FEM-Analysis and dimensioning of a sinkhole overbridging system for high-speed trains at Gröbers in Germany

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ABSTRACT: Roads and railroads crossing sinkhole areas are a problem of increasing importance. The range of possible solutions is dominated by geosynthetic reinforced systems. Although some analytical procedures are available for design, for an increasing number of cases they are not precise enough or not applicable. Numerical procedures provide some help. Based on the project Gröbers on the new high-speed rail link Leipzig-Halle such a case is discussed. The bearing system bridging sinkholes consists of a cement-stabilized soil block with basal geogrid reinforcement. The main problems of design were to check both serviceability and stability of the system for a pre-failure and a partial post-failure state as well. A commercially available FEM-code could not solve all problems, e.g. the post-failure analyses. Thus, analytical procedures were developed and applied. An overview of all checks and corresponding FEM and analytical calculations is shortly presented. The final project specifications were based on the design described. The bearing system is already built.

1 INTRODUCTION

A new railway node in Germany, including eight tracks, is about 800 m long and 120 m wide. This new railway section is situated in a post-mining area near the town of Gröbers. Two of the tracks belong to the new high speed link Leipzig – Halle designed for a speed of up to 300 km/h, the others are ‘conventional’ tracks for passenger- or heavy cargo-trains. The entire area is prone to subsidence due to the previous mining. As a result of regional experience, geological analyses and predictions the possible maximum diameter of a sinkhole ‘funnel’ below the tracks was defined by the owner (German Railroads, DB) with 4.0 m. Unfortunately, the location of all possible existing cavities and galleries in the underground is not known. Consequently, the ‘funnel’ at the terrain or below the tracks could open at any place.

During the first stage of the project many foundation methods for the rail-track system were analyzed (for example: deep foundation on piles, stiff reinforced concrete plate, dynamic compaction after deep excavation, etc.). Finally, a concept developed especially for this project was chosen. This concept is based on two general ideas: first, to ensure serviceability for a given period of time by a ‘bearing system’ overbridging the sinkhole and second, to monitor and register the opening of the sinkhole by a ‘warning system’. Thus, if a sinkhole opens the train speed will be decreased to 100 km/h, and the sinkhole will be re-injected by special techniques.

The bearing system consists of two main elements: a cement-stabilized soil block or ‘bearing layer’ (CSBL) from locally available cohesive soil and a geogrid reinforcement in a thin gravel layer at the blocks base. A warning layer is installed below the geogrid. This paper focuses on design issues of the bearing system only. For more information regarding warning-layer see Ast & Haberland (2002).

The system is depicted in a very simplified way in Figure 1 and in a more detailed way in Figure 2. The system was designed for a design life of at least 60 years and must be able to overbridge sinkholes with a diameter of up to 4.0 m for one month (i.e. during the injection works needed for filling the developed cavity).

Figure 1. Simplified cross section of the described system

Figure 2. Detailed cross section of a part of the described system at Gröbers

The geosynthetic reinforcement has three functions: to reinforce the CSBL before any partial failure occurs (similar to a reinforced ‘plate’ over smaller voids), to ensure a ‘membrane’ bearing effect for a partially collapsed part of the soil block (worst case, large voids), and to ensure for the same case a tied arch mechanism (‘bridge analogy’), ensuring the stability even after a partial collapse.

Based on preliminary calculations and the safety philosophy of the owner in that case the minimum thickness of CSBL was
decided to be ≥ 3.5 m. The expectation of an arch-shaped local collapse at the beginning of the CSBL-failure was based on experience and simplified calculation methods, e.g. Giroud et al. 1990. (For flat non-cohesive systems without arching see e.g. BS8006 1995, Alexiew 1997)).

Ultimate Limit State
(German "GZ1", DIN 1054 new)

Overall stability
(German "GZ 1 C")

Failure of structural components
(German "GZ 1 B")

Cement-stabilized Soil Bearing Layer (CSBL) or "Block"

Geogrid (GG)

A Plastic failure of CSBL due to large plasticized zones

Additional local critical zones ("Bridge analogy")

D before failure of soil "arch" in CSBL

E after failure of soil "arch" in CSBL

B Crest (Ridge) of "arch"

C Springing of "arch"

Settlements at rail level

Complete system

Relative settlements (inclination and deflection) parallel to rail axis

G absolute settlements at any point

H relative settlements (inclination, torsion) perpendicular to rail axis

Figure 3. Overview of the design checks performed.

Among the system stability for at least one month under traffic, serviceability (trafficability) was a major issue. The most rigorous requirement was the inclination (torsion) of a track perpendicular to its axis which was limited to < 1:500 (0.2 %).

2 DESIGN CONCEPT

The design calculations and dimensioning of the bearing system had to be performed according to the German codes for geotechnical structures (DIN 1054 1996) and for soil systems with geosynthetic reinforcement (EBEG 1997).

They both prescribe design analysis for two different limit states: the ultimate limit state (say ‘failure’) and the serviceability limit state (say ‘deformations’); corresponding different partial factors of safety for strengths, internal and external loads etc. have to be applied.

As mentioned above, engineering experience and some simplified pre-design calculations led to the expectations of an ‘arching’ in the high-cohesive cemented soil block. It was additionally decided to perform checks of the overbridging system after failure of the ‘arched’ part of the soil similar to an ‘arch bridge’ (consisting of soil) with a horizontal tie (consisting of the geogrid) between the springings, which is quite unusual for geotechnical engineering. It was decided as well to handle the problem as a 2-D instead of 3-D one, which is on the safer side.

A general overview of the design procedures and checks performed is given in Figure 3. The capitals in the corners are introduced to make citing easier later herein. This design procedures scheme was developed by the authors in interaction with the supervisors not only to meet all "formal" requirements of existing codes, but to check non-codified possible worst cases as well.

3 PROBLEMS, DIFFICULTIES AND POSSIBLE SOLUTIONS

The following items and problems had to be kept in mind respectively had to be solved (note, that the system belongs generally to the highest safety category):

1. Especially due to the high-speed tracks the requirements on the serviceability for at least 30 days of overbridging a sinkhole are extreme.
2. No analytical design procedures are available for reinforced cohesive soil systems overbridging voids.
3. Only insufficiently precise analytical procedures are available for non-cohesive soils and are not available at all for cohesive/cemented soils for calculation of the deformations above a sinkhole.
4. The position of a sinkhole can not be pre-defined beneath a 120 m wide, 6-track-wide cross-section.
5. When such a 'stochastic' sinkhole occurs some tracks can be under traffic and some not; this loaded/non-loaded situation has a stochastic character as well, etc.

Consequently, a sound and safe design can not be performed without using numerical methods. In this case a commercially available geotechnical FEM-code with implied geosynthetic reinforcement was used ("Plaxis"). On the other hand, using a FEM-code generates difficulties in respect of application of different partial safety factors. Additionally, no post-failure analysis is possible which was a general requirement in this case.
Summarizing, FEM was used for the cases (Fig. 3) 'A', 'D' and 'F', 'G' and 'H'. For the case 'E' (soil body has failed, but the geogrid reinforcement still has not) a new combined solution was applied. For the cases 'B' and 'C' ('check of stability of a remaining' soil block after failure of an 'arch-shaped' part of it) an unusual new solution was applied as well.

4. FEM-CALCULATIONS FOR THE CASES 'A', 'D', 'F', 'G' AND 'H' (FIGURE 3)

For the CSBL (soil block) a linear-elastic perfect-plastic model was applied (incl. Mohr-Coulomb). It was defined by the module of deformation E, Poisson's ratio ν, unit weight γ, angle of internal friction φ, angle of dilatancy ψ, cohesion c and (new in that project) the tensile strength of soil $R_{\text{tensile}}$.

While all other parameters are common and can be well defined, establishing a guaranteed value of $R_{\text{tensile}}$ was not easy, although there were no doubts that the value can be significant for cement-stabilized high-cohesive soil material. Three alternatives for estimation of $R_{\text{tensile}}$ were checked: by direct tension test on soil samples (would correspond to CD in Figure 4, but tests are very sensitive and uneasy to perform 'en masse'), by routine shear tests extrapolating the 'Mohr-Coulomb line' to the left and deriving an allowable tension stress (Fig. 4), which is very questionable and by experience-based standardized assumption according to DIN 4093 (1987) defining $R_{\text{tensile}}$ as 10% of the unconfined compressive strength. The latter alternative was chosen resulting in $R_{\text{tensile}}$.

![Figure 4. Estimation of design tensile strength of cemented cohesive soil](image)

Figure 4. Estimation of design tensile strength of cemented cohesive soil

$R_{\text{tensile}}$ definitely on the safer side but allowing for a large quantity of simple tests for later quality assurance with high confidence.

For the geogrid reinforcement only a linear-elastic model was available defined by a constant tensile module $J$ in kN/m ($J = \text{force, kN/m / strain, } \gamma$). The negative consequence and handicap is that theoretically the reinforcement can strain infinitely mobilizing a never ending force and can never fail, which is not correct. This handicap of the FEM-code used has been neutralized later by a separate post-soil-failure-analysis (see below) for the case 'E' in Figure 3.

The geogrids are defined additionally by their coefficient of interaction with the contacting soil. Both moduli and interaction coefficient were known for the family of geogrids 'Fortrac® A' used in this project (with an ultimate tensile strength (UTS) in the range of 1200 to 1400 kN/m at an ultimate strain of less than about 3% and a very low creep tendency, Alexiew (1997)). A specific geosynthetic related issue had to be considered, namely the creep of geogrid under tension during the 30 days of overbridging the opened 4m-void. For this purpose a time-dependent module $J$ was derived based on the isochronous curves of the 'Fortrac® A'-family (Figure 5 as illustration of the procedure: not with real values). Thus, for the geogrid two contrary processes had to be taken into account: on the one hand, creep reduces the allowable strength, on the other hand it leads to some increase of deflection thus reducing the effective tensile force (2nd order theory, elastic ropes and membranes, Moskaliev 1980). Consequently it was not clear in advance, which situation is decisive: the first minutes after sinkhole activation or the 30 days of overbridging. Thus, all calculations for the cases 'A', 'D' and 'H' (Fig. 3) were performed twice: with an one-minute tensile module and with a thirty-days tensile module. The same procedure using two different time-dependent moduli was used for case 'E', see below.

![Figure 5. Assessment of time-dependent tensile moduli: short-term and after some creep](image)

Figure 5. Assessment of time-dependent tensile moduli: short-term and after some creep

It was a significant complication amongst others in comparison e.g. to the project described in (Alexiew 1987), which had to hold the system only for 10 to 20 minutes.

The cases 'F' and 'G' were shown very quickly to be not critical; they are not discussed herein. As 'loads' for the FEM-calculation the unit weight of the soil and the external train load (52 kN/m²) multiplied additionally by a partial factor of 1.40 for dynamic effects were applied for all cases. It may be an issue of philosophy to handle or not the 'opening of 4 m sinkhole' as a 'load' respectively 'action' in the sense of e.g. (DIN 1954 1996, EBGE 1997). In any case the 'action' or 'occurrence' sinkhole opens' controls the design.

The FEM-computing included generally the following steps:
- simulation of construction, then setting all deformations (but not stresses) to zero;
- simulation of the 'opening of sinkhole' and application of the train load; opening at once, train load incrementally up to the design value;
- checking 'A' (Fig. 3): soil body (CSBL) collapses; yes/no;
- if yes, increase soil strength;
- checking 'D' (Fig. 3): tensile force in geogrid is higher than the design strength: yes/no; if yes, take a stronger type;
- checking 'H' (Fig. 3): perpendicular inclination higher than allowed: yes/no; if yes, increase soil stiffness;
- repeat the procedure until all checks are o.k.; the result is a set of minimum requirements on soil and geogrid parameters.
Due to the possible combination of train and void position and time (module of geogrid) the system was computed 26 times (despite the steps for increasing resistance parameters of soil and/or geogrid if not sufficient). Figure 6 shows a typical stress field and Figure 7 a typical vertical deformation just over the void.

Figure 6. Typical stress field from FEM-analysis

Figure 7. Typical vertical deformation of geogrid just above the void

5 ANALYTICAL CALCULATIONS FOR THE CASE ‘E’ (FIGURE 3)

Due to the codes applied (DIN 1054, EDGE 1997), but also due to common engineering sense, a worst case had to be analysed: an ‘arch-shaped’ part of CSBL (soil block) has collapsed (failed), ‘falls down’ and has to be born by the geogrid working as an elastic rope. The tendency can be identified in Figure 6, a simplified model is depicted in Figure 8.

Figure 8. Part of the cemented soil tending to fail just before collapse

The tensile force in the geogrid in such a critical case must be checked, but such a post-soil-failure analysis cannot be performed by FEM. A new procedure was applied by the authors: the stress field in CSBL was analyzed for a just-pre-failure-state in all cases computed, resulting in the definition of the largest possible height $h_0$ of ‘arch block’ after failure, say in the maximum possible vertical load on geogrid after collapse of the (nearly parabolic ‘arch’) (Figs. 8, 9). Considering the tensile strength $R_{\text{max}}$ of stabilized soil in that case results in smaller $h_0$ (Fig. 9) and reduced post-failure load on reinforcement, but this phenomenon was neglected on the safer side. Using a membrane-equation (Moskalev 1980) after some modifications the tensile force in the geogrid was calculated as a function of the parabolic load and, which is very important, depending directly on the tensile modulus $J$ of the reinforcement (Fig. 10).

Figure 9. Evaluation of the height of collapsed arch for post-failure analyses of the geogrid tension

$$H^2 = \frac{(D \cdot J)}{(2 \cdot L)} \quad \text{kN/m} \quad T = \frac{H}{\cos \alpha} \quad \text{kN/m}$$

$J = \text{material-} \quad \text{and time dependent tensile module, kN/m}$

$D_{\text{triangle}} = \left( \frac{q^2 \sin \omega \cdot L^3}{(2 \cdot L)} \right)$

("triangle" via area equivalence to "parabola")

$T \equiv H = \left( 0.02963 \cdot b_o^2 \cdot y^2 \cdot L^2 \cdot J \right)^{1/2} \quad \text{kN/m}$

$y = \text{unit weight of soil, kN/m}^3$

Figure 10. Direct calculation of tensile force in geogrid as a function of its tension module and load after collapse of ‘arch’

Due to the time dependence of the module $J$ (Fig. 5) the calculations were performed for the first minute and for thirty days after block failure as well. This combined analysis using the just-pre-failure-state to evaluate the just-post-failure one developed by the authors resulted in fact in higher requirements on the geogrid than the FEM-analysis in the soil-pre-failure-state (see cases ‘A’, ‘D’ and ‘H’ above), and defined finally the geogrid (Fortræ® 1200/100-10 AM) used in the project.

6 ANALYTICAL CALCULATIONS FOR THE CASES ‘C’ AND ‘B’ (FIGURE 3)

These analyses after a partial collapse of the soil block were not prescribed by any code. They were performed due to common engineering sense and due to the high-safety level of the struc-
ture (what should happen, if...?). They are an additional way to check the remaining CSBL stability after failure of the 'arch', to apply a structural engineering philosophy in geotechnics (bridge analogy) and to reduce the checks to a single comparison of 'action' and 'resistance' in the sense of the new European Design Codes (i.e. in the sense of DIN 1054 1996 and EBGEO 1997 also). The general idea is to handle the remaining part of the CSBL as an arch bridge with the geogrid as a tension member (tie) between the 'springings'.

Thus, the stresses in the 'crest' ('ridge') and the 'springings' have to be checked (Fig. 11). The results regarding $h_y$ (see Section 5) were used herein again, combined with an analysis of the stress field for all cases in the 'crest' and the 'springing'.

In the 'crest' the horizontal normal stress $\sigma_x$ dominates the stress field (but $\sigma_y$ and $\tau_{xy}$ are not zero), and in the 'springing' the vertical normal stress $\sigma_z$ (but $\sigma_x$ and $\tau_{xz}$ are not zero). Because the 'bridge' is made not of concrete but of soil a direct comparison of action vs. resistance e.g. $\sigma_{\text{compression}} \times R_{\text{compression}}$ in the sense of the codes cited is not possible. To make such a single direct comparison possible, the so-called 'q-aux-method' from Alexiuk et al. (1987) was applied reducing the check in both cases ('crest' and 'springing') to the comparison $\sin \psi_{\text{aux}} > \sin \psi_{\text{aux}}$ (Fig. 11 and equations therein), respectively to the requirement for the cement-stabilized soil of CSBL: $\sin \psi_{\text{aux}} \geq \sin \psi_{\text{aux}}$ in all cases. The check was performed for 'crest' and 'springing' for all cases computed, and resulted in higher requirements on the strength parameters $\phi$ and $c$ of the soil then the analysis in case 'A' (Fig. 3). Finally, the 'bridge analogy' analysis defined the required strength parameters of the CSBL for this project.

The final design procedures used the test field experience, but the clear general scheme of checks shown in Figure 3. All detailed pre- and post-failure analyses described were established and performed in the final stage of design, especially the completely new analyses according to 'E', 'B' and 'C' in Figure 3, the time-dependent analysis of geogrid etc. This final design resulted in requirements on the design values of the parameters of soil and geogrid which are in some aspects more rigorous than at the time of the test field.

8 FINAL REMARKS

A complex geotechnical structure for overbridging sinkholes including cement-stabilized soil and geogrids at its base was designed for the German Railroads at Grohers. The bearing system consists of a stabilized soil block and extremely high-strength geogrids.

A clear scheme of checks was established which is conform to the actual German and European codes especially regarding the analysis of the different limit states: the ultimate limit state and the serviceability limit state. All checks and design calculations were performed taking into account such factors as different positions of train loads and sinkhole, time-dependent behavior of geosynthetic reinforcement (and consequently of the entire system) etc.

A commercially available FEM-code was successfully used for many checks required. It was an unavoidable and very useful tool, but FEM-calculations alone were found to be insufficient for this complex problem.

To ensure a sound and really safe design especially for post-failure analysis new analytical procedures or generally new concepts were developed and used: analysis of the critical geogrid tension and deformation after a partial failure of the cemented soil block and analysis of the remaining block after the aforementioned partial failure using an 'arch bridge with tension member'-analogy.

For the serviceability limit state and for the definition of the required design stress-strain (deformation) parameters of soil block and geogrids the FEM-analyses were decisive.

The required short-term and long-term design strength of the geogrid reinforcement was finally evaluated by the post-failure procedure, and the required design soil block strength – by the 'bridge analogy'.

All design calculations and results were approved by the German Railroad Supervising Authority. The resulting requirements on soil and reinforcement became a part of the final pro-
ject specifications. Further information regarding the project can be found in (Ast & Hubal 2001, Leitner et al. 2002).

The construction of the bearing structure discussed herein started in 2001 and was nearly completed until May 2002. Traffic will start in 2002.

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