GEOGRID-REINFORCED BRIDGE ABUTMENTS: REPORT ON A FULL SCALE TEST AND EXECUTED PROJECTS

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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.

The paper deals with the study of a geogrid-reinforced soil (GRS) solution for bridge abutments. 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 sill beam and load from the bridge. 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.

The paper will describe the full scale test in detail and present the important results and findings from the test. Furthermore a short summary of an executed project will be given.

1. INTRODUCTION

Steep slopes and walls from geosynthetic reinforced soil (GRS) became very popular and established practice worldwide due to their advantages, which include: cost-effectiveness, good aesthetics and fine-tuning for optimum functionality. The broad range of available geosynthetic reinforcement allows optimisation and eliminates any limits to height and load capacity. It was to be expected that the next step to be taken would be the use of GRS in a "specialist" area, such as, bridge abutments, which are heavily loaded and have to fulfil stringent requirements with regard to load capacity and all types of deformation.

The first steps originated over 20 years ago, when GRS was used to form the front face, or part of the wing walls, of bridge abutments. In such an application the GRS is not directly

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Figure 1. GRS as front and wing wall facing of bridge abutments (not loaded by the sill beam).

Figure 2. Geogrid-reinforced soil block behind walls as earth pressure relief.

Figure 3. GRS as directly loaded bridge abutment.

loaded by the sill beam, since the load from the sill beam is supported by separate conventional load-bearing systems such as reinforced concrete piles or similar (see Figure 1).

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 (see Figure 2). A 10 to 50 cm gap should be left between the GRS front and the back of the concrete wall, which can be either left open (unfilled) or filled with a very compressible material. The GRS only carries loads from the earth pressure of the backfill and traffic load.

The sill beam also can be placed directly on to the GRS abutment (see Figure 3). In this case the geogrid-reinforced soil body has to take the full load from the bridge and the sill beam.

The contact pressures are typical in the range of 150 to 250 kPa, which are transferred via the sill beam. The width of the sill beam is usually smaller than 2 m and located very close to the front of the bridge abutment. Compared to the normal applications of GRS walls and slopes this can be considered as a very concentrated high load at a critical location.

Due to the increased interest in such application and uncertainties regarding deformation behaviour and maximum load level of a GRS under these conditions HUESKER decided to undertake a full scale instrumented loading test.

2. LOAD TEST

A 4.5 m high vertical geogrid-reinforced soil wall was constructed and tested at the LGA Nuremberg, Germany, simulating a bridge abutment (see Figure 4). A heavily reinforced 1.0 m wide concrete block was used as a sill beam, transferring the load from the hydraulic jacks onto the reinforced soil wall.

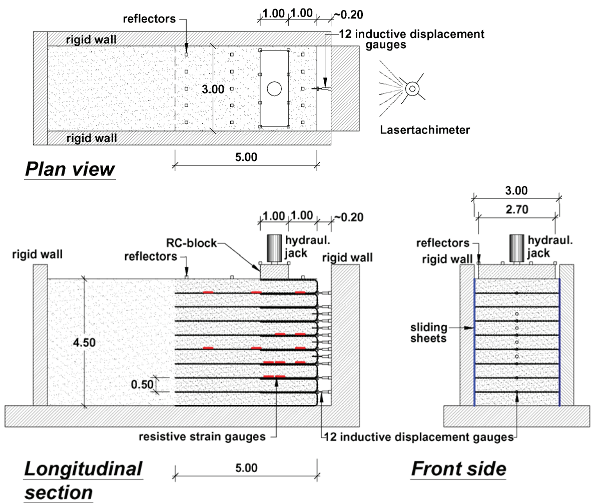

Figure 4. Test set-up and instrumentation of a full scale geogrid reinforced bridge abutment loading test (Alexiew [1]).

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) 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 'wrapped-around', creating a so called "soft facing". The fill was a well graded crushed sandy gravel with an internal friction angle of between 40° and 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. For further details of the test set-up and instrumentation see Alexiew [1].

The main focus of this test was to obtain the magnitude of horizontal and vertical deformation in the usual contact pressure range of 150 to 250 kPa and the ultimate contact pressure, which would drive the GRS to failure.

Two separate tests were carried out. In test 1 the maximum load was 400 kPa, i.e., twice the contact pressure normally experienced under a sill beam. After each load increment a pause took place until the requirements of the plate bearing tests, in accordance with DIN 18134, were reached regarding the change of settlement. The aim of test 2 was to take the GRS block to failure using the full capacity of the hydraulic jacks of 650 kPa.

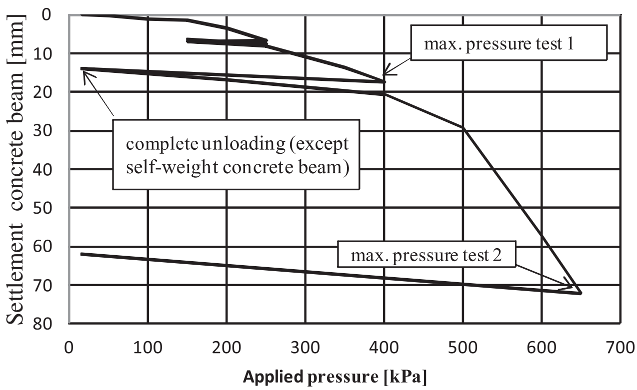

Figure 5. Settlement of the concrete block during the loading test 1 and 2.

2.1. *Important Test Results*

2.1.1. *Settlement of the Concrete Block*

Figure 5 shows the relationship between load and sill beam settlement in test 1 and 2. The shape of the graph at the first two loading-unloading cycles suggests that a certain amount of further compaction takes place between 100 and 250 kPa. The settlements at this load stage are in the range of only 5–8 mm, even including the further compaction at the beginning.

At the maximum pressure of 400 kPa at the end of test 1 the observed settlement was around 17 mm. No failure indication was observed at this load stage.

In the second test the load was increased up to the maximum capacity of the two hydraulic jacks, 650 kPa. At approximately 450 kPa several fine vertical cracks are visible on the bottom edge of the heavily reinforced concrete block, whilst in the GRS wall itself still no failure indication was visible. Beginning at 500 kPa a significant increase in settlement was observed. Up to 600 kPa there were no recognizable symptoms of failure. Between 600 and 650 kPa a small irregular crack finally appeared in the fill surface behind the concrete block and extended towards the rear along the test pit walls. At 650 kPa 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 of soil slipping forward, and downward, as might be expected, never occurred.

2.1.2. *Horizontal Facing Displacement*

Figure 6 shows the horizontal facing displacements for load test 1 and 2. The maximum displacements occurred at the highest measurement point, up to a pressure of 400 kPa, and in both tests amounted to a maximum of approx. 10 mm. From around 500 kPa (i.e., in test 2) the character of the distribution of the deformation changed - the maximum values were no longer at the top edge. A “global bellying out” is 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 corresponded fairly accurately to the area of the strip load on the top projected down to the right at about 45° to meet the facing. All this

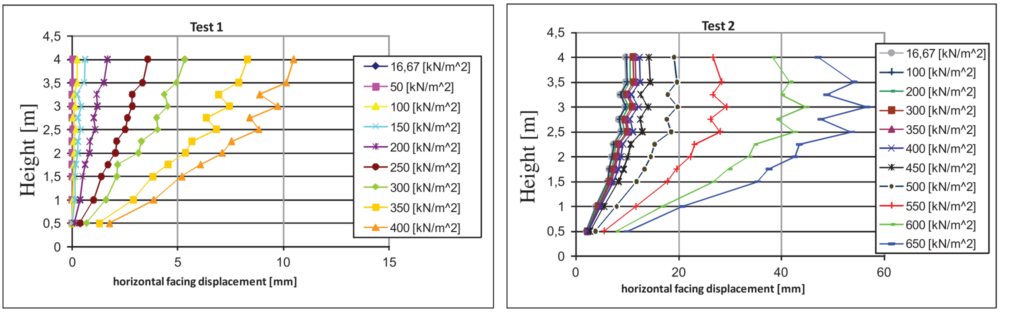

Figure 6. Horizontal facing displacement for load test 1 and 2 (not all load steps are shown).

appears very plausible and corresponds well with common earth pressure theories. The maximum displacement of the soft facing was achieved at 650 kPa at a fairly large value of 56 mm, but under an extreme beam load. From a beam pressure of approximately 500 kPa, Figure 6 (Test 2) clearly shows an increase in the rate of deformation. The (relatively) large displacement from approximately 550 kPa could be taken as a trend in the direction of failure, however up to the end of the test at 650 kPa 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 geogrid-reinforced soil.

2.1.3. *Final Remarks on the Test Results*

The tests presented here on a geogrid-reinforced soil block simulating a real bridge abutment under a sill beam are in no way intended to be a comprehensive detailed scientific analysis. The exercise is much more about testing the behaviour of a system and its performance reserves in a situation that can be related to practice. 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, since the sill beam was only 1 m wide and placed only 1 m away from the edge of the vertical front face, being built as geogrid-wrapped-back wall without any form of stiffening elements in the front. The degree of compaction of the fill in the most sensitive upper zone was only $D_{pr} = 95\%$.

The following remarks can be made:

- A contact pressure under the sill beam of up to 650 kPa (approx. 3 times that normally experienced) led to no obvious component or system failure. However, because there were signs of serious effects taking place, this may be used as a marker for the ultimate limit state.
- A contact pressure of up to 400 kPa (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 to be very good despite the previously found soil density deficiencies.

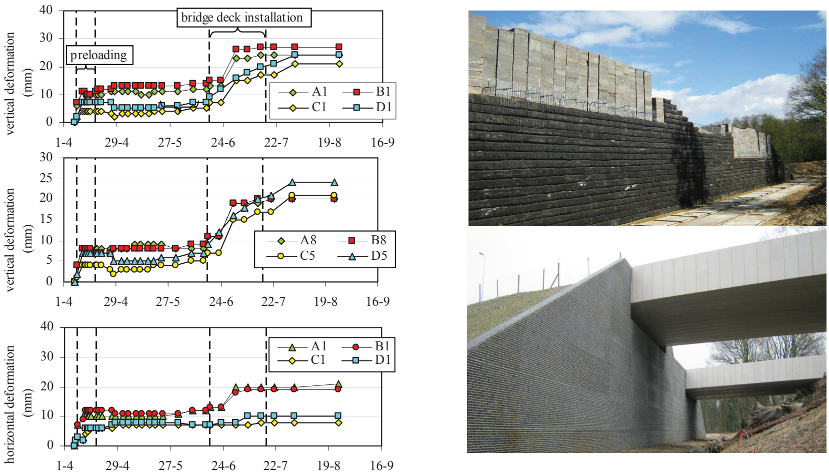

Figure 7. Measurement data of horizontal and vertical deformations during preloading and bridge deck installation as well as picture from the preloaded retaining wall and finished abutment with installed bridge deck (van Duijnen *et al*. [2]).

- The facing consisting of flexible geogrids had no bending stiffness but showed only small local and global deformations (marginal in the relevant load range).

3. CASE STUDY

A new direct connection (A74) between the Dutch A73 and the German Bundesautobahn 61 (BAB 61) in the area of Venlo, the Netherlands was constructed. Part of the construction consists of two 'ecoducts' that guarantee the ecological connection between the north and south side of the road. The left abutment of the viaduct was constructed as geogrid reinforced retaining wall with a max. height of around 10 m. After finishing the retaining wall, but before installing the sill beam and the bridge deck, a preload is applied to activate the initial deformation of the retaining wall. After preloading and finishing the construction of the viaducts, the retaining wall is covered by the Muralex® facing system (gabion like facing) for protection and aesthetical reasons. The horizontal and vertical deformation of the wall were monitored by 26 markers until two months after installation of the bridge, see Figure 7.

The figures show that all vertical deformations are nearly in the same order, both in the top as in the bottom of both embankments. This means that the settlements below the retaining walls dominate. The vertical and horizontal deformations are within in the expected range.

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