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Ultimate bearing capacity tests on an experimental geogrid-reinforced vertical bridge abutment without stiffening facing

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ABSTRACT: The paper deals with geogrid reinforced soil as solution for bridge abutments. Most important results are presented of a real scale test of a 4.5 m high geogrid reinforced vertical soil block loaded directly on top near the edge by a sill beam, high-lightening the low settlements and horizontal displacements measured. In one test, the reinforced embankment was nearly lead to failure, what occurred with a load in the order of 3 times the usual one for this kind of structures.

1 INTRODUCTION

Steep slopes and walls from geosynthetic reinforced soil (GRS) became very popular and established practice due to their advantages: cost-effectiveness, blending in well with the landscape, fine-tuning for optimum functionality etc. The broad range of available geosynthetic reinforcement allows optimisation and eliminates any limits to height and load capacity. The next step to be expected was the use of GRS in an "exclusive" area like bridge abutments, which are heavily loaded and have to fulfil stringent requirements with regard to load capacity and any kind of deformation. They are an intersection of traditional construction types (i.e. reinforced concrete (RC)), soil mechanics and foundation engineering with what today is loosely called "geosynthetic engineering".

2 BRIEF OVERVIEW AND BACKGROUND

The first steps began with the use of GRS to form the front face or part of the wing walls of bridge abutments. The experience with "conventional" GRS slopes and walls (see e.g. Herold & Alexiew (2001), Herold (2002) and Alexiew (2005)) was used.

Here the GRS is not loaded directly by the sill beam: The sill is supported on separate conventional (i.e. RC) load-bearing systems. The GRS wall is built around this conventional system. The bearing and serviceability requirements do not generally differ much from other vertical GRS walls. A more recent example of this type of bridge abutment is presented in Sobolewski & Alexiew (2005).

It is also possible to relieve abutment walls and/or wing-walls of earth pressure by placing a GRS block at the back of them. The GRS only carries loads from the earth pressure of the backfill and traffic load (Jossifowa & Alexiew 2002).

However, for what we should call "real" GRS abutment the sill beam is seated directly on the GRS block. Typical are the significant contact pressures under the sill of 150 to 250 kN/m^2 over a limited area (the width of the sill is usually < 2.0 m) positioned very close to the top edge of the GRS, frequently within 1.0 to 1.5 m. The allowable vertical (most importantly) and horizontal deformations are very much more limited, depending on the bridge system, than for a "normal" GRS wall.

One of the first non-experimental structures of this type was built at the beginning of the 1990s near Ullerslev in Denmark for a road bridge over a railway using high-tenacity PET geogrids Fortrac 110 (Kirschner & Hermansen 1994). Design, detailing and construction were completed without problems and the structure has performed well in service. At the time of construction it was considered a pioneering project.

Despite the Ullerslev project and other activities along the same lines (e.g. Uchimura et al. 1998, Zornberg et al. 2001), it was a long time before a "real" GRS bridge abutment in the German highway network was constructed at Ilsenburg on the German National Road K 1355 (Herold 2002). The knowledge gained is of great importance, including the "psychological" aspects on the part of the client.

Recently a "jointless bridge abutments" research programme was introduced in Germany the experimental part being carried out in cooperation with the author. In a test pit a 5 m high GRS block with vertical wraparound "soft" facing was installed at the back of a moving inwards and outwards RC-wall simulating the movements due to temperature changes in a jointless bridge. Well graded crushed sandy gravel and Fortrac[R] 80/30–35 M geogrids from PVA (Alexiew et al 2000) at 0.5 m spacing were used. The GRS was loaded only horizontally in these tests. The system is recommended for practical implementation (Pötzl & Naumann 2005a, 2005b). For a more detailed information see Alexiew (2007).

After completion of this test series the GRS test wall remained in the test pit for a year.

3 TEST WALL AS A "REAL" BRIDGE ABUTMENT

3.1 *Test set-up and comments*

Then it was suggested that the structure could serve for testing a "real" bridge abutment. Vertical loads could be applied directly from a reinforced concrete beam acting as a sill beam.

The focal point was not the "internal life" in the sense of e.g. internal stresses and strains, but the overall behaviour of the structure:

- What is the contact pressure under the sill that would drive the GRS to failure; this is the only way of estimation of load capacity resources and safety margins (ultimate limit state – ULS).
- How large are the settlements of the sill in the usual pressure range of 150 to 250 kN/m^2 (very important) and how large are the displacements of the facing (serviceability limit state – SLS).

A risky tendency to exclusively concentrate on the serviceability (SLS) in GRS structures and to a greater or lesser extent "neglect" the ULS has been noticed recently in a few publications. In doing this one loses sight of the fact that in these cases the SLS only is not in any respect relevant to safety.

For the new testing programme a rigid slab was fixed in position at a certain distance in front of the GRS (Fig. 1). This meant the structure remained a vertical GRS wall with a "soft" facing of the type "wraparound" without any stiffening elements. Two layers of a smooth membrane between the fill and the side walls of the test pit were installed to minimise friction. A 1.00 m × 2.70 m RC-block was placed as a sill beam 1.0 metres behind the front edge loaded vertically by means of hydraulic jacks (Fig. 1 & 2). Twelve displacement transducers were attached to the facing to measure its horizontal displacement (Fig. 2). Reflectors were attached over the whole surface and to the RC-beam to measure the settlements and any tilting by a precision level (Fig. 1). A data acquisition system recorded permanently all load and deformation data. Figure 3 shows the test set up.

3.2 *Constraints and boundary conditions*

The equipment available and the history of the project imposed certain constraints and boundary conditions

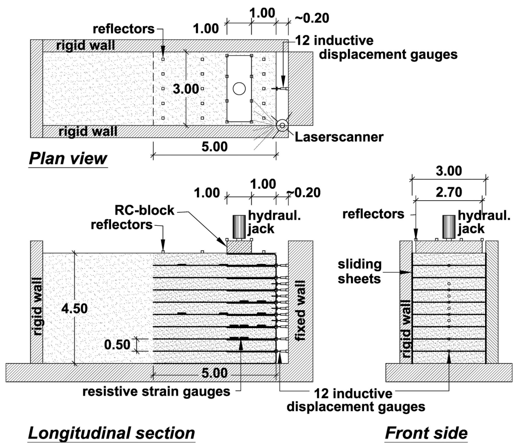

Figure 1. Schematic of the test.

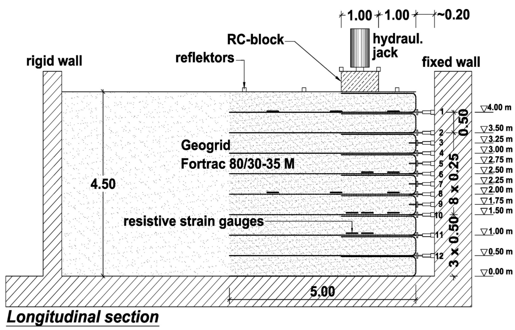

Figure 2. Horizontal displacement transducers.

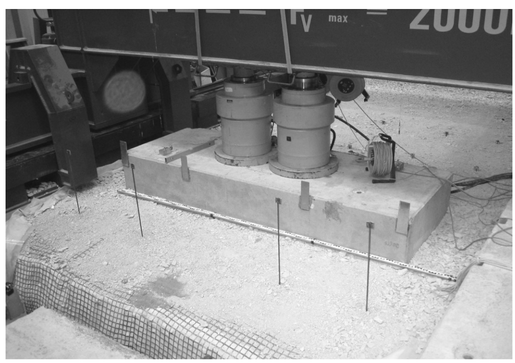

Figure 3. Overview of the test set-up.

to the tests. The height had to be reduced to 4.50 m. The compaction of the crushed sandy gravel fill was found to be only ca. $D_{pr} = 95\%$ in the upper (critical) zone. It was also not clear whether the extreme outer area of the fill at the facing had experienced some loss of density as a result of the earlier tests "jointless bridges".

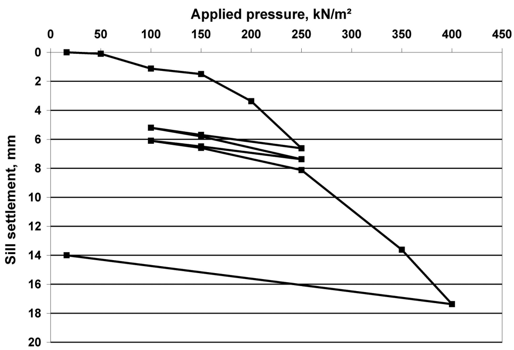

Figure 4. Settlement of the sill beam in Test 1.

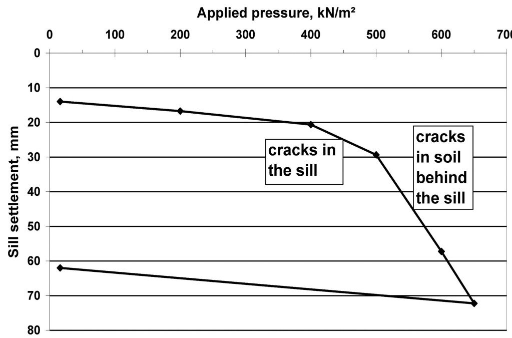

Figure 5. Settlement of the sill beam in Test 2.

The pressure under the loading RC-block (sill) could be taken up to a maximum of ca. 650 kN/m^2.

3.3 *Test procedure*

Two separate tests were carried out. In Test 1 the maximum load was 400 kN/m^2, i.e. twice the contact stress normally experienced under a sill beam (Fig. 4). After each increment a pause took place until there was a reduction in the change of settlement meeting the requirements of the plate bearing tests in accordance with DIN 18134.

The aim of Test 2 was to take the GRS block to failure using the full capacity of the jacks of 650 kN/m^2 (Fig. 5). Following values were recorded (cf. Section 3.1): contact pressure, average settlement of the loading RC-block, the horizontal displacement of the facing and the settlements of the reflectors on the top of the GRS system and on the RC-block.

3.4 *Important test results and comments*

Only the most important results have been included herein. Figure 4 shows the relationship between load and sill beam settlement in Test 1 unsmoothed. The shape of the graph suggests that a certain amount of further compaction may have taken place between 150–250 kN/m^2. It should be born in mind that the top zone of fill had only $D_{pr} = 95\%$ (Section 3.2), and that some loosening of the front part near the beam may have occurred as a result of the horizontal loading of the front area in the earlier "jointless bridges" tests (Section 2).

Let us make an analogy to the well known loading plate test. The increase in settlement in the first loading cycle in the range 150–250 kN/m^2 is ca. 5.9 mm and in the second loading cycle ca 1.3 mm, what indicates an increase in compressive stiffness of $5.9/1.3 = 4.5$. This is an unusually high value and indicates an additional compaction; a value of about 2.0 would have been expected here for a well compacted fill. Obviously, in Test 1 a recompaction of the fill directly under the block and perhaps in the possibly loosened front area takes place together with a higher mobilisation of the reinforcing geogrid in combination with this recompaction. With an a priori good compaction in the critical area the sill beam settlement would be even less than 5–6 mm in the relevant loading range, although even 5–6 mm would satisfy common requirements.

It should be noted that on Figure 4 the "unloading" line from 250 to 100 kN/m^2 is flatter than the "loading" one and that the hysteresis between 100–250–100 kN/m^{2}is parallel to the unloading part of the graph for unloading from 400 kN/m^2 down to 0 kN/m^2. This also indicates an increase in stiffness of the system and a tendency towards identical elastic behaviour at higher loads, or after recompaction and full mobilisation of the system.

Figure 5 shows the sill settlement for Test 2. The graph starts at the residual settlement of 14 mm after unloading in Test 1.

The results for Tests 1 and 2 can be converted into a modulus of subgrade reaction M , MPa/m, according to Winkler: the approximate equivalents are 57 MPa/m for Test 2 and 31 MPa/m and 16 MPa/m for Test 1. The system behaviour in Test 2 is clearly stiffer. Interestingly the common, generally accepted values of modulus of subgrade reaction for a gravel-sand mixture (as here) with good compaction are approximately 50 to 60 MPa/m (similar to 57 MPa/m here in Test 2) and with poor compaction 25 to 35 MPa/m (similar to 16 to 31 MPa/m here in Test 1) (Alexiew et al. 1989). This is one indication more for an insufficient compaction of the fill at the beginning of Test 1. Note: The common values of M apply to loading on a laterally infinite plane, in our case there is a 4.5-high, vertical slope only 1.0 meter away. That a similar M to that on a plane is achieved on the top of a GRS vertical block with a strip load applied close to its edge (whether compacted or not) shows that the geogrids used are acting very efficiently.

In Test 2 at approximately 450 kN/m^2 several fine vertical cracks were visible on the bottom edge of the

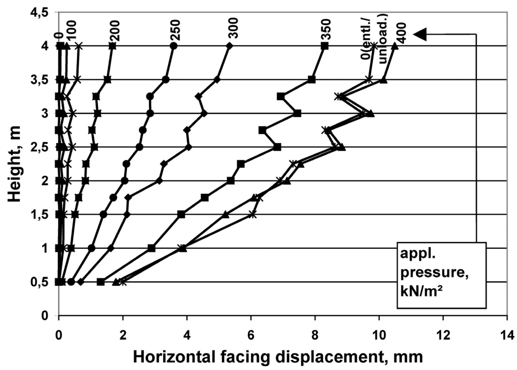

Figure 6. Horizontal displacements of the measured points on the facing for Test 1.

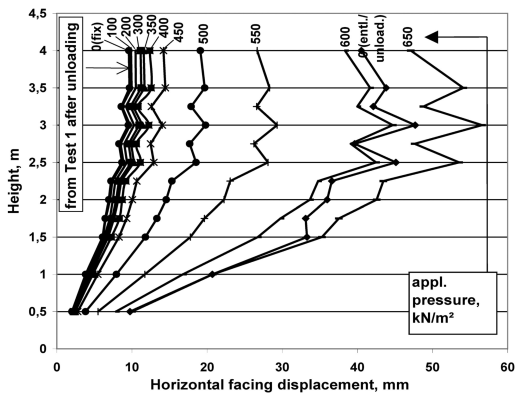

Figure 7. Horizontal displacements of the measured points on the facing for Test 2.

heavily reinforced RC sill beam, whilst in the GRS wall itself there was still nothing significantly amiss. Up to 600 kN/m^2 there were no noticeable symptoms of failure to be seen. Between 600 and 650 kN/m^2 a small irregular crack finally appeared in the fill surface behind the sill beam and extended towards the rear along the test pit walls. At 650 kN/m^2, the full capacity of the jacks was reached and increasingly accompanied by the above mentioned initial signs of failure. A clear failure, such as a failure body slipping forwards and downwards as might be expected, never occurred. It is a question of interpretation as to whether the ultimate limit state was reached or not, but the situation could be used as an "On-the-safer-side-ULS-benchmark".

Now to refer briefly to the horizontal displacement of the "soft" facing. Figures 6 & 7 show the results of Tests 1 and 2 (polygonal, not smoothed!).

Note, that the GRS wall sat directly on the concrete bed of the test pit and was surrounded by smooth membranes only at the sides. As expected, a bowing out at the middle of the front soft "single pillow" was noticed as compared to the points directly at the planes of the reinforcement (Fig. 2). However, the "local" bowing out was very small, probably because of the tensile stiffness of the flexible Fortrac 80/30–35 M geogrids used. Figure 7 (Test 2) starts at the residual displacements after unloading of Test 1 (no reset).

The maximum displacements occur for up to 400 kN/m^2 near the top of facing and in both tests amounts to a maximum of ca. 10 mm (measured in the worst position at a "bellying out" point). Analogically to the "vertical" behaviour of the system, Figures 6 and 7 also indicate an increasing stiffness in the horizontal direction after "recompaction".

A more detailed analysis of this and the relationships between the "vertical" and "horizontal" behaviour will be published separately.

From around 500 kN/m^2 (i.e. in Test 2) the character of the distribution of the deformation changed – the maximum values were no longer near the top. A "global bellying out" was increasingly noticeable between approximately 2.0–2.5 m and the 3.5 m level, together with an equally noticeable increasing curvature to this "bellying out". The position and height of this zone corresponds fairly accurately to the projection of the 1 m strip load (Fig.1) under ca. 45° down to the right at the facing. All this appears very plausible and corresponds well with common earth pressure theories. The maximum displacement of the soft facing amounted to 56 mm at 650 kN/m^2 (between two geogrid planes at $H = 3.0$ m) – a fairly large value, but under an extreme load. However, the "local" bowing of the geogrid at the front between the geogrid planes was still only ca. 9 mm. In relation to the layer spacing of 0.5 m it represents a low value and indicates that the wraparound geogrid is highly efficient. From a load of approximately 500 kN/m^2 there is an increase in the rate of deformation. The (relatively) large displacement from ca. 550 kN/m^2 could be taken as a trend in the direction of failure, however up to the end of the test at 650 kN/m^2 there was no visible breakthrough movement of any failure body at the facing. The results may well speak for themselves as to the remarkable reserve capacity of the GRS block; on removal of the load from 650 kN/m^2 to zero and despite the initial indications of failure the facing moved back ca. 10 mm. In plan view the front remained straight with no bowing in the middle; evidently the slip membrane layers at the pit walls were effective and the system can be idealised as 2-dimensional. This is important for the proposed further analyses and comparison with calculations, which will be published separately.

4 FINAL REMARKS

The tests presented herein on a geogrid-reinforced soil block simulating a real bridge abutment under a sill beam are in no way intended to be a comprehensive scientific analysis. The exercise is much more about testing the behaviour of a system and its reserves (!) in a situation directly related to practice. The use of an already constructed and used for other purposes test object after modification was advantageous in terms of time and costs, but it also brought its own restrictions and deficiencies, e.g. that we would have to live with the known insufficient compaction in the upper zone and the possible looser fill zones near the facing resulting from previous tests. The tests described herein are still fairly recent; and so the following remarks are a first, rather incomplete overview, but the most important points are readily recognisable and can be translated into practice.

The tested arrangement should be seen as a "worst case" scenario:

- The sill beam was only 1.0 m wide and placed only 1.0 m away from the edge
- The front face was vertical
- The facing had no special stiffening elements, being only a geogrid-wrapped-back wall
- The density of the fill in the most sensitive upper zone was only $D_{pr} = 95\%$, with probably loosened zones in the front area near the loading beam, some probably as a result of the previous tests.

The following remarks can be made:

- A contact pressure under the sill beam of up to 650 kN/m^2(approx. 3 times the pressure normally experienced) led to no obvious component or system failure. However, because there were signs of serious effects taking place, the situation could be used as a benchmark for the ultimate limit state.
- A contact pressure of up to 400 kN/m^2(approximately twice the usual value) resulted only in completely acceptable deformations.
- The tested system exhibited technically advantageous, ductile behaviour with no discontinuities and seems to have a substantial reserve capacity.
- The overall performance can be considered very good despite the previously found soil density deficiencies.
- The facing consisting of flexible geogrids had no bending stiffness but showed only small local and global deformations (marginal in the relevant load range).
- The settlement behaviour of the sill (indirectly assessed by the modulus of subgrade reaction) was as if it had been sitting on an infinite horizontal plane and not near a vertical slope; the only plausible explanation is the apparently high effectiveness of the incorporated geogrids.

The author would have no reservation using the structure as built and tested (and ideally with better fill compaction) in practice.

5 ACKNOWLEDGEMENTS

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