GERMAN RECOMMENDATIONS FOR REINFORCED EMBANKMENTS ON PILE-SIMILAR ELEMENTS

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ABSTRACT: The construction of embankments on soft underground is a common problem. In recent years a new kind of foundation, the so-called "geosynthetic reinforced pile-supported embankment", was established. Until now the system behaviour can only be described analytically by simplified geomechanical models. Furthermore, there are simplified calculation procedures, which allow the dimensioning of the geosynthetic reinforcement. In the course of the revision of the EBGEO (German Recommendations for Geosynthetic Reinforced Earth Structures), new recommendations for soil reinforcements above pile-similar elements under static loading were worked out. These new developed analytical methods represent a new State-of-the-Art and enable a realistic and suitable approximation of the bearing behaviour of the composite structure. They differ significantly from the existing methods. The paper describes the new methods of calculation and the construction regulations for this kind of foundation as recommended by the EBGEO.

1 INTRODUCTION

Soil improvement and reinforcement techniques have undergone a significant development during the last decade, especially as a result of the increasing need to construct on soft ground providing economical solutions. Designing structures, such as buildings, walls or embankments on soft soil raises several concerns. They are related to bearing capacity failures, intolerable settlements, large lateral pressure and movement, and global or local instability. A variety of techniques may be used to address the above concerns. These include pre-loading the soft soil, using light-weight fill, soil excavation and replacement, geosynthetic reinforcement and soil improvement techniques.

In recent years a new kind of foundation, the so-called "geosynthetic-reinforced pile-supported embankment" was established (Fig. 1).

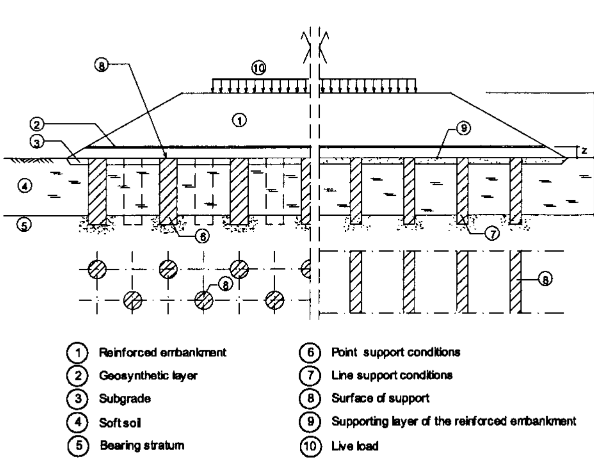


Figure 1 Geosynthetic-reinforced pile-supported embankment

The pile elements (e.g. concrete piles, cemented stone columns, walls etc.) are placed in a regular pattern through the soft soil down to a lower load-bearing stratum.

Three possible support conditions are illustrated in Figure 2. Piles are typically arranged in rectangular or triangular patterns in practice.

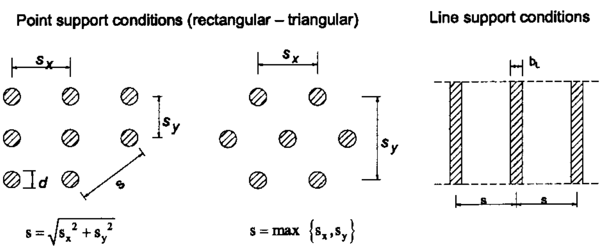


Figure 2 Support conditions and definition of the distance s

Above the pile heads, the reinforcement of one or more layers of geosynthetics (mostly geogrids) is placed.

In Germany the geosynthetic-reinforced pile-supported systems have been used for several applications, especially for highway and railroad embankments (ALEXIEW (1999), (2001)).

The systems have proved to perform well regarding both bearing capacity and serviceability if designed and constructed in an appropriate way (ALEXIEW (1999), (2001)).

Until now the system behaviour can be described analytically only by simplified geomechanical models. Furthermore, there are simplified calculation procedures, which allow the dimensioning of the geosynthetic reinforcement (e.g. HEWLETT (1988), BS 8006 (1995), KEMPFERT 1997), ALEXIEW (2002)). To examine the bearing mechanisms in the system and to derive a better analytical model, a research project has taken place at the Institute of Geotechnics, University of Kassel (KEMPFERT (1999), ZAESKE (2001, 2002)). The developed design procedure will be introduced soon into Chapter 6.9 "Reinforced earth structures on point- or line- shaped bearing elements" (EMPFEHLUNG 6.9 (2003)) of the new edition of the EBGEO (German Recommendations for Geosynthetic Reinforcement).

This new analytical method represents a new State-of-the-Art. It is believed to be more precise and realistic than the "older" procedures available, which was confirmed by experiments (ZAESKE (2001)); at the same time it is more sophisticated and like other procedures available limited mostly to non-cohesive fills. An overview of common procedures today is given e.g. in (ALEXIEW (2002)).

The general load transfer mechanisms, model test results and the new method of calculation and the construction recommendations for this kind of foundation as recommended in Chapter 6.9 of the EBGEO will be described shortly.

2 LOAD TRANSFER MECHANISMS

The stress relief of the soft soil results from an arching effect in the reinforced embankment over the pile heads and a membrane effect of the geosynthetic reinforcement. Due to the higher stiffness of the piles in relation to the surrounding soft soil, the vertical stresses from the embankment are concentrated on the piles, simultaneously soil arching develops as a result of differential settlements between the stiff pile heads and the soft soil between them. The 3D-arches span the soft soil and the applied load is transferred onto the piles and then to the bearing stratum. The redistribution of loads in the embankment depends on the systems geometry, the strength of embankment soil and the stiffness of "piles".

A modified stress-distribution theory was developed (ZAESKE (2001)). Additionally, a concept to take into account the supporting soft soil upwards counter-pressure between the piles in a deformation-related way was introduced including the tensile stiffness of reinforcement and the oedometric modulus of soft soil. Differential equations had to be developed to reflect this interaction (ZAESKE (2001)) (Fig. 3).

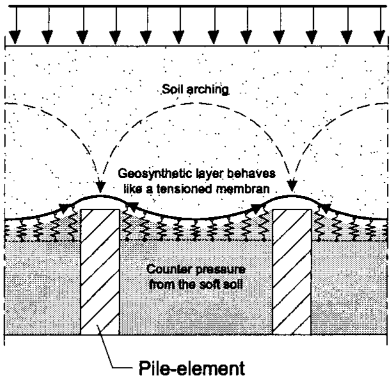


Figure 3 Mechanisms of load transfer and interaction

3 RESULTS OF MODEL TESTS UNDER STATIC LOADING

Three-dimensional well-instrumented model tests in a scale of 1:3 were carried out to investigate the bearing and deformation behaviour and to check and verify the concept and theory mentioned above. A group of four piles was placed in a weak soil of peat in a rectangular grid, above which a reinforced or unreinforced sand fill was placed in different heights (Fig. 4).

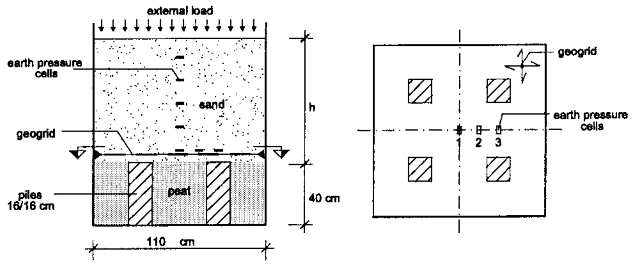


Figure 4 Typical 1:3 scale test arrangement (ZAESKE, 2001)

The stress distribution in the reinforced sand layer was recorded by pressure cells. The part of the load carried by the piles was measured by load-cells and allowed a comparison with the measured stress field in the sand. Under static loading the dependency of the stress transfer on the geometric boundary conditions and the shear strength of the sand fill was verified.

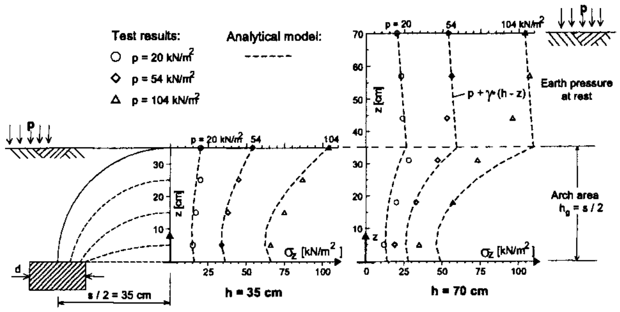


Figure 5 Test results versus analytical model (ZAESKE, 2001)

Vertical rod extensiometers to monitor the settlements of the geogrid and strain measurements in the geogrid reinforcement confirmed the membrane effect and allowed a localisation of the highest tension. The maximum strains were localised in the zone overspanning neighbouring piles (Fig. 6).

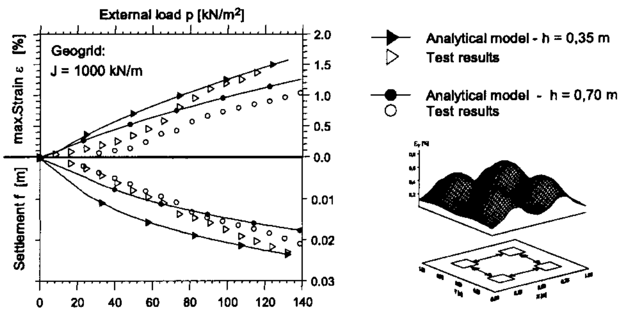


Figure 6 Typical geogrid-strain distribution, predicted and measured values (ZAESKE, 2001)

Similar to field measurements, the strains in the geogrid were found to be relatively small, provided that reaction stress of the underlying soil between the rigid pile elements is mobilised.

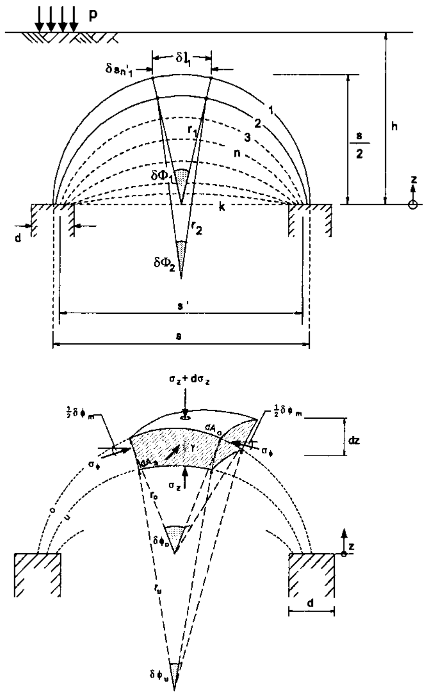
In addition to the model tests, numerical investigations with the finite element method (FEM) were performed for static conditions. The evaluation of the FE-calculations resulted in further information on the stress distribution in the reinforcing layer and the resulting load transfer onto the piles.

After these verifications, the new method became part of Chapter 6.9 of the new edition of the EBGEO (draft) and is explained in the following chapter.

4 DESIGN RECOMMENDATION IN CHAPTER 6.9 EBGEO (DRAFT)

The design procedure recommended in Chapter 6.9 of the EBGEO (draft) (EMPFEHLUNG 6.9 (2003)) is divided into two steps:

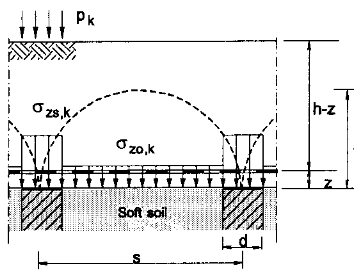
In the first step the load/stress distribution in the embankment is evaluated without considering any geosynthetic reinforcement, which results in the vertical stresses on top of the piles ($\sigma_{zs,k}$) and on the soft subsoil between them ($\sigma_{zo,k}$). The analytical model is based on the lower bound theorem of the plasticity theory and results from pretended directions of the stress trajectories in the reinforced soil body (ZAESKE (2001, 2002)). According to the numerical and experimental results the stress state in the reinforced embankment is divided into a zone, where the earth pressure at rest can be assumed, and an arching region, where the stress redistribution takes place (Fig. 4). Equation (1) shows the differential equation derived from the equilibrium of forces of the three-dimensional soil element in radial direction (Figure 7).



$$-\sigma_z \cdot dA_u + (\sigma_z + d\sigma_z) \cdot dA_o - 4 \cdot \sigma_\Phi \cdot dA_s \cdot \sin\left(\frac{\delta\Phi_m}{2}\right) + \gamma \cdot dV = 0 \qquad (1)$$

Figure 7 Geometry, "arching" and equilibrium of stresses (ZAESKE, 2001, 2002)

The solution of the equation gives the vertical stress $\sigma_z(z)$ inside the arch. The vertical pressure on the soft soil $\sigma_{zo,k}$ results from the limit $z \to 0$, Equation (2).



$$\sigma_{zo,k} = \lambda_1^{\chi} \cdot \left(\gamma_k + \frac{p_k}{h}\right) \cdot \left(h \cdot \left(\lambda_1 + h_g^2 \cdot \lambda_2\right)^{-\chi} + h_g \cdot \left(\left(\lambda_1 + \frac{h_g^2 \cdot \lambda_2}{4}\right)^{-\chi} - \left(\lambda_1 + h_g^2 \cdot \lambda_2\right)^{-\chi}\right)\right) \qquad (2)$$

$$\chi = \frac{d \cdot (K_{krit} - 1)}{\lambda_2 \cdot s} \qquad K_{krit} = \tan^2\left[45° + \frac{\varphi'_k}{2}\right]$$

$$\lambda_1 = \frac{1}{8} \cdot (s - d)^2 \qquad \lambda_2 = \frac{s^2 + 2 \cdot d \cdot s - d^2}{2 \cdot s^2}$$

φ'_k characteristic value of the internal angle of friction (embankment)
γ_k characteristic unit weight of soil (embankment) [kN/m³]
p_k characteristic value of the live load [kN/m²]
h_g vault height [m] $h \geq s/2 \to h_g = s/2$
$h < s/2 \to h_g = h$

Figure 8 Vertical stress $\sigma_{zo,k}$ on the soft soil (EMPFEHLUNG 6.9, 2003)

For more convenience, $\sigma_{zo,k}$ can also be derived from dimensionless design graphs (e.g. Figure 9 for φ'_k = 30°). Figure 4 shows the calculated vertical stress distribution in comparison with results of the model tests

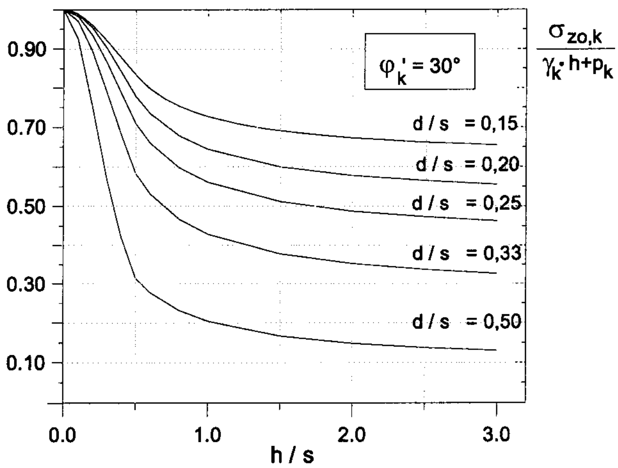


Figure 9 Typical graph for the vertical stress $\sigma_{zo,k}$ on the soft soil between "piles" (EMPFEHLUNG 6.9, 2003)

In the second step, the vertical pressure $\sigma_{zo,k}$ is applied to the geosynthetic reinforcement as external load.

To predict the stresses in the reinforcement, an analytical model is applied based on the theory of elastically embedded membranes (ZAESKE (2001)). The maximum strain in reinforcement (i.e. the maximum tensile force) is concentrated in the band bridging two neighboured piles (despite the common engineering sense, it was confirmed by the experimental work as well). Therefore the analytical model assumes that the maximum stress in the geosynthetic membrane takes place within the width $b_{Ers.}$ and may be calculated based on a planar system (Fig. 10). Biaxial geogrids must be analysed both in x- and y-direction.

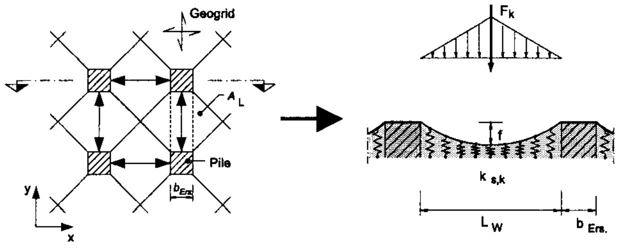
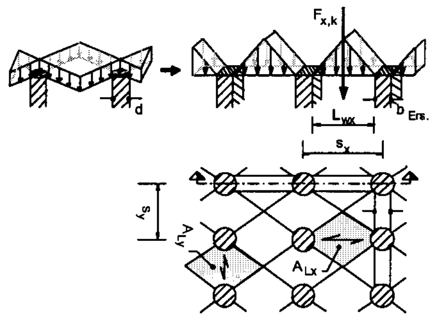


Figure 10 Load transfer and simplified planar (2D) bearing system (ZAESKE, 2001, 2002)

The resulting triangular vertical strip load F_k on the geogrid strip is calculated from the pressure $\sigma_{zo,k}$ and the loaded area A_L (Fig. 11).

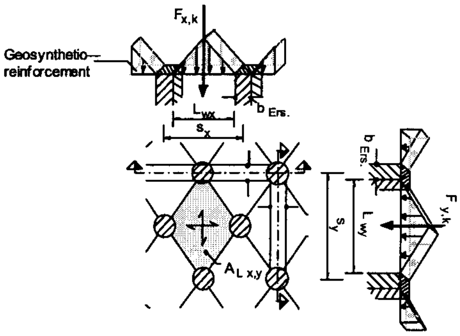
Rectangular grid



$$A_{Lx} = \frac{1}{2}\cdot(s_x \cdot s_y) - \frac{d^2}{2}\cdot \text{atn}\left[\frac{s_y}{s_x}\right]\cdot\frac{\pi}{180} \qquad F_{x,k} = A_{Lx}\cdot\sigma_{zo,k}$$

$$A_{Ly} = \frac{1}{2}\cdot(s_x \cdot s_y) - \frac{d^2}{2}\cdot \text{atn}\left[\frac{s_x}{s_y}\right]\cdot\frac{\pi}{180} \qquad F_{y,k} = A_{Ly}\cdot\sigma_{zo,k}$$

Triangular grid



$$A_{Lxy} = \frac{1}{2}\cdot s_x \cdot s_y - \frac{d^2}{4}\cdot\pi$$

$$F_{x,k} = \frac{J_x}{J_x + J_y}\cdot A_{Lxy}\cdot\sigma_{zo,k}$$

$$F_{y,k} = \frac{J_y}{J_x + J_y}\cdot A_{Lxy}\cdot\sigma_{zo,k}$$

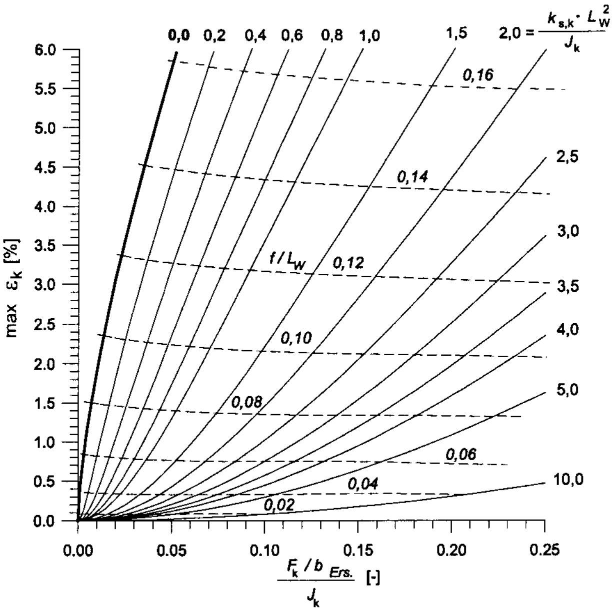
Figure 11 Calculation of the resulting force F_k assigned to the load influence area A_L (EMPFEHLUNG 6.9, 2003)

The influence of the bearing effect of the soft soil between piles is considered by using a modulus of subgrade reaction. A simplified approximation is given in Equation (3); for multiple soft soil layers see EMPFEHLUNG 6.9 (2003).

The maximum strain in the geosynthetic reinforcement results from the tensile stiffness J_k of the geosynthetic, the modulus of subgrade reaction $k_{s,k}$ of the soft soil, the total vertical load F_k and the dimensions b_{Ers} and L_W. Since all geosynthetics tend to creep, the tensile modulus J_k is time-dependent and has to be red out from the real isochrones of the geosynthetic reinforcement; the latter is essential.

In EMPFEHLUNG 6.9 (2003), the value of ε_k respectively f can be out from a dimensionless design graphs, see e.g. (Fig. 12). Finally, the tensile force in the reinforcement $E_{M,k}$ (M = membrane) can be calculated directly as a function of the strain of the geosynthetic, Equation (5).

For two geosynthetic reinforcements the calculated force is divided with respect to the ratio of their tensile moduli.



J_k	tensile stiffness of the geogrid	[kN/m]
L_w	clear distance = (s-d)	[m]
$k_{s,k}$	modulus of subgrade reaction	[kN/m³]

$$k_{s,k} = \frac{E_{s,k}}{t_W} \qquad (3)$$

$b_{Ers.}$ width of support [m]

$$b_{Ers.} = \frac{1}{2}\cdot d\cdot\sqrt{\pi} \qquad (4)$$

Stess in the reinforcement:

$$E_{M,k} = \varepsilon_k \cdot J_k \qquad (5)$$

Figure 12 Maximum strain in the geosynthetic reinforcement (EMPFEHLUNG 6.9, 2003)

The influence of an inclined surface of the reinforced embankment (typically slope) is illustrated in Figure 13.

In addition to the membrane effect, geosynthetics are stressed by horizontal forces. The lateral thrust can be considered on the safer side assuming an active earth pressure condition without any support by "piles" or soft soil (BS 8006 (1995), ZAESKE (2001, 2002)). The concept is conservative.

$$\Delta E_k = E_{ah,k} = \left(\frac{1}{2}\cdot\gamma_k\cdot(h-z)+p_k\right)\cdot(h-z)\cdot K_{ah} \quad (6)$$

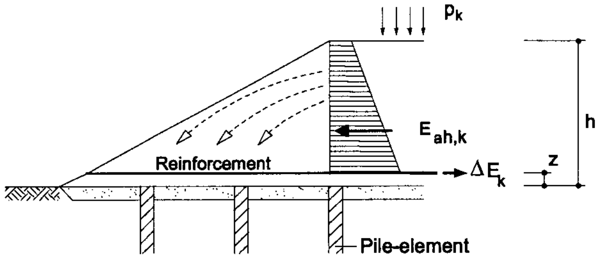


Figure 13 Additional horizontal force in the reinforcement beneath embankment slope (EMPFEHLUNG 6.9, 2003)

5 CONSTRUCTION RECOMMENDATIONS IN CHAPTER 6.9 EBGEO

Based on German and international experience with geosynthetic-reinforced pile-supported embankments, practical reasons, experimental results and the validity of the analytical model following recommendations are established:

5.1 *Pile elements and spacing*

The center-to-center distance s and the pile diameter d of the piles resp. pile caps should be chosen as follows:

- $(s - d) \leq 3{,}0$ m resp. $(s - b_L) \leq 3{,}0$ m: in the case of static loads
- $(s - d) \leq 2{,}5$ m resp. $(s - b_L) \leq 2{,}5$: in the case of heavy live loads
- $d / s \geq 0{,}15$ resp. $b_L / s \geq 0{,}15$
- $(s - d) \leq 1{,}4\,(h - z)$
- The ratio of the modulus of subgrade reaction between the pile elements and the surrounding soft soil $k_{s,p} / k_{s,w} > 100$ (to ensure full "arching" activation assumed in the design); normally, conventional pile-systems fulfil this condition.

5.2 *Geosynthetic reinforcement*

The distance between the reinforcement layer and the plane of the pile/column/wall heads should be as small as possible, in order to achieve maximum efficiency of the geosynthetic membrane. However, it is recommended to have a safe distance (interlayer) between the lowest reinforcement and the pile heads in order to prevent a structural damage of the reinforcement because of shearing at the edge of the pile heads.

- maximum two reinforcement layers (Fig. 14)
- $z \leq 0{,}15$ m for single layer reinforcement
- $z \leq 0{,}30$ m for two layers reinforcement
- for two layers the distance between the geosynthetic layers should be 15 to 30 cm
- design value of the tensile strength $R_{Bd} \geq 30$ kN/m; ultimate strain ≤ 12 %.
- Overlapping is generally allowed, but only just above the pile (caps) and only in the secondary bearing direction; length of overlapping $\geq$ d.

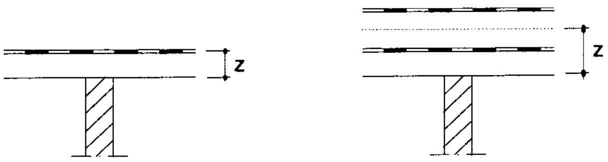


Figure 14 Distance z in the case of one and two reinforcement layers (EMPFEHLUNG 6.9, 2003)

5.3 *Embankment*

For the embankment a cohesionless fill should be used. The angle of internal friction φ'_k should be greater than 30°. Use of low-cohesion soils is also permitted, but not preferred.

(Note: A general issue to be always kept in mind is if the soft soil upward counter-pressure will be available for the entire design life. Not-supported situations should be checked additionally).

6 FINAL REMARKS AND FUTURE PROSPECTS

Geosynthetic-reinforced embankments on point- or line shaped bearing elements ("piles") provide an economical and effective solution for embankments constructed on soft soil, especially when rapid construction and strict deformation of the structure are required.

To examine the bearing effect of the system, large scale model tests and numerical investigations were carried out. Based on these results a theoretical model was developed, which describes the stress-distribution in the embankment and the membrane effect of the geosynthetic reinforcement. The developed design procedure is introduced in the recommendation "Chapter 6.9 - Reinforced soil structures above point- or line shaped bearing elements" and is already released as a draft to the public. Chapter 6.9 will be soon part of the new edition of the EBGEO (German Recommendations for Geosynthetic Reinforcement) (EMPFEHLUNG 6.9, 2003).

The design method provides a realistic approximation of the bearing behaviour of the composite structure under static loading and represents a new State-of-the-Art. Comparisons of the analytical results with model test data and field measurements demonstrate that the new design model still leads to a conservative prediction of the bearing behaviour, at least so far as counter-pressure from the soft soil is available. Recommendations regarding geometry, soils, reinforcement and construction are given as well.

At present, a research project is in progress at the Institute of Geotechnics, University of Kassel, which examines the behaviour of such systems under cyclic loading.

The first model tests show that the arching effect was only formed in a very limited amount and the part of the load carried directly by the piles decreased remarkably, which resulted in an increase of the load on the soft soil and/or in the reinforcement. Due to the reduction of the arching effect, the strains in the geogrid and the settlements of the surface increase considerably.

Because such negative effects of cyclic load have been suspected earlier due to common sense, a quasi-static approach was suggested in KEMPFERT (1997) by applying an additional increasing partial factor to traffic loads, which is at present an acceptable compromise.

Nevertheless the bearing behaviour and the settlements expected under cyclic load are not yet fully explained. The further research required for that specific important issue and is ongoing.

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