Execution and evaluation of large-scale trafficking trials with geogrid-reinforced base courses

L. Vollmert
BBG Bauberatung Geokunststoffe GmbH &Co. KG, Germany (lvollmert@bbgeo.com)

A. Emersleben & N. Meyer
Clausthal University of Technology, Institut für Geotechnik und Markscheidewesen, Germany (ansgar.emersleben@tu-clausthal.de; norbert.meyer@tu-clausthal.de)

M. Beuße
Ingenieurgesellschaft Dr.-Ing. Michael Beuße mbH, Germany (m.beusse@dr-beusse.de)

ABSTRACT: To improve the fundamental data for the prediction of the performance of geosynthetic reinforced unbound granular layers, two series of full-scale trials have been performed, using a truck representative for an axle load of 10t as it is standard for Europe. The first series was used for a systematic variation on pavement thickness, stiffness-modulus and number of layers for the product range of laid and welded geogrids in combination with a separator. The second series has been used for a comparison of different geosynthetics under comparable field conditions under the guidance and supervision of the Clausthal University of Technology. The comparison clearly shows the improvement of the serviceability of unbound granular layers by geosynthetics. The thickness of the unbound granular layer is of primary importance, as well as the stiffness modulus of the reinforcement. Using two layers of geogrids instead of one, but the same sum of stiffness, gives a significant improvement of performance.

Keywords: base course layer, reinforcement, stabilization, serviceability, trial

1 MOTIVE

The positive influence of geosynthetics on the bearing capacity and serviceability of roads is generally known and documented in different, especially English reports. A significant influence is achieved by avoiding the mixture of different soils, by improving the compactability, by limiting deformations due to membrane effects and by stabilizing the grain structure. Details are given in Berg et al. (2000) and Bräu & Vogt (2011), for instance. For the design of geogrids to stabilize and reinforce unbound construction layers in traffic areas and to increase the serviceability, knowledge about the interaction between geogrids and aggregates is required to be able to give a sufficiently reliable forecast on the behaviour of traffic areas. It is generally known that the tensile strength and elongation of the reinforcement product as well as its interaction with the soil is of fundamental importance for the performance of geosynthetic-reinforced base courses.

Whereas the strength and elongation of products can be measured uniaxially and multiaxially and can be described by linear and bilinear functions, a highly non-linear and complex relation has to be considered for the interaction with the soil. The assumption of linearity with the input parameters friction angle and cohesion, multiplied with an interaction coefficient as it is worldwide common practice for the design of geosynthetic reinforcements, is practicable for analysing the limit state of the bearing capacity and is on the safe side. This cannot be assumed for the forecast of limit states regarding the serviceability at low deformation rates and very low absolute amounts of deformation.

For single products or product groups, as for instance laid and welded geogrids made of monolithic flat bars, Ziegler & Timmers (2004) and Müller (2011) developed mathematical solutions for the interaction. These approaches prove to be practicable for the dimensioning and enable, in future, a calculation depending on stress and deformation, even in case of low deformation rates. Nevertheless, it is and remains almost impossible to describe the behaviour of the 3-phase soil mixture comprehensively and thus the interaction between geogrid and soil. The experimental determination of deformation-dependent parameters for soil-geosynthetic combinations will also be required in future.

Also, the possible bearing mechanisms of a reinforcement are manifold and highly depend on the environmental conditions (e.g. fill material, subsoil strength, type and size of the load, geogrid properties) and up to now a clear analytical description is not possible. Accordingly, there is a high number of different design models (Giroud & Noiray, 1981; Jaecklin & Floss, 1988; Houlsby & Jewell, 1990; Giroud & Han, 2004) considering the known bearing mechanisms and influence parameters in different ways. These models/designs are either exclusively based on model assumptions or have been calibrated on the basis of test results. As the calibration or adaptation of the models is often carried out by single product manufacturers with different boundary conditions, the individual dimensionings resulting thereof often lead to widely varying results.

As comparative examinations with identical boundary conditions are missing, the different dimensioning results are, in addition, only insufficiently verified and are comprehensible to a certain extent only. Though comparative examinations on the effectiveness of different products are existing, they are either limited to model tests, which can display the conditions under real load conditions to a limited extent only, or they have been carried out in the scope of construction projects where often a systematical evaluation is hardly possible due to frequently varying soil conditions within the construction field. Up to now, field tests with controlled subsoil und load conditions have been carried out to a limited extent only (e.g. Perkins, 2002; Cuelho & Perkins, 2009; Cuelho et al., 2014).

In addition, the current tendency to reduce the thickness of the construction layers to a minimum and thus to possibly lower the safety of the constructions, requires practical, systematic tests with corresponding documentation and further scientific protection of the design values as well as definition of limit values.

Therefore, BBG Bauberatung Geokunststoffe GmbH & Co. KG initiated a two-part test series which was finally prepared in collaboration with a subsoil expert/geotechnical engineer, design professional and various scientific institutions and implemented by Ingenieurgesellschaft Dr.-Ing. Michael Beuße mbH, Tostedt. Vollmert et al. (2014) report about the background, objective and conception. The first series includes tests to develop and verify product-specific design charts. For the second test series, the Institute of Geotechnical Engineering and Mine Surveying, Department of Geotechnical Engineering, Clausthal University of Technology, was in charge of the conceptional and scientific support of systematic field tests as well as the evaluation of the field tests. The purpose of these tests (second series) was to investigate, under real installation conditions and load conditions, the effect and influence of different reinforcement products on the bearing capacity and serviceability of unbound base courses over weak subsoil. The client made the following significant requirements on the test field to be prepared:

- preparation of in-situ test tracks to assess the serviceability behaviour of base courses with different reinforcements,
- uniform ground conditions, as far as possible, in the total test field with targeted CBR value of 1.5 %,
- exact preparation of the defined thickness of the crushed stone subbase (base course), using a typical road construction material according to ZTVE-StB 09,
- permanent, consistent load on the total test field by a representative load testing vehicle with axle loads of up to 10 t,
- Driving on the test field exclusively in one lane and one direction to simulate the most unfavorable load condition,
- Continuous, widespread monitoring of the subgrade and surface deformations by using suitable methods of measurement

2 SET-UP AND IMPLEMENTATION OF THE TEST FIELDS TOSTEDT

2.1 *Location and load testing vehicle*

The test fields were prepared on an area of a former sand pit near the city of Tostedt, approx. 40 km southwest of Hamburg (Fig. 1). The site was suitable, in particular due to the homogeneous construction of the subsoil with uniform, up to 5.00 m thick alternating layers of fine and medium sand and a very constant groundwater level approx. 1.00 m below the ground level in the area of the test track.

A total test track length of approx. 120 m was available for the tests to be carried out. The accessible width was 5.00 m. The length of the section allowed the design of 8 different test fields with a length of 15 m each. Between the individual test fields a transition area of altogether 5.00 m (2.50 m at the beginning and at the end of each test field) was provided. This area served as overlapping area for the geosynthetic products installed, as area for sampling as well as for additional, destructive tests and soil tests and, at the same time, it prevented a mutual influence of individual test fields.

The Tostedt test fields were designed for a standard axle of 10 t as usual for European construction vehicles, and accordingly a representative 3-axle truck (axle loads 8 t/10 t/10 t) was used.

The wheelbase and the high total weight of the used vehicle led – compared to vehicles with a standard axle load of only 8 tons (e. g. Cuelho et al., 2009; Cuelho et al., 2014) - with the same number of vehicle passes to a static equivalent load which is 67% higher and thus to a correspondingly higher settlement depression below the load vehicle with corresponding strains in the complete system. Besides the deformations resulting from the static total load, mainly strains occur which result from the punctually moving single load of the wheel during the vehicle crossing. In doing so, multiple and locally - to some extent counter-rotating - rotations of the relevant main stresses which occur in the base course and in the reinforcement level, appear during each vehicle crossing. For the documentation of realistic stress situations vehicle-crossing-field-tests carried out with driven wheels were chosen and are preferred to tests carried out with mechanically controlled wheels (Jenner et al., 2002; Watts et al., 2004). As free-field conditions have been chosen, atmospheric influences have deliberately been included into the test results in order to avoid that – compared to the tests with enclosure (e. g. Zander, 2007) - too idealized conditions are shown. These parameters provide for the aim to use the results above all for the calibration of simplified design models and to evaluate the efficiency of products under realistic conditions.

To limit the test duration, a weak subsoil was chosen, because deformations increase more quickly with a soft subsoil and thus the interaction between single construction and product parameters can already be recognized after a relatively small number of vehicle passes. Thereby, the results can be evaluated for small deformations as well as for large deformations. As far as resilient correlations to the characteristic values of the subsoil arise, the results can be extrapolated to stiffer conditions and a higher number of vehicle passes.

To install a specially selected glacial till as topsoil/subgrade, the subsoil beneath the test sites was excavated and the till, after preparation/treatment and adjustment of the water content, was installed by side installation (Fig. 2), compacted by passes with a crawler excavator and levelled to the specified height. On the surface of the till the geosynthetics were laid in transverse direction to the test fields and covered with crushed stone by side installation. The wrap-around of the reinforcement in the edge region was waived, because a sufficient fixation of the edges for the required anchorage was given with the chosen width of the test track and the expected elongations of the reinforcement (among others according to Cuelho et al. 2009) of maximum 2 % as estimated by Rüegger & Hufenus (2003).

The transition areas were used for sampling of cutter cylinders for lab tests, testing of degree of compaction, dynamic probing, CBR tests and plate-loading tests, each of which is leading to a direct impact on the quality of the formation level / planum.

Figure 1: Site plan of the test track Tostedt I and II

Figure 2: Side installation of the glacial till (in layers)

2.2 *Test series I*[1]

To ensure the comparability and transferability of the findings made in the following investigations, the products used were limited to one product group. The products used were geogrids of the product group Secugrid® Q6 made of polyester (PET) and Secugrid® Q1 made of polypropylene (PP) with an almost identical product geometry.

For all trials in the test fields of series I the design included a separating nonwoven and a nonwoven filter layer, as regularly used in European construction practice (Secutex® 151 GRK 3) and as part of composite products.

Fig. 3 shows the set-up of the test fields of series I in longitudinal section. In addition, the changing test parameters from field test to field test are applied, whereas with regard to the three varying parameters base course thickness (h_0), strain stiffness (J) and number of the reinforcement layers only one parameter at a time was changed. Test field 1.8 did not comply with the design criteria and was prepared with a base course thickness once again reduced by the factor of 2 in order to find out to what extent a reduced base course thickness can be compensated, with a higher strain stiffness and robustness of the reinforcement by a factor of 2. The test fields were arranged in the order of the expected decreasing performance, that means the achievable bearing capacity and the size of the ruts at vehicle passes, so that in case of single field failures the other fields remain as unaffected as possible and accessible.

The thickness of the base course was designed for a target value of the ground bearing capacity of about c_u = 45 kN/m² according to the design method of Jaecklin et al. (1988) and Giroud et al. (1981) for N_{10t} = 1000 axle passages. Due to the only partly predictable performance of the reinforcement products, a uniform layer thickness of 0.50 m was chosen for the Tostedt II series.

Additionally, the base course thickness in field 1.1 was increased to 0.63 m, so that according to the *Bemessungsanleitung für Secugrid und Combigrid Geogitter in Tragschichten* (2003) the intended bearing capacity of $E_{v2} \geq 120$ MN/m² could be expected in the plate-loading test. Thus, the aim was to have conditions on the top edge of the crushed stone base course as they are dominating in classified road construction under bound asphalt pavements. The construction in field 1.1 presents a frost-resistant base course design according to RStO 12 (2012). Despite very weak subsoils with plastic deformation properties, the acceptance criterion $E_{v2,Pl} \geq 45$ MN/m² was waived and two reinforcement layers with a separation and filter layer were arranged to ensure the serviceability and deformation properties.

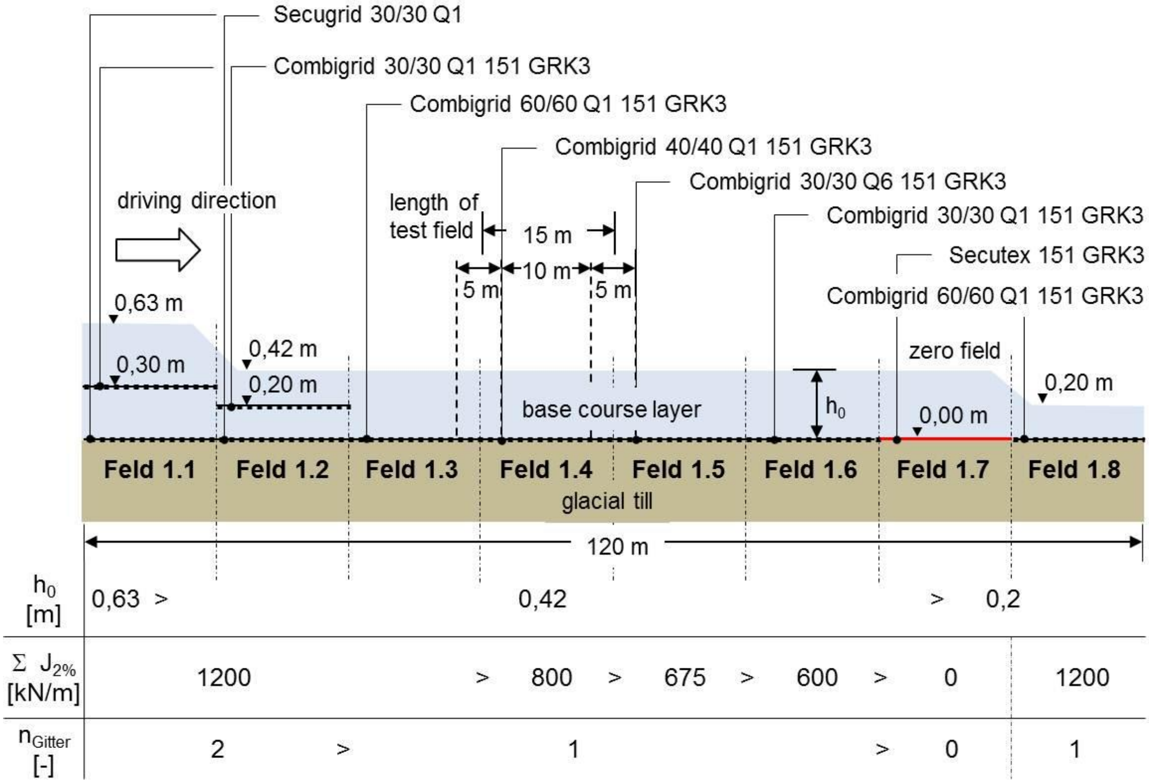

Figure 3: Longitudinal section Tostedt, series I (not to scale), and varying design parameters

2.3 *Series II*[2]

Within the test track, in total 8 different reinforcement elements were investigated and compared with regard to their influence on the load and deformation behaviour of the prepared base course set-up. The investigations included geogrids which are market standard; triaxially and biaxially stretched geogrids, biaxially laid/welded geogrids and knitted geogrids as well as woven fabrics, a combined product from a nonwoven with filaments (knitted) and a mechanically bonded nonwoven. The products have normal strengths of between approx. 20 kN/m and 70 kN/m. Besides the two-dimensional reinforcement products, also a three-dimensional geocell system with a height of 10 cm and a cell diameter of approx. 22 cm in combination with a laid/welded geogrid and a mechanically bonded nonwoven was used. Some characteristic parameters of the geosynthtetics are given in Table 1.

Table 1: Reinforcement products used, test series II

Type	Material[(1)]	Mass [g/m²]	Nominal Strength [kN/m] MD/CMD	Strain @ Nominal Strength [%] MD/CMD
TTT150[(3)]	PP	>200	18,8/17,3	10,3/11,1
TTT170(-G)[(3)]	PP	310	25,9/22,9	9,2/9,9
TSS30(-G)[(4)]	PP	330	30/30	5,8/5,8
HDB15[(9)]	PP	420	≥ 30/≥ 30	≤ 8/≤ 8
NCQ1[(5)]	PP	200	≥ 30/≥ 30	≤ 8/≤ 8
TGPP60[(6)]	PP	240	66/62	9/9
G10[(2)]	HDPE	1235	17,5/-	-/-
HCB25[(7)]	PET	440	≥ 50/≥ 50	≤ 10/≤ 10
NS151/6[(8)]	SP	>150	7,5/11	40/30

(1): PP = Polypropylen, SP = staple fibre, PE-HD, PET = high density Polyethylen(terephthalat); **(2)**: Geocell; **(3)**: triaxial stretched, **(4)**: biaxial stretched; **(5)**: biaxial welded; **(6):** woven; **(7)**: knitted product; **(8):** needlepunched; **(9):** biaxial knitted

All products have a separating component – with the exception of the product TTT150 and the geo-cell which were both installed within the crushed stone subbase. The product NS151/6 consists of a separation layer only and has no reinforcement component. Fig. 4 shows the assignment of the reinforcement elements to the individual test fields as previously specified.
With the exception of field 1 and 7.1, the geosynthetics were installed in one layer, with the separating component down, directly on the surface of the soft/weak layer. In field 1 the reinforcement products were installed in two layers and the base course thickness was increased to 0.65 m. 30 cm above the top of the geogrid type TTT70(-G) which was installed on the weak layer, an additional geogrid of the type TTT150 was placed. In field 7.1 a 10 cm thick geocell layer was installed approx. 35 cm above the reinforcement product type NCQ1 being placed on the soft layer. The geocells were covered with approx. 5 cm base course material up to a total test field thickness of also 0.5 m.

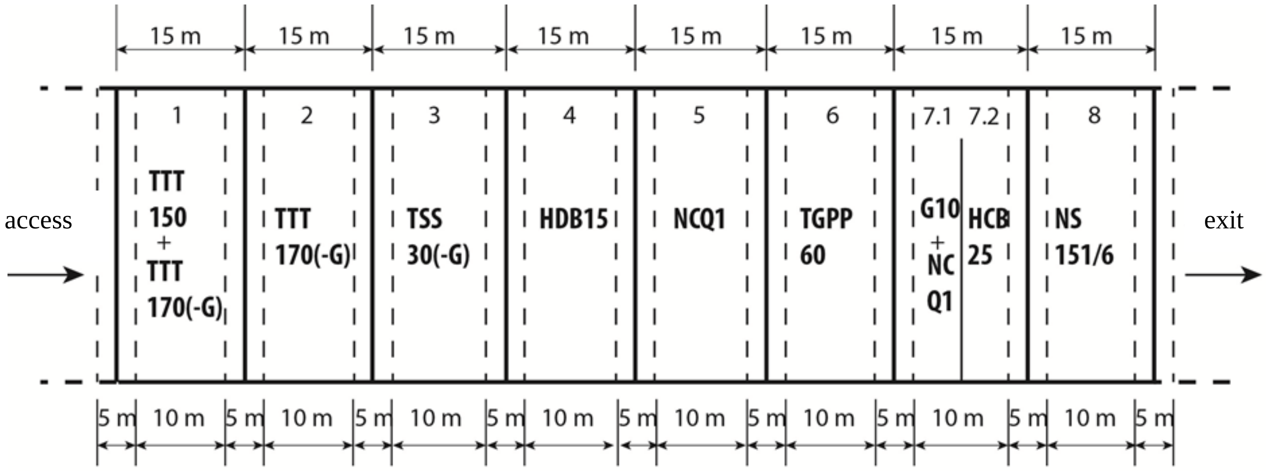

Figure 4: Location of the reinforcement products (schematic)

3 SELECTED RESULTS

The deformations of the test fields were recorded after defined vehicle passes. The evaluation/interpretation of the deformations at the top side of the base course was locally based on those areas for which the ground parameters could be regarded as representative. Under the truck lanes, the deformations at the surface of the base course and at the surface of the soft layer were recorded at the same time (in test series I partially, in test series II completely) and included in the valuation.

3.1 *Test series I*

The installed glacial till was subject to an intensive geotechnical investigation regarding the actual parameters bulk density, water content, undrained shear strength, CBR value etc. – in situ as well as at undisturbed soil samples. Attention was paid to a strong reference to place and time between lab and in-situ results. This enabled correlations between single construction ground parameters, especially between the water content and the CBR value of the subsoil and between the undrained shear strength and the CBR value of the subsoil. In this way, the number of the locally identified CBR values could be intensively completed and consolidated with correlating ground parameters.
The CBR values resulting from the independently determined measuring values such as water content and undrained shear strength show among themselves and also compared to the direct measuring values a very good qualitative as well as quantitative correlation and have locally strong fluctuations. Fig. 5 shows the development of the CBR values over the test field length and the respective average CBR value for the test fields without transition area. The CBR value of the subsoil tends to decrease over the test field's longitudinal axis (x).
Load plate tests were carried out on the test fields which had different degrees of reinforcements and different thicknesses. Most of the test fields, with the exception of the test fields 1.1 and 1.2, revealed very low bearing capacities (Fig. 6). However, relatively good compression ratios were

achieved in all test fields, corresponding well with the measured bulk densities (Fig. 7) prior to the vehicle passes. Consequently, the application of geosynthetics (separation, filtration, reinforcement) obviously allows the proper installation and compaction of the bulk material even on top of extremely weak subsoils.

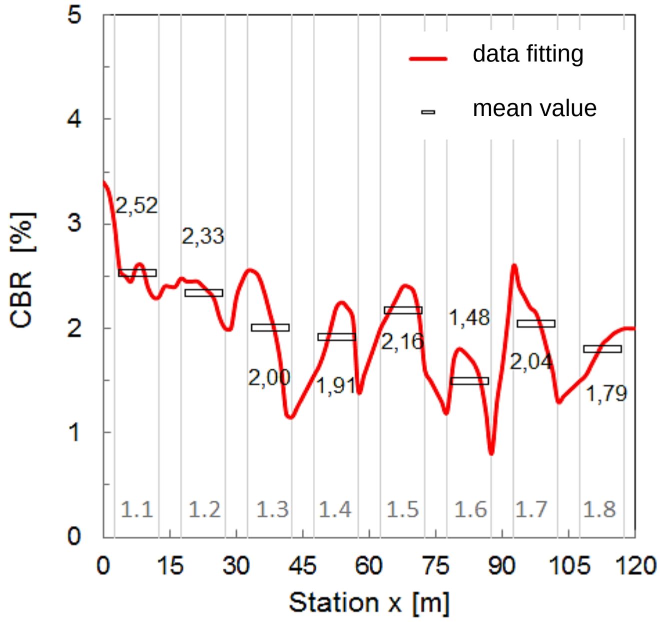

Figure 5: CBR value of the subsoil in series I over the longitudinal axis of the test track (x)

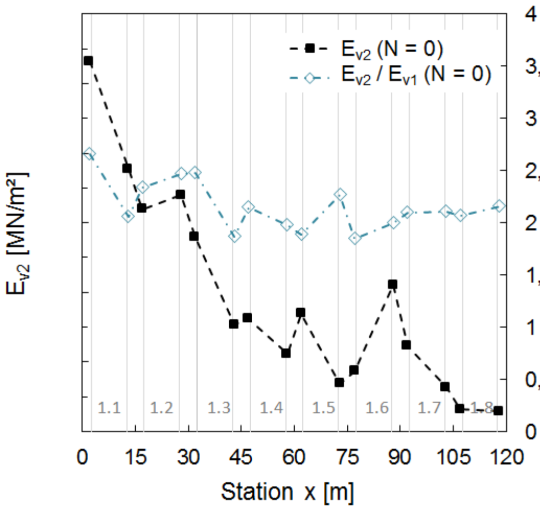

Figure 6: Bearing capacities and compaction ratio, test series I

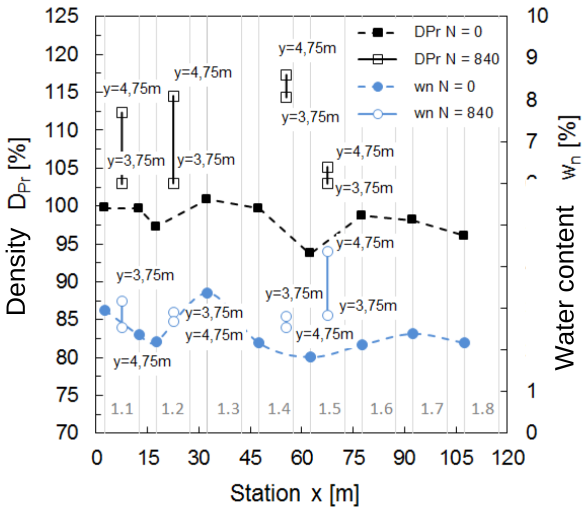

Figure 7: Proctor density and water content of the base course before and after vehicle crossing, test series I

Via the number of vehicle passes the rut areas were recorded in three dimensions with an expensive photo-optical method. The three-dimensional data collection allows the detailed evaluation and follow-up of extreme values and the analysis of deformations at any point within the test track. Thus, the single value of a rut can be exactly assigned to the local subsoil conditions. The deformation differences are determined from the data prior to the vehicle passes and the respective time of the data collection during the vehicle passes. The respective ruts are determined by analysis of maximum and minimum values (Fig. 8) (compare also chapter 3.2, Fig. 12).

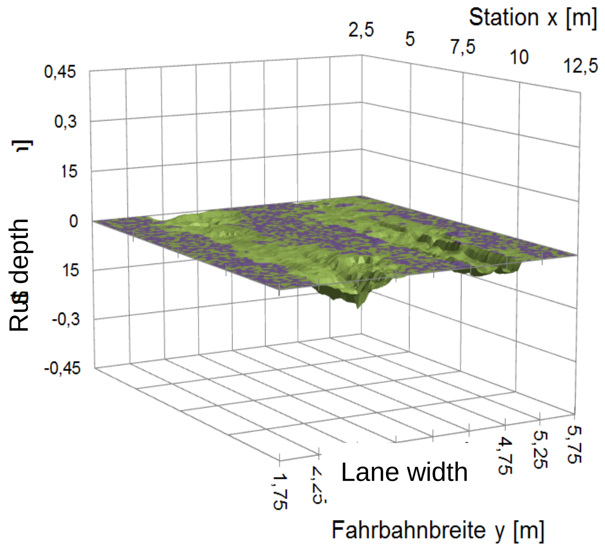

Figure 8: Example of a 3D difference calculation based on photo-optical rut data in enhanced representation

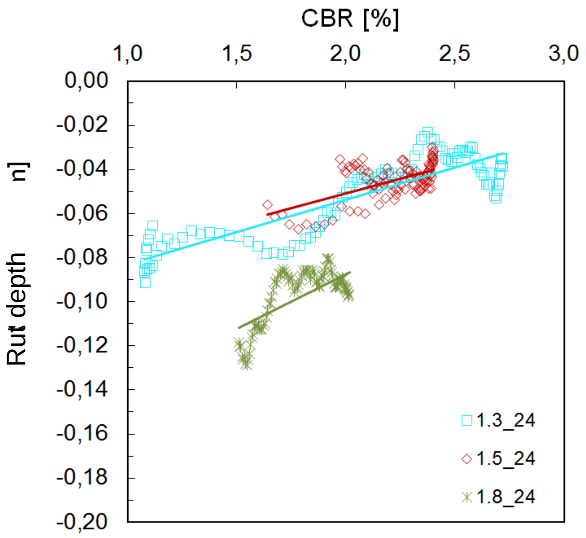

Figure 9: Rut depth (z_N) of single test fields at the same vehicle passes depending on the local CBR value of the subsoil

When applying the rut depth (z_N) on the locally representative CBR value, the rut depth increases with a decreasing CBR value (Fig. 9), as expected. The direct comparison of the balancing functions, in turn, allows a CBR dependent evaluation/assessment of the single test fields. The rut depth (z_N) can be referred to the original thickness (h_0) of the test field and thus results in the relative rut depth (z_n/h_0).

As an example for further evaluations, Fig 10 shows the increase of the relative rut depth depending on the CBR value of the subsoil (a) for an identical total of the installed strain stiffness ($\Sigma J_{2\%}$ = 1200 kN/m) (compare Fig. 3). A differentiation was made between the installation height (h_0) of the considered test fields and the number of geogrid layers (n_{Gitter}) which results in the total strain stiffness. It can be seen that, with a low construction height, the relative rut depth depending on the CBR value of the subsoil (a) increase disproportionately. Thus, with a low construction height a strongly disproportionate rut increase with decreasing subsoil stiffness (smaller CBR) can be expected.

For a bearing capacity of the examined subsoil (CBR value) of 1 % to 3 % CBR a layer thickness < 0.30 m can be referred to as being insufficient. In Fig. 9 it can be seen that the increase of the relative rut depth depending on the CBR value of the subsoil (a) clearly depends on whether the strain stiffness of the reinforcement (in this case $\Sigma J_{2\%}$ = 1200 kN/m) is set up as single-layer system only (n_{Gitter} = 1) or if it is split into two reinforcement layers (n_{Gitter} = 2). With the same CBR value of the subsoil, two reinforcement layers thus significantly reduce the absolute value of the rut as well as the sensitivity with which a structure reacts on variations of the CBR value of the subsoil. Qualitatively, a similar picture is given for parameter b mentioned in Fig. 10.

Fig. 11 shows for the test fields 1.1 to 1.8, with the respective characteristic values of the installed strain stiffness and installation height exemplarily for a CBR value of 1.6 % (c_u ~ 60 kN/m²) the corresponding rut depth for an in each case identical number of load transfers. The rut depth of the non-reinforced test field (field 1.8) is, in this case, around 50 % above that of the reinforced test field with directly comparable installation height.

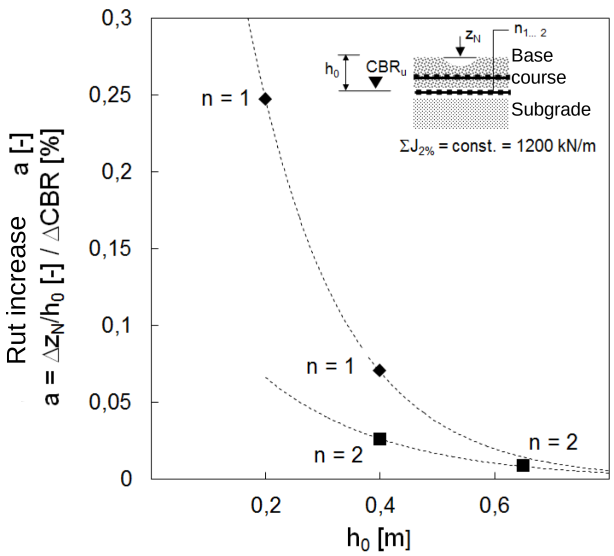

Figure 10: Relative rut increase (a) depending on the installation height (h_0) and the number of the reinforcing layers (n) for the relation z_N [m] = a * CBR [%] – b at a constant sum of the installed strain stiffness $\Sigma J_{2\%}$ = 1200 kN/m

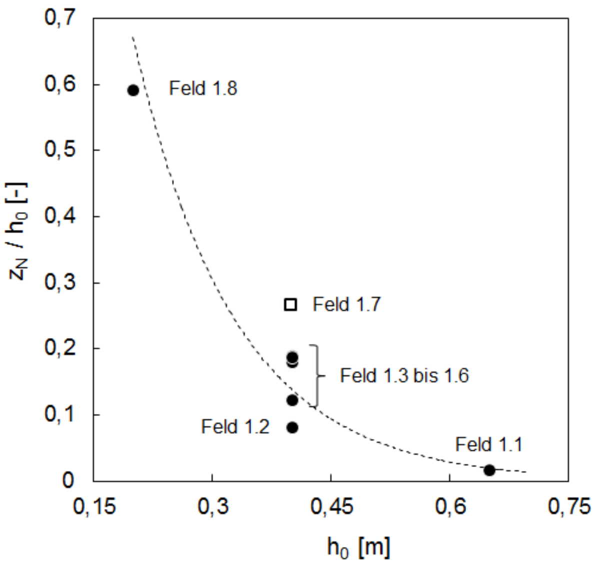

Figure 11: Relative rut z_N / h_0 for CBR 1.6 % and $_{N10\,t}$ = 312

3.2 *Test Series II*

Photographs of the deformations of the test fields were taken after 10, 100, 200, 500, 1000, 1500 and 2950 vehicle passes, (i. e. after each 24.1; 241; 482; 1205; 2410; 3615 and 7110 10 t heavy axle passes). The following evaluation firstly restricts to the left lane of the test track as in test series II - especially in this area - the soil characteristics have continuously been recorded. Thus, the deformations can precisely be assigned to the subsoil characteristics. The deformations in the right lane tend to be smaller than those of the left lane; however, the deformation profile does not differ significantly from the one of the left lane. In all following figures the transition section each with a length of 5.0 m is not shown.
Fig. 12 exemplarily shows the distribution of the surface deformations in the left lane after 10, 100, 500, 1000 and 2950 vehicle passes over the entire test track. It should be noted that the deepest rut depth was defined as maximum deformation.
As well as in test series I the measured maximum deformations in longitudinal direction of test series II show that in most of the test fields local deformation maxima occur which partially significantly differ from the remaining deformation process within the single fields. Typical examples for this are the fields 1, 2, 3, 5, 6, and 7. Here, maxima occur at x = 8 m, 15 m, 28 m, 45 m and 67 m which do not match the remaining deformation process within the single fields and which significantly influence the results. This is particularly applicable to test field 5. All in all considerable differences between the single fields can nevertheless be noted. Thus, especially the test fields 1, 5 and 7.1 show a considerably better deformation behaviour than the test fields 2, 3, 6 and 7.2. The test fields 4 and 8 can only partly be assessed as already after 100 vehicle passes the measured deformations were larger than 100 mm and the fields had to be gravelled again.
Fig. 13 shows the load deformation behaviour of the left lane of all test fields which was derived from the basis of the surface deformations. Thereby, the behaviour was assessed by means of a regression analysis under consideration of the averaged maximum deformations per test field at a defined number of vehicle passes. It should be noted that the partly occurring local deformation maxima in the single fields can to some extent considerably influence the total results.

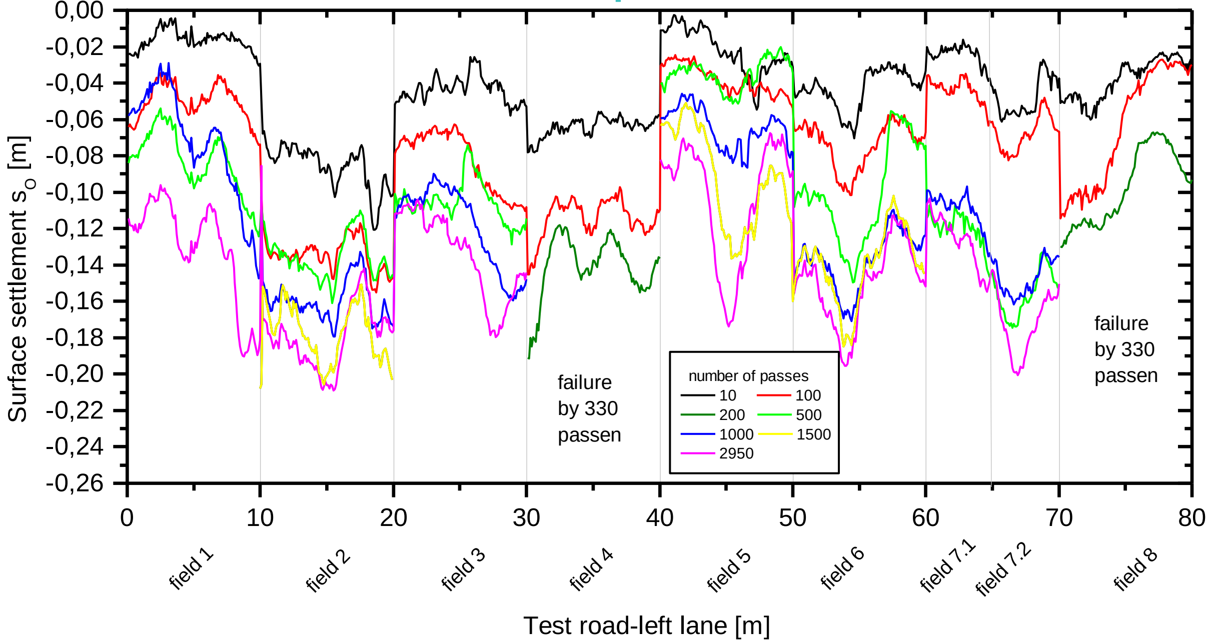

Figure 12: Measured maxima of the surface settlements in the left lane

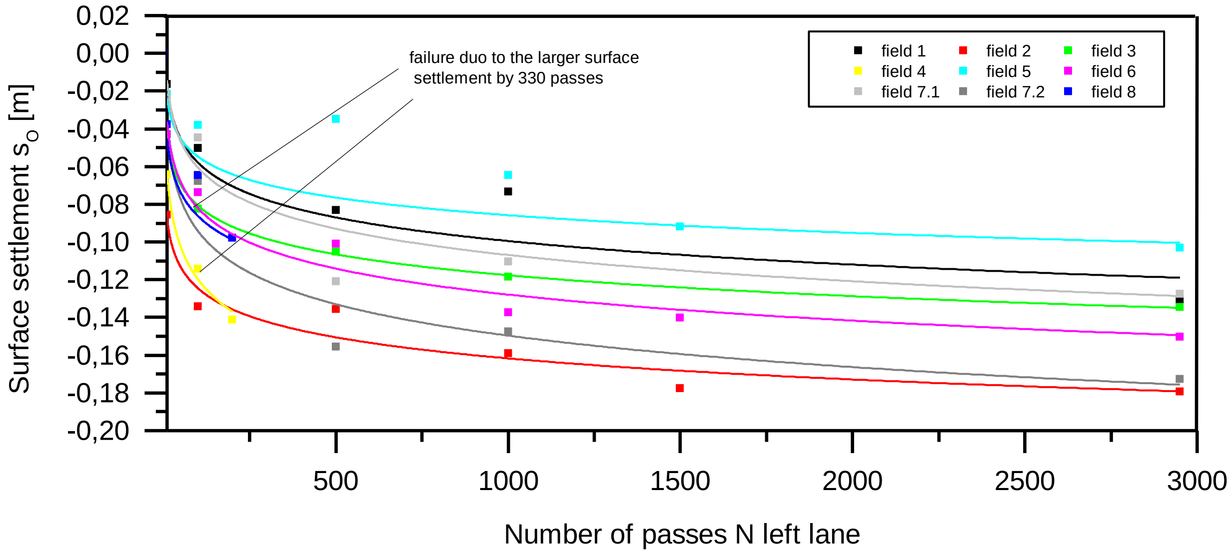

Figure 13: Load deformation behaviour of the base course in the left lane

In analogy to the surface deformations Fig. 14 shows the load deformation behaviour of the subsoil in the left lane of all test fields. It turns out that a better regression analysis is achieved for the surface deformations because of the minor influence caused by the direct truck contact as well as the resulting lane displacements and material entries into the lane.
The deformation assessment of the single test fields however allows an evaluation of the general serviceability, but for the assessment of the single fields among themselves, however, the bearing capacity of the subsoil which is available in the corresponding test fields must be considered.
The assessment of the serviceability under consideration of the available subsoil bearing capacity of the single fields is based on comparative calculations of the determined test results according to the design-method of Giroud et al. (1981). This method allows the determination of the required, unreinforced base course layer thicknesses depending on the number of axle passes (N), the admissible deformations (r) and the available bearing capacity of the subsoil. To adopt the design method to the special parameters of the test field an adjustment factor F(x) was derived for the unreinforced reference field based on comparative calculations and was implemented into the equation.

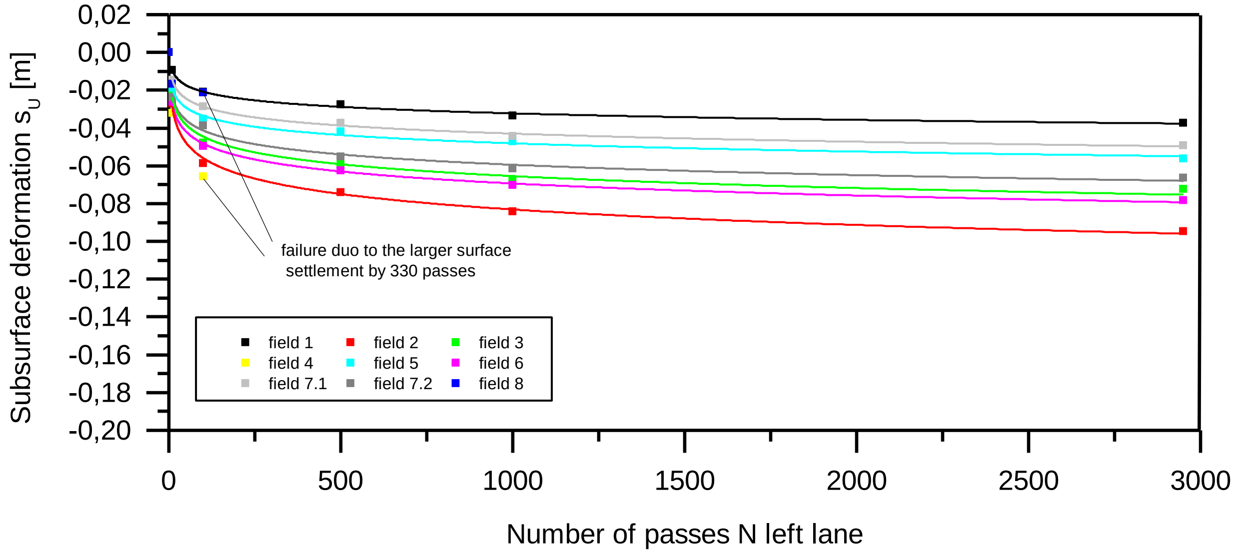

Figure 14: Load deformation behaviour of the soft layer in the left lane

By converting the equation of Giroud et al. (1981) to (N), the maximum number of vehicle passes can - based on the design method adopted to the available test conditions under consideration of the base course layer thickness (h_0), the measured deformation (z_N) and the available bearing capacity of the soft layer - be determined for the respective field in unreinforced condition. Fig. 15 shows a comparison of the calculated possible number of vehicle passes for an admissible rut depth of 50 mm and the passes which have de facto been carried out.
Under consideration of the different bearing capacities of the soft layer the highest improvement of serviceability is achieved - resulting from the installed reinforcements - for an admissible rut depth of 50 mm for field 7.1 by the factor 5.2, for the field 5 by the factor 3.3 and for field 1 by the factor 3.4. For the fields 3 and 6 an improvement of the serviceability by the factor 1.3 can be noted.

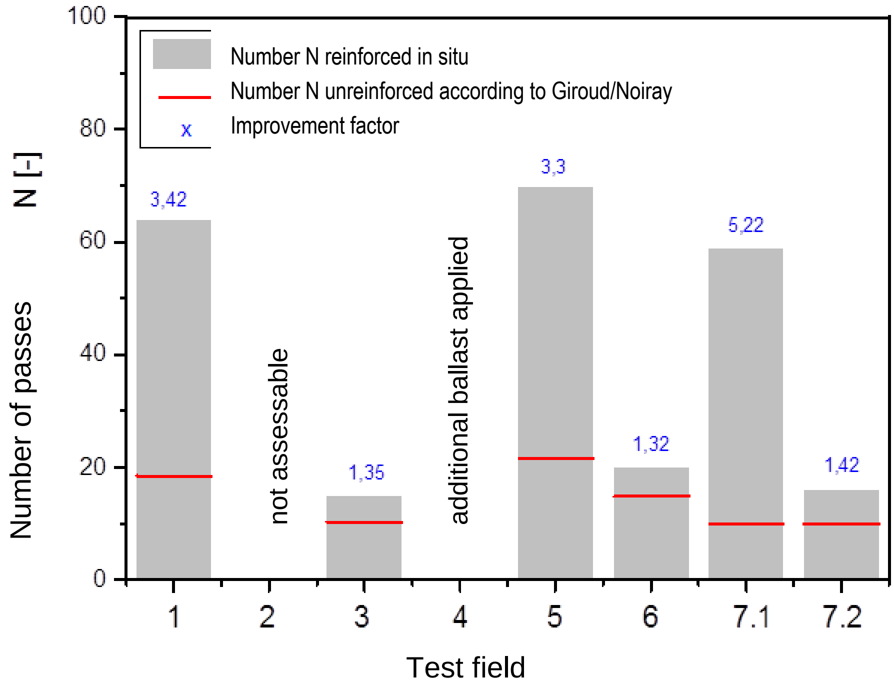

Figure 15: Increase of the serviceability resulting from the reinforcement compared to the calculation-determined number of possible vehicle passes for the unreinforced field at an admissible rut depth of 50 mm

4 SUMMARY

The dimensioning of reinforced base courses often leads to sometimes considerable differences due to the multitude of available design procedures as well the considered different bearing mechanisms of the geogrids. Up to now, comparative studies concerning the serviceability behaviour have mainly been carried out in model tests where the available in-situ parameters can only be shown within a special framework. Comparative studies under identic parameters within the framework of field trials are very limited.

Thus, for the determination of the effects of the base course layer thickness, the number of reinforcement layers and strain stiffness as well as the different reinforcement elements, field trials were carried out in a two-part test series. For this purpose a 120 m long test track was built in a former sandpit. The test track consisted mainly of an approx. 1.00 m thick layer of glacial till with a medium CBR-value of 2.06 % (test series I) and 1.5 % (test series II), respectively, on top of which a 20 cm to 65 cm thick base course layer was installed. The base course layer was systematically built with different reinforcement products and in different reinforcement arrangements. All test fields have a separation and filtration component. Afterwards the test fields are loaded with a 3-axle truck (corresponding to a 2.4 x 10 t standard axle load) having a total weight of 28 tons. During the vehicle passes photos of the ruts which occurred at the surface of the base course as well as of the subsoil deformations were taken at time intervals. The evaluation of the deformations partly reveals considerable differences between the single test fields. Each of the locally measured CBR-values of the subsoil is included into the evaluation.

The evaluation for test series I carried out for one product group (Secugrid / Combigrid) shows the significant effect of the base course layer thickness, the number of layers and the sum of the installed strain stiffness. From the test results clear conclusions can be derived from as to the minimum layer thicknesses and minimum stiffness which are subject to the subsoil and the admissible deformation.

The evaluation of the test fields of the comparative test series II is carried out on the basis of a design procedure for the unreinforced situation considering the variation of the subsoil characteristic values of the glacial till according to Giroud-Noiray (1981) which is adapted to the test parameters. Besides possible savings concerning the base course thickness between approx. 3 cm and 14 cm the evaluations show primarily an improvement of the serviceability. Compared to the unreinforced reference test field an improvement by a factor of 1.3 to 5.2 could be determined for admissible rut depths of 50 mm and 75 mm. Significant differences concerning possible savings for the base course thickness as well as referring to the improvement of the serviceability could be discovered between single test fields.

A continuing evaluation of the test fields can be considered for the calibration of the design procedures in case of small as well as of large deformations. The basic statements concerning the effect of the single parameters and the relationship between single products must be regarded as representative. In this case the determined rut depths correspond to the values of the first vehicle crossing before possible ruts are filled, like it is for example the case in practice, when the bulk goods are delivered. When comparing the here observed absolute values of the deformations with the well-known design approaches (for example Giroud et al., 1981) it must be considered that on the one hand the applied axle load and on the other hand the uniformly distributed load (UDL) of test vehicles deviate upwards. The absolute values of the deformations can deviate depending on the construction process and the load situation and are evaluated for this tested constellation of test series II as maximum values. The investigated constellations with an extremely tracking-stable vehicle crossing without maintenance works, a very continuous vehicle crossing with short intervals and vehicle crossing in only one direction represent a very unfavourable load situation. Continuing evaluations to this are the subject of current investigations (Vollmert, 2016).

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[1]The tests of series I are described and evaluated in detail by Vollmert (2016). This paper represents extracts.

[2]The tests of series II were carried out under the leadership and with scientific monitoring of the Clausthal University of Technology, Institut für Geotechnik und Markscheidewesen. The evaluation and analysis of the data were also provided by the aforementioned institute. Extracts of the results presented by Emersleben et al. (2015) are published here.

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