Causes for the Improved Bearing Behaviour of the Reinforced Two-Layer System

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ABSTRACT: The improvement of the bearing and deformation behaviour by means of a geosynthetic reinforcement placed at the base of a layer of granular fill on the surface of soft clay (called reinforced two-layer system) is examined. Therefore site tests (scale 1:1) have been performed with variations in the manner of the applied geosynthetics (wovens, non-wovens and geogrids) and the thicknesses of the granular fill. The tests were performed until reaching the bearing capacity of the systems. For the reinforced systems increases between 52 % and 108 %, compared to the unreinforced system, were obtained. Subsequently nonlinearelastic-viscoplastic calculations, based on the Finite Element Method (FEM), were performed to evaluate the quality of the received prognoses by comparison with the results of the site tests. Using material parameters exclusively from laboratory tests (no back-calculation analysis) a very good accordance of the FEM calculations with the site tests could be established.

1 INTRODUCTION

The improvement of the bearing and deformation behaviour by means of a geosynthetic reinforcement placed at the base of a layer of granular fill on the surface of soft clay (called reinforced two-layer system) is well known and often used. Nevertheless, the main reasons for the soil mechanical effectiveness of the reinforcement are not exactly known.

Therefore site tests (scale 1:1) have been performed with variations in the manner of the applied geosynthetics (wovens, non-wovens and geogrids) and the thicknesses of the granular fill. By means of an extensive measuring program the main reasons could be found which are responsible for the better bearing and deformation behaviour by placing the reinforcement between the two layers. Subsequently nonlinearelastic-viscoplastic calculations, based on the Finite Element (FE) Method, were performed to evaluate the quality of the received prognoses by comparison with the results of the site tests. Using material parameters exclusively from laboratory tests a very good accordance of the FEM calculations with the site tests could be established.

Based on the site tests and on the numeric calculations, the decisive causes for the better performance of reinforced systems were obtained.

2 MATERIAL PARAMETERS

The site tests and FE-calculations were carried out using three different reinforcements and one kind of granular layer, which was built up in two different thicknesses on the subsoil.

2.1 Subsoil

The field trials were performed in Prien/Chiemsee. The subgrade soil was clay with basic properties presented in Table 1.

Table 1: Material properties of the subsoil

Property	Dimension	Value
Water content w	%	108.3 - 113.5
Plastic limit w_P	%	44.3 - 47.2
Liquid limit w_L	%	121.2 - 126.7
$\varnothing \leq 0.002$ mm	%	52.2 - 54.6
$0.002 < \varnothing \leq 0.06$ mm	%	42.3 - 46.4
$\varnothing > 0.06$ mm	%	≤ 2.5
Undrained cohesion c_u [1]	kPa	11.4
Deformation modulus E [1]	MPa	0.48 - 0.67 [2]
Poisson´s ratio ν [1]	-	0.44 - 0.48 [2]
Hydraulic conductivity k	m/s	$1\cdot10^{-9} - 1\cdot10^{-11}$

[1] from special triaxial test [2] variable with stress level

2.2 Granular layer

For the granular layer a broken sand-gravel-mixture was used (see Table 2). The mechanical parameters were estimated by means of special triaxial tests (described in GOLD (1993)). Especially remarkable is the variability of the shear parameters with the applied stress level. At low lateral pressures high friction angles were estimated which decrease with growing stress level. The value of the cohesion increases with the stress level.

Table 2: Material properties of the subsoil

Property	Dimension	Value
Water content w	%	7.5
Dry density (proctor) $\rho_{d,Pr}$	%	2.23
$\varnothing \leq 0.06$ mm	%	7.1 - 7.9
$0.06 < \varnothing \leq 2.0$ mm	%	22.1 - 23.6
$\varnothing > 2.0$ mm	%	68.5 - 70.8
Friction angle ϕ' [1]	°	41.5 - 51.0[2]
Cohesion c' [1]	kPa	24.0 - 88.0[2]
Deformation modulus E[1]	MPa	50.0 - 220.0[2]
Poisson´s ratio ν[1]	-	0.48

[1] from special triaxial test [2] variable with stress level

2.3 The reinforcements

To get the influence of different geosynthetic products the field trials and also the calculations were carried out using wovens, nonwovens and grids. Some essential material properties are given in Table 3 and Figure 1.

Table 3: Material properties for the reinforcements

Property	Dim.	Woven	Nonwoven	Grid
Mass/unit	g/m²	460	1200	300
Tensile strength	kN/m	220	52	25
Tensile strength (2 % elongation)	kN/m	25.8	0.8	3.0
Strain at max. el.	%	17	65	17

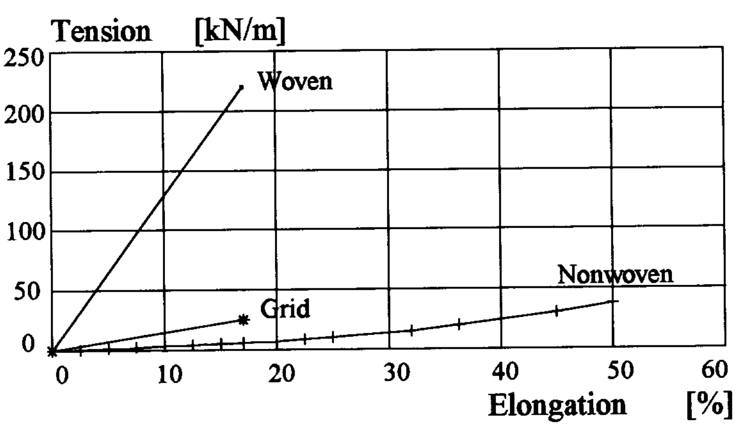

Figure 1 Tension-elongation-behaviour of the geosynthetics

3 SITE TESTS

The trials were carried out in-situ with homogenous clays for the subgrade. The granular layer was built up with heights of 0.15 m and 0.3 m to get the influence of the thickness. Three geosynthetics were used. Including the unreinforced case eight different two-layer systems were examined. The load steps were applied until reaching the bearing capacity of the systems.

Because of the low permeability of the subgrade, it was not possible to apply the loads in such time steps that drained conditions were reached in the subgrade. To have clear conditions concerning the stress situation in the subgrade, the trials were carried out quickly (about one hour for each test), so that perfectly undrained conditions (c_u-case) were obtained.

For the interpretation of the tests and for the analytical calculations it was also necessary to realize a clearly defined stress and strain situation. That means, we didn´t want to have a common three-dimensional stress and strain state, but a plane strain state. For this reason a new loading system was developed (see Figure 2). The loading beam consists of three parts. The two side parts are moved in the same way during the tests as the middle (main) beam, so that for the middle beam a plane strain situation is realized.

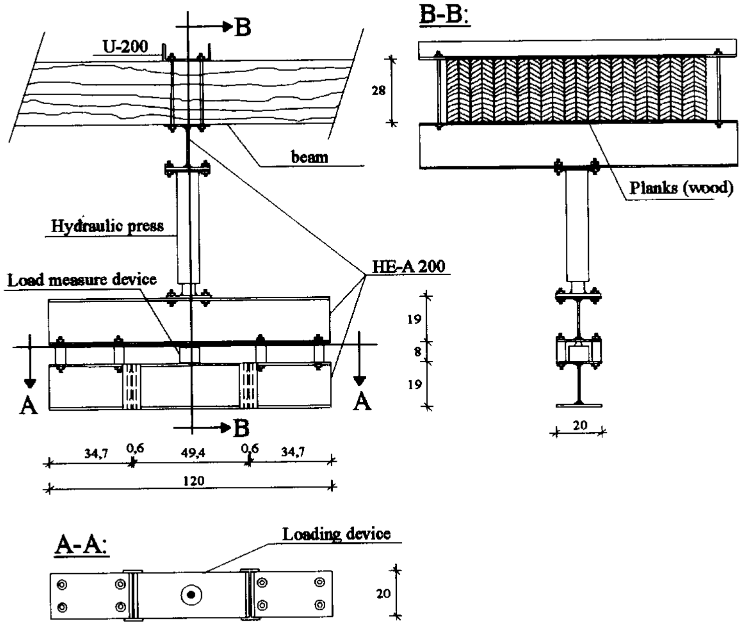

Figure 2 Loading device

Together with the results from Finite Element analysis the site test results are presented in section 5.

4 FINITE ELEMENT CALCULATIONS

The soil continuum is modelled by eight node, isoparametric elements with quadratic shape functions. For the geosynthetics isoparametric bare elements were used. To model the interaction, especially the potential for relative movement and the allowable shear forces between the reinforcements and the soil, thin-layer joint elements were used. As their shear modulus can be defined independently of the Young´s modulus, it is possible to get movements of the soil elements relative to the elements of the reinforcement by large shear distortion of the thin-layer joint elements. Those were additionally supplied with a joint parallel to the reinforcement, to limit the transfer of forces from the soil into the reinforcement by a yield criterion (here: Mohr-Coulomb). The Young´s modulus was taken from the surrounding soil. Shear modulus and friction parameters between reinforcement and soil are determined by pull-out tests and direct shear tests.

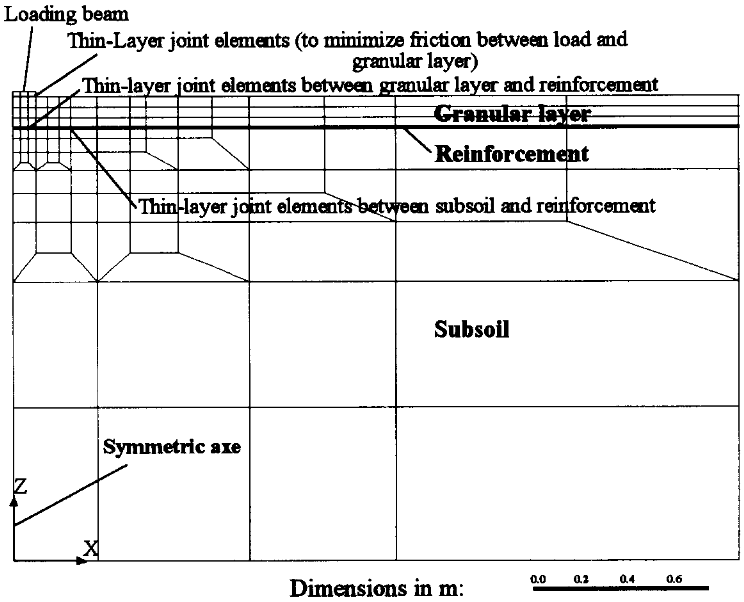

Figure 3 Reinforced two-layer system (FE-model)

Carrying out laboratory tests, it was determined, that the material parameters of the subsoil and especially of the granular layer were variable with the stress level. Therefore it was necessary to extend the existing FE program (MISES3, TDV Austria) to consider the invariability of the material parameter.

The side tests and also the FE calculations were performed until reaching the maximum system load. Therefore it was necessary to consider the geometrical nonlinearity in the calculations. Working in load increments the program code was transformed in a way, that the physical (elastic and shear parameters) and geometrical nonlinearity were taken into account.

To calculate the single load increments the elastic-viscoplastic algorithm was used. Considering the variation of the material parameters between the load increments, our stress-strain relation was nonlinearelastic-viscoplastic.

It should be pointed out that the following presented analytical results are based exclusively on material parameter, determined in our laboratory. A so-called back-calculation analysis was not carried out.

5 RESULTS

Figure 4 shows the measured and calculated deformations in the centre of the load for a two-layer system with a granular layer thickness of 0.15 m. The different kinds of geosynthetics and their deformation reduction is obvious. Beyond that a very good accordance between analyses and test results was determinable.

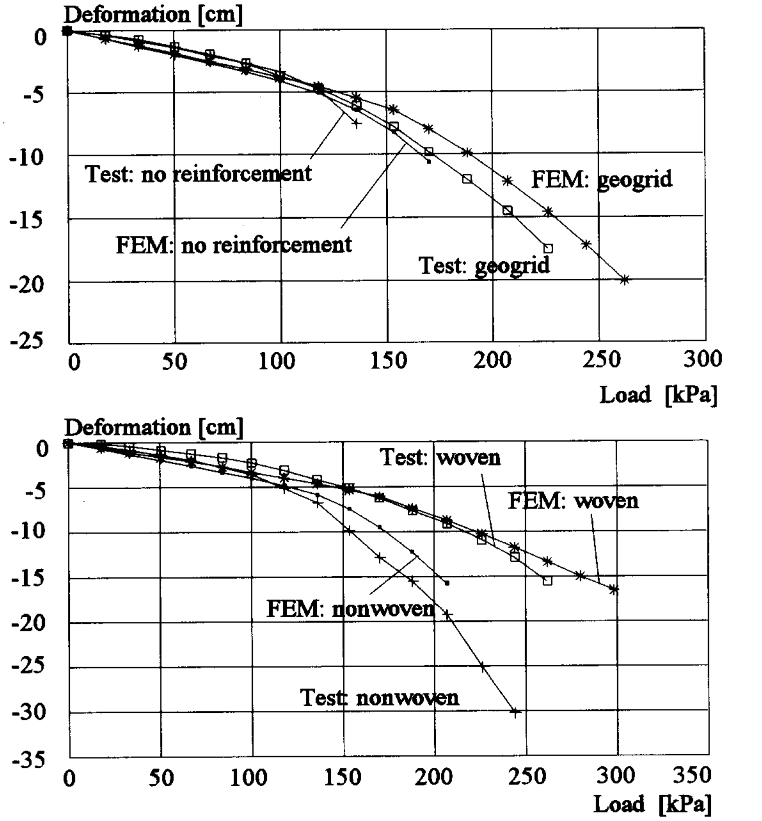

Figure 4 Load-deformation data for systems with 0.15 m granular layer

In Figure 5 the first stress invariant and in Figure 6 the second invariant of the stress deviator in the subgrade immediately underneath the reinforcement for the systems with a granular layer thickness of 0.15 m are presented, as they were estimated by the FE calculations.

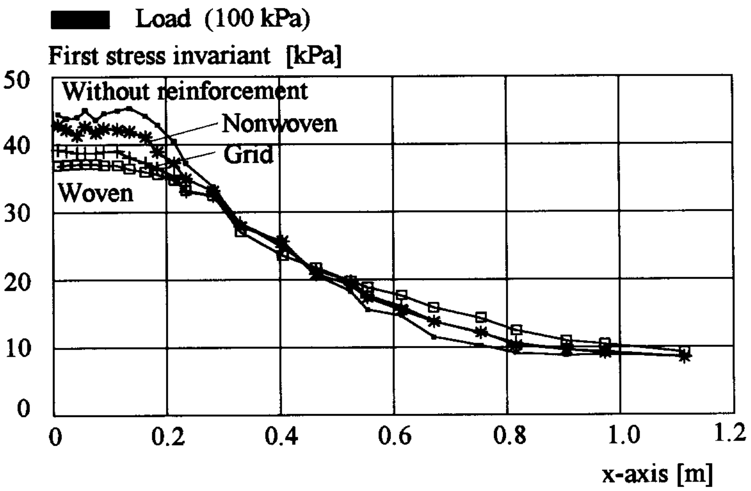

Figure 5 First stress invariants underneath the reinforcement in the subgrade (0.15 m granular layer)

The first stress invariant identifies the hydrostatic stress state. The second invariant of the stress deviator characterises the deviator stresses. For the woven reinforced system the smallest hydrostatic stresses are calculated. But the difference from the unreinforced system is only about 20 %. For the second invariant the factor 2 is determined between unreinforced and woven reinforced systems. At equal loading the deviations from the hydrostatic stress state are essentially smaller for the reinforced system compared with the unreinforced system. This is one of the main reasons for the better load and deformation characteristics of the reinforced systems.

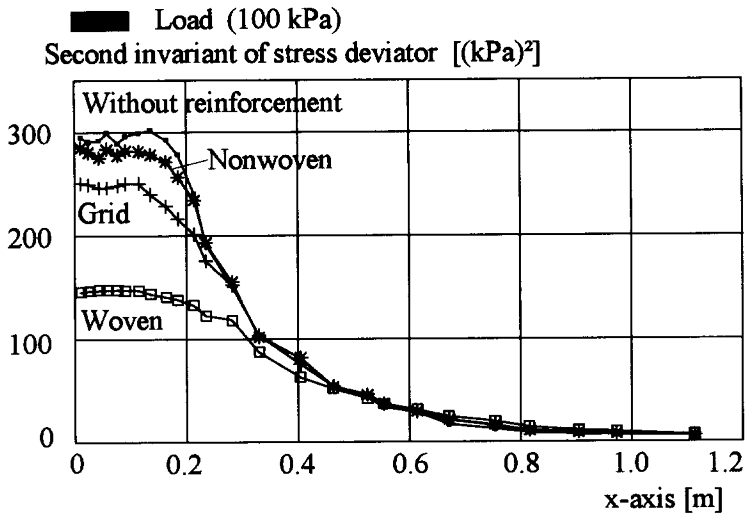

Figure 6 Second invariant of stress deviator underneath the reinforcement in the subgrade (0.15 m granular layer)

Figure 7 shows the calculated (FEM) and measured (field trials) limit loads. For the reinforced systems the load carrying prognosis was very good, especially for the thicker granular layer. The FEM was not able to model the main failure cause for unreinforced systems, which was the punching procedure of the granular layer. Therefore, the load capacities of unreinforced systems have been overestimated for 12 % to 22 %.

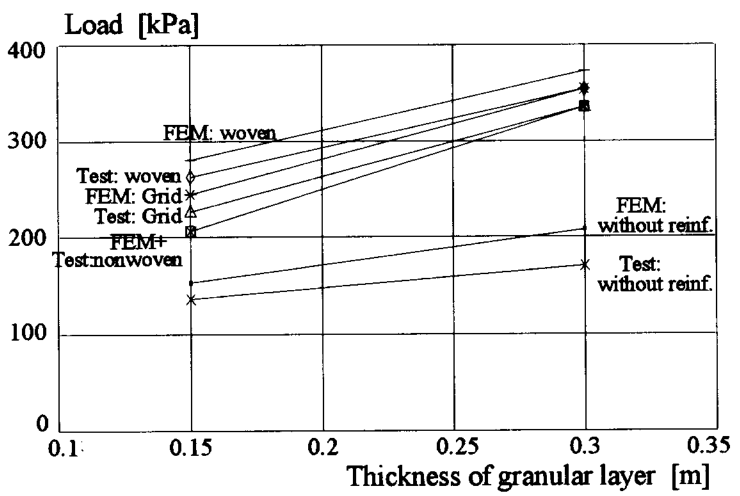

Figure 7 Limit loads by filed trials and FE calculations

6 CONCLUSIONS

With some soil mechanical extensions of the FE-program system the calculations showed a good agreement with the site tests. A very good prognosis could be achieved for the bearing capacity of reinforced systems, the maximum deformation and the maximum strains and stresses of the reinforcement. The load capacities of unreinforced systems can well be determined with the Kinematical Element Method, because it is possible to model the mentioned main failure cause.

The relevant reasons for the improvement of the bearing and deformation characteristics of the reinforced two-layer system in comparison with the unreinforced system can be summarised as follows:

- With reinforcement the granular layer is able to transmit shear forces on an essentially higher level without collapsing. The function of (better) load spreading is maintained also on a high deformation level.
- The reinforcement effects a tendentious reduction of the maximal hydrostatic stress level (= first invariant of the stress tensor), and a considerable reduction (up to 50 % referred to the unreinforced system) of the deviatoric stress level (= second invariant of the stress deviator) immediately underneath the reinforcement, in the subgrade (see Figure 6). The reason therefore consists in the fact that the shear forces are transferred from the granular layer into the reinforcement. This results in a more favourable stress situation for the subgrade material in the reinforced as in the unreinforced case. Higher loading capacities are therefore given for reinforced systems because bigger yielding geometries are calculated.
- The membrane effect has only a subordinate influence on the increase of the load capacity of a reinforced system. Merely very stiff reinforcements and large system deformations cause an not negligible influence on the system load capacity resulting from the membrane effect.

7 REFERENCES

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