, Φ_g, Φ_{local}, Φ_{fs}, Φ_{fb} and Φ_c are influence factors that account for the effects of global and local reinforcement stiffness, facing stiffness, face batter and soil cohesion, respectively. The coefficient of lateral earth pressure is calculated as $K = 1 - \sin\phi$ with $\phi = \phi_{ps}$ = secant peak plane strain friction angle of the soil. However, it should be noted that parameter K is used as an index value and does not imply that at-rest soil conditions exist in the reinforced soil backfill according to classical earth pressure theory. For the four RMC walls in the current study, the values for influence factors are: Φ_g = 0.32, 0.26, 0.24 and 0.66 for Walls 1, 2, 5 and 6, respectively; Φ_{local} = 0.83 to 1.13, Φ_{fs} = 0.51, Φ_{fb} = 0.86 and Φ_c = 1. The reader is directed to the paper by Bathurst et al. (2008) for details to calculate these values for the RMC walls. To keep comparisons between the two calculation methods independent of choice of friction angle, the same value of peak friction angle is used in both sets of calculations (i.e. $\phi = \phi_{ps}$ = secant peak plane strain friction angle of the soil). For the polymeric reinforced soil walls the reinforcement loads were computed using secant stiffness values taken at 1000 hours and 2% strain as recommended for design. Recall that the WWM for Wall 6 was selected to represent an idealized “geogrid” with an ultimate strength value comparable to that of the polymeric materials in this study but with very much greater stiffness. Hence, the predicted reinforcement loads for this wall are computed as if this wall was a geosynthetic reinforced soil wall.

The “measured” maximum reinforcement load in a layer is calculated as:

$$T_{max} = J(t, \varepsilon_{max}) \times \varepsilon_{max} \quad (5)$$

where the secant stiffness value (J) corresponds to the time (t) since beginning of construction and the measured peak internal strain value in the layer (ε_{max}). The accuracy of the estimation of reinforcement loads using secant stiffness values from laboratory in-isolation constant load tests, duration of loading and measured strain has been demonstrated by Walters et al. (2002) by direct comparison with connection loads measured in Wall 1 and an instrumented field structure with in-line load cells attached directly to reinforcement layers.

Measured and predicted maximum reinforcement loads are plotted in Figures 2 and 3 for the four RMC walls at end of construction and a common surcharge level of 50 kPa, respectively. The measured data show that the reinforcement loads are very much more uniform with depth for the polymeric reinforcement walls and average loads are much lower than values computed using the AASHTO (2002) Simplified Method for geosynthetic reinforced soil walls. The difference in measured and predicted load values using the Simplified Method increases with depth of layer below the wall crest. The discrepancy can be expected to be even greater if lower friction angle values were used in the calculations (i.e. constant volume values, or peak values from direct shear or triaxial compression tests that are not corrected to higher plane strain values). The values predicted using the K-stiffness Method generally capture the more uniform load distribution with depth and are closer in magnitude to measured values for the three polymeric (geogrid) walls. However, both the AASHTO Simplified Method and the K-stiffness Method do not do as well capturing the measured loads for the WWM wall. It can be argued that the AASHTO Simplified Method is a better choice to predict loads for the WWM wall because maximum predicted loads are greater than measured values and hence safer for design. An explanation for the poor performance of the K-stiffness Method for this wall is that the method was calibrated for field walls having measured global stiffness values (i.e. Equation 1 with J computed using actual measured strains at end of construction and laboratory creep data) in the range of 90 to 1500 kN/m^2 with most walls in the range of 400 to 600 kN/m^2 (Bathurst et al. 2008). These values fall well below the value of about 5200 kN/m^2 for the WWM structure.

It should be noted that the K-stiffness Method is an empirical-based method with coefficient terms determined by back-fitting to measurements recorded in a database of instrumented field walls with surcharge levels generally less than S = 1 m. Nevertheless, the method does very well at an equivalent surcharge height of about S = 3 m for the polymeric-reinforced RMC walls.

4 CONCLUSIONS

This study has shown that reinforcement loads in reinforced soil walls are significantly influenced by the global stiffness of the reinforcement consistent with trends predicted by the empirical-based K-stiffness Method (e.g. Allen et al. 2003, Bathurst et al. 2008). Comparison of computed maximum internal reinforcement loads (i.e. not at the connections) with predicted values, using the AASHTO Simplified Method for geosynthetic reinforced soil walls and the K-stiffness Method, show that the latter was more accurate and able to capture the uniform reinforcement load distributions for the three polymeric-reinforced (geogrid) walls in this study. The AASHTO method was found to over-estimate reinforcement loads by up to a factor of eight for these walls.

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