THE PERFORMANCE OF REINFORCED UNPAVED SUB-BASES SUBJECTED TO TRAFFICKING

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ABSTRACT: Geosynthetics are used in pavement construction to reinforce the sub-base layer, when the underlying material is weak, or when the thickness of the pavement needs to be kept to a minimum. However, the effectiveness of this technique is difficult to quantify and is best investigated by undertaking full scale Trials.
Trafficking Trials have been carried out in the Pavement Test Facility at the Transport Research Laboratory (UK) to assess the ability of a range of different geosynthetic reinforcements to reduce the surface deformation of an unpaved road. Twelve different reinforcements were included in the Trials; these included integral grids (punched and stretched or, extruded grids, woven grids, bonded junction grids, composite grids and woven geotextiles. The performance of the reinforcements was quantified in terms of pavement stiffness, and surface deformation of the sub-base.
This paper present details of the test facility, methodology and the principle results. The relations between deformation under the wheel path with the number of wheel passes are presented. The results are compared with the standard unpaved road design equation derived by Giroud and Noiray.

1 INTRODUCTION

Geosynthetic reinforcements may be used to reinforce the granular layers of a road and so improve the in-service performance of the pavement. A wide range of products with various forms are sold for this purpose and the discerning highway engineer may well be forgiven for hesitating before selecting the best product for his scheme.

Currently there are no indications from laboratory tests of the influence that the geosynthetic material will have on the performance of the pavement under trafficking. Trafficking Trials have shown that materials perform differently and these Trials, using a number of different products, indicate how materials could be included in standard sub-base design based on performance rather than laboratory testing data.

Two Trials have been undertaken, encompassing a range of 12 geosynthetic reinforcement products. The Trials were performed at the Pavement Test Facility (PTF) at the Transport Research Laboratory (TRL Limited).

This paper describes general arrangement for these Trials and presents the results of this work. The paper builds upon and extends the work reported by Jenner et al (2002)

2 GENERAL ARRANGEMENT FOR THE TRIALS

The PTF comprises a pit 10 m wide 25 m long by 3 m deep, containing a clay subgrade on which experimental pavements are constructed. A gantry, spanning the pit supports a road wheel that traffics backwards and forwards across the full width of the test pavement, as shown in Figure 1.

The general arrangement for the two Trials was the same, and comprised a conditioned subgrade overlain by a compacted granular sub-base; the geosynthetic reinforcements were installed on the surface of the subgrade.

2.1 *Test arrangement*

Trafficking lanes, 2.4m wide, were constructed across the width of the pit; these were each subdivided into 3 sections. A different reinforcement material was installed in each Section. The layout for Trials A and B are depicted in Figures 2 and 3 respectively. Section 2c of Trial B was the only Section that included two layers of reinforcement, the upper layer being at the mid height of the sub-base layer.

Figure 1 Pavement test facility

2.2 *Preparation of the subgrade*

The subgrade was a grey silty London clay, of very high plasticity.

Prior to the start of the Trials a series of tests were undertaken to determine the relations between the CBR value and the Moisture Condition Value (MCV).

Tests were also undertaken to determine a relation between the CBR value of the clay and the Cone Index (CI) value measured with a penetrometer developed by the British Military Engineering Experimental Establishment. The relation between CBR value and CI of the penetrometer, fitted with a 20mm cone, was found to be:

$$CBR(\%) = 0.033xCl \qquad (1)$$

where CI is the mean for the top 150mm of subgrade.

2.3 *Conditioning and placement of the subgrade*

Trial A - The top 500mm of the subgrade was excavated and carefully conditioned by a cycle of wetting and rotavating to provide a homogeneous material with a CBR of 2 per cent. The conditioned clay was then placed and compacted into the pit in layers in accordance with Series 600 of the Specification for Highway Works (SHW) (MCHW 1): i.e. a compacted layer thickness of 150 mm was achieved with 8 passes of a Rammax RW2100 compactor. The mean thickness of the conditioned subgrade layer was approximately 550 mm.

Trial B - Clay excavated from the pit was progressively wetted and rotavated as required, to provide a homogenous material with an MCV equivalent to a CBR value of 2 per cent. The conditioned clay was then placed and compacted in accordance with Series 600 of the SHW (MCHW 1): i.e. a compacted layer thickness of 110mm was achieved with 12 passes of a Rammax RW2400 compactor (mass 1039kg/m per roll). The mean thickness of the conditioned subgrade layer was approximately 550mm.

For both Trials, the mean CBR value of the placed subgrade was determined for each sub-section, from nine tests, spaced at 1m intervals along the centreline and lines 750mm either side, using the cone penetrometer. The mean CBR value for each section is presented in Table 1.

Table 1 Subgrade strength CBR (%)

Section	Lane 1	Lane 2	Lane 3	Lane 4
Trial A				
a	1.52	1.58	1.49	1.52
b	1.63	1.59	1.49	1.53
c	1.42	1.49	1.47	1.50
Trial B				
a	2.14	2.32	2.25	-
b	2.16	2.25	2.21	-
c	2.17	2.14	2.11	-

Section	Lane				<<< Direction of trafficking >>>
	1a 40kN woven geotextile (Lotrak 40)	2a 40kN punched and stretched integral grid (Tensar SS40)	3a 30kN punched and stretched integral grid (Tensar SS30)	4a Unreinforced Control	
	1b 40kN welded, bonded grid (Secugrid40)	2b 30kN extruded, integral grid (Tenax LBO 330)	3b 30kN extruded, integral grid (Tenax LBO 330)	4b 20kN punched and stretched integral grid (Tensar SS20)	
	1c 40kN woven grid (Telegrid 40)	2c Unreinforced Control	3c 40kN composite grid (TRC 40)	4c 30kN punched and stretched integral grid (Tensar SS30)	

Figure 2 Layout of reinforcements for Trial A

Section	Lane				<<< Direction of trafficking >>>
	1a 40kN knitted grid (Fornit 40/40)	2a 30kN punched and stretched integral grid (Tensar SS30)	3a Unreinforced Control		
	1b Unreinforced Control	2b 30kN punched & stretched, integral grid (Tensar SSLA 30)	3b 40kN woven grid (Enkagrid 40)		
	1c 20kN punched and stretched integral grid (Tensar SS20)	2c Double layer of 30kN punched and stretched integral grid (Tensar SS30)	3c 60kN welded, bonded grid (Secugrid 60)		

Figure 3 Layout of reinforcements for Trial B

2.4 *Geosynthetic reinforcements*

The geosynthetic reinforcements were installed directly onto the surface of the subgrade. Sections without reinforcement were termed Control Sections. The location, product form, and nominal short term strength of each geosynthetic are presented in Figures 2 and 3 for Trials A and B respectively.

2.5 *Preparation and placement of the sub-base*

The sub-base material consisted of a crushed granite aggregate that conformed with the requirements of the 800 Series of the SHW (MCHW 1). The material was placed and compacted in (two layers) in accordance with the SHW (MCHW 1); the target thickness for the sub-base layers was 150mm, For Trial A the sub-base layer was increased to 160mm due to compensate for the lower than anticipated subgrade strength.

Care was taken in placing and spreading the sub-base to minimise damage to the geosynthetic reinforcements, and due to the low CBR value of the clay to minimise possible deformation of the subgrade surface.

For each Trial and Section, the mean thickness of the sub-base layer was determined from a level survey of some 30 points on the surfaces of the subgrade and sub-base. The mean sub-base thickness for each Section is provided in Table 2.

Table 2 Thickness of the sub-base layer (mm)

Section	Lane 1	Lane 2	Lane 3	Lane 4
Trial A				
a	320	322	331	316
b	328	323	334	318
c	320	331	327	304
Trial B				
a	288	286	280	-
b	292	292	283	-
c	292	275	284	-

2.6 *Pavement stiffness*

The stiffness of the pavement was assessed using a Falling Weight Deflectometer (FWD), as described by Sorensen and Hayven (1982).

The FWD was manually positioned over the Trial pavement to minimise possible disturbance to the pavement and because of the confined space. For all tests, the FWD was fitted with a 300mm diameter segmental loading plate, and the weight fell from a predetermined height to produce a stress of 150kPa on the surface of the sub-base.

The pavement stiffness (E) was calculated from the following equation:

$$E = \frac{2qa(1-\nu^2)}{d} \tag{2}$$

where:

q is the stress under the plate
a is the radius of the plate
ν is Poisson's ratio of the sub-base (taken as 0.45)
d is the maximum deflection at the centre of the plate

Prior to trafficking three tests were undertaken on the centreline of each Section to measure the pavement modulus on the compacted surface of the sub-base. The mean modulus for each Section is given in Table 3.

Table 3 Surface modulus (MPa) prior to trafficking

Section	Lane 1	Lane 2	Lane 3	Lane 4
Trial A				
a	40.8	41.9	41.2	37.4
b	44.4	36.6	44.6	41.9
c	37.7	42.4	37.5	35.3
Trial B				
a	27.1	34.6	29.4	-
b	30.8	33.5	28.8	-
c	33.0	25.6	29.6	-

3 ARRANGEMENT FOR TRAFFICKING

The PTF may be operated with a twin or single road wheel, and axle loadings from 23kN to 100kN. For both Trials a dual wheel pair was used with an axle loading of 40kN, i.e. about half of a standard axle. The wheel passes were bi-directional and canalised across the centre of each Section at a speed of 15kph. The maximum number of wheel passes was 10,000. The vertical deformation was monitored at intervals during trafficking, using an optical level. Failure of a Section was deemed to have occurred if the vertical deformation in the wheel path exceeded 80mm.

When the failure of a subsection occurred the wheel was halted, and the surface profile was determined. Also, by excavating a trench through the sub-base the surface profile of the subgrade was also determined. Before recommencing trafficking, the sub-base was reinstated and brought up to its original level to prevent a ‘step’ developing between adjacent subsections. If the failed subsection contained reinforcement, a sample of the geosynthetic was recovered for inspection and a ‘patch’ of new material was placed over the bottom of the trench before reinstating the sub-base.

4 RESULTS

The relation of increasing vertical deformation in the wheel path, with increasing number of wheel passes, for each lane is presented graphically. The results for Lanes 1, 2, 3 and 4 for Trial A are presented Figure 4a, 4b, 4c and 4d respectively. The corresponding results for Lanes 1, 2, and 3 of Trial B are presented in Figures 5a, 5b and 5c.

Measurements of surface modulus were recorded for Trial A, after 2,000 passes on Lanes 1 and 4, and after 5,000 passes on Lanes 3 and 4. Due to excessive deformation, measurements could not be taken on Sections 1c, 2c and 4a. The modulus values and the percentage change from the pre-trafficking values, are presented in Table 4.

Measurements of surface modulus were not taken during the trafficking of Trial B.

Table 4 Surface modulus (Mpa) and percentage change of modulus measured during trafficking

Section	Lane 1	Lane 2	Lane 3	Lane 4
No. Of passes	2,000	5,000	5,000	2,000
a	26.4 (35)	38.4 (8)	38.2 (7)	*
b	35.7 (20)	33.7 (8)	36.9 (17)	28.7 (32)
C	*	*	24.4 (35)	28.2 (18)

* Not tested

5 DISCUSSION

5.1 *Trial A.*

Only four of the reinforced Sections 2a, 2b, 3a and 3b achieved the target of 10,000 passes; all of these contained integral grids and exhibited a slower rate of deformation. However 5b and 4c were also integral grids and these Sections failed after only 5,000 passes. It is interesting to note that Control Section 2c failed at 2,000 passes whereas the Control Section 4a failed after only 500 passes. Thus it is apparent that though the pavement condition in Lanes 2 and 3 were similar, that of Lane 4 was probably significantly less strong. This hypothesis is supported by a consideration of the magnitude of the reduction of the surface modulus of these Sections after the commencement of trafficking.

An inspection of Table 4 shows that the surface moduli for all Sections reduced after trafficking, but there was a wide variation in the magnitude of the changes, which ranged from 7 to 38 per cent. This would go some way to explaining the different performance of the same or similar reinforcements in different Sections. Explanation of this variation is not certain, but the subgrade was conditioned outside the PTF and it is conceivable that inclement weather may have dampened some of the clay, though normal practice required that such clay should be discarded.

A further explanation for the poor performance of Lane is that the sub-base thickness was about 5 percent thinner than for the other Lanes.

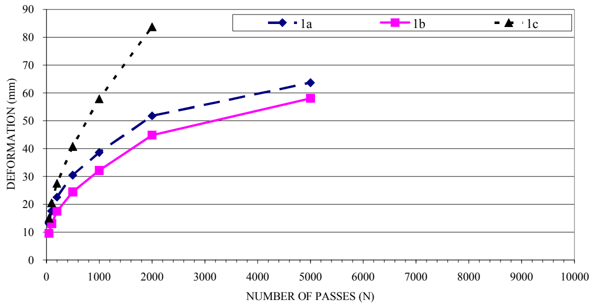

Figure 4a Trial A: Development of deformation in Lane 1

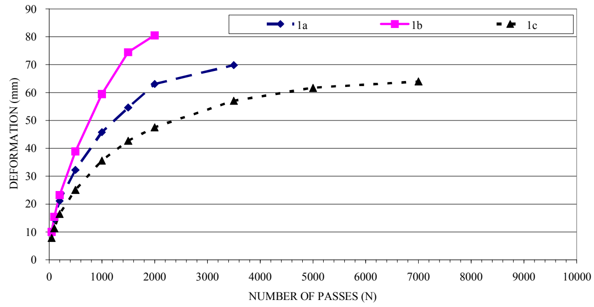

Figure 5a Trial B: Development of deformation in Lane 1

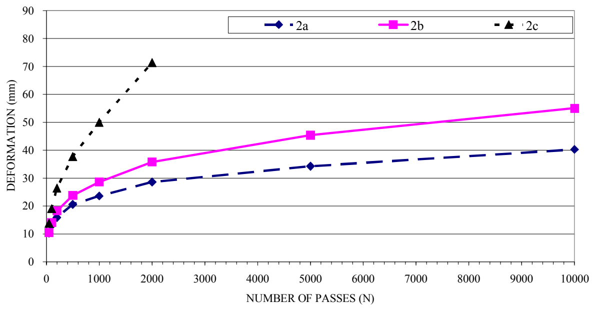

Figure 4b Trial A: Development of deformation in Lane 2

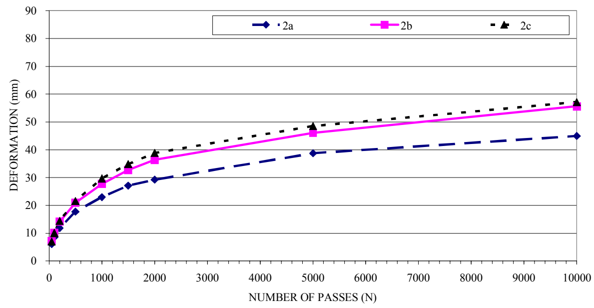

Figure 5b Trial B: Development of deformation in Lane 2

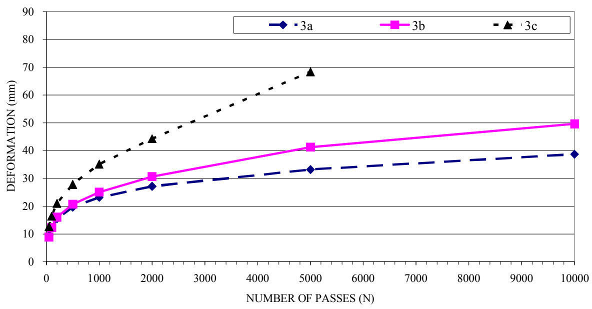

Figure4c Trial A: Development of deformation in Lane 3

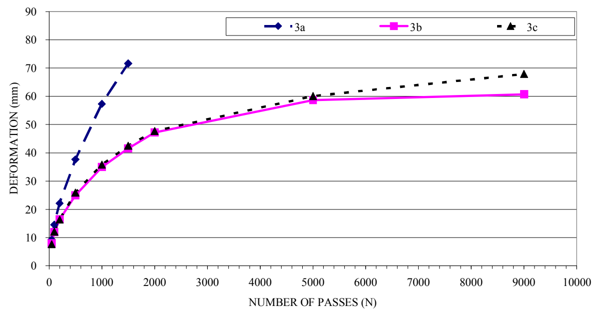

Figure 5c Trial B: Development of deformation in Lane 3

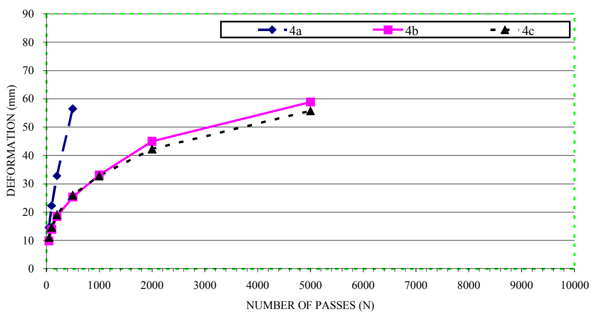

Figure 4d Trial A: Development of deformation in Lane 4

5.2 *Trial B*

The results show that the Sections incorporating the integral grids demonstrated the best performance, by exhibiting a slower rate of deformation and achieving the target of 10,000 passes. The Control Sections exhibited the worst performance.

Section 2a and 2c contained the same reinforcement material, but 2c contained a second layer at the mid-height of the sub-base layer. Perhaps surprisingly, this Section performed less well than the other. It is conjectured that because the reinforcements were only 150mm apart the reinforcing effect of one layer interacted with the other. Had the sub-base layer been thicker, a greater separation of the reinforcements could have been achieved thereby allowing each to act independently and so providing the maximum benefit to the road.

The Sections in Trial A and B reinforced with the same materials exhibited a similar performance.

The initial measurements of pavement stiffness were generally lower than for Trial A. This consistency suggests that Trial B may permit a better comparison of benefits of particular reinforcements than did Trial A. The lack of knowledge for the reduction in surface modulus is to be regretted. As seen previously this quantity provides a sound indication of non-uniformity within the pavement, and therefore may also provide an indication of the likely performance of the Section.

5.3 *A comparison of the results*

Reinforcement of the granular layers within a road pavement is frequently used when the underlying material is weak, or when the thickness of the pavement needs to be kept to a minimum. All the reinforced Sections described above performed better than the un-reinforced Control Sections. But the above results clearly show that different reinforcements are unlikely to provide the same degree of improvement to the road's performance.

The criterion was used to select the depth of the sub-base, is based on the relation, suggested by Giroud and Noiray (1981), between layer thickness and number of standard axles to generate 40mm deformation.

$$Log_{10} N = \frac{h(CBR)^{0.63}}{190} \qquad (3)$$

where N = number of standard axles
h = thickness of sub-base for a rut depth of 75mm (40mm deformation)

For a target of 10,000 passes and a CBR of 2 per cent, the required depth of an un-reinforced sub-base layer is 491mm. To estimate the equivalent depth for a reinforced sub-base the empirical so called $^{1}/_{3}$ rule, used by some manufactures, gives a depth of 329mm. For the Trials at TRL, this value was further reduced to 300mm to ensure that measurable deformations would be recorded.

The number of passes required to achieve 40mm deformation are summarised for Trials A and B in Figures 6a and 6b respectively; (N.B. Section 3a, Trial A, had not achieved a deformation of 40mm at 10,000 passes). The Figures show that some Sections developed 40mm deformation with a surprisingly small number of passes, and it is evident that some reinforcements provided a substantially better resistance to deformation than others in the early life of the pavement.

It is notable that the response of the Control Sections were much the same, with the exception of 4c, Trial A; this gives weight to the previous surmise that this Lane was notably weaker than the others.

From a knowledge of the number of passes required to generate 40mm deformation, the CBR value (from Table 1) and the mean thickness of the sub-base layer (D1) (Table 2), then equation (3) can be used to calculate a theoretical required sub-base depth (D2) as shown in Table 4. Included in the Table are (i) the ratio of theoretical to the actual depth, and (ii) the reduction of the theoretical depth required to equal the actual depth, expressed as a per cent of the actual depth.

The ratio of the theoretical to the actual depth is an indication of the degree of conservatism that is implicit in calculation. The values of this ratio range from 1.09 to 1.79, indicating that though equation (3) will determine a safe solution, it may overestimate the sub-base depth by up to 44 per cent.

The data in Table 4 show that the so called $^{1}/_{3}$ rule for estimating the depth of a reinforced sub-base is not applicable for all reinforcement products.

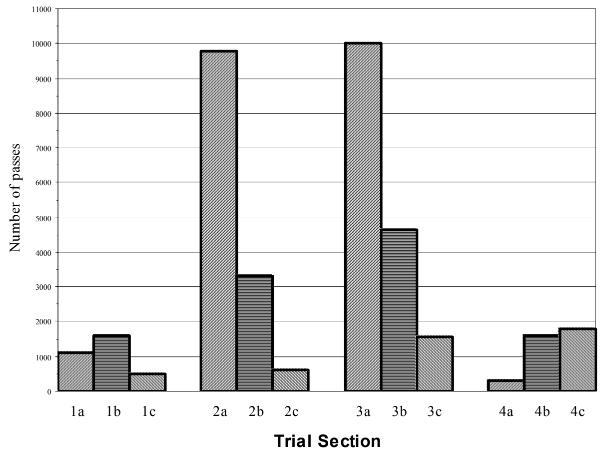

Figure 6a Trial A: Number of passes at 40 mm deformation

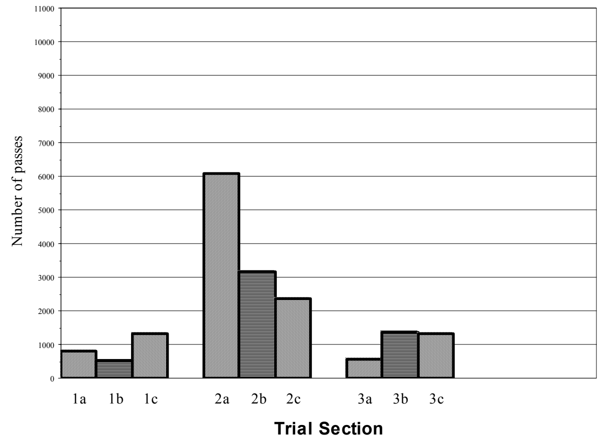

Figure 6b Trial B: Number of passes at 40 mm deformation

Table 4 Comparison of actual and theoretical sub-base depth

Section	No. of passes	D1 (mm)	D2 (mm)	Ratio D2/D1	Reduction (%)
Trial A					
1a	1100	320	444	1.39	28
1b	1610	328	448	1.37	27
1c	480	320	408	1.28	22
2a	9800	322	568	1.77	43
2b	3300	323	499	1.55	35
2c	600	331	411	1.24	19
3a	10000	331	591	1.79	44
3b	4650	334	542	1.62	38
3c	1550	327	476	1.45	31
4a	300	316	362	1.14	13
4b	1590	318	465	1.46	32
4c	1770	304	478	1.57	36
Trial B					
1a	800	288	342	1.19	16
1b	530	292	319	1.09	8
1c	1310	292	364	1.25	20
2a	6100	286	423	1.48	32
2b	3150	292	399	1.37	27
2c	2350	275	397	1.44	31
3a	570	280	314	1.12	11
3b	1380	283	362	1.28	22
3c	1340	284	371	1.31	23

6 CONCLUSIONS

The following conclusions have been drawn from this work.

Incorporating geosynthetic reinforcement into the granular layer of an unpaved road, will improve the road's in-service performance. However, different geosynthetics will provide different levels of improvement.

The change in surface modulus (FWD), prior to and in the early stages of trafficking, may provide an indication of the likely performance of the road in-service. Such measurements could provide a useful method of assessing the likely in-service performance of a road, and is worthy of further investigation.

The calculated values of the required depth of the sub-base layer, determined using equation (3) derived by Giroud and Noiray (1981), provided a safe solution for all the reinforcements used in the Trials. But the calculated depths were overly conservative for some reinforcements.

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