Early life performance of cement and foamed bitumen stabilised reclaimed asphalt pavement under simulated trafficking

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ABSTRACT: A simulated pilot-scale recycled pavement was constructed in the laboratory by mixing reclaimed asphalt pavement (RAP) material and crushed limestone with selected binders. The RAP and crushed limestone were blended in two proportions and various mix formulations were generated by stabilising with foamed bitumen only, cement only and foamed bitumen plus small amounts of cement. The Nottingham Pavement Test Facility was used to realistically traffic the various pavement sections using a loaded wheel. The early life performance of the various mixtures under traffic was evaluated by monitoring their deformation, surface condition, and resilient strains at the bottom of the recycled layer. It was found that at early life foamed bitumen bound mixtures showed excellent stiffness properties when compared with traditional unbound bases. The materials tend to fail primarily in rutting and no visible fatigue cracking was observed. The rutting resistance of foamed bitumen mixes was dependent on the mixture proportions and penetration grade of bitumen generating the foam. The addition of small amounts of cement significantly enhances the rutting resistance of foamed mixes. The stiffness of RAP mixtures stabilised with cement only was found to be similar to or better than that of conventional asphalt road base materials.

1 INTRODUCTION

The cold in-place recycling technique is an attractive option in road maintenance and rehabilitation. This technique uses a special machine to (1) mill to full or partial depth an existing deteriorated asphalt layer and, often, a predetermined amount of underlying unbound materials, and (2) inject binders and/or stabilising agents such as cement, bitumen emulsion or foamed bitumen and mix them with the recycled material before re-compacting the composite to produce a new stabilised base course. Typically, only a thin overlay surfacing is required on top of a stabilised base course. In addition to a number of advantages such as conservation of resources and energy, preservation of environment, and retention of road geometry, this technique can improve bearing strength and riding quality at a relatively low cost.

In the United Kingdom, road recycling is becoming an increasingly important activity for highway maintenance. Recently a comprehensive set of guidelines on this subject were introduced in TRL reports TRL386 (Milton & Earland 1999) and TRL611 (Merill et al. 2004) which include all aspects of road recycling practices such as site evaluation, thickness and mixture design methods, construction techniques, and specifications.

2 OBJECTIVE AND SCOPE OF INVESTIGATION

A considerable amount of the data available from the literature on cold mix in-situ stabilisation is based on analysis of cores obtained from stabilised layers several months after completing the project. This is understandable as coring cold bituminous stabilised layers (i.e. emulsion or foamed asphalts) before this time is not readily achievable in many cases as a consequence of the low early life strength of such mix types. Similarly, due to the difficulties in testing fragile or weakly bound cold mixes in their early life, a lot of laboratory based investigations are based on testing fully cured specimens.

In this study it was decided to investigate the early life mechanical performance of cold mix stabilised asphalt layers, in particular RAP stabilised base course mixes. It is hoped that the data and insight from this research will lead to a better understanding of early life performance of cold stabilised RAP layers.

In this paper, the performance of a range of stabilised recycled mixes under realistic trafficking conditions under a moving loaded wheel was investigated. The recycled pavement materials were composed of various proportions of reclaimed asphalt pavement (RAP) and virgin limestone aggregate. The stabilising agents included foamed bitumen and cement.

3 TEST PROGRAMME

A trial section of a cold recycled pavement was constructed in the Pavement Test Facility (PTF) which is housed within the Nottingham Centre for Pavement Engineering laboratory. The trial pavement was divided into several smaller sections composed of various mixture proportions and binder types. In the first stage of the investigation, mix design tests were carried out to optimise the properties and/or content of binder for each mixture type. The mixtures were subsequently mixed and compacted at optimum binder contents. Once the pavement section was fully constructed and allowed to cure for a fixed duration, the various sections were trafficked using a single loaded wheel. The performance of the pavement was assessed at frequent intervals by monitoring the magnitude of accumulated permanent surface deformations (rutting) and transient strains at the bottom of the stabilised layer in the wheel path during trafficking.

4 MATERIALS

4.1 *Reclaimed asphalt pavement*

Reclaimed asphalt pavement (RAP) was collected from an asphalt producer with an in-plant asphalt recycling facility. The RAP was originally milled from various asphalt roads and brought together into one stockpile. Composition analysis was performed to determine the properties of RAP and its extracted components. The results are shown in Table 1.

4.2 *Mineral aggregate*

Virgin Limestone aggregate was collected and stored separately in six stockpiles according to the following sieves: 20 mm, 14 mm, 10 mm, 6 mm, passing 6 mm (dust) and passing 0.075 mm (filler). The addition of Limestone filler was necessary as the RAP was found to be deficient in fine particles passing 0.075 mm sieve. The Limestone aggregates were dried and weighed according to required proportions before mixing with RAP materials.

Table 1. Properties of RAP

Recovered bitumen		
Binder content (%)		4.1
Penetration (25 °C, 100 g, 5 s) (0.01 mm)		25
Softening point (ring & ball) (°C)		63.2
Viscosity @ 120 °C (mPa s)		253
Gradation		
Sieve size	Percent Passing	
	RAP	Extracted Aggregate
20 mm	93.7	99.1
14 mm	80.6	87.9
10 mm	64.8	74.4
6.3 mm	44.8	60.5
2.36 mm	24.6	44.3
1.18 mm	14.3	35.2
0.600 mm	8.4	28.7
0.300 mm	3.5	20.0
0.150 mm	1.1	13.2
0.075 mm	0.4	9.7

4.3 *Foamed bitumen*

Foamed bitumen is produced by injecting air and water droplets under high pressure into a pre-heated penetration grade bitumen. As the water turns into steam, bitumen changes from the liquid state into foam. This is mainly a physical rather than a chemical process. The life of the foam at ambient temperature is very short, measured in seconds. Soon after production, the foam bubbles quickly collapse thus reverting the bitumen back to its liquid state and gradually regaining its viscous condition.

Foaming technology was first introduced by Professor Ladis Csanyi (Csanyi 1957) and then developed by Mobil Oil in the 1960s by creating an expansion chamber. In the mid-1990s, the equipment manufacturers Wirtgen developed this system by creating the Wirtgen WLB-10 laboratory foaming plant in which both air and water are injected into the hot bitumen in an expansion chamber as shown in Figure 1.

In this study, two penetration grade bitumens, PG50/70 and PG70/100, were selected for the production of foamed bitumen. Basic properties of the bitumens including penetration, softening point, and viscosity were determined and the results are presented in Table 2.

Foamed bitumen is commonly characterised in terms of its Expansion ratio (ER) and Half-life (HL). During the bitumen foaming process, the foamed bitumen would expand to a maximum volume and then the bubbles would collapse completely. ER is defined as the ratio between maximum volume achieved in the foam state and the volume of bitumen after the foam has completely dissipated. HL is the time that the foam takes to collapse to half of its maximum volume. For a given temperature, there is an optimum percentage of added water by mass of bitumen that produces the most effective ER and HL of the foamed bitumen.

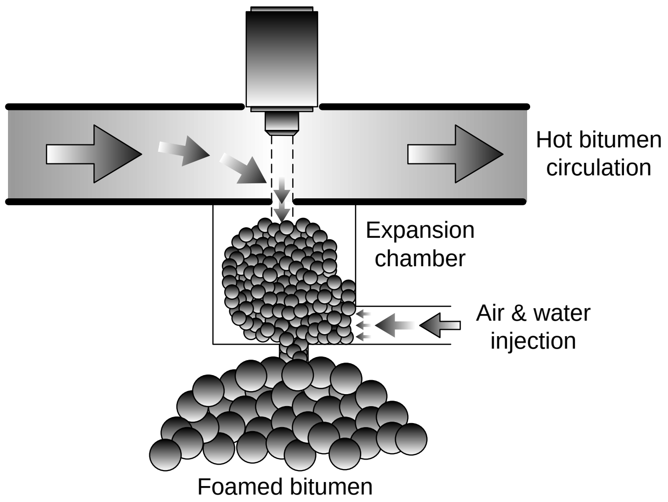

Figure 1 Foamed bitumen produced in an expansion chamber

Table 2. Basic properties of bitumen PG50/70 and PG70/100

Properties of bitumen	PG	
	50/70	70/100
Penetration (25 C, 100 g, 5 s) (0.01 mm)	56	76
Softening point (ring & ball) (˚C)	53.1	49.6
Viscosity @ 140 ˚C (mPa s)	401	262
Viscosity @ 160 ˚C (mPa s)	162	114
Viscosity @ 180 ˚C (mPa s)	83	57

In order to determine the temperature and amount of added water that produce suitable foaming characteristics, the two bitumens were subjected to foam production at 140 ˚C, 160 ˚C and 180 ˚C and at various water contents using the Wirtgen WLB-10 laboratory foaming plant. The foamed bitumen characteristics are presented in Figure 3.

At temperatures at or above 160 °C, the expansion ratio of foamed bitumen was higher than that at 140 °C although the half-life was slightly shorter. Wirtgen (2001) recommends a minimum expansion ratio of 8 times and half-life of 6 seconds. Based on the results, a temperature of 160 ˚C and a water content of 2.0% by mass of bitumen were selected to create the most stable foam for both bitumens. The corresponding maximum ER and HL values were 15 times and 9 seconds for PG50/70, and 17.5 times and 16.5 seconds for PG70/100 respectively.

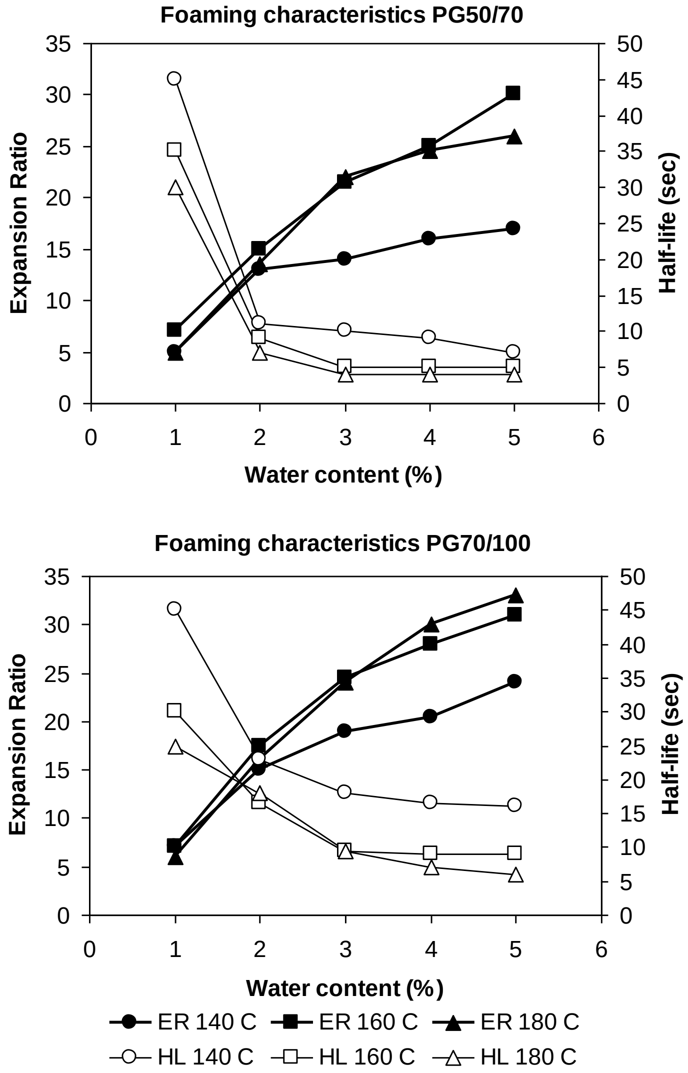

Figure 3 Foaming characteristics of bitumen grade PG50/70 and PG70/100

4.4 *Cement*

The cement used in this study was normal Portland cement type I.

5 MIX DESIGN

5.1 *Mixture proportions*

In this study, two proportions of RAP to aggregate were considered:

Proportion 1 – (RAP : Aggregate) = (75 % : 25 %) by mass.

Proportion 2 – (RAP : Aggregate) = (50 % : 50 %) by mass.

The composition of limestone aggregate was designed based on Fuller's equation and the gradation of the aggregate should fall into the recommended envelope (Milton & Earland 1999). The composition of the aggregate used in this study is presented in Table 3.

Table 3. Composition of limestone aggregate

Aggregate size (mm)	Proportion by mass (%)
20 mm	25
14 mm	12
10 mm	13
6 mm	8
Dust	42

Aggregate and RAP were blended together and 5% of filler by weight of dry aggregate (i.e. RAP + limestone) was added to adjust the gradation curves of the mixtures. Figure 4 shows the gradation curves of these mixtures. In the case of cement bound mixtures, the filler component was omitted as it was argued that the cement would act as filler in such mixtures.

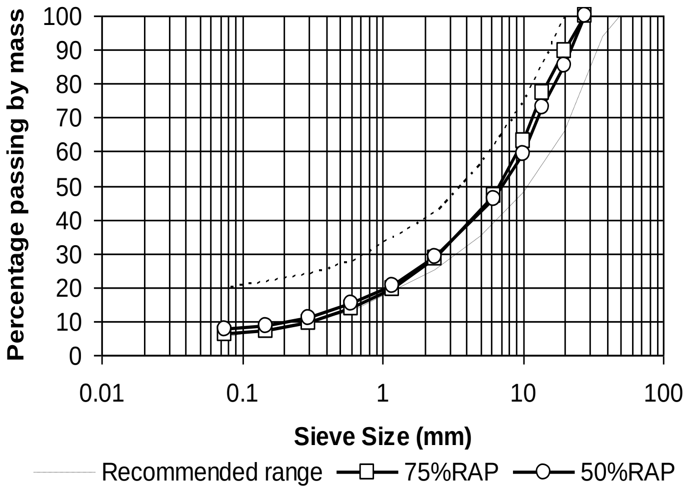

Figure 4 Gradation of studied blends

RAP and virgin aggregate blends were treated with foamed bitumen and/or cement. Four mixtures were primarily treated with foamed bitumen and two with cement. In one of the foamed bitumen mixtures (Mix 4), 1.5% cement was added by mass of dry combined aggregate. In total, six mixtures were made up according to the following combinations:

Mix 1: Foamed bitumen PG50/70 treated 75%RAP
Mix 2: Foamed bitumen PG70/100 treated 75% RAP
Mix 3: Foamed bitumen PG70/100 treated 50% RAP
Mix 4: Foamed bitumen PG70/100 treated 50% RAP + cement 1.5 %
Mix 5: Cement treated 75% RAP
Mix 6: Cement treated 50% RAP

5.2 *Design for foamed bitumen bound mixtures*

Mix design was performed to establish the optimum binder content of each mixture. The optimum foamed bitumen content was determined by a common procedure as described in *Wirtgen Cold Recycling Manual* (Wirtgen 2001). The optimum moisture content of each aggregate mixture was determined from the modified Proctor test. Foamed bitumen was produced at the selected temperature and water content as previously described. The indirect tensile strength (ITS) test was conducted on 100 mm diameter by 63.5 mm high Marshall cylindrical specimens compacted to 75-blows using a Marshall hammer, in both the dry and wet conditions. The optimum foamed bitumen content was considered as the bitumen content that produces the maximum soaked ITS whilst maintaining a minimum retained ITS value of typically around 75% (defined as the ratio between soaked and dry ITS) (Wirtgen 2001). Table 4 summarises the optimum binder contents and other related results of mix design.

Table 4. Mix design results

Parameters	RAP proportion	
	75%	50%
Optimum binder content (%)	2.5	2.7
Maximum dry density (Mg/m^3)	2.02	2.20
Optimum moisture content (%)	4.2	4.5

5.3 *Design for cement bound mixtures*

The mix design procedure for cement bound mixtures was to determine optimum water content, followed by the determination of the cement content required to meet the minimum strength requirements. In this study, the minimum cube compressive strength was targeted at 4.5 N/mm^2 as suggested by Milton & Earland (1999) for a road with less than 0.5 million standard axles. The samples were compacted using a vibratory compactor. A pair of 150 mm cubes were then produced from the materials at the optimum moisture content, each pair having a different cement content. The cubes were crushed at 7 days and the cement content necessary to achieve the target strength was determined. The mixture porportions adopted for both cement bound mixtures (Mixes 5 & 6) were as follows:

– 6.0% normal Ordinary Portland cement.
– Optimum moisture content = 6 %

6 PILOT-SCALE TRIAL

6.1 *General*

In this part of the investigation, recycled asphalt pavement base courses were constructed and tested in the PTF. The PTF, which is an approximately half-scale facility, was used to produce repeated traffic loading onto the pavement via a moving wheel under controlled speed and load conditions. Details of PTF equipment are described elsewhere (Brown & Brodrick 1981).

The PTF foundation consisted of a 450 mm crushed limestone subbase sitting on top of a Keuper Marl clay subgrade. It was found from dynamic cone penetration (DCP) tests that the clay subgrade was very weak with a CBR value of less than 1%. Light falling weight deflectometer (PRIMA 100 LWD) tests (Carl Bro 2006) indicated that the stiffness at foundation level (see Fig. 5) varied from 37 MPa to 81 MPa with an average of approximately 60 MPa.

The thickness of the trial pavement was purposely designed to ensure that the pavement would suffer some degradation within a reasonable number of load applications. It was decided to construct the pavement as thinly as practically possible. Thus based on the compaction criteria, the trial stabilised pavement layer thickness was selected at 80 mm which was around four times the maximum aggregate size.

6.2 *Construction*

In the case of foamed bitumen bound mixtures, the materials were weighed and then mixed in a Hobart mixer at the optimum moisture and binder contents based on the mix design results. Foamed bitumen was produced at 160 °C with 2% water content. However, due to a limitation of the mixer's capacity, the materials could only be mixed in 6 kg batches. Thus the mixtures had to be stored in sealed containers at room temperature (20 ± 5 °C) and it took approximately 2 weeks to manufacture enough quantities for construction of all the stabilised layer sections in the PTF pit.

In the case of the cement bound materials, the RAP, limestone aggregate, cement and water were weighed and thoroughly mixed in a concrete mixer on the day of compaction.

For the foamed bitumen plus cement mixture, the RAP and aggregate were treated with foamed bitumen first and the product was stored in sealed containers for up to about 10 days. On the day of compaction, cement and an additional quantity of water were then mixed with the foamed bitumen treated material using a concrete mixer.

The materials were then placed into the PTF pit, spread and compacted. A Wacker VP1340A plate compactor was used to compact the materials in a single layer. Before compaction, the moisture content of the materials was measured. The materials were weighed so that following compaction to the required thickness, their dry densities would not be less than 95% of their corresponding maximum dry density values as achieved in the laboratory compactability tests. The time required to lay and compact all the sections in the trial pavement was such that most sections would have been left to cure in the compacted state for at least 14 days before trafficking commenced. The exception was the section that was composed of foamed bitumen plus cement as the binder, which was cured for 7 days before trafficking.

6.3 *Instrumentation*

Embedment strain gauges, two in the transverse direction and one in the longitudinal direction, were installed at the bottom of the recycled pavement layer in each section and all gauges were installed directly underneath the wheel path. Two pressure cells were also installed in one of these sections. The pavement layout and instrumentation are illustrated in Figure 5.

6.4 *Trafficking*

The performance of the pavement under traffic was evaluated by repeatedly applying wheel loads onto the pavement in the PTF as shown in Figure 6. The trafficking was canalised, i.e. successive wheel passes being superimposed without lateral distribution of the wheel tracks across the pavement.

The trial pavement had two lanes. Both lanes were trafficked with an equal number of load applications on each day of testing. This ensured that the performance of the different mixtures could be compared directly without having to consider the effects of differential curing between the various test sections. Trafficking was carried out at an approximate velocity of 3 km/hr. The number and magnitude of the loads applied to each lane are schematically illustrated in Figure 7. The early life strengths of the foamed sections were relatively low, and to avoid premature damage, the magnitude of the first applied wheel load was selected at the lowest practical level (3 kN) that can be comfortably applied using the PTF. The sections were trafficked and the accumulation of permanent deformations was monitored at this load level. When the rate of increase of surface deformations reduced to an insignificant level, the load applied was subsequently doubled in magnitude. In this experiment, the first 5000 passes were at a wheel load of 3.0 kN, the next 10,000 passes were at a wheel load of 6.0 kN, and the remaining passes were at a wheel load of 12 kN. Trafficking was terminated when the cumulative number of passes was equal to 45,000 passes per lane. The tyre pressure was 600 kPa for all applications, equivalent to that in typical heavy goods vehicle tyres.

6.5 *Monitoring*

At intervals of approximately 1000 load applications, the strains at the bottom of the recycled pavement layer were recorded. At every 3000 to 4000 wheel passes, the pavement profile was measured using a straight edge and the pavement surface was visually inspected. After completion of all trafficking, a number of cores were extracted from each section using the dry coring technique to provide samples for laboratory testing.

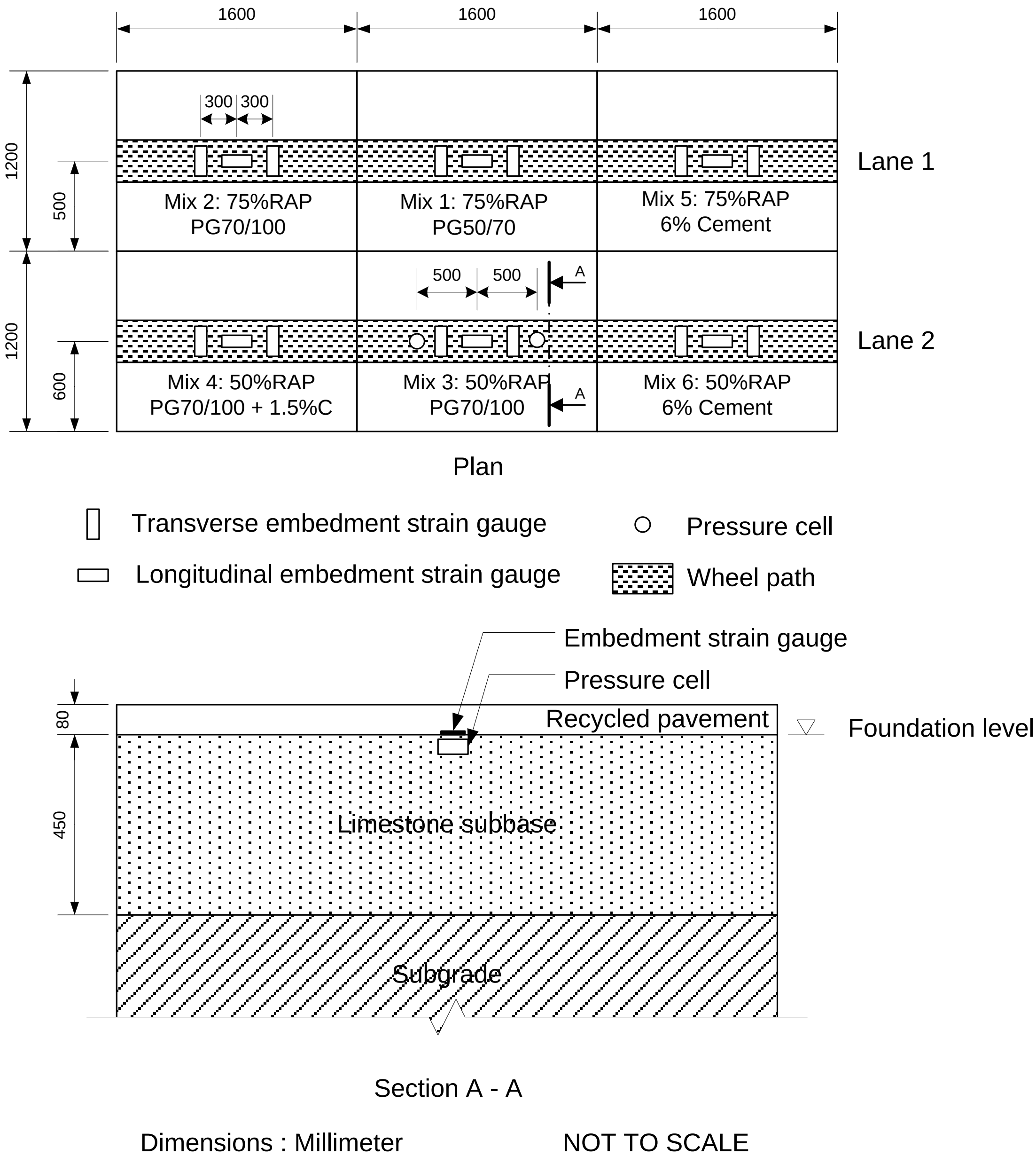

Figure 5 Trial pavement layouts

Figure 6 Trafficking trial pavement in the PTF

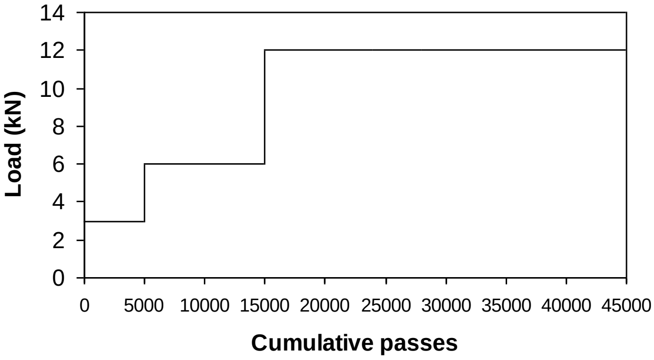

Figure 7 Loading schedule

7 RESULTS

7.1 *Visual inspection*

The pavement surface was inspected every 3000 to 4000 loaded wheel passes. No evidence of distress was observed in the cement bound sections. The foamed bitumen bound sections with no cement additive started to rut as soon as the first load was applied (i.e. 3 kN). Rutting occurred only in the wheel path. Elsewhere, other than in the wheel path, the transverse profile of the pavement remained unchanged. It was also observed that, on every occasion that the magnitude of wheel load was increased, there was a significant immediate rise in rut magnitude. At each load level, the rutting rate was found to gradually decrease with increasing number of wheel passes. When the wheel load was increased to 12 kN, i.e. the maximum load selected in this investigation, and when no more significant increases in rut depth were noticed, it was decided to terminate the test. Trafficking was thus terminated after 45,000 wheel passes per lane.

Between 15,000 and 20,000 passes, i.e. soon after the wheel load was increased to 12 kN, longitudinal cracks were observed at both sides of the wheel paths on the foamed bitumen stabilised sections. Cracks were not observed in the foamed section with added cement or on the cement stabilised sections. These cracks were the result of excessive rutting and are not believed to have been caused by fatigue cracking.

7.2 *Permanent deformation*

The surface deformations of each section were measured at 8 points at equal intervals along the wheel path. The average permanent surface deformation under the wheel path of the trial sections is shown as a function of load applications in Figure 8. Rutting was developed to varying degrees in all foamed bitumen bound sections but no noticeable surface deformation was observed in either of the cement only treated sections. In terms of amount of rutting, the mixtures can be ranked as follows (from best to worst):

– Cement bound mixtures
– Mix 4 - 50%RAP PG70/100 + cement,
– Mix 3 - 50%RAP PG70/100,
– Mix 1 - 75%RAP PG50/70,
– Mix 2 - 75%RAP PG70/100.

For foamed bitumen bound materials, the differences in the rut depth values and profiles between different mixtures indicate that the rutting potential of stabilised mixtures depends on the binder type, mixture proportion and the presence of cement. In general, foamed bitumen bound mixtures that contain a higher proportion of RAP and a softer binder exhibited greater deformations.

The effect of mixture proportions can be seen by comparing Mix 2 and Mix 3. Up to the first 5000 load applications, just before the load level was increased to 6 kN, the average rut depth in Mix 2 was approximately 6 mm and that of Mix 3, which contained less RAP, was 2 mm. At 15,000 load applications, average deformation of Mix 3 was still about one third that of Mix 2. However, when the load was increased to 12 kN, the difference in rut depth between these two mixtures decreased. The final deformation at 45,000 load applications of Mix 2 was about 13.5 mm while the deformation of Mix 3 was about 12 mm, i.e. a difference of only about

1.5 mm. The effect of mix proportions on permanent deformation was more pronounced during early life and at low loads, but when the mixture was subjected to higher loads, this effect was less significant.

The penetration grade of the binder also affected the permanent deformation during the early life and low load. This effect can be shown by comparing the performance of Mix 1 and Mix 2, which contained the same amount of RAP but different binder grades. After the pavement had been subjected to 5000 passes at 3 kN load, the rut depth of Mix 1, containing the harder grade binder, was about 33% that of Mix 2. When the pavement was loaded with another 10,000 passes at 6 kN, the rut depth of Mix 1 increased to about 67% that of Mix 2. During the application of the 12 kN wheel loads, although Mix 2 deformed on average by a slightly greater amount, the permanent deformations of both materials were approximately equal with a difference of 1 mm or less. The harder penetration grade bitumen produces a foam that improves the mixture resistance to permanent deformation. However, when the mixture is subjected to higher loads, a harder penetration grade bitumen makes little difference compared to a softer grade bitumen.

Unlike conventional hot mix asphalts, foamed bituminous bound composites are visually very different from fully coated hot mixtures. In foamed mixes the fine aggregate and filler components are preferentially coated by the bitumen. It was clearly visible in this experiment that the RAP and coarse aggregate particles in the foamed bitumen bound mixtures, though bound together, were not fully coated by the bitumen. Also noticeable was the visible voided nature of the mixtures. Mix deformation was therefore likely to depend on both particle interlock as well as binder stiffness. At low stress levels, the binder contribution to the mix response was relatively high resulting in improved response with the use of a harder grade binder. On the other hand, at high loads, the binder contribution to the mix behaviour was less evident and the deformation behaviour of the mixtures was primarily governed by the aggregate interlock regardless of the binder grade.

The effect of adding a small amount of cement is clearly observed from the performance of Mix 4. The magnitude and rate of deformation of Mix 4 was clearly smaller than that of all the other foamed bitumen mixtures. When the test was terminated at 45,000 load applications, all three foamed bitumen bound sections with no added cement developed significant surface deformation with an average rut depth greater than 12 mm. On the other hand, the foamed bitumen bound section containing 1.5% cement deformed on average by only 5 mm. This 60% decrease in rutting indicates a significant increase in permanent deformation resistance caused by the inclusion of cement as an additive.

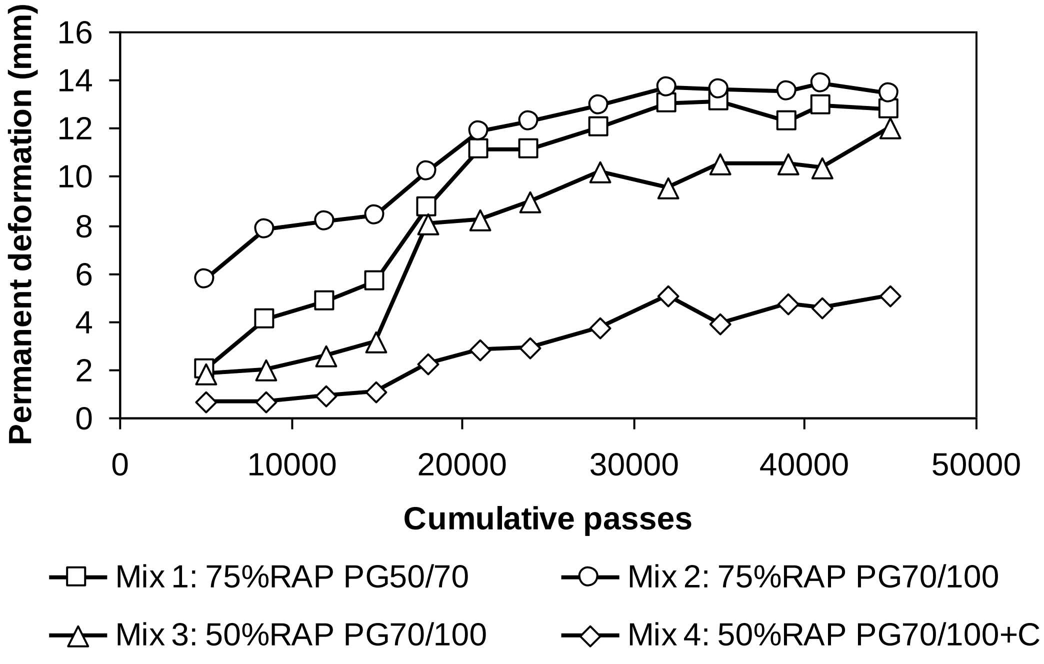

Figure 8 Average surface permanent deformation of foamed bitumen bound mixtures

As part of an earlier study carried out on emulsion stabilised RAP mixes with added cement, Montepara & Giuliani (2002) carried out analysis at the molecular level and were able to prove that when cement is added to bituminous emulsions, a "new" binder is not generated. The rigidity effects are due to two simultaneous mechanisms, i.e. the emulsion breaking and the cement hydrating within the watery phase of the bituminous emulsion. The hydrates formed in situ fill part of the porosity of the composite and contribute towards its cohesion and rigidity. A very similar set of conclusions are to be expected when foamed bitumen is used instead of a bituminous emulsion.

7.3 *Strains at bottom of the stabilised layer*

7.3.1 *Typical shape of strain responses*

Strain gauge data acquisition was performed at every 1000 passes. Figure 9 shows the typical shape of horizontal transversal and longitudinal strain gauge responses located at the underside of the recycled materials. The readings shown were taken at 28,000 passes. Depending on the position of the wheel in relation to the strain gauge location, there are two types of response, i.e tensile strain (positive) and compressive strain (negative). The transverse and longitudinal strain gauges showed different responses, in that the transverse response was normally in tension whereas the longitudinal response showed a combination of compression and tension.

The transverse strain response shape can be explained in simple terms as follows; the wheel load causes a deflection bowl with maximum deflection directly under the load. The circumferential stresses at any radial distance away from the centre of the load will always be tensile and will decrease in magnitude as the load centre moves further away from the strain gauge. When the wheel is close enough to the strain gauges, the deflection bowl causes the strain gauges to flex and results in a measurable tensile response.

The scenario for the longitudinal gauges is slightly different. As a simple analogy, if one imagines a infinite beam on an elastic foundation, the beam will deflect downwards underneath and close to the point of load application, the bending moment at the underside of the beam causing tensile stresses, but at a certain distance away from the load the beam will be subjected to a slight hogging moment thus causing compressive stresses at the underside of the same beam. Hence, as the wheel moves towards the strain gauge, an initial compressive stress is followed by a tensile stress wave as the wheel approaches and travels over the strain gauge location. As the wheel moves away from the gauge, exactly the reverse would occur. The amount of peak compressive strain recorded in the longitudinal direction was approximately one third or less in relation to the peak tensile strain.

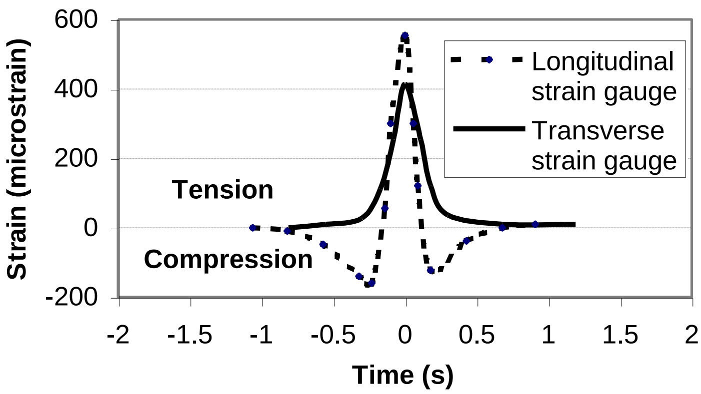

Figure 9 Typical profiles of transverse and longitudinal recorded strains.

7.3.2 *Characteristics of strain responses*

Figure 10 shows the transient strain responses for all sections during trafficking. There were 18 gauges in total and 16 of those gauges survived the compaction process undamaged. Each section had two transverse and one longitudinal strain gauge, and this allowed the elimination of data from gauges generating unreliable data.

As expected, and except for the response of Mix 2, the deflections and hence tensile strain values of all sections increased as the wheel load was increased in magnitude. This can be seen as sudden jumps in the strain profiles in Figure 10. The unexpected readings from the gauges in Mix 2 may imply lack of cohesion between the surface of the gauge and the surrounding mixture.

The recorded tensile strains appeared to be relatively stable during trafficking under wheel loads of 3 and 6 kN. This was followed by a gradual reduction in strain response with time during the 12 kN wheel load trafficking. This is quite reasonable as the strength of the foamed bitumen bound materials increased with time due to the curing effect. Once again, these trends were not observed from the strain gauges of Mix 2.

With the exception of Mix 6, the strain values obtained from the longitudinal gauges, in general, were higher than those from transverse gauges. It is difficult to form an opinion on this matter as the number of sections tested was inadequate to justify this as a generalised observation. However, the overall trends obtained were logical.

7.4 *Back calculated modulus*

In this study, an attempt was made to estimate the modulus of the stabilised layers. This problem was approached with the aid of a linear elastic software "BISAR" (Shell 1998). This software is normally used as a forward calculation tool in pavement analysis. Using BISAR, the stresses and strains can be calculated at any point within the structure for any multi layered construction, assuming each layer is infinite in the horizontal dimensions, knowing the thickness, modulus and poisson's ratio of each of the layers and the magnitude and locations of the applied loads.

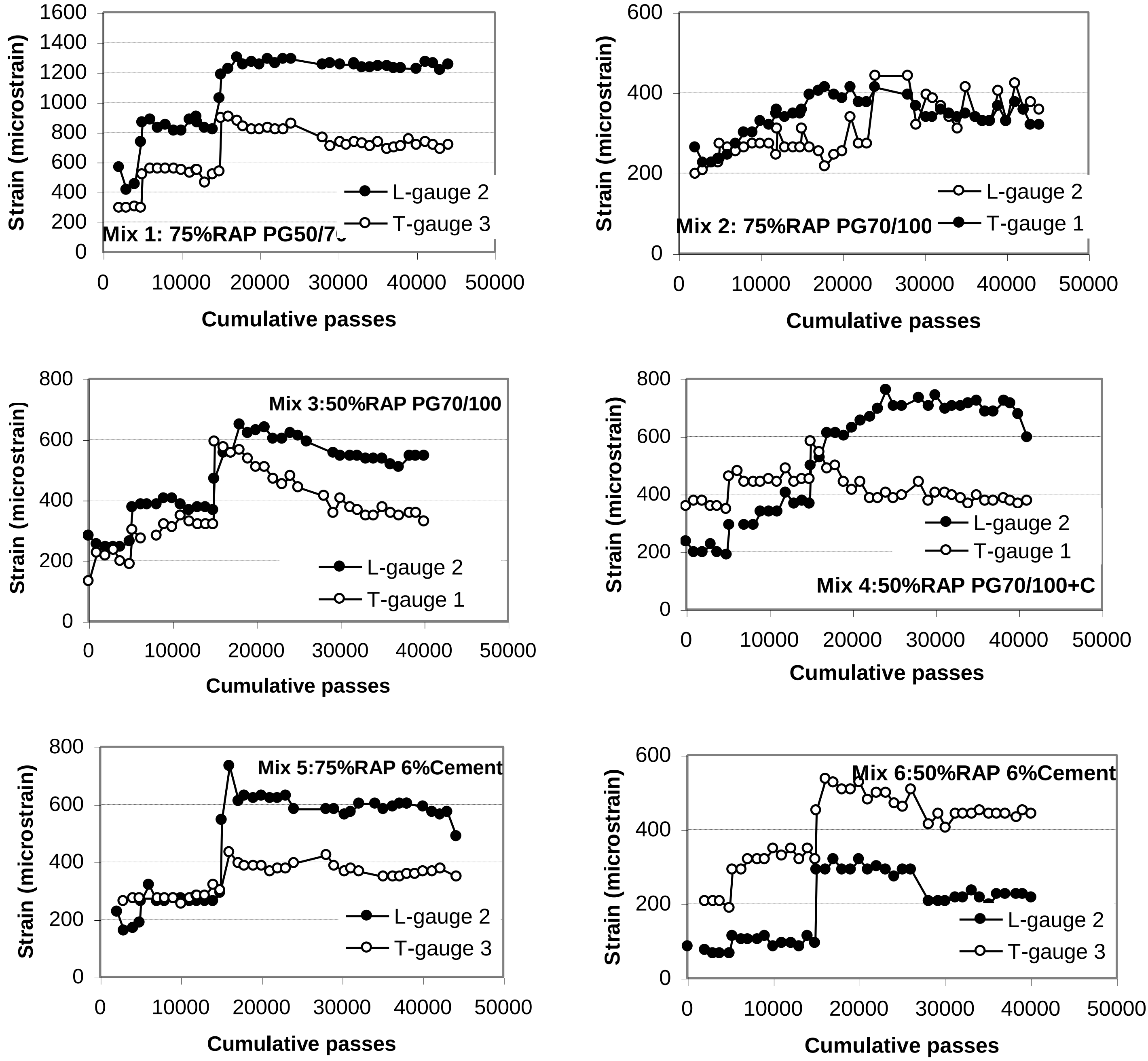

Figure 10 Measured transient tensile strain results

To calculate the modulus of a stabilised layer based on the measured strains and using BISAR software, the following procedure was devised:

The first step was to simplify the layer system from an actual 3-layered construction to a 2-layered system, since the individual moduli of the granular sub-base and soil sub-grade layers were unknown. Using the dynamic plate test over the sub-base surface, the combined modulus of the sub-base + sub-grade (= foundation) was measured at about 60 MPa. Therefore the pavement was analysed as a 2 layered system, in which the stabilised material is referred to as the first layer with a thickness of 80 mm and the second layer has a modulus 60 MPa with an infinite thickness.

The second step was to generate a small data-base relating predicted tensile strain values underneath the wheel to a range of stabilised layer moduli values at each of the 3 wheel loads applied. These data can be fitted to develop a strain-modulus equation at each applied wheel load. Figure 11 shows the relationships resulting from this step.

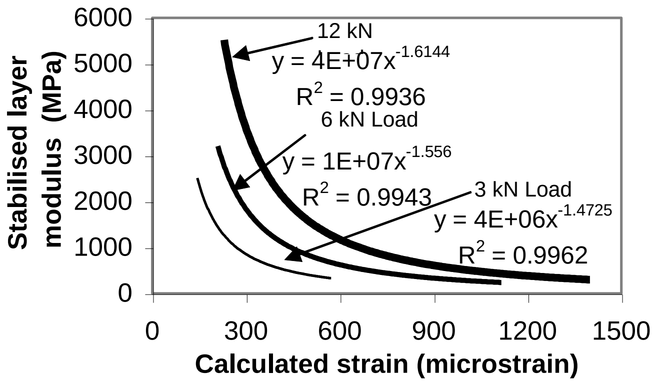

Figure 11 Calculated strain-modulus relationship of a stabilised layer based on a 2 layered BISAR model.

The final step was calculating the modulus of the stabilised layer using the strain-modulus equation, at each wheel load, in which the experimentally measured strain value becomes the input data.

Table 5 presents an overview of the results of the modulus calculations. The modulus results appear to span a relatively wide range. This was primarily caused by the scatter of output from the various strain gauges within each test section. It is also easy to appreciate that the responses were greatly affected by fluctuations in load, plus variability in layer thickness and foundation stiffness across the test pit. Taking an overview of all the results, the modulus of an 80 mm foamed asphalt layer on a 60 MPa foundation at early life is a function of mixture composition and extent of curing. Foamed bitumen bound recycled asphalt mixes had stiffness values ranging from around 300 MPa at the lower end to about 2500 MPa at the upper end, whilst for the cement bound recycled layer, the stiffness values ranged from 1000 to 3000 MPa.

Table 5 An overview of calculated modulus values

Mix	Calculated modulus during trafficking (MPa)
1	300 - 800 (at the beginning) 350 -1000 (at the end)
2	Data was not reliable
3	1000-1300 (at the beginning) 1500-2500 (at the end)
4	500 -1500 (at the beginning) 600 -2500 (at the end)
5	1000-1500 (at the beginning) 1500-2500 (at the end)
6	1500-2000 (at the beginning) 2000-3000 (at the end)

For comparison purposes, the modulus of a granular base course with a thickness of 80 mm resting on a granular foundation having a modulus of 60 MPa was estimated using the Shell design procedure (Claessen et al. 1977), to be about 90 MPa. The modulus of a typical hot rolled asphalt road base of 80 mm thickness subjected to a vehicle speed of 3 km/hr and having a binder stiffness of 20.5 MPa at 20 °C was estimated to be approximately 2900 MPa (Brown & Brunton 1988, pp.14-15). It can thus be seen that in terms of modulus, foamed bitumen bound recycled mixtures exhibit excellent early life characteristics when compared with conventional unbound base materials and, as the curing time increases and the mixtures densify with additional trafficking, may become as good as those of typical asphalt road base materials. The modulus of cement bound recycled materials at the relatively short curing times tested in this investigation was also found to be similar to or better than that of typical hot rolled asphalt base mixtures. It is, however, significantly lower than a typical cement bound base due to the inclusion of bitumen in the RAP.

Figure 12 presents an interesting feature of one of strain gauge responses from Mix 4. Unlike other gauges, the transient strain response increased gradually during trafficking especially at wheel loads of 6 and 12 kN, which implied a gradual reduction in modulus. It is hypothesised that with respect to this gauge's response, one of the main reasons for the reduction in modulus of the stabilised layer is that cracks may have developed in the layer during trafficking. This prediction was difficult to confirm without additional supporting data, such as conducting flexural beam tests to determine whether the mix will crack at the strain levels imposed on the mix in the PTF. In an earlier study on fatigue resistance of foamed asphalt material using the Indirect Tensile Fatigue mode of testing (ITFT) (Sunarjono 2006), it was found that initial cracking occurred at a very low number of cycles but that this was followed by a long period of crack propagation (approximately 60% of total fatigue life). On the other hand, the crack propagation period noted for a conventional 20 mm dense bitumen macadam in the ITFT test is around 10% of the total fatigue life (Read 1996). Such trends can be partially explained by the low modulus of foam stabilised materials and the fact that the binder is not as continuous as in conventional asphalt; both factors are expected to retard the progress or propagation of cracks beyond the point of initiation.

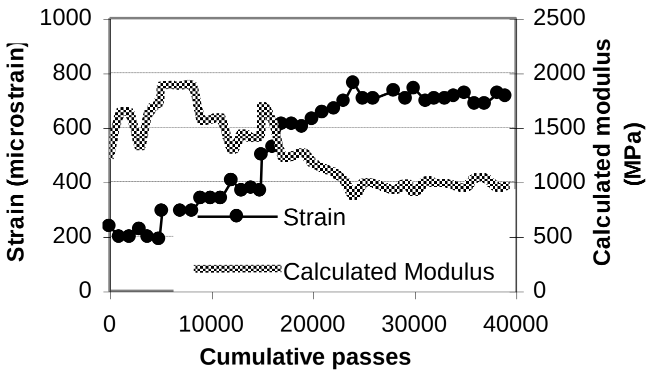

Figure 12 Observations of the stabilised layer modulus reduction during trafficking in Mix 4.

7.5 *Stiffness of cored specimens*

At the conclusion of the trafficking phase, the trial sections were cored. Unfortunately, it was not possible to conduct coring with any ease because the aggregates were weakly bound by foamed bitumen. Wet coring was impossible because the water would damage the bonding of material, and therefore it was decided to use a dry coring technique. Initially, a coring barrel diameter of 100 mm was tried but this did not produce any intact cores. Using 150mm diameter coring was only slightly more successful. Coring was carried out in the wheel path (trafficked) and away from the wheel path (un-trafficked). It was possible to core Mixes 3 and 4 at both positions, however all the un-trafficked cores obtained from Mixes 1 and 2 were damaged during the coring process.

The stiffness modulus of the cored specimens was measured using the ITSM test and the results are shown in Figure 13. The calculated modulus values at the end of trafficking for each section are also superimposed on this figure.

The ITSM stiffnesses of the un-trafficked specimens in Mix 4 are about three times higher than those of the trafficked specimens. The lower end of the calculated modulus band for this section coincides with the average trafficked ITSM value (about 500 MPa). However, in the case of Mix 3 the ITSM stiffnesses of both trafficked and un-trafficked specimens were very similar values (about 300 MPa), and far lower than the calculated modulus values as shown. Sample disturbance and damage during coring of low stiffness mixtures such as Mix 3 are likely to have caused a reduction in the measured ITSM values. This applies to both the trafficked and un-trafficked sections. The trafficked specimens ITSM stiffness values in Mix 1 also coincided with the lower limits of the calculated modulus for this section (about 400 MPa), whereas the ITSM values of the trafficked specimens of Mix 2 were the lowest of all (about 200 MPa). Results from the cement stabilised mixes 5 and 6 are not available at the time of preparation of this paper.

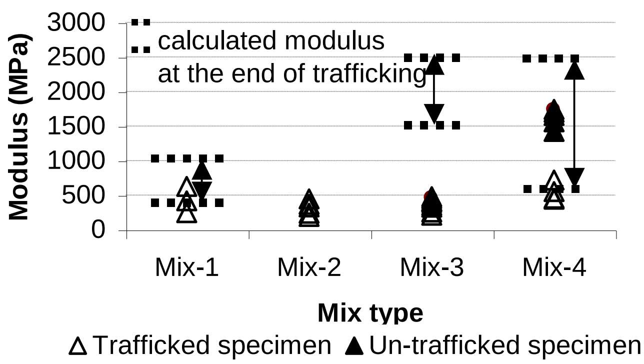

Figure 13 Comparison between actual ITSM values of cored specimens from the four foamed bitumen stabilised sections and the calculated modulus limits from strain gauge readings.

8 CONCLUSIONS

The early life performance of a pilot-scale trial pavement consisting of various recycled asphalt pavement mixtures has been investigated under traffic loading. From the results and observations in this study, the following conclusions can be drawn:

1 Foamed bitumen bound recycled materials, when subjected to repeated traffic load, tend to fail in rutting mode rather than fatigue cracking.
2 The resistance to rutting of foamed bitumen bound materials is affected by the amount of RAP in the mixture and also the penetration grade of bitumen used to generate the foam.
3 Foamed bitumen bound mixtures that contain a higher proportion of RAP and a softer binder exhibited greater deformations. This effect is less significant as the magnitude of load and curing periods increase.
4 The addition of a small percentage of cement to the foamed bitumen bound mixtures significantly enhances the resistance against rutting failure.
5 Overall analysis of the strain gauge results shows that the modulus of foamed bitumen bound recycled materials at the early life is much higher than that of conventional unbound base materials and can be as high as that of traditional asphalt base material. The predicted modulus of cement bound materials was also comparable to that of traditional asphalt base material.

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