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Numerical analysis of reinforced embankments on soft soils

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ABSTRACT: This paper presents a numerical study on the performance of reinforced piled embankments and reinforced retaining walls on compressible foundation soils. Parameters such as pile spacing, foundation stiffness, wall height and number and stiffness of reinforcement layers were varied. The results obtained allowed the identification of relevant parameters and the importance of a proper modelling of the reinforcement. The behaviour of case-histories in the literature support the findings of the numerical analyses

1 INTRODUCTION

Geosynthetic products can be effectively used to reinforce embankments on soft soils. The presence of the reinforcement increases the factor of safety of the embankment, allows a faster rate of construction, steeper embankment slopes and minimises fill consumption and construction time. In most of the cases it is required that the geosynthetic reinforcement presents large tensile strength and tensile stiffness to fulfil its role as an effective reinforcing element.

Geosynthetic layers can also be combined to piles with caps at the base of the embankment. In this case the piles transfer the fill loads to a deeper and stronger soil layer beneath the soft deposit, reducing embankment settlements. This type of solution is of interest not only for problems of embankments on soft clay deposits, but also for embankments on collapsible soils. This is the case in some regions in the city of Brasilia, Brazil, where vertical deformations due to soil collapse can reach as much as 15%. The presence of the reinforcement layers can provide a better distribution of vertical load to the piles and to reduce the vertical stresses transmitted to the soft soil, if an effective arching mechanism can be mobilised in the fill material.

The design of piled embankments with reinforcement is still a complex task, where many variables involved are still difficult to quantify. This is the case of the required reinforcement strength and tensile stiffness, number of reinforcement layers and spacing between reinforcement layers. Some limit equilibrium methods are available in the literature for the design of piled embankments with and without reinforcement (Terzaghi 1943, John 1987, Hewlett and Randolph 1988, BSI 1995 and Russel and Pierpoint 1997). However, the complex nature of the interaction between materials in this type of problem limits their application. This favours the utilisation of more powerful numerical tools, such as finite difference or finite element methods to predict embankment behaviour. Nevertheless, even the latter methods have their own limitations to capture the actual mechanism of soil reinforcement in this type of situation.

This paper presents some numerical studies of geosynthetic reinforced embankments built on compressible foundations. The presence of piles at the base of the embankment and how numerical simulation relates to the observation of the performance of real structures is also approached in the present paper.

2 ANALYSIS OF PILED EMBANKMENTS WITH GEOSYNTHETIC LAYERS

2.1 *Characteristics of the cases analysed*

Figure 1 shows a typical cross section of a piled embankment with reinforcement layers. The aim of the study at this stage is to assess the behaviour of the region around a pile in the central part of the embankment. The computer code FLAC (Itasca 1995) was used in the numerical analyses. The characteristics of the materials used in the simulations are summarised in Table 1. A linear elastic analysis was used at this stage under plane strain conditions. The piles (0.3 m diameter) were modelled as equivalent walls to satisfy plane strain conditions. Interface elements were employed at the pile-soft soil and fill-reinforcement interfaces. The adhesion between pile and soft soil was assumed as equal to 10 kPa (zero interface friction angle). The construction of the embankment was staged, similar to the conditions found in real works. It was assumed a value of

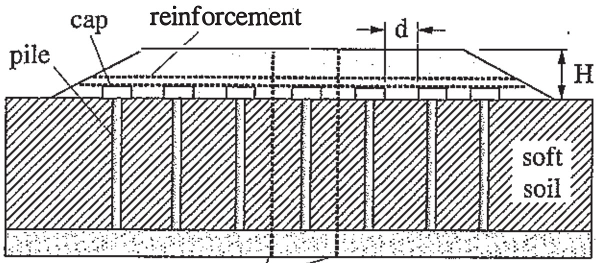


Figure 1. Geometrical characteristics of the problem.

Table 1. Material properties used.

Property	Fill	Soft Soil	Pile
Young modulus (MPa)	40.0	2.0	1.4 x 10^4
Poisson coefficient	0.3	0.3	
Unit weight (kN/m^3)	19.6	14.7	24
Thickness (m)	6.0	6.0	0.3$^{(*)}$

Note: (*) Pile diameter.

Young modulus for the layer of soil below the soft soil equal to 60 MPa and Poisson ratio of 0.3.

The caps, when present, were assumed rigid with dimensions 1 x 1 x 0.5 m (width x length x height).

The spacing between piles was varied to investigate the influence of this factor on the general behaviour of the system. The following values of pile spacing were analysed: 1.8, 2.4, 3.0 and 5.2 m. The spacing values of 1.8 and 2.4 m would yield pile efficiencies of 90 and 75% by the design methodology proposed by Hewlett and Randolph (1988).

The number and tensile stiffness of the reinforcement layers were varied as part of the investigation. Problems with up to 3 reinforcement layers above the piles were analysed. The spacing between reinforcement layers was kept constant at 0.25m. The values of reinforcement tensile stiffness (J) used were 300, 1000, 4000 and 40000 kN/m. The reinforcement layers can be simulated by FLAC as cable elements or beam elements. Both types of elements were used to assess the influence of the type of element on the results obtained.

Additional information of the analyses carried out can be found in Sa (2000).

2.2 *Results obtained*

Figure 2 shows the variation of average vertical displacement of the foundation surface between pile caps as a function of the distance between cap faces (d, in Figure 1). A 15 to 21% reduction of settlements was can be observed for the case of the embankment with piles and caps in comparison to the embankment on the soft soil alone up to distances between caps of 1.5m. The addition of one layer of reinforcement (J = 300 kN/m) to the system with

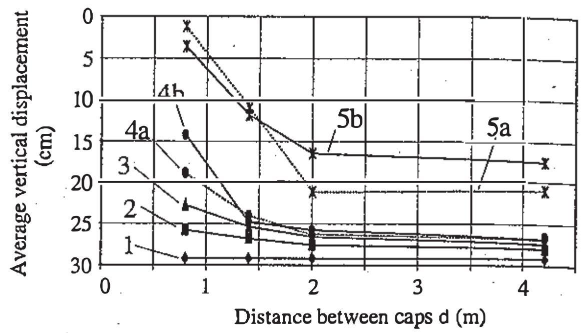


Figure 2. Average vertical displacement on the foundation surface between pile caps.

piles and caps reduced the vertical displacement even further (up to 20 to 50%) for lower values of d. As d increases the benefit of the presence of piles and one layer of reinforcement is reduced. For the case of 3 layers of a stiffer reinforcement (J = 4000 kN/m) a significant reduction of the vertical displacement of the foundation surface can be noted, even for larger values of d. Figure 2 also shows that the differences coming from modelling the reinforcement by different types of elements were smaller than 20%.

The influence of the number and stiffness of the reinforcement layers on the vertical displacement of the foundation soil surface can be visualised in Figure 3, as a function of the distance between pile caps. It can be observed that the greater the stiffness or the number of reinforcement layers the smaller the vertical displacement for values of d below 2 m (H/3). For larger values of d the stiffness of the reinforcement is less important to the reduction of vertical displacements than the number of reinforcement layers.

The reinforcement tensile stiffness and number of reinforcement layers have a marked effect on the average vertical displacement at the embankment surface between pile caps, as shown in Figure 4, for d = 2m. It should be noted that the number of reinforcement layers influences more the vertical displacements than the reinforcement stiffness for values of J above 1000 kN/m.

Figure 5 shows the influence of reinforcement stiffness and number of reinforcement layers on the average vertical stress on the surface of the foundation soil. Reductions of vertical stresses from 15 up to 83%, with respect to the situation of the foundation soil alone, can be observed for d < 2 m (d/H<0.33), depending on the number of reinforce-

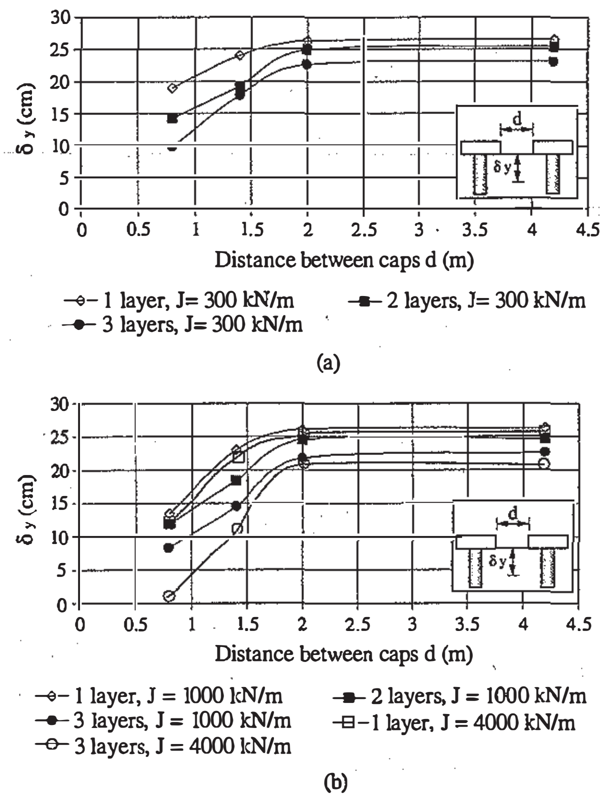


Figure 3. Influence of reinforcement stiffness and number of reinforcements on foundation surface average settlements.

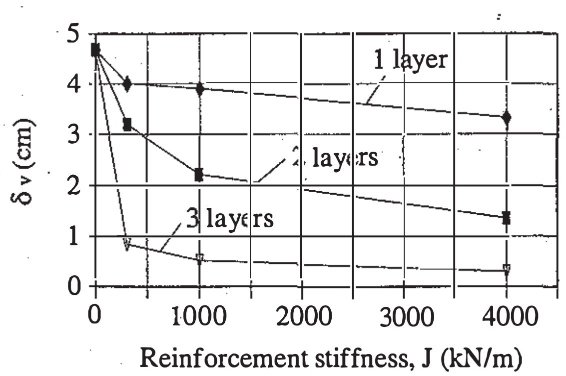


Figure 4. Average vertical displacement at the fill surface for different numbers of reinforcement layers (d = 2m).

ment layers and reinforcement stiffness. For d = 4.2 m (d/H = 0.7) and piles with caps with three layers of a 4000 kN/m stiff reinforcement the reduction of vertical stresses is of the order of 33% in comparison to the case of the foundation soil alone.

The influence of the ratio between pile modulus and foundation soil modulus on the average vertical stress on the foundation soil surface is presented in Figure 6 for d = 2.0m. The presence of piles and caps only and piles, caps and a layer of reinforcement, with J = 1000 kN/m, are considered in that

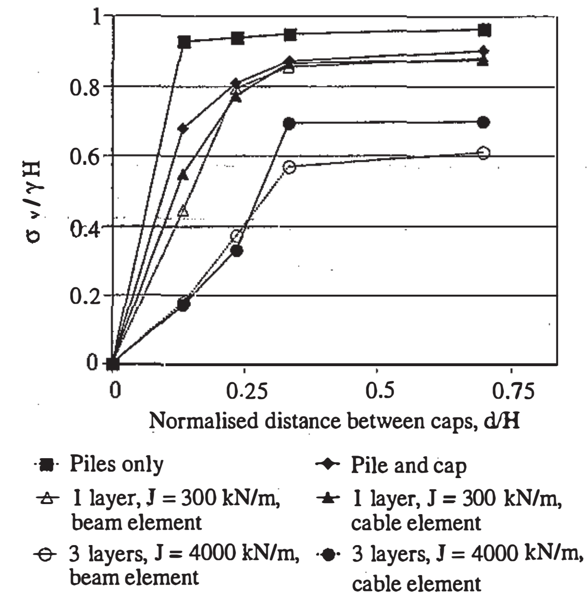


Figure 5. Average vertical stress on the foundation soil surface.

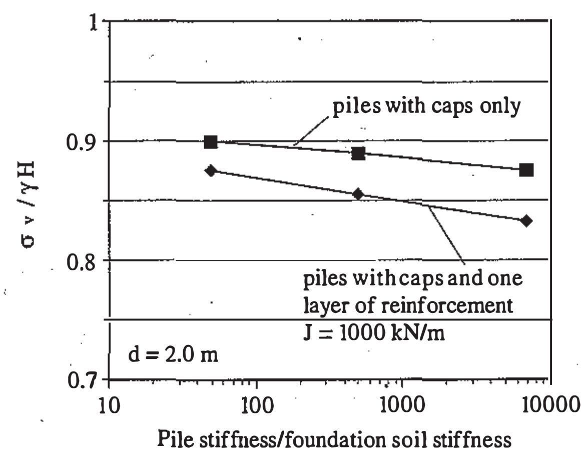


Figure 6. Influence of the relative stiffness of the piles.

figure. It can be observed that the increase of the relative stiffness of the pile above a value of 100 had a marginal effect (less than 5%) on the reduction ofvertical stresses transferred to the foundation surface. The effect of pile stiffness increase was slightly more relevant when the reinforcement layer was present.

Figure 7 presents the variation of the maximum mobilised reinforcement force versus reinforcement stiffness for the case of an embankment with piles, caps and three layers of reinforcement, for d = 2m. The results show that the distribution of tensile force between layers is not uniform, with the reinforcement layer at the bottom being 3 times more loaded than the top reinforcement layer.

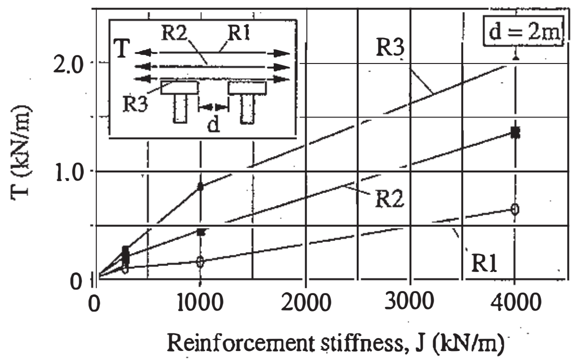


Figure 7. Forces mobilised in the reinforcement layers.

3 REINFORCED WALLS ON COMPRESSIBLE FOUNDATIONS

The behaviour of reinforced soil structures such as the one presented in Figure 8 was also investigated in the present work. The computer code FLAC was also employed in the numerical simulations. It is important to point out that the reinforced walls were designed using the program ReSlope (Leshchinsky 1995). The aim was to have a preliminary deformation analysis of a reinforced wall designed using current limit equilibrium methods and the influence of the foundation soil stiffness on the general behaviour of the structure.

The general characteristics of the walls studied are presented in Table 2. Three wall heights were investigated and the reinforcement layout chosen was the one given by the program Reslope for the condition of uniform reinforcement spacing and length (stronger and stiffer reinforced mass).

The backfill material was assumed as having a Young modulus of 20 MPa, Poisson coefficient of 0.3 and unit weight of 18 kN/m^3. The properties of the foundation soil in each case are presented in Table 3. Three foundation soils with Young modulus values ranging from 10 to 120 MPa were analysed, yielding to different levels of foundation stiffness. Both elastic and elastic-plastic analyses were carried out. Table 4 shows the parameters adopted in the elastic-plastic analyses conducted. The construction of the wall was staged, simulating real construction conditions.

The reinforcement tensile stiffness (J) values used in the analyses were 1200, 1800 and 3600 kN/m. Interface elements were used between soil and reinforcement layers. The friction angle at these interfaces was adopted as 31°. Different values of the interface stiffness (normal, K_n, and shear, K_s) were used. For a rigid interface both the normal and the shear stiffness of the interface were equal to 99000 MPa/m. Based on direct shear tests (Tupa 1994), more realistic values of interface stiffness were used

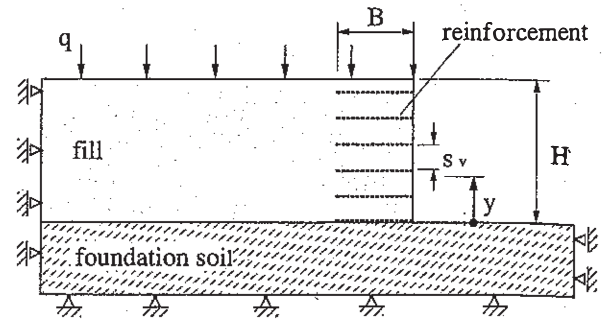


Figure 8. Reinforced wall on compressible foundation.

Table 2. Characteristics of the walls analysed

Wall	H (m)	s_v (m)	n	B (m)
1	3,0	0,3	10	1,7
2	6,0	0,3	20	3,4
3	12,0	0,3	40	6,9

Notes: (1) H = wall height, s_v = reinforcement spacing, n = number of reinforcement layers, B = reinforcement length; (2) See also Figure 8.

Table 3. Properties of the foundation soil.

Foundation type	Young Modulus (MPa)	Poisson ratio	Unit weight (kN/m^3)
A	120	0.3	22
B	60	0.3	19
C	10	0.3	17

Table 4. Parameters used for the elastic-plastic analysis.

Soil	c (KPa)	S_u (KPa)	ϕ (deg.)	E (MPa)
Fill	5	-	35°	20
Foundation A	-	1200	0	120
Foundation B	-	600	0	60

Notes: c = soil cohesion, S_u = soil undrainded strength, ϕ = soil friction angle and E = soil Young modulus.

in some of the cases. In these cases the values of K_n and K_s adopted were 9000 and 30 MPa/m, respectively.

Additional information on these numerical analyses can be found in Dellabianca (1999).

3.1 *Results obtained*

Figure 9 shows the results of horizontal wall face displacements (normalised by the wall height) for different values of foundation stiffness. It can be observed that the softer the foundation soil the larger the wall horizontal displacement, particularly close to the wall toe. This behaviour has been also identified in model tests (Palmeira and Monte 1997) and in real structures, as will be discussed later in this work.

The distribution of vertical stresses along the base of the reinforced wall 2 on foundation soil B can be seen in Figure 10. The distribution of vertical stresses is rather uniform along most of the length of the wall base. For regions close to the wall face it can be observed the influence of the presence and value of the interface elements stiffness on the results obtained.

Figure 11 presents the distribution of tensile loads along the reinforcement length at the base of wall 2 (K_n = 9000 MPa/m and K_s = 30 MPa/m) for different values of foundation stiffness. It can be noted that the foundation stiffness can have a major influence on the tensile loads at the base of the structure. For the softer foundation condition the reinforcement was subjected to compression, rather than tension, close to the wall face. This can be attributed to the pattern of horizontal displacements in the foundation soil. A similar behaviour was also observed when the number of reinforcement layers was varied in wall 2 (with varying safety factors and/or conditions assumed in the design of this wall with program ReSlope). For reinforcement layers at mid-height and at the top of the wall the values of foundation stiffness used in this work had little effect on the distribution of tensile forces and maximum tensile force in the reinforcement.

For the conditions of the walls there was little difference between predictions using a linear elastic and an elastic-plastic model, as can be seen by the results of normalised wall face horizontal displacements in Figure 12, except for the regions close to the ground surface and wall base.

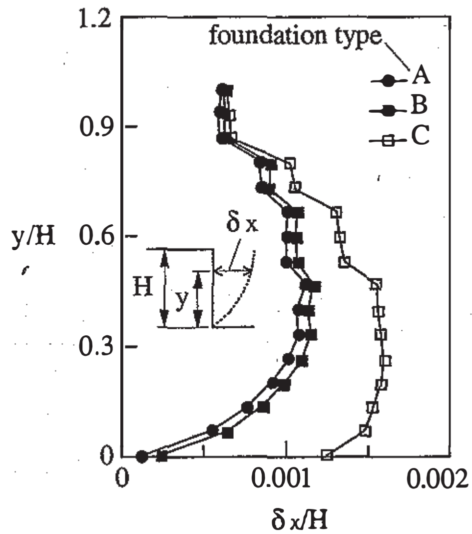


Figure 9. Horizontal displacements of the wall face – Wall 2.

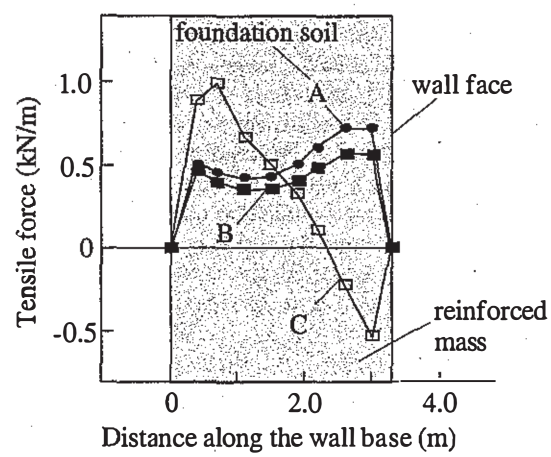


Figure 11. Distribution of tensile forces in the reinforcement at the wall 2 base.

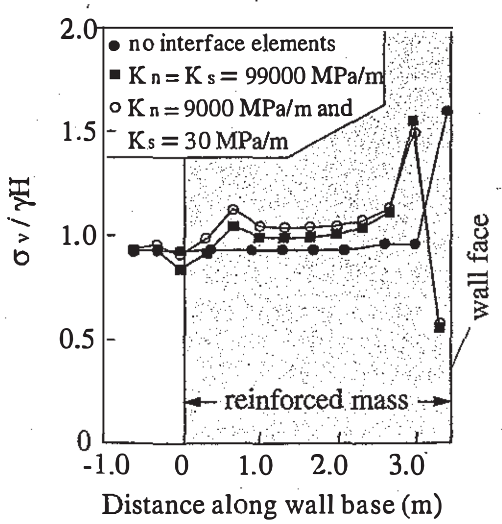


Figure 10. Vertical stresses on the wall base (Foundation B).

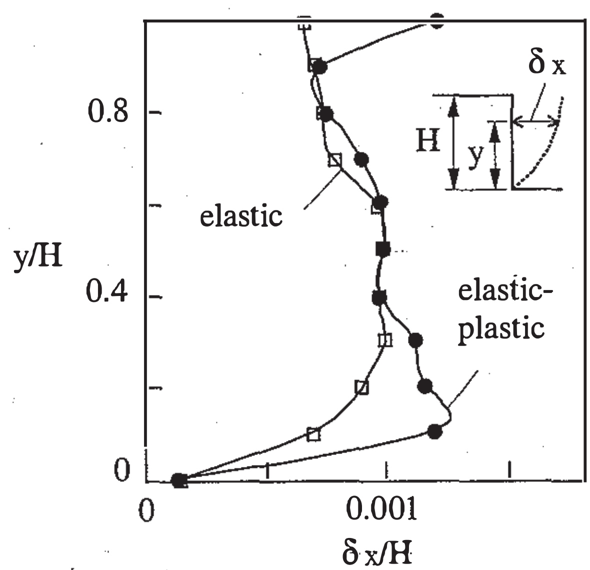


Figure 12. Elastic versus elastic-plastic analysis – Wall 1 and foundation soil B.

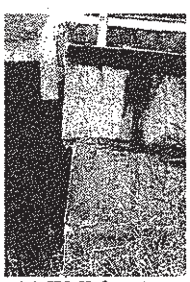
4 CASE-HISTORIES BEHAVIOUR

To illustrate the link between numerical simulations and real structure performance, some case-histories are presented and their general behaviour evaluated.

The reinforced structures to be discussed below are piled geotextile reinforced abutments built on soft soils as part of the crossings of the rivers Itariri, Subauma and Sauipe in the BR-101 highway, along the Brazilian north-east coastline. Soft organic clay deposits are common in the region, with undrained strengths varying typically between 10 to 60 kPa. Detailed information on these case-histories can be found in Fahel (1998) and in Palmeira and Fahel (2000 and 2001).

The Itariri, Subauma and Sauipe abutments heights are 3.8, 1.75 and 2.0m, respectively. The former two abutments were built on piles with caps. The pile diameter was 0.25m and the piles crossed the entire soft soil thickness. The pile caps measured 1 x 1 x 0.3 m and the pile spacing was 1.5m (d = 0.5 m). The Sauipe reinforced abutment was built directly on a 4.5m thick layer of sand overlying the a 5.7m thick soft clay foundation. Rather extensible woven (Subauma wall) and non woven geotextiles (Itariri and Sauipe walls) were used.

For the pile spacing and cap geometry used, the settlements of the piled abutments were negligible, which is consistent with the results of the numerical analyses presented earlier. Relative displacements between wall panels and rotation of the wall face were observed for the Sauipe abutment (built without piles) as shown in Figure 13 (a), as well as a significant vertical settlement (29 cm) close to the bridge (Fig. 13 b). The flexibility of the reinforced soil mass was a major component for the reduction of damages to the abutment due to settlements. After rather minor repairs, these embankments have been performing well since.



(a) Wall face

(b) Bridge-embankment contact

Figure 13. Damages to a reinforced abutment caused by foundation settlements (Fahel 1998).

5 CONCLUSIONS

This paper presented a numerical analysis study of reinforced embankments and walls on compressible foundation soils. The results obtained showed the benefits of the combination of reinforcement layers and piles beneath the embankment.

The analysis also compared different ways to model the reinforcement layer and some differences of results can be observed, although of limited relevance. The use of interface elements is very important for an accurate modelling of the reinforcement.

Horizontal wall face displacements and reinforcement forces are influenced by the foundation stiffness. For the conditions investigated in this work the use of a conventional limit equilibrium design approach yielded rather stiff walls when uniform reinforcement lengths and spacing were employed, even for the softer foundation soil.

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