On the use of high strength geosynthetic basal reinforcement layers for embankment construction on very soft soils

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ABSTRACT
Embankments on soft subsoils are one of the typical problems of modern geotechnical engineering. High strength geosynthetic reinforcement are used in this application as a technical, ecological and financial beneficial solutions for more the 35 years by now. Over the time a lot of experience and knowledge has been gathered in regard to the system short and long term behavior. Site measurements have disclosed some unexpected results, which have been meanwhile confirmed by further site measurements and numerical simulation. One phenomenon is the unequal stress or load distribution, respectively, if multilayer systems with small vertical distance between the single layers are constructed.

The paper present two case studies, where among others strain measurements have been done on the geosynthetic. The results will be shown and interpreted as well as used for a calibration of a numerical model. The numerical is used to gain a better understanding of the system behavior.

RESUMEN
Los terraplenes en suelos blandos son uno de los problemas típicos de la ingeniería geotécnica moderna. El refuerzo geosintético de alta resistencia se utiliza en esta aplicación como una solución técnica, ecológica y financiera beneficiosa desde hace más de 35 años. A lo largo del tiempo se ha acumulado mucha experiencia y conocimiento sobre el comportamiento del sistema a corto y largo plazo. Las mediciones del sitio han revelado algunos resultados inesperados, que entretanto han sido confirmados por otras mediciones del sitio y simulaciones numéricas. Un fenómeno es la distribución desigual de la tensión o de la carga, respectivamente, si se construyen sistemas multicapa con una pequeña distancia vertical entre las capas individuales.

El documento presenta dos estudios de caso, en los que se han realizado, entre otras cosas, mediciones de la deformación del geosintético. Los resultados se mostrarán e interpretarán y se utilizarán para la calibración de un modelo numérico. La numérica se utiliza para obtener una mejor comprensión del comportamiento del sistema.

1. INTRODUCTION

In the last four decades, the use of geosynthetic reinforcements for the construction of the embankments on soft subsoils has become popular around the world. Before the development of advanced design and calculation methods for such an application, it was essential to construct test embankments in order to assess the possible response of the system to different aspects such as construction rate or tensile behavior of the basal reinforcement in short and long terms and for single or multiple layer arrangements. For instance, the Federal Highway Research Institute in Germany initiated projects for investigation and measurement of large-scaled embankments on soft soil reinforced with geosynthetics. The main objective of conducting such trial embankments were to evaluate the short- and long-term behavior of the system as well as failure/deformation mechanisms and to assess the interactions between the soil and high strength geosynthetic reinforcements. Through these investigations, the existing design concepts (mainly in terms of stability analysis) and the corresponding assumptions were verified.

The first project in this paper, has been a trial embankment where two cross sections were constructed with reference to the federal interstate highway B 211 in the proximity of city Grossenmeer. In this trial, the first cross section with steeper slopes and rapid construction rate, called “test embankment”, was built to assess the possibility of the quick construction of the main core of the road and later integration of that into the standard highway embankment by reshaping the slopes. The second cross section was gently constructed from the beginning with the standard geometry of the highway embankment, called “reference embankment”. Both cross section remained in the final embankment. In both embankments, the construction was carried out on the immediate soft soil without any ground replacement and the evolution of the system responses such as strain in the basal reinforcement and the settlements with time were monitored. In order to better understand the hydro-mechanical behavior of the system and to analyze the influence of the material characteristics on the model response, a series of numerical analyses have been carried out to simulate the behavior of reference embankment.

The second project described in this paper is about an embankment on soft soils where two layers for the basal reinforcement have been used. The embankment was constructed in 2001 as part of the Federal Highway A 26 in the north of Germany. The subgrade in this area often consists of clay and peat with layer thicknesses of up to 13 m. Due to the low shear strength additional measures had to been taken to secure stability of the embankment. Measurements have been taken in a section, where the embankment height was 16 m to investigate the strain of the layers during the construction time and after end of the construction.

This paper will present the results obtained from measurements and a numerical model that is validated based on the field measurements of the first project.

2. INTERPRETATION OF STRAIN MEASUREMENTS

A very common and widely used method to estimate the tensile force of geosynthetics is performed by the measurement of strains. Based on the "stress – strain" behavior of the geosynthetic for short-term as well as long-term loading the activated strain in the geosynthetic then gets converted to an activated tensile force. Assuming that the tensile force remains constant during the period of observation and / or low strains this procedure has enabled satisfying conclusions with regard to the behavior of the geosynthetic reinforced structure in many cases. However, it is important to understand that this procedure may lead to mistakes especially if the loading of the geosynthetic is not constant over time. In such cases strain measurements only will not be sufficient to get appropriate information about the tensile force. This is amongst others due to the following reasons:

- The tensile force versus strain curves are estimated in air. This relation does change, once the geosynthetic is in contact with soils, e.g. by particle intrusion or interlocking
- The tensile force versus strain curves are time depending due to creep and relaxation processes
    - Creep is defined as the elongation over time under a constant load
    - Relaxation is defined as the tensile force reduction under constant strain
    - Creep and relaxation depend on the raw material, the load magnitude, the load duration and the temperature
- The loading ratio (tensile force in relation to the maximum tensile strength) might vary over the time

Such applications where it is hardly possible to predict the exact tensile force of the geosynthetic are applications such as basal reinforcement of embankments on soft soil. This is mainly due to the consolidation process (increasing settlement and deformation) and the increasing sub grade strength due to the consolidation (increasing stiffness and support). Degrading natural contents in predominantly organic soils, changing groundwater levels and creep of the soil itself may be other reasons. Furthermore creep and relaxation processes of the reinforcement occur at the same time and influence each other, e.g. creep will reduce the tensile force in the geosynthetic and therefore it will also reduce the creep rate of the geosynthetic. Therefore certainty about the activated tensile force over time will only be obtained by a direct tensile force measurement.

An estimation of the tensile force can be obtained by the use of the so called isochronous curves. The isochronous curves represent the stress – strain behavior over time for a constant load level or a constant stress level.

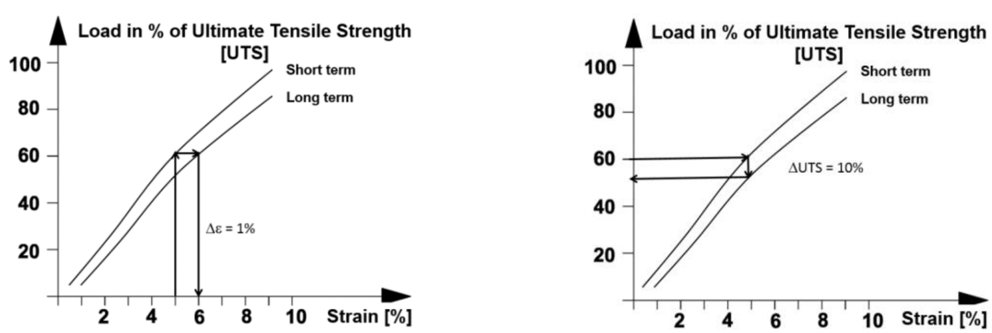


Figure 1: Estimation of the creep for give load (left side) and estimation on load reduction for a given constant strain (right side) by the use of the isochronous curves.

The left diagram in Figure 1 shows how to estimate the elongation due to creep under a constant load over time. In this example a strain of 5% corresponds to a load ratio of 60%, so for a geosynthetic with e.g. an ultimate tensile strength of 100 kN/m a tensile force of 60 kN/m would have been activated. To estimate the extra strain due to creep under a constant load ratio of 60% a horizontal line is constructed from the point of the short term stress-strain curve at 5% to intersect with the long term stress-strain curve and from there down to x-axis, where the long term strain for this load ratio can be taken. In this case it is

6%. The difference between the short term and the long term strain is the expected creep for the long term application, in this case 1%.

In case the strain measurements do not change over time, relaxation might occur, which does result in a reduced load ratio. Assuming the strain of 5% stays constant over time, the relaxation can be estimate by constructing a vertical line from the point of the short term stress-strain curve at 5% to intersect with the long term stress-strain curve and from there back to y-axis, where the long term load ratio this strain level can be taken. In this case it would 50%. The difference between the short term and the long term load ratio is the expected creep for the long term application, in this case 10%, which means the 60 kN/m tensile force would reduce to 50 kN/m.

In both examples the boundary conditions have been chosen to either be a constant load ratio or a constant strain level, in reality such “ideal” conditions may not occur, which is why from strain measurements only a careful estimation of the acting tensile force can be done. As mentioned above more certainty would be obtained by a direct tensile force measurement.

For the most raw materials used for reinforcing geosynthetics, such as Aramid, PVA or PET, most of the strain occurs with the first couple of days and only small changes are occurring afterwards.

Nevertheless, strain measurements are very important and helpful measurements for understanding and observing the system behavior which is demonstrated in the following two case studies.

3. CASE STUDY: GROSSENMEER

The construction of the highway at Grossenmeer with the length of 2 km that also included the trial embankment with a height of 4.5 m, the earthwork was took about 1 year starting on June 1986 that was followed by a consolidation period of 15 months. In this field, the weak subsoil consists of a 3 to 5 m thick peat and organic silt with that is under laid by a dense sand. According to the preliminary soil investigations, the peat had a unit weight of γ= 11 to 13 kN/m^3 and undrained shear strength of cu=8 kN/m2 while the sand has a unit weight and friction angle of γ=18 kN/m^3 and ϕ =32.5°, respectively. The slopes of the embankment were constructed with the slope of 1V:3H. In order to enhance the safety factor of the embankment against global failure, a basal reinforcement with a short-term nominal tensile strength of 400 kN/m (Stabilenka® 400) was adopted. To monitor the system behavior, settlements, pore water pressures, inclination and ground water level in the subsoil were measured.

3.1 Construction process

In the reference embankment section, the highway is constructed stepwise in 5 layers with a thickness of 1.5 m, 0.9 m, 0.7 m, 0.7 m and 0.5 m, respectively. In this frame, the settlements and strains have been measured after 4 days from the installation of the 1st layer and after 30 days for the 2nd to 5th layers. Figure 2 shows the geometry of the embankment and the arrangement of the layers.

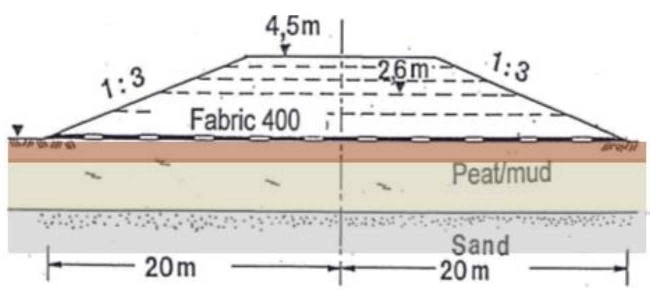


Figure 2. Geometry of the embankment and the thickness of layers (Blume, 2015).

3.2 Measurements

The behavior to the basal reinforcement over time is observed since 33 years by strain measurement devices. Those devices are installed only in one have of the embankment, starting at the center towards the slope in a distance of 2 m. In total 9 strain measurements devices have been placed. The last measurements on this sections has be done in august 2010.

Figure 3 displays the strain measurements from the year of 1986 until 2019. For a better overview, only 4 selected measurement series are shown. The first shown measurement curve was taken in 1986 after the construction of the embankment including the pre-loading has be finished. Due to consolidation process and the associated settlements the strain within the reinforcement increases (measurement curve from 1988). In 1990 the consolidation beneath the embankment has reached about 90% and the excess height has been removed. No significant settlements is expected to occur after that. It can be observed, that the strain in the slope areas does decrease after removal of the extra embankment load. Most probably this results from the reduced spreading forces in the slope and also the reduced overburden pressure, which might allow the geotextile to move a bit (reduced pressure for anchoring the geotextile in the slope area) and thereby reducing the strain. Comparing the strain values in the center part of the embankment a further increase between 1988 und 1992 can be seen. The strain curves from 1992 and 2019, a time period of 27 years, are more or less identical. The deviations are more or less within the measurement tolerances.

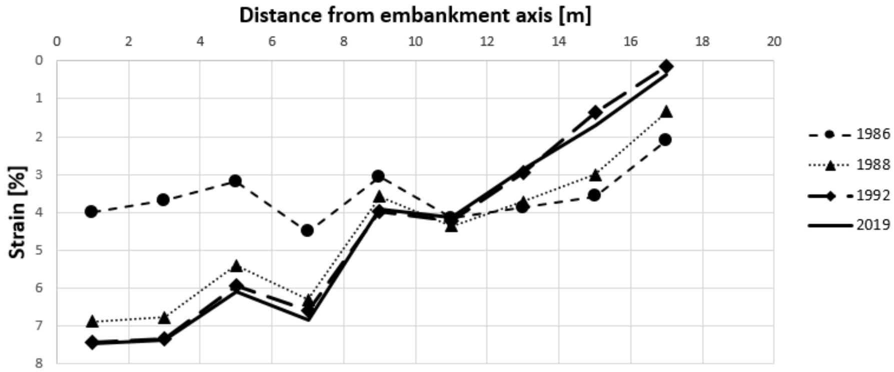


Figure 3. Strain measurements in respect to the location of the measurement device over time from the year of 1986 to 2019.

It can also be observed that the strain values are quite significant and that even after 33 years the woven geosynthetics reinforcement is very much tensioned, which means it is still providing significant tensile forces to the system.

As explained before most of the creep strain happens at the first couple of days. For the used product the extra strain due to creep tested in air between for a strain of 7,5%, which corresponds to a load ratio of about 60% is about 0,4% between 1 year and 40 years. The reduction in tensile load due to relaxation at this stress level would be around 3%, so instead of providing a tensile force of 240 kN/m to the system, a tensile force of 228 kN/m would be provided to the system. Due to the missing tensile force measurement only those extreme interpretations can be concluded.

Figure 4 displays the strain over time for three different location from the embankment axis. It can be seen, that approximately after around 1100 days (~3 years) the strain remains more or less constant. This corresponds quite well with the consolidation process, which has reached after around 3,5 years a degree of 90%. After that it appears that no further additional loads have been transferred to the geogrid.

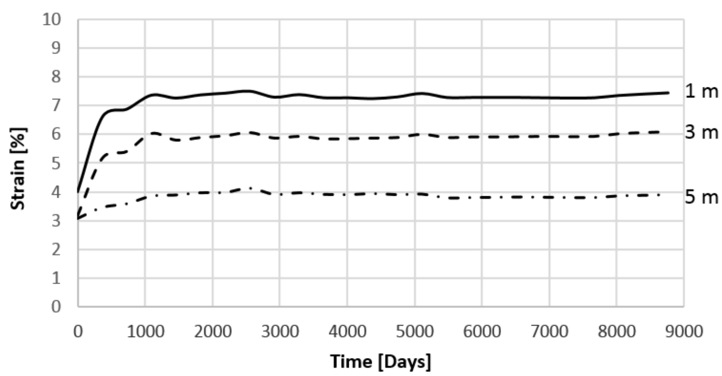


Figure 4. Strain over time for different distance from the embankment axis in [m].

3.3 Numerical simulation of the trial embankment

In order to assess the behavior of the embankment at different conditions, a series of numerical simulations have been carried out using commercially available finite element (FE) package PLAXIS^{2D}. In these numerical simulation, the hydro-mechanical (HM) coupled interactions during the construction as well as the consolidation have been taken into account. In the numerical models, 15-noded triangular elements have been used to simulate the system assuming a plane-strain condition. To assure no influence of the boundaries and mesh size on the model responses, a trial analysis has been conducted and the size of the model and the discretization pattern are relevantly chosen in a manner that the model responses are independent from these numerical aspects. Accordingly, the depth of the model was assumed to be 40 m while it is extended 40 m from tow of embankment toe (i.e. 60 m from the embankment axis). Moreover, a finer mesh is considered at the base of the embankment to ensure no locking and singularity effect due to short-term large deformations. To avoid excessive restriction of the deformations at the boundaries, only the out of plane displacements at the sides and bottom of the model were restricted.

To initiate the geostatic stress in the subsoil prior to the construction, an initial phase calculation in conjunction with K_0 analysis is carried out. As the rate and process of the construction of the embankment plays a significant role in the coupled HM behavior of the system, the stepwise construction stages have been addressed in the numerical modeling in accordance with the realistic time of the construction and consolidation, as mentioned in section 3.1. In the present FE model, it has been assumed that placement of each layer takes 1 day that is followed by a 30-day consolidation period. Figure 5 shows the geometry of the system, discretization scheme as well as the stages of construction in numerical simulations.

3.3.1 Properties of the materials

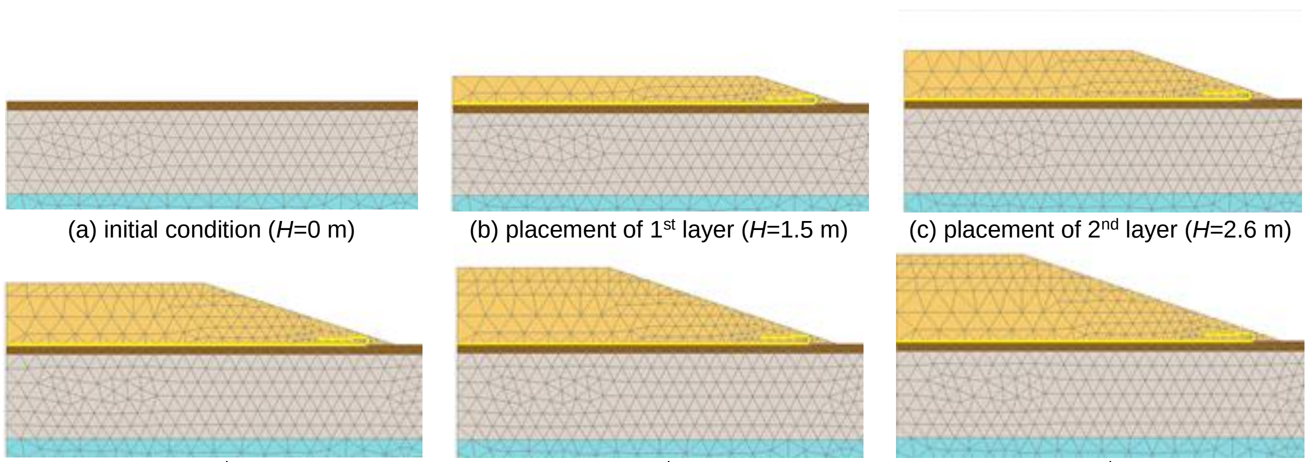
To simulate the behavior of the subsoil in the numerical investigation, the Hardening Soil model (HS) developed by Schanz (1998) has been adopted. Hardening soil model is one of the mostly referred constitutive model in geotechnics that can be used for wide range of soil. This model has been developed in the frame of classical elasto-plasticity in accordance with a multishape yield surface that consists of a cone and a cap. The hardening in this model is governed by a double hardening law, namely shear and volumetric hardening that is applied to the cone and cap surfaces, respectively. In this model, three stress dependent stiffness parameters, namely E_{50}, E_{oed} and E_{ur} have been used for primary triaxial, oedometric and unloading/reloading conditions, respectively. For more information about the details of the HS constitutive model, one is referred to Schanz (1998) and Schanz et al. (1999).

As the laboratory element test results on the subsoil are not available, the constitutive parameters have been initially approximated from the literature and experience on similar soft soils. However, in the second stage, the proper soft soil parameters (i.e. top and bottom peat layers) have been identified in accordance with the monitoring data and the field observations on the deformation mechanisms. The parameters used in the numerical investigations are presented in Table 1.

To address the evolution of permeability in soil during the construction of the embankment and corresponding consolidation, the relationship between the actual and initial permeability (k and k_0) and void ratio proposed by Taylor (1943) is adopted in the present study. According to this concept, the evolution of the void ratio (Δe) due to compression and consolidation consolidation leads to evolution of the permeability as:

$$\log (k/k_0) = \Delta e/c_k \qquad [1]$$

where c_k states the permeability parameter for the soft subsoil (e.g. peat). In this study, c_k is assumed to be equal to 1.0 while the initial void ratio (e_0) for the top and bottom peat layers is assumed to be 1.6 and 2.0, respectively.



(a) initial condition (H=0 m) (b) placement of 1st layer (H=1.5 m) (c) placement of 2nd layer (H=2.6 m)

(c) placement of 3rd layer (H=3.3 m) (d) placement of 4th layer (H=4.0 m) (e) placement of 5th layer (H=4.5 m)

Figure 5. Details of the Geometry, discretization and construction stages in the FE models.

Table 1. Parameters of the different soil layers.

	E_{oed}^{ref}	E_{50}^{ref} [MPa]	E_{ur}^{ref}	υ_{ur} [-]	m [-]	c [kPa]	ϕ	ψ [°]	e_0 [-]	$k_x = k_y$ [m/s]	c_k [-]	γ_{unsat}	γ_{sat} [kN/m^3]
top peat	1	1	1	0.2	0.6	10	15	0	1.6	1×10^{-7}	1	12	13
bottom peat	0.3	0.3	1.2	0.2	0.8	0	12	0	2.0	5×10^{-8}	1	12	13
Sand	15	15	45	0.2	0.5	0	30	0	0.5	1×10^{-6}	10^{12}	18	19
Fill material	60	60	60	0.2	0.5	0	32.5	2.5	0.5	1×10^{-5}	10^{12}	19	19

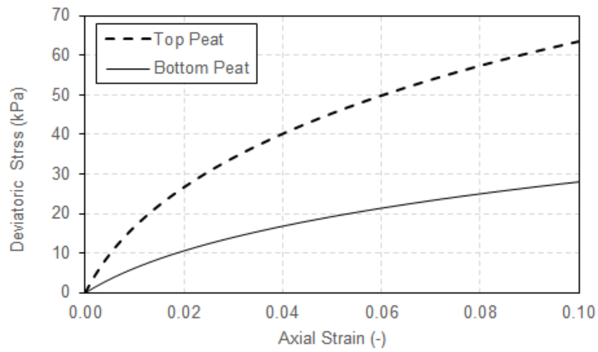
p_{ref}= 100 kPa

3.3.2 Calibration of the parameters and validation of model

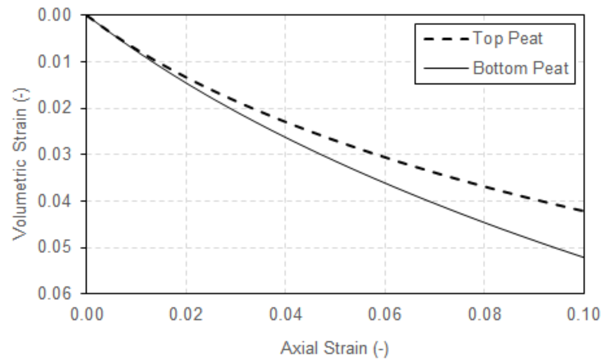
In the first stage, to assure the relevance of the parameters identified for soft peat layers, mechanical behavior of the top and bottom peat under triaxial and oedometer loading is numerically investigated using in-built PLAXIS soil lab test tool. The results obtained from such virtual laboratory tests have been presented in Figure 6.

After the identification of parameters, the construction and consolidation process of the embankment have been simulated and the time dependent numerical results have been compared with the field measurements. This comparison between the numerical results and monitoring data has been illustrated in Figure 7. As seen, the top and bottom peat have a shear strength of about 67 and 28 kPa at 10% strain when it is subjected to 100 kPa confining stress. According to the field observation and the available literature on the soft soil in the region of Grossenmeer, such behavior is realistic.

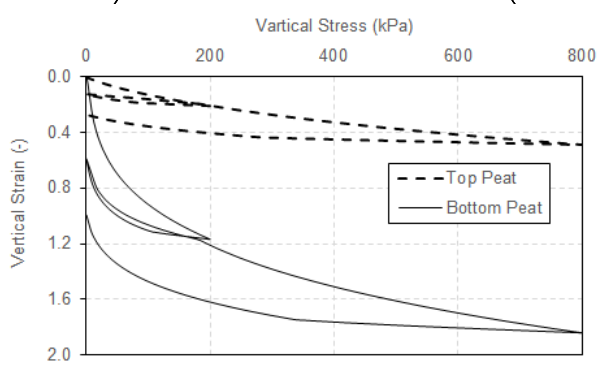
By having the soil parameters calibrated, next stage was to verify the reliability of the numerical model. For such a validation of the FE model, the construction period and approximately 1 year after that is simulated and the results in terms of stain and settlements are compared. Figure 7 shows the comparison between the numerical results and field measurements of the settlement of embankment base. According to Figure 7, the numerical results are in an excellent agreement with the measurements.



(a) deviatoric stress vs Axial strain in triaxial loading (confining stress 100 kPa)



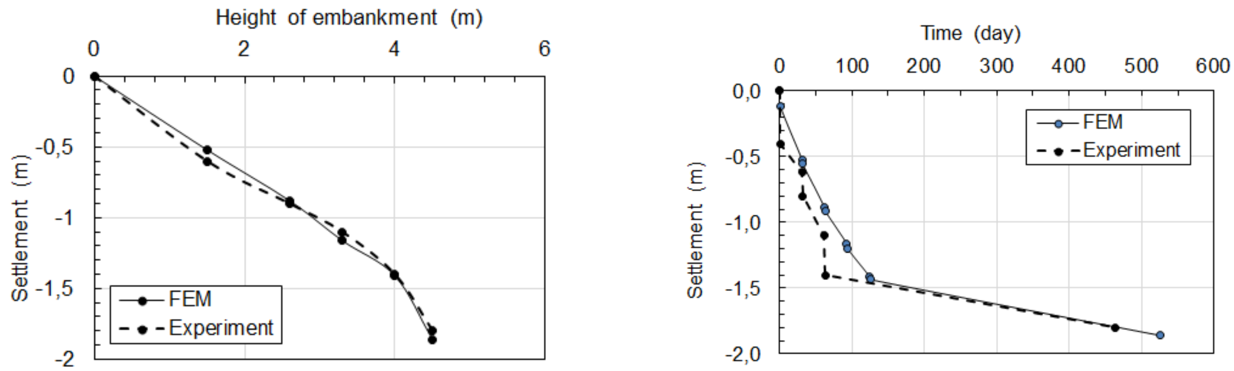
(b) volumetric strain vs Axial strain in triaxial loading (confining stress 100 kPa)



(c) vertical stress vs vertical strain in oedometer loading

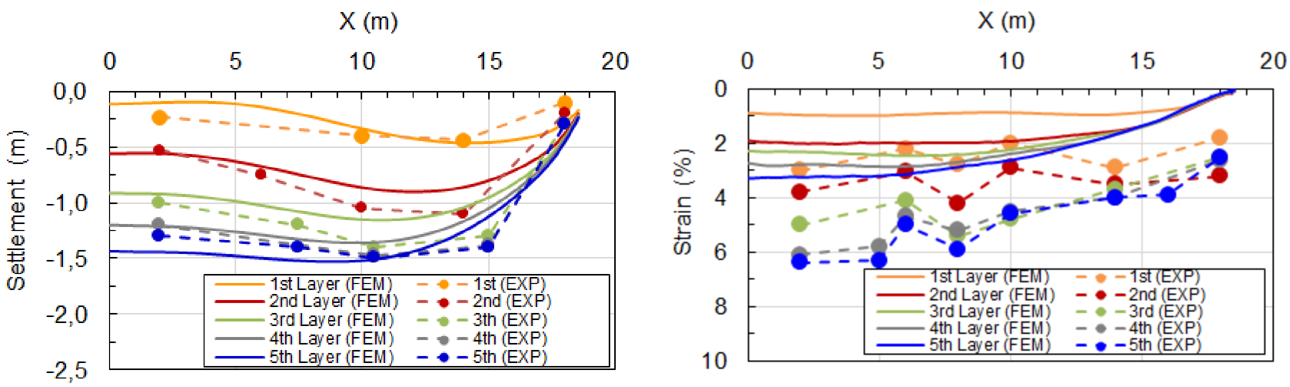
Figure 6. The results obtained from virtual soil triaxial and oedometer lab test.

To verify the model in terms of the deformations and strain in the basal reinforcements, the numerical results have been compared with the measurements as shown in Figure 8. As seen, despite an excellent agreement between the models of the settlement over the length of reinforcement, the strain distribution obtained from numerical simulations slightly deviate from the measurements. To realize a possible reason for this disagreement, it has to be noted that strain (ε) has been back calculated from force (*F*) distribution in the reinforcement and its tensile modulus (*EA*) in accordance with $\varepsilon = F/EA$. Due to the non-axial deformations in the reinforcement, such translation of strain as a vector in reality to a scalar from the back calculation can lead to some inaccuracy. To better assess this issue, a geometrical analysis on a single case carried out and the real strain distribution in the reinforcement is directly calculated from the updated coordinate of its node. This comparison, which has not been presented here, illustrated a better match between the numerical results and measurements.



(a) settlement vs height of embankment (b) evolution of settlement with time

Figure 7. Comparison of measured settlement at the base of the embankment with numerical results.



(a) vertical displacements in reinforcement (b) distribution of strain in reinforcement

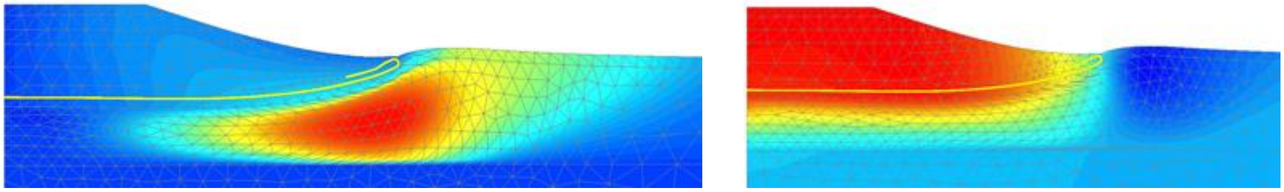
Figure 8. Comparison of settlement and strain in the basal reinforcement in the construction period.

3.4 Numerical results and discussuíons

In order to better identify the embankment system in terms of the deformation mechanism and failure kinematics, influence of the material properties and construction speed is examined through a parametric study based on the validated FE model. In this section, first the general behavior of the system and then the results obtained from this parametric study will be presented and discussed.

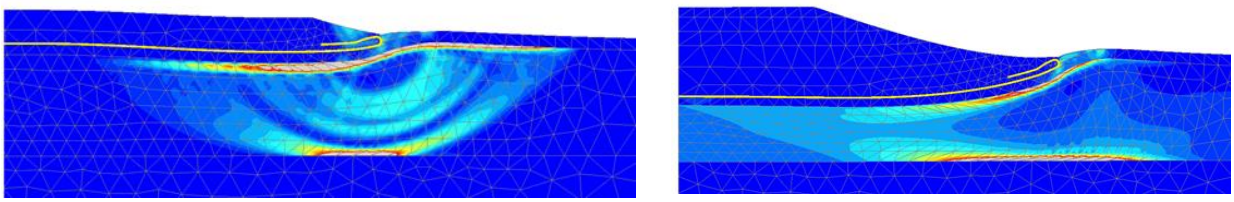
3.4.1 General assessment of system behavior

To identify the behavior of a geotechnical system, it is essential to evaluate the mechanism of deformation and/or failure in the system. To do that, first the deformations and potential modes of failure in the subsoil in short and long terms have been analyzed. Figure 9 presents the deformations in the subsoil and the embankment body at the end of construction. As seen in Figure 9, the soft subsoil would be squeezed out under the slope of embankment while the embankment settles almost uniformly at the end of construction. With reference to the variation of the position of point with largest settlement as shown in Fig. 8(a), it can be expected that the possible mode of failure varies with time. To assess this phenomenon, the contour of deviatoric strain distribution in short term (during the construction) and in long term (during the final consolidation period) have been illustrated in Figure 10. According to Fig. 10, the shear failure in the slope is the most probable mode of the failure while the lateral spreading (lateral squeezing) is the governing failure mechanism in long term after the consolidation of the subsoil.



(a) horizontal displacement (b) vertical displacement

Figure 9. Contour of soil displacement in the long-term



(a) short-term (b) long-term

Figure 10. Mode of the failure in short- and long-terms.

Combining the displacements in the basal reinforcement and the results shown in Figure 10, a dual deformation mechanism as depicted in Figure 11, can be expected. As seen, the shear failure and the corresponding settlements govern the design in the short term while the compression zone at the core of the embankment would be activated due to the compression/consolidation of the subsoil under the weight of embankment.

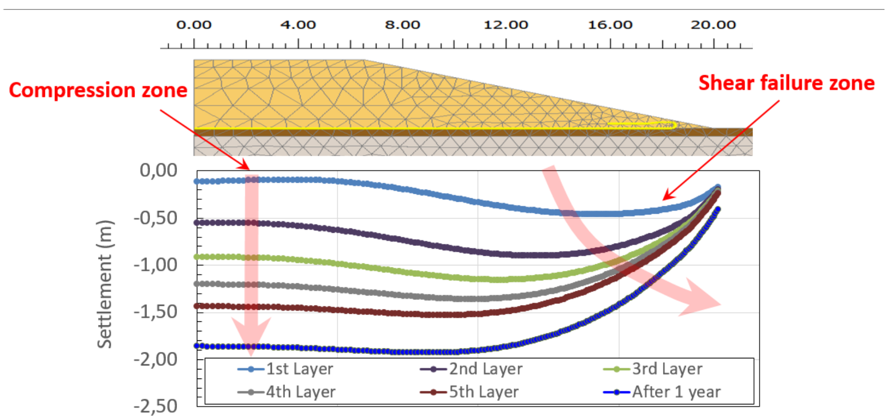


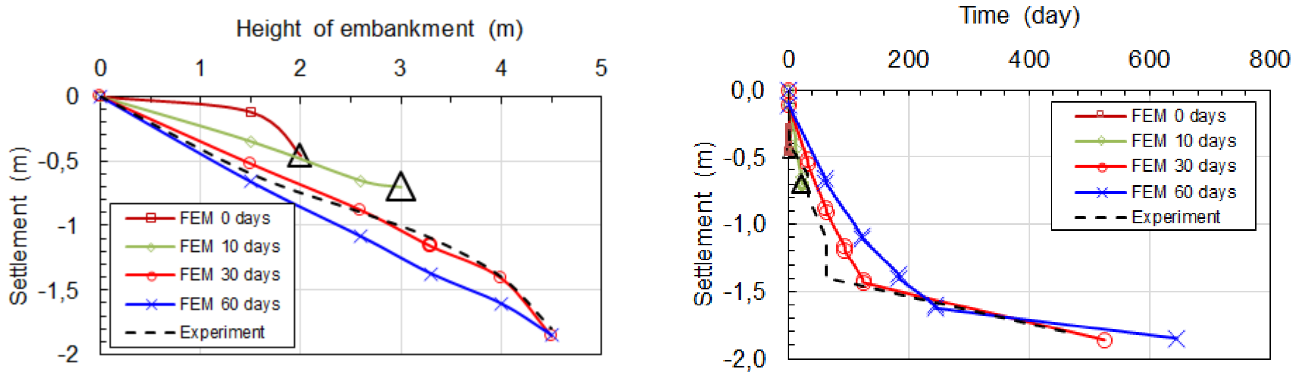
Figure 11. Evolution of mode of deformation and failure with time.

3.4.2 Construction rate

To study the impact of the construction speed on the stability and settlement of the embankment, 4 different scenarios have been considered as:

I. Construction without any break (0 days)
II. Construction with 10 days break between each phase
III. Construction with 30 days break - real construction of "reference embankment"
IV. Construction with 60 days break

The results obtained from numerical simulation of the above-mentioned scenarios have been presented in Figure 11 where the collapse of the construction has been shown by large triangular symbols. As seen in Figure 11, scenario I (i.e. no break in construction) leads to the collapse of the embankment at the height of 2 m while allowing 10 days of consolidation time after placement of each layer permits the construction of the embankment up to the height of 3 m. However, further construction is not possible at this rate. The construction of the embankment with the full height of 4.5 m can be achieved once when the consolidation period of 30 days or more is taken into account. According to Fig. 11 (a), as slower the construction rate as larger the settlements during the construction. Therefore, there would be a better possibility to compensate the settlements at the end of construction. The comparison between the final settlements for the scenarios III and IV, it can be seen the final construction at the end of consolidation is almost identical for these two cases.



(a) settlement vs height of embankment

(b) evolution of settlement with time

Figure 11. Time dependent evolution of settlements in the construction period for different construction rates.

Comparing figures 10 and 11, one can say that the quick construction of the embankment leads to the shear failure at the slope while the slow construction result in more uniform settlement due to concurrent mobilization shear and compression zones in the system.

3.4.3 Stiffness of soft subsoil

One of the most critical aspects in the analysis of the embankments on soft soils is investigating the impact the variation of the initial stiffness of the soft soil on the system behavior. To study this phenomenon, 3 different scenarios have been taken into consideration where the stiffness of the bottom peat varies in the range of 300 to 900 kPa as:

I. $E_{oed}^{ref} = E_{50}^{ref} = 300$ kPa
II. $E_{oed}^{ref} = E_{50}^{ref} = 600$ kPa
III. $E_{oed}^{ref} = E_{50}^{ref} = 900$ kPa

In all of these scenarios, it is assumed that the un-/re-loading stiffness is equal to $4E_{oed}^{ref}$ while the other parameters if the soft bottom peat remain unchanged. Results obtained from numerical analysis for scenarios I~III are presented in figure 12.

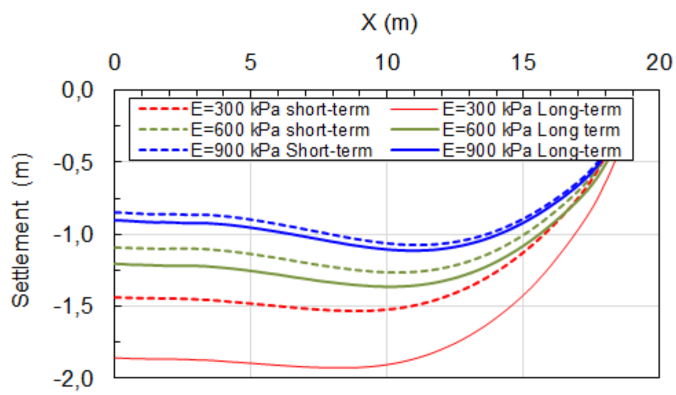


Figure 12. Displacement of the reinforcement in short and long terms for various stiffness of subsoil.

According to figure 12, the difference between the short and long term deformations is more significant for the lower stiffness in the soft subsoil. Additionally, the magnitude of the maximum settlement is, to a large extent, dependent on the stiffness of subsoil. It also can be seen that the settlement at the slope would be larger than the settlement at the center of the embankment in case of higher stiffness of soil. It means, the shear deformation in the proximity of the slope would remain more dominant compared to the compression of the subsoil at the core of the embankment. The numerical results show that the maximum tensile force in the basal reinforcement would be equal to 145, 139 and 127 kN/m for scenarios I, II and II, respectively.

3.4.4 Tensile modulus of reinforcement

In order to study the impact of the tensile modulus of on the behavior of the embankment, 3 scenarios have been defined. In these scenarios, the tensile strength of the basal reinforcement is assumed to be adequate to assure safe construction of the embankment while only the tensile modulus has been varied as:

I. J = 2000 kN/m/m
II. J = 4000 kN/m/m
III. J = 8000 kN/m/m

In these scenarios, the properties of the soil have been assumed to remain constant in accordance with Table 1. Figure 13 shows the deformations of the basal reinforcement in short and long terms for different scenarios. As seen, the tensile modulus of the basal reinforcement does not have a dramatic influence on the magnitude and regime of the deformations. However, it has to be noted that adequate tensile strength of the basal reinforcement is required to assure sufficient safety factor for the system during the construction and operation periods. Comparing the displacement shape of the reinforcement with highest and lowest tensile modulus, it can be seen that the maximum settlements at the vicinity of the slope and the center of the embankment are almost identical for the reinforcement with higher tensile modulus. It means that a slightly more uniform settlement profile can be expected for basal reinforcement with higher tensile modulus while for the lower tensile modulus (i.e. J=2000 kN/m/m) slightly larger settlement at the slope of the embankment is observed.

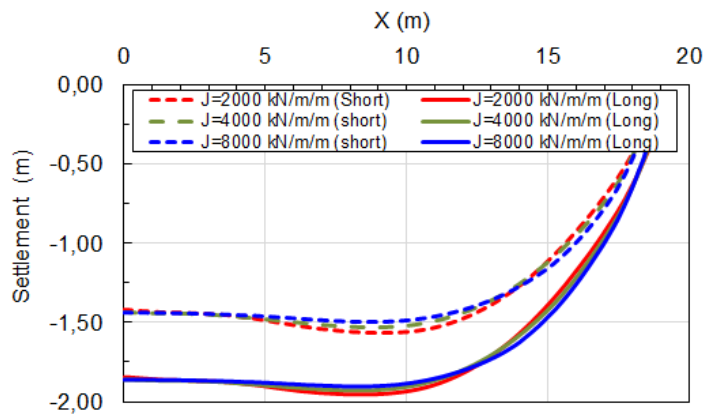


Figure 13. Displacement of the reinforcement in short and long terms for various tensile modulus of reinforcement.

4. EMBANKMENT CONSTRUCTION AT THE FEDEREAL HIGHWAY A26 CLOSE TO HAMBURG, GERMANY

The construction of the federal highway A 26 started in 2001 in the north of Germany close to Hamburg. The upper 13 m of the subgrade consists mainly of peat and clay with a very low strength. Different options have be analyzed for the construction of the embankment. For financial, technical and ecological reasons the so called pre-loading and consolidation method has been chosen, where the short term and long term stability of the embankment is provided by the use of high-strength woven geosynthetics at the embankment base. A detailed project description can be found at Alexiew and Blume (2012).

In one section an embankment as bridge approach with a total height of up to 16 m close to an existing railroad under traffic had to be constructed, so besides the vertical settlements, also the horizontal deformation has been an important aspect. Stability calculation resulted in a required design resulted in a required ultimate tensile strength (UTS) greater than 1100 kN/m. Also it was recommend to use one strong layer in the project, for contractual reasons two layers had to been installed. Therefore two woven reinforcements products with an UTS of 600 kN/m haven been installed with a distance of 50 cm below the16 m high embankment. Besides deformation measurements also the strain in the lower and upper layer haven been measured during and after the construction.

Figure 5 displays the strains in the two geosynthetic reinforcement layers close to the centerline of the embankment as well as the load steps over time. First of all there is clear mobilization of both reinforcement layers with increasing embankment height. Furthermore it can be seen, that even there a two identical products placed (same raw material, same UTS and same tensile stiffness) the load distribution between the two layers divers significantly. The lower layer is taking nearly twice as much load as the upper layer, also they a relatively close to each other (50 cm below a 16 m high embankment). Referring to the bended beam theory, an increasing strain towards the lower layer was expected, but not in the magnitude as shown in figure 5.

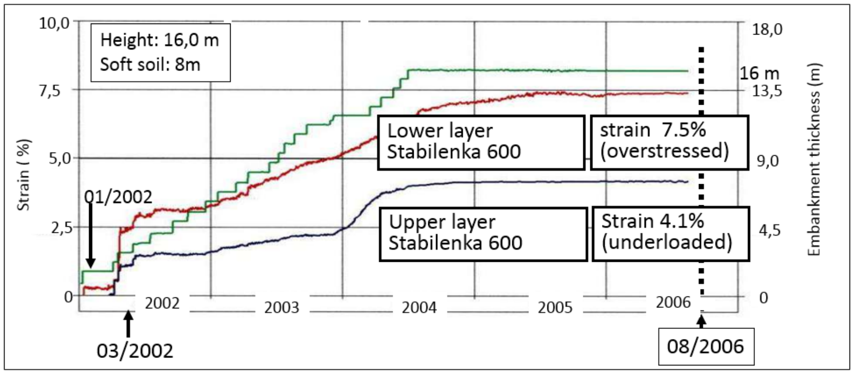


Figure 5. Total height of the embankment (green line related the right y-axis) and strains in the geosynthetic layers (red and dark blue line related the left y-axis) over time.

Beside the economic benefit of using only layer, the technical requirement can be clearly concluded from this experience. Using two layer of equivalent technical properties will not results in an even stress distribution, whereas using only layer a clear load allocation to this layer will occur. From the technical point of view, this results in higher or better said in the “designed” safety, whereas a multiple layer arrangement for those applications may result in an overstressed bottom layer and insufficient used top layer. Same observation have been reported by e.g. Row and Li (2003).

5. CONCLUSION

The paper reports about two case studies, where among others strain measurements have been conducted and in one case they are even ongoing for 33 years. The measurements have been done to gain a better system understanding at the beginning of the use of geosynthetic reinforcement products as basal reinforcement. They are very helpful in understanding the system behavior as well as control the construction speed. In order to assess the system behavior in different conditions, a numerical model is calibrated and validated in accordance with the field measurements. Afterwards, a series of numerical analyzes have been conducted to examine the behavior of the system in different conditions.

Based on the field observations/measurements and numerical results, the following remarks can be drawn:

1. Both case studies do confirm that the use of geosynthetic reinforcement is a successful and sustainable technique to construct embankments on very weak subsoils
2. From economical and technical perspectives the use of a single strong layer is recommended in comparison to a multiple layer arrangement with equal strength
3. By strain measurements alone do not allow to differentiate between creep and relaxation. Additional tensile force measurements would be need. However, a rough estimation can done by the use of the isochronous curves.
4. Numerical results show that very quick construction of the embankment leads to high excess pore pressure in the subsoil that results in an undrained shear failure of the slope.
5. The maximum settlement at the base of the embankment reduces by increasing the stiffness of subsoil while the difference between the short and long term settlements of the embankment reduces with an increase in the stiffness of the soft subsoil.
6. The tensile modulus of the basal reinforcement does not have a significant influence on the magnitudes and profile of the settlement. The use of reinforcement with higher tensile modulus leads to slightly more uniform settlement at the base of embankment.

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