PERFORMANCE OF AGGREGATES IN GEOGRID-REINFORCED SOILS USED FOR PROTECTION AGAINST SURFACE COLLAPSE INTO UNDERGROUND VOIDS

S. Schwerdt
Anhalt University of Applied Sciences, Dessau, Germany
O. Naciri
Tensar International GmbH, Bonn, Germany
C.G. Jenner
Tensar Limited, Blackburn, UK

ABSTRACT: Geosynthetics have been used increasingly to protect traffic routes constructed over subsidence-prone areas from collapse. A common design practice for a geogrid reinforcement is the use of the practical code BS 8006 which is based on the membrane theory. This theory considers the soil above the geosynthetic as being a load only. The supporting effect of the overlying soil and the interaction between soil and geogrid are not taken into account. Anhalt University of Applied Sciences Dessau, Germany carried out major tests which consisted of the bridging of subsidence by using a single layer and multiple layers of a stretched and stiff geogrid. This paper describes the test results and compares them with the results from numerical calculations.

1 INTRODUCTION

The use of geosynthetics to reinforce the soil in order to protect traffic routes in areas prone to subsidence has become state-of-the-art technology. A number of publications have reported on the experiences of this method of construction and on the design methods used (eg Lawson et. Al., 1994; Gourc, Villard, 2000; Schwerdt et al., 2003).

The voids are mostly bridged by using geosynthetics with a high tensile strength and a high short-term tensile strength. Depending on the situation single layer or multiple layers of geosynthetics are installed. The load-bearing performance is based on the membrane theory (BSI 1995; Giroud et al., 1990).

This theory considers the soil above the geosynthetic as being a load only and any supporting effect of the overlying soil is not taken into account. Furthermore this method does not consider the fact that the ribs of stiff geogrids provide a bearing surface for the interlocking fill particles.

Tensar International Germany asked Anhalt University of Applied Sciences, Dessau, Germany to carry out major tests which consisted of the bridging of subsidence by using a single layer or multiple layers of a stretched and stiff geogrid (Tensar® SS 30 and Tensar® SS 40). This paper describes the test results and compares them with the results from numerical calculations.

2 MAJOR SUBSIDENCE TEST

2.1 *Test program*

Several major subsidence tests were carried out in which the number of geogrid layers, the dimension of the void and the height of the upper soil varied. Table 1 summarises the major subsidence tests carried out.

Table 1 Major tests carried out

Trial	1	2	3
Number of geogrid layers	1	1	2
Quality control strength of the geogrid T_{ult} [kN/m]	30	40	30+40
Height above the lower geogrid layer [m]	0,70	1,60	1,60
Void diameter [m]	1,68	1,68	2,00

2.2 *Running the tests*

The major subsidence tests were carried out in the test pit of Anhalt University of Applied Sciences in Dessau. In trial 1 and 2 a circular 1,68 m diameter subsidence was formed and spanned with a single geogrid layer. In trial 3 a 2 m diameter void was originally planned. However, due to the arching effect only a rectangular void of a x b = 1,2 x 1,6 m was formed beneath the lowest geogrid layer.(paragraph 2.3.2).

The arrangement of the trial is shown on Figure 1 as an example of the double-layer geogrid reinforcement (Trial 3). The load applied on the geogrid consisted of the self weight of the overlying soil layers and of variable static and dynamic loads.

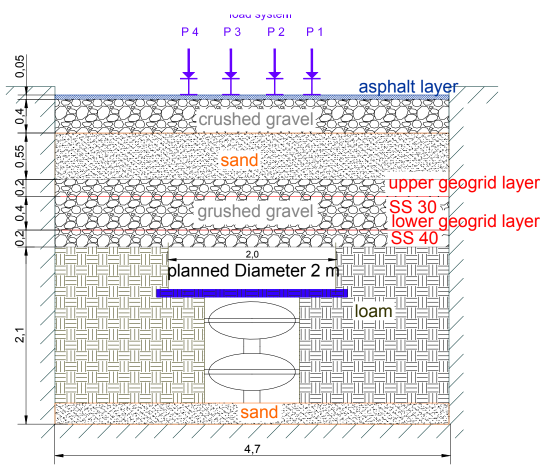

Figure 1 Longitudinal section of the trial arrangement with double-layer geogrid reinforcement.

In this trial the geogrid layers were embedded in a layer of 80 cm thick crushed gravel (0/32 mm). A 0.55m thick sand (0/8) layer and a 0.4m thick crushed gravel (0/32) layer were installed above the reinforced gravel layer. The top course was a 5 cm thick asphalt layer which was chosen to be thin in order to exclude an arching effect with the walls.

For trial 1 the geogrid was placed into a 75 cm thick reinforced layer, 10 cm above the cohesive soil. A 5 cm thick asphalt layer was placed on top of it. For trial 2 the reinforced layer was 50 cm thick. A sand layer (75 cm) and a crushed gravel layer (40 cm) and an asphalt layer (5 cm) were placed on top.

The geogrids used were stretched geogrids of polypropylene with a short-term tensile strength (quality control strength) T_{ult} = 30 kN/m (SS 30) and T_{ult} = 40 kN/m (SS 40).

The variable load was applied using four test cylinders which were arranged behind each other. The dynamic load was applied with a time delay between the test cylinders. The maximum force per test cylinder was 50 kN. With overlapping and a time delay this arrangement was able to simulate a rolling load which was equivalent to a lorry twin-wheel travelling at a speed of 60 kph. During a period of two weeks the dynamic load was applied more than 300.000 times for trial 2 and 3, and after that the load was increased. In trial 1 the planned number of loadings was not achieved because failure occurred after only 2.500 passes.

Extensive measuring equipment measured the most important values such as the depressions of the geogrid layers, the soil layers and the pavement, the strains of the geogrid and also the vertical stresses in the different layers. Figure 2 shows the measuring instruments placed on the geogrid.

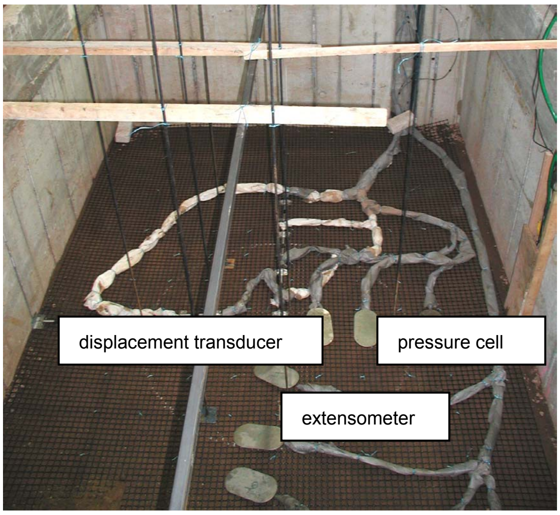

Figure 2 Geogrid and measuring instruments

2.3 *Test results*

2.3.1 *Trials with a single layer geogrid reinforcement*

During trial 1, with only 0.70 m of soil above the geogrid, a large increase of the strain in the geogrid occurred after only about 500 passes. After about 2500 loadings the limiting stress of the material was reached and it failed. The depressions of the geogrid increased in the same way and reached shortly before failure its maximum value. The reason for the collapse is the small thickness of the soil layer, 0.70 m ,above the geogrid. The top of the load-bearing arch in the soil layers reached the surface of the pavement, i.e. the load-bearing arch in this trial was unstable.

In trial 2, with 1.6 m of soil above the geogrid, the load-bearing arch remained constant during the trial procedure as planned. The maximum strains of the geogrids were ε = 1.0 % after the end of the trial and at this time the depressions were 13.2 cm. The measured vertical stresses on the geogrid right above the void were clearly smaller than the calculated values based on own load and traffic load (see figure 3).

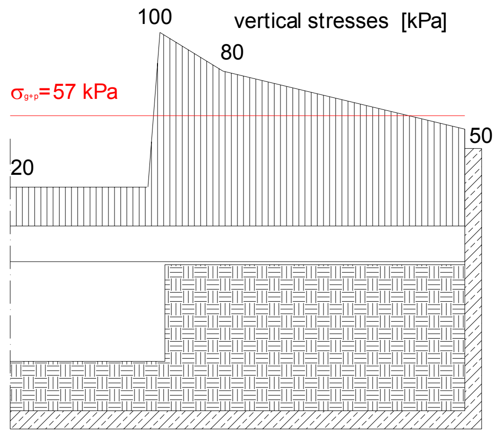

Figure 3 Vertical stresses after the end of the traffic load as planned

During the additional load phase with the value of the planned traffic load doubled the arch´s vertex was rapidly climbing up. After 3600 passages the geogrid failed.

Both tests with stiff geogrid reinforcement have shown, that an protection of the cavity is possible with one layer of an rigid geogrid if the height of the capping layer above is sufficient. This height has to be chosen according to the size of the cavity, the selected geogrid and the traffic magnitude.

2.3.2 *Trials with double-layer geogrid reinforcement*

In the trial with a double-layer geogrid reinforcement the load was increased in several stages. In the first loading stage the planned loading of 50 kN per test cylinder was repeated 300,000 times. After that the load was increased up to a maximum of 120 kN per test cylinder which is equivalent to 2.4 times of the load planned. During this additional loading phase 150,000 load repetitions were applied. The total amount of load repetitions was 450,000. Figure 4 shows the exposed geogrid above the subsidence after the end of the test no cracking or damage could be found.

Figure 4 Exposed geogrid reinforcement after end of the test

Only small strains were measured in both geogrid layers during the planned loading period. The strain reached a maximum of 0.15 % in the lower geogrid layer and 0.1 % in the upper geogrid layer. During the additional loading period the strains increased to 0.45% in the lower and 0.2 % in the upper geogrid layer. These very low strains are also confirmed by the measurement of the depression. After completion of the planned trial the maximum depression was 0.8 cm and after the additional loading phase the measured value was 1.4 cm. In the overlying layers the depressions are clearly smaller. The depressions after the completion of the additional loading can be seen in figure 5.

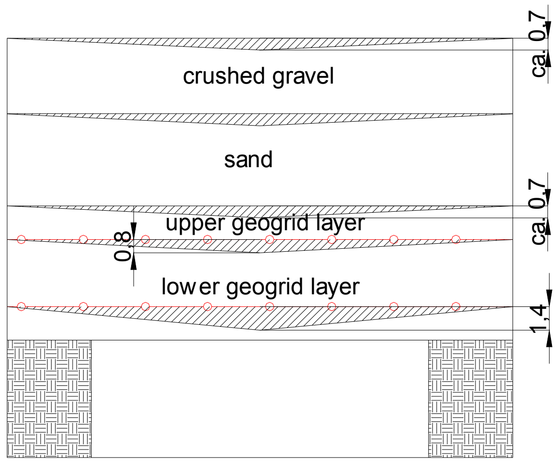

Figure 5 Depressions of the geogrid layers and of the overlying measuring planes [cm]

The strains measured show that a load-bearing arch was formed above the subsidence. Typically the vertical strains in this area are clearly smaller than those next to the void.

After the completion of the trial and the removal of the overlying soil layers it was found that the void beneath the geogrid was smaller than originally planned. This was due to the formation of the load-bearing arch which also reached beneath the lower geogrid layer (See figure 6).

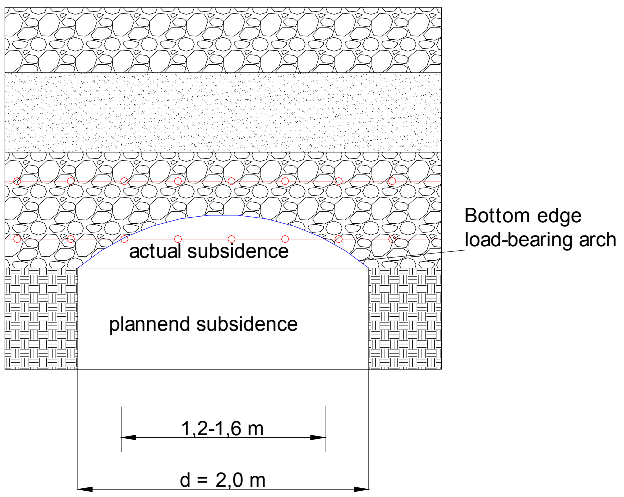

Figure 6 Actual void and assumed bottom edge of the load-bearing arch

2.3.3 *Summary of the test results*
The major subsidence tests show that it is possible to span voids using a single layer of stiff and stretched geogrids. However, the relationship between the height of the overlying soil to the void diameter has to sufficiently large. The use of multiple layers of stiff geogrids to span over voids results in a stable load-bearing arch which starts from beneath the lower geogrid layers. The use of multiple layers of stiff geogrids to bridge voids results in the formation of a stable load-bearing arch which already begins to develop beneath the geogrid layers. The interlocking mechanism between the granular fill and the geogrid layers creates an effective load distribution. The high bearing capacity of this construction could be seen clearly even with an enormous increase of the traffic load when this did not cause considerable depressions of the pavement surface. Also the strains of the geogrid were far below the limit strain value.

The trials show that the interlock between the multiple layers of stiff geogrids and the granular fills ensure a safe bridging of voids.

3 NUMERICAL CALCULATIONS

3.1 *Calculation models*

The numerical calculations were carried out with the calculation programs Plaxis® and Sofistik®. The program Plaxis® analyses plane stress states and three-dimensional deformation states, whereas the program Sofistik® considers three-dimensional stress and deformation states.

3.1.1 *Calculations with the program Plaxis®*
The calculations were made for the plane stress state and a vertical section was selected through the test pit. The subsidence was considered to be limited on two sides with subsidence Perpendicular to the selected plane being unlimited. This corresponds to the plane state of the bridging of clefts when the presented analytical calculation methods are used. The strain stiffness of the geosynthetic was chosen to be E^*A = 800 kN/m for the lower geogrid layer and E^*A = 600 kN/m for the upper geogrid layer. The soil layers were modelled according to the Mohr-Coulomb modeland the asphalt bearing course was regarded as a linear-elastic layer. An embedding layer was modelled around the soil layers. This layer had the shear parameters found in the friction test between the granular material and the geogrids. Figure 7 shows the different layers in the model.

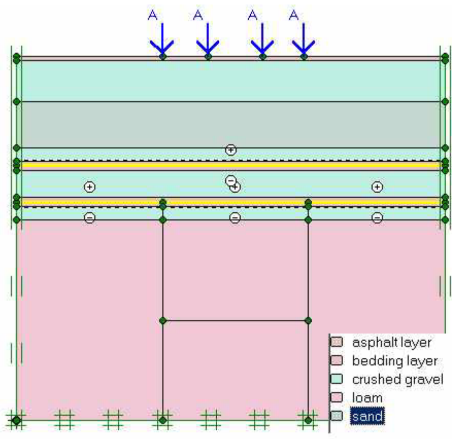

Figure 7 Model of the layers for the calculations with Plaxis®

3.1.2 *Calculations with the program Sofistik®*
The section forces and element values are calculated with the ASE modulus of the program. The geosynthetics can be simulated by the use of a special element type "membrane". It is also possible to model the geogrids by the use of truss elements and cables. Typical features of the membrane element:

- only membrane section forces are taken
- any distortions and strains are possible
- it takes large torsion forces and distributes the resultant membrane forces in the correct direction, i.e. forces develop perpendicular to the imaginary middle area of the element
- a prestress (even orthotropic) can be defined
- the anisotropic tensile strength-strain behaviour is considered

Node, linear and area loads can be chosen. All loads can be defined independently of the chosen element. Non-linear calculations can be carried out in order to consider the failure of individual elements (cable under pressure, rising of bedded plates) and to include crack, friction and flow effects of spring and bedding elements..

The test pit was simulated with the dimensions a = 4.7 m, b = 3.0 m and d = 4.0 m. The axis of the potential subsidence was in the middle of the pit and the whole area was overlain with membrane elements which modelled the geogrid reinforcement. The soil layers were modelled with volume elements.

The calculations were based on a three-dimensional stress and deformation state. This means that the system automatically calculates the vertical stresses on the geogrid layers from the loads.

The geogrid was considered as linear-elastic. The determination of the elasticity modulus was based on the Hook model. Figure 8 shows the calculation model.

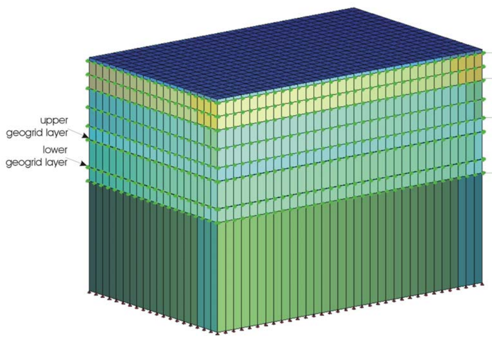

Figure 8 Model of the calculations with Sofistik®

3.2 *Calculation results*

3.2.1 *Calculations with Plaxis®*
The finite element program Plaxis® analyses the occurring stress, forces and deformations for the plane case in a vertical section. Due to the void a load transfer takes place between the soil layers above the subsidence and the lateral zones. The formation of a shell-shaped load-bearing arch above the void can be seen clearly in figure 9. The vertical stress beneath the load-bearing arch decrease clearly. However, at the edge of the subsidence a concentration of the stress develops.

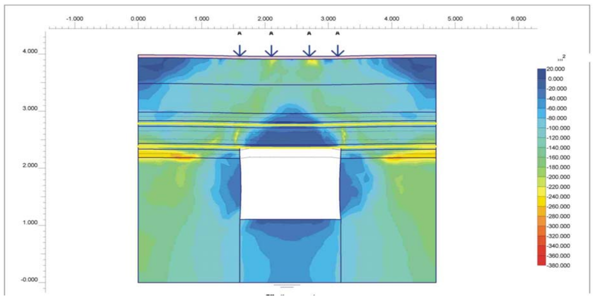

Figure 9 Calculated stress distribution using Plaxis® (additional test)

The calculated depressions of the surface and the strengths of the geogrid-layers can be taken from the table 2.

Table 2 summarises the calculated depressions and the tensile strength of the geogrid layers and on the surface.

	planned loading		additional loading	
	trial	calculation	trial	calculation
d_s [cm]	< 1.0	1.2	1.4	3.8
upper geogrid layer				
d [cm]	0.9	1.3	0.8	4.0
F_{md} [kN/m]	0.5	0.6	1.1	1.5
lower geogrid layer				
d [cm]	1.2	1.3	1.4	4.1
F_{md} [kN/m]	1.2	1.4	3.4	3.2

3.2.2 *Calculations with Sofistik®*
The individual loads of the traffic loads were converted into area loads. The calculations were carried out for both the planned and increased traffic load (additional load). Table 3 shows the calculation results.

Table 3 Comparison of the trial results to the calculation results with Sofistik®

	planned loading		additional loading	
	trial	calculation	trial	calculation
d_s [cm]	< 1.0	0.6	1.4	1.2
upper geogrid layer				
d [cm]	0.9	0.6	0.8	0.8
F_{md} [kN/m]	0.5	2.3	1.1	3.3
F_{cmd} [kN/m]	--	1,4	--	2.6
lower geogrid layer				
d [cm]	1.2	0.6	1.4	0.8
F_{md} [kN/m]	1.2	0.9	3.4	4.9
F_{cmd} [kN/m]	--	0.75	--	4.8

3.2.3 *Summary of the calculation results using numerical methods*
There is a good correlation between the results of the numerical calculations and the measured tensile load results of the trials. The differences regarding the depressions are bigger.
The comparison between the results shows that the program Plaxis® provides sufficiently accurate results for the design calculations of isotropic geosynthetics. However, under anisotropic conditions numerical calculations need to be carried out using programs which consider three-

dimensional stress and deformation states. The program Sofistik® is one example.

4 SUMMARY AND CONCLUSIONS

The results of three major subsidence tests with stiff and stretched geogrids were presented. The trials showed that a stable and safe bridging of a void can be achieved, if the soil layer above the geogrid is sufficiently thick so that a stable load-bearing arch can be formed. The bearing capacity of a double-layer reinforcement of stiff geogrids embedded in a layer of crushed gravel was particularly good.

The following numerical calculations confirmed the test results. It was shown that the often used numerical programs, i.e. Plaxis® and Sofistik®, prove a good correspondence between test and calculation results.

5 REFERENCES

BSI (1995): BS 8006 Code of practice for reinforced / strengthened Soils and other fills, British Standard Institution

DGGT (1993): EVB – Empfehlungen für Verformungen von baulichen Anlagen, Deutsche Gesellschaft für Geotechnik, Verlag Ernst & Sohn, Berlin, 1993

Giroud et.al. (1990): Design of Soil Layer-Geosynthetic Systems Overlying Voids; IN: Geotextiles and Geomembranes 9 (1990), S. 11-50

Lawson, C.R.; Jones, J.F.P.; Kempton, G.T.; Passaris, E.K.S. (1994): Advanced Analysis of Reinforced Fills over Areas Prone to Subsidence; In: Proc. of the Fifth International Conference on Geotextiles, Geomembranes and Related Products, Singapore, pp 311-316

Schwerdt, S.; Paul, A.; Thurm, S. (2003): Die Überbrückung von erdfall- und tagesbruchgefährdeten Bereichen beim Neubau von Straßen-Beispiele für ausgeführte Sicherungslösungen in Sachsen-Anhalt, In: Proc. 14. Nationale Tagung für Ingenieurgeologie, Kiel, DGGT, pp. 319-325