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# GERMAN RECOMMENDATIONS FOR REINFORCED EMBANKMENTS ON PILE-SIMILAR ELEMENTS

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ABSTRACT: The construction of embankments on soft underground is a common problem. In recent years a new kind of foundation, the so-called "geosynthetic reinforced pile-supported embankment", was established. Until now the system behaviour can only be described analytically by simplified geomechanical models. Furthermore, there are simplified calculation procedures, which allow the dimensioning of the geosynthetic reinforcement. In the course of the revision of the EBGEO (German Recommendations for Geosynthetic Reinforced Earth Structures), new recommendations for soil reinforcements above pile-similar elements under static loading were worked out. These new developed analytical methods represent a new State-of-the-Art and enable a realistic and suitable approximation of the bearing behaviour of the composite structure.

## INTRODUCTION

Soil improvement and reinforcement techniques have undergone a significant development during the last decade, especially as a result of the increasing need to construct on soft ground providing economical solutions. Designing structures, such as buildings, walls or embankments on soft soil raises several concerns.

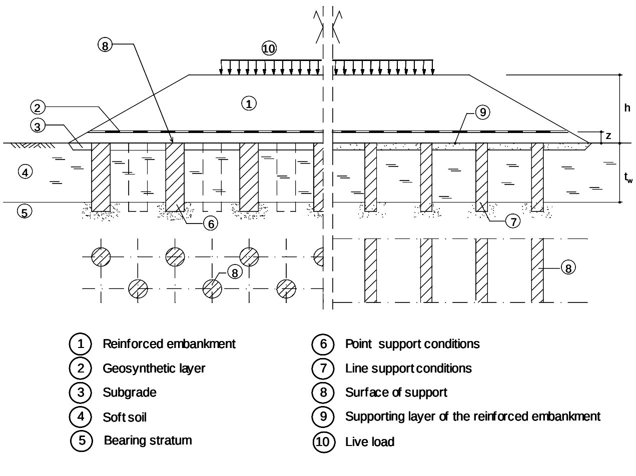


Fig. 1 Geosynthetic-reinforced pile-supported embankment

They are related to bearing capacity failures, intolerable settlements, large lateral pressure and movement, and global or local instability. A variety of techniques may be used to address the above concerns. These include pre-loading the soft soil, using light-weight fill, soil excavation and replacement, geosynthetic reinforcement and soil improvement techniques. In recent years a new kind of foundation, the so-called “geosynthetic-reinforced pile-supported embankment” was established (Fig.1). The pile elements (e.g. concrete piles, cemented stone columns, walls etc.) are placed in a regular pattern through the soft soil down to a lower load-bearing stratum.

Three possible support conditions are illustrated in Fig.2. Piles are typically arranged in rectangular or triangular patterns in practice.

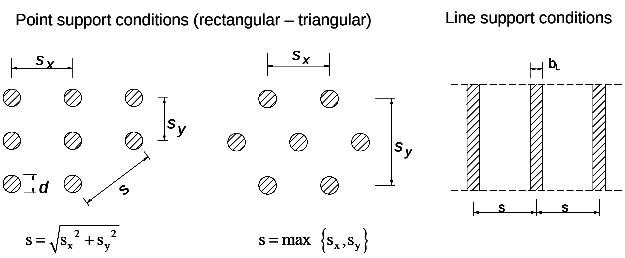


Fig. 2 Support conditions and definition of the distances

Above the pile heads, the reinforcement of one or more layers of geosynthetics (mostly geogrids) is placed.

In Germany the geosynthetic-reinforced pile-supported systems have been used for several applications, especially for highway and railroad embankments (Alexiew and Gartung 1999), (Alexiew 2001).

The systems have proved to perform well regarding both bearing capacity and serviceability if designed and

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constructed in an appropriate (Alexiew and Gartung, 1999).

Until now the system behaviour can be described analytically only by simplified geomechanical models. Furthermore, there are simplified calculation procedures, which allow the dimensioning of the geosynthetic reinforcement (e.g. Hewlett and Randolph 1988), (BS 8006 1995), (Kempfert et al. 1997), (Alexiew 2002). To examine the bearing mechanisms in the system and to derive a better analytical model, a research project has taken place at the Institute of Geotechnics, University of Kassel (Kempfert et al. 1999), (Zaeske 2001), (Zaeske and Kempfert 2002). The developed design procedure will be introduced soon into Chapter 6.9 "Reinforced earth structures on point- or line-shaped bearing elements" (Empfehlung 6.9 2003) of the new edition of the EBGEO (German Recommendations for Geosynthetic Reinforcement). This new analytical method represents a new State-of-the-Art. It is believed to be more precise and realistic than the "older" procedures available, which was confirmed by experiments (Zaeske 2001); at the same time it is more sophisticated and like other procedures available limited mostly to non-cohesive fills. An overview of common procedures today is given e.g. in (Alexiew 2002).

The general load transfer mechanisms, model test results and the new method of calculation and the construction recommendations for this kind of foundation as recommended in Chapter 6.9 of the EBGEO will be described shortly.

## LOAD TRANSFER MECHANISMS

The stress relief of the soft soil results from an arching effect in the reinforced embankment over the pile heads and a membrane effect of the geosynthetic reinforcement. Due to the higher stiffness of the piles in relation to the surrounding soft soil, the vertical stresses from the embankment are concentrated on the piles, simultaneously soil arching develops as a result of differential settlements between the stiff pile heads and the soft soil between them. The 3D-arches span the soft soil and the applied load is transferred onto the piles and then to the bearing stratum. The redistribution of loads in the embankment depends on the systems geometry, the strength of embankment soil and the stiffness of "piles".

A modified stress-distribution theory was developed (Zaeske 2001). Additionally, a concept to take into account the supporting soft soil upwards counter-pressure between the piles in a deformation-related way was introduced including the tensile stiffness of reinforcement and the oedometric modulus of soft soil. Differential equations had to be developed to reflect this interaction (Zaeske 2001) (Fig.3).

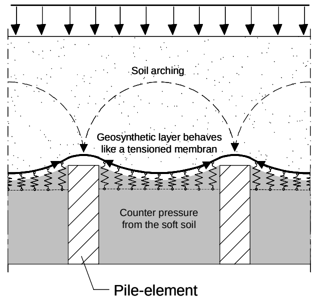



Fig. 3 Mechanisms of load transfer and interaction

## RESULTS OF MODEL TESTS UNDER STATIC LOADING

Three-dimensional model tests in a scale of 1:3 were carried out to investigate the bearing and deformation behaviour and to check and verify the concept and theory mentioned above. A group of four piles was placed in a weak soil of peat in a rectangular grid, above which a reinforced or unreinforced sand fill was placed in different heights (Fig.4).

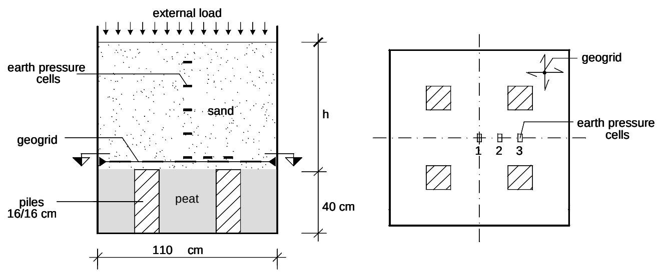



Fig. 4 Typical 1:3 scale test arrangement

The stress distribution in the reinforced sand layer was recorded by pressure cells. The part of the load carried by the piles was measured by load-cells and allowed a comparison with the measured stress field in the sand. Under static loading the dependency of the stress transfer on the geometric boundary conditions and the shear strength of the sand fill was verified.

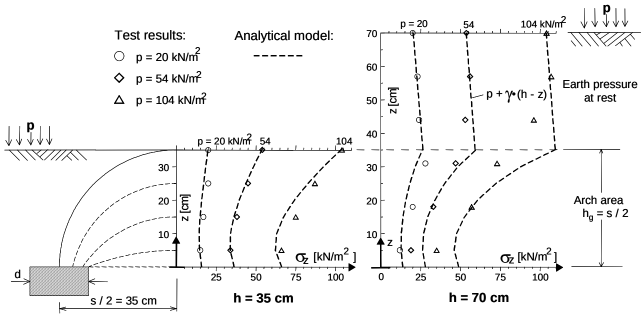



Fig. 5 Test results versus analytical model

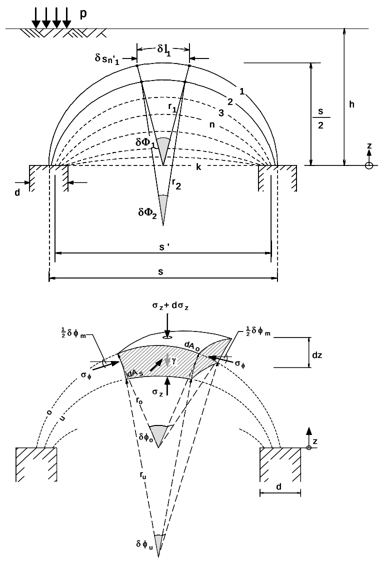
Similar to field measurements, the strains in the geogrid were found to be relatively small, provided that reaction stress of the underlying soil between the rigid pile elements is mobilised. In addition to the model tests, numerical investigations with the finite element method (FEM) were performed for static conditions. The evaluation of the FE-calculations resulted in further information on the stress distribution in the reinforcing layer and the resulting load transfer onto the piles.

After these verifications, the new method became part of Chapter 6.9 of the new edition of the EBGEO (draft) and is explained in the following chapter.

## DESIGN RECOMMMENDATION EBGEO

The design procedure recommended in Chapter 6.9 of the EBGEO is divided into two steps:

In the first step the load/stress distribution in the embankment is evaluated without considering any geosynthetic reinforcement, which results in the vertical stresses on top of the piles ($\sigma_{zs,k}$) and on the soft subsoil between them ($\sigma_{zo,k}$). The analytical model is based on the lower bound theorem of the plasticity theory and results from pretended directions of the stress trajectories in the reinforced soil body. According to the numerical and experimental results the stress state in the reinforced embankment is divided into a zone, where the earth pressure at rest can be assumed, and an arching region, where the stress redistribution takes place (Fig.4). Equation (1) shows the differential equation derived from the equilibrium of forces of the three-dimensional soil element in radial direction (Fig.6).




$$-\sigma_z \cdot dA_u + \left(\sigma_z + d\sigma_z\right) \cdot dA_o - 4 \cdot \sigma_\Phi \cdot dA_s \cdot \sin\left(\frac{\delta\Phi_m}{2}\right) + \gamma \cdot dV = 0 \qquad (1)$$

Fig. 6 "Arching" (Zaeske 2001), (Zeaske and Kempfert 2002)

The solution of the equation gives the vertical stress $\sigma_z(z)$ inside the arch. The vertical pressure on the soft soil $\sigma_{zo,k}$ results from the limit $z \to 0$, Equation (2). For more convenience, $\sigma_{zo,k}$ can also be derived from dimensionless design graphs (e.g. Fig.7 for $\sigma'_k = 30°$). In the second step, the vertical pressure $\sigma_{zo,k}$ is applied to the geosynthetic reinforcement as external load.

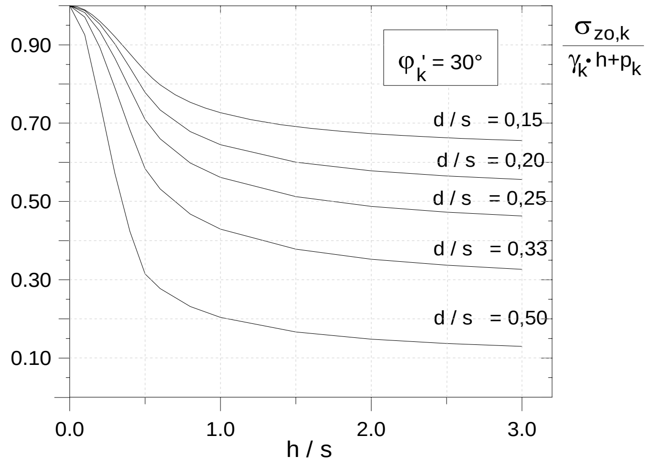



Fig. 7 Vertical stress $\sigma_{zo,k}$ on the soft soil

To predict the stresses in the reinforcement, an analytical model is applied based on the theory of elastically embedded membranes (Zaeske 2001). The maximum strain in reinforcement (i.e. the maximum tensile force) is concentrated in the band bridging two neighboured piles (despite the common engineering sense, it was confirmed by the experimental work as well). Therefore the analytical model assumes that the maximum stress in the geosynthetic membrane takes place within the width $b_{Ers.}$ and may be calculated based on a planar system (Fig.8). Biaxial geogrids must be analysed both in x- and y-direction.

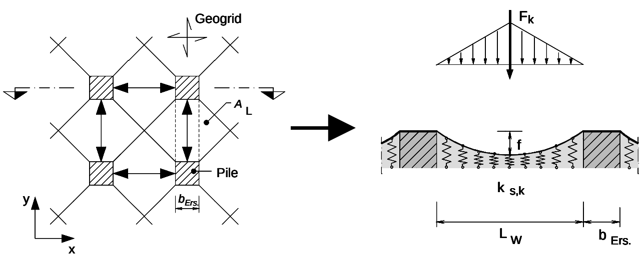



Fig. 8 Load transfer and simplified planar (2D) bearing system (Zaeske 2001), (Zeaske and Kempfert 2002)

The resulting triangular vertical strip load $F_k$ on the geogrid strip is calculated from the pressure $\sigma_{zo,k}$ and the loaded area $A_L$ (Fig.9).

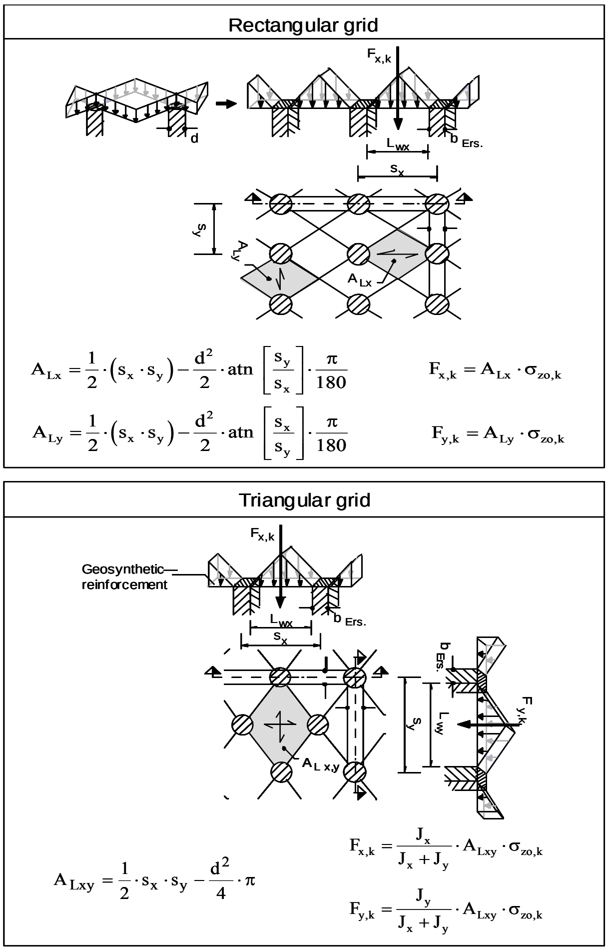


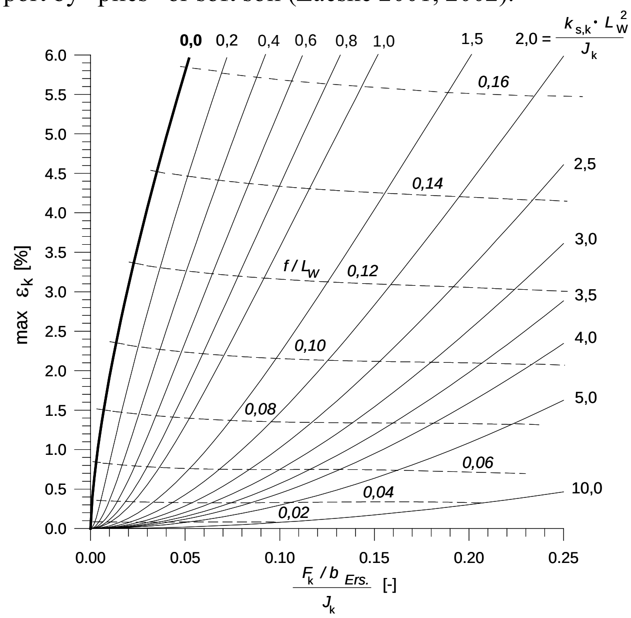

Fig. 9 Calculation of the resulting force $F_k$ assigned to the load influence area $A_L$

The influence of the bearing effect of the soft soil between piles is considered by using a modulus of subgrade reaction.

The maximum strain in the geosynthetic reinforcement results from the tensile stiffness $J_k$ of the geosynthetic, the modulus of subgrade reaction $k_{s,k}$ of the soft soil, the total vertical load $F_k$ and the dimensions $b_{Ers}$ and $L_w$. Since all geosynthetics tend to creep, the tensile modulus $J_k$ is time-dependent and has to be red out from the real isochrones of the geosynthetic reinforcement; the latter is essential.

In Empfehlung 6.9 (2003), the value of $\sigma_k$ can be taken from a dimensionless design graphs, see Fig.10). Finally, the tensile force in the reinforcement $E_{M,k}$ (M = membrane) can be calculated directly as a function of the strain of the geosynthetic, Equation (5). For two geosynthetic reinforcements the calculated force is divided with respect to the ratio of their tensile moduli.

By an inclined surface of the reinforced embankment geosynthetics are stressed by additional horizontal forces. The lateral thrust can be considered on the safer side assuming an active earth pressure condition without any support by "piles" or soft soil (Zaeske 2001, 2002).




| | | |
|---|---|---|
| $J_k$ | tensile stiffness of the geogrid | [kN/m] |
| $L_w$ | clear distance = (s-d) | [m] |
| $k_{s,k}$ | modulus of subgrade reaction | [kN/m³] |

$$k_{s,k} = \frac{E_{s,k}}{t_W} \qquad (3)$$

| | | |
|---|---|---|
| $b_{Ers.}$ | width of support | [m] |

$$b_{Ers.} = \frac{1}{2} \cdot d \cdot \sqrt{\pi} \qquad (4)$$

Stess in the reinforcement:

$$E_{M,k} = \varepsilon_k \cdot J_k \qquad (5)$$

Fig. 10 Maximum strain in the geosynthetic reinforcement

## CONSTRUCTION RECOMMENDATIONS

Based on German and international experience with geosynthetic-reinforced pile-supported embankments, practical reasons, experimental results and the validity of the analytical model following recommendations are established:

The center-to-center distance s and the pile diameter d of the piles resp. pile caps should be chosen as follows:

- (s - d) ≤ 3.0 m resp. (s - $b_L$) ≤ 3.0 m: in the case of static loads
- (s - d) ≤ 2.5 m resp. (s - $b_L$) ≤ 2.5: in the case of heavy live loads
- d / s ≥ 0.15 resp. $b_L$ / s ≥ 0.15
- (s - d) ≤ 1.4 (h - z)

The distance between the reinforcement layer and the plane of the pile/column/wall heads should be as small as possible, in order to achieve maximum efficiency of the geosynthetic membrane. However, it is recommended to have a safe distance (interlayer) between the lowest reinforcement and the pile heads in order to prevent a structural damage of the reinforcement because of shearing at the edge of the pile heads.

- maximum two reinforcement layers
- z ≤ 0.15 m for single layer reinforcement
- z ≤ 0.30 m for two layers reinforcement
- for two layers the distance between the geosynthetic layers should be 15 to 30 cm
- design value of the tensile strength $R_{Bd}$ ≥30 kN/m; ultimate strain ≤ 12 %.
- Overlapping is generally allowed, but only just above the pile (caps) and only in the secondary bearing direction; length of overlapping ≥ d.

## PROJECT "RAILWAY HAMBURG – BERLIN"

As part of the improvement of the existing railway line Hamburg-Berlin, the section Büchen-Hamburg and the section Paulinenaue-Friesack were upgraded in 2003 by the German Rail company (Deutsche Bahn AG), to allow a train speed of 230 km/h. Due to very soft organic soil layers (peat and mud) and the insufficient bearing capacity of the embankment, an improvement of the railway embankment was necessary in this sections. As improvement method a reinforcement of the embankment with geogrids over columns, installed with the Mixed-in-Place method (MIP, can be characterized as a wet deep mixing technique) was executed (Fig.11).

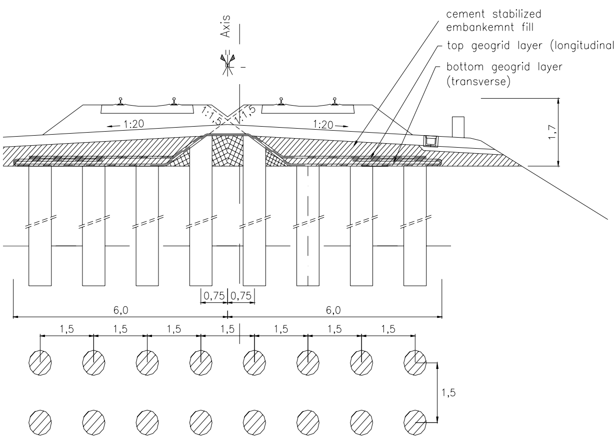



Fig 11 Foundation system Section Büchen-Hamburg

Between two improved sections, better soil conditions are given, therefore no columns were installed. In this 75 m long part only a reinforcement of the embankment with two Geogrids was executed. In the sections with columns, underneath a 3 to 5 m fill of medium dense packed silty and gravely sand with slag and organic admixtures, very soft peat and mud layers, with a total thickness of 0.5 to 2 m, are present. The peat has a water content of 80 to 330% and an organic content between 25 and 80%. Underneath these soft layers, slightly silty sand layers with a thickness up to 8 m are present, which are medium dense packed. At the base of the sand layers, boulder clay is present, which has a soft to stiff consistency and a water content of 10 to 20%.

During the improvement work, a single track operation at 90 km/h was maintained. The operated track was secured by sloping the ballast bed, the protective layer and the embankment (Fig.12). This made possible the construction of the geogrid reinforcement across the total embankment width. The MIP-columns were installed after the excavation of the protective layer. Prior to the setting of the MIP material, the columns generally were shortened to a level of 1.7 m below top of rail during the following excavation stage. The columns adjacent to the embankment axis, however, couldn't be shortened to 1.7 m below the rail level, which resulted in a cover of less than 1.5 m on top of the columns. Nevertheless, this option was favoured over a sheet pile wall, for instance, since the retracting of sheet piles could lead to unexpected settlements.

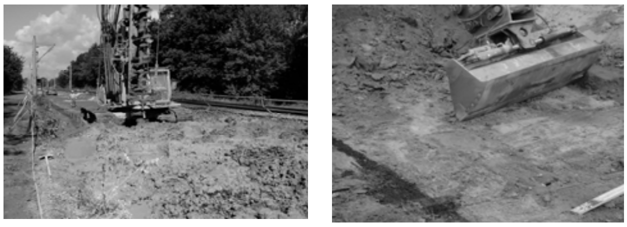


Fig 12 Installation of MIP-columns (left), shortening of the MIP-columns (right).

The MIP-columns were installed using a single auger (Fig.12). A cement slurry is injected continuously into the soil during the penetration as well as during the retrieval of the auger. Due to the rotation of the auger, the cement slurry is mixed with the soil. The MIP-technique is free of vibrations and displacements and therefore had no effect on the ongoing railway traffic on the other track. The cement columns (diameter 0.63 m) were installed in a square 1.5 x 1.5 m grid. The composition of the binder (water, cement and bentonite) and the water/binder ratio (approx. 1.0) was determined in laboratory tests on trial mixed samples. During the 1st improvement stage (track Hamburg-Berlin), approx. 800 l/m$^3$ binder were mixed into the soil. During the 2nd stage (track Berlin-Hamburg), the binder was mixed into the soil to the extent where a homogenous soil / binder mixture was obtained. This resulted in a variable, soil dependant binder quantity. The depth of the columns was determined on the basis of cone penetration tests prior to column installation.

On top of the MIP-columns two layers of Fortrac® PVA geogrid type M 400/30-30 were placed (Fig.11). Since the geogrids are loaded in longitudinal direction only, the short-term tensile strength in transverse direction was put at only 30 kN/m, whereas the required short-term tensile strength in longitudinal direction was put at 400 kN/m.

The 1st geogrid layer was placed in transverse direction directly on top of the MIP-columns. This geogrid was rolled up near the embankment axis during the 1st construction stage, and later laid across the whole embankment in the 2nd stage. The 2nd geogrid layer was placed in longitudinal direction (Fig.11).

To obtain a uniform bearing platform for the ballast bed, 2.5 to 3% cement was added to the filling material. The top of this cement stabilization was roughened to ensure a sufficient friction with the upper protective layer. To avoid an influence of hydrolysis of the cement, polyvinylalcohol was used as geogrid material.

The settlement behaviour of the tracks was monitored by means of geodetic measurements of the outer rail of both tracks. The measurements were conducted in 3 measurement sections each 20 m in consisting of 5 measuring points with a spacing of 5 m. These measurement sections were set up at locations with unfavourable soil conditions. The results of the settlement measurements over 6 months of train operation are presented in Fig.13. On both tracks the train speed was up to 160 km/h. The measurements show, that the track Hamburg-Berlin has settled up to 7 mm in a period of 6 months after reopening the track. This settlement can be considered as small since usually a settlement of 10 mm to 15 mm will occur, due to compaction of the ballast bed, the protective layer and embankment, even if the soil conditions are favourable. Also, it has to be considered~~,~~ that the geogrids have to deform slightly to become active.

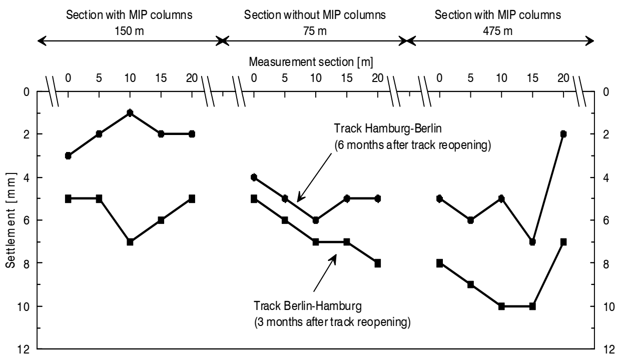



Fig 13 Settlement measurements

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