Load Tests on a 1:1 Model of a Geogrid-Reinforced Bridge Abutment

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ABSTRACT: The paper deals with geogrid reinforced soil as solution for bridge abutments. Preliminary results are presented of a real scale test of a simulated 4,5m high geogrid reinforced vertical soil block loaded directly on top near the edge by the bridge sill beam, high-lightening the low settlements and horizontal displacements measured. In one test, the reinforced embankment was nearly lead to rupture, what occurred with a load in the order of 3 times the usual one for this kind of structure.

RESUMO: O artigo lida com a solução de solo reforçado com geogrelha para encontros de ponte especificamente para o problema da transição do aterro para a superestrutura. São apresentados resultados preliminares de ensaios em escala real simulando um aterro reforçado com geogrelha com 4,5m de altura carregado diretamente no topo pela estrutura da ponte, destacando-se os baixos valores de deslocamentos verticais e horizontais medidos. Em um teste, o aterro reforçado foi levado à ruptura, o que ocorreu apenas para uma carga da ordem de 3 vezes a usual para este tipo de estrutura.

KEY-WORDS: Geogrid, Reinforced Soil, Bridge Abutment.

1 INTRODUCTION

Steep slopes and walls from geosynthetic reinforced soil (GRS) became very popular and established practice not only in Europe 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. It was to be expected that the next step to be taken would be 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”, with all the related ways of thinking, traditions, experiences and backgrounds.

2 BRIEF OVERVIEW AND BACKGROUND

The first steps were taken rather cautiously about 20 years ago: They began with the use of GRS to form the front face or part of the wing walls of bridge abutments. Engineers made use of their experience with “conventional” GRS slopes and walls. Further information can be found in e.g. (Herold & Alexiew 2001), (Herold 2002) and (Alexiew 2005).

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. Although the task is still relatively complex in terms of design, detailing and execution, 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) (Fig. 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 (Fig. 2). A 10 to 20 cm vertical gap should be left between the

Figure 1. Bridge abutment walls on the Via Baltica near Riga with Terrae Blocks

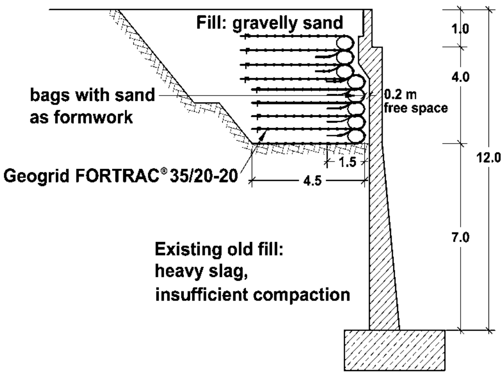

Figure 2. Relieving abutment walls Jerkovo, Bulgaria

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 (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 large contact pressures under the sill of the order 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 as part of a road bridge over a railway line (Kirschner & Hermansen 1994) (Fig. 3). 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.

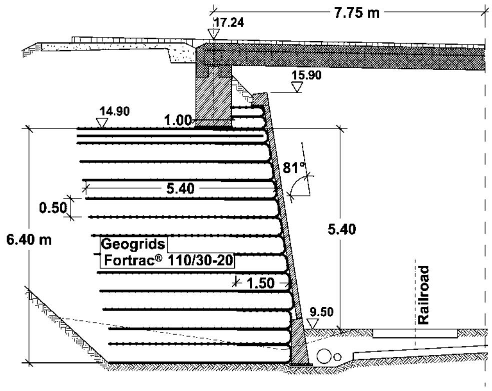

Figure 3. Bridge abutment at Ullerslev, Denmark

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 breakthrough occurred with a GRS bridge abutment in Germany. The first "real" GRS abutment in the German highway network was constructed at Ilsenburg on the German National Road K 1355 (Herold 2002) (Fig. 4) and incorporated a very much more extensive programme of monitoring than was installed at Ullerslev. Geometry and height are generally more modest than at Ullerslev, however the knowledge gained is of great importance, including the "psychological" aspects on the part of the client.

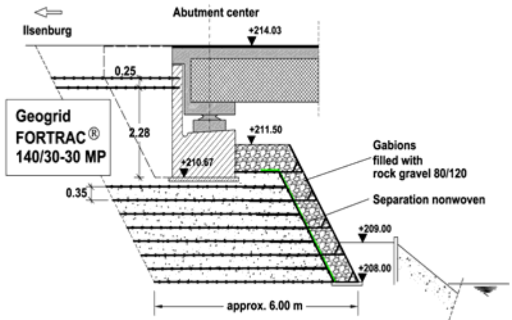

Figure 4. Bridge abutment on the Highway K 1355, Ilsenburg, Germany

Then bridge designers also became directly involved and showed increased interest. Professor Pötzl has introduced a highly interesting research programme with the experimental part being carried out in cooperation with HUESKER, Germany. The idea was to install a GRS block with vertical facing at the back of an abutment front wall to permit this wall to move freely inwards and outwards due to temperature changes of a jointless bridge. The GRS wall was 5.0 m high, made from a well graded crushed sandy gravel and reinforced every 0.5 m with layers of Fortrac^{R} 80/30-35 M geogrids from PVA (Alexiew 2000) with wrapped-back ends. It was loaded only horizontally at the front face by the back and forward movement of a stiff slab, which simulated the temperature-related movements mentioned above. Soft EPS blocks were installed in the gap at the front of the geogrid facing. No vertical loads were introduced into the system. The system is proving to be very suitable and a practical implementation is recommended (Pötzl & Naumann 2005a, 2005b).

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

3. TEST WALL AS A "REAL" BRIDGE ABUTMENT

3.1. TEST SET-UP AND COMMENTS

After the above test series for the "jointless bridges" the test wall had stood in a quasi-original condition in the test pit for more than a year, it was suggested that the structure could serve as a 1:1 model for testing a "real" bridge abutment, i.e. vertical loads could be applied directly from a reinforced concrete beam acting as a sill beam. The emphasis of the proposed investigation was not on the "internal life" in the sense of e.g. internal stresses and strains. The investigation was to concentrate much more directly on the overall behaviour of the structure:

- How large are the settlements of the sill in the usual loading range of 150 to 250 kN/m^2 (very important) and how large are the displacements of the facing (there importance depends on the facing system); (serviceability limit state - SLS).
- 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).

A dangerous tendency to predominantly or 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 <u>not</u> in any respect relevant to safety.

For the purposes of this new testing programme a rigid slab was fixed in position at a certain distance in front of the GRS; the soft EPS blocks were removed (Fig. 5). This meant the structure remained a GRS structure with a free vertical facing of the type "wraparound wall" without any kind of stiffening elements. Two layers of a smooth membrane between the fill and the side walls of the test pit were installed to minimise friction between the soil and the concrete: A 1.00 m x 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. 5 & 6). Twelve displacement transducers were attached to the facing to measure its horizontal displacement at points at the mid-height of individual "pillows" and at the heights of the geogrid layers, i.e. in there contact surfaces (Fig. 6).

Reflectors were attached over the whole surface and to the RC-beam to measure the settlements and any tilting. A precision level was used to record their displacements (Fig. 5).

A data acquisition system recorded all load and deformation data using a high frequency reading cycle. In addition the most important data were displayed numerically and graphically on monitors in real time. Figure 7 shows the test set up and Figure 8 shows a view of the facing with the displacement transducers.

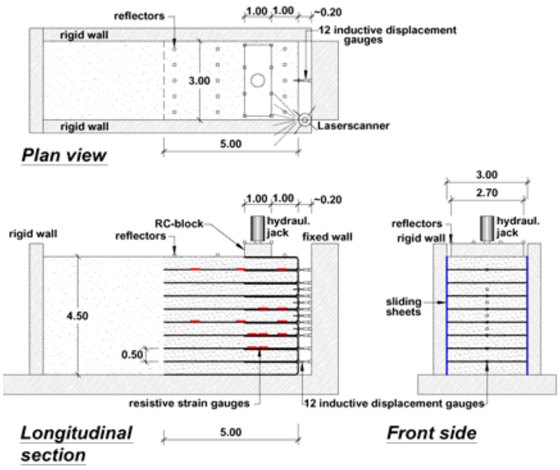

Figure 5. Schematic of the test

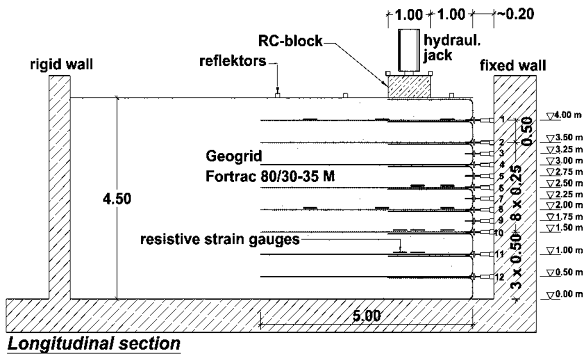

Figure 6. Horizontal displacement transducers

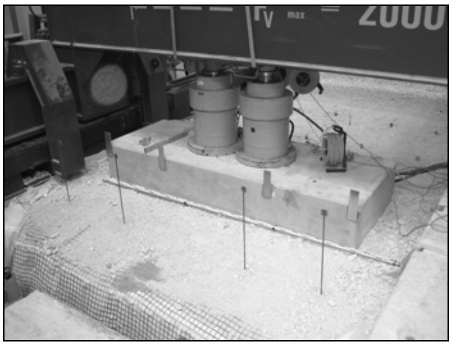

Figure 7. Overview of the test set-up

Figure 8. Geogrid wrapped-back facing

3.2. CONSTRAINTS AND BOUNDARY CONDITIONS

The equipment available and the history of the project imposed certain constraints and boundary conditions to the tests. The height had to be reduced to 4.50 m. Furthermore the proper functioning of the resistive strain gauges installed previously was called into question. The degree of compaction of the fill was approximately only 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 mentioned above. The pressure under the loading RC-block (sill) could be taken up to a maximum of about 650 kN/m^2, which nevertheless represented about 3 times the pressure normally experienced.

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. Two loading-unloading cycles between 100 and 250 kN/m^2 were applied in this test. The system was completely unloaded after reaching 400 kN/m^2. The load increments can be seen in Figure 9. After each increment a pause took place until there was a reduction in the increase in settlement in compliance with the criteria for plate bearing tests in accordance with DIN 18134.

The aim of Test 2 was to take the GSR block to failure. The loading steps can be seen in Figure 10, on which the system was brought up to the maximum applicable stress of approximately 650 kN/m^2 before being completely unloaded again. Following values were automatically recorded (cf. Section 3.1): contact pressure (via the jacks), average settlement of the loading block (wire extensometer), horizontal displacement of the facing (displacement transducers) and the strain of the Fortrac 80/30-35 M geogrids using the resistive strain gauges. The signals of the latter appeared to be very unreliable and were not taken any further. After reduction of the rates of settlement at every load increment or decrement stage all settlements of the reflectors on the top of the GRS system and on the RC-block (cf. Section 3.1) were measured with a precision level to an accuracy of 0.1 mm, from which settlements, settlement profiles and any tilting of the block were determined.

3.4. IMPORTANT TEST RESULTS AND COMMENTS

For reasons of brevity and expediency only the most important results relevant to practice and to the assessment of the system as a "real" bridge abutment have been included herein. Figure 9 shows the relationship between load and beam settlement in Test 1 unsmoothed. The load increments and the measured settlements can be seen in the graph.

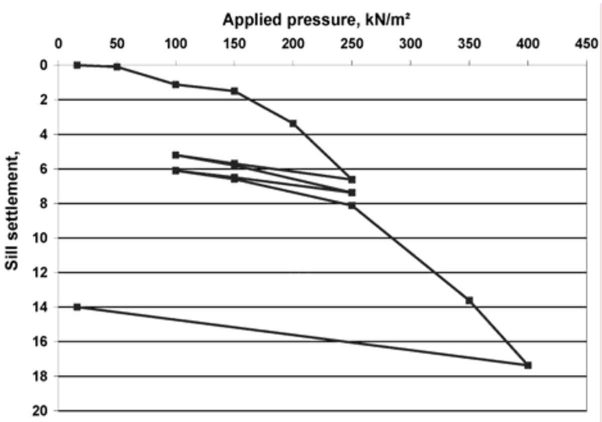

Figure 9. Settlement of the loading block in Test 1

The shape of the graph suggests that a certain amount of further compaction may be 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 of the GRS system near the beam may have occurred as a result of the horizontal loading of the front area in the earlier "jointless bridges" test (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 approximately 5.9 mm and in the second loading cycle approximately 1.3 mm. Under otherwise identical boundary conditions, this indicates an increase in compressive stiffness of 5.9/1.3 = 4.5. This is an unusually high increase in stiffness and indicates an additional compaction; a value of about 2.0 would have been expected here with well compacted, non-cohesive, well graded soil. There would appear to be two possible reasons for the increase in stiffness in Test 1: Recompaction of the fill directly under the block and perhaps in possibly loosened front area and a higher mobilisation of the reinforcing geogrid in combination with this recompaction. In any case the conclusion is that with a priori good compaction in the critical area under consideration (which would also bring with it greater geogrid effectiveness in the composite system) the sill beam settlement would be even less than 5 - 6 mm in the relevant loading range, although even 5 - 6 mm would satisfy normal requirements without further action.

It should be noted that on Figure 9 the "unloading" graph 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 successful recompaction and full mobilisation of the system.

Figure 10 shows Test 2 in a similar way to Figure 9. The first point starts at the residual settlement of 14 mm remaining after unloading in Test 1.

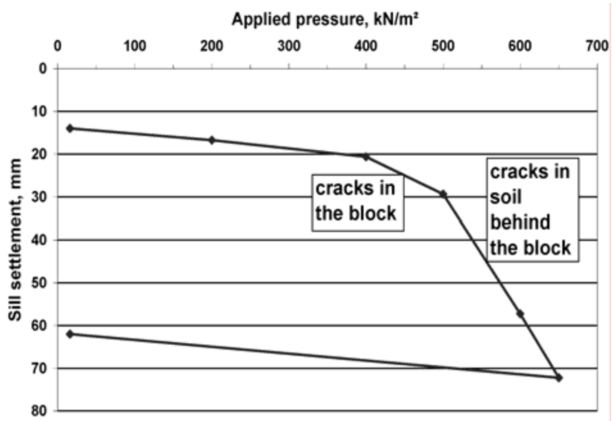

Figure 10. Settlement of the loading block in Test 2

The results for Tests 1 and 2 can be converted into a modulus of subgrade reaction 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 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 to35 MPa/m (similar to 16 to 31 MPa/m here in Test 1) (Alexiew et al. 1989). Important: The common values 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 modulus to that of a plane is achieved on the top of a geogrid-reinforced vertical soil block with a strip load applied close to its edge (whether compacted or not) shows that the reinforcement used is acting very efficiently. In Test 2 at approximately 450 kN/m^2 several fine vertical cracks were visible on the bottom edge of the heavily reinforced RC block; whilst in the GRS wall itself there was still nothing significantly amiss. Only from 500 kN/m^2 was there a significant increase in settlement. Up to 600 kN/m^2 there were no recognisable 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 loading block 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 of soil slipping forward and downward as might be expected, never occurred. It is a question of interpretation as to whether the ultimate limit state was reached or not.

Now to refer briefly to the horizontal displacement of the measured points on the "soft" (see Sections 1 & 2) facing. Figures 11 & 12 show the results of Tests 1 and 2 (polygonal, not smoothed; say the plots do not show the true shape of the facing; the real single layers are more "rounded", see e.g. Figure 8).

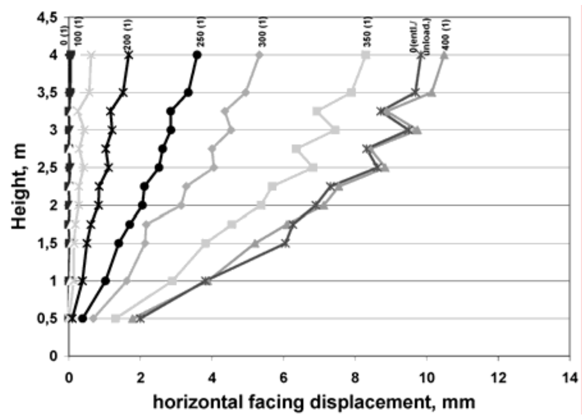

Figure 11. Displacement of the measured points on the facing for Test 1

It should be noted that the GRS structure 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 measured points directly at the planes of the reinforcement (see Figure 6). However, the increases in local bowing out were very small, probably because of the tensile stiffness of the flexible Fortrac 80/30 - 35 M geogrid used. In Figure 12 (Test 2) the residual displacements after unloading of Test 1 (Figure 11) are taken into account.

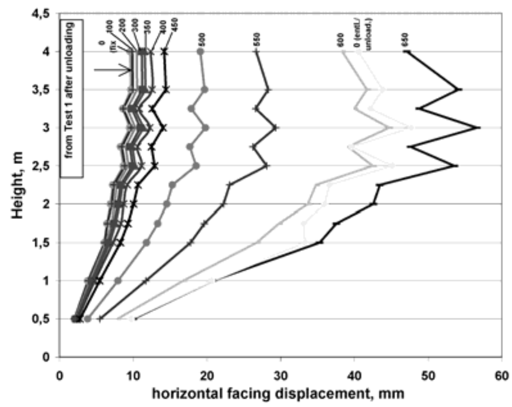

Figure 12. Displacement of the measured points on the facing for Test 2

The maximum displacements occur up to a pressure of 400 kN/m^2 at the highest measurement point and in both tests amounts to a maximum of ca. 10 mm (measured in the worst position at a "bellying out" point at a height of 4.0 m, the height of the top edge of the GRS system was at 4.50 m and contains a restraining geogrid return, see Fig. 6). As with

the “vertical” behaviour of the system (see above), the graphs on Figures 11 and 12 also record an increasing stiffness in the horizontal direction after “recompaction”.
For reasons of brevity a more detailed analysis of this and the relationships between the “vertical” and “horizontal” 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 at the top edge. 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 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 appears very plausible and corresponds well with common earth pressure theories. The maximum displacement of the soft facing is achieved at 650 kN/m^2 (at the bowing out at the front between two planes of reinforcement at H = 3.0 m) was a fairly large value of 56 mm, but under an extreme beam load. However, the local bowing of the geogrid at the front between the neighbouring planes of reinforcement was still only approximately 9 mm. Measured at a layer spacing of 50 cm (between the reinforcement planes) this displacement represents a good low value and indicates that the wraparound geogrid is highly efficient. From a beam pressure of approximately 500 kN/m^2, Figure 12 (Test 2) clearly shows an increase in the rate of deformation. The (relatively) large displacement from approximately 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 geogrid-reinforced soil; on removal of the load from 650 kN/m^2 to zero and despite the initial indications of failure the front face moved back approximately 10 mm. Viewed in plan the front remains straight with no bowing in the middle; evidently the slip 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 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, from the point of view of “we want to know”. The use of an already constructed test object after modification was advantageous in terms of time and money, however, it also brought its own restrictions and deficiencies (Section 3.2), including that we would have to live with the known, somewhat insufficient, compaction in the upper layers and the possible looser fill zones in the front face zone resulting from the tests done for other purposes. 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 outer skin (facing) had no bending stiffness, being only a geogrid-wrapped-back wall, without any form of stiffening
- The degree of compaction of the fill in the most sensitive upper zone was only D_{pr} = 95%, with probably loosened zones in the front face 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 marker 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 loading beam (indirectly assessed by converting 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 highly effectiveness of the incorporated reinforcement and the geogrids used.

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

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