REINFORCING FOUNDATION LAYERS ON SOFT SUBGRADE

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ABSTRACT: The aim of the research project on the “Design of Geosynthetic Reinforcement” is to develop guidelines for the design of serviceability and ultimate limit states. Field tests on a 1:1 scale concentrated on questions relating to the improvement of the compaction properties and long-term bearing capacity of foundation layers on a soft subgrade reinforced with geosynthetics. Ten different geosynthetics were tested for the reinforcement layer. These included five geogrids, one geocomposite, two nonwoven geotextiles and two slit tape woven geotextiles (one deliberately weak). A site in a brick clay mining pit was available for the test track. The subgrade consisted of a clayey silt with a very low load-bearing capacity. The test track was built with three 0.2 m thick layers. The 1st of these was statically compacted, the 2nd and 3rd were dynamically compacted. Loose recycled material (crushed concrete) was used to build the foundation layers. The state of the track was monitored from installation to removal, focusing on static plate compression tests and profile measurements to assess the formation of ruts. The geogrids were instrumented to measure the strains. The results show that geosynthetic reinforcement only has an effect on a soft subgrade (CBR coefficient < 3 %), which allows the geosynthetic to deform, thus mobilising tensile forces.

1 INTRODUCTION

Reinforcement is used in foundation layers to improve the characteristics of the subgrade with regard to compaction and load-bearing capacity under use (road traffic). Strengthening the foundation layers involves two different processes, which may develop simultaneously (Rüegger and Hufenus, 2003):

- Improving the compaction in foundation layers on soft subgrade, with the aim of achieving the minimum values required for the foundation layer with the lowest possible depth of layer, or with the lowest possible compaction cost. This task relates to a temporary effect during construction.
- Improving the load-bearing capacity of the foundation layer, and thus reducing the necessary depth of the layer (Izvolt et al., 2001) or extending the service life by reducing deformations affecting in service use (Su et al., 2002).

Optimal development of these processes can, either individually or in combination, bring about a significant saving in foundation materials (usually fine gravel).

In order to be able to work out the economic advantage associated with reinforcement, the overall depth of the construction track must not exceed 0.4 m. If the layer depth is > 0.4 m, it is almost impossible to prove the effect of reinforcement (Pospisil and Zednik, 2002).

Field and laboratory trials concentrated on the open question of the extent to which geosynthetics contribute to the improvement of the compaction characteristics and the suitability for use of foundation layers on soft subgrade (Hufenus et al., 2003). The research results serve as the basis for an appropriate standard. The project is still running. The conclusions drawn from the results of the investigations and presented here represent the opinions of the authors alone.

2 FIELD TRIALS

Field trials on a scale of 1:1 were undertaken in the autumn of 2002 in order to ascertain the effect of geosynthetics on the load-bearing capacity of foundation layers on soft subgrade. Compaction and in-service tests were undertaken on the foundation layers on a construction track of modest dimensions, reinforced with various geosynthetics.

2.1 *Selection of geosynthetics*

Table 1 contains a list of the geosynthetics used, with details of the rolling width in each case, and the mesh width of the geogrids. Seven very different reinforcing geosynthetics were used (nos. 02, 27, 28, 32, 42, 44 and 46), with weighting given to the most representative selection possible with regard to raw materials and type of manufacture. A nonwoven separating geotextile (41), which can take moderate weight, and a woven slit tape geotextile (no. 45), which was deliberately too weak, were also included. Nos. 32, 42 and 46 were incorporated with and without an additional nonwoven separator (no. 40), while no. 27 was only included in combination with the nonwoven geotextile.

Table 1 Geosynthetics used in field experiment

No	Field	type of geosynthetic	width [m]	grid [mm]
02	10	PP slit tape woven	5.15	-
27	9	extruded biaxial PP grid in 5 layers	4.50	60 x 60
28	2	PVC coated woven PET grid	5.10	20 x 20
32	5/6	PET flat rib grid	4.75	32 x 32
40	V1	PP nonwoven (separation)	5.00	-
41	12	PP nonwoven (reinforcement)	5.00	-
42	3/4	PVC coated woven PVA grid	5.20	40 x 40
44	11	PET yarn reinforced PP nonwoven	5.20	8.5 x 8.5
45	1	PP slit tape woven	5.15	-
46	7/8	extruded biaxial PP grid	3.80	65 x 65

As a general principle, those products that are conventionally used to reinforce foundation layers were the ones used here. These are products that are effective biaxially, and are able to withstand approximately the same force in both directions. The Young's modulus (Bloise and Ucciardo, 2000) is particularly important at an elasticity of < 3 % (Bergado et al., 1998). Table 2 contains details about the strength of the various products, mobilised at different values of axial strain (manufacturer's data).

Table 2 Tensile strength of the products in machine (MD) and cross direction (XD)

No	Field	tensile strength per metre width [kN/m]					
		at 2 % strain		at 5 % strain		max.	
		MD	XD	MD	XD	MD	XD
02	10	12	12	30	30	65	65
27	9	6	10	14	20	22	35
28	2	9	9	14	14	55	55
32	5/6	10	10	20	20	30	30
40	V1	0.2	0.1	0.3	0.2	10	10
41	12	0.4	0.3	0.6	0.4	20	20
42	3/4	12	12	32	32	40	40
44	11	7.5	7.5	22	22	50	50
45	1	2	2	8	8	30	30
46	7/8	11	12	22	25	30	30

2.2 *Test track*

An area within a brickworks clay pit was available for use as a test track. The subgrade was a clayey silt. The test track was constructed adjacent to an existing road, with a length of about 130 m (Figure 1). This allowed for installation from the side, so that the test track was not previously subject to traffic, nor put under strain by installation equipment prior to compaction. The track was located along the outer north east edge of the pit. Water streamed over and out of the embankment during and immediately following heavy rainfall, and this was held back by the track.

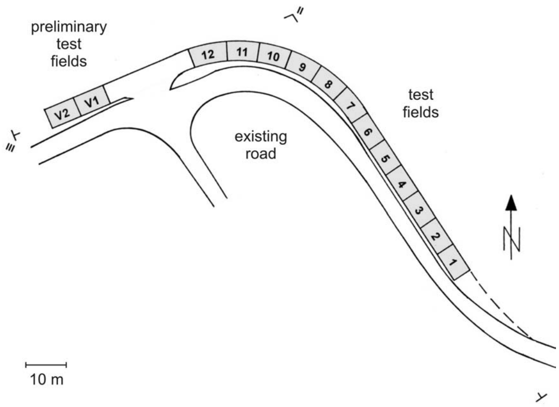

Figure 1 Test track with divisions between test fields

The cost of the test run was relatively high, and errors in installation had to be avoided as much as possible. It was therefore important to know in advance what intensities would generally be permitted with regard to compaction and traffic loading. No geosynthetic material (Zone V2) or only a separation geosynthetic (Zone V1) was therefore included at the end of the run (Figure 1). This enabled the installation, compaction and traffic characteristics to be tested in a section that demonstrated equal or worse conditions compared with the track (no inherent reinforcing).

A similar research project (Schad, 2001) failed, partly because factors that exerted an influence, such as weather conditions and trafficking during construction, caused major variations in the research results, and these prevented any reasonable assessment from being made.

2.3 *Ground parameters*

Loose recycled rubble was used to form the foundation layer. The material was broken down to a maximum grain size of 64 mm, and the fine portion (with a diameter of < 8 mm) was sieved out. The particle size for layers 1 and 2 then lay between approx. 8 - 64 mm (Figure 2). Because there was a limit to the proportion of small particles, the material demonstrated low sensitivity to changes in the water content, and was sufficiently porous so that meteorological and percolating water was conducted quickly into the lateral drainage ditches.

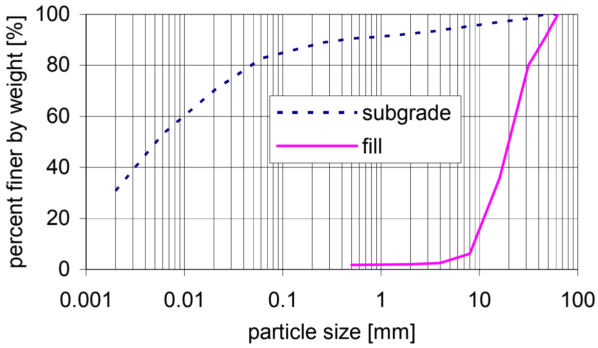

Figure 2 Particle size distribution in the subgrade and fill

In contrast, finer grained recycled material, with a particle size of 0 - 32 mm was used for the third layer, in order to achieve an improvement in density and hence interlocking, so that less particle movement within the material itself occurs when the ruts are driven over during trafficking.

The subgrade was classifiable as CM (medium plasticity silty clay). The distribution of particle sizes can be seen in Figure 2. The undrained shear resistance c_u of the subgrade was measured directly in the field using a Pilcon shear vane. A penetrometer is used to determine the CBR coefficients at depths of approx. 0.2, 0.4 and 0.6 m (as for the shear vane). Figure 3 contains the CBR and c_u values measured in test Fields 1 - V2 (average of the values gained at depths of 0.2, 0.4 and 0.6 m).

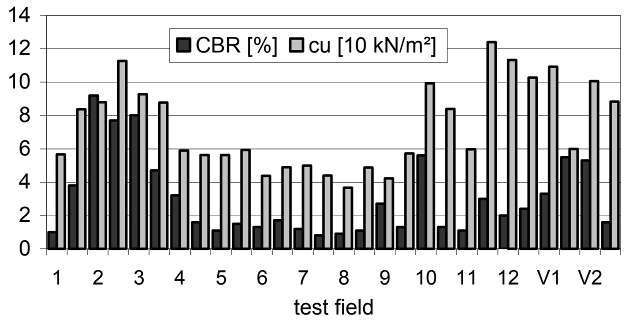

Figure 3 CBR and c_u values for the subgrade

2.4 *Equipping and instrumenting the test fields*

The distribution of the geosynthetic samples to the fields of the actual test track (0 - 96 m) is shown in Figure 4. This shows the set-up with the distribution of the geosynthetic sample (grey: grid underlaid with nonwoven separator) and the orientation of the geosynthetic material (arrow = direction of production), as well as the position of the strain gauges and the profile used to measure the ruts (broken line). Only one reinforcing layer was anticipated for the subgrade in each case.

The geosynthetics were equipped with strain gauge instrumentation in order to detect the local short-term and long-term deformation.

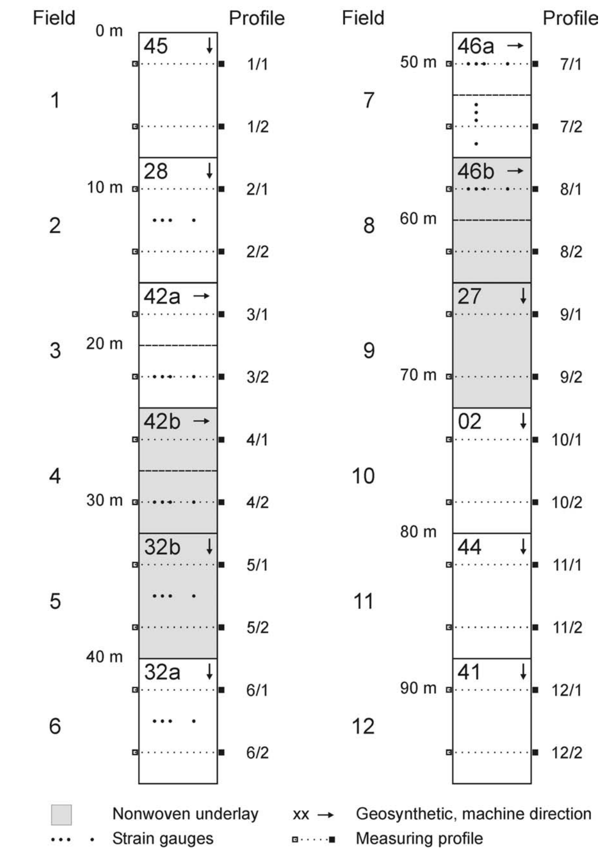

Figure 4 Test track set-up

The selection of the sample subjected to instrumentation was made according to technical practicability (the potential for attaching the strain gauges without making any relevant alteration to the force-deformation characteristics of the test object concerned). 4 strain gauges were fitted in a line at right angles to the track axis for each sample under test. Figure 5 shows their positions. 4 additional strain gauges were fitted in the direction of travel in Field 7.

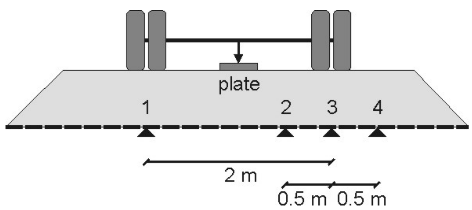

Figure 5 Positioning of the transverse strain gauges

2.5 *Setting up the test track*

The plan for the test track was for 3 layers of 0.2 m thick fill (Figure 6), with the 1st layer compacted statically and the 2nd and 3rd layers compacted dynamically.

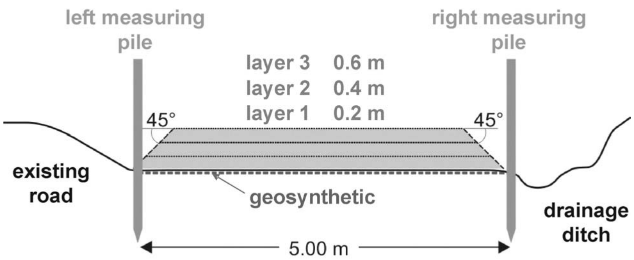

Figure 6 Cross section of the test track

The test track was created and removed as follows:

- Local widening, side ditches and through-cuts for water drainage were constructed, followed by irrigation of dry sections of track, adjustment of subgrade with sand, and static leveling by roller.
- Geosynthetics and cabling for the strain gauges were laid.
- Construction track and test track was laid with 0.25 m loose ballast. The construction and test track were compacted purely statically with a 2.5 t tandem flat roller (Bomag BW 120), 3 - 4 passes (compacted depth 0.2 m).
- Trafficking test over 1st layer with 13 t truck, 2 times in each case for plate load tests, 6 further passes.
- 2nd layer laid (a further approx. 0.25 m ballast): dynamic compaction, roller with constant energy (Bomag Variocontrol BW 177 with a weight of 8.0 t), 3 - 4 passes (compacted depth 0.2 m), computer registration of the response.
- Trafficking test over 2nd layer with loaded truck (10 driving passes with 22 t, 10 passes with 28 t).
- 3rd layer laid (recycled material 0 - 32 mm), 3rd layer compacted in same way as for 2nd layer.
- Trafficking test over 3rd layer with loaded truck (10 and then 50 driving passes with 28 t).
- Ballast cleared to approx. 0.05 m over the subgrade (geosynthetics) with a hydraulic excavator for all profiles, final excavation by hand shovel.

The condition of the track and the geosynthetics was monitored by instruments from installation to removal, using CBR measurements (CBR penetrometer), shear vane measurements (Pilcon), static and dynamic plate load tests, specific gravity measurements, a dynamic falling weight deflectometer (FWD), the overall dynamic compaction control and the profile measurements (ruts) and strain gauges on the geogrids.

3 COMPACTION CONTROLS

Control of the compaction was carried out in the field, using the following procedure:

- Measurement of the dry/wet density using radioactive isotopes (Troxler apparatus)
- Static plate load test
- Dynamic plate load test
- Overall dynamic compaction control
- Dynamic falling weight deflectometer (FWD)

3.1 *Static plate load test*

Plate load tests using a one-hour plate load device (D = 300 mm) were carried out on the 1st and 2nd recycled layer to measure the deformability and load-bearing capacity of the foundation layer.

Two to three static plate load tests in each case were undertaken on the 1st recycled layer (d = 0.2 m) in Fields 1 - 12 and in V1 and V2. Figure 7 shows the corresponding results, where E_{V1} and E_{V2} are the Young's moduli of the 1st loading and reloading cycle respectively (cf. Figures 8 and 9).

Because the subgrade was so soft, it was rarely possible to achieve the maximum potential required to meet the standards with an initial load of 0.5 MN/m^2. Consolidation during the test also made it impossible to expect a settlement change of < 0.02 mm/min in accordance with the standards. Nevertheless, the results can be used for purposes of comparison between the fields, since all tests were carried out under the same conditions.

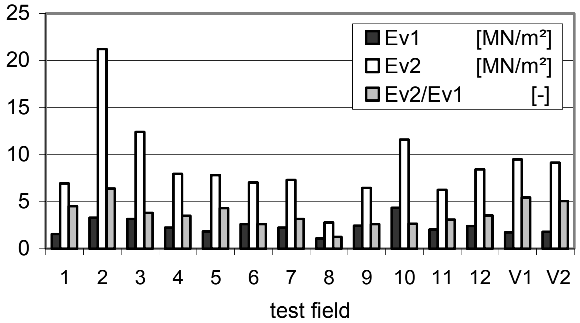

Figure 7 Mean E_{V1} and E_{V2} values on the 1st recycled layer

The tests showed that the influence of the reinforcement on the compression moduli measured by means of plate load tests (initial and repeated loading) is marginal, since the deformations are too small to mobilize forces with significant vertical components.

An exception to this is the measurement on the very thin foundation layers of approx. 0.2 m, when the pressure forces under the plates still clearly reach through to the subgrade. The plate load test then becomes a load-bearing test as shown by the examples in Figure 8 (without reinforcement, with non-woven separator only, No. 40) and Figure 9 (with reinforcing grid, No. 32), whereby the former case gives a settlement of 55 mm and the latter 18 mm for a 1st loading cycle to 350 kN/m². This was the maximum value for Field V2, whereas a vertical stress of significantly more than 500 kN/m² is achieved for the field with the geogrid reinforcement.

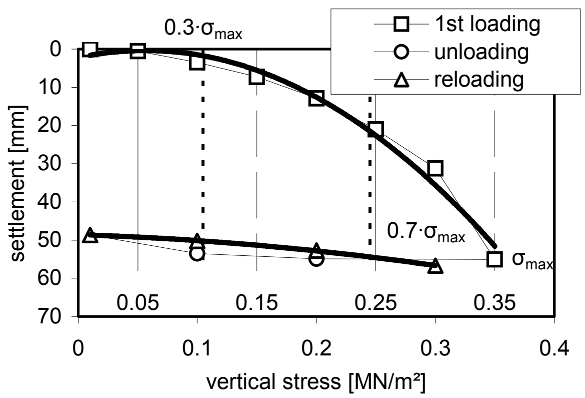

Figure 8 Vertical stress - settlement diagram for Field V2

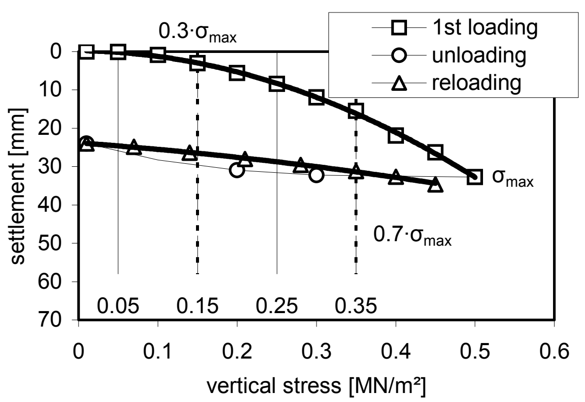

Figure 9 Vertical stress - settlement diagram for Field 6

3.2 *Overall dynamic compaction control*

Overall dynamic compaction control is a technique used to measure the load-bearing capacity of compacted ground with the help of the movement behaviour of the dynamically excited roller. By measuring and analyzing the acceleration within the vibrating roller, conclusions can be drawn about the dynamic stiffness E_{vib} and/or the degree of compaction of the areas rolled over. The depth of measurement for this is greater than the depth of compaction.

In Figure 10, the roller measurement values E_{vib} deduced from compaction of the 2nd recycled layer (d = 0.4 m) are compared with the E_{V2} values (static plate load test) obtained subsequently.

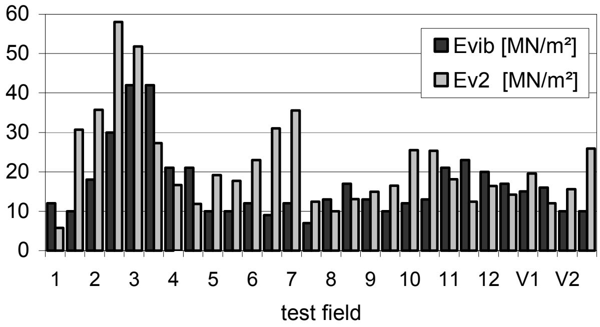

Figure 10 E_{vib} and E_{V2} values for the 2nd foundation layer

In the dynamic compaction of deeper layers (over 0.4 m depth) with rollers weighing in the region of 50 - 80 kN, signs of differences can only be detected in the achievable compaction values (plate load tests, dynamic compaction control using a roller). The compaction energy is not fully effective on a soft subgrade, because this will dissipate in spite of the reinforcement, since the substrata offers too little resistance.

3.3 *Effect of reinforcement on compaction*

The measurement results are highly dependent on the local subgrade conditions (CBR, c_u), which means that appropriate calibration will have to be carried out for evaluation to take place. It appears that given comparable subgrade conditions and an equal depth of layer, with reinforcement, the degree of compaction (measured by in situ density) is approx. 10 - 30 % more than without any reinforcement. Open, rigid (rigid node) geogrids also have a somewhat enhanced effect compared with closed or open webs that can not be smoothly laid, and which naturally require initial deformation before they can become taut and withstand tension. This agrees with the observations made by Kenny (1998).

4 RUT FORMATION AND DEFORMATION

4.1 *Profile measurement*

The profile measurement used to assess the formation of ruts is carried out using a cross bar specially developed for this field test. This cross bar rests on the left and right measuring piles driven in on either side of the track, and the distance of the piles from the track is given by measuring sticks (Figure 11). A selection of measurement results is shown in Figures 12 (Field 6) and 13 (Field 1), where the thickness of the layers is measured in relation to the initial level of the subgrade (= 0 mm).

Figure 11 Measuring out the ruts

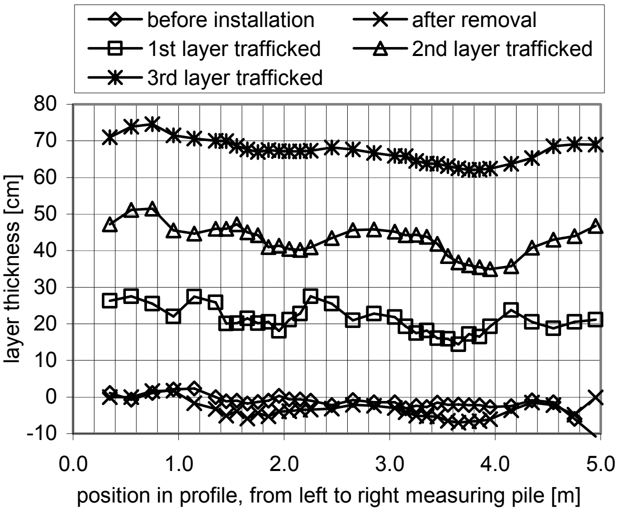

Figure 12 Development of ruts in Field 6 (Sample 32)

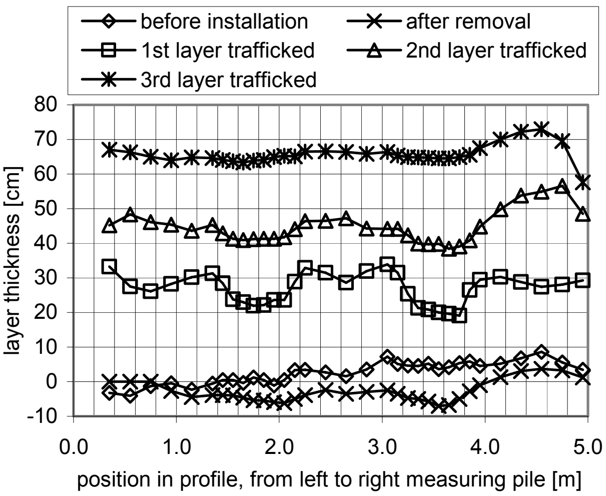

Figure 13 Development of ruts in Field 1 (Sample 45)

With reinforcing geosynthetics able to withstand a force per metre width of > 8 kN/m at 2 % axial strain, ruts form at a depth of between 20 and 40 mm in very soft subgrade (CBR < 1.5 %). For a more flexible geosynthetic (sample no. 45, approx. 2 kN/m at 2 %), ruts of up to just less than 100 mm develop in an equivalent subgrade, whereas the depth of rutting in the foundation layer (0.2 m deep) reached up to 150 mm. A strain of approx. 5 % in the geosynthetic can be calculated as a result. The mobilised force per metre width then stands at about 8 kN/m.

4.2 *Strain measurements*

Figure 14 shows the results of dynamic (continuous) strain measurements taken during compaction and driving passes over the 3 layers (Field 8). The measurements taken during the time the track is being trafficked demonstrate relatively large strain maxima of more than 1% (1st layer trafficked), under the direct load influence. Nevertheless, the residual deformations still stay modest, as shown in Figure 14.

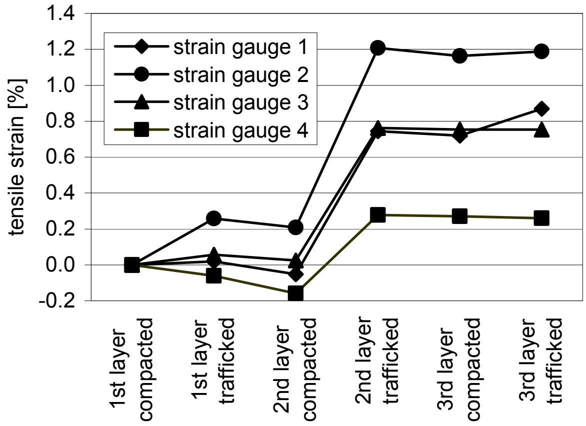

Figure 14 Reaction of the strain gauges to the various loads

The greatest degree of plastic deformation of the subgrade and the equivalent prestressing of the geosynthetic was achieved when driving over the 2nd layer (0.4 m). It seems that residual ruts were formed in the subgrade under this load and lateral anchoring of the geosynthetics was also provided by the covering of 0.4 m compacted fill. In the 3rd layer, (0.6 m) there were only minor additional deformation maxima beyond the existing residual deformations recorded in the reinforcement, and these did not increase any further.

Figure 15 contains static, point-by-point strain measurements taken during a plate load test (reaction of the 4 strain gauges in Field 8). The plate load test on the 1st layer (0.2 m) over strain gauge 2 (figure 5) generated similar deformations in the reinforcing geosynthetic as for driving upon it (highest values beneath the direct load). When the loading was removed, the deformations receded and the permanent prestressing remained comparably minor, in the same way as when a truck drove over the 1st layer (0.2 m).

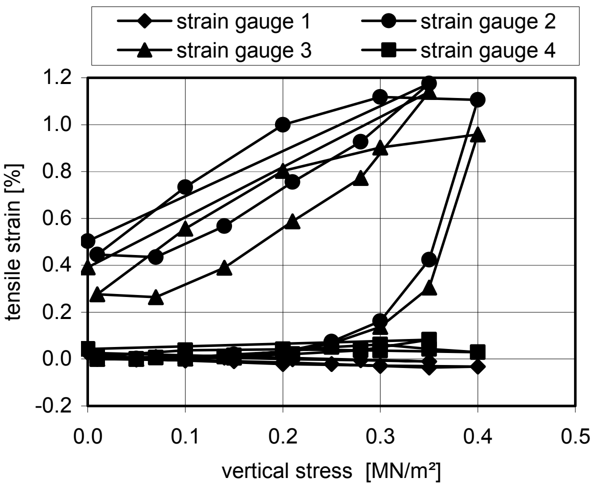

Figure 15 Reaction of the strain gauges to the plate load test

From the 2nd layer onwards (> 0.4 m), very little extra deformation was generated in the geosynthetic by the plate load test, since the effective depth of the 0.3 m diameter plate load was too small.

To summarize, the results of the deformation measurements lead to the following conclusions:

- Both compaction in thin layers and driving over thin layers (< 0.4 m) generates strains (and therefore tensile forces) in the longitudinal and transverse directions in reinforcing geosynthetics on very soft subgrade.
- Compaction seldom leads to development of residual strains. Once the roller has passed onwards, short-term strains recede.
- Trafficking the field with a loaded truck results in the triggering of comparable strains and/or forces. The development of residual ruts in the subgrade then results in residual geosynthetic strains, with the corresponding prestressing in the transverse direction within the geosynthetic.
- The formation of ruts in the subgrade is a precondition for residual prestressing. Such ruts can only develop on very soft subgrade and beneath thin layers (< 0.5 m).

These comments apply to the geogrids with strain gauges, since other products could not be instrumented for technical reasons, and relevant measurements are therefore unavailable. However, since the other tests (compaction measurements) did not show clear, product-specific differences, the statements made may apply generally, as long as products with comparable stiffness (mobilised force per metre width at 2 % strain) are used and compared.

The relatively good agreement of the deformation calculated from the formation of the ruts in the subgrade (measured after the track was reinstated) with the residual strains measured by the strain gauges in the geosynthetic at the same location after the test was completed was of interest.

In order that the surface ruts can be limited to a maximum of 70 - 100 mm in very soft or soft subgrade on thin foundation layers (depths 0.2 - 0.3 m), the ruts formed in the subgrade must be limited to a maximum of 30 - 50 mm. The strains calculated along the ruts beyond the extended formation are then 0.5 - 1 % and roughly correspond to the residual strains in the geosynthetics. Greater strains of up to about 2 %, with the corresponding mobilised force, result when the geosynthetic is under the direct load (when the roller or the truck is passing).

Such strains correspond in conventional reinforcing geosynthetics (stiffness 400 - 600 kN/m) to a force mobilization of 8 - 12 kN/m. For thin layers (low load), the forces mobilised, calculated from the geosynthetic strain and its stiffness, equate in practical terms to the maximum possible lateral anchoring forces, as stated by Rüegger and Hufenus (2003).

5 CONCLUSIONS

Geosynthetic reinforcement only has an effect on a soft subgrade (CBR coefficient < 3 %), which allows the geosynthetic to deform, thus mobilising tensile forces. Forces of up to around 8 - 12 kN/m may be mobilized in the geosynthetic. Higher traction forces might have no effect, due to the limited lateral anchoring forces.

Residual strains in the geosynthetic remain below 2 % under static loads, but may reach somewhat higher levels with dynamic loads. Because all the geosynthetics exhibit comparable load-strain behaviour (stiffness modulus), the type under consideration is somewhat irrelevant, regarding reinforcement on soft subgrade.

The formation of ruts is reduced with a depth of layer of up to 0.4 m. With thicker layers, the reinforcement no longer has any significant effect. If geosynthetics able to mobilise tensile forces of at least 8 kN/m at 2 % deformation are used, an improvement of approx. CBR = 1 % or to c_u = 25 kN/m^2 can be expected in the subgrade.

The profile of the foundation layer (thickness) can be measured for these improved subgrade characteristics in respect of both the compaction and the ability to bear driving loads (rut formation). This leads to a considerable reduction compared with the depth of layer that would be required without reinforcement, and leads to significant savings in materials.

The results of the tests do not enable any comments to be made about the positive effect of reinforcement on the ultimate behaviour of a road with an asphalt surface.

6 ACKNOWLEDGEMENTS

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