Stabilization of soft organic soils with cement columns using the Mixed-in-Place technique (MIP) for a railway embankment

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ABSTRACT: The very soft organic subsoil of 2 sections of the railway line Hamburg-Berlin was improved by installing cement columns with the Mixed-in Place method, which can be characterized as a wet deep mixing technique, and by reinforcing the embankment with geogrids. In this paper, firstly the soil conditions and the improvement measures are described in general. Then a short discription of the theoretical bearing and deformation behaviour and the design and calculation method is given. Furthermore the installation of the MIP columns and quality control measures are described. Finally some results of settlement measurements are discussed.

1 INTRODUCTION

As part of the improvement of the existing railway line Hamburg-Berlin, the section Büchen-Hamburg was upgraded in 2003 by the German Rail company (Deutsche Bahn), 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 two sections with a total length of 625 m near the railway station Büchen..

As improvement method a reinforcement of the embankment with geogrids over cement columns was executed. The cement columns were installed with the Mixed-in-Place method (MIP). During the improvement works, a single track operation at 90 km/h was maintained..

2 SOIL CONDITIONS

The railway embankment consists of medium dense packed silty and gravely sand with slag and organic admixtures. Underneath the 3 to 5 m fill, 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%.

3 IMPROVEMENT

The basic purpose of a reinforced embankment over columns is to relieve the soft soils of the load by distributing the loads through the columns to a bearing layer (here: boulder clay).

The cement columns (diameter 0.63 m) were installed in a square 1.5 x 1.5 m grid using the MIP-technique, which can be characterized as a wet deep mixing technique.

Using a single auger, 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.

On top of the MIP-columns two layers of Fortrac® PVA geogrid type M 400/30-30 were placed (fig. 1). 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 cover over the columns generally has a thickness of 1.5 m, i.e. the requirements of the guideline DS 804 (Deutsche Bahn) are complied with.

Between the two improved sections, no MIP-columns were installed. In this 75 m long part only a reinforcement of the embankment with two Geogrids was executed. In the transition zones, between the improved sections and non-improved embankment, a geogrid reinforcement of the embankment was carried out over a length of 10 to 20 m.

Since there existed no proved experience with the improvement technique using Mixed-in-Place Columns for railway tracks in Germany and up to then this technique had only been applied in sandy soils with an organic content of max. 3%, an ‘individual approval’ from the German Federal Railways Office (Eisenbahn-Bundesamt EBA) had to be obtained.

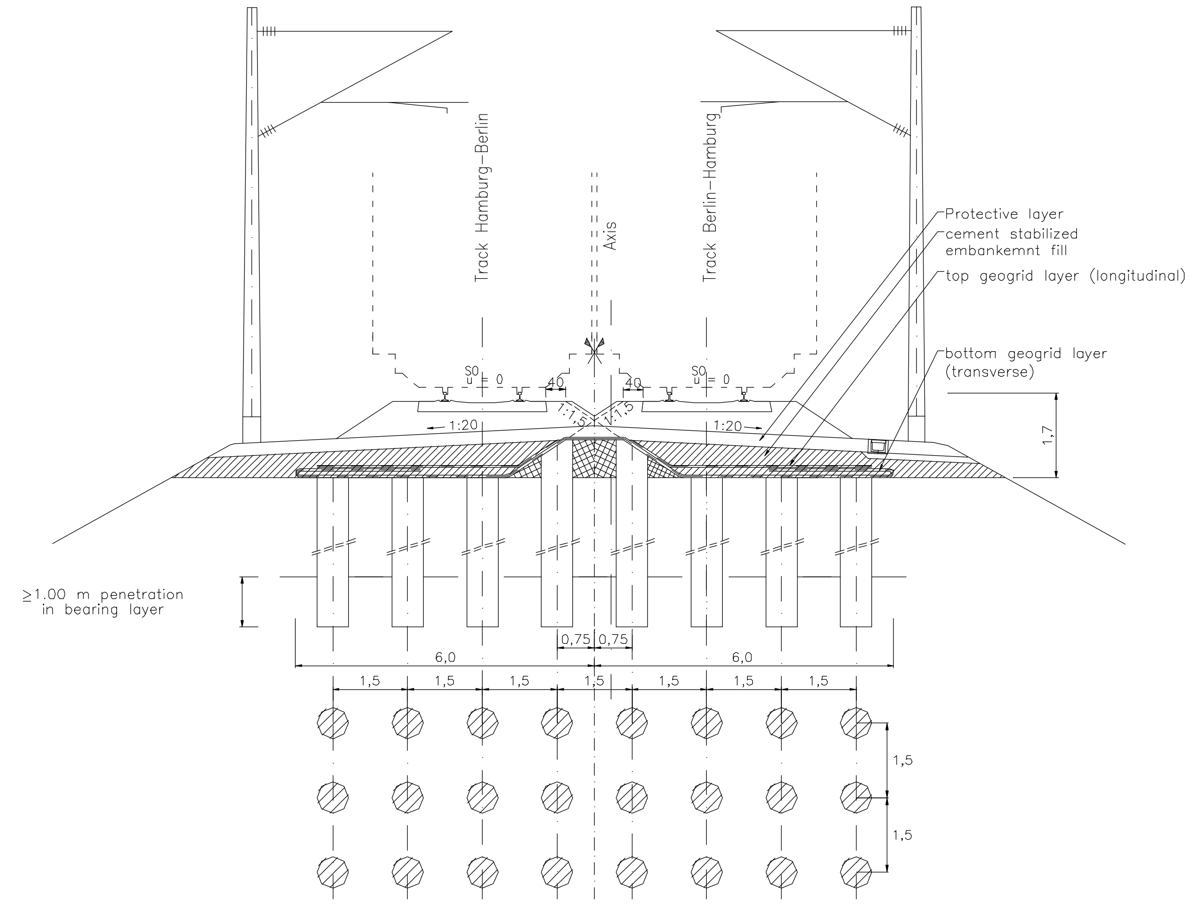

Figure 1. Foundation system.

4 CALCULATION AND DESIGN

4.1 Theoretical Background

The stress relief from the soft soil results from an arching effect in the reinforced embankment over the pile heads and a membrane effect of the geosynthetic reinforcement, see figure 2 (Kempfert et al., 2004).

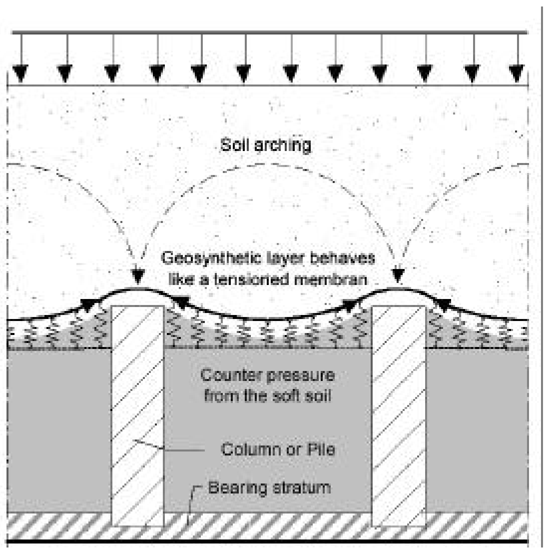

Figure 2. Mechanisms of load transfer and interaction.

Due to the higher stiffness of the columns 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 column heads and the surrounding soft soil.

The 3D-arches span the soft soil and the applied load is transferred onto the piles down to the bearing stratum The stress distribution can be modelled in various ways. Figure 3 shows, for example, a system consisting of several arching shells (Zeaske, 2001; Zaeske and Kempfert, 2002).

This model leads to a differential equation, which is a function of the described vertical stresses σ_z [z] in the arching system (Zaeske, 2001):

$$-s_z \cdot dA_u + \left(s_z + ds_z\right) \cdot dA_o - 4 \cdot s_F \cdot dA_s \cdot sin\left(\frac{dF_m}{2}\right) + ? \cdot dV = 0 \quad (1)$$

For the areas above the arches a load depending stress distribution is assumed. The effective stress on the soft soil stratum σ_{zo} results from the limiting value consideration z $\rightarrow$ 0 with t = height of the load depending arch, so equation (2) can be formulated.

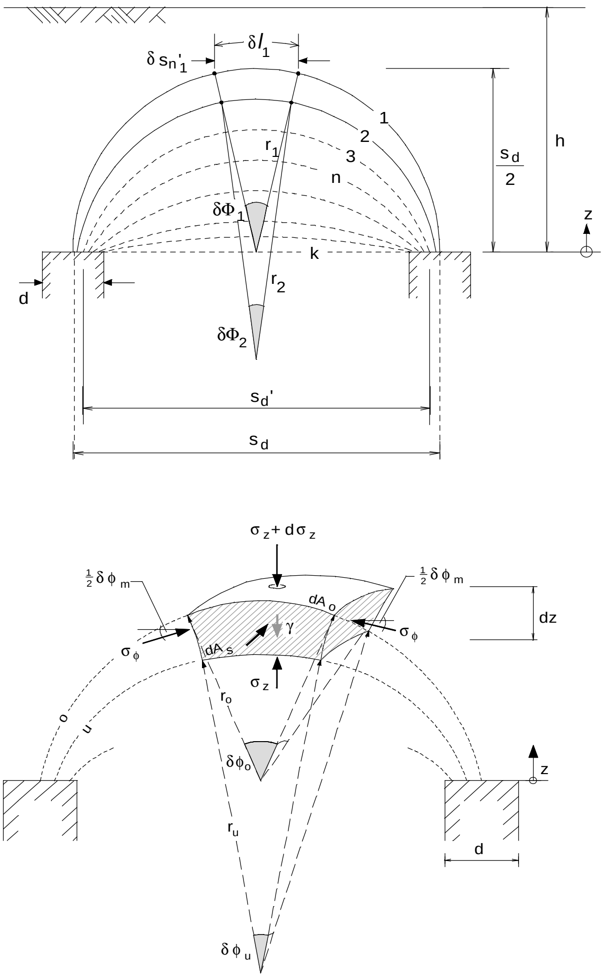

Figure 3. Theoretical arching model.

$$s_{zo} = l_1^{\,c} \cdot \left(g + \frac{p}{h} \right) \cdot$$

$$\left(h \cdot \left(l_1 + t^2 \cdot l_2 \right)^{-c} + t \cdot \left(\left(l_1 + \frac{t^2 \cdot l_2}{4} \right)^{-c} - \left(l_1 + t^2 \cdot l_2 \right)^{-c} \right) \right) \tag{2}$$

with:

$$\chi = \frac{d \cdot (K_{krit} - 1)}{l_2 \cdot s_d},$$

$$l_1 = \frac{1}{8} \cdot (s_d - d)^2,$$

$$l_2 = \frac{s_d^{\,2} + 2 \cdot d \cdot s_d - d^2}{2 \cdot s_d^{\,2}}$$

$$K_{krit} = \tan^2 \left[45° + \frac{j'}{2} \right]$$

Simplified σ_{zo} can also be derived from dimensionless diagrams (DGGT, 2003). In figure 4 analytically calculated stresses are compared with those resulting from model tests (Zaeske and Kempfert, 2002).

The loading of the reinforcement is expressed by the differential equation (3) of the elastic supported cable, in which the vertical displacement z and the horizontal force H, according figure 5 (Zeaske, 2001; Zaeske and Kempfert, 2002) are the unknown variables.

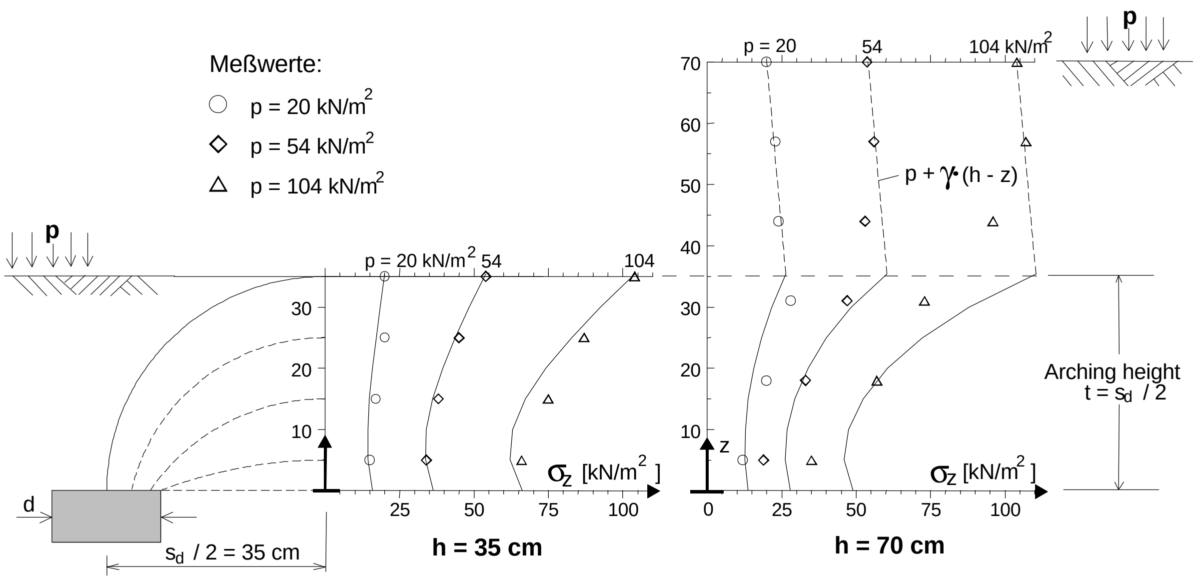

Figure 4. Stresses in bearing layer: theoretical vs. tests (height of the bearing layer h = 35 und 70 cm).

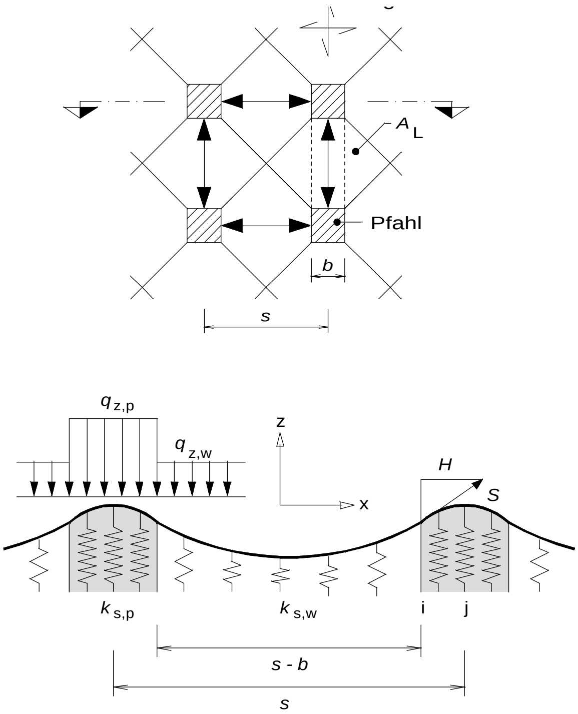

Figure 5. Bearing system for membrane effect.

$$\frac{d^2z}{dx^2}=\frac{q_z}{H}+\frac{k_s\cdot z}{H} \tag{3}$$

with $H=\dfrac{2\cdot\int_0^i\sqrt{1+z_W{'}^2}\,dx+2\cdot\int_i^j\sqrt{1+z_P{'}^2}\,dx-l_0}{2\cdot\int_0^i\left(1+z_W{'}^2\right)dx+2\cdot\int_i^j\left(1+z_P{'}^2\right)dx}\cdot J$

Finally the loading of the reinforcement S can be calculated directly as a function of the elongation ε (J = stiffness) of the geosynthetic (for dimensionless diagrams, see DGGT, 2003):

$$S[x]=e\,[x]/J=H\cdot\sqrt{1+z'^2[x]} \tag{4}$$

4.2 Design and German Recommendations

The design of this bearing system was done according to German Recommendations for Reinforcements with Geosynthetics EBGEO (DGGT, 2003).

The aim of these Recommendations is to harmonize and further develop the methods, according to which reinforced earth structures are designed, calculated and carried out. Since 1989 the Recommendations have been drawn up by the working group for earth reinforcements of the German Society for Geotechnical Engineering under the name "EBGEO" and are similar to a set of standards. The recommendations help in designing and calculating reinforced earth structures, unifying approaches to loads and methods of calculation and improve the profitability of reinforced earth structures.

The recommendation “Chapter 6.9 – Reinforced soil structures above point- or line shaped bearing elements” is based on the described theoretical background and contains e.g. dimensionless diagrams for calculation of the vertical stresses and the tension forces in the geogrids. It was released as a draft to the public in 2003. Chapter 6.9 will soon be part of the new edition of the EBGEO. For further information, see Kempfert et al. (2004a and 2004b).

Using the Recommendations the required short-term tensile strength in longitudinal direction was calculated to 400 kN/m.

5 MIP-COLUMNS

5.1 Installation

The MIP-columns were installed after the excavation of the protective layer (fig. 6). 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 (fig. 7).

Figure 6. Installation of MIP-columns.

Figure 7. Shortening of the MIP-columns.

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/m3 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 operated track was secured by sloping the ballast bed, the protective layer and the embankment, according to the requirements of the German railway guideline Ril 836. This made possible the construction of the geogitter reinforcement across the total embankment width.

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.

The depth of the columns was determined on the basis of cone penetration tests prior to column installation.. In total, 3,260 MIP-columns of a length between 5 and 8 m were installed (in total 21,000 m).

5.2 Quality control

In order to prove that the MIP-columns can comply with the design criteria also in very soft organic soils, a quality plan was set up.

As part of this plan, installation parameters such as the penetration depth, the penetration and retrieval speed, the amount of binder etc. were continuously recorded and controlled for each column.

To verify the homogeneity of the columns, core samples were taken out of 11 test columns by the use of liner samplers (fig. 8, 9 and 10).

The visual inspection of the core samples showed that a homogenous soil-binder mixture, with only local small peat enclosures could be produced. Some of the test columns were partially excavated, to determine the extension of these peat enclosures. For laboratory testing purposes, wet grab samples were extracted from 4.5% of the columns.

Per 500 m^3 of treated soil, 6 unconfined compression tests were carried out after 28 days, to determine the unconfined compressive strength q_u .

The results of the unconfined compression tests are presented in fig. 11.

According to the tests, unconfined compressive strength after 28 days of all samples exceeded the design criteria of qu ≥ 2.2 MN/m2. The differences between the compressive strength of the individual samples are probably due to different quantities of binder.

Figure 8. Installation of Liner.

Figure 9. Liner samples.

Figure 10. Column after liner extension.

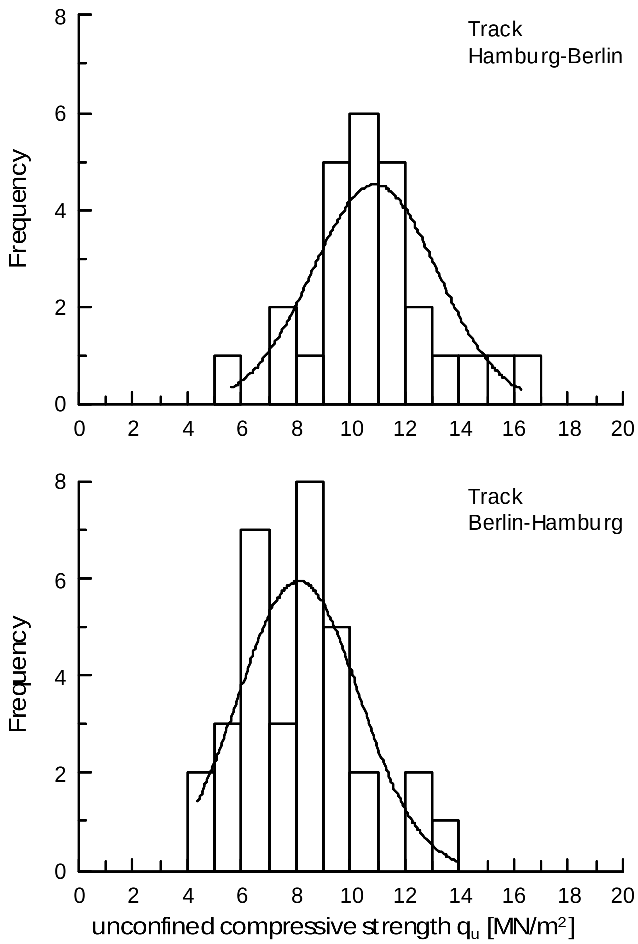

Figure 11. Unconfined compressive strength.

6 REINFORCED EMBANKMENT

6.1 Geogrids

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. To avoid damage to the geogrid, the MIP-columns were scraped off prior to the setting of the treated soil. Near the embankment axis, the excavation, the placing of the geogrids and the backfilling took place in 6 m broad sections during periods of no rail traffic. The 2nd geogrid layer was placed in longitudinal direction. Fig. 12 shows the placing of the geogrids.

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 short-term tensile strength in transverse direction differs from the technical delivery conditions for geogrids according to the German Railway Standard BN 918039 (Deutsche Bahn), which states that the short-term tensile strength in transverse direction should be at least 20% of the short-term tensile strength in longitudinal direction.

Figure 12. Placing of geogrids.

6.2 Filling material

For filling material between and on top of the geogrids, a gap graded gravel-sand mixture (soil group SI according to the German Standard DIN 18196) with a coefficient of uniformity ≥ 6 was used. The filling material was stabilized with 2.5% to 3% cement by using a mixing plant.

The filling material was placed in layers of maximum 30 cm thickness in accordance to the Ril 836. Each layer was compacted to a degree of compaction of at least D_{Pr} = 98%.

7 MONITORING

In accordance with the clauses of the ‘individual approval’ from the Federal Railways Office, 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 length, consisting of 5 measuring points with a spacing of 5 m. These measurement sections were set up at locations with unfavourable soil conditions.

The by now available measurements cover 6 months of train operation on the Hamburg-Berlin track and 3 months of train operation on the Berlin-Hamburg track, from the time of their respective reopening. On both tracks the train speed was up to 160 km/h. The results of the settlement measurements are presented in fig. 13. 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.

The measurements also show in both improved sections, that the settlement of the track Hamburg-Berlin, are not greater than the settlement of the track in the section in which no MIP-columns were installed.

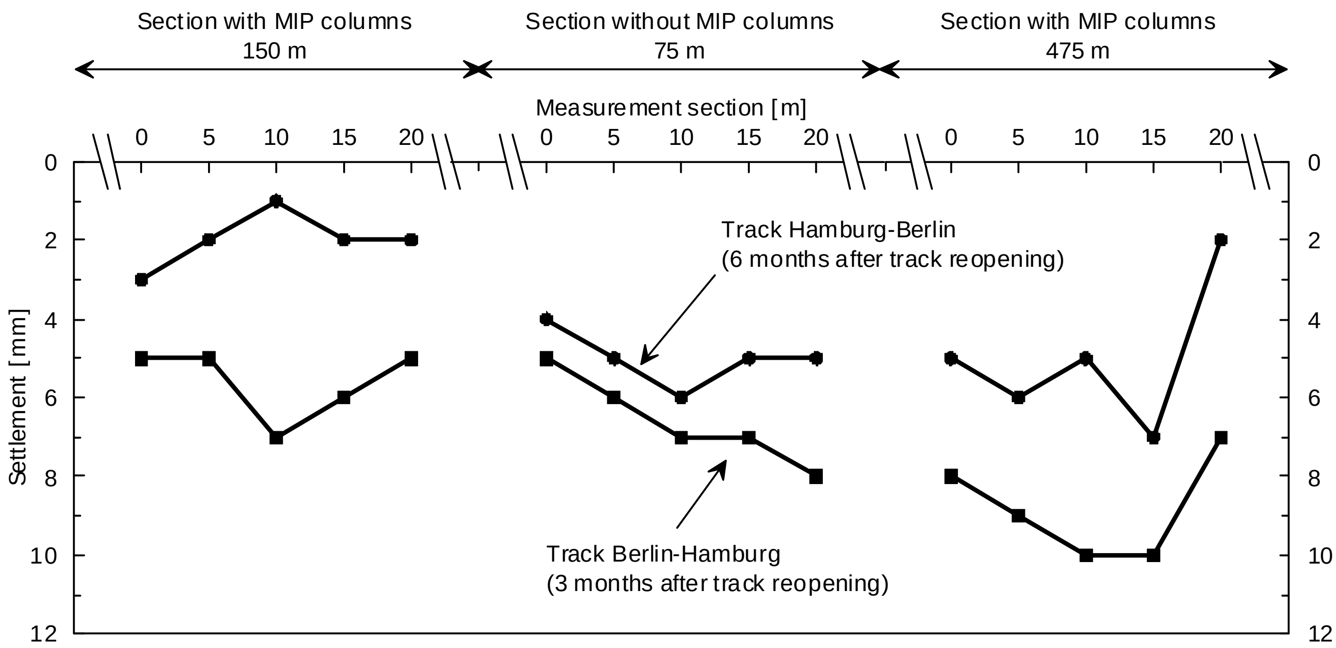

Figure 13. Settlement measurements

On the opposite track, Berlin-Hamburg, the settlements after 3 months amount up to 10 mm, i.e. the settlements of this track are approx. 3 mm greater than the settlements of the track Hamburg-Berlin.

From the measurements, it can be recognized that the measured settlements mainly are due to the compaction of the ballast bed and the protective layer, and hardly no settlements occur under rail traffic.

In general, it can be concluded that the effectiveness and success of the executed improvement of the very soft organic layers and embankment could be proved by means of the settlement measurements (see also Raithel et al. 2004).

8 CONCLUSIONS

To allow a train speed of 230 km/h, the very soft organic soil (peat and mud) of two small sections of the railway line Hamburg-Berlin was improved by installing cement columns with the Mixed-in-Place method. Also the embankment was reinforced with geogrids. In total, 3,260 MIP-columns of a length between 5 m and 8 m were installed (in total 21,000 m). During the improvement works, a single track operation at 90 km/h was maintained.

As part of the quality plan, several installation parameters were recorded. Also liner samples of the treated soil were taken out of some columns. By means of laboratory tests on stabilized soil samples, it was verified that the design criteria were met. The effectiveness of the executed improvement measures was proved by means of settlement measurements

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