Geosynthetics, J. Kuwano & J. Koseki (eds)
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Study of the face deformation of brick faced reinforced walls

Correia, A.A.S. & Pinto, M.I.M.
Department of Civil Engineering, University of Coimbra, Portugal

Lopes, M.L.
Department of Civil Engineering, University of Porto, Portugal

Keywords: reinforced soil walls, deformation predictions

ABSTRACT: For the development of the soil-reinforcement interaction, the retaining walls must deform. However, this deformation should not exceed a certain threshold value known from previous experiences. The need to predict and control the deformations in retaining structures reinforced with geosynthetics is therefore of utmost importance. The limit equilibrium methods traditionally used in the design of this type of structures do not supply any information concerning the deformations. The deformation, which is dependent of a large number of factors, can be determined by numerical analyses and eventually by other less complex methods. This work describes a simple method based on a simple theoretical model, which was found to be suitable and quick to predict the face deformations of the brick faced masonry walls reinforced with geosynthetics. The results were successfully compared with laboratorial measurements.

1 INTRODUCTION

A great amount of research was already carried out both in laboratory and in the field in order to better understand the behaviour of the retaining structures reinforced with geosynthetics and of their materials. Nevertheless all the developments, much remains to be done, namely the development of design methods that bring deformation into consideration.

The design of the retaining structures reinforced with geosynthetics should be based on the study of several limit states (states beyond which the structure no longer satisfies the design performance requirements, according to ENV 1991-1, 1994). The selected design situations should be sufficiently severe and varied so as to encompass all reasonable conditions, which can be foreseen to occur during the construction and the use of the proposed structure (Jewel, 1996). The selected design states should satisfy both the stability and the deformation requirements. The later requirement is specially important due to the extensible nature of the reinforcement. Compatibility of deformations between soil and reinforcement is a key issue.

The design methodology should (ENV 1991-1, 1994):

– fulfil the stability requirement by Ultimate Limit States (ULS) analysis;
– fulfil the deformation requirement by Serviceability Limit Sates (SLS) analysis.

The most widespread design methods for soil structures reinforced with geosynthetics are based on Limit Equilibrium Methods (LEM) and are well established nowadays (McGown, 2000). These methods are in the great majority of situations safe but very conservative. These methods (LEM) only allow the analysis of safety as far as the ULS is concerned and do not regard for deformations. However, design will only be completed with the analyses of the SLS. This can be done by numeric analyses or eventually by less complex methods.

Due the current lack of design methods concerned with the deformation requirement, the authors develop a method to predict the face deformation of brick faced reinforced retaining walls. This work describes the method developed and presents some of the comparisons made.

2 BRICK FACED REINFORCED RETAINING WALLS

2.1 *Presentation*

The brick faced reinforced retaining walls combine the advantages of a brick wall with the soil reinforcement technique.

The behaviour of this type of walls has been studied for quite some time. Pinto (1992) carried out a laboratorial study on small model reinforced soil retaining walls (scale 1:5) of those walls previously studied by Walsh (1987). Later, in 1999, Pereira modified a two-dimensional numeric model to allow the study of brick faced reinforced soil retaining walls. Finally Correia (2003) and Correia et al. (2004a, 2004b, 2005) studied the applicability of some known simple design methods to this type of walls.

The walls studied are constructed on a rigid foundation ground, as the great majority of those investigated previously by the authors referred. All the walls (Figure 1) are 300 mm high, 240 mm long, 20.5 mm thick, built on top of a rigid strip foundation. The reinforcement is a non woven geotextile with an arrangement that combines two different lengths (80 and 120 mm) with two vertical spacings (3 and 4 layers of bricks). A surcharge load, is applied to the whole area of the free surface of the backfill.

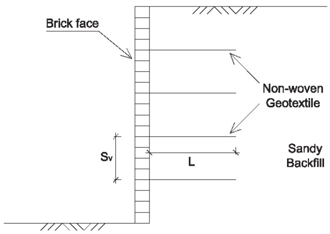

Figure 1. Brick faced reinforced retaining wall.

The characteristics of the materials used in the construction of the walls and of the respective interfaces are presented in Table 1.

A simple nomenclature was defined for the different walls studied in order to allow a more clear presentation and discussion of the results and that is shown in Table 2.

Table 1. Characteristics of the materials and interfaces.

Backfill (sand)	γ (kN/m^3) 16.3	ϕ'_p (°) 40	c′ (kN/m^2) 0
Reinforcement	t_{GTX} (mm) 0.3	T (kN/m) 1.4	E (kPa)$^{(1)}$ 13 200
Brick face	γ (kN/m^3) 24	b (N/mm^2)$^{(2)}$ 0.03	E (kPa)$^{(3)}$ 50 000
Soil-reinforcement	Ca (kN/m^2) 0	$f_{s/GSY} \times$ tg ϕ' 0.73	
Soil-brick face	Ca (kN/m^2) 0	f $\times$ tg ϕ' 0.75	

(1) elastic modulus for the unconfined reinforcement; (2) brick mortar bond strength; (3) brick mortar bond stiffness

Table 2. Nomenclature defined for the studied walls.

Nomenclature	Reinforcement arrangement	
	Vertical spacing, S_v	Length, L
4-S	each 4 brick rows (60 mm)	80 mm
3-S	each 3 brick rows (45 mm)	80 mm
4-L	each 4 brick rows (60 mm)	120 mm
3-L	each 3 brick rows (45 mm)	120 mm

2.2 *Wall failure mechanism*

The inclusion of short sheets of reinforcement into the traditional brick masonry wall is responsible for a behaviour significantly different from that displayed by the unreinforced walls, as far as the failure mechanism is concerned.

Walsh (1987), Pinto (1992) and Pinto and Cousens (1996) described that, independently of the reinforcement arrangement, the reinforced walls always begin to move in a similar way to those traditional unreinforced walls. The walls rotate as a rigid body around a crack developed (and visible along all the extension of the wall) in the first bed joint mortar-brick, i.e., at the foundation level. This movement continues until failure occurs, on unreinforced walls and on walls reinforced every 4 brick rows. Walls reinforced every 3 brick rows show a complete different mechanism, because they develop a second crack during loading, at about mid-height of the wall. After the development of this second crack the wall starts to move as two independent rigid blocks (Figure 2) until failure occurs.

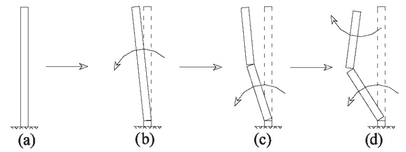

Figure 2. Brick faced reinforced retaining wall failure mechanism for walls 3-S and 3-L (Pinto, 1992).

3 PROPOSED METHOD FOR DETERMINING THE FACE DEFORMATION

3.1 *Introduction*

Face deformation predictions of reinforced retaining structures can be done by numeric analyses. In spite of their enormous advantages, the numeric analyses are not of widespread use by designers because of its enormous costs, especially due to a higher demand for a much more complete characterization of the materials, interfaces, etc.

This section describes a new method proposed by the authors for prediction of the face deformation of the brick faced reinforced retaining walls. This is a very simple method, based on a simple theoretical model, and was shown to give good results when compared with the measurements taken on the same type of walls.

3.2 *Proposed method*

The method proposed herein predicts the deformations of the brick faced reinforced retaining walls and therefore allows the verification of the deformation requirement (SLS). It takes into account that deformation depends strongly on the relative

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deformability of the constituent materials. That is, it considers not only the deformability of the face (masonry), the deformability of the reinforcing elements (geotextile) and the deformability of the backfill material (sand) but also the compatibility between all these deformations.

The theoretical model considered in this study is a beam simply leaning on flexible supports (spring type) to simulate the reinforcing elements and subject to a theoretical diagram of horizontal earth pressures (Figure 3). The support at the base level has one degree of freedom (rotation) in order to simulate the crack developed at the foundation level, phenomenon that was observed in all the walls tested in laboratory and described above. The deformability of the beam is conditioned by the bending stiffness of the brick masonry face (Table 1). The flexible supports are simulated by connecting rods with axial stiffness and tensile strength defined by the reinforcement (Table 1). The stiffness of the connecting rod can vary along the height of the wall to allow the inclusion of the confinement effect on the reinforcement due to the backfill above.

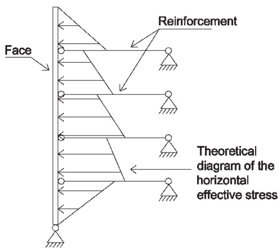

Figure 3. Theoretical model for the proposed method.

The theoretical diagram considered for the horizontal earth pressures acting on the face of the wall intends to take into account the effect of the deformability of the soil and the compatibility of deformations requested for the equilibrium. It is essential that the diagram of the horizontal stresses acting on the model beam simulate closely the real stresses acting on the face of the wall (taken herein as those measured during laboratory programme and reported by Pinto, 1992; Pinto and Cousens, 1996). The predictions of the horizontal stresses acting on the face of the wall need to be made in a simple way, which is essential for a method that pretends to be simple, but still as much accurate as possible. For that reason, the following equation was used (based in studies described by Osman et al., 1979) to allow the assessment of the maximum tensile force mobilized in the different reinforcing elements:

$$T_{\max}(z) = \sqrt{\frac{6 \times K^{2.5}}{L}} \times \sigma_v \times S_v \times S_h \times \sqrt{H - z} \quad (1)$$

where S_v and S_h are the vertical and horizontal reinforcement spacing respectively, L is the length of the reinforcement (taken as uniform along the height of the wall), H is the height of the wall, σ_v is the vertical stress acting on the reinforcing element, K is the earth pressure coefficient and z is the depth of the reinforcing element.

Assuming that the tensile force mobilized in each reinforcing element is entirely balanced by the horizontal stress in the area of its influence, then it is possible to estimate the horizontal stress diagram. The variation of the horizontal stresses in depth is then quantified by considering two horizontal earth pressure coefficients (correspondent to at rest and to active states, i.e., K_o and K_a respectively) to act immediately on top and bellow the reinforcement in that order. The maximum tensile force obtained for the surface level must be neglected, as there is no reinforcing element at that level. The value of the maximum tensile force at the base level is zero, and therefore the same value was considered for the horizontal stress as can be seen in Figure 3. This value is due to the presence of the foundation, which is an element of high stiffness (Correia, 2003 and Correia et al., 2005).

4 RESULTS

This section shows the results of the application of the proposed method for the prediction of the deformation of the face of the wall.

The stiffness modulus of the reinforcement was considered to be constant along the total height of the wall and with the value of E = 13 200 kPa (Table 1). A study was conducted to analyse the influence of the confinement effect on the stiffness modulus of the reinforcement. The results showed a negligible effect, probably due to the small levels of vertical stress acting on the small model walls studied.

Figure 4 shows the predictions of the deformation of the face of the walls obtained by using the proposed method. This Figure shows also the deformations determined from the numerical analysis and those measured during laboratory studies, all of them correspondent to a surcharge load of 2.445 kPa. It can be seen that there is a very good agreement between the deformation predicted by the proposed method and the deformation measured during laboratorial studies, for walls 3-L and 4-L. Comparisons on walls with shorter reinforcing elements (walls 3-S and 4-S) are not as good as on the previous walls. Correia (2003) and Correia et al. (2004a, 2004b) explain that these walls fail by pullout of the reinforcement because of the insufficient pullout length: the face of the wall and the reinforcing elements move partially together, which means that the reinforcement is not working fully as reinforcement. This mechanism seems to be more important on walls

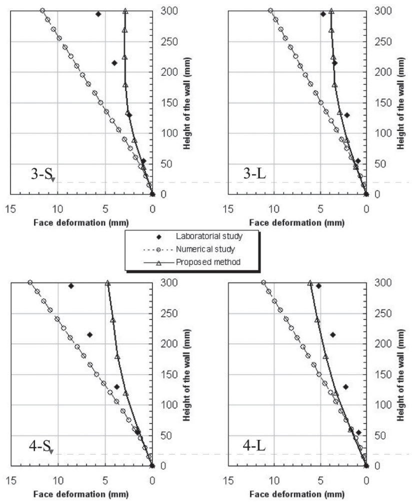

Figure 4. Face deformation for a surcharge load of 2.445 kPa.

reinforced each 4 brick rows (Figure 4 shows indeed that is the situation with less agreement between the measured and predicted deformation), which were the walls reported as deforming in a more similar manner to the unreinforced walls.

The profile of the wall 3-L is shown in Figure 5. It can be seen the great similarity between this profile and the failure mechanism shown in Figure 2 c).

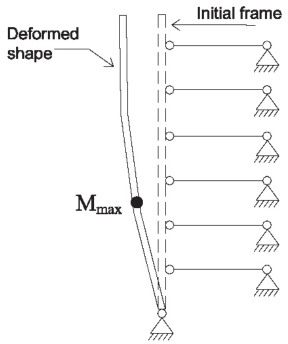

Figure 5. Face profile predicted for the wall 3-L.

5 CONCLUSIONS

The design of the retaining structures reinforced with geosynthetics should be based on the stability (ULS) and the deformation (SLS) requirements.

The method proposed here to predict the deformation of the face of the brick faced walls reinforced with geosynthetics is a very simple method, based on a simple theoretical model, and gives good results when compared with measurements taken on the same type of walls.

ACKNOWLEDGEMENTS

The authors wish to extend special thanks to the financial support given by: “Fundação para a Ciência e Tecnologia” (FCT) through the Project PBIC/C/CEG/2387/95 and the Scholarship FMRH/BIC/6003/98; – FCT, POCTI and FEDER through the Project POCTI/ECM/42806/2001.

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