Tensar reinforcement of asphalt: laboratory studies

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INTRODUCTION

In most countries, pavement design methods and techniques for specifying asphalt mixtures are largely empirical. Encouraging progress is being made in the application of theoretical concepts and the use of mechanical properties of paving materials towards improving design practice. Economic advantages will be apparent as present research knowledge is implemented. There are, however, a number of problems in asphalt pavements for which solutions using conventional materials are unlikely to be completely successful, even with the use of new design techniques. These problems include the development of rutting caused by permanent deformation in heavy vehicle wheel tracks under high temperature conditions and the propagation of cracks through asphalt surfacings (overlays) placed on existing cracked pavements. These latter are known as 'reflection cracks'.

MECHANICAL PROPERTIES OF THE POLYMER GRID

The material supplied for use in this project was Tensar ARI polypropylene grid which has the geometric properties shown in Fig. 1. This is a biaxially orientated product with larger dimensions in the 'primary' direction and hence, greater strength and stiffness in this direction too. All the tests were carried out on the same batch of material.

Stiffness of Tensar ARI

For pavement applications, the grid would be subjected to repeated applications of low strains (or loads). The test arrangement developed to measure the load/strain relationships under these conditions involved a 0.5 m wide x 0.8 m long test specimen held by special clamps in a servo-hydraulic testing machine. The test involved applications of increasing amplitudes of controlled deformation at 1 Hz using a sinusoidal waveform and measurement of the corresponding load amplitude. Cyclic strains up to 0.8% were applied.

In view of the importance of assessing the influence of exposure to the elevated temperatures experienced in asphalt paving, a number of grid specimens were heated in various ways and retested after cooling. Fig. 2 shows the load displacement curves for a typical specimen at one amplitude traced from the X-Y plotter used to monitor results. A reduction in stiffness is apparent as a result of the exposure to elevated temperature (in this case hot asphalt placed at 165°C). The load displacement curves exhibit slight hysteresis, but the response is essentially linear. The tests were carried out at 20°C.

Fig. 3 shows the results from a typical set of tests on another specimen and has been developed from data such as that in Fig. 2. Each point represents the peak load and strain during increasing and decreasing increments.
In all tests the load was applied in the primary directions.

From the slopes of the lines, such as those in Fig. 3, the values of Young's modulus and of elastic stiffness were calculated. The former was calculated on the basis of stress determined using the minimum cross-sectional area of the ribs (see Fig. 1). This area was 3.65 mm² per rib and the average Young's modulus from nine tests on unheated grids was 14.7 GPa. Since the specification of cross-sectional area is somewhat arbitrary, it was considered more useful to describe elastic behaviour in terms of elastic stiffness, being the load per unit width divided by the strain. The mean value from all nine tests for this was 1.2 MN/m.

The reduction in stiffness after heating and cooling depended on the exposure temperature and the medium in which it was applied. Fig. 5 summarises all the test results to date including the use of a variety of containing media offering different degrees of restraint to the grid which had a tendency to shrink on heating. For typical asphalt paving temperatures of 140°C, stiffness is about 60% of that for new material.

The tests were all carried out on relatively small pieces of grid and the restraint may not be representative of that which could develop in a pavement. A significant force (typically 0.8 kN/m) builds up in the grid if it is rigidly restrained while being heated. Developments in the production process and placement techniques on site will change the detailed characteristics, so additional testing will be required when these matters have been pursued further.

Fatigue Characteristics of Tensar ARL

Four cyclic load fatigue tests were carried out in the servo-hydraulic machine on test specimens 360 mm long x 250 mm wide at 20°C. A mean strain of between 8.7 and 13% was applied and a cyclic strain of ± 0.25% was superimposed using a sinusoidal waveform at 15 Hz. The load was monitored and a typical result is shown in Fig. 5. This indicates that the cyclic load remained unchanged over the 370,000 cycles involved. This implies no decrease in elastic stiffness and, hence, no suggestion of fatigue failure. The mean load, however, relaxed in a manner characteristic of visco elastic materials. The same pattern was apparent in all four tests and this stress relaxation behaviour had earlier been noted in static tests at lower mean strain levels.

THE ASPHALTIC MATERIALS

The mixtures used in this investigation were all made to current British Standard Specifications (British Standard Institution, 1973, a and b) and included not rolled asphalt (HRA) and dense bitumen macadam (DBM) wearing course...
and hot rolled asphalt basecourse. The aggregate grading curves are shown in Fig. 6. In U.K. practice, the wearing course materials would be used in approximately 40mm thick layers at the road surface while the basecourse mixture would be used in layers of about 60mm thickness below. Both materials could feature in either overlays or new construction.

The maximum aggregate sizes varied between 10 and 20mm (see Fig. 6) and this factor should be considered in relation to the grid aperture size of 63 x 45mm (see Fig. 1).

![Fig. 6 Asphalt grading curves](image)

Bitumen does not stick to polypropylene so continuity between the grid and the asphalt relies entirely on interlock. Furthermore, to avoid creating a plane of weakness (low resistance to shear stress) by inclusion of the grid, there must be sufficient continuity of asphalt through the grid apertures. The proportions of Tensar ARI were considered adequate for aggregate sizes up to 20mm but the aperture size is a parameter which could be varied and is the subject of current research.

Table 1 gives details of the binder content for each mix and the grade of bitumen used characterised by its penetration (British Standards Institution, 1974). The average air void contents are also indicated as a measure of the state of compaction and this is discussed in more detail in a subsequent section of the paper.

The grading curves of Fig. 6 show that the HRA's have gap gradings while the DBM has a continuous grading. Resistance to permanent deformation is inherently better for the DBM type of mix, which mobilises higher aggregate interparticle friction than the HRA. The latter, however, because of its higher binder content and harder bitumen, has the better tensile strength.

Various slabs and beams of these mixtures were prepared, both reinforced and unreinforced. Details are presented in the appropriate section of the paper.

### Table 1 - Details of Asphalt Mixtures

<table>
<thead>
<tr>
<th>Mix Type</th>
<th>Bitumen Content (% by Mass)</th>
<th>Bitumen Penetration Grade</th>
<th>Air Void Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HRA wearing Course</td>
<td>7.9</td>
<td>50</td>
<td>5</td>
</tr>
<tr>
<td>DBM wearing Course</td>
<td>5.0</td>
<td>100</td>
<td>9</td>
</tr>
<tr>
<td>HRA Base Course</td>
<td>5.7</td>
<td>50</td>
<td>2</td>
</tr>
</tbody>
</table>

**ELASTIC STIFFNESS OF REINFORCED ASPHALT**

The relationship between uniaxial stress and strain in bituminous material is termed "stiffness" after Van der Poel (1954). Under conditions of high strain rate such as occur in pavements subjected to moving wheel loads, asphalt behaves in an essentially elastic manner, although small residual strains do develop and the accumulation of these lead to surface rutting.

The stresses transmitted to the lower layers of a road and those set up in the asphalt layer are strongly dependent on the elastic stiffness of asphaltic materials. This parameter is therefore most important for design computations.

Elastic stiffness is a function of temperature and loading time. For a rolled asphalt wearing course mixture at 20°C and typical vehicle speed of 80 km/hr, the elastic stiffness would be about 2 GPa. The DBM would be similar while the HRA basecourse would be a little stiffer. Under similar conditions, the Young's modulus of Tensar ARI was found to be about 18 GPa which could reduce to, say, 9 GPa as a result of its exposure to elevated temperatures during paving. Hence, the order of magnitude of the stiffness is similar. In a unit cross-sectional area of asphalt, the percentage of reinforcement ribs is likely to be very low. In view of these factors it seems unlikely that the presence of the grid will increase the effective stiffness of the composite. Exceptions could occur at high temperatures when asphalt stiffness will be low and the stiffness modulus ratio will be larger. It seems unlikely, though, that major improvements to stiffness under low strain conditions will be realised. The real benefits will come from situations where large strains can develop (permanent deformation or opening of cracks) when the grid should be effective. A few experiments were carried out to check the low strain stiffness of reinforced HRA wearing course. A pair of rectangular test specimens 100 x 89mm in cross-section and 250mm long were cut from a reinforced and an unreinforced slab. In the reinforced case, two ribs of Tensar ARI were located along the centre of the specimen. Steel loading plates
were glued onto each end.

Cyclic, axial load tension-compression tests were performed with LVDT's attached over gauge lengthson opposite faces of each specimen to monitor the small deformations accurately. Stresses between 600 and 1200 kPa were applied at a frequency of 16 Hz which corresponds to about 100 km/hr traffic speed. The test temperature was 20°C. The results are detailed in Table 2, readings have been taken after as few cycles as possible.

Table 2 - Results of axial load elastic stiffness tests on HRA wearing course

<table>
<thead>
<tr>
<th>Specimen Details</th>
<th>Axial Stress (kPa)</th>
<th>Axial Strain (microstrain)</th>
<th>Elastic Stiffness (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforced</td>
<td>657</td>
<td>133</td>
<td>5.2</td>
</tr>
<tr>
<td>Void Content</td>
<td>921</td>
<td>200</td>
<td>4.6</td>
</tr>
<tr>
<td>= 9.11</td>
<td>1135</td>
<td>300</td>
<td>3.8</td>
</tr>
<tr>
<td>Unreinforced</td>
<td>652</td>
<td>133</td>
<td>5.0</td>
</tr>
<tr>
<td>Void Content</td>
<td>888</td>
<td>183</td>
<td>4.9</td>
</tr>
<tr>
<td>= 7.8%</td>
<td>1124</td>
<td>267</td>
<td>4.2</td>
</tr>
</tbody>
</table>

It will be noted that the levels of stiffness were almost identical on both specimens. The state of compaction in the reinforced one was poorer than in the other, which is one of the consequences of including the grid. Hence, there may be some compensating effects resulting in the same stiffness, but no improvement was obtained.

A further pair of tests were conducted using the configuration shown in Fig. 7 which is a beam on an elastic support. Cyclic loads were applied through the loading pad at 5 Hz and a temperature of 20°C. The horizontal strain at the bottom of the beam was determined from the LVDT shown in Fig. 7. The results for reinforced and unreinforced beams are shown in Fig. 8 from which it will be noted that there is no significant difference between the two. The strains involved in this case were larger than in the axial load tests.

![Fig. 7 Asphalt beam on elastic support](image)

Fig. 8 Stiffness from beam tests

RESISTANCE TO PERMANENT DEFORMATION

A series of wheel tracking tests was carried out on 1.2 x 0.34m slabs of all three mixes and, in each case, an identical pair was manufactured with one containing the reinforcing grid. Table 3 presents details of all the slabs showing the method and level of compaction, the thickness and the position of the grid.

The slabs were cast in steel formwork and compacted using a hand operated vibrating roller. For tests E to H, the slabs were compacted in two stages with the grid being placed upon the first layer after rolling. This was considered to be most representative of likely site practice. In the other cases, the grid was placed on an uncompacted first layer with rolling being effected after placement of the second.

A study of the void contents in Table 3 shows that the presence of the grid did inhibit compaction slightly. In some of the HRA wearing course slabs, the void contents in the top and bottom parts of the slab were determined and these results are shown in Table A. Clearly, compaction at both levels provides more uniform densities in the asphalt.

The slabs were tested in the Nottingham Pavement Test Facility (Brown and Brdicka, 1981). Some staging was constructed to provide a constant support for all the test slabs with a resilience representative of the site situation.

All tests were carried out at 30°C. For all but tests I and J a wheel contact pressure of 415 kPa and speed of 8 km/hr were used. The width of the contact area was about 150mm. A profilometer was used to measure the transverse surface profile at intervals during the tests which generally were continued to 50,000 wheel passes.

For tests I and J, the contact pressure below the tyre reduced to 250 and the lateral position of the wheel was varied to produce a realistic transverse distribution of passes.
Table 3 - Details of Slabs after compaction

<table>
<thead>
<tr>
<th>Test</th>
<th>Slab No.</th>
<th>Material Construction Method</th>
<th>Slab Thickness (mm)</th>
<th>Tensile depth divided by 'layer thickness'</th>
<th>Air Void Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1</td>
<td>T1</td>
<td>86</td>
<td>0.71</td>
<td>7.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>86.7</td>
<td>0.1</td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>2</td>
<td>T2</td>
<td>86</td>
<td>0.77</td>
<td>4.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>86.7</td>
<td>0.1</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>3</td>
<td>T3</td>
<td>93</td>
<td>0.54</td>
<td>5.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>93.7</td>
<td>0.1</td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>4</td>
<td>T4</td>
<td>77</td>
<td>0.47</td>
<td>6.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>77.8</td>
<td>0.1</td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>5</td>
<td>T5</td>
<td>104</td>
<td>0.49</td>
<td>2.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>104.7</td>
<td>0.1</td>
<td></td>
</tr>
<tr>
<td>F</td>
<td>6</td>
<td>T6</td>
<td>104</td>
<td>0.63</td>
<td>3.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>104.7</td>
<td>0.1</td>
<td></td>
</tr>
<tr>
<td>G</td>
<td>7</td>
<td>T7</td>
<td>87</td>
<td>0.40</td>
<td>2.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>87.8</td>
<td>0.1</td>
<td></td>
</tr>
<tr>
<td>H</td>
<td>8</td>
<td>T8</td>
<td>89</td>
<td>0.69</td>
<td>8.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>89.7</td>
<td>0.1</td>
<td></td>
</tr>
<tr>
<td>I</td>
<td>9</td>
<td>T9</td>
<td>107</td>
<td>0.53</td>
<td>1.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>107.8</td>
<td>0.1</td>
<td></td>
</tr>
<tr>
<td>J</td>
<td>10</td>
<td>T10</td>
<td>110</td>
<td>0.5</td>
<td>2.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>110.7</td>
<td>0.1</td>
<td></td>
</tr>
</tbody>
</table>

In order to illustrate the results, data from tests A, E, G and I have been reproduced in Figures 9 and 10. This includes the HRA wearing course with both types of construction (A and E), the DBM (Test G) and the HRA base course (Test I).

Fig. 9 shows the build up of permanent deformation during each test and Fig. 10 illustrates the final transverse surface profiles. It is clear that substantial reductions in permanent deformation are apparent in all tests with that in the DBM particularly marked. The lower levels of deformation in Test I resulted from the reduced applied stress on a more resistant mix. The tests on DBM (G and H) only went to about 5000 passes as this material was poorly compacted (see Table 3) and early failure developed. The lateral restraining effect of the reinforcement was, however, particularly apparent in these tests as illustrated by the data in Table 5. These were obtained from horizontal measurements between markers on the slab surface 200mm apart across the wheel track.

The reductions in rut depth for the various groups of tests are summarised in Table 6.

Omitting G and H, where the test conditions proved rather extreme, it is apparent that better performance was achieved when the grid was placed prior to compaction of the asphalt above and below it in one operation. Placing the grid at a well-defined interface probably results in poorer interlock.

RESISTANCE TO REFLECTION CRACKING

The experimental arrangement illustrated in Plate I and Fig. 11 was used to apply cyclic loads to beams cast over a preformed crack representative of an overlay above a concrete pavement joint, considered to be the most severe source of reflection cracking.

The beams were 525mm x 150mm x 100mm deep and a layer of Tensar AR1 was placed in various positions as detailed on the schedule of tests in Table 7. The beam width was such that three primary ribs provided the reinforcement. All beams were made of HRA wearing course material and compacted to the same void content (10%).

The 10mm wide discontinuity below the beam was provided by a pair of high quality plywood...
Table 4 - Influence of Reinforcement and Construction Method on Compaction

<table>
<thead>
<tr>
<th>Construction Method</th>
<th>Type of Slab</th>
<th>Air Void Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Slab</td>
<td>Bottom of Slab</td>
</tr>
<tr>
<td></td>
<td>Unreinforced</td>
<td>7.3</td>
</tr>
<tr>
<td></td>
<td>Reinforced</td>
<td>7.2</td>
</tr>
<tr>
<td>Compaction of complete slab</td>
<td>Unreinforced</td>
<td>2.3</td>
</tr>
<tr>
<td></td>
<td>Reinforced</td>
<td>3.3</td>
</tr>
</tbody>
</table>

Table 5 - Transverse Surface Deformation for Tests G and H between 1000 and 30,000 Passes

<table>
<thead>
<tr>
<th>Test</th>
<th>Transverse Deformation (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>unreinforced</td>
</tr>
<tr>
<td>G</td>
<td>47</td>
</tr>
<tr>
<td>H</td>
<td>28</td>
</tr>
</tbody>
</table>

Fig. 9 Build up of permanent deformation sheets. A resilient support in the form of a piece of rubber was located below the plywood. (see Fig. 11).

Load was applied through a rubber based loading platen 200mm wide placed at the beam centre. A loading frequency of 5 Hz was used and the load was cycled between 1.2 and 8.3 kN in all tests. This gave a peak contact stress below the loading pad of 275 kPa.

An LVDT was located to measure horizontal displacement across the plywood gap (Fig. 11). Under test, the gap acted as a crack inducer and the progress of crack propagation on either side of each beam was monitored visually. This was facilitated by painting the beams white. The average height of the crack tip above the bottom of the beam was recorded at intervals during the test. Initially the cyclic deformation across the gap was approximately 0.05mm giving an average local strain of 0.5% which is very high for asphalt.

Fig. 10 Transverse profiles after testing

Fig. 12 summarises the results of these experiments showing mean crack lengths from the replicated tests. When the grid was placed at the bottom of the beam no cracking was observed. In similar unreinforced beams after corresponding numbers of load cycles, the cracks had propagated above mid-depth and the beams could be regarded as having failed. When the grid was placed at intermediate depths, although there was crack development and growth, it was controlled. In addition, the cracks were of hairline type indicating the effectiveness of the grid in holding the asphalt together.

This point is well illustrated in Plate 2. The results for the intermediate grid positions were apparently inconsistent as better perfor-
higher magnitude than a typical asphalt mix at 20°C and a loading time representative of traffic moving at 80 to 100 km/hr.

3. Under cyclic loading representative of severe conditions in a pavement, Tensar ARI showed no indication of fatigue failure but did exhibit mean load relaxation.

4. The presence of Tensar ARI in hot rolled asphalt did not increase its elastic stiffness.

5. Inclusion of Tensar ARI in three different asphalt mixes increased the resistance to rutting substantially.

6. Best rutting resistance was obtained when the grid was compacted in an asphalt sandwich rather than by placing it on a precompacted surface and then covering with further asphalt.

7. The presence of Tensar ARI impeded compaction of the asphalt slightly but this did not prevent the improved performance noted in 5.

8. A layer of Tensar ARI placed immediately over a discontinuity, representative of a concrete road joint, prevents the development of a reflection crack in the asphalt overlay. Inclusion of the grid within the overlay thickness inhibits the crack growth and limits its width.

ACKNOWLEDGEMENTS

The research described in this paper has been made possible by two co-operative research awards from the Science and Engineering Council in conjunction with Netlon Ltd. The Authors are grateful to Dr. F.B. Mercer and his colleagues at Netlon and Professor Sir Hugh Ford, Chairman of the SERC/Netlon research steering committee together with its members for providing a stimulating environment in which to pursue their studies. The support and encouragement of Professor P.S. Pell, Head of the Civil Engineering Department at Nottingham is also gratefully acknowledged.
Fig. 11 Details of crack initiation

Plate 1. Apparatus for reflection crack testing

Fig. 12 Crack growth

Table 7 - Details of Beam Tests

<table>
<thead>
<tr>
<th>Beam No.</th>
<th>Total No. of Load Cycles (Thousands)</th>
<th>Position of Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>102</td>
<td>Unreinforced</td>
</tr>
<tr>
<td>2</td>
<td>140</td>
<td>Unreinforced</td>
</tr>
<tr>
<td>3</td>
<td>206</td>
<td>Unreinforced</td>
</tr>
<tr>
<td>4R</td>
<td>103</td>
<td>Bottom</td>
</tr>
<tr>
<td>5R</td>
<td>167</td>
<td>Bottom</td>
</tr>
<tr>
<td>6R</td>
<td>218</td>
<td>Bottom</td>
</tr>
<tr>
<td>7R</td>
<td>241</td>
<td>3/4 depth</td>
</tr>
<tr>
<td>8R</td>
<td>279</td>
<td>3/4 depth</td>
</tr>
<tr>
<td>9R</td>
<td>245</td>
<td>mid-depth</td>
</tr>
<tr>
<td>10R</td>
<td>250</td>
<td>mid-depth</td>
</tr>
</tbody>
</table>

The references are as follows:


Together with the advice of Mr. K.E. Cooper.
Structural behaviour of Tensar reinforced pavements and some field applications

R. Haas, University of Waterloo

INTRODUCTION

If pavement reinforcement could reduce the thickness of layers and/or extend pavement life in a cost and performance effective way, it would certainly be a viable alternative to conventional designs.

The potential of a new polymer geogrid, Tensar, in achieving this objective was realized in 1979 shortly after it was introduced, and a comprehensive research program was initiated in 1980. The objectives of the program were to evaluate the behavior and effectiveness of Tensar in paved road structures. An integrated set of laboratory and field experiments were planned and carried out. The laboratory experiments were felt to be vital not only for assessing behavior and establishing design parameters but also as a basis for intelligent planning of the field trials.

The laboratory experiments were carried out as "model" tests to simulate full-scale pavements and full-scale dynamic loads, but under controlled conditions.

It is the purpose of this paper to describe the laboratory experiments and results, the analyses conducted and some of the field applications.

Experimental and Analytical Program Objectives

Reinforcement can occupy several alternative locations in a road structure, ranging from the subgrade to the subbase and base layers to the asphalt layer. Its relative effectiveness depends on the strength of the layers, particularly that of the subgrade, their thickness and the number of loads to be carried. Thus, each individual design situation dictates where the reinforcement should be placed, and its relative effectiveness.

We carried out an initial analytical investigation, using elastic layer theory, for a wide range of paved road designs. The results indicated that the Tensar geogrid offered particular potential if used in thicker asphalt layers where high traffic volumes occur. Consequently, the experimental program concentrated on this application, as described in the following sections, but current investigations are covering the full range of road structures.

The experimental program was designed to include varying thicknesses of reinforced and unreinforced asphalt on subgrades of varying strength. Its main goal was to thoroughly investigate, under a variety of controlled conditions, the mechanical behavior and load-carrying capabilities of Tensar reinforced flexible pavements and to compare with unreinforced (control) sections.

As well, the results of the program were to be used to verify and/or modify the elastic layer theory and develop initial design procedures for reinforced asphalt pavements.

THE EXPERIMENTAL INVESTIGATION

Test Facility

The pavement sections were constructed in a test pit at Royal Military College (RMC) in Kingston, Ontario. Dimensions of the pit are 6 m by 2.4 m wide by 2 m deep.

Dynamic loads, to simulate dual truck tires, were applied through electro-hydraulic actuators and a circular plate.

Controlled Variables and Sequence of the Experiments

The experimental program was divided into five series of tests, called loops. Each loop involved one set-up of the test pit, in which half of the pit was reinforced and the other half was left as a control section. For each loop, the asphalt thickness and the subgrade condition (either dry or saturated) were the controlled variables. Between 4 and 9 tests were performed for each loop, with each test representing a different location within the test pit. The five loops are described in Table 1.

The design of these test loops involved a logical sequence to assess certain parameters related to reinforcement evaluation. For example, the first loop was designed to compare the behavior and performance of the reinforced section with an unreinforced section of the same thickness (150 mm) on a weak subgrade. Permanent deformation and vertical deflections were monitored throughout the test until complete failure occurred on the unreinforced section.

In loop 2, strain carriers were installed at the bottom of the asphalt layer to compare tensile strain, in the critical zone, between reinforced and unreinforced sections of the same thickness (165 mm) on a stronger subgrade. Results of these two loops were of major
importance since they compare the reinforced sections with unreinforced sections under identical geometric, loading and environmental conditions.

### TABLE 1

<table>
<thead>
<tr>
<th>Loop No.</th>
<th>Test No.</th>
<th>Asphalt Thickness</th>
<th>Subgrade Condition</th>
<th>Description</th>
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<tbody>
<tr>
<td>(1) 1</td>
<td>1</td>
<td>150 mm</td>
<td>Dry</td>
<td>Control</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>150 mm</td>
<td>Dry</td>
<td>Reinforced</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>150 mm</td>
<td>Saturated</td>
<td>Reinforced</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>150 mm</td>
<td>Saturated</td>
<td>Control</td>
</tr>
<tr>
<td>(2) 1</td>
<td>1</td>
<td>165 mm</td>
<td>Dry</td>
<td>Control</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>165 mm</td>
<td>Dry</td>
<td>Reinforced</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>165 mm</td>
<td>Dry</td>
<td>Reinforced</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>165 mm</td>
<td>Control</td>
<td></td>
</tr>
<tr>
<td>(3) 1</td>
<td>1</td>
<td>200 mm</td>
<td>Saturated</td>
<td>Control</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>200 mm</td>
<td>Saturated</td>
<td>Reinforced</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>150 mm</td>
<td>Saturated</td>
<td>Reinforced</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>200 mm</td>
<td>Saturated</td>
<td>Control</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>200 mm</td>
<td>Saturated</td>
<td>Control</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>150 mm</td>
<td>Saturated</td>
<td>Control</td>
</tr>
<tr>
<td>(4) 1</td>
<td>1</td>
<td>115 mm</td>
<td>Dry</td>
<td>Control</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>115 mm</td>
<td>Dry</td>
<td>Reinforced</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>115 mm</td>
<td>Dry</td>
<td>Reinforced</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>115 mm</td>
<td>Dry</td>
<td>Reinforced</td>
</tr>
</tbody>
</table>

Upon achievement of the first objective (basic comparisons between reinforced and unreinforced), the second objective was to find the equivalent thickness of the reinforced layer. Loops 3 and 4 were designed for this purpose. In loop 3, two unreinforced sections (200 mm and 250 mm) were tested against a thinner reinforced section of 150 mm on weak subgrade. Results of this loop, subsequently described, showed that a value of (50 - 100 mm) equivalent thickness may represent the reinforcement effect.

Based on this finding, loop 4 tests were performed with an unreinforced section of 250 mm and a reinforced section of 200 mm to confirm the minimum saving value (50 mm).

The last loop was designed to compare the vertical stresses on the subgrade (strong subgrade) for reinforced and unreinforced asphalt sections of the same thickness (115 mm).

### Load Applications

Loads were applied through a 300 mm (12 in.) diameter rigid circular plate placed on the pavement surface. The loading pulse was sinusoidal, with an amplitude or peak of 40 kN for each cycle, and a frequency of 10 Hz. The loading program was designed to represent typical traffic loadings on pavements under operating conditions. The cyclic loading was carried out until certain defined criteria for failure, as subsequently described, were reached.

After certain numbers of selected cycles, dynamic loading was discontinued and static loading sequence [5-10 static cycles] applied as a time lengthened, step-wise approximation of one cycle of loading. In addition to obtaining static load response per se, this static loading sequence was necessary for monitoring the array of displacement gauges, strain gauges and strain carriers in each section.

### Test Materials and Construction

The subgrade for each loop consisted of a 1.2 m depth of medium to coarse sand, compacted at an optimum moisture content of 11.5 percent using a Plate tamper. Moisture content and compactive effort were carefully controlled using a Troxler nuclear densitometer. For the "weak" subgrade tests it was flooded to full saturation from below, to the sand-asphalt interface.

The asphalt used was a local Ministry of Transportation and Communications of Ontario grade H4 hot mix. A 25 mm lift (62°F) was first placed on the subgrade for all tests. The mesh was then placed on half the pit, the other half being left unreinforced, as the control section. Next, reinforced and unreinforced halves were covered with one additional 50 mm of asphalt and compacted using four passes of a plate tamper. Additional uniform 25 mm to 75 mm lifts were then placed and compacted.

The Tensar grid used was an ATR type with about 50 mm by 50 mm openings. For loops 1, 2, 3 and 5, strain gauges were bonded to the top and the bottom of the ribs at locations covering a wide area under the loading plate in order to monitor strains.

### Instrumentation

The general arrangement of instrumentation used to monitor the pavement sections during testing is shown in Figure 1. Data access was through a PDP 11/34 computer at pre-programmed intervals (2).

Foil-type (120 ohm) strain gauges were used to record the magnitude and distribution of elastic and plastic tensile strains generated in the reinforcement elements as a result of the loading.

![Figure 1 Schematic Illustration of Test Set Up and Instrumentation](image-url)
Mastic "strain carriers", consisting of two (120 ohm) strain gauges embedded in a 150 mm square by 12 mm thick mastic plate, were used for each test set-up of the last four loops. These strain carriers were placed directly under the centerline of the loading plate and at the subgrade-asphalt interface.

Dial gauges were located on the rigid loading plate and at radial distances. They were read during static load cycles to determine the elastic and plastic surface deflection profile after various numbers of load cycles.

For loop 5, a circular plate pressure cell was embedded in the subgrade directly below the loading area for each test set-up. Depth of burial was about 50 mm below the sand/asphalt interface.

**Failure Criteria**

In order to objectively compare the performance of the reinforced and control sections, certain failure criteria were established. Failure was said to have occurred if:

1. A permanent deformation of 30 mm was measured;
2. The development of extensive cracks, or
3. A steady increase in the measured values of stress, surface deflection and/or horizontal strain at the interface.

It is noteworthy that the failure of the mesh did not have to be considered in the criteria. The reason is that the strains on the mesh on all loops did not exceed 30% of its yield strain of 15%.

**RESULTS OF THE EXPERIMENTS**

A large amount of information was collected. Some of it is summarized in Ref. (3, 4), while Ref. (5) contains a comprehensive documentation. It is only possible herein to present some typical results illustrating the absolute and relative behavior of the reinforced sections. There are many fundamental considerations and implications of this behavior, which are covered in Ref. (5).

Loop 1 was designed to compare reinforced and unreinforced (control) sections at a constant asphalt layer thickness of 150 mm under both strong (dry) subgrade and weak (saturated) subgrade conditions. The results (5) showed that the unreinforced section on a strong subgrade started to fail after 200,000 cycles of the 80 kN load, in terms of increasing static deflection and fatigue cracking. Both sections were carried to 500,000 cycles, with the reinforced showing no signs of failure. As well, the "angle of curvature" (5), a measure of the load spreading capabilities of the reinforced layer, was significantly less for that layer. The foregoing results were confirmed for the saturated or weak subgrade condition, although the absolute number of loads carried were substantially lower.

Loop 2, involving a strong subgrade condition, utilized extensive instrumentation (see Fig. 1). Some of the results are shown in Figures 2 to 5. Figure 2 shows that maximum elastic deflection (under static loading) after various numbers of load cycles is not substantially higher for reinforced as compared to unreinforced. However, the angle of curvature (Fig. 3) is about 50% less for the reinforced section, as is the elastic tensile strain at the bottom of the layer (Fig. 4), both of which have major implications for permanent deformation and fatigue behavior. Figure 5 shows that permanent surface deformation is substantially less for the reinforced section. At the 20 mm failure criterion shown, the unreinforced at best carried only 110,000 loads while the reinforced section carried 320,000 loads, a threefold increase.

Loop 3 was designed to investigate the equivalent thickness effect of reinforcement, under weak subgrade conditions. Figure 6 illustrates some of the results, in terms of permanent deformation. The 150 mm reinforced section carried about 80,000 loads compared to only 34,000 loads for the 200 mm unreinforced and 92,000 loads for the 250 mm unreinforced. In other words, 150 mm of reinforced nearly compares to 250 mm of unreinforced. When the much longer total loading time is taken into account for the reinforced section (because of having to read the instrumentation, under static load conditions, after various numbers of cycles), it actually performed as well or better than the unreinforced section, as shown by Abdelhalim (5).
As well, the unreinforced sections were severely cracked at the end of the test while the reinforced section surface was still quite sound.

Loop 4 was designed to confirm the previous results. The major conclusions were that thickness savings for reinforced sections can range from 50 to 150 mm and that the reinforcement would have a relatively greater effect for weaker subgrade and lower stability mix conditions. Another important conclusion was that maximum elastic surface deflection, a major criterion of behavior used in the pavement field, was inadequate for evaluating reinforced pavements. A much better criterion is angle of curvature.

Loop 5 involved pressure cells in the subgrade under each test section, using a thin asphalt layer (115 mm) and a strong subgrade. Figure 7 shows that the vertical compressive stress was 20 to 40% higher for the unreinforced section, also, the angle of curvature was about 20% higher by the end of the test. Both these results represent an important observation as to the load spreading ability of the reinforcement.

APPLICATION OF ELASTIC LAYER THEORY

The experimental program also provided the opportunity to examine the validity of elastic layer theory under simulated field conditions. Furthermore, it also provided enough information about asphalt pavement responses (elastic deflections, horizontal tensile strains and vertical stresses) under a wide range of variables.

A multi-layer elastic model, BISAR (6) was used to predict surface deflections, horizontal strains and vertical stresses. Table 2 shows that the predicted values under the maximum load (40 kP) were quite comparable to measured values, for the unreinforced sections.

For reinforced sections, the application of the theory is more difficult because of assumptions which must be made on interface conditions, effective thickness of the reinforcement, etc. An example, using a minimum thickness of 2.5 mm for the reinforcement, an effective modulus of 10 GPa (1,500,000 psi), and a Poisson’s ratio of 0.35 plus moduli of 690 MPa (100,000 psi) for the asphalt and 34.5 MPa (5,000 psi) for the subgrade, is given in Figure 8. It shows the vertical compressive strain reduction on the subgrade ranging from a maximum of about 40% for a 600 mm asphalt layer to about 10% or less when a thicker (300 mm or greater) layer is used. This compares with the measured reductions in vertical compressive stress on the subgrade (see Figure 7).

SOME FIELD APPLICATIONS

The use of Tensar geogrid in field applications is being carried out in three areas: (a) regular paving operations, where the Tensar is in the asphalt layer, (b) installation of Tensar at some other point in the road structure, i.e., at the subgrade-base interface, and (c) pavement spot improvements and trench cut repairs where the placement is by hand. Construction technology is being developed for the first area. The second area has seen some field trials, primarily where soft subgrade soils such as peat are involved. The third area was seen as an opportunity to obtain some early field installations and observations of behavior.

<table>
<thead>
<tr>
<th>Thickness (mm)</th>
<th>Asphalt modulus (MPa)</th>
<th>Subgrade modulus (MPa)</th>
<th>Measured Deflection (mm)</th>
<th>Tensile strain (µε)</th>
<th>Predicted Deflection (mm)</th>
<th>Tensile strain (µε)</th>
<th>% error in Deflection</th>
<th>% error in Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>115</td>
<td>2997</td>
<td>33.3</td>
<td>1.57</td>
<td>605</td>
<td>1.54</td>
<td>569</td>
<td>2.0</td>
<td>3.0</td>
</tr>
<tr>
<td>165</td>
<td>1399</td>
<td>13.3</td>
<td>1.81</td>
<td>680</td>
<td>1.80</td>
<td>715</td>
<td>1.0</td>
<td>5.0</td>
</tr>
<tr>
<td>200</td>
<td>965</td>
<td>11.6</td>
<td>1.98</td>
<td>845</td>
<td>2.05</td>
<td>769</td>
<td>4.0</td>
<td>5.0</td>
</tr>
<tr>
<td>250</td>
<td>1233</td>
<td>15.2</td>
<td>1.69</td>
<td>440</td>
<td>1.76</td>
<td>632</td>
<td>1.0</td>
<td>3.0</td>
</tr>
</tbody>
</table>

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Consequently, a program of such pavement spot improvements and trench cut repairs was undertaken in 1983, with an approximately 20 installations being completed. These have been more fully reported elsewhere (7, 8) and are only briefly summarized as follows. Basically, the spot improvements involved removal of the deteriorated asphalt, and any underlying material if required, followed by preparation and levelling of the affected area, hand placement of the Tensar (after cutting to size), hand placement of the asphalt and finally compaction with conventional rollers. Trench cuts involved similar procedures, with more than one layer of Tensar used where required by depth and backfill soil conditions.

Figures 9 and 10 show a typical pavement spot improvement area with Tensar. In first photo, the deteriorated surface has been removed and the Tensar is being trimmed to size. The second photo shows how the hot mix is hand placed over the grid. This was followed by conventional rolling.

Figure 9: Typical pavement spot improvement area, with Tensar being trimmed to size.

Figure 10: Hand spreading of the asphalt hot mix over the Tensar.

Because the spot improvements and trench cuts involve relatively softer areas, reinforcement can be a particularly useful approach to gaining more uniformity in support conditions. This was verified by a limited amount of dynamic deflection testing (using a Dynaflect device) which showed reductions usually in the order of 30% in surface deflection when Tensar was used.

These field applications were felt to be quite successful. Field crews found it easy to work with the material. The only caution is that the asphalt mix should not be over about 140°C; otherwise the Tensar will distort. Observations of performance will continue in 1984 and beyond.

CONCLUSIONS

The research described in this paper has shown that flexible pavements can be effectively reinforced with the polymer geogrid, Tensar. This involves asphalt thickness savings ranging from 50 mm to 100 mm, or the ability to carry two to three times more traffic loads for equal thicknesses.

The material can also be very effectively used in pavement spot improvements and trench cut repairs.

ACKNOWLEDGMENTS

This paper is based largely on the work of Dr. A.O. Abdelhalim. His contributions are gratefully acknowledged, as are those of many others such as Professors Jarrett and Bathurst of Royal Military College, Mr. William Phang of the Ontario Ministry of Transportation and Communications, Mr. Jamie Walls and Ms. Louise Steel of the PMS Group and many colleagues at Gulf Canada Ltd. and the Tensar Corporation.

REFERENCES


Construction of Tensar reinforced asphalt pavements

G. J. A. Kennepoth, The Tensar Corporation, and N. I. Kamel, Gulf Canada Ltd

BACKGROUND

There has been no lack of attempts to reinforce asphaltic concrete in layered pavement systems to assure longevity of satisfactory pavement performance. Asphaltic concrete exhibits an ultimate tensile strength which is usually less than the ultimate compressive strength by an order of magnitude and, therefore, reinforcement is expected to yield several important advantages, such as:

- increased tensile strength
- greater resistance to cracking
- longer fatigue life
- cohesion of pavement after cracking
- improved shearing resistance due to lateral restraint
- reduced cost from material savings

Previous attempts to improve the tensile strength of bituminous concrete included the incorporation of natural and synthetic fibres and fabrics - in either random or aligned orientation - as well as steel welded wire mesh.

Most field trials with fibres, starting from the addition of cotton fibre in the 1930's (1) to the use of current pavement fabrics (2), have experienced modest or unqualified success owing mostly to low tensile strength and high creep under load of the fibre.

Welded wire or expanded steel mesh, normally associated with reinforcement of Portland cement pavements, have been extensively field tested on air field runways and highway pavements in 1950's and 1960's. Following the construction of test sections in the U.S.A. (California, Illinois, Massachusetts, Michigan, Minnesota, New Jersey, New York, Pennsylvania, Texas, Wisconsin) and Canada, (Ontario), the use of steel mesh in asphalt concrete was discontinued, in spite reports claiming significant reflection cracking reduction.

While factors such as corrosion and cost of steel had an influence, the lack of success is primarily attributed to construction problems. According to Davis (3) the steel mesh had a tendency to curl at the edges, and irregularities in the mesh produced bulges and caused pavement failure. Transverse cracks of the mesh splices were caused by expansion of the wire under the hot mix, some creep of the mesh occurred in front of the paver allowing significant bulges to develop and the mesh was often springy during installation and compaction. In summary, experience with wire mesh paving has shown that the mesh must be kept absolutely flat on the underlying road, otherwise bulging and springiness under rolling develops and later cause break-up of the pavement.

With the advent of Tensar geogrid, a new candidate for pavement reinforcement is now available with a more suitable set of material properties. The objective of this paper is to describe in subsequent sections geogrid property requirements for performance as well as for construction, types of reinforced pavement structures and the development of geogrid installation procedures (Figure 1).
KENNEPOHL AND KAMEL

GEORGRID PROPERTY REQUIREMENTS

A comparison of suggested, property requirements with the properties of various reinforcement materials presented in Table 1 shows that Tensar geogrid ARI meets the desired criteria best. The key features are high tensile strength, high modulus, and a compatible coefficient of expansion. Specific tensile strength is defined as tensile strength/particle gravity.

<table>
<thead>
<tr>
<th>Property</th>
<th>Desired</th>
<th>Steel Mesh</th>
<th>Glass Fibre</th>
<th>Polypropylene</th>
<th>Tensor Geogrid</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spec. Tensile Strength</td>
<td>25</td>
<td>45</td>
<td>500</td>
<td>38</td>
<td>225</td>
</tr>
<tr>
<td>Strength (MPa)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spec. Elastic Modulus</td>
<td>26</td>
<td>26</td>
<td>29</td>
<td>1.5</td>
<td>12</td>
</tr>
<tr>
<td>(GPa)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Coefficient of</td>
<td>5-7</td>
<td>0.12</td>
<td>0.2</td>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td>expansion ((\times10^{-5})/°C)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The geogrid ARI has been specifically designed for asphalt pavement reinforcement. The biaxial open mesh structure provides for bond and interlock with the matrix. Alternate structure designs can be made easily if required for optimization of performance. The geogrid is manufactured from polypropylene polymer for high thermal stability in contact with hot paving mix.

In addition, a heat soaking process is applied to reduce thermal shrinkage to a maximum of 3% at 140°F. Excessive shrinkage during placement of the hot mix can result in movements of the mat and build-in cracks.

The extent of shrinkage during installation and cooling of the mat is not only a function of the initial mix temperature, but also the cooling rate of the mat and the degree of restraint due to aggregate interlock, friction, and weight. Figure 2 illustrates the synergic effects on shrinkage from a pilot test simulating various temperature, mix types and mat thickness conditions.

The effective shrinkage under field conditions was studied in several off-road, full-scale experiments. Shrinkage and associated edge cracking were not observed in test sections with heat stabilized ARI geogrid for specified temperature and laying procedure.

Shrinkage force measurements reported by Brown (5) show that on cooling some tension is retained. Therefore, under controlled conditions the geogrid can possess a useful pretensioning force in hot-laid asphalt pavements.

TYPES OF REINFORCED PAVEMENT STRUCTURES

There are several options or types of structures for reinforcing layered, flexible pavement systems which will determine the applicable construction procedure. Depending on the design criteria, environmental subgrade and paving material condition, the geogrid may be placed in the surface, base or subgrade layers.

The most common geogrid applications can be represented by the following three types of reinforced pavement structures:

1. Asphalt concrete in new construction
2. Asphalt concrete overlays
3. Gravel base or subgrade

In full-depth pavements, the geogrid is placed at the bottom of the asphalt concrete slab for largest reduction in tensile strain as reported by Abdelhalim (3). If such pavement is placed on unbound base or subgrade, the geogrid may be installed at the interface ensuring adequate interlock with the hot mix. However, in case of a very hard surface, such as an old pavement, the geogrid may have to be sandwiched between the first and second lifts to provide sufficient interlock. For the reduction of permanent deformation and rutting, the geogrid is placed near the centre of the slab. The postulated mechanism as reported by Brown (4) for this behaviour of Tensar geogrid is the reduction of plastic flow of the asphalt mix away from the stressed area.

For overlaid pavements the preferred construction procedure is the placement of the geogrid between a levelling course and the surface lift. This method of construction will give a better interlock with the asphalt concrete and also reduce possibility of debonding. If the geogrid is placed directly upon the aged asphalt surface or alternately upon a Portland concrete surface, a tack coat should be applied. Further field trials are needed to investigate long term performance. It may be necessary to improve the bond with the old pavement by increasing the contact area using a geogrid with a larger mesh opening. Further information is also required for a proper strength and thickness design of the overlay to avoid excessive shearing which could result in delamination.

The reinforcement of unbound base or subgrade does not require special installation techniques. The geogrid can be laid by hand as in established geotechnical applications, such as unpaved roads.

In pavement spot repairs the reinforcement with geogrids is somewhat easier to apply because the construction procedure, including the placement of the paving mix, is done manually. Therefore, it is appropriate and feasible to install the Tensar geogrid also manually.
INSTALLATION METHODS

Pavement Spot Repair

Geogrids have been successfully used in pavement maintenance for repairing of spot distresses on asphalt and Portland cement pavements and in utility trench cuts. While the utility trench cuts apply to roads and streets, the repair of spot distresses extends to highways, parking lots, industrial yards, etc. Installations of Tensor geogrid at more than 20 job sites indicated the following major benefits:

- improved load bearing characteristics
- greatly reduced differential settlement
- increased pavement life from reduced deflection of up to 30%

The installation of the geogrid involves no changes in construction procedure. It can be done with conventional equipment and materials. The geogrid is simply cut with tin snips to the required size and hand laid. Based on experience from various types of field applications an installation manual has been developed for the repair of a variety of localized pavement distress conditions including potholes, ruts, alligator cracking and base distortions. There are three types of repairs where it is advantageous to include geogrids:

1. **Surface (Skin) Patch**
   Hand placed overlay (patching mix) is spread directly on the distressed area of the pavement. A piece of ARI approximately cut to the size of the patch area is placed at the bottom for reinforcement.

2. **Replacement (Deep) Patch**
   The cracked pavement and in some cases also the base is removed as deep as is necessary to reach firm support. In repairing SS1 or SS2 (for DTN > 100) is placed in the granular material and ARI in the asphalt mix for reinforcement of the patch.

3. **Trench (Utility) Cuts**
   When pavement and subgrade is removed to install or repair a utility, Tensor geogrid can be used effectively in the backfill and patch as shown in Figure 3.

Placement of Tensor SS1 on the bottom of the excavation with the edges curled up on the sides of the trench will provide a firm work platform, support for the pipe or conduit, and reduce contamination of granular backfill (e.g. all or clay). Geogrid placed over the pipe or conduit will protect the utility from damage by large aggregates and reduce the stress on the pipe. Where appropriate, the backfill may be reinforced with additional layers of geogrid. This will reduce differential settlement, lateral pressure against pavement base structure and associated damage during compaction.

The pavement spot repair with geogrid is now being introduced to the paving industry.

The use of geogrids in spot repair applications will, no doubt, help to acquaint the highway and paving engineer with the potential uses and benefits of geogrids in pavement structures, construction and rehabilitation projects alike. However, for full-scale paving projects manual installation of geogrid is not economical and specialized geogrid laydown equipment must be used.

**Tensioning Method**

Tensor laydown equipment has been developed for tensioning of the geogrid flat against the base before placement of the paving mix by the finisher. This technique is applied to paving with geogrid in new construction or overlay applications which involve the installation of large volumes of geogrid at regular paving speeds. To-date, six full-scale field trials have been carried out in the United Kingdom and Canada (Table 2), as well as several off-road test sections.

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>DATE</th>
<th>TYPE OF CONSTRUCTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Highway 25, Ontario, Canada</td>
<td>November/81</td>
<td>overlay on asphalt</td>
</tr>
<tr>
<td>Conwy Island, UK</td>
<td>Spring/82</td>
<td>reconstruction over PCC (rubber laydown)</td>
</tr>
<tr>
<td>Carreysway A 631, Nottinghamshire, UK</td>
<td>Fall/82</td>
<td>overlay on PCC</td>
</tr>
<tr>
<td>Motorway M 621, West Yorkshire, UK</td>
<td>Fall/82</td>
<td>reconstruction/overlaying (under base course)</td>
</tr>
<tr>
<td>Highway #6, Sault Ste Marie, Canada</td>
<td>July/83</td>
<td>cold-mix surfacing</td>
</tr>
<tr>
<td>Sudbury, Ontario, Canada</td>
<td>November/83</td>
<td>overlay on asphalt (instrumented)</td>
</tr>
</tbody>
</table>

The tensioning of the geogrid proved to be quite a critical feature of the installation. Similar to the experience reported for paving with steel mesh (3), it was found that the geogrid must be held absolutely flat against the underlying base in order to attain proper compaction. The applied tension force must not only flatten the geogrid, but also overcome the shearing forces acting on the geogrid by the wheels or crawlers of the finisher. The required tension is in the order of 2.5-3.0 kN/m depending on such variables as the type of surface, paving mix fluidity and thickness, paving train wheels (or crawlers) and weight, including size and load on dump truck.
Because the high tensile strength polymer geogrid is more pliable than steel, it can be handled easier than steel mesh. Any slackness in the geogrid that might develop by the action of the paving equipment or minor relaxation of the geogrid can be picked up by the continuous tensioning equipment which moves immediately ahead of the paving train.

The first prototype tensioning equipment, shown in Figure 4 was designed and constructed by K.C. Welding Corporation, Bartlesville, Oklahoma. It consists of a 4 metre wide steel frame with a dispensing hanger for a roll of Tensar geogrid and two rubber-sleeved steel drums. These drums function as slipping rollers and release the geogrid as the equipment moves ahead. The lower roller is piloted and maintained at a preset tension by a disc brake at each end.

The laydown equipment can be used as an attachment to a front-end loader or tractor with a 3 point hitch. For shipment from job site to job site it is mounted on a small skid (1.2 x 3.7 m) and transported on a truck or trailer.

Several test sections have been constructed successfully using the tensioning method (Figure 5). However, further experience is still required to develop and perfect the operation and construction procedures for full-scale applications. The major advantages of this method includes:

- Minimum interference with normal paving operations
- Low installation cost, i.e., 2 unskilled laborers plus about $5,000 for the laydown equipment
- In-place pretensioning of reinforcement mesh and elimination of shrinkage in longitudinal direction.

When the geogrid is placed at the interface of the base and the asphalt concrete mat, the interlock with the mix may be weak, especially where the base is solid and hard. Also, on hard surfaces some damage may be incurred to the tensioned geogrid by heavy paving equipment. It is mainly for these reasons that an alternate installation, the roll-in method is being studied.

**Fig 5** Paving over geogrid which is being held in tension by the laydown equipment ahead of the paving train.

**Roll-in Method**

Rolling the Tensar geogrid into the hot mix immediately after the paver and prior to compaction has been investigated as an alternate method to incorporate the geogrid into asphalt pavement. The task to push the mesh into the mat appeared initially quite difficult because the mix is not fluid like Portland concrete mix and any load exerted onto the mesh is distributed in a snow-shoe like manner.

Initial experiments with a first generation roll-in equipment consisting of a set of steel discs and a 1 metre wide vibrator screed have been encouraging. A second 4 metre wide prototype is shown in Figure 6. It features a holder for a roll of geogrid, a set of steel discs and immediately following an adjustable vibrator screed with weights. The vibrators are connected by hose to and driven by the hydraulic system of the finisher.

**Fig 6** Prototype geogrid roll-in equipment pulled behind a paver.
The weight of the roll-in equipment and the resulting drag on the paver proved to be a limiting factor. Further experiments and development of this equipment is still required. Major advantages expected from the roll-in method include:

- Improved interlock of the geogrid with the mix
- No extra vehicle needed for installation
- No damage to geogrid by paving equipment or overtensioning

**RECYCLING OF TENSAR REINFORCED PAVEMENTS**

As the recycling of old and worn asphalt pavements becomes more and more the accepted procedure for rehabilitation and reconstruction of pavements in North America, it is important to establish (a) whether the reinforced asphalt pavement can be recycled and (b) if the cost of recycling is increased because of the reinforcement. In a cost benefit analysis of geogrid a significant demerit would be assessed if the geogrid reinforced pavement had no recycling value.

The first field trial of rotomilling a Tensar reinforced pavement took place in October 1983 at the Nelson Quarry, Burlington, Ontario (Figure 7). Several test sections constructed there previously with Tensar ARR were milled successfully to a depth of 5 cm with a Goetz "Scrapitplane" rotomill. At normal milling speeds the Tensar geogrid was cut into pieces of about 3 or 4 mesh units. The milling drum became wrapped in pieces of Tensar geogrid, but did not appear to affect the operation.

**REFERENCES**


Also


About ten tons of the rotomilled asphalt pavement has been stockpiled at the quarry for a future plant-scale recycle trial. Before recycling it will be necessary to remove by screening the geogrid pieces from the rotomilled product. Separate laboratory experiments indicate that geogrid pieces smaller than 2 cm do not affect the quality of the asphalt paving mix. While the evaluation is not yet complete, it has already been established that Tensar reinforced pavements can be recycled without any significant increase in operating cost.

**CONCLUSIONS**

Significant progress has been made to develop the required construction technology for asphalt pavement reinforcement after the viability, cost and performance benefits have been established over the past 3 years. The main conclusions of this development as well as points for future activities are summarized here.

1. Tensar geogrid has distinct advantages over other reinforcement materials with regard to required construction and performance properties.

2. The installation and construction procedure varies with the type of reinforced pavement structure.

3. Manual installation of geogrid can be used for pavement spot repair and reinforcement of unbound granular base and subgrade.

4. Automated laydown equipment and specific installation technology has been developed for the reinforcement of asphalt concrete.

5. Reinforced asphalt pavements can be recycled.

6. Field data must be obtained to support and confirm laboratory results and to develop application design procedures.
Construction of a reinforced asphalt mix overlay at Canvey Island, Essex

This paper describes the design, specification and construction of the first application in April 1981 and a subsequent application in 1982, of a reinforced asphalt or bituminous material overlay using Nation geogrids.

The reinforced asphalt mix overlay was designed to strengthen a severely damaged concrete road pavement over a very weak saturated estuarine silt and clay subgrade. Polymer grid reinforcement being specified to increase tensile strength of the thick asphalt mix overlay and deter propagation of reflective cracking from the underlying cracked concrete layer.

G. R. Pooley, Mobil Oil Company Ltd

CONSTRUCTION OF A REINFORCED ASPHALT MIX OVERLAY AT CANVEY ISLAND, ESSEX

INTRODUCTION

Canvey Island is a primarily residential area but has located on the island extensive gas and petroleum product storage facilities. The site of the application of the geogrid was the B1014 road known as Somers Avenue. This road is one of only two roads connecting the island with the mainland, and consequently, besides providing a major route for every day traffic, would also play a vital role in the event of a large scale emergency.

The road which is of two lane construction was originally built in 1972 of reinforced concrete, 150 mm thick, on 175 mm of ash sub-base, constructed over considerable depths of estuarine silt and clay with a very high water table. By 1978, after carrying less than one million standard axles (msa), the concrete had suffered extensive damage over its 1.5 km length requiring constant maintenance. The necessary maintenance repair cost was estimated at £576,000 in 1981.

Various strengthening and reconstruction options were considered, but following the design recommendations of the author a thick asphalt mix overlay incorporating Nation geogrids was selected for the first phase of restrengthening in 1981. With the success of this phase a second phase was undertaken in 1982, at a total cost for the two phases of only approximately £220,000.

PAVEMENT AND ASPHALT MIX DESIGN

Pavement Design Options

Soil tests on the very weak saturated subgrade soil indicated a California Bearing Ratio (OCR), generally lower than 2% and in many cases less than 1%. For the pavement design the following design criteria were stated by the highway authority.

Design life, 20 years
Commercial vehicles in each direction in 1981, 433
Annual traffic growth rate, 3%.

Initially various reconstruction alternatives involving the removal of the existing reinforced concrete and ash sub-base pavement were considered. These may be summarised as follows.

<table>
<thead>
<tr>
<th>Pavement Type</th>
<th>Layer Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Flexible bituminous on granular sub-base</td>
<td>205 mm bituminous 650 mm sub-base</td>
</tr>
<tr>
<td>2. Reinforced concrete slab on granular sub-base</td>
<td>250 mm concrete 410 mm sub-base</td>
</tr>
<tr>
<td>3. Full depth asphalt directly on to subgrade</td>
<td>420 mm bituminous</td>
</tr>
</tbody>
</table>

In addition overlays of flexible polymer modified bituminous materials and a 175 mm thick concrete overlay to the existing damaged concrete pavement were considered. However, the uncertain performance of such overlays, and the cost, inconvenience and impracticability of completely reconstructing the damaged concrete pavement led to the selection of a thick asphalt mix overlay incorporating Nation geogrids.

As part of the construction process it was decided to break up what was remaining of the existing concrete slabs to form a 'flexible sub-base', and to prevent any stress concentrations, thus reducing the possibility of reflective cracking through the overlay.

The thickness of the asphalt mix overlay was selected following pavement analyses using the Nottingham University analytical computer.

design programmes. Any improvement in tensile strength due to the incorporation of the geogrids was not included in the pavement analysis. The overlay design consists of a 210 mm thick basecourse layer of gravel gap graded rolled asphalt with 65% coarse aggregate content, topped with a 40 mm thick rolled asphalt wearing course as the surfacing, giving a total overlay thickness of 250 mm of asphalt mix.

Asphalt Mix Design

To achieve both high fatigue resistance and low permanent deformation the rolled asphalt mixes were designed using the total mix Marshall method. Based on the pavement design analyses calculated values for voids, binder content, and quotient and stability were set to achieve the required flexible but deformation resistant performance. The gap grading being selected in preference to a continuous aggregate grading because of its greater fatigue resistance. The mix design criteria together with the actual material properties achieved are summarised below.

<table>
<thead>
<tr>
<th>Wearing Course Surfacings</th>
<th>30% coarse aggregate content</th>
</tr>
</thead>
<tbody>
<tr>
<td>14 mm nominal size, 50 pen bitumen</td>
<td>Design Properties</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Mix Property</th>
<th>Bitumen Content %</th>
<th>Mix Void, %</th>
<th>VMA, %</th>
<th>Minimum VMA, %</th>
<th>Maximum Stress, KN</th>
<th>Minimum Quotient KN/mm</th>
<th>Lab Density g/cc</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design</td>
<td>7.0-8.2</td>
<td>3.5</td>
<td>14.5</td>
<td>19.3</td>
<td>3.0</td>
<td>1.0</td>
<td>-</td>
</tr>
<tr>
<td>Actual</td>
<td>7.0</td>
<td>3.5</td>
<td>19.3</td>
<td></td>
<td></td>
<td>1.3</td>
<td>2.316</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Basecourse</th>
<th>65% coarse aggregate content</th>
</tr>
</thead>
<tbody>
<tr>
<td>20 mm nominal size, 50 pen bitumen</td>
<td>Design Properties</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Mix Property</th>
<th>Bitumen Content %</th>
<th>Mix Void, %</th>
<th>VMA, %</th>
<th>Minimum VMA, %</th>
<th>Maximum Stress, KN</th>
<th>Minimum Quotient KN/mm</th>
<th>Lab Density g/cc</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design</td>
<td>4.9-5.8</td>
<td>5-10</td>
<td>16.4</td>
<td>16.4</td>
<td>4.5</td>
<td>1.25</td>
<td>-</td>
</tr>
<tr>
<td>Actual</td>
<td>5.0</td>
<td>5.8</td>
<td>5.8</td>
<td>2.1</td>
<td>2.324</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Geogrid Reinforcement**

To increase the tensile strength of the overlay and deter propagation of reflective cracks from the concrete layer, Petlon geogrids made from polypropylene were included as horizontal reinforcement near the base of the overlay.

Tensile strength tests were undertaken on single lengths of Tensar AK geogrid using a Houndsfield Tensometer at a loading rate of 3mm/min. These tests being of single lengths of Tensar only, took no account of the important grid effect when in service. However, they did confirm the considerable potential tensile strength of the geogrid as an asphalt reinforcement. The results obtained are summarised below.

Failure mode - splitting across the mesh joint node.
Load at failure - 0.99 to 1.09 KN
Approx. stress at failure - 150 to 200 MPa
Strain at failure - 16 to 24%

**PAVEMENT CONSTRUCTION**

The construction of phase I commenced in April 1981 and was planned so that only one lane of the road was closed at any time. Construction was undertaken in the following stages.

1. Break existing concrete.
2. Lay first layer of asphalt mix.
3. Place geogrid.
4. Lay second layer of asphalt mix.
5. Install new road edge kerbs.
6. Complete laying of asphalt mix.

During the construction of phase I a number of practical problems were experienced and overcome, and the experience gained was used to develop the procedure for phase II. The most important aspects of the experience gained are discussed in the following sections.

**Breaking of Existing Concrete Pavement**

The design intention was to break the concrete slabs into a system of interconnected flexible blocks of approximately 300 to 400 mm in size. By this means it was hoped to achieve a firm foundation for the reinforced asphalt overlay without the problems of stress concentrations and subsequent reflective cracking through the asphalt layers. Fracture of the concrete was to be achieved by use of a dead weight drop hammer falling onto the concrete slabs in a regular pattern. Upon completion of the fracturing of the slabs a thin layer of small size graded stone was to be spread over the concrete and rolled as a binding and regulating layer prior to laying the asphalt mix over. However, experience showed that this procedure was unsuitable because of the following problems.
1. A spade pointed tip was provided with the drop hammer instead of the club foot tip anticipated. This pointed tip tended to penetrate the concrete slab rather than cause cracking of the surrounding area.

2. A regular pattern of hammer drops was proposed, but the concrete proved to be of variable strength. In some areas the hammer tip fully penetrated the slab and in others several drops were required to break the concrete.

3. Some areas of the concrete slab proved to be doubly reinforced with steel reinforcement also in the top of the slab. Where the slab surface was badly broken by the hammer tip some of the steel reinforcing had to be cut away.

To overcome these problems and produce the fractured pattern required the following procedure was recommended.

A club foot hammer tip was used to weaken the concrete slabs in a suitable pattern determined by the concrete strength and the operators experience. By this method the slab was only weakened and the required complete fracturing of the slab into interconnected blocks was achieved by subsequently rolling the slab with a heavy 20 T9 dead weight roller. This caused the weakened slab to bend and crack without extensive shattering of the surface. It was found using this method that no stone blinding and regulating layer was necessary and a firm but flexible foundation was achieved.

Any particularly weak areas of concrete that did shatter extensively or had suffered previous damage making them unsuitable for overlay strengthening were removed and replaced with approximately 150 mm of compacted asphalt mix base. To reduce penetration of this asphalt base into the very weak subgrade a layer of Netlon GE 131 geogrid was placed on the subgrade prior to laying the asphalt mix. The alternative to the use of this full depth asphalt replacement would have required excavation and placing of 850 mm of crushed rock material as required by conventional procedures.

Achieving the Geogrid - Pavement Bond

The importance of obtaining a good bond between the asphalt pavement and the geogrid, to achieve both pavement strength and the required tensile reinforcement, was recognised throughout. In the original design concept it was proposed to achieve this by penetration of the asphalt mix through the geogrid by rolling the geogrid into the surface of the first layer of asphalt mix. This first layer was to be hand laid with no compaction so that it would allow penetration of the geogrid. However, again problems occurred and amendments to the proposed procedure were necessary.

1. At the start of Phase 1 the specified Tenax A11 with its rectangular mesh size of 30 mm x 65 mm was not available, and it was necessary to use Netlon GE 131 with its circular mesh size of only 27.5 mm. The nominal size of the asphalt mix was 20 mm and insufficient mix penetration was achieved causing poor bond at the geogrid layer with the Netlon GE 131.

2. The original intention was to lay a loose layer of asphalt mix 40 mm thick, either by hand or without use of the paver screed tamper and compaction. Then place the geogrid on the surface of this loose layer and roll it into the asphalt mix. This would then be followed by a second layer of asphalt mix 60 mm thick laid by paver and compacted normally. Both methods of laying the first loose layer were attempted but due to inadequate asphalt mix thickness, distortion and movement of the geogrid during rolling into the surface, little satisfactory penetration of the geogrid was achieved. A method of 'sandwiching' the geogrid between two layers of asphalt mix was subsequently developed.

To achieve a satisfactory bond using the Tenax A11 geogrid reinforcing the following procedure was adopted on the basis of these experiences.

A 50 mm thick layer of asphalt mix was laid and compacted using a conventional paver and roller. Onto the surface of this layer the geogrid was unrolled and a 50 mm thick layer of asphalt mix was hand spread over the geogrid. The sandwich of asphalt mix and geogrid was then completely compacted to give the required pavement strength and bond. Satisfactory bond being proven by the cores cut from the completed pavement. Following the sandwich layers the remaining layers of 150 mm of asphalt mix were placed by conventional paver methods.

Placing the Geogrid

Because of the design intention to roll the geogrid into the surface of a loose layer of asphalt mix and the subsequently developed sandwiching procedure, it was felt unnecessary to fix the geogrid to the asphalt mix layer in any way. However, various methods of nailing the geogrid to a compacted layer of asphalt mix were attempted but due to geogrid distortion these were not very successful and were abandoned as unnecessary. However, a number of problems were experienced mainly because of the distortion of the geogrid, both during manufacture and when placing on site.

1. Phase I of the project commenced before the manufacturing plant producing Tenax A11 was in production, and the first rolls of Tenax subsequently received were manufactured before the commissioning of the new plant was complete. As a consequence some of the geogrids used were distorted not only in their mesh pattern...
and size but in the roll shape itself. This caused two problems directly related to the distortion.

a. the geogrids did not lay flat on the surface of the asphalt mix and in some cases penetrated the top of the second asphalt mix layer.

b. because of the distortion of the rolls, ensuring continuous overlapping of adjacent rolls of geogrid was difficult, and the overlap width was not constant.

Later non-distorted production runs of the Tensar solved both these problems.

2. During the compaction of the second layer of asphalt mix it was found necessary to roll the asphalt mix in one direction only, and particularly not to change direction of the rolls on the asphalt layers until they had stabilised. This was not done, rolling in two directions caused the geogrid to stretch in opposite directions with severe distortion and pushing of the geogrid through the surface of the asphalt layers occuring. Again this problem was particularly apparent if the geogrid was already distorted during manufacture.

3. The original rolls of Tensar ARI were manufactured with a thick edge strip approximately 30 mm in width. To make it easier to place the geogrid flat it was necessary to remove this edge strip before placing the geogrid on the asphalt mix.

To place the geogrid on the first asphalt layer and hold it flat, with the appropriate overlap of 150 mm between adjacent rolls, the following procedure was adopted which does not require any method of fixing or tensioning the geogrid other than hand placing of the second asphalt layer. This method also allows the geogrid free movement and avoids any localised tensioning of the geogrid.

Place by paver and compact asphalt mix layer 50 mm thick. Unroll geogrid onto surface of first asphalt mix layer, closely followed by the second asphalt mix layer which is discharged from the rear of the delivery vehicle and spread by hand. Heaps of asphalt mix being used to hold the geogrid flat as it is unrolled further, but not restraining, thus avoiding any distortion as the second asphalt mix layer, 50 mm thick, is spread over the geogrid. Roll the sandwich of asphalt mix and geogrid, avoiding change of direction of the rolls on the fresh asphalt mix until the sandwich has stabilised.

Pavement Compaction

To ensure satisfactory compaction of the asphalt mix layers was achieved the contractor was required to undertake a compaction trial area, 9.0 m x 3.5 m, including the proposed method of sandwiching the geogrid. Compaction achieved being measured by the density of cores cut from the trial area. Percentage requirements of not less than 95% of the Laboratory Design Mix Density (LDM), were required in the trial areas, and from these trials not less than 92% of the Job Standard Mix Density was required for the actual pavement overlay. Results from the cores showed that no problems of achieving satisfactory compaction were experienced with either the use of Nation CR 131 or Tensar ARI geogrids. The following core density results were achieved for the basecourse asphalt mix.

Laboratory Design Mix Density, (LDM) = 2.3240
Trial Area Density = 2.2550, (92% of LDM)
Overlay Densities = 2.2213 to 2.3146, (96 to 99% of LDM)

In order to avoid damaging the polypropylene polymer used to manufacture the geogrids the maximum temperature of the asphalt mix was restricted to 130°C when in contact with the geogrid. The core density results quoted demonstrate that this restriction caused no problems in achieving satisfactory asphalt mix compaction. Temperatures of generally 130°C are considered a suitable maximum based on this experience. With the development of new geogrid polymers specially created to remain heat stable this maximum temperature may be increased if considered necessary to achieve satisfactory compaction with less workable asphalt mixes.

Pavement Performance

During construction of the overlay it was found necessary to open one section to traffic before the basecourse was completed to the specified thickness of 210 mm. Only 100 mm of asphalt mix incorporating the geogrid had been placed when the section was opened to traffic for two days, resulting in some extensive damage, visible as cracking and crazing of the surface of the asphalt mix. It was decided to leave this section in place and continue construction of the overlay as a comparative section for considering subsequent performance of the overlay. To date this 'weaker' section of overlay is performing totally satisfactorily.

A further small section of overlay in part of phase I was also constructed without benefit of the geogrid reinforcement. Again to date this section is performing as well as adjacent sections, but this may be a consequence of the greater foundation strength in the area of this unreinforced section.

To assess the performance of the overlay design, pavement deflection readings using a deflectograph vehicle were undertaken in October 1981. Average deflections of 46.5 10⁻² mm at 14°C were recorded. These indicate a life expectancy of 5 to 8 million standard axles, (mae), compared to the original design requirement of 4 mae.
POOLEY

Having now carried up to 0.7 and all sections of the overlay are performing totally satisfactorily. Performance monitoring is being undertaken which confirms the suitability of the overlay design and the construction methods adopted.

CONCLUSIONS

A thick asphalt mix overlay incorporating Netlon geogrids to strengthen a severely damaged concrete pavement on a very weak subgrade has performed satisfactorily since 1981. A total of approximately 1,500 m² of Netlon CE 131 and 10,500 m² of Tensar AK1 being employed to reinforce the overlay which was completed in two phases, the second phase being undertaken in 1982.

Deflectograph measurements taken on the completed pavement overlay confirm that the predicted life of the strengthened pavement exceeds the design life by between 20 to 100%.

A method of sandwiching the geogrid between two asphalt mix layers, with the second layer laid by hand has been developed and used successfully. Holding the geogrid in place with heaps of asphalt mix allows the geogrid to take up its required placement without distortion or the need for elaborate fixing or pre-tensioning techniques.

Because of its larger aperture size Tensar AK1 is to be preferred to Netlon CE 131 to ensure that satisfactory bond with the asphalt mix layers is achieved. No problems in achieving satisfactory compaction of the asphalt mix with conventional plant were experienced even though a maximum asphalt mix temperature in contact with the geogrid of 130°C was specified.

Cost of supplying and placing the geogrid was approximately £1.20 per sq. m. The additional cost, including hand laying as necessary, as a proportion of the pavement cost depends upon the pavement thickness but is probably between 5 and 15%. This additional cost will obviously reduce as more experience is gained, larger applications are undertaken, and mechanical placing methods are developed. However, even with these additional costs the use of geogrids offers considerable financial advantages as demonstrated by the project described in this paper where the total restrengthening overlay cost was 50% lower than the alternative estimate for maintenance repairs alone.

In addition geogrids offer the possibility of technical solutions where previously no solutions existed. The use of geogrids, used sensible as an option within a pavement and asphalt mix design system that can take account of the beneficial effects of these geogrids, offers considerable potential to the design engineer in the future.
Pavement reinforcement: report on discussion

T. L. H. Oliver, Netlon Ltd

The discussion session was preceded by a contribution from the floor by Dr Jiamnejad of Sunderland Polytechnic Civil Engineering Department. The department has recently initiated a research programme to investigate the behaviour of three-dimensional grids for road base construction. The grids comprise 200mm deep vertical walls welded together intermittently. The shape of the cells is a function of the tension applied prior to filling with sand. The grid may be expanded up to 40 times the folded thickness.

Using a 1.5 cubic metre test pit, plate bearing tests have been conducted on layers of reinforced and unreinforced sand. Some plate bearing testing of block paving, with and without reinforcement of the sand road base has also been carried out. Pressure pads were installed beneath the sand road base to measure subgrade stress.

Preliminary test results have shown that subgrade stress is reduced for the reinforced case due to improved stress distribution through the reinforced road base. Vertical deflection of the pavement was also reduced.

It was noted that in paper 5.1, under the heading “Mechanical Properties of the Polymer Grid” the authors state that exposure to elevated temperatures results in a reduction in stiffness of the grid. Current UK guidelines permit asphalt to be delivered at temperatures between 125 degrees Centigrade and 190 degrees Centigrade. Concern was expressed by the session chairman that as a surfacing contractor he would be forced to work within a reduced range of laying temperature when using polymer grid reinforcement. This may reduce the achievement of adequate compaction and surface finish difficult especially during winter on wind chilled sites.

Prof. Brown agreed that laying temperatures would need to be restricted to below 160 degrees Centigrade adding that for most mixes workability is adequate at this temperature. He also noted that laying temperatures have been restricted to 150 degrees Centigrade for sulphur asphalt in order to reduce pollution from fumes and this had been implemented quite adequately. The 190 degrees Centigrade referred to is the maximum delivery temperature in the UK. The normal maximum compaction temperature in the UK is 145 degrees Centigrade, so this would be the critical temperature.

Reference was also made to paper 5.4 which concluded that adequate compaction was achieved with a maximum laying temperature of 135 degrees Centigrade.

During his presentation Prof. Brown had mentioned that pre-stressing of the reinforcement could be advantageous though it may be impractical. Commenting on this, one delegate suggested that a significant pre-stress may already be occurring during installation due to the shrinkage effect acting on the restrained grid. It was agreed that this would occur in the field, and indeed had been reported in paper 5.3.

When asked where the reinforcement would be located in new construction consisting of base course plus 40mm of wearing course, Prof. Brown stated that he would not contemplate putting reinforcement shallower than 40mm. Ideally the grid would be sandwiched within the base course, or conceivably placed between base course and wearing course.

In paper 5.2, the experimental results from Loop 2 indicated a significant reduction in the angle of curvature (Fig 3).

The question was asked whether this improvement was truly the effect of the reinforcement during loading, or whether it was possible that the reinforcement serves to improve compaction of the asphalt, and the resulting reduction in air voids content is the reason for the apparent increase in stiffness. In answer, reference was made to paper 5.1, Table 3, which shows that the presence of a grid actually impedes compaction of the asphalt. Despite the lower densities achieved under the same compactive effort, the reinforced sections still gave better performance.

In paper 5.3, Table 1 compares the specific tensile strengths of various materials. One delegate commented that this made the polymer materials appear far stronger than steel and this was misleading. What was important he argued, was that in order to reinforce anything a material needed to be extremely stiff, and in this aspect tensar did not have the best performance.

Dr Kneepkohl explained that the table had been included to enable some cost/benefit comparisons to be made. He apologised if the information had proved misleading. With regard to the second point, reference was made to paper 5.1 where the elastic stiffness of reinforced asphalt is discussed. The authors of that paper explained that as there was no significant difference between the elastic properties of the reinforced layers and the unreinforced layers the analysis would remain the same. The reinforcing effect comes into play when assessing the life of the pavement. The reinforcement will limit cracking and reduce rutting. This is how the performance of the reinforcement should be assessed and not purely on the initial elastic analysis.

In response to a question concerning recycling Dr Kneepkohl reported that a reinforced pavement had been taken up with a road milling machine. Laboratory testing of the material had indicated that it is suitable for reuse. 10 tonnes of the material have been stored for recycling trials due to commence in Spring 1984.

While accepting that the subject of the Symposium was Polymer Grid Reinforcement, one delegate commented that...
in reality the subject was Tensar. We expressed concern
that some of the papers lacked objectivity, and that no
comparisons were made with other criteria or methods of
operating. In this connection the author of paper 5.4
was questioned as to what basis had led to the
incorporation of a grid in 1981, since the pavement
design used did not appear to take reinforcement into
effect.

Mr Pooley explained that at the time of design not only
was the effect of reinforcement unable to be modelled,
but the performance of a cracked concrete base was also
unknown. An assumption was made that when broken up it
would perform similarly to a granular base. Because of
concern with the accuracy of this assumption, some
additional insurance was sought. What was known of
Tensar at the time indicated that it was stronger than
other fabrics available and it was selected for that
reason.

In response to the question of objectivity two
participants

Mr Bumah, Chairman
Professor Brown
Professor Haas
Mr Pooley
Dr Kennepohl
Dr Sammied
Dr Hoare
Mr Bridle
Professor Bell
Mr Smith