PHYSICAL AND NUMERICAL ANALYSES OF A GEOGRID-REINFORCED SOIL SYSTEM FOR BRIDGE ABUTMENTS

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ABSTRACT: The paper deals with a study of a geogrid-reinforced soil (GRS) solution for bridge abutments, which was tested in a real full scale test at the LGA Nuremberg and also analyzed with analytical and numerical methods.

In the first part of the paper the full scale test will be presented. A 4.5 m high vertical GRS wall was directly loaded by a 1 m wide reinforced concrete beam and hydraulic jacks, which were located near the top front edge. The concrete beam, simulating the bridge sill beam, was subjected to several loading and unloading cycles by the hydraulic jacks, where the load was increased up to three times the normal load for this kind of structure. Settlements and horizontal facing deformations were measured during the test.

The paper presents analytical analyses of the full scale test. The analytical design procedures include methods commonly used worldwide e.g. Bishop, Krey, Janbu and Block Sliding. The focus of the analytical analyses is on the ultimate limit state.

Furthermore numerical analyses of the full scale test will be presented. The numerical analyses were made using FEM with the commercially available Plaxis V9.2 program. Different material models for the soil, such as Mohr-Coloumb, Hardening Soil and Hardening Soil with Small Strain Stiffness, have been used. The focus of the numerical analyses is on both the ultimate and serviceability limit state. A comparison of the analyses results between the different material models and with the measurements from the full scale loading test will be presented.

The comparisons made between analytical and numerical procedures on the one hand, and the behaviour tested 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.

1. Introduction

The use of geogrid reinforced soil as retaining walls in various applications is meanwhile a well known, accepted and established construction method, whereas the use of geogrid reinforced soil walls as directly loaded bridge abutments still has to contest for acceptance. Picture 1 shows a Fortrac® geogrid reinforced bridge abutment in the Netherlands before and after installation of the facing.

In comparison to conventional geogrid reinforced soil structures geogrid reinforced bridge abutments experience very concentrated and heavy loads and have to fulfil stringent limitations in terms of deformation. To gain better understanding of the system behaviour and to encourage the trust in the system a real scale test was performed at the LGA Nuremberg, Germany. Since the presentation of the first test results in 2007 (Alexiew 2007) a strong increase in the number of constructed Fortrac® geogrid reinforced bridge abutments is noticed. First results of the analytical and numerical analyses of the full scale test were published in 2008 at the EuroGeo4 in Edinburgh (Alexiew & Detert 2008).

Picture 1. Fortrac® geogrid reinforced bridge abutment without and with facing (System Muralex®)

2. Test set-up of the real scale test

A test set-up at the LGA Nuremberg previously used for a different research project (Plötzel & Naumann, 2005a, 2005b) was slightly modified and reused for the real scale test of a directly loaded geogrid reinforced bridge abutment. The 4.5 m high vertical wall was loaded by heavily reinforced 1.0 m wide concrete block, simulating the sill beam, and hydraulic jacks, applying the load from the bridge. The concrete block was placed only 1.0 m away from the edge of the vertical wall and transferred the load from the hydraulic jacks onto the reinforced soil wall. The wall was reinforced by 9 layers of geogrids (Fortrac® 80/30-35M) from PVA with an ultimate tensile strength of 80 kN/m at only 5% strain. 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 wrapped around, so the wall got a so called "soft facing" without any bending stiffness. 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 deformation of the wall surface and horizontal deformation of the wall facing during the test, see 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).

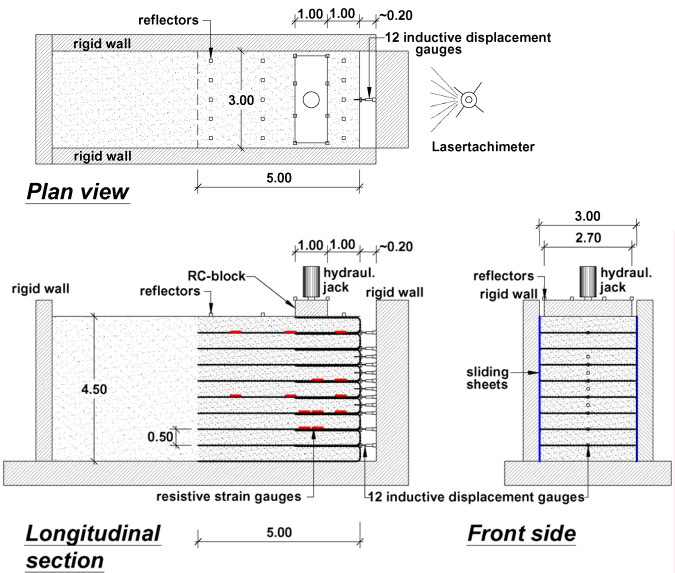

Figure 1. Test set-up and instrumentation of real scale geogrid reinforced bridge abutment loading test (Alexiew 2007)

3. 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, which is the common contact pressure from a sill beam, see 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 leads to 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.

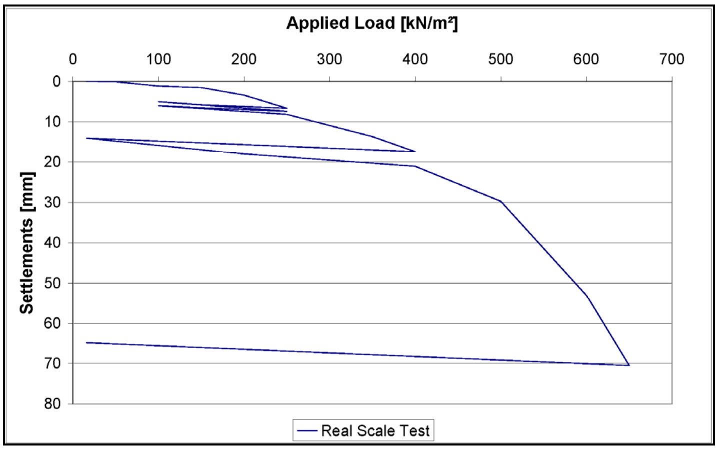

Figure 2. Settlement of the concrete block during the loading test

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 suggest 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 did not occur.

4. Analytical and numerical analyses of the obtained test results

In the following chapter the results of the analytical and numerical analyses are compared to the results from the full scale test. Some already published findings (Alexiew & Detert, 2008) will be repeated for the better understanding of the following statements.

Different analytical limit equilibrium design methods have been used for a comparison of the obtained test results. All calculations are done with characteristic values, which mean partial factors of safety were set to 1.0. 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/m² was applied 1.0 m from the edge of the 4.50 m high vertical reinforced soil wall. The analytical analyses with the methods of Bishop and of Krey (circular failure plane) and "Block-sliding" (polygonal failure plane) matched fairly well with the observed beginning failure mechanism of the real scale test. The results of all three methods predicted a nearly 100% usage of the bearing capacity of the reinforced soil structure. Only the analysis with Janbu predicted a clear failure of the structure with an overloading of about 30% resp. 19%, when applying a correction factor. A good agreement was found between the analytical circular failure plane and the potential numerical failure plane, which was obtained by marking the locations of the maximum tensile forces in the geogrid since these locations correspond with the location of the maximum shear deformation in the structure, see figure 3.

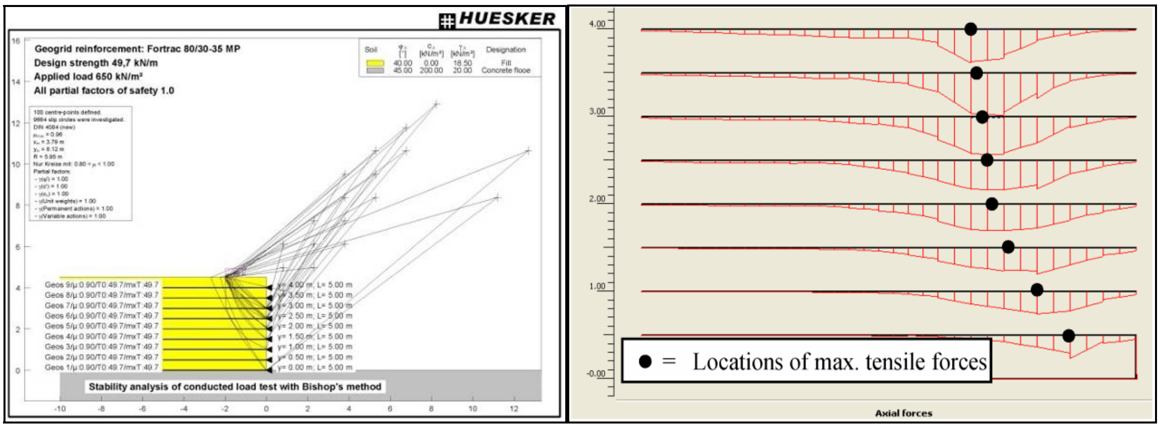

Figure 3. Comparison of the potential analytical and numerical failure plane

A comparison was made between the measured sill beam settlements of the reinforced vertical soil wall with a numerical laterally infinite half-space model. No geogrid reinforcement was used in this numerical model. The shear strength parameter and unit weight of the soil are the same as in the analytical analysis. The deformation modulus of the soil was varied until a good agreement between the measured and calculated load-settlement curve was obtained. 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. The load-settlement curve of the full scale test and the numerical model are shown in figure 4.

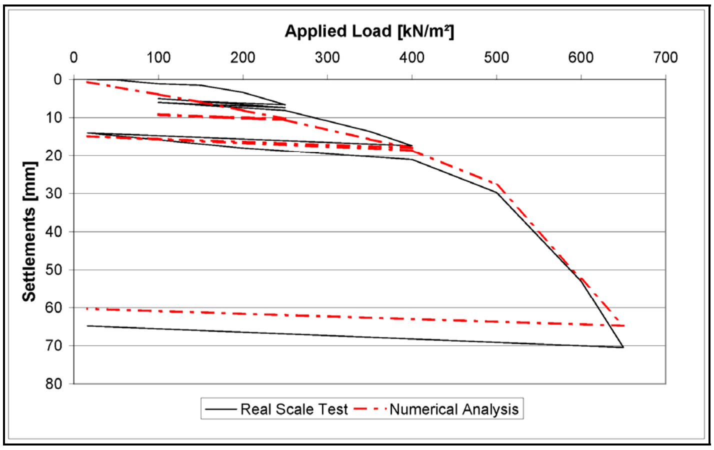

Figure 4. Settlement of the concrete block from the loading test and the numerical analysis

A good agreement of the load-settlement curves was found for a deformation modulus of the soil of about 90 MN/m² with a poison’s ratio of 0.25, which can be transferred into an oedometric modulus of about 110 MN/m². These are plausible values for the fill used. This comparison demonstrates the high efficiency of the geogrid reinforcement used, which is capable to compensate completely the missing elastic half-space to the right of the loading beam resp. wall facing.

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. A tensile stiffness was assigned to the geogrid elements. This stiffness was derived 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 $\geq$ 1.0 and 0.9 to 1.0 respectively. Thus, no interfaces between geogrids and soil were applied in the numerical model (perfect bond).

It has to be mentioned here that a precise simulation of the construction process is not possible. In reality a temporary formwork is used, which acts as support during the soil installation and compaction, which is hard to model. Furthermore a certain amount of tensile force develops during the installation process since the soil layers are not perfectly smooth and the soil is re-arranging during compaction.

The numerical study was done with the Mohr-Coloumb model (MC), the Hardening Soil model (HS) and the Hardening Soil model with Small Strain Stiffness (HSsmall). The soil parameters used are shown in table 1.

Soil model	γ	E_{50}^{ref}	E_{oed}^{ref}	E_{ur}^{ref}	m	c	ϕ	η	p^{ref}	$\gamma_{0,7}$	G_0^{ref}
	[kN/m³]	[MN/m²]	[MN/m²]	[MN/m²]	[-]	[kN/m²]	[°]	[-]	[kN/m²]	[-]	[kN/m²]
MC	18,5	-	110	-	-	0,1	40	0,33	100	-	-
HS	18,5	110	110	330	0,5	0,1	40	0,25	100	-	-
HSsmall	18,5	110	110	330	0,5	0,1	40	0,25	100	$2*10^{-7}$	$4,8*10^{5}$

Table 1. Soil parameters used in the numerical analyses

Figure 5 shows the load settlement curves of the three numerical calculations. Surprisingly only small differences regarding the shape of the load-settlement curves using the different soil models could be observed. The load-settlement curves demonstrate a nearly linear behaviour, whereas the biggest settlements are obtained by the MC and the smallest with the HSsmall model. The shape of the numerical load-settlement curves differ clearly from the measured load-settlement curve, which behaves obvious stiffer until the final settlement is reached. As mentioned before, there are different effects during the construction process, which are difficult if not impossible to model. Furthermore the geogrid behaviour is modelled elastoplastic, whereas the real stress-strain curves of the geogrid are more complex. Various researchers have pointed out, that the "in-air" tested geogrid behaviour may differ from the "in-soil" embedded geogrid behaviour. Hence the performance of the reinforced soil is more complex than just the sum of the soil and the "in-air" tested geogrid behaviour. A better curve fitting may also be achieved by variation of the soil parameters, which will be analysed in future.

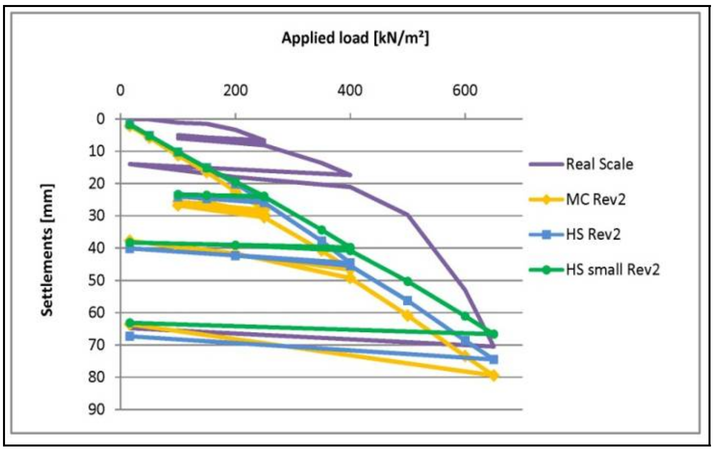

Figure 5. Comparison of the load-settlement curves using different soil models

For the further analysis only the HS model was used, since it is more stable then the MC model for the analyzed problem and less parameters had to be assumed compared to the HSsmall model.

A variation of the geogrid stiffness was analyzed in a further numerical analysis, see figure 6. The stiffness of the geogrid for the “HS_Rev3” curve is twice and the stiffness of the “HS_Rev4” curve is three times the “in-air” stiffness. It can be seen, that a better agreement is reached for the first loading-unloading circles but an inferior agreement for the last loading circle.

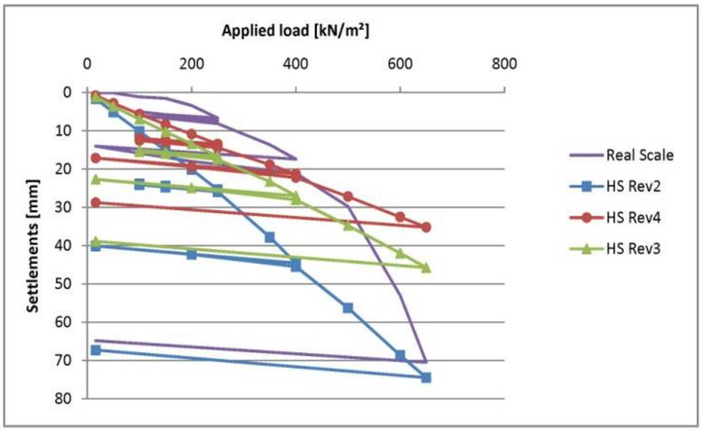

Figure 6. Comparison of the load-settlement curves using the HS model and different geogrid stiffnesses

The same tendency can be observed for the horizontal deformation of the vertical wall face, as shown in figure 7 for three different loading stages.

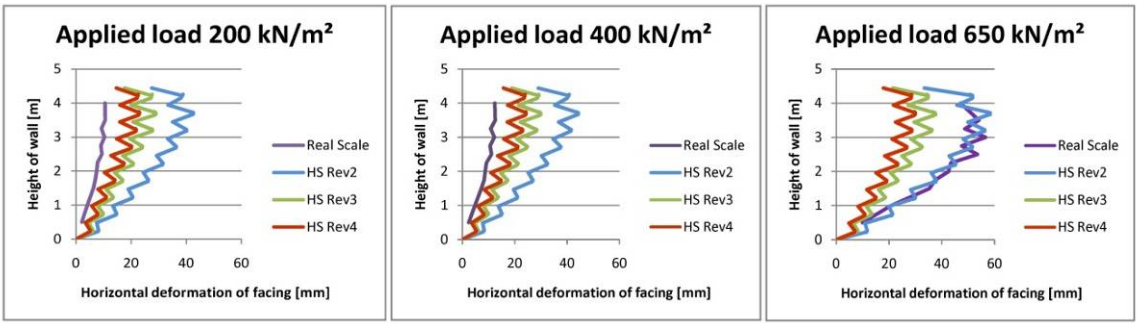

Figure 7. Comparison of the horizontal facing deformation using the HS model and different geogrid stiffnesses

Even if the prediction of the horizontal and vertical deformation using numerical analyses does not match perfectly the measured values, the numerical analyses still confirm the high bearing capacity of the reinforced soil structures at low deformation.

It was demonstrated before in this paper, that the reinforced soil body is capable to compensate completely the missing elastic half-space to the right of the loading beam. Similar conclusions can be derived from the torsion resp. the differential settlement between both ends of the simulated sill beam in the numerical model, which is only 0.134° resp. 2.3 mm under a loading of 350 kN/m² and 0.45° resp. 0.7 mm under a loading of 650 kN/m². The application of a three times stronger geogrid, which results in higher stiffness, would result in 0.017° resp. 0.3 mm under 350 kN/m² and 0.11° resp. 2 mm under 650 kN/m².

The activated forces within the geogrids seem to fit very well with the design forces of the above mentioned analytical calculations, see figure 8. This is an important observation, since there are discussions, that geogrid reinforced soil structures maybe designed over conservative, since only small deformations are observed at constructed and instrumented retaining walls. As it can be seen from these numerical calculations, the maximum tensile forces of the geogrids are only activated over a small length resp. area, which is the zone of the biggest shear deformation of the structure. So even, if the deformation of the whole structures is small, there can be bigger deformations within the wall, which are generating significant tensile forces in the geogrid. The maximum force under a loading of 350 kN/m² is about 29 kN/m, which correspond very well with the design strength of the Fortrac® 80/30-35M, including all reduction and partial factors of safety according to the current DIN 1054.

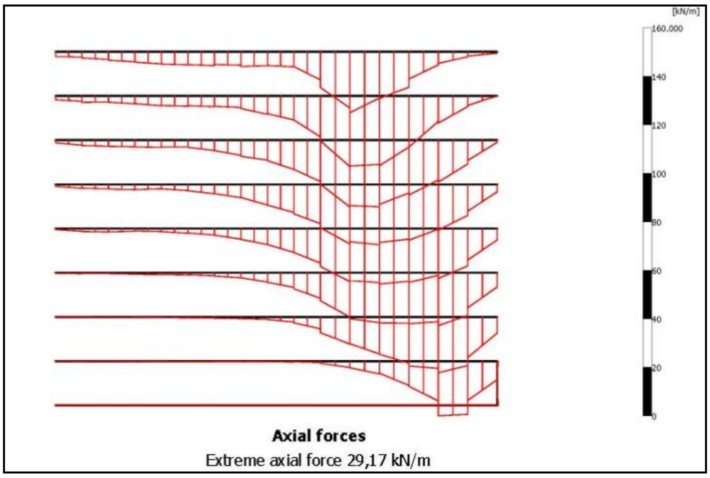

Figure 8. Activated forces in the geogrid under a loading of 350 kN/m²

5. Conclusion

The paper presents the most important test results of a real scale loading test of a geogrid reinforced vertical soil wall used as bridge abutment, which are published in detail in Alexiew, 2007. 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 meet the stringent requirements.

Different analyses of the obtained test results using analytical as well as numerical methods are presented in this paper. Good agreements are found between the results of the real scale test and the analytical and numerical analyses. All analyses confirm the high bearing capacity of the reinforced soil structure under the very concentrated and heavy loads as well as the compliance with the stringent requirements regarding the deformation of a bridge abutment.

It is found that the exact simulation of the step-wise construction of a geogrid reinforced soil structure is not easy and some limitations exist.

The most important findings from the FEM analyses are:

- The influence of the soil model (MC, HS and HSsmall) on the results of the numerical calculation in the case studied is smaller than expected. Therefore all further calculations are made with the HS model.
- It is 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.
- In the case studied it was not possible to simulate the sill beam 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 beam settlements and facing bulging. The numerical model is reacting softer then the real structure.

- 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, but an inferior agreement was found for the higher loads.
- 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 activated geogrid forces in the numerical simulation correspond well with the tensile forces used in the analytical calculation.
- 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, especially in cases of very high beam loads. For lower loads the FEM results seem to be on the safe side from the point of view of load-deformation behaviour.

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