Influence factors on the performance of geosynthetic reinforced and pile supported embankments

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ABSTRACT: Geosynthetic reinforced and pile-supported embankments (GPE) are often used to transfer traffic loads into a bearing layer through a soft underground. In the last decade this construction method has well proved itself in the practice. Nevertheless, there are several factors that can influence load transfer mechanism of the GPE-system which are not investigated in detail so far. These are, for example, the influence of pile arrangement pattern, the lateral spreading problematic in the slope zone, influence of cyclic loading, effect of product structure and the number of reinforcement layers. In the last years, the Department of Geotechnics of the University of Kassel has carried out a lot of experimental and numerical investigations to study these special conditions. This paper summarizes the results of these research works and shows modified approaches to tackle with the above factors.

1 INTRODUCTION

With the help of geosynthetic reinforced and pile supported embankments, known as GPE, static and dynamic traffic loads can be transferred directly to the bearing layer and thus relief the soft underground. The GPE-system is successfully used in the construction industry since beginning of the 1990s. The main application area of the GPE system is railway and road embankments on soft to very soft underground. In the last years, many researchers have dedicated themselves to this topic with different type of model concept. A summarized overview of the different models can be found in Heitz (2006).

The current method of design of a GPE-system is given for example in EBGEO (2010) (German recommendations for geosynthetic reinforcements), which is developed based on the arching model by Zaeske (2001) (see also Zaeske & Kempfert, 2002). However, this design method does not yet include all factors affecting the GPE system. Rather it partly lies on the safe side. These factors include the effect the pattern of the pile-like elements arrangement, the effect of lateral spreading, the influence of cyclic loading on the load transfer mechanism (arching effect), effect of product structure and number of reinforcement layers. For further investigation of these factors a series of model tests and numerical studies had been conducted at the Department of Geotechnics, University of Kassel. The results of these investigations are presented in the following.

2 INFLUENCE OF PILE ARRANGEMENT

The pile-like elements in GPE-systems can be arranged in rectangular or triangular pattern as shown in fig.1. The unrolling direction of the geogrids always takes place thereby in the embankment longitudinal axis. Experiences show that the triangular arrangement can favorably influence the load transfer mechanism of a GEP-system.

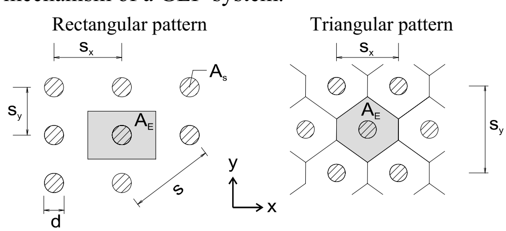


Figure 1. Arrangement of pile-like elements

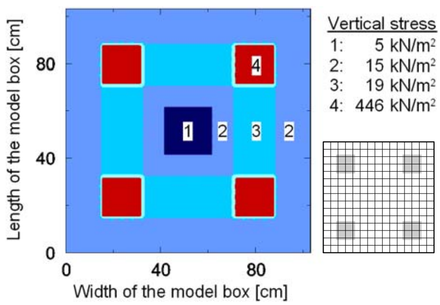
According to EBGEO (2010) a triangular arrangement of the piles is considered in the computation simply as a turned rectangular pattern, since no investigation is available so far in this direction. Hence, a series of test was conducted with which two different types of geogrids and two pile arrangement patterns have been investigated. Details about the model tests can be found in Kempfert et al. (2008) and Lüking et al. (2008).

For rectangular arrangement of the pile like elements, the model test results showed that the largest strains, hence the largest tensile stress, are transferred to the piles through the shortest direction between the piles. On the other hand a very small strain was measured at the center. The strain in triangular pattern showed quite another trend. The largest strains were recorded by strain gauges in the diagonal direction between the piles. Whereas those strain gauges arranged in the direction of the shortest distance between the piles had been subjected to less strain.

To analyze the stresses on the top of the geogrids and hence the soil arching in the embankment, back analyses of the model tests are carried out using the FEM program RSTAB Version 5. For more information about the numerical calculations, see Kempfert et al. (2008)

The calculated vertical soil pressure distributions at the top of the geogrids are shown in fig. 2. The geogrids are directly located above the top of the piles.

a)



b)

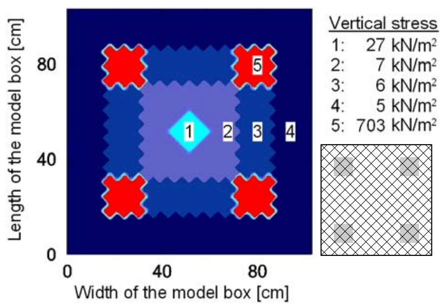


Figure 2. Vertical soil pressure distribution on the top of the geogrids, a) rectangular and, b) triangular pile arrangements

It can be observed from fig. 2b that the stress concentration on the top of the pile is higher for triangular grid system compared to rectangular grid system, which indicates a stronger soil arching. The soil pressures on the top of geogrids are relatively uniformly distributed and are smaller compared to rect-angular grid. However, there exists a higher local stress concentration in the middle of the geogrids, which reflects a possible base support for the higher arching.

Because of the different vertical soil pressure distribution compared to the rectangular pattern, it is assumed that the form of the soil arching is also different. The smaller soil pressure in the direction of the shortest distance between the piles indicates that no linear base support is available for the soil arching. On the other hand, the higher local stress concentration in the middle shows a point base support for the soil arching at this location.

The different form of the base support results in a reduced span length of the soil arching, and hence a stronger soil arching may be possible. The higher stress concentration on the top of the piles compared to rectangular pattern may support this hypothesis.

To take in to account the above mention effect of triangular pattern a modification has been recommended to the EBGEO (2010) design approach. The modification is on the spacing s between the piles, which were used in the calculation of the vertical soil pressure on the surface of the geogrids. Here, instead of $s = \max(s_x, s_y)$ as recommended in EBGEO (2010) $s = s_L$ will be used in the modified approach (fig. 3). A comparison of the maximum reinforcement strain according to the different approaches is indicated in fig. 4.

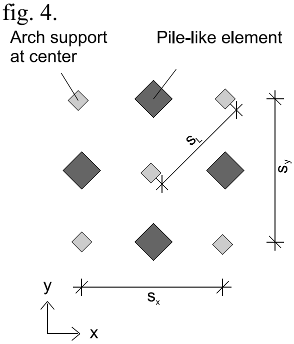


Figure 3. Modified spacing

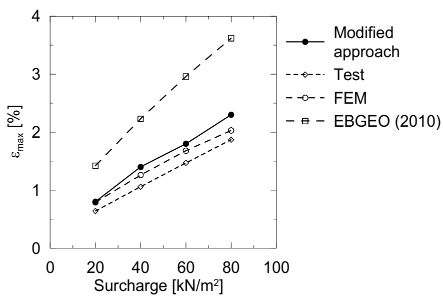


Figure 4. Comparison of strain in geogrids

3 INFLUENCE OF LATERAL SPREADING

In the slope zone of the embankment of a GPE-system the underground is subjected to additional lateral stresses due to the spreading effect of the slope. In practice, the spreading stresses are assumed equal to the active earth pressure at a section through the crest of the embankment. The spreading stresses influence the stability of the bearing system and possibly may result a horizontal displacement of the pile-like-elements or a horizontal displacement of the toe of the embankment slope. The horizontal forces must be transferred to reinforced elements such as horizontally lied geosynthetics reinforcement.

With increasing embankment heights, the spreading forces, and as a result, the tensile forces in the reinforcement will be dramatically increased and lead to higher deformations in the system. Both the membrane effect (arching effect) and the spreading effect influence the behaviour of the bearing system (such as pile elements) and the tensile forces in the reinforcement. Therefore, there is a high need to analyse and evaluate these effects for higher embankments.

The determination of the shear stresses and the horizontal deformations at the embankment base as well as the tensile forces in the geosynthetics reinforcement are followed through a series of large-scale model tests under variation of underground conditions. Similarly, the horizontal force on head of pile element due to spreading effect has also been measured and analysed. The large-scale model test results have been verified using a finite element method. Details of the model tests can be referred to the works by Fahmy et al. (2009) and Fahmy (2008).

Plane strain FE-models were used to analyze the model tests of un/reinforced embankments on soft underground without pile-like elements, whereas three-dimensional FE-models had been employed in the case of a piled soft underground. Fig. 5 shows selective results of the FE-computation and comparison with measured values. As it can be seen from Figure 5a, the calculated and measured strains in the geogrids agree very well in the case of underground without pile-like elements. Whereas, the calculated strains in the base reinforcement on top of a pile like elements show a large difference (fig. 5b). This may be attributed to the simulation of the geogrids as a membrane. The geogrids seems to behave differently as a membrane, especially when it is laid on a point support system. This phenomenon has also been reported by Zaeske (2001), Jenck et al. (2005) and Heitz (2006). Based on the authors own results such as shown in Figure 5b and back analysis of model test results from the literature (for e.g. Zaeske 2001, Heitz 2006) a factor about 3.5 is derived between calculated and measured values.

A series of numerical parameter studies on the prototype using 3D-FEM are performed under different parameter variations such as embankment height, slope, underground stiffness, geogrids stiffness and number of layers, etc. It is obvious that the spreading and the membrane forces increase with increasing height. The result of the numerical study also confirms that under steeper slope the shear stress at the slope base is greater, and consequently the resulting spreading forces are greater. The effect of the slope is more noticeable in high embankments than the lower embankments.

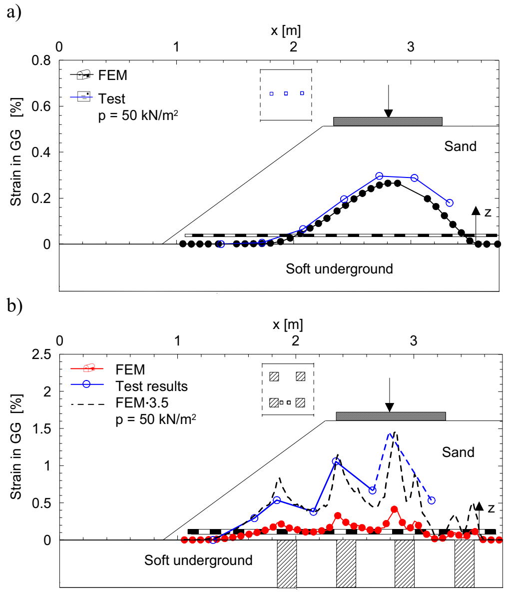


Figure 5. Strain in reinforcement (GG) a) unpiled and b) piled embankment

Both the spreading and membrane forces in reinforcement are also observed to be smaller in the case of stiffer underground than soft underground. This is attributed to the small shear deformations of the stiffer underground.

EBGEO (2010) recommends two approaches (option 1 and 2) for the determination of the total tensile force in reinforcement. In option 1 the total tensile force is taken as the sum total of the membrane force ($F_{G,M}$) and the spreading force ($F_{G,S}$). This approach is similar to that recommended by BS 8006 (1995).

$$F_G = F_{G,M} + F_{G,S} \tag{1}$$

Option 2 is similar to the approach by Love & Milligan (2003) and it is based on the concept that basal reinforcement can only have one tension in the transverse direction of embankment. The reinforcement in this case should be designed for whichever is the greater: the membrane force or spreading force, but not their sum. The same approach was also adopted by Klobe (2007).

$$F_G = max \begin{cases} F_{G,M} \\ F_{G,S} \end{cases} \tag{2}$$

Both options of EBGEO (2010) can lead to an overestimation of the tensile force in the reinforcement as compared to the FEM-results, especially in the case of high embankments. This is mainly attributed to the assumption that the spreading force is equal to the horizontal active earth pressure force at a section through the crest of the embankment and ignorance of the stiffness of the underground.

Fahmy (2008) introduced a modification to EBGEO (2010) method based on the assumption that the section, through which the horizontal earth pressure force is determined, is not always fixed at the embankment crest, rather it moves towards the toe depending on the height of the embankment as shown in fig. 6.

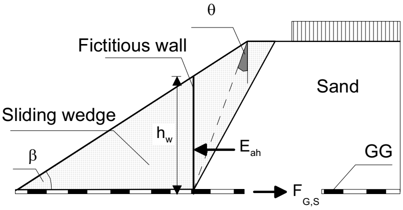


Figure 6. Sliding soil wedge

The position and the height of the fictitious wall h_w depends mainly on a vertical angle θ from the slope crest. The critical angle θ can be determined by equating the so calculated spreading force with that obtained from FEM. The earth pressure force E_{ah} on the fictitious wall which is assumed equal to the spreading force $F_{G,S}$ can be calculated using the earth pressure theory.

By comparing the tensile forces due to spreading determined analytically and numerically, the critical angles for different embankment heights are identified for the case of a peat underground (E_s=0.8 MN/m^2) and embankment slope of 1:1.5 (reference model). For embankment height h_1 up to 5m, the critical angle is found to be $\theta = 0°$ and for h_1=10m, $\theta = 30°$. For other underground conditions and different embankment slopes, adjustment factors are introduced to the tensile force $F_{G,S}$ by Fahmy (2008) as follows:

$$F_{G,S} = E_{ah} \cdot (0.92 - 0.0085 \cdot h_1) \cdot E_s^{(-0.4768 - 0.0165 \cdot h_1)} \cdot$$
$$\cdot (0.962 + 0.0504 \cdot h_1) \cdot \left(\frac{1}{tan\,\beta} \right)^{(0.0334 - 0.099 \cdot h_1)} \quad (3)$$

where E_{ah} ($h = h_w$) is the earth pressure force for the reference model.

Fig. 7 shows a comparison of the tensile forces in the reinforcement due to spreading according to the modified and the EBGEO (2010) approaches for different underground conditions and embankment slopes.

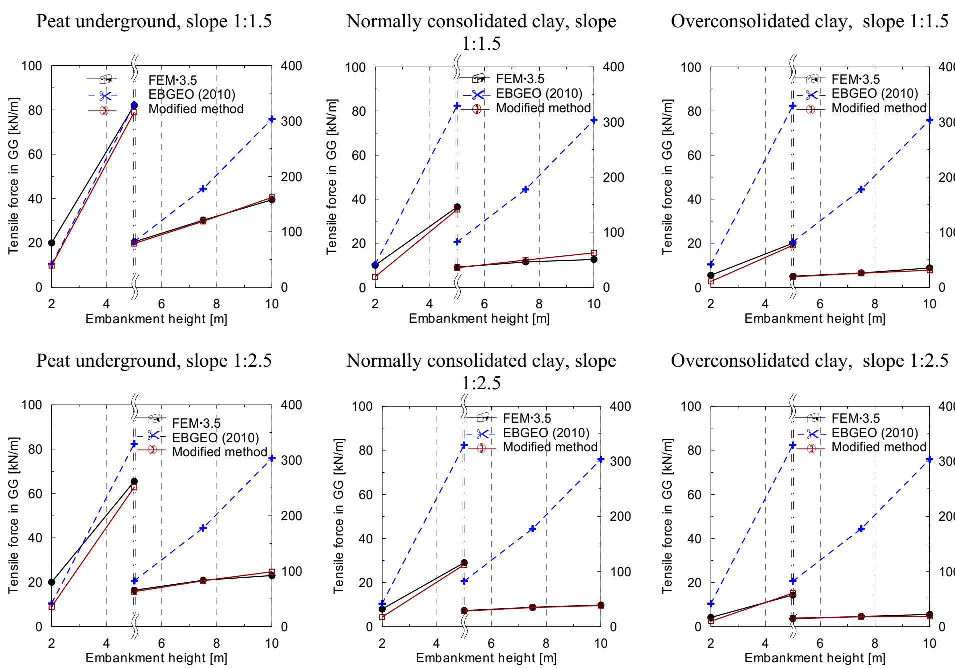


Figure 7. Comparison of tensile forces in reinforcement due to spreading according to the EBGEO (2010) and the modified approaches at a surcharge pressure of p = 30 kN/m^2

4 INFLUENCE OF CYCLIC LOADING

Whereas the system behaviour of GPE-system under static loading is well-known, its behaviour under cyclic loading is not yet fully explained. For this purpose a lot of research had been carried out to examine the soil pressure distribution above the pile-heads and the bearing effect of the geosynthetic reinforcement. In the following a summary of the research works by Heitz (2006), Heitz & Kempfert (2007) and Heitz et al. (2008) will be presented.

Without geosynthetic reinforcement the soil arching can only be formed in a very limited extent under cyclic loading. The geogrids stabilise the system under cyclic loading and reduce the settlements of the ground surface. The reduction of the soil arching and the punching mechanism leads to considerably increase in strain in the geogrids. Especially the lowest reinforcement layer near the pile heads suffered the most. Based on the results of model tests a soil arching reduction factor κ was introduced, i.e.,

$$\kappa = \frac{E_{stat}}{E_{zykl}} \tag{4}$$

where E_{stat} and E_{zykl} are soil arching ratios due to static and cyclic loading respectively. Fig. 8 illustrates this factor depending on the ratio of the sand fill height and pile spacing *h/s,* frequency *f*, and amplitude σ_c.

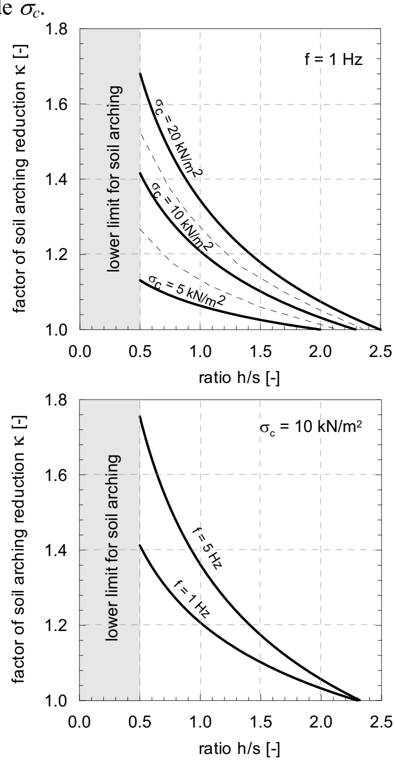


Figure 8. Soil arching reduction factor

Heitz 2006 proposed a modified method which takes into account the soil arching reduction factor and an increase of strains in geogrids. The increased vertical soil pressure $\sigma_{z0,zykl}$ on the soft soil between the piles is given by:

$$\sigma_{z0,zykl} = \frac{(\gamma \cdot h + \sigma_{stat}) \cdot A_E}{A_E - A_S} \cdot \left(1 - \frac{1}{\kappa}\right) + \frac{1}{\kappa} \cdot \sigma_{z0}^{stat} \tag{5}$$

where σ_{z0}^{stat} is calculated according to EBGEO (2010). Fig 9 illustrates a comparison between the calculated values according to equation (5) and the model test results. The calculated stresses using equation (5) lie on the safe side.

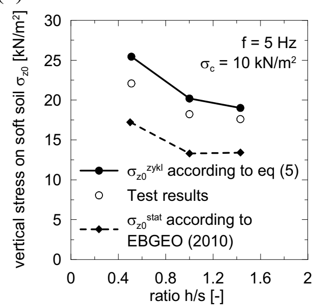


Figure 9. Comparison of equation (5) and model test results

5 INFLUENCE OF PRODUCT STRUCTURE AND NUMBER OF LAYERS

The structure and the number of reinforcement layers may affect the load transfer mechanism of reinforced embankment on pile-like elements. The model tests carried out to investigate these influences are reported by Kempfert et al. (2007) & (2009). Two geogrid products, mainly GW and GL were considered in the study with one to three reinforcement layers. GW 60 PET geogrids are made of woven synthetic yarns and have a protective polymer coating. The ribs are woven together at cross points. The GL 60 PET is flat geogrids made from interlaced extruded bars (J=850 kN/m for both).

It was observed that the lower layer of reinforcement is the most efficient in carrying the applied load. However, with increasing embankment height there is a clear reduction in membrane effect. The share of the load transfer by the different layers of reinforcement is given in Table 1.

Table 1: Share of the load transfer

Number of layers	Model tests	EBGEO (2010)
2	1:0.65	1:1
3	1:0.65:0.35	not considered

Dividing the required tensile strength of geogrids into several layers may lead to a more unfavourable system performance. Thus, it is recommended to limit the number of layers to a minimum as much as possible. The above top layer of three layered geogrids contributes the least to the load transfer mechanism and it may additionally damage the development of the soil arching. Hence, it is not recommended in the practice.

The product structure effect on the development of strain in the reinforcement strips and the deformation behaviour of the whole GPE-system under cyclic loading is illustrated in fig. 10 & 11. It can be observed from fig. 10 that the GEP-system with multi-layer reinforcement suffers less settlement compared to one layer reinforcement. The effect of settlement reduction from two to three layers of reinforcement is clearly smaller than that of one to two layers of reinforcement. It appears from fig. 11 that the strain in the strips of the geogrids in the lowest layer increases with number of cycles. This is a clear indication of the reduction of the soil arching effect during cyclic loading.

Moreover, no major difference in system performance (settlement and strain in the reinforcement) can be observed between the different products for the same number of reinforcement layers.

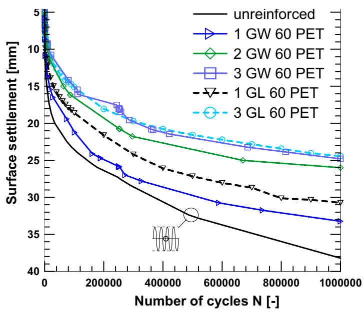


Figure 10. Surface settlement of the GEP-system

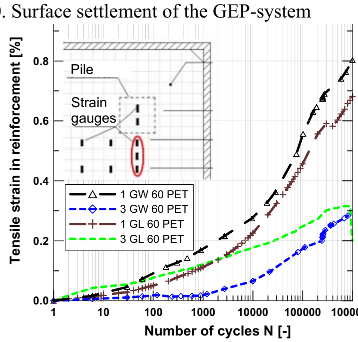


Figure 11. Selected strains in reinforcement

6 CONCLUSION

A summary of factors influencing the design of a GEP-construction is given in the paper. These include: pile raster arrangement, lateral spreading, cyclic loading, product structure and number of reinforcement layers. These factors are not usually considered in practice in the design of the GEP-system. Adjustment factors and modified approaches have been recommended.

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