BRICK FACED REINFORCED RETAINING WALLS – COMPARISONS BETWEEN DIFFERENT DESIGN METHODS

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ABSTRACT: Since their first use in the sixties, the retaining structures of reinforced soil have suffered a notable development. Important progresses were made in the understanding of their behaviour, as well as in the constructive processes. Nevertheless the developments, much remains to be done, especially regarding the design methods. The present work aims to contribute to a better understanding of the design methods of this type of structures, especially for brick faced reinforced retaining walls. A study is presented concerning the applicability of some analysis methods to the design of walls of brick masonry reinforced with geossynthetics. The results obtained from the different design methods used in the study (Rankine, Coulomb, planar failure surface, circular failure surface and two-wedge failure surface) are compared with results obtained on previous studies on laboratorial tests as well on a numeric analysis, on brick faced reinforced retaining walls. It is known that the quality of the results supplied by the several calculation methods is directly related to the approach of the real failure surface of the wall. The results obtained showed that the real failure surface of the wall is closely predicted by the circular and two-wedge failure surface. The later method provided the best predictions both for surcharge loads at failure and failure surfaces.

1 INTRODUCTION

During the last 30 years a great amount of research was carried out both in laboratory and in the field with the objective of better understand the behaviour of the retaining structures reinforced with geosynthetics and of their materials. Nevertheless all the developments, much remain to be done, especially regarding the development of suitable design methods.

Due to the current lack of design methods for the design of brick faced reinforced retaining walls, the authors studied the applicability of five known design methods, namely Rankine, Coulomb, plane failure surface, modified Bishop and two-wedge failure surface. Comparisons between the results predicted by application of these design methods with those observed in laboratorial (Pinto, 1992) and numeric studies (Pereira, 1999), allowed the authors to reach important conclusions on the validity of these referred design methods when used in brick faced reinforced retaining walls with geosynthetics.

2 BRICK FACED REINFORCED RETAINING WALLS

The brick faced reinforced retaining walls combine the advantages of a relatively resistant and durable face (as the brick masonry one), with the technique of soil reinforcement, resulting in a significant increase of the resistant capacity.

The behaviour of this type of walls has been studied for some time, initially in United Kingdom where this technique was born and patented, and more recently in Portugal. In 1992, Pinto carried out a laboratorial study on small model reinforced soil retaining walls (scale 1:5) of those prototype 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.

The brick faced reinforced soil retaining walls were further studied by Correia in 2003 and part of those studies are described herein. These walls are constructed on a rigid foundation ground, which correspond to those walls investigated by Pinto (1992) and Pereira (1999). These walls (Figure 1) are 300 mm high, 240 mm long and with a brick face 20.5 mm thick, constructed on top of a rigid strip foundation 15 mm high, with the same length and thickness of the face. The reinforcement material is a non woven geotextile. The reinforcement arrangement combines two different lengths (80 and 120 mm) with two vertical spacing (3 and 4 layers of bricks, i.e., 45 and 60 mm respectively). A surcharge load, q (kPa), is applied at the fill surface (top of the wall).

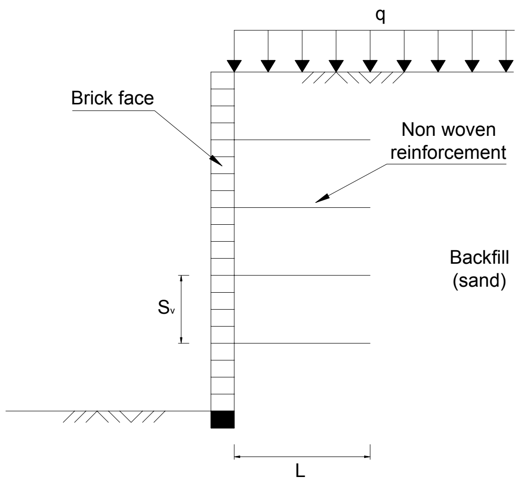

Figure 1 Brick faced reinforced retaining wall

The characteristics of the materials of the walls are presented in Table 1. Table 2 shows the characteristics of the interfaces. In order to make the presentation and discussion of the results more easily, a simple nomenclature was defined for the different walls studied and that is shown in Table 3.

Table 1 Characteristics of the materials

Backfill (sand)	γ (kN/m^3)	ϕ'_p (º)	c' (kPa)
	16.3	40	0
Reinforcement	t (mm)	T (kN/m)	ε_f (%)
	0.3	1.4	15
Brick face	γ (kN/m^3)	b (N/mm^2) (1)	
	24	0.03	

(1) Brick/mortar bond strength

Table 2 Characteristics of the interfaces

Soil-reinforcement	C_a (kN/m^2)	$f \times tg\phi'$
	0	0.73
Soil-brick face	C_a (kN/m^2)	$f \times tg\phi'$
	0	0.75

Table 3 Nomenclature defined for the studied walls

Nomenclature	Reinforcements arrangement	
	Vertical spacing, S_v	Length, L
4-s	each four brick layers (60 mm)	80 mm
3-s	each three brick layers (45 mm)	80 mm
4-l	each four brick layers (60 mm)	120 mm
3-l	each three brick layers (45 mm)	120 mm

3 DESIGN METHODS

The design methodology used in this study is based on the traditional concept of a Global Factor of Safety FS_G (although it is known that the equilibrium of the soil reinforced structures is too much complex to be represented by a single Factor of Safety). This option is however justified by the widespread design practice.

The design methods for the analysis of external and local stability were not used herein, as none of these limit states are important to the walls studied, according to Pinto, 1992 and Pereira, 1999. Therefore, in this work, great importance was given to the internal stability design methods. The limit states due to tensile failure and pull-out failure were considered, and 5 known design methods were used: Rankine, Coulomb, Plane failure surface, Bishop modified and Two-wedge surface failure. The main characteristics of each of these methods are described in this section.

For all of the methods employed in this study, the reinforcement layers were assumed to remain horizontal during surcharging. The results obtained are discussed in section 4.

3.1 *Rankine and Coulomb*

Rankine's method is classified as a local equilibrium method as it studies each reinforcement layer individually. The horizontal stress diagram area corresponding to each reinforcement layer must be balanced by the tensile force mobilized in that reinforcement layer. Consequently, the maximum tensile force developed occurs on the lowest reinforcement layer. This force is then compared to the reinforcement tensile strength and bond strength mobilized along the reinforcement anchorage length.

The Coulomb's method, which is classified as a global equilibrium method, considers the overall contribution of all the reinforcement layers, without any concern about establish the portion mobilized in each reinforcement layer. The total force on the reinforcements, which must be equal to the active earth pressure, is then distributed to each reinforcement layer by assuming a linear distribution with depth (according to Schlosser and Vidal, 1969). In this method both the force and moment equilibrium can be considered. The maximum tensile force developed occurs on the lowest reinforcement layer. This force is then compared to the reinforcement tensile strength and bond strength mobilized along the reinforcement anchorage length.

Both methods (Rankine and Coulomb) have the same potential failure surface (Figure 2).

3.2 *Plane failure surface*

This method considers that the internal instability can occur along a plane failure surface (Figure 2). This is also a global equilibrium method and therefore the safety analysis of the wedge defined by the failure surface is made by a simple force equilibrium. Only the reinforcement layers intersected by the failure surface are considered to contribute to the stability. The mobilized force on each reinforcement layer intersected by the failure surface is the smallest of the following values: tensile strength and bond strength developed along the reinforcement anchorage length.

By repeating the calculations for different potential failure surfaces, the critical failure surface can be found which correspond to that of the lowest value for the Factor of Safety, FS_G. During this study, the inclination to the horizontal, β, of the failure surface varies from ϕ' to $45+\phi'/2$, while the height of the intersection point in the wall face also change.

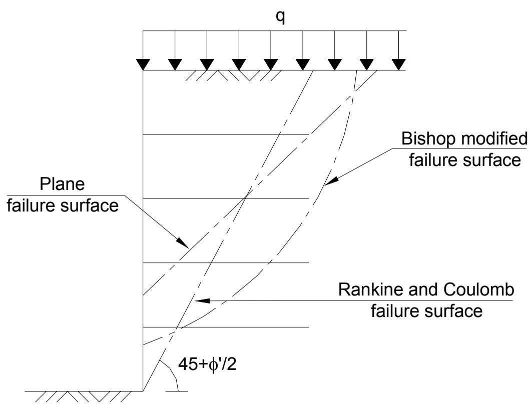

Figure 2 Failure surfaces correspondent to the different design methods used in the stability analysis

3.3 *Bishop modified (circular failure surface)*

This method, also considered as a global equilibrium, considers that the internal instability can occur along a potential circular failure surface (Figure 2).The safety analyzes is made by considering the division of the potential instable soil mass in vertical slices followed by the moment equilibrium driven to the centre of the circle. Similar to the previous method described, only the reinforcement layers intersected by the failure surface are considered to contribute to the stability. The mobilized force on each reinforcement layer intersected by the failure surface is the smallest of the following values: tensile strength and bond strength developed along the reinforcement anchorage length.

The critical surface can be found (the one corresponding to the lowest value of the Factor of Safety, FS_G), after assuming different potential failure surfaces. In these surfaces both the radii R and the centre of the circle can vary.

3.4 *Two-wedge surface failure*

This method is also classified as a global equilibrium method. The stability analysis is made by considering the equilibrium forces of the two wedges as represented in Figure 3 a and b. Therefore, for a group of surfaces with the same nodal point (x, y) but different inclinations (θ_1 and

θ_2), the interwedge equilibrium forces are calculated, F'_{c1} and F'_{c2}, respectively on the wedge 1 and 2 (Figure 3 b). For the nodal point in study and for each combination of angles θ_1 and θ_2, the sum of the forces F'_{c1} and F'_{c2} must be null, i.e., there will be only equilibrium when FS_G assumes a single value. On the other hand, for the nodal point considered, the minimum value of this FS_G corresponds to a single pair of angles θ_1 and θ_2.

By considering a mesh of nodal points it is possible to find the most critical failure surface, i.e., the surface with the lowest value of the Factor of Safety, FS_G.

According to Woods and Jewell (1990) the inclination of the interwedge frictional forces are assumed to be null, i.e., the interface strength between the wedges is considered to be insignificant.

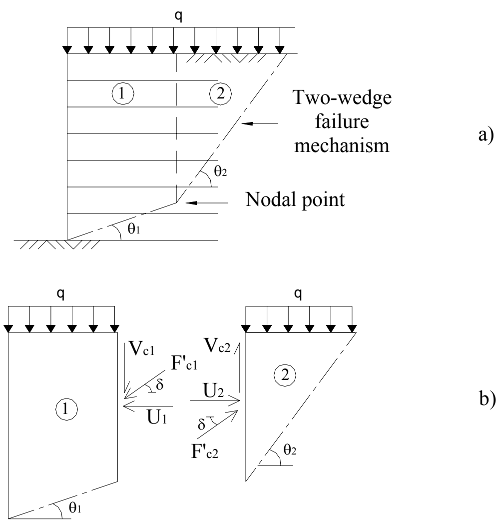

Figure 3 Two-wedge failure mechanism

4 ANALYSIS OF RESULTS

4.1 *Rankine and Coulomb*

Table 4 summarises the results obtained by the Rankine and Coulomb's design methods when applied to the study of the internal stability of the brick faced reinforced retaining walls.

Table 4 FS_G predicted by the Rankine and Coulomb methods (q = 2.445 kPa)

Design method		3-s	4-s	3-l	4-l
Tensile failure	Rankine	14.06	10.84	14.06	10.84
	Coulomb (Forces equilibrium)	14.72	10.97	14.72	10.97
	Coulomb (moment equilibrium)	17.08	13.17	17.08	13.17
Pullout	Rankine	NA(1)	NA(1)	NA(1)	0.60
	Coulomb (Forces equilibrium)	3.20	2.10	7.41	5.11
	Coulomb (moment equilibrium)	2.26	1.62	6.92	5.11

(1) not applicable

The results show that for both methods the walls fail by pullout mechanism, which is in accordance to the behaviour observed by Pinto (1992) and Pereira (1999). With the exception of Rankine' method, all the walls exhibit internal stability ($FS_G \geq 1.0$). Due to the fact that the Rankine' method is classified as a local equilibrium method then in order to assure internal stability all the reinforcement layers must be intersected by the potential failure surface. This does not happen for the walls 3-s, 4-s and 3-l, which shows that the reinforcement layers in these walls have an insufficient length (in Table 4 this situation is referred as not applicable: NA). Although in the wall 4-l all the reinforcement layers intersect the potential failure surface, as the upper reinforcement layer exhibit an insufficient anchorage length, the Factor of Safety is lower than 1.0.

4.2 *Plane failure surface*

The results obtained by using the plane failure surface design method on the walls under investigation are presented in Table 5. The results show internal stability for all the walls studied ($FS_G \geq 1{,}0$), although the wall 4-s is near the limit equilibrium. The lower value of the Factor of Safety on this wall can be explained by the fact that this wall has the lowest amount of reinforcement.

Table 5 FS_G predicted by the plane failure surface, Bishop simplified and Two-wedge failure mechanism (q = 2.445 kPa)

Design method	3-s	4-s	3-l	4-l
Plane failure surface	1.26	1.07	1.87	1.52
Bishop simplified	0.92	0.77	1.23	1.07
Two-wedge failure mechanism	0.82	0.82	1.21	0.96

4.3 *Bishop modified (circular failure surface)*

Table 5 summarises the results obtained by applying the Bishop modified design method. With this method and for all the walls, the critical failure surface never intersects the reinforcement layers.

The increment of the reinforcement length leads to an elongation of the critical failure surface into the backfill, involving a greater volume of soil, and therefore a higher resistant force is mobilized.

These results are qualitatively in conformity with Pinto (1992) and Pereira (1999).

4.4 *Two-wedge failure surface*

The results obtained by applying the two-wedge failure surface design method to the study of the internal stability of the analyzed walls are summarized in Table 5. This table shows that only the wall 3-l presents safety relatively to the internal stability. This fact is comprehensible because this wall has the higher amount of reinforcement.

It can be observed that the reduction of the vertical spacing produces an improvement in the behaviour of the walls, only on walls with long reinforcement (3-l and 4-l). Hence, according to this method of design, walls 3-s and 4-s present an insufficient reinforcement length.

Once again, these results are qualitatively in agreement with Pinto (1992) and Pereira (1999).

4.5 *Comparisons of predicted and observed failure surface*

It is known that the quality of the results supplied by the different design methods is directly related with the accurately approach of the real failure surface of the structure (Gomes *et al.*, 1994; Lanz, 1992; Palmeira *et al.*, 1996). Figure 4 presents the critical failure surfaces predicted by the several design methods in study, as well as the failure area obtained in the numeric studies performed by Pereira (1999), which are similar to those observed in the laboratorial studies carried out by Pinto (1992). These authors present only the results for the wall 4-s but they referred identical results for the other walls studied.

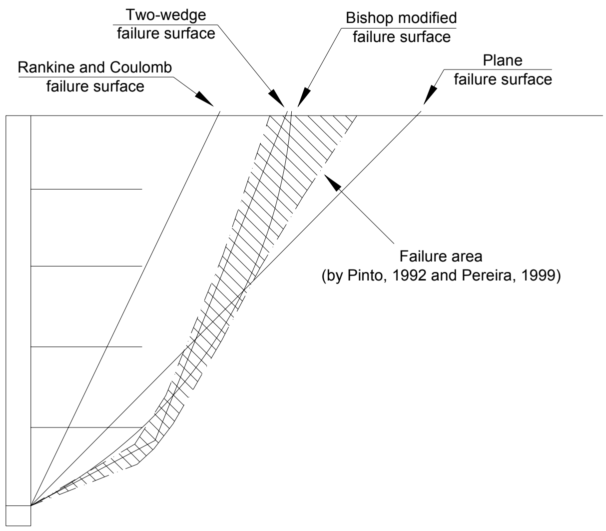

Figure 4 Failure area and critical failure surfaces predicted by different design methods for the wall 4-s

By analysing Figure 4 it can be concluded that the approach of the real failure surface made by applying the Rankine, Coulomb and plane failure surface methods is of weak quality. Therefore it is expected that the results supplied by application of these methods are also of weak quality.

Both the Bishop modified and the two-wedge failure surface methods seem of good quality approach, since the critical failure surfaces predicted by them are very close to the observed failure area. In fact, if both methods are considered simultaneously, the failures surfaces can be considered for the limits of the real area. For that reason it is expected that the results obtained by these methods are of good quality.

5 CONCLUSIONS

The critical failure surface predicted by the Rankine and Coulomb methods was developed for the situation of a homogenous and unreinforced soil, supported by a rigid structure, which in fact exhibit a different behaviour from that of the brick faced retaining wall reinforced with geosynthetics. Both Coulomb and plane failure surface methods suggest the existence of internal stability for several analyzed walls, which is in contradiction to the behaviour observed by Pinto (1992) and Pereira (1999). For these reasons, among others, it is not advisable the application of the Rankine, Coulomb and plane failure surface methods for the internal stability design for this type of walls.

For the remaining design methods, i.e., Bishop modified and two-wedge failure surface, both methods are more conservative for walls with shorter reinforcement (4-s and 3-s), although the same no longer happens for walls 3-l and 4-l. Nevertheless, it can be concluded that the two-wedge failure surface method is always the most conservative one. Furthermore, the two-wedge failure surface method is far more versatile in the research of the critical failure surface, and also gives a closer real failure surface, which is in agreement with Woods *et al.*, 1990; Silva, 1991; Pinto, 1992; Lopes, 1992; Palmeira *et al.*, 1996; Jewell, 1996. The study described in this paper shows that the two-wedge failure surface method is the most suitable for the internal stability design of brick faced retaining walls reinforced with geosynthetics constructed on a rigid foundation ground.

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