Two measurement programs of a "classic" and of a "novel" geosynthetic structure

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ABSTRACT: The paper handles two measurement programs and their results. The first program was applied to a highway embankment on soft soils with basal high-strength reinforcement at Grossenmeer in Germany. The embankment was built in summer 1986, the use of basal reinforcement (being today "classic"), was at that time a novelty. Among others strains in the woven were measured from the same beginning and continued under traffic for many years. Most important results were published in the 90ies and in 2010. The measurements continued until 2016, i.e. over totally thirty years. To our knowledge this is the longest measurement program on geosynthetic reinforcement ever performed. Latest results for seven years more after 2010 are published and commented. The second program is significantly shorter, but applied to a novel system of geosynthetic encased columns (GEC) installed to reduce lateral trust of soft subsoil behind a highway bridge abutment at Berne in Germany. Among others settlements and horizontal stresses at four depths were registered; the latter is a focal point here. Construction and measurements at Berne started in January 2013 and continued under traffic until September 2016, i.e. over three years and a half. Intermediate data until July 2015 were published in 2016. The present paper includes latest results (i.e. for more than one additional year under traffic) and comments.

Keywords: soft soil, embankments, basal reinforcement, geosynthetic encased columns (GEC), lateral stress relief, measurements

1 INTRODUCTION

In Germany for the construction of highways and autobahns on soft soils the so called preloading method in combination with strong basal geosynthetic reinforcement is in the meantime a well established procedure. The development was closely connected to the development of design procedures and geosynthetics in Germany.

Since the 70ies, the Federal Highway Research Institute (BASt) has been involved in such projects. Measuring programs have been performed on large-scale test embankments focusing also on high-strength geosynthetic reinforcement (Alexiew, Blume & Hillmann, 2010, BASt, 2005, Blume, 1996, Blume & Hillmann, 1996).

Among others the entire highway B 211 at the town of Grossenmeer was built in 1986 comprising test and measurement sections using a high-strength basal reinforcement (“Grossenmeer embankment”). For the first time strains of the woven were measured as well. The project being followed up until now (30 years) is reported focusing on long-term strains.

The Geotextile Encased Columns (GEC) foundation system for embankments on soft soils was introduced some 20 years ago and is now considered State-of-the-art. The GECs consist of compacted granular fill similar to common stone columns with one decisive difference: they are confined in a high-strength woven geotextile encasement controlling their behavior. Thus, they work properly even in extremely soft soils and a wide range of fills including sand can be used. Recently bridge approaches on soft soils were constructed at the new German Federal Road B212n in the Northwest of Germany as a bypass of the City of Berne. For the first time the GECs adjacent to the piled bridge abutments have additionally to reduce the lateral pressure in depth on the rigid piles. An extensive measurement program was installed. The specifics of the lateral stress relief are described together with the most important measurement data, comments and conclusions.

2 GROSSENMEER EMBANKMENT ON SOFT SOIL

2.1 *Description of the project*

Based on positive results from test and pilot embankments in Germany (Alexiew & Blume, 1999, BASt, 2005) the so called preloading method in combination with strong basal geosynthetic reinforcement was recommended in 1986 for the new federal highway B 211 at the town of Grossenmeer as a first "official" (say non-experimental) project as a long-term permanent application of the concept. To gain additional information for further projects of this type, measurements were performed on two sections: the so called “test” (TS) and “reference” (RS) sections.

The TS was a crash test (4.5 m of sand installed in 4 days with steep 1V:2H slopes).

The RS was a standard one with 1V:3H slopes built up slowly over one year. For the first time direct strain measurements of the high-strength basal reinforcement by strain gauges were foreseen.

Both TS and RS were integrated later on into the final structure.

The weak subsoil consists of layers of peat and organic silts of 3 to 5 m. Stability calculations were performed according to DIN 4084 (Bishop), for details see Blume (1995 & 1996), BASt (2005) and Alexiew & Blume (1999). To achieve the average preloading height of 4.5 m and the required FOS = 1.2 (global stability, temporary stage), a reinforcement tensile force of about 200 kN/m was required (Blume 1995 & 1996). For reasons of deformation compatibility with the compacted sand and to restrain lateral “spreading”, max 5 % strain in the short-term and max 6% total strain (short-term plus creep) for several years under 200 kN/m tensile force were allowed. Based on the isochrones a high-tenacity polyester woven with an UTS = 400 kN/m and max 10 % ultimate strain was selected. The highway (incl. of the integrated TS and RS) was opened to traffic in October 1990. The strain measurements continue until now (see below).

2.2 *Description of the tests*

The main aim of both the TS and RS was to provoke a high stress in the woven, selecting sections having particularly unfavorable subsoil conditions. Under the TS (crash test) extremely high reinforcement stress had to be generated. Nevertheless, failure had to be avoided because of the intended later integration in the standard final highway embankment. At the TS section the focus was more on the short-term behavior under the worst case conditions of first consolidation steps under quick loading. At the RS section (standard profile with 1V:3H slopes, slow construction) the intention was to follow up the strain measurement for a longer period of at least some years.

2.3 *Measurement results and comments*

Details and results for the TS section with "short-term" measurements can be found in Alexiew & Blume (2010). Useful lessons were learned: overestimation of soft soil strength can quickly result in a high stress-ratio of reinforcement than predicted reducing its safety margin significantly or generating a "squeezing" of soft subsoil; weaker local softer spots across the embankment can always appear resulting in local overstressing (the soft soil is not so homogeneous as usually believed). Consequently, a conservative design should be strongly recommended.

In 1993 the strain measurements in the TS were terminated due to adjacent building activities. Thus, only the results of the continued long-term measurements from the “reference section” RS are reported herein. Note that they have started in 1986. Settlement measurements were performed until 2010 (over 24 years), when unfortunately the settlement gauges were destroyed. However, a sufficient number of the strain gauges on the woven are still intact (after 30 years!); read-out of data is still possible using the in the meantime no more up-to-date, but still functioning read-out device from 1986.

Generally, maximum strains between 6.8 % and 7.6 % have been measured.

Figure 1 shows the data for the centerline measurements only representing the maxima of settlement and strains.

So far, at the centre of the embankment, maximum strains of 7.6 % and maximum settlements of between 190 cm and 207 cm have been measured. The strains remain almost unchanged (ca. 7.3 to 7.4 %) after removal of the pre-loading and opening of the traffic in October 1990 until August 2016.

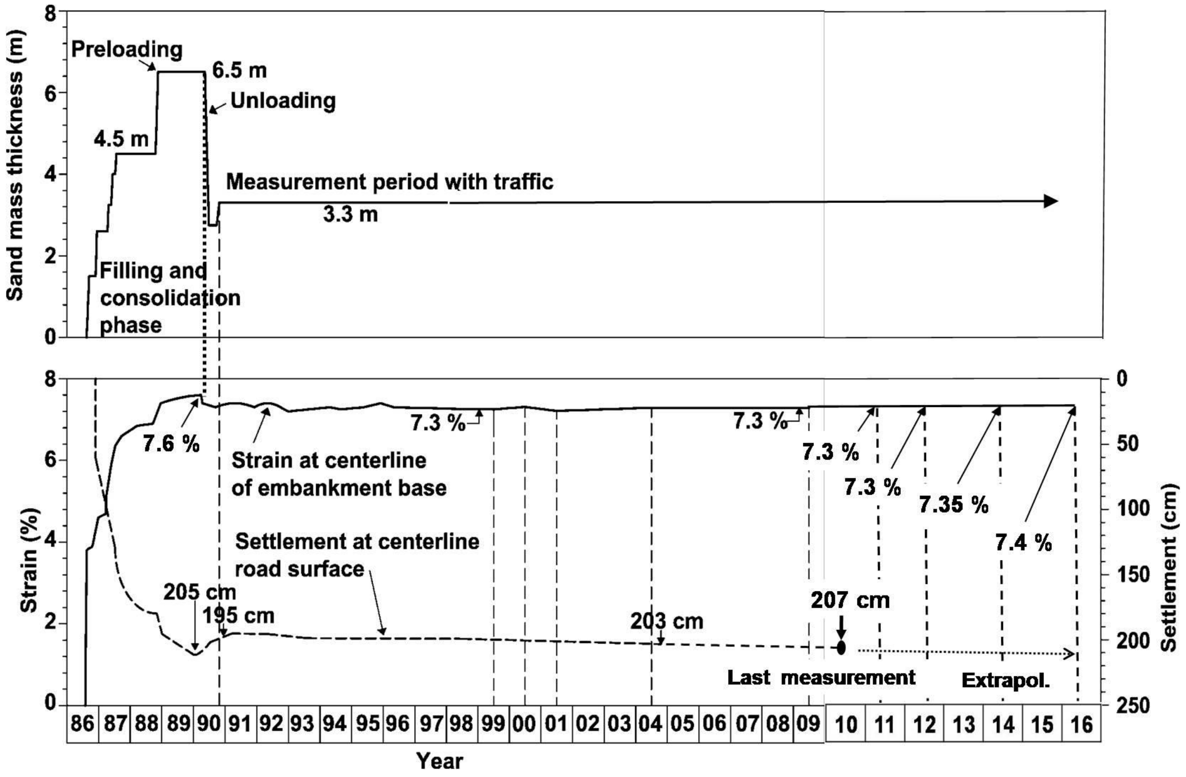

Figure 1. Grossenmeer highway embankment over 30 years (1986 - 2016): embankment height, settlements and strains in the basal reinforcement.

2.3.1 *Settlement behavior*

The settlement rate due to secondary consolidation (creep) from 1990 (constant embankment height, traffic) until the last measurement in 2010 amounts to ca. 0.6 cm/year; it is practically constant (Figure 1). Due to this practical linearity a simple extrapolation to 2016 as shown in Figure 1 seems to be acceptable; the settlement in 2016 should be about 210 cm. That means, a total settlement of about 15 cm over 25 years under traffic has taken place. This is an acceptable value for an embankment without any support (e.g. by piles/columns) or subsoil improvement. There were no problems with trafficability (driving velocity of 100 km/h). Obviously the height and duration of preloading has been sufficient to reduce preventively also long-term settlements. A leveling of the superstructure was done as recently as in 2016.

Main lessons learned:

Pro: Highway embankments of high category associated to generally stringent long-term deformation limitations can be successfully built on very soft soils using the combination of sufficient preloading and proper basal geosynthetic reinforcement.

Contra: A long construction (totally four years) and especially preloading time (here two years) to allow for a sufficient primary consolidation and reduction of later creep settlements. Import and export of a huge fill volume due to preloading.

2.3.2 *Strains in the basal reinforcement*

It is interesting to see how the reinforcement responses to the embankment height/surcharge. It follows in a quite precise way the construction of embankment, the additional preloading and then the unloading (Figure 1). This confirms by the way indirectly once more the applicability of relatively simple design procedures like Bishop for such systems in the sense of reflecting action and reaction.

Some additional strain takes place over time under constant load, obviously due to creep in the woven (look in detail and compare the load and at the strain graphs for 1986 – 1989).

The maximum strain amounts to 7.6% (end of preloading stage). The reinforcement strains exceeded the values of 5 % (short-term) and 6 % (long-term) assumed in design as an average for the project, say for the average subsoil strength. The real mobilized tensile forces in the "reference section" RS handled here were significantly higher; however, this had been intended, see Section 2.2 above.

For the reinforcement used the maximum strain of 7.6% corresponds to a stress ratio of ca. 70% in the short term and to ca. 60% in the long term (inclusive of creep), which is quite high for the geosynthetic used and more or less "at the edge".

It is interesting to compare the proportionality of reinforcement strains to embankment height/load. It is very different during loading and unloading. As a rough comparison (Figure 1): a loading of 6.5 m results in a strain increase of 7.6% (1,2% per 1 m embankment height); the unloading of 3.2 m results in a strain decrease of 0.3% only (0.1% per 1 m height). Obviously the strains remain quasi "imprinted" in the system. A possible explanation: the bond to the strongly deformed embankment base and subsoil (?) plus significant viscous-plastic behavior (?). Note, that so high tensile forces are definitely no more needed for equilibrium/stability after halving the embankment height and for higher subsoil strength after consolidation.

The practically constant strain of 7.3% to 7.4% in the post-construction stage (Figure 1) indicates at least formally that the reinforcement is still under a constant tension after 25 years, which is hardly to believe. However, the strains remain "imprinted" also in the long-term. It is difficult to evaluate the corresponding effective tensile force due to the unknown relaxation in soil embedment. Based on the long-term stress-strain behavior (isochrones, but note: tested "in air") of the reinforcement used a stress ratio of ca. 55% should be expected.

Main lessons learned:

Under an embankment on soft soil the basal reinforcement can behave in a very different way during loading and unloading. It is generally a known behavior of polymers; however, in the case here the order of magnitude is surprising: the difference in response is more than 10 times, see above. A possible explanation beside the viscous-plastic behavior is the "retaining" influence of bond to the contact soils arresting the shortening/unloading of reinforcement.

Once generated, the strains can remain "imprinted" also in the long-term despite the strongly reduced need of tensile force for global equilibrium.

3 BERNE BRIDGE ABUTMENT: STRESS RELIEF GECS

3.1 *Description of the project*

The new German Federal Road (Bundesstrasse) B212n is conceived as a bypass of the City of Berne in Northern Germany. The allowed driving velocity is 100 km/h, thus the requirements concerning serviceability are quite stringent. The route comprises nine bridges and viaducts with high embankment bridge approaches. Construction started in 2009. Typical for the region are soft saturated soils (holocene clay and peat, alluvium) with a thickness of 10 to 15 m followed by pleistocene sandy layers. Usually the ground water level (GWL) equals the terrain. The entire B212n is positioned on embankments of varying heights, the highest ones at the bridge approaches.

Typical solutions in such cases are embankments with basal reinforcement without or with strip drains and temporary overloading (pre-consolidation, see Chapter 2 and Blume et al. (2006), Alexiew & Blume (2010), Alexiew & Blume (2012)). However, at Berne this scheme could not be applied for the high bridge approach embankments due to the limited construction time and the risk of inacceptable post-construction creep settlements. Finally the foundation on so called geotextile encased columns (GEC) was found to be the optimal solution. The GECs are pile-similar elements consisting of compacted sand encased by high-strength low-strain geotextile tubular encasements as an engineered element controlling their behavior. For more details see e.g. Alexiew et al. (2012) and Alexiew & Thomson (2014) including also further references.

For the northern approach embankment at the river Hunte two problems had to be solved: crossing an old landfill and reducing the lateral pressure in depth on the piles of the bridge abutment (Figure 2). The latter challenge is the focal point herein. For the "crossing the landfill issue" see Alexiew et al. (2016) including also a very detailed geotechnical information. Typical values of the undrained unconsolidated shear resistance s_U of peat and clay are in the range of 10 to 15 kPa, the drained angle of internal friction ϕ' in the range of 20° to 25°. The old landfill was extremely inhomogeneous (Alexiew et al. 2016).

The height of embankment at the abutment is about 7 m.

Design for the GEC foundation system was performed according to EBGEO (2011) being until now the only Code worldwide handling the system. GECs with a diameter of 0.6 m were chosen in a triangular pattern with an area (replacement) ratio of 12.4% (Figure 3).

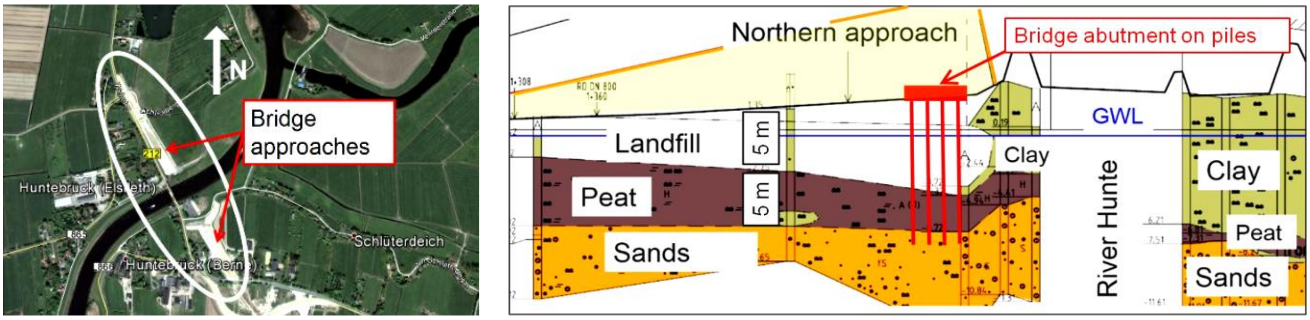

Figure 2. Berne bypass: Overview of crossing the river Hunte (left)
and simplified longitudinal cross section (right); GWL=average ground water level

The common range is 10 to 20% (Alexiew & Thomson 2014). Very high-strength high-modular geotextile encasements were adopted with a tensile modulus in "ring" direction of more than 7000 kN/m because of the very stringent deformation limitations.

The German pile design recommendations EA Pfähle (2007) comprise a simplified procedure to protect a pile group against lateral pressure from an adjacent embankment asking for a factor of safety (FOS) > 1.4 for the global stability of the embankment in direction to the piles. This requirement resulted in the GEC group shown in gray in Figure 3, right. For the purpose of unification columns and encasements were kept the same as for the rest of approach embankment, but the number of GEC rows and the area ratio were increased until reaching a FOS > 1.4. Additionally - based on regional experience - it was decided to limit the total lateral (horizontal) pressure $\sigma_{h\ tot}$ to max 50 kN/m^2 and/or to reduce the lateral pressure to less than 50% in comparison to a non-supported solution without GECs. A simplified FEM analysis was performed as well (due to brevity not handled here). The final area ratio for the "relief GEC group" amounts to 17.5% instead of 12.4% for the rest of embankment.

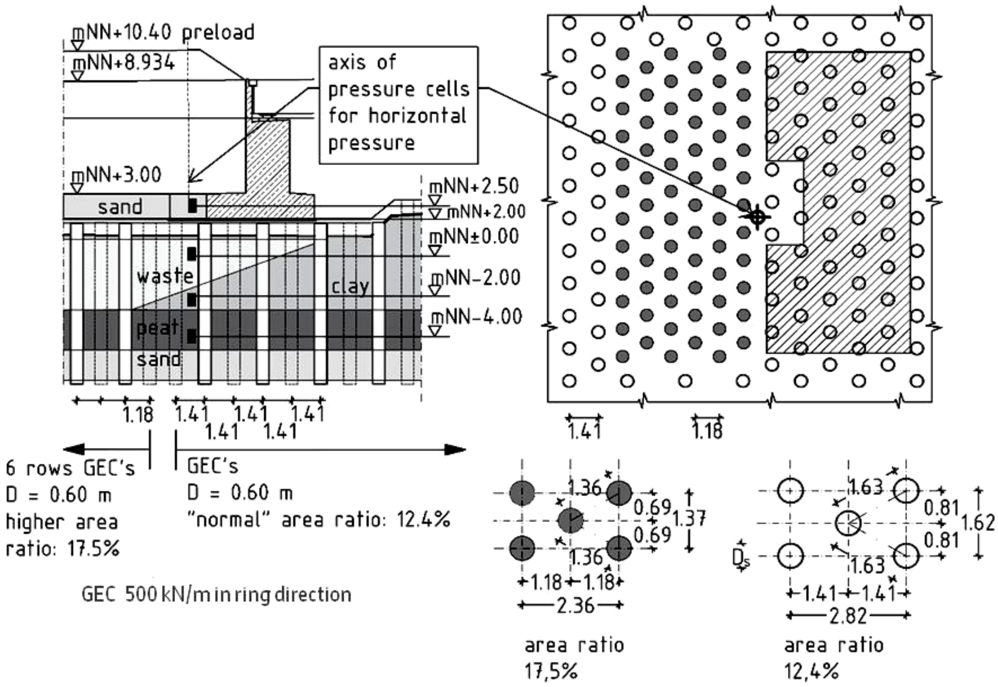

Figure 3. GEC foundation near the abutment: longitudinal cross-section (left) and plan view (right);
mNN=average sea level

3.2 *Monitoring program*

A monitoring program was implemented due to lack of research and experience especially regarding the lateral pressure in depth adjacent to GEC, due to the importance of validation of design calculations and due to the strict limitations in terms of acceptable lateral pressure and settlements. Earth total pressure cells (EPC) were installed between the GEC stress-relief group and the abutment piles (Figure 3). Their levels are in the sand fill of the working platform, the waste in the landfill, the clay and the peat. Settlements were also measured on top of embankment to check the general behavior of the GEC system and to adjust the gradient if needed.

3.3 *Measurement results and comments*

3.3.1 *Settlements*

The measurements continued until October 2016. Settlements on top of the sand layer behind the abutment at about mNN+3.00 (Figure 2, left) are shown in Figure 4 for a point at the embankment axis positioned about 15 m to the left (say behind) of the abutment. The embankment height above the sand platform at that point is about 5.5 m (i.e. the maximum surcharge on the plane of measurements is about 100 kN/m²). As expected, the settlements correspond to the surcharge. They do not exceed 20 cm: a low value under this geotechnical circumstances (Figure 2, right, and Alexiew et al 2016). What is even more important: there is no settlement increase under constant load (after January 2014). The settlement measurements seem to confirm the suitability of the solution chosen.

Main lessons learned: A properly designed GEC foundation can also eliminate settlements due to creep of soft soil.

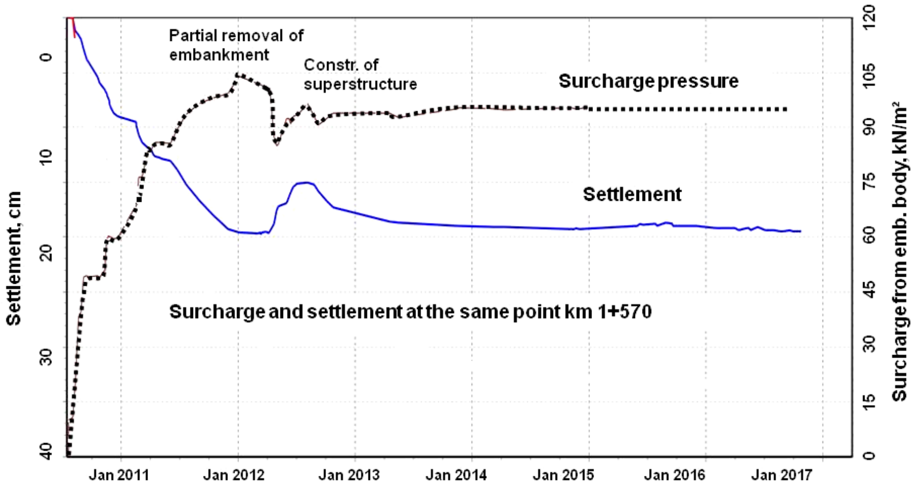

Figure 4. Typical settlements at level mNN+3.00 (see Figure 3, left)

3.3.2 *Lateral pressure in depth behind abutment*

Due to brevity only the graphs of development of total horizontal $\sigma_{h\ tot}$ are displayed in a simplified way in Figure 5. Results until August 2015 have been published and commented in Alexiew et al (2016). Figure 5 includes additional results for 14 months more until end of measurements in October 2016.

The local deviations of the graphs reflect deviations of the ground water level (GWL) connected to some extent to the water level in the river Hunte; they change the pore water pressure u as a part of $\sigma_{h\ tot}$. Because the sand fill at mNN+2.50 is quite above the average GWL at mNN 0.00 (varying usually by about 1.0 m) the total $\sigma_{h\ tot}$ is less influenced by the GWL changes, it undergoes less deviations and the σ_h -graph is smoother in contrast to the other layers. However, there is a surprise in the latest values in comparison to the measurements until August 2016: there is an increase of 8 kN/m² corresponding to 0.8 m waterhead; the authors don't find an explanation for that; may be an EPC disfunction?

Another generally new fact of significance are the peak values in the clay, the waste and the peat about January-March 2016. The most probable explanation is the assembly and presence in this period of the prefabricated bridge structure on top of the approach embankment before final installation (the bridge was constructed using the so called time shifting method). The $\sigma_{h\ tot}$ in the clay even exceeds for a short period the "allowed limit" of 50 kN/m². The final bridge construction procedure had not been communicated in time to the geotechnical designers, thus this short-period additional surcharge had not been taken into account by them. However, this can be interpreted as a not-planned demonstration of the robustness of the solution on GECs and the right choice of safety factors.

After this moment (i.e. after the bridge structure removal) all $\sigma_{h\ tot}$ -values permanently decrease in an almost parallel way, reaching in the (permeable) peat almost zero.

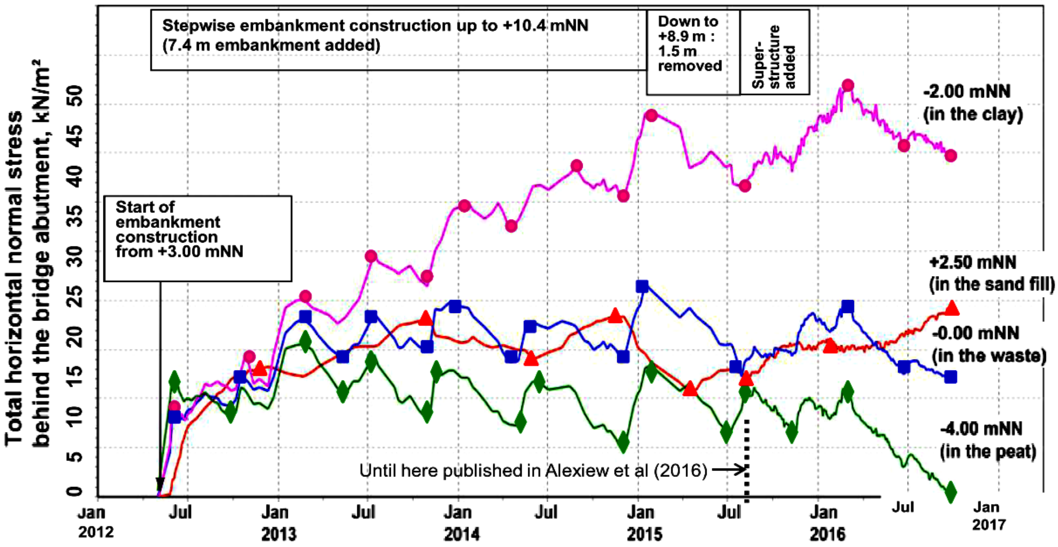

Figure 5. Total horizontal stresses $\sigma_{h\ tot}$ at four levels in four different layers behind the bridge abutment over time (see also Figure 3).

From the practical point of view the most important fact is that the highest $\sigma_{h\ tot}$ (as it has to be expected: in the clay) is less than the half of total σ_h to be expected without the stress relief GEC group (predicted by EBGEO 2011 and the simplified FEM analyses) demonstrating the correctness of design approach and the suitability of GECs as lateral pressure relief measure. Even in the period of the bridge assembly it exceeds only by 2 kN/m² the max allowed value of 50 kN/m². It should be also noted that in all problematic layers the final trend of $\sigma_{h\ tot}$ is downwards.

Generally the “total σ_h - behavior” of the clay and of the peat meets the expectations taking into account their position and permeability.

From special interest are the measurement results for the old waste being something of a rarity and positioning its behavior between the behavior of the clay and of the peat.

It seems that the GEC system has proved to be an efficient solution also as a stress relief system. Similar experience was gained in an almost time-parallel project in Brazil (Schnaid et al 2014, 2017).

Main lessons learned:

GECs can be successfully applied for lateral stress relief in different soils. The degree and development of stress-reduction effect depends (as it should be expected) on the type of soil.

ACKNOWLEDGEMENTS

Many parties participated in the two projects presented: consultants, researchers, investors, officials and supervisors. The authors being only a part of this engineering constellation would like to express their appreciation to all participants for their activities and efforts.

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