Railroad embankment with reinforced slopes and base on stone columns

D. Alexiew, H. Pohlmann
Huesker Synthetic GmbH & Co., Gescher, Germany

K. Lieberenz
HTW Dresden, Germany

Keywords: Embankments, Slopes, Railways, Geogrids, Piles

ABSTRACT: As part of the “German Unity Traffic Projects” initiative, Deutsche Bahn AG (German Railways) has arranged to extend the important supra-regional west-east rail link between Uelzen and Stendal. The reconstruction and extension section 51 at “Harper Mühlenbach” is located on an embankment up to 6.5 meters high with a track-level width of over 11 meters, because it is being extended and modified for two tracks suitable for speeds of 160 km/h. Since widening of the embankment base was out of the question for ecological and economic reasons, the consultant planned to install “over-steep” geogrid-reinforced slopes having an inclination of 45°. A horizontal, high-strength geogrid reinforcement on mortar-cemented stone columns was used for the embankment foundation, thereby increasing overall stability and providing a virtually settlement-free foundation for the embankment body. In this way, a combined structure was created, consisting on both sides of geogrid-reinforced earth slopes being in the same time completely set on from-toe-to-toe installed geogrids on “piles”. Such a combined structure was designed and constructed for the first time for the German Railways. Construction was in 1999, the structure is being under traffic since December 1999.

1 INTRODUCTION AND PROBLEM TO HAND

In 1999, as part of the exercise of closing the gap, the Stendal-Uelzen Extension and Reconstruction Section, originally a single-line stretch but with railroad right-of-way for two tracks, was electrified and enhanced to take a design speed of 160 km/h. This railway line was first put into service for single-line operation in 1873 and was subsequently extended to two tracks between 1900 and 1906. In 1945, railway traffic was suspended in the region of the occupation-zone border, then some of the ballast and track was removed in 1951.

To build up the line to v = 160 km/h, the permanent way now needed to be widened from about 10 m to 11.30 m track-level width and masts for the electrification had to be installed.

In the region of the low-lying “Harper Mühlenbach” plain, the line passes through the “Elbufer-Drawehn Nature Park” national park between 71.160 km and 71.600 km, going over an embankment which is up to 6.5 m in height.

There had been no traffic or maintenance work on this embankment for more than forty years. The slopes were greatly overgrown and full of roots. The embankment body comprised insufficiently compacted sand with alternating silty components, and had an underlying, soft layer of decomposed peat up to 1.90 m thick at ground surface level. The ground water sometimes reached up to ground surface level (the toe of the embankment). Because of the low shear strength of the peat layer in particular, the existing embankment slopes having an inclination of 30° to 35° had not sufficient stability.

The following work therefore needed to be done:

- widening of the track level (the crest of the embankment) to produce the required standard profile for a two-track, electrified line and
- increasing the stability of the embankment, its foundation and its slopes taking into account the increased dynamic influence of a train speed of 160 km/h.

In the national park area, the construction method and the technology had to be chosen so as to avoid interference with the natural surroundings (including the toe of the embankment), very limited and no more than temporary demand on space outside the embankment itself and restriction of temporary construction roads to a minimum.

As a result, in the chosen method of construction, the embankment was to be left as it was at the toes of the slopes on both sides (i.e. no base widening), so steeper grassed slopes were to be constructed to enlarge the crest width by the higher slope inclination, and the embankment (respectively its foundation) was to be strengthened from a working level situated on the embankment itself.

2 SUGGESTED SOLUTION

The response to the problem envisaged removing the embankment crest down to at most 3.0 m below the future top edge of sleepers. From this working level the embankment and its foundation were then to be strengthened (“working downwards”) and the embankment crest was to be widened (“working upwards”), so as not to interfere with the natural surroundings at (and outwards) the embankment foot (Fig. 1).

The final solution accepted consists generally of two components:

- an embankment foundation provided by mortar cemented stone columns (extending downwards from the above-mentioned level) and a continuous, horizontal, full-surface (from toe to toe) geosynthetic reinforcement on the tops of the columns and
- widening of the embankment crest by providing steep, grassed slopes in geosynthetic-reinforced soil on both sides from the above-mentioned level upwards.

The first system is principally referred to as a “geosynthetics reinforced soil body on piles/columns” and the second as an “over-steep geosynthetics-reinforced slope”. This construction project was the first one on which Deutsche Bahn (German Railways) used a combination of these two systems.

The two construction methods respectively systems and (additionally) their combination required approval and individual consent (certification) (Federal Railways Office 1998, Federal Railways Office 1999) from the German Federal Railways Office (EBA) - representing the German State in the matter of railways safety - and similar technical declarations/certifications from the DB (German Railways) (DB, GB Network 1997, DB Network 1999) as owner and user of this novel combined structure. Inter alia, both called for particular consideration of the applied dynamic load, technical accompaniment by an expert and monitoring (measurement program) during the first some years in use to ensure long-term serviceability and stability of the structure having a design life of at least 100 years.

In the national park area, the construction method and the technology had to be chosen so as to avoid interference with the natural surroundings (including the toe of the embankment), very limited and no more than temporary demand on space outside the embankment itself and restriction of temporary construction roads to a minimum.

As a result, in the chosen method of construction, the embankment was to be left as it was at the toes of the slopes on both sides (i.e. no base widening), so steeper grassed slopes were to be constructed to enlarge the crest width by the higher slope inclination, and the embankment (respectively its foundation) was to be strengthened from a working level situated on the embankment itself.

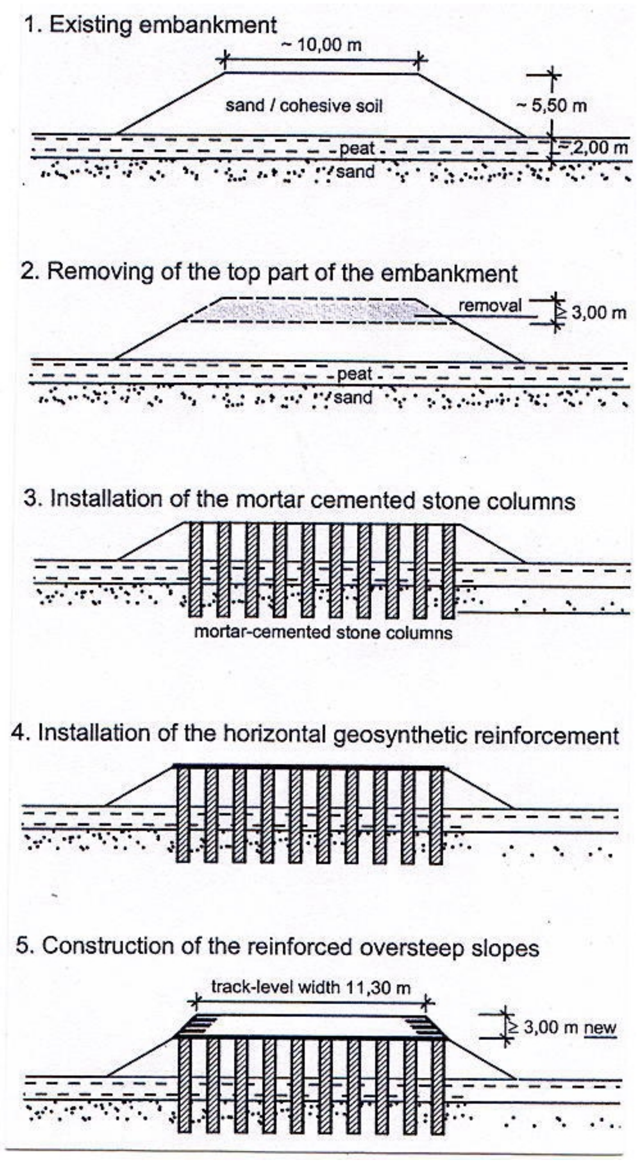

Figure 1. Suggested solution and construction stages

3 DIMENSIONING OF GEOSYNTHETIC REINFORCEMENT ABOVE THE MORTAR-CEMENTED COLUMNS

The suggested use of a horizontally reinforced soil body (embankment) on columns/piles as foundation system was submitted as separate proposal by the contractors as being technically and economically more advantageous than the originally tendered sheet pile walls with interconnected anchoring. The new solution calling for high-strength geogrids was accepted after appropriate design calculations and dimensioning according to the state-of-the-art (carried out by Huesker (Huesker Synthetic GmbH & Co. 1998).

The dimensioning performed according to (Kempfert, H.-G. et al. 1997) with some modifications was based on a 4.50 m high embankment above the columns as relevant cross-section. The principle of the system's working (functioning) and dimensioning is based on the fact that, after redistribution of stress in the point-supported embankment body, a portion of the stresses generated by dead-load and traffic surcharge is transferred directly to the columns/piles, while the remaining portion (which would otherwise inadmissibly overstress the soft subsoil between the columns) is absorbed by the membrane-type supporting effect of the horizontal reinforcement. Under certain circumstances, a portion of the upwardly directed reaction stress of the subsoil between the "piles" can be included in the equation as counter-pressure reducing the tensile forces of the membrane.

The allowed total strain (short-term + creep) of the geosynthetic reinforcement over the 120 year service-life of the structure was limited in this case to a maximum of 3%, which is definitely a restriction on the safer side. For design purposes, a global average value of the undrained shear strength in situ c_u (or s_u) was used for calculation of the upward reaction stress of the soft soil between the columns according to (Kempfert, H.-G. et al. 1997). The spread forces at the edges of the embankment, which have to be neutralized by the reinforcement (no inclined columns for H-forces), were calculated as described in (Kempfert, H.-G. 1997, British Standards Institution 1995). It is a simplified approach on the safe side according to which these forces can be calculated from the active earth pressure, built up between the reinforcement layer and the crest of the embankment, and assigned to the geosynthetic layer perpendicular to embankment axis, resulting in significant tensile forces.

Dimensioning (Huesker Synthetic GmbH & Co. 1998) was carried out with a purpose-developed computer program which also allows optimisation. The design calculations were performed for both the limit state of serviceability (limitation of total strain and of creep-strain of reinforcement) and the ultimate limit state (rupture of reinforcement). Performing the calculations for two limit states was the most important modification of the method in (Kempfert, H.-G. et al. 1997). For estimation of the design strength of reinforcement for the ultimate limit state analysis the procedure according to (DGGT 1997, Forschungsgesellschaft für Straßen- und Verkehrswesen 1994) has been applied. Geometry, loads and soil characteristics had been pre-defined by the tender documents.

After calculating the design tensile forces required for the ultimate and serviceability limit states respectively, the required "as produced" short-term strength (ultimate tensile strength) was back-calculated for both limit states. For the ultimate limit state (rupture) the back-calculation was based on product-specific, approved rupture(strength)-related reduction factors, and on the so called "isochrones" of reinforcement (being product-specific also, and describing the load-time-strain behaviour) for the serviceability limit state (strains). These back-calculations resulted in the following required short-term strengths:

- 400 kN/m transverse to embankment axis (membrane supporting effect + spread forces)
- 150 kN/m along embankment axis (membrane supporting effect)

An additional strength-reduction factor RF_{dyn} in regard of dynamic train loads, which is usually being required by German Railways did not have to be taken into account in this case due to the relatively large installation depth of the geogrids on the cemented columns (mostly > 4.00 m below top edge of sleepers).

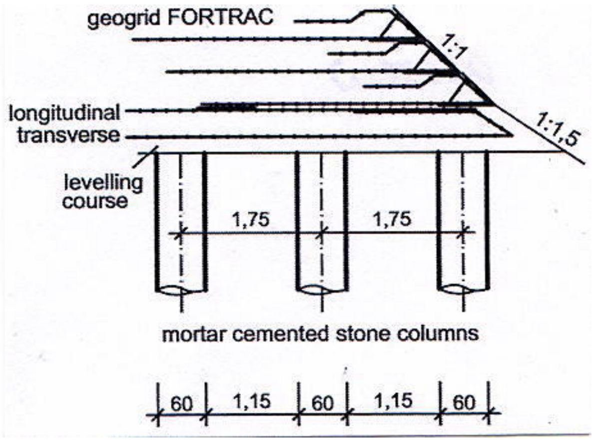

Figure 2 Geogrid reinforcement of embankment on the mortar-cemented stone columns (from toe to toe over the entire embankment base, not shown)

A simplified design drawing of a part of the geogrid-reinforced embankment on cemented columns is shown on Figure 2.

For reasons of time, it was not possible to provide for detailed expert examination and discussion of an alternative dimensioning proposal for the geosynthetic reinforcement on piles/columns put forward by Huesker, which concentrates not (only) on the total strain of the reinforcement, but primarily focuses on the limitation of increase in creep strain in the reinforcement from end of construction stage to end of design life of the structure. This long-term increase in strain was believed to be more critical than the short-term strain and even more than the total strain itself, because deformations due to creep can not be compensated after putting the system in use. This alternative proposal could possibly have allowed a further optimization.

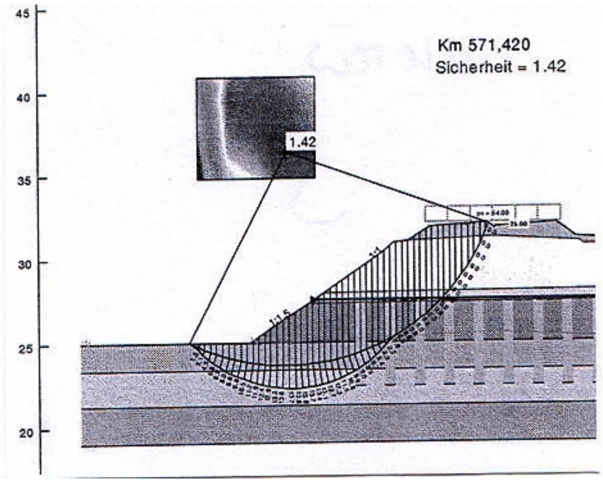

Figure 3. Analysis of global external stability

Global external overall stability was also analyzed, resulting in a sufficient factor of safety (Fig. 3).

An important part in gaining final acceptance for the system played also (Alexiew, D., Gartung, E. 1999).

As already mentioned, this project involved a “combined structure” comprising a pile-based reinforced soil body and over-steep geogrid reinforced slopes. Both systems were dimensioned separately. A “proven” dimensioning method, for instance, as described in (DGGT 1997) or to DIN 4084, was chosen for the over-steep geogrid reinforced slopes.

4 DIMENSIONING OF STEEP GEOGRID REINFORCED SLOPES

According to the concept design for the reconstruction, the embankment crest was to be broaden by means of over-steep geogrid reinforced slopes on both sides. The dynamic (cyclic) stresses produced by train traffic called for extra specifications for the geogrid, because it was to lie within the dynamic field of influence in the slopes. An Individual Permit which was already to hand for the steep slopes reinforced with geogrids (although of a different type according to the primary concept design) therefore contained an extra reduction factor RF_{dyn} for evaluation of design strength.

Another part of the separate proposal of the contractors mentioned above (see chapter 3) involved also replacement of the HPDE geogrids provided for under the Individual Permit with high tensile-strength, flexible geogrids in polyester from the Fortrac® family. Because of this “change in geogrid”, renewed design stability calculations had to be carried out on the steep geogrid-reinforced slope for this part of the separate proposal.

Parameters like geometry, soil characteristics and the short-term strength (ultimate tensile strength) of 55 kN/m and 110 kN/m, the vertical space between layers and the lengths of the geogrids were assumed as in the the existing Individual Permit for the sake of simplification and to speed up procedure. However, independently approved, product-specific reduction factors were used in determining the design tensile strength of the “new” geogrids, and the required reduction factor RF_{dyn} was considered differently according to installation depth (depth below top of sleepers).

The better creep behaviour of the “new” geogrid compared with the “earlier” one in HDPE meant that the stability design calculations could include distinctly higher design tensile strengths for the reinforcing elements. It should be said that only at this stage, the client called for an additional reduction factor which had not been provided to date. This factor is intended (correctly) to allow for weakening of the geogrid by cuts (recesses) for the railway pole foundations.

Following discussions with all parties involved, this reduction factor was set at $RF_{cut} = 1.20$. Additional H-forces produced by the mast piles were ignored because accurate information on same was not available at the time of the new design calculations (HUESKER Synthetic GmbH Co. 1998) and in any case, the “new” geogrid (see above) provided the slope with sufficient resources despite any such recesses.

The stability analysis was carried out on six different embankment cross-sections. In each case, the right and left slopes of the embankment were both examined to DIN 4084 or (DGGT 1997) for slope stability. Design loads were based on the German Railways Standards DS 804 and DS 836. Centrifugal force influences, braking forces and side-impact forces were ignored. Internal and external stability of the steep geogrid-reinforced slope was calculated by Bishop’s method according to DIN 4084 and additionally with polygonal sliding surfaces based on the block-sliding method. The safety factor $\eta \geq 1.40$ required to DIN 4084 (Load case 1, i.e. basic load combination) was verified in all cases (unlike (DGGT 1997), which involves partial factors of safety, an overall DIN-based safety factor was taken into account).

5 STRUCTURAL SYSTEM AND CONSTRUCTION WORK

A special quality-assurance program was developed for execution and supervision of the construction work, stipulating details on stages of work, materials to be used and the necessary qualification, certification and quality control.

The basics of the construction process are shown in Figure 1 and can be described as follows:

- ⇨ Removal of top (old) embankment body and preparation of working level, with temporary storage of removed material outside of the construction site.
- ⇨ Installation of mortar-cemented stone columns within a 1.75 m axial grid (i.e. 10 rows of piles per cross-section).
- ⇨ Verification of load-bearing capacity using a trial load on selected columns and, derived from the foregoing, quality-assurance verification by recording depths, quantities of materials and compaction/activation by the applied electrical energy.
- ⇨ Levelling of height of columns and covering with a levelling course.
- ⇨ Installation of geosynthetic reinforcement over the cemented columns (Fig. 2).
  - levelling course, 20 cm, coarse-grained soil (see above)
  - geogrid with unrolling direction (= main direction of tensile force) transverse to embankment axis: 400 kN/m
  - soil layer, 30 cm, coarse-grained soil
  - geogrid with unrolling direction (= main direction of tensile force) longitudinal to embankment axis: 150 kN/m
  - soil layer, 20 cm, coarse-grained soil; new embankment body from that point upwards, see below.

The bottom geogrid layer was installed continuously without interruption, overlaps or joints, transverse to the embankment axis; side overlaps of adjoining layers amounted to 0.5 m.

The top geogrid layer (longitudinal to embankment axis) was longitudinally overlapped by 3.0 m where necessary; side overlapping of adjoining layers amounting to 0.5 m in this case also.

A load-bearing capacity of $E_{v2} \geq 45$ MN/m^2 according to DIN 18 134 and a compaction degree of $D_{pr} \geq 0.97$ were verified as required for the top test horizon (surface of top 20 cm soil layer, see above).

- ⇨ Construction of embankment with steep geogrid-reinforced slopes:
  The steep slopes were built up in 2 to 7 soil-, respectively. geogrid-layers depending on location to provide the required track-level width of 11.30 m. An inspection path was also provided at the toe of the steep slope. Construction work basically followed the “Textomur®” system (Fig. 4); the Fortrac® 110/30-20 and 55/30-20 geogrids were installed by the wrapped-back method with basic anchor lengths from 4.0 to 2.0 m and wrapped-back lengths from 1.50 m to 2.00 m . Appropriate overlaps and openings (cuts) were provided for the foundations of the catenary masts to be installed at a later date.

The new embankment body was largely rebuilt from the soil of the removed (old) embankment, partially homogenizing the mixture of poorly graded fine to medium sand with silty components and leaving out the softened, cohesive soil which could not be used. An angle of internal friction of 32.5°, a degree of compaction of $D_{pr} \geq 0.97$ and a load-bearing capacity of $E_{vd} \geq 30$ MN/m^2 (dynamic-plate module) were required and duly verified.

The construction stage of the slopes is shown in Figure 5.. Figure 6 provides an overall view of the continuous building operation. In the background, the mortar-cemented stone columns are still being produced, and the beginnings of the over-steep slopes are already to be seen in the foreground.

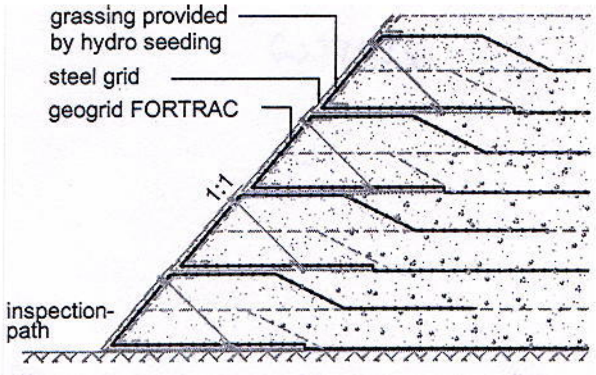

Figure 4. Detail of the geogrid-reinforced over-steep slope with „green“ grassed facing

Figure 5. State of construction work on over-steep slopes

- ⇨ Installation of track-level layer and frost blanket (PSS/FSS):
  A PSS/FSS 0.30 m thick from KG1 was installed (“KG 1” is a defined non-cohesive coarse “grain-size mixture” according to DB AG-TL 918 062) over the entire width of the embankment, levelled and compacted.
- ⇨ Grassing of steep slopes/slopes:
  Grassing was provided by
  - placing fertile soil on the front side of the layer
  - installing a vegetation-bearing fabric as irrigation protector and base for seeding and
  - a hydro seeding with a seed recipe for dry locations.

The initial growth was already relatively dense, and the luxuriant nature of the embankment will allow the locally typical variety of vegetation to develop in a relatively short period of time.

⇨ Track installation:
Just one track was laid in ballast on the track level to begin with.

The earthworks were finished in March/April 1999. The superstructure was then built on top and above this section, the remaining construction phases in keeping with construction logistics. The line was put into service after heavy-load test train runs in December 1999.

Figure 6. View of part of the “Harper Mühlenbach” construction site; different construction stages can be identified

6 FINAL REMARKS

A comprehensive measurement (monitoring) program was installed to confirm calculated stability and long-term serviceability. Measurement programs conducted by DB (German Railways) on geogrid-reinforced systems on piles also contributed decisively to development of the state of the art (see Alexiew, Gartung, 1999) for example).

The following geotechnical and geodesic measurement programme was provided to record the deformation behaviour of the embankment in magnitude and development over time, and to trace the activation of the load-bearing system:

- extensometers in the embankment axis to record settlement and the percentage settlement attributable to subsoil, soft layer and embankment body respectively;
- vertical inclinometers beside the outer rows of columns to record horizontal displacement with depth;

- horizontal inclinometers above the column heads in the geosynthetic reinforced level to record vertical displacement and settlement differences (settlement profile cross the embankment axis);
- horizontal inclinometers under the track-level protective layer to record vertical deformation (settlement/lifting) near the tracks;
- geodesic measurements in 3 measurement profiles to record deformation at the surface of the soil body/superstructure.

A separate report is to be published on the outcome of the programme of measurements as soon as appropriate measurement results are to hand. Initial analyses would indicate that the load-bearing system is activated and working effectively in keeping with the planning and dimensioning concept.

As with all major projects like the one just described, accompanied by a substantial degree of innovation, a number of institutes, firms and individuals were and still are involved in its overall development, not just the authors. While for reasons of space it is impossible to provide an individual mention, we would like to thank everyone for their commitment and application.

REFERENCES

Alexiew, D., Gartung, E. 1999. Geogrid Reinforced Railway Embankment on Piles – Monitoring 1994-1998. *Proceedings Geossinteticos '99. 1st South American Symposium on Geosynthetics*: 403-411.

British Standards Institution (BSI). 1995. BS 8006:1995. *Code of Practice for Strengthened/Reinforced Soils and Other Fills.* London.

DB, GB Network. 1997. NGT 39. *Fachtechnische Stellungnahme zur Ausführung von Steilböschungen bei der Dammertüchtigung Harper Mühlenbach.*

DB Network, NEF 1. 1999. *Unternehmensinterne Genehmigung für die Aufständerung der Dammkrone mit geogitterbewehrten Steilböschungen aus säulengegründeten geogitterbewehrten Erdkörpern, Dammsanierung Harper Mühlenbach with Professional Opinion on User Declaration*, NPZ 25 Vo.

DGGT. 1997. *Empfehlungen für Bewehrungen aus Geokunststoffen – EBGEO.* Berlin: Ernst & Sohn

Federal Railways Office (EBA) 1998. *Bau einer geokunststoffbewehrten Steilböschung. Individual Permit. 21.41 Ibzb*. Bonn.

Federal Railways Office (EBA) 1999. *Zustimmung im Einzelfall für den Bau eines geogitterbewehrten Dammes auf einer Gründung mit säulenartigen Tragelementen und geokunststoffbewehrter Tragschicht. 21 41 Ibzb.* Bonn.

Forschungsgesellschaft für Straßen- und Verkehrswesen. 1994. *Merkblatt für die Anwendung von Geotextilien und Geogittern im Erdbau des Straßenbaus.* Cologne.

Huesker Synthetic GmbH & Co. 1998. *ABS Stendal-Uelzen. Bemessung der Bewehrung über den Fertigmörtelstopfsäulen und der geogitterbewehrten Steilböschungen. Pr. No. 98-316 and 98-219*. Gescher.

Kempfert, H.-G., Stadel, M., Zaeske, D. 1997. Berechnung von geokunststoffbewehrten Tragschichten über Pfahlelementen. *Bautechnik 12/97*: 818-825.