Reinforced Earth Structures to Relieve Walls of Earth Pressure

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ABSTRACT: Earth pressure on walls or concrete retaining structures can reach significantly high values for high structures. The reduction of earth pressure results in a reduced stress on these structures. The concept described in this paper is to reduce the earth pressure by means of wrap back geotextile reinforced earth structures (GRE). GRE structures are a well established construction method. By means of these walls the earth pressure can be reduced to zero, leaving a gap in between wall and GRE. The paper presents two case studies, where wrap back walls have been used behind walls to reduce the earth pressure. For this application the deformation behaviour of the GRE is very important. Some examples are given, showing the huge capacity of GRE in load bearing combined with small deformation. Deformation measurements of laboratory tests as well as on executed projects are shown. At the end of the paper the benefit of an earth pressure elimination resp. reduction is shown on a virtual example.

Keywords: Earth pressure relief, geotextile reinforced wall, deformation behaviour

1 INTRODUCTION

Earth pressure on walls or bridge abutments can be a driving component in the design. Especially for high structures the issue can become very costly. The reduction in earth pressure leads to a reduction of the loading and therefore of the required cross section of the wall. Furthermore deformations are reduced significantly, since a reduced or no earth pressure is acting.

The construction of a reinforced earth wall with a wrap back facing represents a meanwhile well-established method of reducing earth pressure. By leaving a gap of 10 to 50 cm between the concrete wall and the reinforced earth structure, the earth pressure on the facing wall is reduced to zero. For this application it is mandatory that the gap remains over the time, i.e. no big deformation of the geotextile reinforced earth wall is allowed to occur.

2 LOAD BEARING AND DEFORMATION BEHAVIOR OF WRAP BACK WALLS

Due to their economical, technical and ecological advantages, geosynthetic reinforced earth 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 experiences gained within the last years and the wide range of available geosynthetic reinforcements even resulted in the use of the first geosynthetic reinforced earth structures as bridge abutments, which do experience very concentrated and heavy loads and have to fulfill stringent limitations concerning their deformation behaviour. In this application the sill beam of the bridge is placed directly on to the GRS abutment, (see Figure 1). The geogrid-reinforced soil body has to take the full load from the bridge and the sill beam.

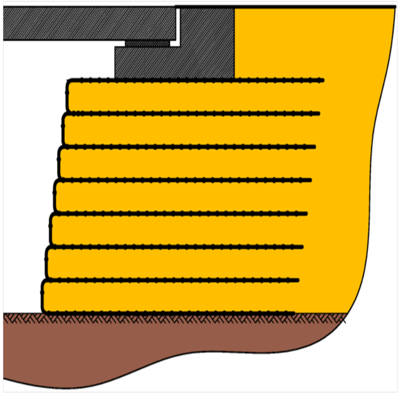

Figure 1. Sketch showing a geotextile reinforced earth structure as directly loaded bridge abutment

Two examples are shown in figure 2. In the picture on the left a temporary bridge is supported by geotextile reinforced abutments, at both ends one and another support in the middle. On the right side a permanent support of a pedestrian bridge can be seen. In both cases there are no stiffening elements at the front of the wrap back structures. On the left the undisguised GRE can be seen, whereas on the right side the wrap back structure is covered by the facing system Muralex, which gives the wall a gabion-like look.

Figure 2. Temporary and permanent bridge abutments in the Netherlands

Before these bridge abutments were constructed it was necessary to gain sufficient knowledge and confidence about the applicability of GRE for the use as support structures of bridges, with regard to their bearing and deformation behaviour. Therefore a real scale loading test was performed.

2.1 *Real Scale Loading 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 3). A heavily reinforced 1.0 m wide concrete block was used as sill beam, transferring the load from the hydraulic jacks onto the reinforced soil wall.

This concrete block was placed only 1.0 m away from the back of the vertical wall. The wall was reinforced by 9 layers of PVA geogrid (Fortrac 80/30-35M) with an ultimate tensile strength of 80 kPa. 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 a friction angle of $\varphi' = 40°$ to 45°, depending on the compaction grade. Various monitoring devices were installed on top and in front of the wall in order to monitor vertical deformations of the wall surface and horizontal deformations of the wall facing, during the test (the installed strain gauges did not work properly and the measurement readings have been contradictive. Therefore they have not be used in the evaluation of the system behavior). For further details see Alexiew [1].

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

Two separate tests were carried out. In test 1 the maximum load was defined to 400 kPa, i.e. twice the contact pressure normally experienced under a sill beam. Each load step was maintained in accordance to the requirements of plate bearing tests regarding the change of settlement of DIN 1813. The aim of test 2 was to drive the GRS block to failure using the full capacity of the hydraulic jacks of 650 kPa.

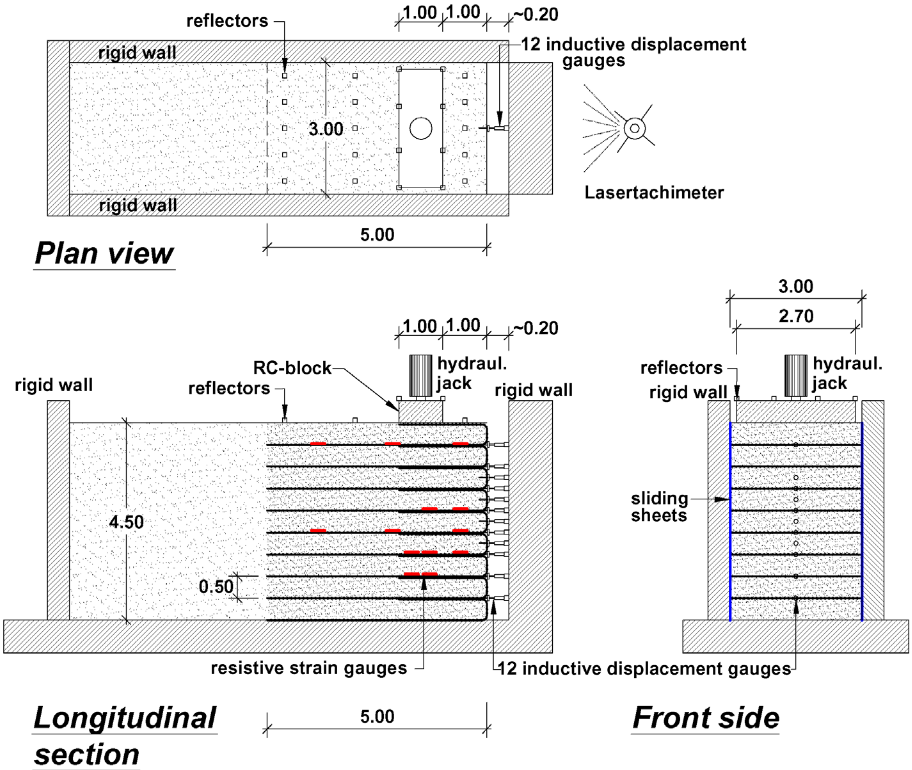

Figure 3. Test set-up and instrumentation of a full scale geogrid reinforced bridge abutment loading test (Alexiew [1])

Figure 4 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 a settlement of around 17 mm was observed. No failure indication was observed at this load stage.

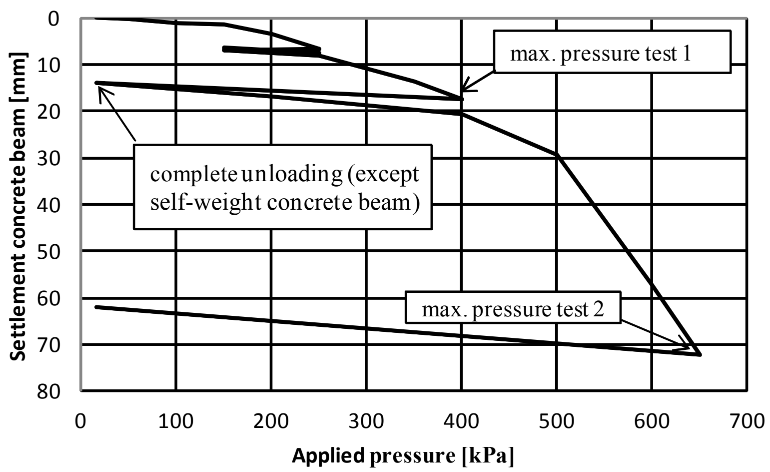

Figure 4. Load settlement curve of the GRE

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 became visible on the bottom edge of the heavily reinforced concrete block, whilst the GRE wall itself showed no failure indication. 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 appeared in the fill surface behind the concrete block which 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.

2.1.1 *Horizontal Facing Displacement*

Figure 5 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) on 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".

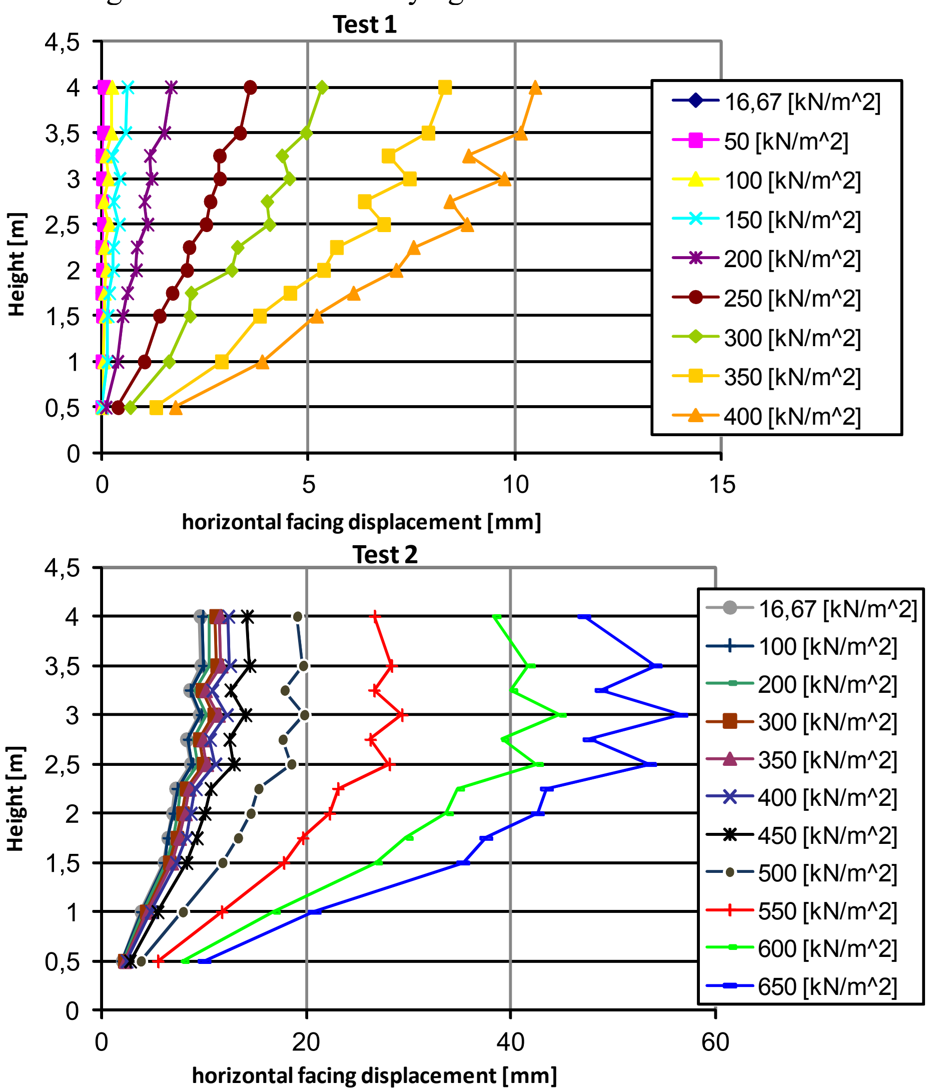

Figure 5. Horizontal facing displacement for load test 1 and 2 (not all load steps are shown)

The position and height of this zone correspond to the area of the strip load on the top projected down to the right at about 45° to meet the facing. The monitoring results are very plausible and correspond 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 5 (Test 2) shows an increase in the rate of deformation. The relatively large displacement from approximately 550 kPa can be taken as a trend towards 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 speak for themselves as to the remarkable reserve capacity of the geogrid-reinforced soil.

2.2 *CASE STUDY*

Similar experiences have been made in the following case study. A new direct connection (A74) was constructed between the Dutch highway A73 and the German Bundesautobahn 61 (BAB 61) in the area of Venlo, Netherlands. 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 was applied to activate the initial deformation of the retaining wall. After preloading and finishing the construction of the viaducts, the retaining wall was covered by the Muralex® facing system (gabion like facing) for protection and aesthetical reasons. The horizontal and vertical deformations of the wall were monitored by 26 markers until two months after installation of the bridge, see figure 6.

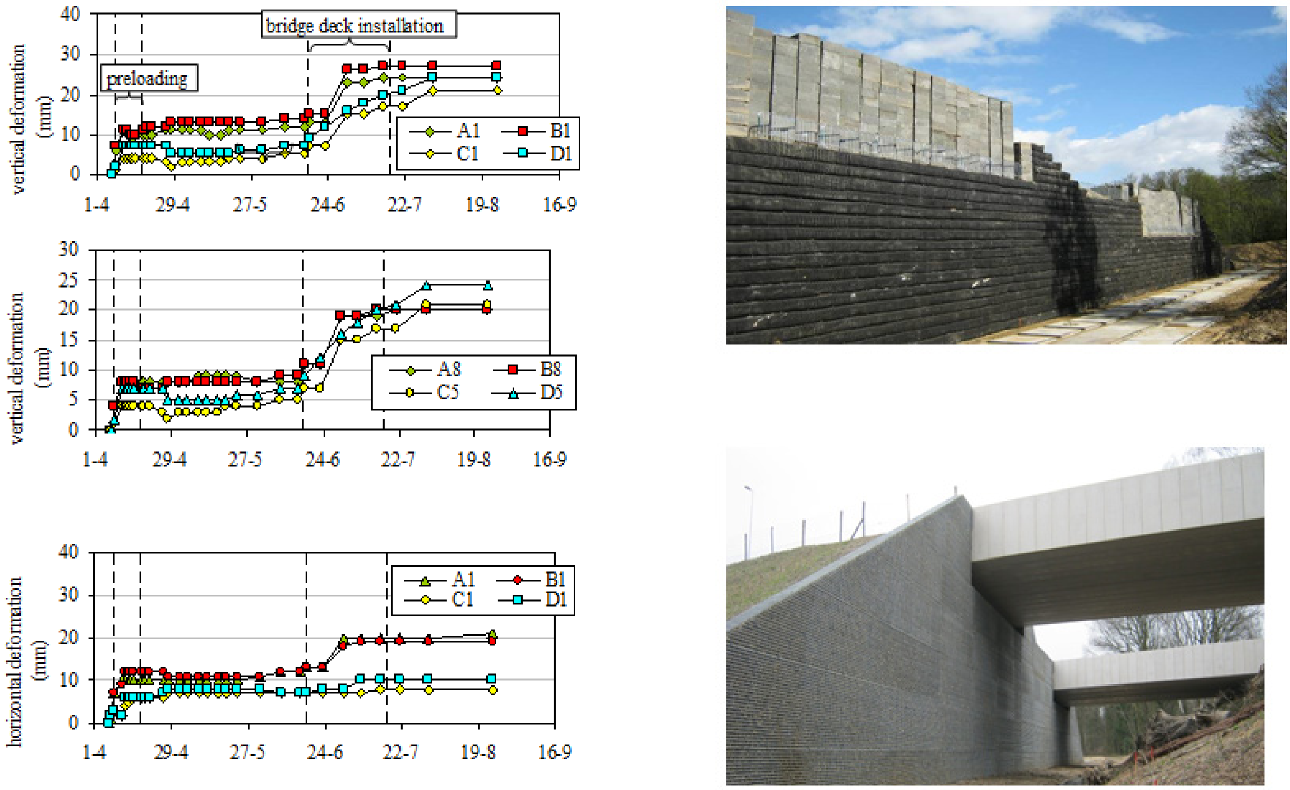

Figure 6. Measurement data of horizontal and vertical deformations during preloading and bridge deck installation as well as pictures of the preloaded retaining wall and finished abutment with installed bridge deck (van Duijnen *et al*. [2])

The monitoring data show that the vertical deformations are nearly in the same order, both at the top as at the bottom of both embankments. This means that the settlements below the retaining walls dominate the vertical deformation, not the settlement within the wall itself. More important though in regard of the application of GRE for the purpose of earth pressure reduction are the horizontal deformations, which are at the maximum around 2 cm for the 10 m high GRE under full load.

2.3 *Conclusions*

The real-scale test as well as the measurement data of the executed project demonstrate the high bearing capacity in combination with low deformations of the GRE structure, especially in horizontal direction. This is a very important issue, when using these kinds of walls to reduce or eliminate the earth pressure by leaving a permanent gap between wall and GRE. It has to be emphasized that a proper compaction of the GRE is a precondition for the low deformation behavior.

3 CASE STUDIES OF GEOTEXTILE REINFORCED WALLS TO REDUCE EARTH PRESSURE

Figure 7 shows the principle of such an application. Between the retaining structure and the GRE a gap is kept and therefore the earth pressure on the retaining structure is eliminated completely.

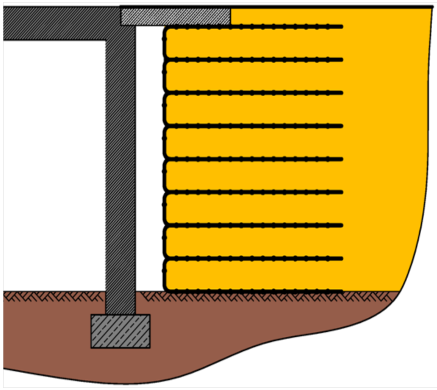

Figure 7. Principle sketch of earth pressure relief

3.1 *Earth pressure relief and shock wave protection*

An impressive example has been constructed in the Netherlands at the BP refinery. To protect the new office building in case of an explosion of storage tanks, an embankment was built around the building. In case of emergency (explosion), this embankment shall lead the shock wave above the building. To reduce the earth pressure of the 14 m high embankment on the building, a wrap back wall was constructed leaving a gap of 50 cm, as shown in figure 8.

Figure 8. 14 m high earth pressure relief at an office building using GRE to leave a gap between earth embankment and wall

3.2 *Over-steep GRE as Earth Pressure Relief on Concrete Walls along an Ecoduct*

Another impressive example was the construction of an ecoduct to allow animal crossing with a slender and over-steep concrete structure. This structure was not ment to take any earth pressure. Therefore again a wrap back wall, this time with steel elements in the front due to the over-steep facing of 110°, was used. Figure 9 gives an impression of this project. Again the gap between the GRE and the concrete structure is clearly visible.

Figure 9. Oversteep GRE (110°) as earth pressure relief on slender concrete walls.

3.3 *Parameter Study*

A parameter study was carried out in order to evaluate the efficiency of the reduction of earth pressure by means of GRE constructions.

This study is based on virtual boundary conditions. The constructions considered can be used either as retaining or abutment walls. The study includes an analysis of a total reduction of earth pressure in relation to the height of the retaining/abutment wall. Furthermore the effects of a percentage reduction of earth pressure was examined. The results can be summarized as follows.

Figure 10 yields that the potential economical savings (in relation to the required reinforcement of a concrete wall) rise with increasing height of the wall. It can be seen that with rather small heights of up to 5 m the savings of reinforcement gained by a reduction of the earth pressure are about 20%. With increasing height of the retaining wall the economical effect of an earth pressure reduction becomes obvious; the savings are rising significantly.

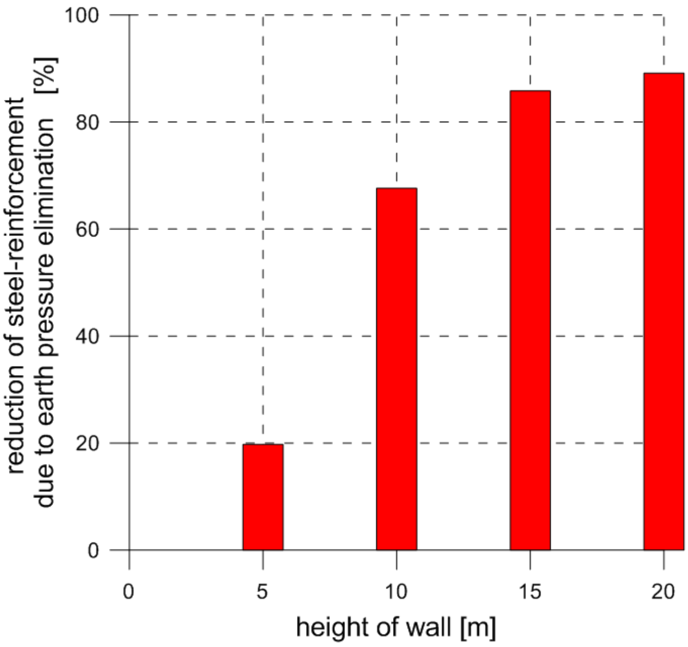

Figure 10. Reduction of steel-reinforcement due to earth pressure elimination according to the assumed height

Figure 11 yields that the cost (by reduction of the cross section of the concrete and reinforcement) is rising with increased earth pressure reduction. The displayed example shows that with a retaining wall of about 6 m height and a friction angle of φ' = 30° (cohesion c' = 0 kPa) of the filling/soil behind the wall a cost reduction of about 50 % can reached.

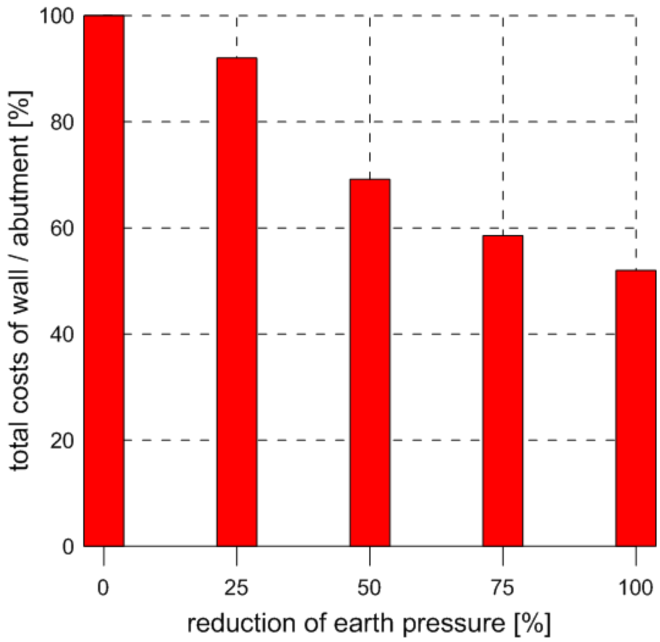

Figure 11. Total costs according to the reduction of earth pressure

4 CONCLUSION

GRE walls are capable to bear great loads with low deformation. This can be seen in the presented measurement results of the real-scale test as well as the instrumented bridge abutment. Two elected case studies were presented where GRE structures have been used to reduce the earth pressure on the main building such as an 14 m high wall of an office building as well as 9 m high oversteep (110°) slender concrete wall of an ecoduct. A parameter study shows, that the total costs of a 6 m high retaining (concrete) structure or an abutment can be reduced by about 50% due to the effect of earth pressure elimination by using a GRE structure.

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