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Interactions in reinforced bearing layers over partial supported underground
Interaction de couches de sol porteuses, armées, avec un sous sol qui par endroit ne cède pas

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ABSTRACT: Large - scale (1 : 3) model tests have been performed to evaluate the behaviour of horizontal geosynthetic-reinforced bearing layers (res. embankments) on soft subsoil supported partially by piles. The system has been applied often especially for highway and railroad embankments. Until now the system behaviour can be described analytically only by simplified geomechanical models - mainly its ultimate limit state. The most important test results are shortly presented and compared with the available commonly used dimensioning procedures.

RÉSUMÉ: Dans cette contribution sont présentées les résultats d'essais-modèles, qui ont étés réalisés pour l'examination de l'interaction de couches de sol porteuses, armées, avec un sous sol qui par endroit ne cède pas, par ex. des pieux. Le système porteur, que l'on retrouve souvent pour la réalisation ou l'assainissement de barrages, digues, ne peut être décrit analytiquement que par un modèle simplifié. Une comparaison des résultats d'examen avec des modèles analytiques existants sont faits et sont notés les différences, respectivement les sources d'erreurs.

1 INTRODUCTION

The construction of fills on unstable underground is a common problem, e.g. the foundation of embankments on soft soil. Solving the problem e.g. by using high-strength horizontal geosynthetic reinforcement in the base of embankment has become common engineering practice, but suitable deformation behaviour often requires long waiting time for consolidation settlement. In recent years a new kind of foundation was established, where this disadvantage is nearly eliminated: the so called "piled embankments". Piles (or pile-similar elements) are driven in a regular screen disposition into the in-situ soil down to bearing soil, transferring the loads directly downwards and decompressing the soft soil significantly. Over the pile caps, a reinforcement by one ore more layers of geosynthetics (mostly geogrids) is placed and above this the embankment will be provoked. The most efficient version of the system - with horizontal geosynthetic reinforcement - is described e.g. in John (1987).

During the last years the system has been successfully applied for important structures incl. sections of the German Railways by using high-strength geogrids. High bearing capacity and very low long-term deformations under traffic-loads have been registered, Alexiew et al. (1995), Gartung et al. (1996).

2 BEARING BEHAVIOR

Due to the higher stiffness of the piles relative to the surrounded soft soil, the vertical stresses are concentrated in the area over the piles, simultaneously the stresses over the soft soil reduces. The stress field in the embankment is strongly inhomogeneous. The stress / strain distribution depends generally on following factors: ratio of supported / unsupported base area, ratio of embankment height / pile-spacing, parameters of embankment soil (strength and moduli), support (counter-pressure upwards) of soft subsoil between piles. In any case, the settlement between piles is larger than directly on the tops of them, generating strains (and tensile forces) in the horizontal geosynthetic reinforcement. Therefore the geosynthetics are clamping like a membrane over the unyielding piles and the degree of the load transference by the piles increases additionally. Only small residual stresses are demanding the soil between the piles.

Until now there is no analytical approach which describes precisely this bearing behavior of the system embankment - reinforcement - piles - soft subsoil. Nevertheless, there are simplified calculation procedures, which allow for dimensioning of the geosynthetic reinforcement, e.g. acc. to John (1987), British Standard BS 8006 (1994), Kempfert et al. (1997).

Usually the design precedure is divided into two steps. As a first step, the load / stress distribution in the embankment is evaluated without considering any geosynthetics-reinforcement, resulting in vertical stresses on top of piles and on soft subsoil in-between. For this, particularly the methods of Marston (1973), used e.g. in John (1987) and in British Standard BS 8006 (1994) and the method of Hewlett & Randolph (1988) considered in Kempfert et al. (1997) are common. The latter seems to be better - founded and confirmed by tests in the 80's. Whereas Marston (1973) proceeds by empirical arching coefficients, that are estimated in dependence of the height of the embankment, the stiffness and the diameter of the piles, Hewlett & Randolph (1988) assume a series of thick-walled 3D-shells in the embankment (Fig. 1a). The maximum stress redistribution via the shell to the pile-supports is derived from classical soil-plastification criteria at the top ("key") and the toes of the 3D-arch. The assumed typical stress distribution is shown in Figure 1b.

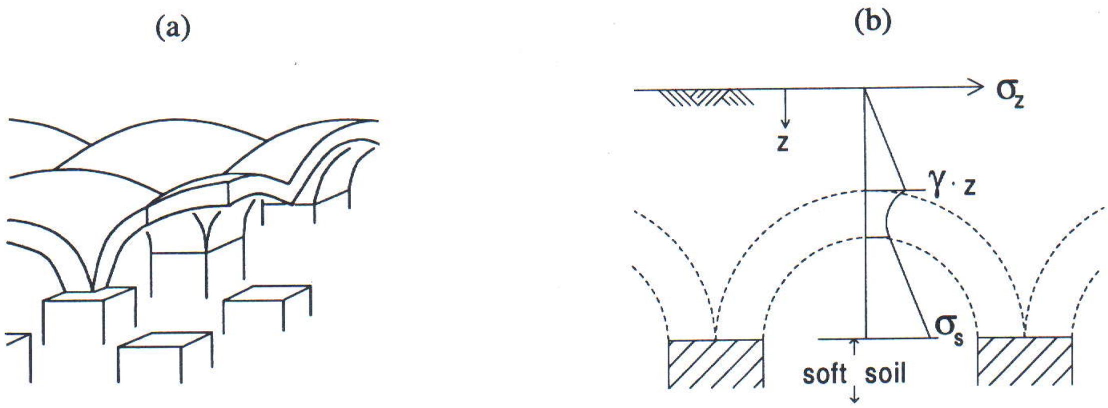


Figure 1. 3D-arching model acc. to Hewlett & Randolph (1988)

As a second step, the vertical stress σ_s (Fig. 2a) is applied to the geosynthetic reinforcement as external loading. The forces in the reinforcement are calculated e.g. by using the equations of the chord, overspanning the spacing between adjacent piles. Herewith a strip of the geosynthetic reinforcement, loaded by the vertical stresses σ_s over assigned influence areas, is considered. This second step is quite similar in John (1987) and British Standard BS 8006 (1994) and more precisely evaluated in Kempfert et al. (1997) (Fig. 2b).

In all cases σ_s is set to be constant between the piles (Fig. 2a). A possible counterpressure σ_{up} (upwards) from the partially compressed upper zone of soft subsoil between piies, which reduces the tension in reinforcement, is still a matter of discussion and safety philosophy (e.g. for permanent structures a decrease of subsoil phreatic line will eliminate any counterpressure); a possible assumption for σ_{up} is described in Kempfert et al.(1997).

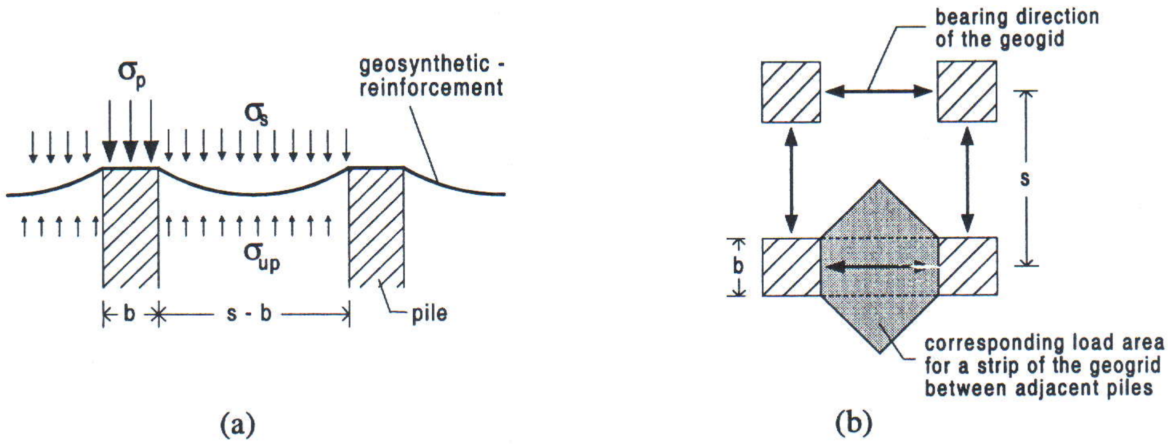


Figure 2. load distribution in the area of the reinforcement (a) and assumed bearing directions for geogrids (b)

Summarizing: the common analytical procedures available today separate stress redistribution, load on reinforcement and membrane action of the reinforcement. No interaction of soil and tensioned "membrane" is taken into account. Although the structures dimensioned this way are proved to behave well, a better understanding of the complete behavior is necessary.

3 MODEL TESTS

For a better understanding of the complete behaviour of the systems discussed and to check/verify so far as possible the analytical models available, large-scaled 3D-model tests have been performed, see Figure 4. A poorly graded sand with d_{50} = 0,35 mm was used at D_{pr} = 100 % for all tests. The sand height h was varied. The test geometry corresponds to common structure geometry's at a 1 : 3 - scale. The peat was built in under slight compaction. Tests were performed without any reinforcement and with two different model polyester biaxial geogrids modeling commonly used high-strength polyester Fortrac® - geogrids in the prototypes, with affin stress/strain behavior and bond coefficients. The geogrids were installed in a frame that could slide vertically; a sliding film was installed inside the entire case walls. Increasing statical surcharge p was applied by a soft pillow. Additional cyclic load tests have been performed. Geogrid strains and deflections, soil pressures and effective loads on piles have been measured.

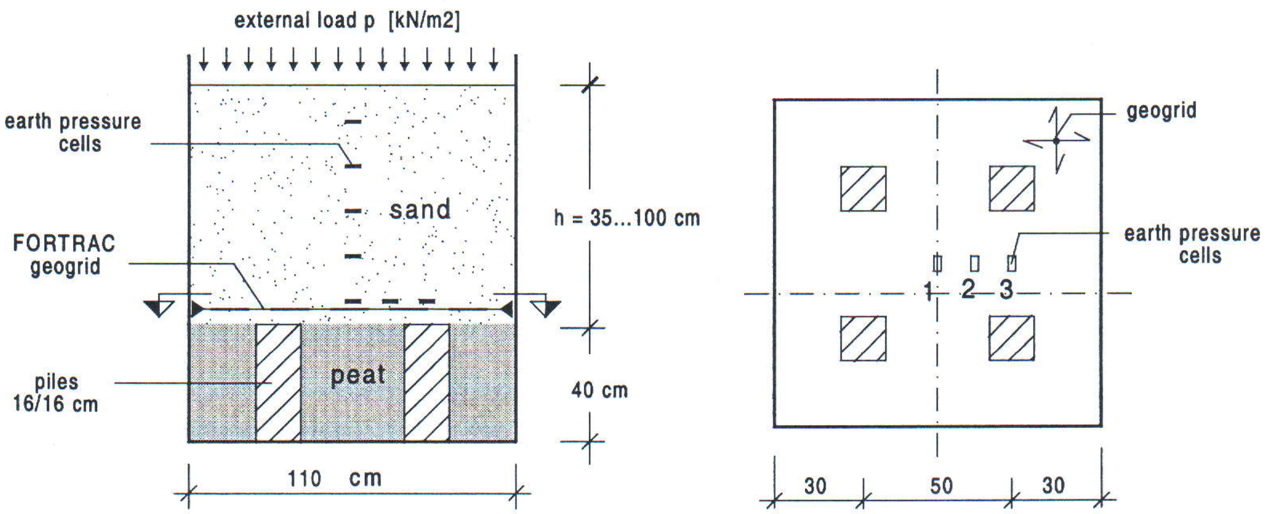
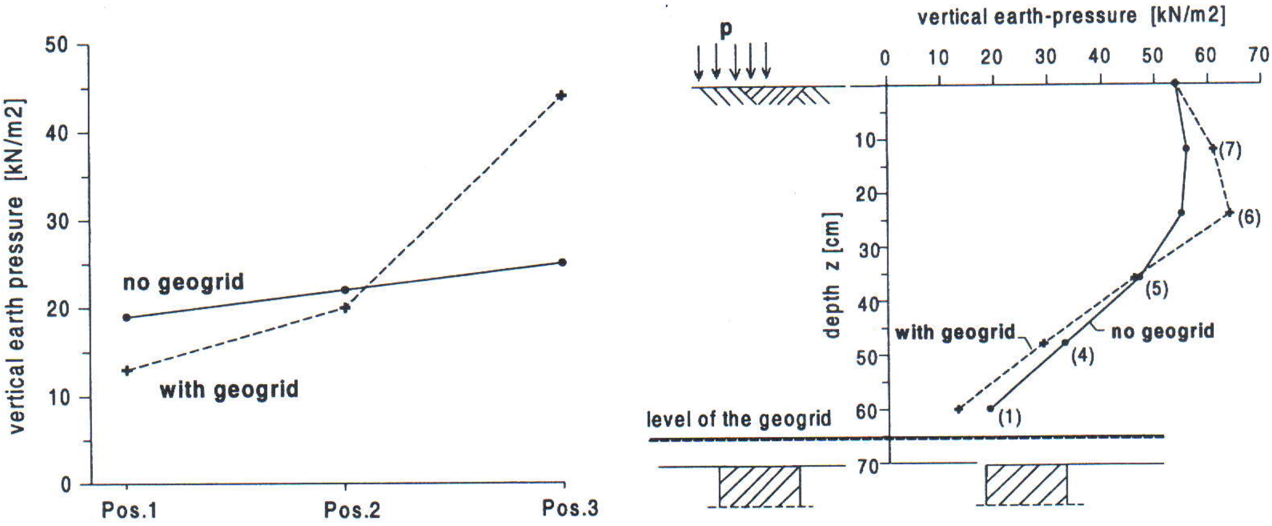


Figure 3. Model testing device

4 RESULTS OF MODEL TESTS UNDER STATIC LOAD

The interaction of stress in the embankment and the tension in the geosynthetic reinforcement can be displayed by the pressure measurements in the fill and the deformation measurements of the geogrid. The stronger the reinforcement, the more significant the stress redistribution. The tendency is depicted in Figure 4a (for pressure cell positions see Fig. 3). Without reinforcement the vertical stress is nearly constant in all zones between the piles, a very significant load redistribution occurs due to geogrid.



(a) (b)

Figure 4. Vertical pressure in the plane just above peat (a) and in the center-line of the system (b) for a static surcharge of $p = 54$ kN/m^2

The data of the strain gauges on the geogrid confirm the high stress level in the zone overspanning neighboring piles. Figure 5 is a graph of measured strains parallel to the Y - axis (it is not a picture of the deformed grid), showing clearly higher strains (say: tensile forces) on the "short axis" between piles.

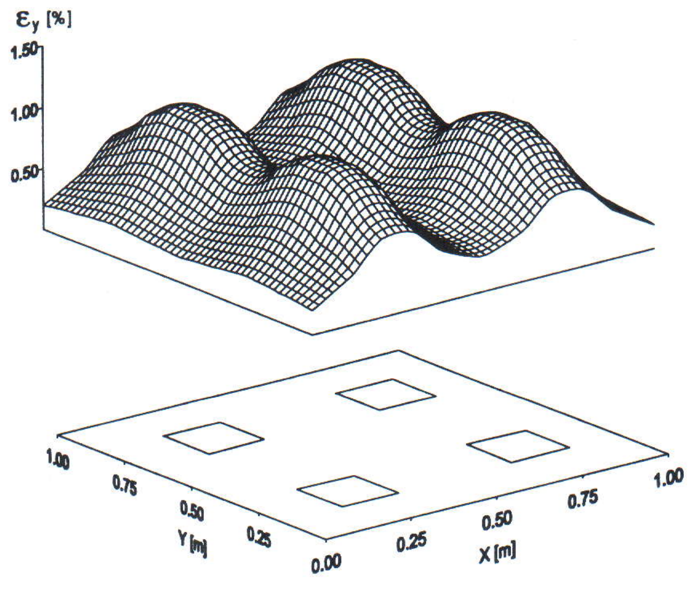
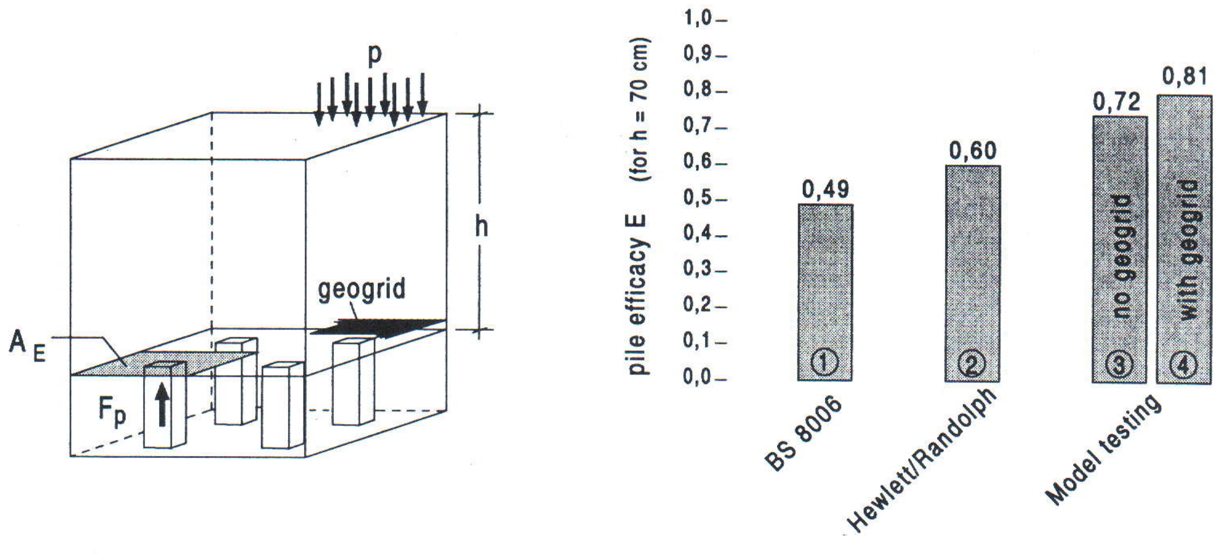


Figure 5. Geogrid strains in Y-direction

Due to the axial symmetry of the system and the biaxial (orthotropic) geogrid the strains parallel to the X-axis should be the same. It is evident, that the strain (stress) in reinforcement varies in a wide range, corresponding to the registered vertical stress variation in the "reinforced cases", and to different peat counterpressures also. Note, that in the analytical procedures used today the vertical pressure σ_s (Fig. 2a) is set to be constant. The combination with the additional assumption, that this constant σ_s is applied on the entire "influence area" (Fig. 2b) results in dimensioning on the safer side. Significant stress redistribution has been registered in vertical direction also. Figure 4b depicts the distribution of vertical pressure in the center-line of the model for a typical case. It becomes clear that a 3D-arching according to the simplified model of Hewlett & Randolph (1988) (Fig. 1) doesn't take place. The stress redistribution zone in the embankment due to mobilized shear stresses covers a large part of embankment's height, and not only a space inside a 3D-arching. This fact is even more clearly demonstrated in the case of 35 cm embankment height (not shown), where stress redistribution starts just below sand surface; the zone with this specific stress-field is higher than assumed by the semispherical 3D-shell. The test results can be analyzed from the point of view of the so called "pile efficacy" (or stress redistribution ratio), defined as (Fig. 6a):

$$E = \frac{F_P}{F_{AE}} = \frac{F_P}{(\gamma \cdot h + p) \cdot A_E} \qquad (1)$$

(1) say, what part of the total load is born by the piles. The evaluation of data of the pressure cells below the piles shows, that E is relatively independent of applied external load. Figure 6b shows a comparison of E registered in the tests under static loading conditions and E calculated according to two common methods mentioned above.



(a) (b)

Figure 6. Stress redistribution ratio for static load ("efficacy of pile support"): calculated and registered

Note: column 4 in Figure 6b includes the membrane bearing action of reinforcement, transferring more loads to the piles. Strictly speaking, this is the "reinforcement + pile" efficacy.

Additionally some tests under cyclic load were performed at frequencies being critical for the sand used. Remarkable less favorable stress redistribution was registered resulting in higher geogrid tension. For thin embankments the reinforcement alone prevented the system from punching. Obviously, attention has to be paid in such cases, say higher geogrid strengths are required. Tests are ongoing, results will be reported separately.

5 CONCLUSIONS

Field measurements, FEM analysis (not discussed herein) and the static model tests described herein lead so far generally to the conclusion, that in case of static loads and mobilised soft subsoil counterpressure the simplified analytical procedures commonly used today don't reproduce the stress redistribution effects precisely enough and tend to overestimate required reinforcement forces. For cyclic loads preliminary results are definitely less favourable. Further research is needed for better understanding of the interaction embankment - "membrane" reinforcement-piles-soft subsoil, especially from the point of view of serviceability. All registered data until now refer to cases in which soft soil embedment (counterpressure) was available and mobilised. On a long-term basis it could be not the case.

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