

European analytical calculations compared with a full-scale Brazilian piled embankment

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ABSTRACT: Measurements have been carried out in a full-scale Brazilian basal reinforced piled embankment. The subsoil beneath the geosynthetic reinforcement (GR) had been excavated before the installation of the reinforced embankment. The embankment was relatively thin in comparison to the pile cap spacing, resulting in relative much load on the GR. GR strains and settlements have been measured, making it possible to validate analytical models.

Many analytical design models for the design of piled embankments distinguish two calculation steps; calculation step 1; the arching and calculation step 2; the load deflection behaviour of the GR. This paper presents the comparison of the full-scale test with a new step 1-model; the Concentric Arches Model (Van Eekelen et al. 2013) and several step 2 models. The new Concentric Arches model is an extension on the models of Hewlett and Randolph (1988) and Zaeske (2001) and takes into account the 3D nature of the arching. The new model was developed because experiments had showed a load distribution on the GR in a piled embankment that could not be explained with the existing models. The new model does explain the measured load distribution.

Keywords: arching, piled embankments, load transfer platforms, geosynthetic reinforcement, full-scale tests

1 DESIGN GEOSYNTHETIC REINFORCEMENT FOR PILED EMBANKMENTS

Many analytical design models for the design of piled embankments distinguish two calculation steps; calculation step 1 and step 2, explained in Figure 1. The next sections explain these steps and summarize some analytical models describing these steps. After that, these models will be compared with measurements in a full-scale Brazilian basal reinforced piled embankment.

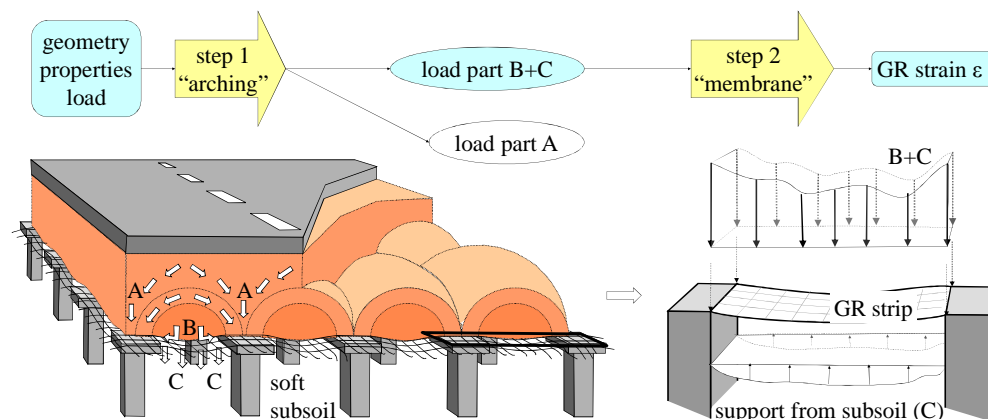


Figure 1. The GR design is done in two steps: (step 1) arching behaviour determining the distribution of the load into A, B and C and (step 2) the load deflection behaviour that calculates the GR strain from load part B+C.

1.1 Step 1: arching

Step 1 is the arching behaviour in the fill. This “arching step” divides the total vertical load into two parts: load part *A* and the ‘residual load’ (*B+C* in Figure 1). Load part *A*, also called the ‘arching’, is the part of the load that is transferred to the piles directly. The two most frequently used arching models in Europa are the models of Hewlett and Randolph (1988, see Figure 2*a*) and Zaeske (2001, also cited in Kempfert et al., 1994, see Figure 2*b*). An extension of this model is the Concentric Arches model (Figure 2*c*), presented recently by Van Eekelen et al. (2013). This model was specifically developed to meet the observation that a major part of the load on the GR concentrates on the GR strips between adjacent piles, and that the load distribution on these strips approaches the inversed triangular shape, as shown in Figure 3*c*. This is further discussed in the next section.

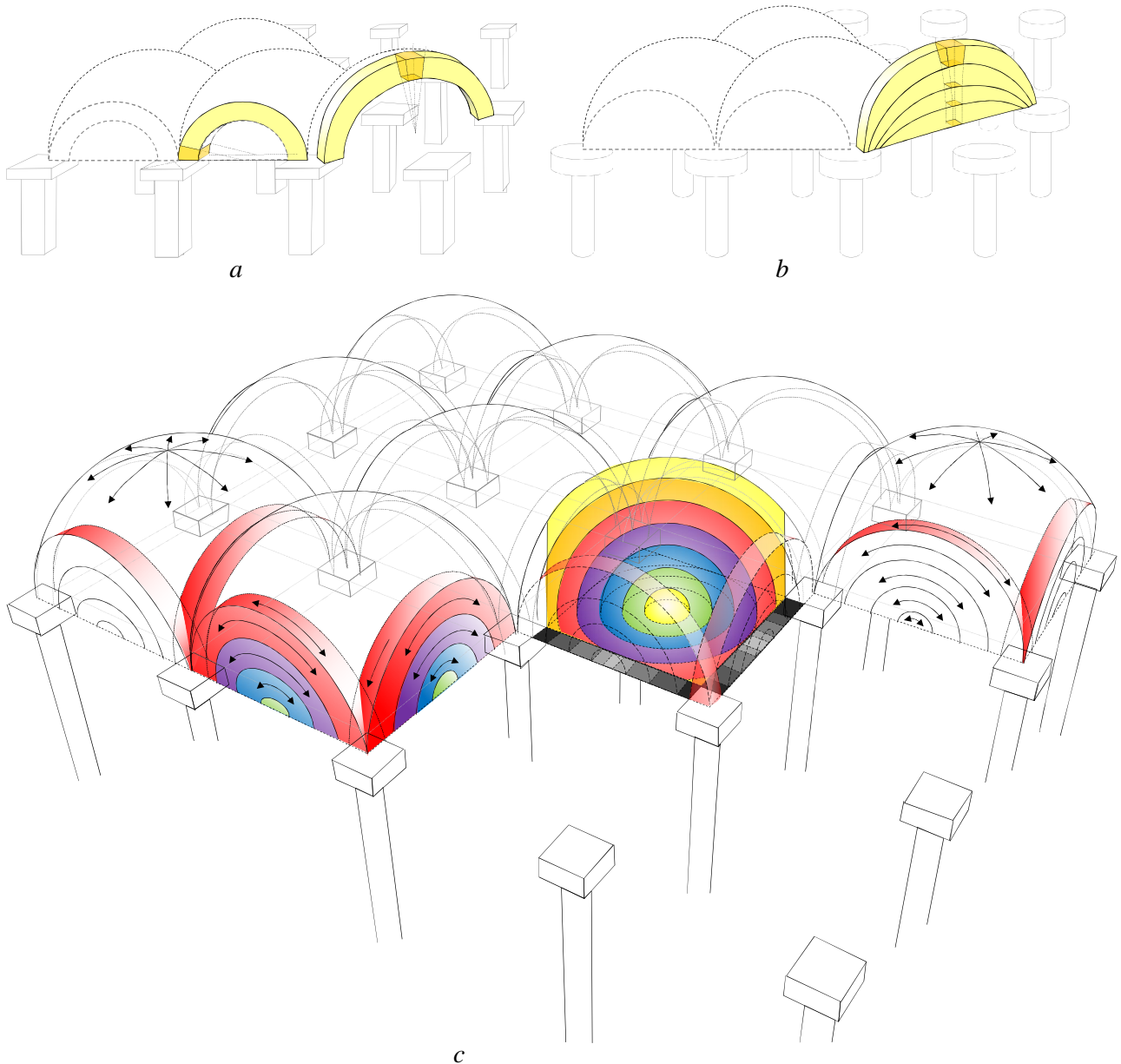


Figure 2. Models for step 1 (“arching”, see Figure 1); (a) Hewlett and Randolph (1988), (b) Zaeske (2001, also described in Kempfert et al, 2004, adopted in EBGeo, 2010 and CUR226, 2010) and (c) the Concentric Arches model (Van Eekelen et al., 2013).

1.2 Step 2: load deflection behaviour

Calculation step 2 describes the load-deflection behaviour of the geosynthetic reinforcement (see Figure 1 and 3). In this calculation step, the ‘residual load’ that results from step 1 is applied to the GR strip between each two adjacent piles. And possibly the GR strip is supported by subsoil. From this, the GR strain is calculated. An implicit result of step 2 is that the ‘rest load’ is divided into a load part B, which goes through the GR to the piles, and a part C, resting on the subsoil, as indicated in Figure 1.

Zaeske (2001), between several other researchers, showed the great influence of the application of a sufficient stiff GR in a piled embankment. The concentration of load on strips between the piles is only found for GR basal reinforced piled embankments, not for piled embankments without GR. Therefore, it is necessary to make a distinction between arching models for piled embankment with and without GR. This paper focuses on GR reinforced piled embankments only.

Two issues are of major importance in step 2: Firstly, the load distribution on the GR strip has a strong influence on the resulting calculated GR strain. See Figure 3. Van Eekelen et al. (2012a and b) concluded that this distribution approaches the inverse triangular distribution more than for example the triangular distribution (EBGEO (2010), CUR (2010), Zaeske (2001)) or the uniform distribution (BS8006, 2010).

Secondly, the subsoil support has a major influence. BS8006 never accepts calculating with subsoil support. EBGEO (2010) and CUR (2010) accept subsoil support. However, the most extreme situations occur when the subsoil support gets lost. This can for example occur due to dewatering of the subsoil or settlement of the soft subsoil due to the weight of a working platform below the GR. The case study presented by Almeida et al. (2007) is interesting, because the influence of the subsoil has been eliminated by excavating the subsoil. This rest of this paper only considers the situation without subsoil support.

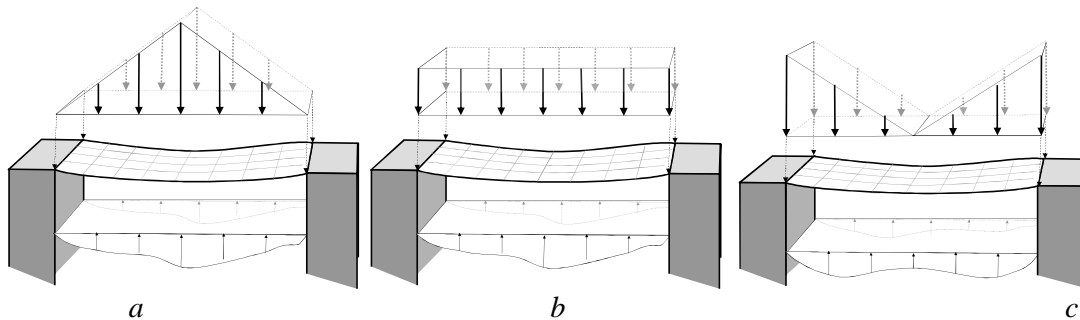


Figure 3. Calculation step 2 with three different load distributions: (a) triangular (Zaeske, 2001, EBGEO, 2010, CUR, 2010) (b) uniform (BS8006, 2010) (c) inverse triangular (Van Eekelen et al., 2012a and b)

1.3 Validation of arching and load distribution on the GR

It is suggested using the Concentric Arches model and an inverse triangular load distribution. Van Eekelen et al. (2013) validated the Concentric Arches model with laboratory experiments, numerical calculations of Le Hello et al. (2009) and full scale tests (Van Eekelen, 2012, Van Duijnen, 2010). Van Eekelen et al. (2012b) validated the inverse triangular load distribution with laboratory experiments. The present paper gives additional validations.

2 FULL SCALE TEST

Almeida et al. (2007, 2008) and Spotti (2006) presented a full scale tests carried out in the Barra da Tijuca District of Rio de Janeiro. Part of the test area is presented in Figure 4. The pile caps are square with sides 0.80 m and placed on piles that were driven in a square array with a centre-to-centre spacing $s_x = s_y = 2.50$ m.

The embankment is relatively thin: at the test location average $H=1.25$ m, so that $H/(\sqrt{2} \cdot (s_x - a)) = 0.52$. This is lower than 0.80 and thus would not be accepted in Germany and also lower than 0.66, the minimum value in the Netherlands for the usability phase. For research purposes however, this height is very interesting as this low embankment height gives a relatively high load on the GR. During construction phases this height does occur so that analytical models need to be able to describe this situation as well.

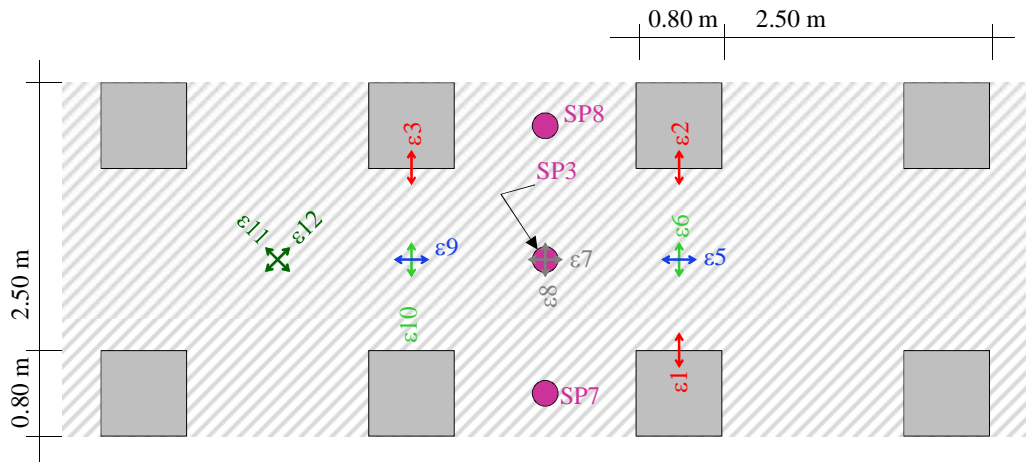


Figure 4. Layout of the considered part of the experimental site in the Barra da Tijuca District (Spotti, 2006, Almeida et al., 2007, McGuire et al., 2009). The dashed area has been excavated prior to the installation of the reinforced embankment.

A 1 m deep excavation was made prior to the GR installation. The excavated area is dashed in Figure 4 and also indicated in Figure 5. A geogrid was kept taut by loading the edges with fill. Photos taken at the site show that the geogrid was indeed relatively taut but also show that some parts of the reinforcement did sag a little prior to placement of the fill. Compaction of the lower fill layers was carried out with care, using light equipment.

A single layer of Fortrac 200/200-30 polyester (PET) biaxial knitted geogrid was placed over the pile caps with an underlying layer of non-knitted geotextile to reduce abrasion between pile cap and geogrid.

The fill material consisted of a clayey sand compacted to at least 95% of the standard Proctor maximum. It has a compacted unit weight $\gamma = 18.0 \text{ kN/m}^3$.

The GR deflection was measured with settlement plates (SP) installed at the GR level at the locations indicated in Figure 4. GR strain gauges (ε) consisted of an ingenious system that measured the strain with strain gauges on a steel bar that has been attached with a reaction spring to the GR. The system has been described by Almeida et al. (2007) and gave reliable-looking results. Their locations are indicated in Figure 4.

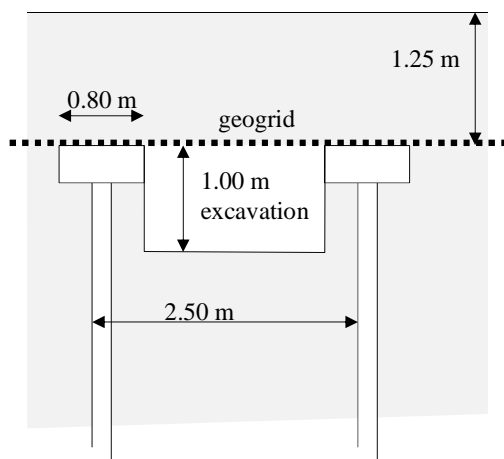


Figure 5. Cross section of the full scale test

3 COMPARISON MEASUREMENTS AND ANALYTICAL CALCUALTIONS

3.1 Determination of parameters

The stiffness J (kN/m) of the geogrid depends on the tensile force and the loading time. The average of the larger values of the measured GR strains is 1.5% (see Figure 7). The total loading time is 188 days. The $J_{1.5\%, 1 \text{ month}} = 1637 \text{ kN/m}$ and the $J_{1.5\%, 1 \text{ year}} = 1594 \text{ kN/m}$. We assume that the average of these values is $J_{1.5\%, 188 \text{ days}} = 1615 \text{ kN/m}$. This value will be used in the calculations.

Shear tests on fill samples showed an average friction angle $\varphi = 42^\circ$ and an average cohesion $c = 18.9$ kPa. However, the considered analytical calculation models have been developed for cohesion-less embankment fill. Therefore, it was necessary to find an equivalent friction angle φ for $c = 0$. McGuire et al. (2009) have found (after some back-calculations from measurements in nearby 2D test fields): $\varphi = 68^\circ$ for $c = 0$ kPa. This seems to give satisfying results with their model, although their value for φ seems unreasonably high.

To prevent wrong interpretation because of a wrong choice for the friction angle, a wide range of values for φ was used in the calculation of the present paper. Table 1 summarizes the parameters that have been used in the calculations.

Table 1. Parameters used in the calculations

Unit	Friction angle fill	Cohesion fill	Subgrade reaction	Centre-to-centre distance piles	Width square pile caps	Height fill	Stiffness GR along and across	Surcharge load
γ	φ	c	k	$s_x = s_y$	a	H	J	p
kN/m ³	o	kPa	kN/m ³	m	m	m	kN/m	kPa
18.0	23-68	0	0	2.50	0.80	2.50	1615	0

3.2 GR strains

Most design models assume that the highest GR strain occurs within the GR strips with a maximum at the edges of the pile caps. This is indeed confirmed by the measured GR strains presented in Table 2 and Figure 6: the greatest strains occur at the pile cap edges, the second largest GR strains occur at and along the GR strips. This has also been found by Zaeske (2001) and Van Eekelen et al. (2012a).

The strains on the GR strip, but in the cross direction ($\epsilon 5$ and $\epsilon 9$) are much smaller. The same is found for the strains in the diagonal direction in the centre of four piles ($\epsilon 11$ and $\epsilon 12$). The GR strains in the centre of four piles, in the directions parallel to the pile array have also been measured. One of them ($\epsilon 7$) is surprisingly large, but still smaller than the GR strains measured in and along the GR strips. The other one ($\epsilon 8$) is small, according to the expectations and findings of Zaeske (2001) and Van Eekelen (2012a).

Table 2. Measured GR strains

Edge of pile caps			On and parallel to GR strips		On GR strips, cross direction		Centre of 4 piles, parallel to pile array		Centre of 4 piles, diagonal direction	
$\epsilon 1$	$\epsilon 2$	$\epsilon 3$	$\epsilon 6$	$\epsilon 10$	$\epsilon 5$	$\epsilon 9$	$\epsilon 7$	$\epsilon 8$	$\epsilon 11$	$\epsilon 12$
2.05%	1.73%	1.50%	1.50%	1.36%	0.51%	0.32%	1.14%	0.97%	0.25%	0.63%

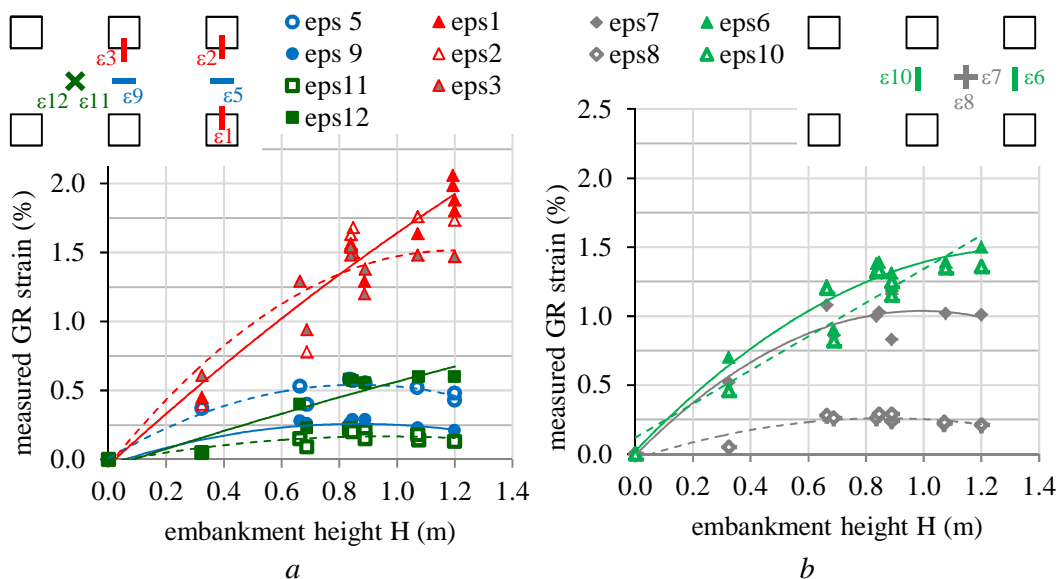


Figure 6. Measured strains confirming the assumption of most calculation models that most strains occur in and parallel to the GR strips with a maximum at the edges of the pile caps

Figure 7 compares the calculated and measured strains in and along the GR strips. As explained before, the fill has cohesion while the calculation models have been developed for cohesion-less material. The cohesion can be compensated by increasing the friction angle ϕ . As it is not clear how much this increase should be, a large range of values for ϕ have been given on the horizontal axis.

Figure 7a shows the maximum strains, at the edge of the pile caps. Figure 7b shows the minimum GR strains at the GR strip, thus in the centre of the GR strips. Both the measured and the calculated strains are indeed larger at the pile caps than in the centre of the GR strips. It is expected that an increasing ϕ gives a decrease of GR strain. All three step 1 models, Hewlett and Randolph, Zaeske and the Concentric Arches model meet this expectation.

Hewlett and Randolph (1988) already recommended not using their model for low embankments. Their model predicts a much too high GR strain indeed. For safe but economic designs, we need analytical models that approach the measurements closely and from the safe side. That means that the calculated GR strains should be the same or larger than the measured strains. This agreement is best for the Concentric Arches model in combination with an inverse triangular load distribution.

In common design, it is recommended to neglect the cohesion. Figure 7 also shows that consequences of doing so, and thus calculating with $\phi=42^\circ$. The Concentric Arches + Inverse triangular model would give a prediction that is not far from the measured value, but on the safe side. The other models give more, and sometimes far more GR strain, leading to a more conservative design.

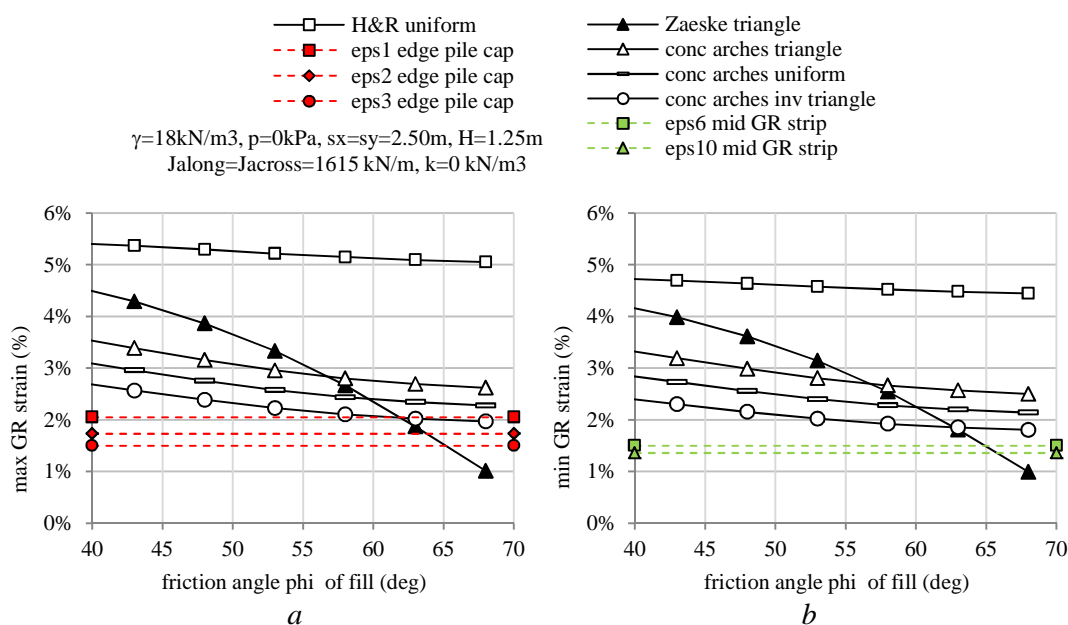


Figure 7. Comparison measured and calculated GR strains (a) maximum GR strains at the pile cap edge and (b) minimum GR strains in the centre of GR strips. Calculations with three step 1 models (Figure 2); Zaeske (2001), Hewlett and Randolph (1988) and the Concentric Arches model (Van Eekelen et al. 2013) and three step 2 - load distributions (Figure 3); triangular, uniform and inverse triangular.

3.3 GR deflections

The GR deflection has been measured with settlement plates but can also be determined from the relationship between vertical deflection and GR strain. This relationship can be determined if we assume a shape of the deformed GR. For example a parabolic shape ($z = ax^2$), a 3rd order polynomial ($z = ax^3$) or a 4th order polynomial ($z = ax^4$). Each of these GR shapes corresponds with a load distribution, as shown in Table 3 and explained in Van Eekelen et al. (2012b). Van Eekelen et al. (2012a) showed that the GR shape approaches at least the 3rd order polynomial. Table 3 gives settlements calculated from measured strains.

Table 3. Settlement calculated from measured GR strain assuming a shape of the deformed GR

shape deformed GR	load distribution	Settlement calculated from measured GR strain (m)		
		$\epsilon_1 = 2.05\%$	$\epsilon_2 = 1.70\%$	$\epsilon_3 = 1.50\%$
parabola: $z = ax^2$	uniform	0.143	0.129	0.121
3 rd order polynomial: $z = ax^3$	inverse triangular	0.132	0.118	0.111
4 th order polynomial: $z = ax^4$	approximately parabolic	0.118	0.101	0.094

Figure 8 compares the measured and calculated GR deflection of the GR strips. The settlements calculated from the measured GR strains have been calculated using the 3rd order polynomial. Comparison between the prediction with the Concentric Arches model + inverse triangular and the measurement of ϵ_1 show that the shape of the deformed GR must be in between the 3rd and the 4th order polynomial. This means that the load distribution approaches the inverse triangular distribution more than the uniform distribution.

The deflection measured with the settlement plates is significantly larger than the value calculated from the GR strains. This suggests that the GR had initially some slag at this location. McGuire et al. (2009) and Bezuijen et al. (2010) explain this difference between measured settlement and measured strain as a result of some initial slag (too much GR length), resulting in some initial sagging of the GR and less GR strain per deflection increment. Slag has not been included in the analytical predictions of the settlement.

Vertical equilibrium requires that the vertical component of the tension developed within the reinforcement counteract the vertical pressure produced by the overlying embankment. If the geogrid sags a bit prior to fill placement, the vertical component of the developed tension is larger. This means that less tension, and therefore less strain, is necessary to satisfy equilibrium than for reinforcement without initial slag.

The concentric arches + inverse triangle in Figure 8 show a reasonable agreement with the settlements calculated from the strain gauges. Better than any of the other models.

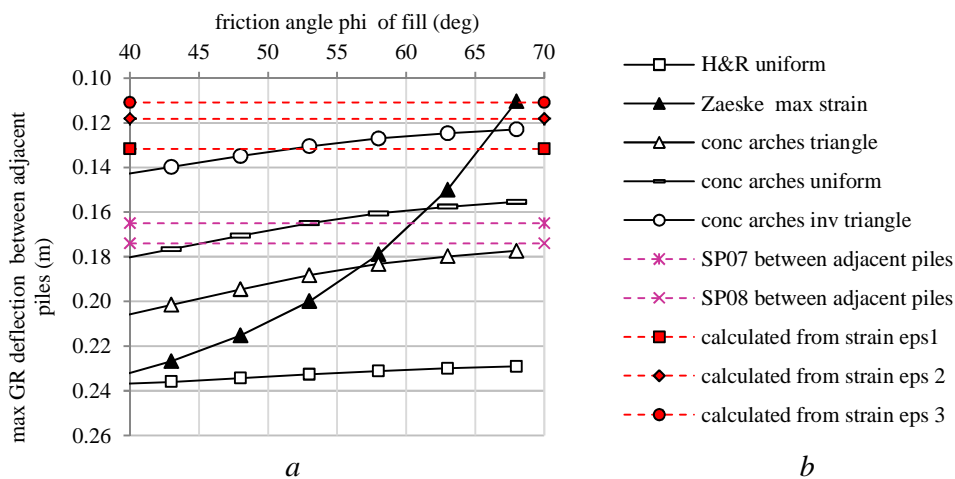


Figure 8. Comparison measured and calculated GR deflection. Calculations with Zaeske (2001), the Concentric Arches model (Van Eekelen et al. 2013) in combination with three load distributions; triangular, uniform and inverse triangular. The measured settlements are larger than those calculated from the measured strains. This is probably because there has been some initial slag, as explained in the text. Slag has not been included in the analytical calculations.

4 CONCLUSIONS

A full scale test has been carried out in the Barra da Tijuca District of Rio de Janeiro in Brazil. The project gives valuable results as the subsoil has been excavated away prior to the embankment installation and the embankment has a relatively low height in comparison to the pile cap spacing. The fill consisted of clayey sand.

The GR strains have been measured successfully at several locations. The measured GR strains are the largest in and along the GR strips. The largest strains within the GR strips have been found at the edges of the pile caps. This strain concentration in the GR strips and the maximum at the edges of the pile caps is in accordance with the considered analytical design models.

This paper considers different analytical design models for the design of piled embankments. They distinguish two calculation steps; calculation step 1 (arching) and step 2 (load deflection behaviour). Three step 1 models have been considered. The first two are the models of Hewlett and Randolph (1988) and Zaeske (2001). Both models are frequently applied in Europe. The third model is the Concentric Arches model (Van Eekelen et al., 2013) which is an extension of the first two. Also three load distribution models for step 2 have been considered: a triangular, uniform and inverse triangular load distribution.

The results of these analytical models have been compared with the measured GR strains. As the models have been developed for cohesion-less fill, it was necessary to adapt the value for the friction angle φ of the fill. To prevent doing a wrong choice, a wide range of values for φ has been considered.

The measured GR strains agree best with the Concentric Arches model for calculation step 1 in combination with the inverse triangular load distribution for step 2.

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