# ANALYTICAL AND NUMERICAL ANALYSES OF A REAL SCALED GEOGRID REINFORCED BRIDGE ABUTMENT LOADING TEST

**Dimiter Alexiew[1] & Oliver Detert[2]**

[1] *Huesker Synthetic GmbH. (e-mail: dalexiew@huesker.de)*
[2] *Huesker Synthetic GmbH. (e-mail: detert@huesker.de)*

**Abstract:** The paper deals with the study of a geogrid-reinforced soil (GRS) solution for bridge abutments. In the first part of the paper a full scale test is briefly presented summarizing the important parameters and results required for further theoretical analyses.

A 4.5 m high geogrid reinforced vertical soil wall was directly loaded, near the top front edge, using a reinforced concrete block and hydraulic jacks, simulating the bridge sill beam. Loading and unloading cycles were performed, where the load was increased up to 3 times the normal load for this kind of structure. Settlements and horizontal facing deformations were measured during the test.

In the second (and main) part of this paper analytical and numerical analyses of the full scale test are presented. The analytical procedures include methods commonly used worldwide e.g. Bishop, Janbu, Block Sliding etc. The numerical analyses were made using FEM with the commercially available Plaxis V8.6 program.

The analytical analyses focus on the ultimate limit state and the numerical analyses on both, serviceability and ultimate limit state. The comparisons made between analytical and numerical procedures on the one hand, and test behaviour on the other hand, will assist in gaining a better understanding of the systems behaviour and application, and for better guidance in relation to the appropriate design procedures and assumptions for heavily loaded geogrid-reinforced bridge abutments, both in regards of ultimate and serviceability limit state.

**Keywords:** back analysis, bridge abutment, design method, finite element, full-scale test, geogrid reinforcement

## INTRODUCTION

Due to their economical, technical and ecological advantages, geosynthetic-reinforced soil (GRS) walls and slopes have become a very popular and common solution. Realized projects show that there are hardly any limitations concerning height, inclination and shape. The gained experiences in the last years and the wide range of available geosynthetic reinforcements resulted in the use of the first geosynthetic reinforced earth structures as bridge abutments. Bridge abutments experience very concentrated and heavy loads and have to fulfil stringent limitations of deformation. To gain more information about the capability of reinforced earth structures as bridge abutments a real scale loading test was performed (Alexiew, 2007). Aside from the presentation of the most important obtained test results, analytical and numerical studies are conducted and presented in the following sections.

## TEST SET-UP OF THE REAL SCALE TEST

The GRS-vertical wall was constructed and tested at LGA Nuremberg. The test set-up was originally used for a research program in Germany to investigate the behaviour of integral bridges with a geogrid-reinforced earth structure behind the abutments (Pötzl & Naumann 2005a, 2005b). This set-up was slightly modified and reused for a real scale test of a directly loaded geogrid reinforced 4.5 m high vertical soil wall, simulating a bridge abutment. A heavily reinforced 1.0 m wide concrete block was used as a sill beam, which transferred the load from the hydraulic jacks onto the reinforced soil wall. This concrete block was placed only 1.0 m away from the edge of the vertical wall. The wall was reinforced by 9 layers of geogrids (Fortrac 80/30-35M) formed from PVA with an ultimate tensile strength of 80 kN/m. The layers were 5.0 m long and the spacing between the layers was 0.5 m. In the front of the wall the layers were wraparound, so the wall got a so called “soft facing”. The fill was a well graded crushed sandy gravel with an internal friction angle of about 40° to 45°, depending on the compaction grade. Various measurement devices were installed on top and in front of the wall to measure vertical deformations of the wall surface and horizontal deformations of the wall facing during the test, Figure 1. Several loading cycles were performed up to a maximum load of about 650 kN/m². For more details of the test set-up and instrumentation see Alexiew (2007).

## SOME TEST RESULTS

The performance of the GRS bridge abutment fulfilled the stringent requirements concerning the deformation behaviour and bearing capacity. Even with a poor compaction in the upper part of the wall of only $D_{pr}$ = 95%, the settlements are in the range of 5-8 mm for a loading range between 100 to 250 kN/m$^2$, the common contact pressure from a sill beam, Figure 2. The shape of the graph for the first load cycles between 100 kN/m$^2$ and 250 kN/m$^2$ suggests that a certain amount of further compaction took place during these load cycles. This allows the assumption, that with a good compaction at the beginning of the loading test the settlements would have been less. But even these settlements comply with the requirements for bridge abutments.

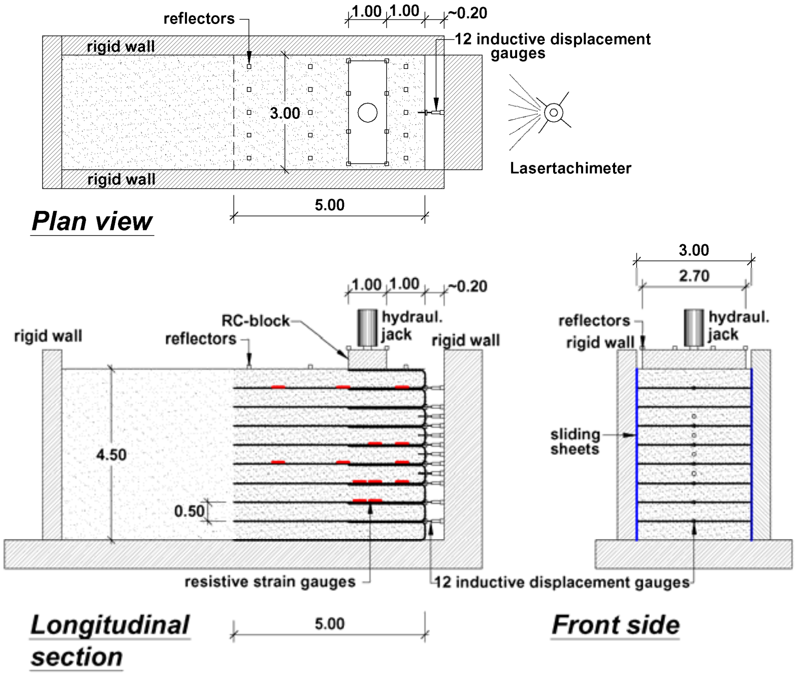


**Figure 1.** Test set-up and instrumentation of a real scaled geogrid reinforced bridge abutment loading test (Alexiew 2007)

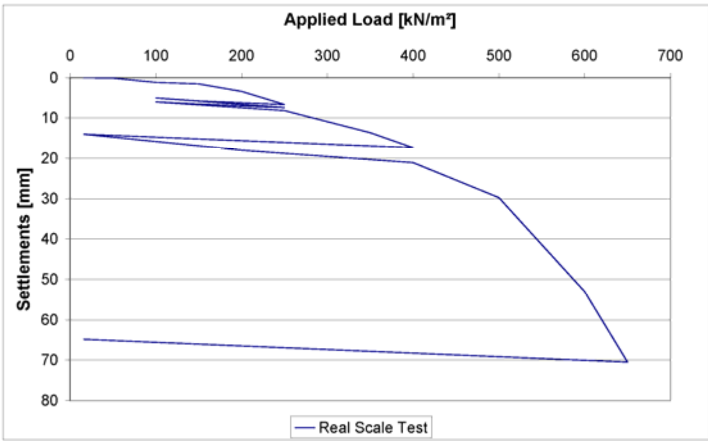


**Figure 2.** Settlement of the concrete block during the loading test

In the last loading cycle, the load was brought up to the full capacity of the hydraulic jacks of about 650 $kN/m^2$. The significant increase of settlements could be interpreted as the beginning of a possible failure mechanism. At this load level small irregular cracks appeared in the fill surface behind the concrete block and extended towards the rear along the test pit walls. A clear failure, such as a failure body of soil slipping forward and downward never occurred.

These and some further test results are analysed in the following sections with the help of analytical and numerical studies. For a more detailed and comprehensive description of the test results see Alexiew (2007).

## ANALYTICAL ANALYSES OF THE OBTAINED TEST RESULTS

Different analytical limit equilibrium design methods are used for a comparison of the obtained test results. All calculations are done with characteristic values, that means without any partial factors of safety. The ultimate tensile strength of the geogrids of 80 kN/m was reduced applying the product-specific reduction factors for installation and compaction damage and creep for 1 year, what is believed to be an acceptable assumption taking into account the pre-history of the test wall (Alexiew 2007). Therefore the design tensile strength was set to 49.7 kN/m. The internal friction angle of the soil is set to 40° and the unit weight to 18.5 $kN/m^3$. As in the real scale test the load of 650 $kN/m^2$ was applied 1.0 m from the edge of the 4.50 m high vertical reinforced soil wall.

The analyses are made with the commercial software “GGU-Stability”. The analytical methods of Bishop and of Krey (circular failure plane) and Janbu and “Block-sliding” (polygonal failure plane) are used. All analyses are performed according to the actual DIN 1054 based on partial safety factors, similar to the Eurocode EC-7, and resulting in the so called degree of ultilisation $\mu$ instead of the former global factor of safety FOS resp. $\eta$. Note, that $\mu$ is the relation of driving forces to resisting forces resp. moments, say $\mu > 1.0$ means failure, and $\mu = 1.0$ the exact equilibrium. Figure 3 shows the model of the analysed wall and the calculation result according to Bishop. The maximum degree of utilisation in the Bishop analysis is 0.97, so the system is stable but close to failure. This matches fairly good the test results from the real scale test, where a significant increase of settlements and facing bulging was observed under this load conditions, obviously approaching failure (but without reaching a clear failure).

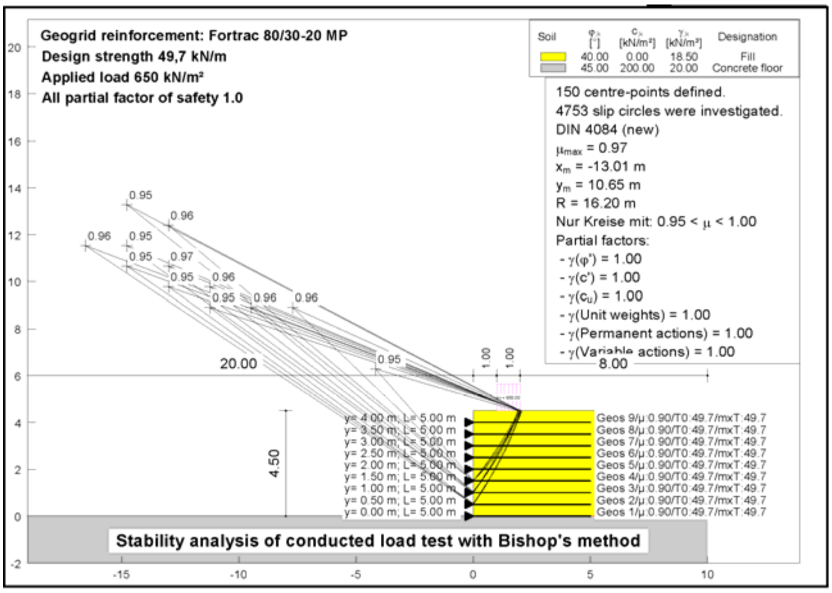


**Figure 3.** Stability analysis with Bishop's method

Similar results are obtained with Krey and the block sliding method, as shown in Table 1. Only the calculation with the method of Janbu shows an utilisation degree $\mu$ greater then 1. Since Janbu normally is used for shallow slip bodies in slopes, a correction factor has to be applied on this utilisation degree, which still results in a $\mu$ of 1,19.

**Table 1.** Utilisation degrees $\mu$ according to different analytical limit equilibrium methods ($\mu > 1.0$ → failure, $\mu = 1.0$ → exact equilibrium, $\mu < 1.0$ → stable)

| Method | Bishop | Krey | Block Sliding | Janbu | Janbu (corrected) |
|---|---|---|---|---|---|
| Degree of utilisation | 0.97 | 0.98 | 0.93 | 1,30 | 1,19 |

The measurements of the horizontal wall facing deformations show a strong bulging in height of about 2.5 to 3.5m (Alexiew 2007). Comparing this with the locations of the most critical slip bodies according to the calculations with Janbu and the block sliding method, shows that they coincide with the location of the maximum horizontal deformation of the facing, Figure 4. Alexiew (2007) suggests that the maximum horizontal bulging of the facing at high sill pressures coincides roughly with the projection of the strip load on the facing under an inclination of 45° to the horizontal, what is an angle somewhere between the internal friction angle φ and the Rankine's angle of active earth pressure $\theta_a = 45° + \varphi/2$. Based on the analyses herein it seems possible to predict the zone of maximum horizontal bulging in a more precise way using polygonal rigid-body-based analytical procedures like Janbu or Block-sliding, Figure 4.

## NUMERICAL ANALYSES OF THE REAL SCALE TEST

Further analyses were performed with numerical simulations. For these studies the commercial FEM software Plaxis 2D Version 8.6 was used. Two different studies were done.

### Comparison of the reinforced vertical soil wall with a laterally infinite half-space

In Alexiew (2007) a simplified comparison of the measured sill settlement on the GRS-wall with the settlement of the same sill beam on a non reinforced infinite plane according to Winkler was done. It was shown, that the registered load-settlement behaviour of the beam positioned only 1.0 m away of the edge of the 4.5 m high vertical GRS-wall is practically identical to calculated settlements based on Winkler's model for a homogeneous even plane: an indication of the high efficiency of the geogrid reinforcement used. In a first approach a modulus of subgrade reaction according to Winkler was estimated to be around 57 MPa/m for the last loading cycle. A literature study showed, that this value is in the range of generally accepted values of modulus of subgrade reaction for a gravel-sand mixture with good compaction, which are between 50 to 60 MPa/m. To verify this simplified comparison, a numerical parameter study was performed herein, using not Winkler's model but the supposingly more precise model of the infinite elastic half-space in combination with the Mohr-Coulomb failure criterion. The numerical model is shown in Figure 5.

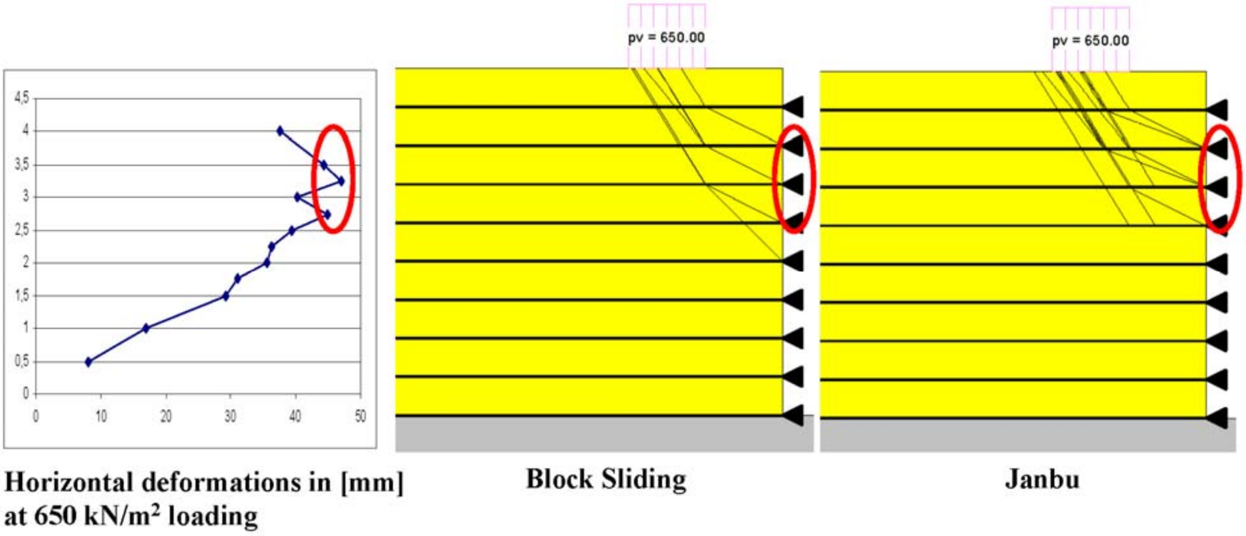


**Figure 4.** Comparison of the maximum horizontal deformation of the wall facing and the locations of the most critical slip bodies

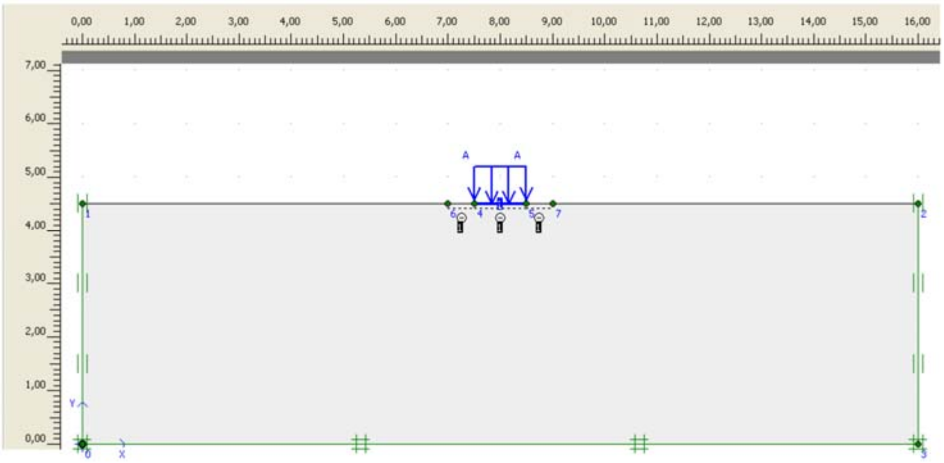

**Figure 5**. Numerical model for a comparison of the vertical GRS-wall with a laterally infinite elastic half-space

The concrete beam is simulated by using a rigid plate element. Since several loading and unloading cycles had to be simulated and an increasing stiffness of the structure due to the loading cycles was observed in the real scale test the Hardening Soil Model implemented in Plaxis was used for the parameter study. As in the real scale test the loading and unloading cycles were simulated. In the parameter study the modulus of the soil was varied. A good agreement of the load-settlements curves between the real scale test and numerical study was found, Figure 6.

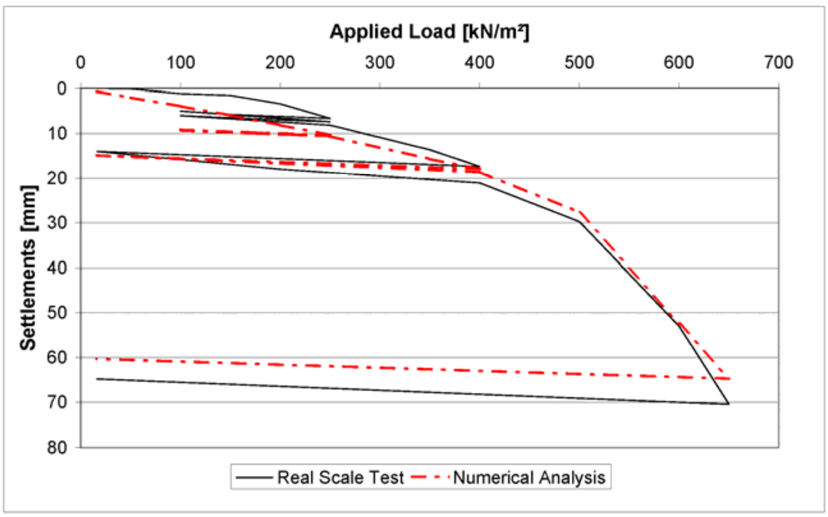


**Figure 6.** Settlement of the concrete block from the loading test and the numerical analysis

The parameters used are shown in Table 2. The oedometric modulus of 110 MN/m$^2$ equals a deformation modulus of the subgrade of about 90 MN/m$^2$ with a poison ratio of 0.25. These are plausible values for the fill used. It was found out again similar to the Winkler based analysis in Alexiew (2007) mentioned above, that the geogrids compensate completely the missing elastic half-space to the right of the loading beam.

**Table 2.** Used soil parameters for the HSS-Model in the numerical studies with Plaxis

| $\gamma$ [kN/m$^3$] | $E_{50}^{ref}$ [MN/m$^2$] | $E_{oed}^{ref}$ [MN/m$^2$] | $E_{ur}^{ref}$ [MN/m$^2$] | m | c [kN/m$^2$] | $\varphi$[°] | $\nu$ [-] | $p^{ref}$ [kN/m$^2$] | $K_0^{nc}$ [-] |
|---|---|---|---|---|---|---|---|---|---|
| 18,5 | 110 | 110 | 330 | 0,5 | 0,1 | 40 | 0,25 | 100 | 0,357 |

**Numerical simulation of the real scale loading test of the geogrid reinforced vertical bridge abutment**

In a further numerical study the step-wise construction of the wall and the subsequent loading cycles were simulated and analysed. The geometry of the numerical model corresponds to the geometry of the reinforced test wall, whereas the concrete floor and the back wall of the test pit are represented by fixed boundaries, Figure 7. The identical soil parameters including the estimated soil modulus from the first numerical study were used. A tensile stiffness was assigned to the geogrid elements. This stiffness was derivated from the load-strain curves resp. the isochrones of the geogrid. The geogrid material behaviour was defined as elastoplastic with a maximum tensile force of 49,7 kN/m, as before in the analytical analyses. Tests with the geogrid-family used and common soils in the shear mode and pull-out mode had resulted in coefficients of interaction (bond) of ≥ 1.0 and 0.9 to 1.0, respectively. Thus, no interfaces between geogrids and soil were applied in the numerical model (perfect bond).

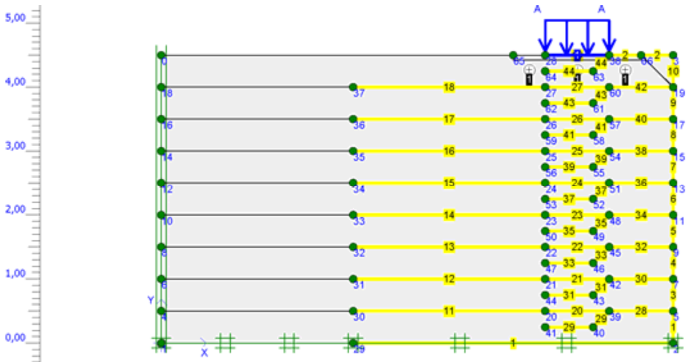


**Figure 7.** Numerical model of the vertical geogrid reinforced bridge abutment

During the numerical study it was noticed, that to comply with the measured test results for the vertical and horizontal deformations and also with the ultimate bearing capacity from the real scale test, the tensile stiffness *J* of the geogrids had to be adjusted. In order to fit in a best way with the test results an additional variation of the tensile stiffness of the geogrids was applied. It turned out that in any case a discrepancy between the tested and the numerical load-settlement curves occurred. Applying a tensile stiffness derived from the load-strain curves resp. the isochrones led to an overestimation of the settlements in all load cycles, whereas a decreasing discrepancy for higher loads was observed. A significant increase of settlements could be noticed at a load level between 550 kN/m² and 600 kN/m². By increasing the tensile stiffness the accordance of the load-settlement curves especially for higher loads improved, whereas the significant increase of settlements was observed at about 600 kN/m², compare with load-settlement curve for "Numerical Simulation using J2" in Figure 8. By applying a very high tensile stiffness, the accordance for the lower load cycle improved, but big differences can be observed for the last load cycle. Furthermore the signs of a beginning failure could not be noticed, compare with load-settlement curve for "Numerical Simulation using J3" in Figure 8.

Similar observations are made for the horizontal deformations of the wall facing, Figure 9. Shown are the measured and numerically estimated horizontal wall deformations at load levels 200 kN/m² and 650 kN/m² of the last loading sequence resp. load cycle. Again it can be observed, that a significant overestimation of the horizontal wall deformations occurs for the lower load levels in the numerical analysis and a good agreement is found for the last load level.

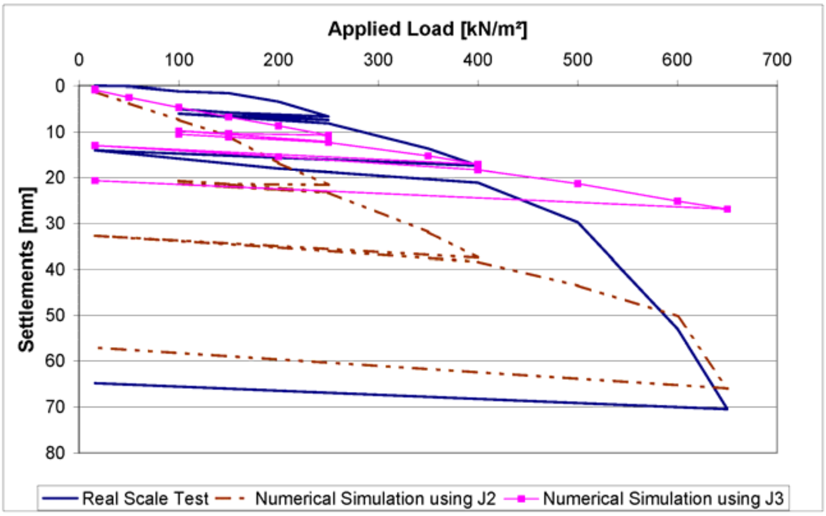


**Figure 8.** Comparison of the settlements of the real scale test and the numerical analyses with tensile stiffness *J2* (dashed line) and a tensile stiffness *J3* (curve with squares)

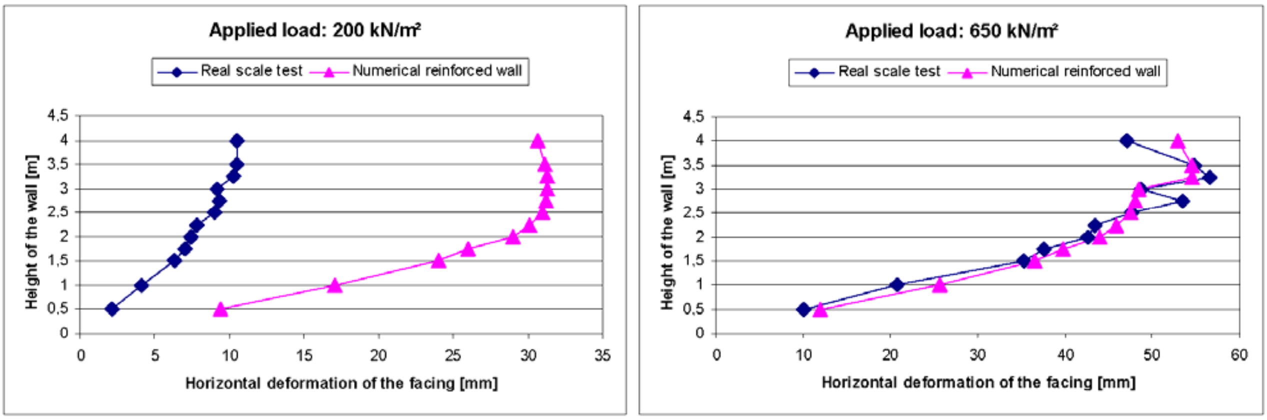


**Figure 9.** Measured and numerically estimated horizontal wall facing deformation for the load levels 200 kN/m² and 650 kN/m² of the last loading sequence

The authors suggest at present two possible explanations for the discrepancy between FEM-simulation and real scale test.

- The most plausible explanation is that the simulation with the software used does not reflect in an appropriate way the early mobilisation of the high-tensile modulus geogrids mainly during construction and compaction and mabye also the early stages of loading. Due to that the numerical analysis overestimates both settlements and horizontal facing bulging in those early loading stages. As shown before, a better agreement for the low loading stages was achieved simulating higher geogrid tensile stiffness than the real values provoking artificially in the numerical analyses an earlier mobilisation of the geogrids.
- Another explanation could be the prehistorie of the wall. Before the loading tests described other test series had been performed on the same wall. During these previous tests the wall was loaded horizontally at the front by a moving full-height RC-plate (Alexiew 2007, Pötzl & Naumann 2005a, 2005b). Maybe because of this a decompaction of the wall fill especially near the front area had taken place. This undefinable process was not numerically simulated herein.

Note, that comparative numerical analyses using the Mohr-Coulomb model (not shown herein) resulted also into an insufficient agreement with the test results.

The maximum activated forces in the geogrids at a load of 650 kN/m$^2$ are similar to the tensile strength used in the analytical analyses. By marking the locations of the maximum tensile forces with dots on the geogrid, the contours of a probable failure figure are obtained. As shown in Figure 10 again a good agreement is found between the analytical failure circle from Bishop, Figure 3, and the possible failure line from the numerical study.

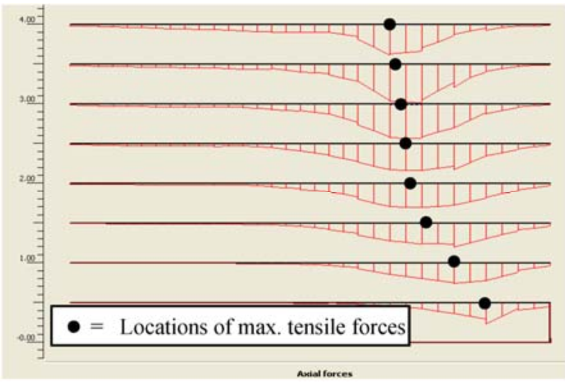


**Figure 10.** Activated tensile forces of the geogrids at a load of 650 kN/m$^2$, locations of maximum tensile forces are marked with a dot

The principle stresses at a load of 650 kN/m$^2$ are presented in Figure 11. Despite the so called "soft facing" of the vertical wall, the load is transferred nearly vertically through the reinforced soil to the rigid bottom. This indicates the high capability of the high-modulus low-creep geogrid used to confine the soil even by a "soft" wraparound facing, "redirecting" the stresses downwards.

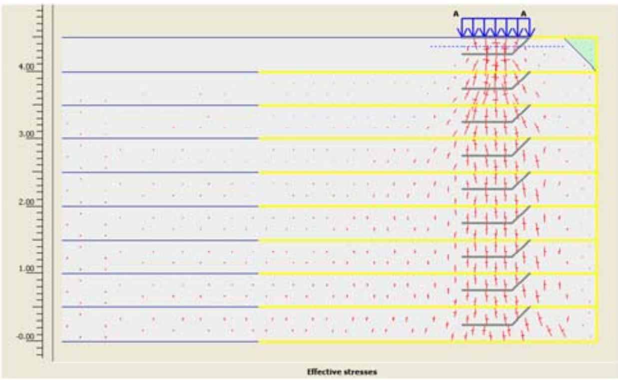


**Figure 11.** Principle stresses are "redirected" downwards by the wraparound facing

**CONCLUSION**

This paper presents first the most important test results being published in detail elsewhere (Alexiew 2007) of a real scale loading test of a geogrid reinforced vertical soil wall used as bridge abutment. The test results demonstrate the high capability of geogrid reinforced soil walls and the versatility as bridge abutment since the bearing capacity and also the deformations fulfil the stringent requirements.

The obtained test results are now analysed herein first with different analytical limit equilibrium methods. The calculations are made with the non-factored (say: "characteristic") parameters of the soil and the load. The tensile strength of the geogrid is reduced only due to creep and installation damage: no additional partial factor of safety is applied. The circular failure plane methods of Bishop and of Krey and the polygonal failure plane methods of Janbu and block-sliding are applied. Very good agreements are found between the analytical methods and test results, only Janbu seems to be a bit conservative.

In a second step numerical FEM analyses were performed in order to simulate the registered load-deformation behaviour of the test wall for the full load range from 50 kN/m$^2$ up to 650 kN/m$^2$ (when the test wall approached failure) under the loading sill beam. The most important foundings from the FEM analyses are:

- It was confirmed that for the system under discussion the load-settlement behaviour of the sill beam on top of the reinforced wall is similar to that of an even unreinforced infinite half-space (with the same soil parameters) demonstrating the efficiency of the reinforcement used.
- It was not possible to simulate the sill settlement and the horizontal displacements of the wall facing in a precise way for the lower loads. For them the numerical simulation overestimates both sill settlements and facing bulging. Possible explanations and comments are given.
- An artificial increase of the tensile stiffness (modulus) of the geogrids in the FEM-simulation results in a better simulation of wall behaviour, especially for the lower load range.
- The points of maximum tensile force in the geogrids at the maximum load of 650 kN/m$^2$ from the FEM-analysis correspond very well to the critical Bishop-circle from the analytical analyses.
- The wraparound facing from the geogrids Fortrac 80/30-35M without any other stiffening elements seems to provide sufficient support/confinement redirecting the main stresses downwards probably due to the grids tensile stiffness.

- Despite the problems faced, FEM seems to be an acceptable tool for analysis and modelling of the general tendencies in the behaviour of the prototype test wall. A main problem seems to be the appropriate simulation of the early geogrid mobilisation.

**Corresponding author:** Dr Dimiter Alexiew, Huesker Synthetic GmbH, Fabrikstrasse 13-15, Gescher, Nordrhein-Westfalen, D-48705, Germany. Tel: ++49 2542 701290. Email: dalexiew@huesker.de.

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