EVALUATION OF THE FIELD AND LABORATORY FATIGUE PROPERTIES OF ASPHALT MIXES

M.A. Moffatt  
Senior Research Engineer  
ARRB Transport Research

G.W. Jameson  
Principal Research Scientist  
ARRB Transport Research

G.D. Foley  
Principal Research Engineer  
ARRB Transport Research

K.G. Sharp  
Principal Research Scientist  
ARRB Transport Research

ABSTRACT

This paper summarises the findings of an Austroads/AAPA-sponsored accelerated loading trial conducted to examine the fatigue performance of asphalt mixes. Although over 500,000 cycles of the ALF 80 kN dual-wheel load were applied to four test sites during two stages of testing, no fatigue failure could be induced in any section, with the only distress observed being surface deformation related to shear failure of the underlying unbound base and sub-base layers. The results of an extensive series of response-to-load testing using H-bar strain gauges were also inconclusive because, whilst the effect of temperature on asphalt strain was clearly demonstrated, the results were counter intuitive in terms of the effect of load magnitude and type, and loading rate, on pavement response. Unfortunately the usefulness of the strain gauge data was severely limited because of the large corrections that needed to be applied to take account of variations in pavement temperature. An extensive series of laboratory testing was also undertaken on both laboratory-prepared samples and field specimens and the results are also summarised in the paper.

Following the completion of the trial, the performance data were evaluated, along with data obtained from other accelerated loading trials and in-service field trials, to assess whether an asphalt fatigue shift factor should be adopted in the Austroads pavement design system. A number of trials were evaluated, including a recent trial conducted in the USA and the OECD DIVINE Project. The results were inconclusive in that some studies supported the introduction of shift factors whilst others did not.

If it were decided to adopt an asphalt fatigue shift factor in the Austroads Pavement Design Guide, then presumably it will be a factor that results in the Austroads design method having an average reliability of 50%, rather than the current situation, where the average reliability of asphalt pavements is 85-95%. Given this, consideration would need to be given to adjusting the subgrade failure and cemented materials fatigue relationships so that they also result in the design procedures for these two distress modes having average design reliabilities of 50%. The incorporation of shift factors will also require the design reliability traffic multipliers to be recalculated because the use of shift factors to relate laboratory or predicted performance to field performance should not be considered in isolation from other pavement design issues.

Keywords: accelerated pavement testing, laboratory, asphalt, shift factors, design reliability, response to load, fatigue
INTRODUCTION

The development and harmonisation of national standards for roads is the responsibility of Austroads, the Association of Australian State Road Authorities. The need for a nationally-based approach to pavement research in Australia was recognised in the Austroads Strategy for Pavement Research and Development, which was published in 1987, and updated in 1992 and 1995. The Strategy is currently being again revised and a draft has been prepared (Austroads 1999).

The Australian Asphalt Pavement Association (AAPA) have also developed an R&D Strategy, and one objective under Topic 3.1 (Asphalt) of their original Strategy was to “Develop equipment and test methods to measure the fundamental properties of asphalt, establish representative values and incorporate into design procedures”. The thrust of this objective has been carried through to their latest Strategy (AAPA 1997), viz. “Finalise and validate test procedures with the objective of achieving national uniformity”.

In order to help the achievement of these aims, Austroads, with support from AAPA, have been supporting an R&D program, the broad aim of which is to improve the quality and performance of asphalt mixes. To date, standard testing equipment (the Materials Testing Apparatus (MATTA)), standard test methods for determining the stiffness and creep properties of asphalt, and mix design methods (based on the use of the Gyropac and the MATTA) have been developed to the stage where full-scale testing could be conducted to check/verify these procedures. To this end, an extensive evaluation of the rut-resistance properties of asphalt mixes under accelerated loading was conducted (Sharp et al. 1996; Oliver et al. 1997). Austroads’ APRG Report No. 18 “Selection and design of asphalt mixes – Australian provisional guide” was also issued in May 1997 in partnership with ARRB Transport Research (ARRB TR) and AAPA.

While the use for road surfacings of asphalt mixes with improved rut-resistant properties is of considerable benefit to the road community, the predominant application of such materials (thin surfacing on granular material) requires that the fatigue resistance properties be not excessively compromised by improving permanent deformation resistance. Establishing that such mixes are superior to conventional mixes with respect to rut-resistance therefore required that the fatigue properties of these mixes also be established.

In addition, delays have occurred in the formulation of a laboratory method for the determination of the fatigue properties of asphalt mixes because of the complexity of the problem, the perceived need to await recommendations from the Strategic Highway Research Program (SHRP) and the lack of a suitable test that could be conducted on the MATTA. Recent developments, however, had led to the development of a draft standard test method for the determination of the fatigue properties of asphalt that is suitable for use on the MATTA.

Another major related issue is that the Austroads (1992) Pavement Design Guide, which is currently being revised, does not provide performance relationships for the design of pavements incorporating ‘non-conventional’ asphalt mixes, or ‘aged’ mixes. Indeed, there is some doubt as to the validity of the current fatigue performance models for conventional mixes.

In response to these needs, an Austroads/AAPA-sponsored study was conducted, during which the fatigue performance of two conventional dense-graded asphalt mixes, one composed of a conventional binder and one composed of an SBS binder, was evaluated under both field and laboratory conditions (using the Accelerated Loading Facility (ALF) and the MATTA flexural fatigue testing apparatus).

Following the completion of this study (Stage I), it was agreed that there was a need for further evaluation of the conventional Class 320 mix, and a stiffer (Class 600) mix, with a target binder content of 4.8% by mass, under both field and laboratory conditions.
(Stage II). The asphalt mixes were designed in the same way as those tested during Stage I of the study.

This paper summarises the findings of the accelerated loading trials, including the results of relevant laboratory characterisation studies. Following the completion of the trial, the performance data were evaluated, along with data obtained from other accelerated loading trial and in-service field trials, to assess whether an asphalt fatigue shift factor should be adopted in the Austroads pavement design system. The results of this assessment are also presented.

DETAILS OF TEST SECTIONS

The ALF site was about 58 m long and 38 m wide and testing was conducted at two sites centred on chainage 26 m (Stage I) and 16 m (Stage II). In both cases the test sections were placed “end-to-end” in order that ALF could traffic either two 6 m lengths of each section at the same time or one 12 m length of each section. The site was wide enough to allow trials to be also conducted along offsets 1.5 m from the centreline of testing.

Figure 1 shows the plan of the site, the locations of the two experiments and a sectional view of the site tested in Stage II, including the various thicknesses of asphalt constructed.

Stage I of Testing

The specified thickness of asphalt in Stage I of testing from (site) chainages 0-5 m, and chainages 53-58 m, was 120 mm after compaction to enable cores of sufficient thickness to be collected to allow for the possibility of testing being conducted in terms of SHRP protocols developed at Texas A&M University. The nominal cross-section of the pavement testing in Stage I of testing is shown in Figure 2. The specified thickness
of asphalt between chainages 8 m and 50 m was to be 70 mm after compaction, although subsequent levelling using the ARRB TR Walking Profiler showed that the actual compacted thickness varied between 75 and 85 mm. The asphalt was ramped gradually between chainages 5-8 metres, and chainages 50-53 m, so that the first two requirements could be met.

The mixes tested during Stage I were as follows:

- A conventional "control" mix with Class 320 binder chosen to be representative of current VicRoads surfacing mixes (designated F1).
- The "control" mix (F1 grading) but with an SBS polymer binder substituted for the Class 320 binder (designated F3). This mix was nominated by the Austroads PMB Working Group, which also provided a generic binder specification.

The F1 "control" mix was designed and constructed on the basis that fatigue failure would occur after about 50,000 cycles of the 80 kN ALF dual-wheel load based on Austroads Pavement Design Guide predictions, whilst the F3 (SBS) mix was selected as an example of a mix which should have a much longer life than the conventional mix.

The decision to place the asphalt in a single 70 mm thick layer was based on the dimensional requirements for samples to be tested in the MATTA Beam Flexure apparatus as well as practical construction considerations.

<table>
<thead>
<tr>
<th>Stage I</th>
<th>Stage II</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphalt (70 mm)</td>
<td>Asphalt (55 mm)</td>
</tr>
<tr>
<td>Crushed Rock Base (150 mm)</td>
<td>Crushed Rock Base (150 mm)</td>
</tr>
<tr>
<td>Imported Sand Sub-Base (500 mm)</td>
<td>Imported Sand Sub-Base (500 mm)</td>
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<tr>
<td>Crushed Rock Sub-Base (100 mm)</td>
<td>Initial Design Surface level</td>
</tr>
<tr>
<td>Lime-Stabilised Imported Subgrade (300 mm)</td>
<td>Lime-Stabilised Imported Subgrade (300 mm)</td>
</tr>
<tr>
<td>Imported Clay Subgrade (200 mm)</td>
<td>Imported Clay Subgrade (200 mm)</td>
</tr>
</tbody>
</table>

Figure 2: Cross-section of test pavements in both stages of testing (nominal thicknesses only)

**Stage II of Testing**

The specified thickness of asphalt in Stage II from chainages 0-5 m, and chainages 53-58 m, was 75 mm after compaction in order that slabs could be removed of sufficient thickness that beams, 50 mm square, could be cut for fatigue testing in the laboratory. The specified thickness of asphalt between chainages 8 and 50 m was 55 mm after compaction. Once again, the asphalt was ramped gradually between chainages 5-8 metres, and chainages 50-53 m, so that the first two requirements could be met.

The mixes tested during Stage II were the same Class 320 mix tested in Stage I (designated F1A) plus a stiffer (Class 600) mix, with a target binder content of 4.8% by mass (designated F7).

The Class 600 mix was selected as an example of a stiffer mix which should have a longer life than the conventional mix. This mix was adopted, rather than a mix containing a polymer modified binder, because the results of the Stage I testing had
demonstrated that no distress was likely to be observed in such a mix in the time available, this, of course, being related to the amount of available funding and also climatic considerations.

**INSTRUMENTATION**

For Stage I of testing, H-bar strain gauges were placed under the asphalt layer in both sections in order that the response to load (viz. horizontal tensile strain) under both FWD and ALF loading could be measured and these strains used to estimate fatigue lives based on the Austroads (1992) asphalt fatigue life performance model. The H-bars were placed under the asphalt layer, transverse to the direction of ALF loading, along a line between the two offsets used for ALF testing and at ALF chainages 1, 2, 3, 4, and 5 m in the F1 section and ALF chainages 7, 8, 9, 10 and 11 m in the F3 section.

Partial Deflection Gauges (PDGs) were installed within both test sections for the purpose of measuring the vertical deflections within the asphalt and crushed rock base layers. The PDG gauges were monitored during the response to load testing and the output from the PDGs at one offset was also monitored during initial ALF trafficking.

The mid-depth temperature of the pavement was continuously recorded using thermocouples.

H-bar strain gauges were again used in Stage II of testing. However, it had been noted during the analysis of the data collected during Stage I that, because all the H-bar strain gauges had been placed transverse to the direction of ALF trafficking, the longitudinal strain had to be estimated based on the data measured transverse to the direction of traffic and also the results of CIRCLY (MINCAD 1996) analysis. For Stage II, the H-bars were placed, alternatively, in both directions at half-metre intervals along a line between sites used for ALF trafficking. The response of these gauges was measured during response-to-load testing conducted both before and after the ALF trafficking.

**LABORATORY TEST PROGRAM**

The objectives of the ALF asphalt fatigue trial laboratory characterisation program were to:

- confirm the current fatigue test method for the ranking of mixes;
- relate the laboratory results with field (ALF) performance, including verification of the current fatigue life prediction models (e.g. Shell);
- assist in determining the desirable fatigue parameters needed as input into the Austroads mix design procedures; and
- relate binder properties with mix properties and performance.

For Stage I of testing, samples of the Class 320 (F1) and SBS (F3) mixes were prepared using the following methods:

**Method I** Plant-mix, laboratory compacted slabs and cores. Slabs were manufactured by rolling compaction using a pedestrian roller. Cores were manufactured using the Gyropac.

**Method II** Laboratory-prepared mix compacted in the laboratory. Mix components were obtained from the supplier and transported to the laboratory for mixing. The binder content used was in line with that determined from field cores.

**Method III** Plant-mix, field compacted slabs (400 mm long x 300 mm wide x 70 mm high) cut from the from untrafficked pavement areas at the completion of the trial. Cores (100 mm diameter x 63.5 mm high) were also obtained from the ALF pavement.
The structure of the laboratory test program for Stage II was similar in format to that carried out in Stage I except that testing of laboratory-mixed and compacted samples using the aggregate and binder used in the trial (Method II) was not carried out. Indirect tensile resilient modulus testing of all mixes was carried out by ARRB TR on Gyropac specimens and field cores using the MATTA in accordance with the current Australian Standard, except that the test temperatures were 10, 20 and 25°C. The bulk density and binder content of each core were also determined.

Fatigue testing of laboratory-prepared mixes was carried out by ARRB TR on beam specimens using the IPC Beam Fatigue Apparatus and the NARC interim test method (viz. controlled strain at 20°C, 10 Hz, etc.). The mixes were tested at a minimum of three strain levels (400, 600 and 800 µε), with a minimum of three replicates at each strain level. The F1 mix was also tested at 250 µε. In addition, testing was conducted on samples of the mixes cut from the pavement.

Additional fatigue testing was carried out by ARRB TR on the F1 mix at 10°C and some limited fatigue testing was conducted on field samples of the F1 mix when: (1) cut in the direction transverse to ALF trafficking, and (2) cut in the longitudinal direction (direction of rolling).

In the original laboratory characterisation program for Stage II of testing, the field-compacted samples were to be tested for resilient modulus at three temperatures and for fatigue at two temperatures on beams cut in the transverse and longitudinal direction with respect to direction of compaction. However, it was later decided to extend the laboratory testing program to include the characterisation of the viscosity and fatigue properties of the F1 (Class 320) and F3 (SBS) mixes tested in Stage I, and the F7 (Class 600) mix, both in the ‘as received’ condition and also after laboratory ageing. The ‘as received’ samples of the Stage I and Stage II mixes had been field aged for 26 months and five months respectively.

The results of Phase I of laboratory testing program associated with Stage II of the trial are reported in Baburamani and Luke (1999). Following the completion of Phase I, it was decided to proceed only with one part of the originally-mooted Phase II of testing, viz. fatigue testing of the F1A (Class 320 mix, Stage II) and F7 (Class 600) mixes. This testing was conducted at four strain levels and two temperatures on beams cut parallel to the direction of rolling in order that the results could be compared to those obtained on beams cut transverse to the direction of trafficking.

**SUMMARY OF RESULTS OF STAGE I OF TESTING**

Trafficking was conducted in three phases between June 1996 and July 1997. The results of the laboratory characterisation of the mixes are reported in Baburamani (1997) and Baburamani and Hogan (1997), whilst a summary of these results, together with the results of the analysis of the ALF trafficking data, is presented in Foley et al. (1998).

Investigation using Fuji Prescale Film (Fuji 1986) showed that the most uniform pressure across the width of a truck tyre was achieved at a pressure of 600 kPa and this tyre pressure was therefore adopted for Stage I of testing.

The fatigue life of each mix was defined as the number of ALF loading cycles at which 50% of the trafficked area had surface fatigue cracking of at least 1.0 m/m² in extent. After three phases of testing and up to 595.5 kilocycles of the 80 kN and 90 kN ALF dual-wheel load (or 2.3 x 10⁷ repetitions of the Standard Axle loading) being applied to some sections, there was no cracking observed in any section. In addition, there was no increase in maximum pavement deflection or curvature during testing, confirming that no cracking had been initiated at the base of the asphalt layers. As no fatigue cracking was observed under ALF loading it was not possible to rank the performance of the mixes in terms of observed field performance.
Figure 3: Typical pavement profile at completion of trafficking (Stage I of testing)

The only distress observed was deformation of the surface of the pavement. A pavement investigation was undertaken to confirm the mode of failure of the pavement. The cross-sections of the Class 320 and SBS sections are shown in Figure 3. The trenching confirmed that the observed failure was due to shearing of the crushed rock base and sand sub-base layers rather than to fatigue failure of the asphalt surfacing.

Based on the longitudinal strains estimated from the transverse strains measured under the 80 kN ALF dual-wheel load at 20 km/h, and allowing for the ALF narrow transverse load distribution, the current fatigue relationship recommended in the Austroads Pavement Design Guide predicted that cracking would not occur until between 1,000 and 1,900 kilocycles of loading (average of 1,360 kilocycles), compared with the original estimated design life of 50 kilocycles and the total number of cycles actually applied of almost 600 kilocycles (of which about 160 kilocycles were applied using the 90 kN dual-wheel load).

The experiment was initially designed to produce a horizontal tensile strain of $\approx 400 \mu\varepsilon$; however, from FWD testing and pavement modelling, the typical calculated strain was $364 \mu\varepsilon$. When the Shell fatigue relationship was applied to a back-calculated strain level of $364 \mu\varepsilon$ under ALF loading, the predicted number of cycles to "failure" was 158 kilocycles, or about three times the predicted design life. However, based on this predicted strain, no cracking, or increase in pavement curvature, was observed under ALF loading after 3.8 times the Shell-estimated fatigue life.
Pavement lives were also estimated assuming the relationship developed during the earlier Mulgrave ALF trial (Jameson, Sharp and Vertessy 1992). Based on these calculated fatigue lives, the performance of the trial pavement, suggested that either the asphalt was experiencing lower strains than the 364 µε value derived using the back-calculated material moduli and/or that the Mulgrave relationship was not applicable to this trial.

The mean maximum deflection ($D_0$) and curvature ($D_0$-$D_{200}$) values were similar to those recorded during the Mulgrave trial, indicating that the strength of the pavements was similar. The deflection data did not support the contention that the pavement tested in Stage I was substantially stiffer or stronger than those tested at Mulgrave, which otherwise may have explained the reason for the lack of cracking observed during Stage I of testing.

A check of the back-calculated moduli values for the crushed rock base layer was undertaken using partial deflection depth gauge (PDG) data because the predicted asphalt fatigue lives varied markedly depending on the modulus adopted for the supporting granular layer. The crushed rock deformation data, measured by the PDGs, was compared with layer deformations using the derived pavement model and there was very close agreement (within 2%). The back-calculated layer moduli were therefore considered to be acceptable.

It is considered that pavement temperatures were sufficiently controlled, and low enough, that any variation would not account for the longer than expected fatigue life.

No degradation of the joint between the two mixes was observed during trafficking. It was therefore possible to obtain satisfactory test data close to the asphalt joint and it is recommended that this strategy be adopted for future trials.

As no pavement distress was observed during trafficking, a considered estimation of the length of test section required could not be provided. However, the deflection bowl data suggested that the sections were very uniform and, on this basis, it was considered that a test section length of 6 m would be adequate (i.e. sufficient data could be obtained) provided this degree of uniformity was achieved in later trials. On that basis, this strategy was again adopted for Stage II of testing.

The main findings of the Stage I laboratory study were as follows:

- The indirect tensile modulus of the Class 320 asphalt at 20°C taken from the field was lower than the modulus of samples compacted using laboratory gyratory compaction (80 cycles). The indirect tensile moduli of the SBS and Class 320 mixes at 20°C were greater than the flexural stiffnesses.

- The laboratory fatigue performance of the Class 320 asphalt was not influenced by the method of mixing or by the method of compaction. However, the relative laboratory fatigue lives of the two mixes varied greatly depending on the method of sample preparation.

- The fatigue lives estimated using the predicted longitudinal strains and the laboratory-based relationship were between 36 and 62 times the design estimate of 50 kilocycles. However, the stiffness of the asphalt during the laboratory testing would be far greater than that under ALF loading due to the higher rate of loading. When the laboratory-based fatigue relationship was applied to the back-calculated strain level of 364 µε, the predicted number of cycles to “failure” was 344 kilocycles, or about seven times the predicted design life. On the other hand, based on this predicted strain, no cracking was observed under ALF loading after 1.7 times the laboratory-estimated fatigue life.

- The predicted laboratory fatigue life for the predicted strain under ALF loading was probably lower than that observed in the field because the effects of vehicle wander and crack healing cannot be replicated in the laboratory. On the other hand, whilst
the correction factors that may apply to the combined effects of both these phenomenon are not known, it is considered that they would explain only a part of the difference observed in this trial.

- Observation of the condition of the laboratory fatigue beams at the “end of their life”, i.e. after a 50% loss of initial modulus, revealed no cracking of the asphalt, this being the criterion upon which the asphalt was to be deemed as “failed” under ALF trafficking. Cracking did not initiate in the field and the structural integrity of the pavement had not decreased, indicating that the asphalt modulus was relatively constant and had not decreased over time – and the lack of increase in pavement curvature over time supports this. Hence there would appear to exist a period of life between the loss of 50% of the initial modulus and the cracking of the asphalt. As no cracking or increase in pavement curvature was observed during trafficking, it was not possible to monitor the rate of decrease in asphalt modulus over time.

- Based upon the measured strains prior to ALF trafficking, and using a proposed model developed to estimate the fatigue life of PMB asphalt (Foley et al. 1998), the expected life of the SBS modified surfacing ranged between 22,000 and 82,000 kilocycles of the ALF 80 kN load at 20 km/h.

- The measured laboratory fatigue life was 3.4 times greater than the life predicted using the proposed SHRP fatigue relationship. Consequently, it was concluded that the proposed SHRP relationship was of limited value in predicting the performance under ALF in this experiment.

The fact that the performance of the Class 320 asphalt far exceeded design predictions was therefore probably due to a combination of factors as follows:

- Slightly higher than targeted bitumen content of the mixes, resulting in a less stiff and hence more fatigue resistant mix.

- Lower than targeted air voids in the mixes resulting in increased fatigue resistance.

- Stiffer than originally designed (sand) sub-base which provided greater support to the asphalt surfacing than originally anticipated.

- Part of the testing period extended into the warmer months of the year.

- The inherent conservatism of the existing Austroads fatigue relationship.

**SUMMARY OF RESULTS OF STAGE II OF TESTING**

A total of 500,000 cycles of load, composed of 50,000 cycles of the 50 kN dual-wheel load and 450,000 cycles of the 80 kN dual-wheel load, were applied simultaneously to both test sites. In this test the tyre pressure was increased to 690 kPa. There was no increase in maximum deflection or curvature during the testing and no suggestion that any fatigue failure was being induced in either mix (see Figure 4). The only distress observed was surface deformation towards the end of trafficking which was later identified as being related to shear failure of the sand sub-base layer. This result was identical to that observed during Stage I of testing.

Data from three FWD surveys were back-calculated to determine the insitu elastic moduli of the pavement materials. As expected, the results showed that the modulus of the asphalt was temperature sensitive and that the moduli of the crushed rock and sand layers were dependent upon the FWD loading stress. The analysis also showed that the natural subgrade material was not stress dependent (at least within the range of the FWD loading imposed), and that its vertical modulus could reasonably be fixed at a value of 150 MPa. The backcalculation analysis was less successful in
characterising the stabilised imported subgrade material and, for this reason, it was
decided to adopt an ‘intuitive’ stress-independent modulus of 200 MPa for this material
in further analysis.

Following the completion of the ALF trafficking, an extensive series of response-to-load
testing was conducted. H-bar strain gauges had been installed, in directions
transverse and longitudinal to the direction of trafficking, at the bottom of the asphalt
layer during construction and response was measured under the following ALF loading
conditions: 50 kN dual-wheel load, 80 kN dual-wheel load, and 50 kN super single
wheel load. Testing was conducted at the normal ALF operating speed of 20 km/h and
at ‘creep’ speed (nominally 2-3 km/h).

Before any modelling of the strains within the asphalt layer was conducted the modulus
of the asphalt was estimated by examining the laboratory test data conducted on
samples of asphalt removed from the ALF site. Relationships were established which
enabled the flexural moduli determined in the laboratory to be estimated for a range of
temperatures for both asphalt mixes and at both ALF loading speeds.

Horizontal strains at the bottom of the asphalt layer were calculated using CIRCLY and
the pavement model previously established, and the results were compared with
pavement temperature and measured strains for the various loading conditions. The
results clearly demonstrated that the H-bar results for transverse strain between the
wheels (or at the edge of the single wheel) did not match the theoretical model, the
strains being compressive, whilst the H-bar strains were small and tensile. This
difference was possibly related to the nature of the assumed circular loading patch
used in the theoretical model. Such a condition is unlikely to occur in the field,
especially when the wall stiffness of the tyres is considered.

There was a reasonable agreement between the measured and modelled strains in the
longitudinal direction beneath one wheel of the dual-wheel assembly. Data from the
transverse, as well as the longitudinal gauges, beneath the Class 600 asphalt were compared with the modelled strains and asphalt temperature and it was found that the strains measured in both directions were very similar to each other and also in reasonable agreement with the predicted strains, though always lower than the predicted strains.

In order to directly compare all results, the H-bar strain gauge data were corrected to a standard reference temperature. Temperature correction factors were derived by dividing the predicted strains at various asphalt temperatures by the predicted strain at the reference temperature. It was necessary to correct the data to four pavement temperatures (15, 18, 21 and 24°C) to take account of the variations in temperature during the response-to-load testing. Temperature correction charts were prepared by dividing the predicted strains, for each loading combination, by the predicted strain at that temperature.

Unfortunately, the analysis of the H-bar strain gauge data did not confirm the intuitive expectations regarding the influence of load, loading time and wheel configuration on measured strain. The fact that the temperature during ALF loading (approximately 12°C) was generally lower than that during the response-to-load-testing (15-24°C) hampered the analysis because the temperature correction factors that often had to be applied were so large as to make the corrected data unrealistic. This was especially the case for the response-to-load testing conducted under the 80 kN dual-wheel load (about 24°C) compared to the ALF trafficking with the same load (about 12°C). Ideally, of course, response-to-load testing should be conducted at a pavement temperature as close as possible to that operating during ALF loading. The practicality of this strategy in a country such as Australia is, however, very difficult because the number of weeks where operating temperatures are sufficiently low and consistent is limited. Whilst a pavement cooling system was used with some success in Stage I of the trial, it was only capable of reducing temperature by 2 or 3°C and the costs associated with operating the system formed a significant proportion of the total budget. This trial did, however, again highlight the very significant influence of temperature on strain gauge data.

A comparison of the estimated number of cycles of load to induce a 20 mm rut at the surface with the number of repetitions of strain at the top of the sand layer suggested that the number of cycles predicted using the Austroads (1992) subgrade performance model was about three times that based on the accumulation of surface rutting. However, examination of trenches suggested that shear failure occurred in the sand layer rather than the accumulation of plastic deformation in all layers.

The main findings of the laboratory study were as follows:

- The air void content of field-compacted samples averaged 7.6% for the Class 320 (F1A) and 9.1% for the Class 600 (F7) asphalts.

- For both laboratory-compacted and field-compacted samples, the resilient modulus of the mix composed of the harder Class 600 binder did not appear to be significantly different from that of the mix which was composed of the softer Class 320 binder.

- The fatigue test results on the plant-mixed, laboratory-compacted samples indicated that the initial flexural stiffness of the two mixes was very similar, and that, contrary to normal expectation, the performance of the Class 600 mix was marginally superior to that of the Class 320 mix.

- The Class 600 mix had a lower flexural stiffness (considering the air voids in the samples) and superior fatigue performance (higher fatigue life) than the Class 320 mix at all initial strain levels.
The fatigue performance of the field-compacted Class 320 mix at 20°C was clearly inferior to that of the Class 320 mix tested in Stage I. This was possibly related to the higher air voids of the beams tested. The fatigue performance of the Class 320 mix tested in Stage I and the Class 600 mix was generally very similar.

The fatigue performance of beams cut parallel to the direction of rolling was slightly superior to that of beams cut transversely to the direction of rolling.

The fatigue performance of the Class 600 mix was significantly superior to that of the Class 320 mix at 10°C, particularly at the lower initial strain level (≤400 με). The fatigue performance of the Class 320 mix tested in Stage I was also superior to that of the Class 320 mix tested in Stage II by at least by a factor of two. As observed with the fatigue results obtained at 20°C, the fatigue performance of the Class 320 mix tested in Stage I and the Class 600 mix at 10°C was also similar. The reasons for the anomalous stiffness and fatigue characteristics of the Class 600 mix are not clear, and detailed characterisation of the Class 600 binder may be required.

An attempt was also made to derive ‘shift factors’ between the laboratory prediction and observed performance. This is discussed in the next section of this paper.

INCORPORATING SHIFT FACTORS FOR PREDICTING THE FATIGUE LIFE OF ASPHALT

One of the major anticipated outcomes of the ALF trial had been the derivation of ‘shift factors’ between the laboratory predicted fatigue life and the observed fatigue life under accelerated loading in order that recommendations could be made as to whether a shift factor should be included in the year 2000 edition of the Austroads Pavement Design Guide.

In the previous two editions of the Austroads Pavement Design Guide (Austroads 1987 and 1992), the allowable loading, in terms of asphalt fatigue, is calculated using the Shell (1978) asphalt fatigue relationship. However, when the Guide was first published it was not appreciated that this relationship was developed by Shell to predict the laboratory fatigue life of asphalt mixes from mix properties. In the Shell (1978) Pavement Design Manual, the allowable traffic loading of asphalt in-service is estimating by multiplying the allowable laboratory fatigue lives by shift factors. This brings into question the current use, in the Austroads Guide, of the Shell laboratory fatigue relationship without shift factors.

ARRB Transport Research Ltd was commissioned by Austroads and AAPA to review existing information on the relationship between observed and predicted pavement performance, including the origins of the current Austroads design procedures to estimate asphalt fatigue life and the Shell laboratory fatigue relationship.

The origins of the current Austroads design procedures to estimate asphalt fatigue life, including the Shell fatigue relationship, are discussed in Jameson (1999). It is now known that the Shell (1978) general relationship between the maximum tensile strain in asphalt produced by a specific load and the allowable number of repetitions of that load was developed to predict the laboratory fatigue life of asphalt mixes rather than the field fatigue life. This is discussed by Jameson et al. (1992) and later by Potter (1999) in his description of the background to the current Austroads Pavement Design Guide.

It is also noted by Jameson (1999) that these Shell shift factors seem to have been based on theoretical calculations: no published data was found relating the predictions of the Shell Pavement Design Manual with the in-service performance of roads.
Figure 5: Asphalt fatigue relationships used to derive Austroads design curvature relationship

Fatigue relationship for asphalt modulus of 4,000 MPa taken from Paterson and Maree (1978)

It is of interest to also consider the fatigue relationships (Figure 5) used to develop the curvature function-allowable loading relationship used in the Austroads Guide (1992) overlay design procedures. Fatigue relationships for moduli of 4000 MPa, 8000 MPa and 20,000 MPa were used because, according to Anderson (1982), these three fatigue relationships were considered appropriate for Victoria. The relationship for an asphalt modulus of 4,000 MPa is that reported by Paterson and Maree (1978) for a continuously-graded asphalt with a modulus of 4500-10,000 MPa and 5% air voids. Anderson (1982) did not state the origins of the relationships used for asphalt moduli of 8,000 and 20,000 MPa. The allowable loadings calculated using these fatigue relationships (Figure 5) produce fatigue lives 6-25 times those of the Shell laboratory relationship recommended in the Austroads (1992) Guide for the design of new pavements. As the Paterson and Maree relationship was considered appropriate for Victoria, it brings into question whether the Shell laboratory fatigue relationship is overly conservative.

**Review of Accelerated Loading Trials**

**Callington ALF Trial**

Data is available from the Callington ALF trial (Jameson, Kadar and Sharp 1997) which could be used to assess whether there was a shift factor between performance under ALF loading and laboratory test data. However, the varying temperature conditions in the experiments would make this analysis difficult and the laboratory testing was conducted prior to the standardisation of testing equipment and procedures developed as part of the Austroads/AAPA-sponsored research conducted throughout the 1990s.

**Mulgrave ALF Trial**

In the Mulgrave ALF trial (Jameson, Sharp and Vertessy 1992), seven experiments were completed on six full depth asphalt pavements composed of a nominal 120 mm of dense-graded asphalt. Extensive laboratory and field testing was conducted to complement the ALF trial.
Analysis of the data suggested that, for the trial mix under accelerated loading, about 50% of the pavement had crocodile fatigue cracking at the end of the Austroads predicted asphalt fatigue life. However, it should be recognised that:

- the results were obtained under accelerated loading over a five-month period of testing during winter/spring when the asphalt was only 12 months old – less severe cracking than this may have occurred with normal trafficking over 15-30 years due to crack healing; and
- the back-calculated subgrade modulus values used to estimate the asphalt strains were substantially higher than those based on the laboratory and field estimated CBR values normally assumed for pavement design. Back-calculated subgrade modulus values in excess of 100-200 MPa were common, even though the subgrade had a laboratory soaked CBR of 3-4. It was concluded that if subgrade modulus continues to be modelled this conservatively, then the Shell laboratory fatigue relationship would be associated with less than 50% of the area with severe cracking.

Based on the trial findings it was recommended that the Shell laboratory fatigue relationship should continue be used to estimate the fatigue live of roads in-service, without the use of shift factors.

**OECD DIVINE Project**

In the mid-1990s the OECD initiated a major international infrastructure research project known as DIVINE (Dynamic Interaction of Vehicle and Infrastructure Experiment) (OECD 1997). A major component of the program was a study of the effects of dynamic wheel loads on pavement life by comparing the performance of a test pavement when it was subject to dynamic wheel loads representative of steel spring and air bag suspensions.

A flexible test pavement consisting of 80 mm of asphalt over a prepared granular base and select subgrade was constructed and trafficking was applied using the circular test track, CAPTIF, which is owned and operated by Transit New Zealand.

The allowable loading of the test pavement under CAPTIF loading was calculated (Sharp, Jameson and Yeo 1996) using:

- layer moduli back-calculated from FWD testing, and
- the Austroads mechanistic pavement design procedures, including the Shell laboratory fatigue relationship.

For an asphalt modulus of 2,000 MPa, the mean predicted fatigue life was 74,000 CAPTIF loading cycles. However, fatigue cracking was not observed until about 900,000 CAPTIF load repetitions. The observed number of load repetitions to 1 m/m² and 3 m/m² of cracking under CAPTIF with a steel spring suspension were about 950,000 cycles and 1,700,000 cycles, respectively (Sharp and Moffatt 1996).

This data suggested that asphalt fatigue shift factors of about 13 and 23 were required to predict the allowable loading to cracking intensities of 1 m/m² and 3 m/m², respectively.

**Asphalt Fatigue (Dandenong) ALF Trial**

In order to derive shift factors between observed and predicted fatigue performance, it is obviously necessary that fatigue failure be observed in the field. As this did not occur, then it was only possible to derive ‘lower bound’ values based on the number of cycles of load actually applied (see Moffatt et al. 1999).

Using back-calculated layer moduli and the Austroads Guide procedures, the Class 320 asphalt mix tested in Stage I of the trial was predicted to fatigue after about 158,000 cycles of the 80 kN ALF dual-wheel loading. However, as already discussed, no fatigue cracking was observed after about 440,000 cycles of the 80 kN and 160,000
cycles of the 90 kN ALF load. In terms of an asphalt fatigue shift factor for use with the Austroads Guide procedures, the results suggested a shift factor having a minimum value of 3.8.

### Table 1
Estimated ‘Lower Bound’ Shift Factors: Stage II of Dandenong ALF Trial

<table>
<thead>
<tr>
<th>Asphalt mix</th>
<th>Shell¹</th>
<th>Austroads²</th>
<th>Laboratory³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class 320</td>
<td>500 / 880 = ≥0.6</td>
<td>500 / 585 = ≥0.9</td>
<td>500 / 670 = ≥0.7</td>
</tr>
<tr>
<td>Class 600</td>
<td>500 / 120 = ≥4.2</td>
<td>500 / 80 = ≥6.3</td>
<td>500 / 210 = ≥2.4</td>
</tr>
</tbody>
</table>

¹ Shell (refer to Section 7.5.1 of Moffatt et al. 1999)
² Austroads (refer to Section 7.5.1 of Moffatt et al. 1999)
³ Laboratory (refer to Section 7.5.2 of Moffatt et al. 1999)

\[ N = \left( \frac{6918 \cdot (0.856 \cdot V_b + 1.08)}{S_{mix}^{0.36} \cdot \mu e} \right)^5 \]

\[ N = \left( \frac{1901 \cdot V_b - 13118 - 0.3173 \cdot S_{mix}}{\mu e} \right)^{4.1} \]

In terms of Stage II of the trial, Table 1 compares the shift factors corresponding to the various prediction methods discussed by Moffatt et al. (1999). A perusal of Table 1 shows that, for the Class 320 mix, the actual number of loading cycles (500,000) was less than both the predicted lives (Shell and Austroads) and the laboratory life. In other words, the ‘lower bound’ values of the shift factor were less than 1. For the Class 600 mix, the ‘lower bound’ values of the shift factor ranged between 2.4 and 6.3.

**Caltrans Accelerated Loading Trial**

The Caltrans Accelerated Pavement Testing (CAL/APT) Program was a joint effort between Caltrans (the California Department of Transportation), the University of California at Berkeley (UCB), the Division of Roads and Transport Technology of the Council of Scientific and Industrial Research (CSIR) of South Africa and Dynatest Consulting Inc.

The program utilised two Heavy Vehicle Simulators (HVS) to test full-scale pavements in a controlled environment at the UCB’s Richmond Field Station. An extensive laboratory testing program complemented the full-scale accelerated loading testing. Between 1995 and 1997, the fatigue performance, under HVS loading, of the following two pavement types was examined:

- nominal 150 mm of asphalt on 75 mm of asphalt-treated permeable base on 400 mm of granular sub-base, and
- nominal 150 mm of asphalt on 500 mm of granular base and sub-base.

The results are reported in Harvey et al. (1996, 1999a and 1999b).

Jameson (1999) compared the observed asphalt fatigue performance under HVS loading with that predicted using the Austroads Guide (1992) design guidelines. As the Austroads Guide does not provide detailed procedures for the design of asphalt-treated permeable bases, and as this pavement type is not commonly used in Australia, the analysis was confined to the results of testing of the two sections with granular bases.

The comparison was complicated by the fact that there was a lack of bonding between the two asphalt layers, resulting in fatigue cracking of the upper layer but no apparent
cracking in the lower layer. Consequently, the Austroads predicted lives were undertaken assuming both full bonding and full slippage between the asphalt layers.

Assuming full bonding between the asphalt layers, a shift factor of 1.8-3.5 was required to predict the fatigue life to the initiation of surface cracking (1 m/m²) and a factor of 3.1-5.4 to the beginning of crocodile cracking (5 m/m²). Assuming full slippage between the layers, a shift factor of about 13 was required to predict the fatigue life to the initiation of surface cracking and a factor of about 21 to the beginning of crocodile cracking. As the asphalt layers were neither fully bonded nor fully slipping, appropriate shift factors would lie between these two conditions. Jameson (1999) recommended that the analysis be checked when the revised Austroads procedures for the estimation of in-service asphalt moduli from laboratory flexural moduli became available.

**Review of In-Service Field Trials**

**Austroads Long Term Pavement Performance Program**

Pavement monitoring of 19 Australian test sections has been under way for five continuous years as part of an Austroads-funded project on long term pavement performance (LTPP). These sections include both those set up specifically as LTPP sites, some of which are also incorporated into the SHRP database, and also sites specifically established in tandem with ALF trials.

The overall objectives of the LTPP study are to:

- enhance asset management strategies and Austroads pavement design procedures through the use of improved pavement performance models based on an improved understanding of the behaviour of pavement structures (SHRP-LTPP program); and
- compare the results of accelerated pavement testing studies with actual road pavement performance (ALF-LTPP program).

Details of those sites which include asphalt layers are presented in Table 2.

Koniditsiotis (1999) presented the preliminary analysis associated with the four asphalt sites in South Australia (SA02, SA03, SA05, SA10) and the two asphalt sites in Queensland (QL13, QL14), all of which form part of the ALF-LTPP program.

The results to date are inconclusive in relation to shift factors; however, it is anticipated that future monitoring of these sites will enable a comparison of observed asphalt fatigue life with the life predicted using the Austroads Guide procedures.

**VicRoads Reviews of Performance of Asphalt Performance**

Kruize, Jameson and Bethune (1990) reported the findings of about ten years of monitoring of the performance of Victorian heavy duty flexible pavements. Of particular relevance to the current study were the conclusions drawn regarding the performance of the Monash Freeway (formerly South-East Arterial) in Melbourne. The pavement consisted of 75-100 mm of asphalt and a granular base and sub-base.

Using the Austroads Guide procedures, including the Shell laboratory fatigue relationship, the allowable loadings of these pavements were calculated to be $2 \times 10^6$ to $5 \times 10^6$ ESAs. After about $10^7$ ESAs of loading, 90-95% of the length of these pavements had not fatigue cracked.
Table 2
Details of LTPP Test Sections

<table>
<thead>
<tr>
<th>State</th>
<th>Test Section Number</th>
<th>SHRP/ALF</th>
<th>Location1</th>
<th>Road</th>
<th>Pavement Composition</th>
<th>Lane2/Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>NSW</td>
<td>NS20 ALF Somersby</td>
<td></td>
<td>Pacific Fwy</td>
<td>asphalt/macadam</td>
<td>1/S</td>
<td></td>
</tr>
<tr>
<td>QLD</td>
<td>QL04 SHRP Ipswich</td>
<td></td>
<td>Warrego Hwy</td>
<td>asphalt/CR base/CTSB</td>
<td>1/W</td>
<td></td>
</tr>
<tr>
<td></td>
<td>QL13 ALF Beerburrum</td>
<td></td>
<td>Bruce Hwy</td>
<td>asphalt/CTCR</td>
<td>1/S</td>
<td></td>
</tr>
<tr>
<td></td>
<td>QL14 ALF Beerburrum</td>
<td></td>
<td>Bruce Hwy</td>
<td>asphalt/CTCR</td>
<td>1/S</td>
<td></td>
</tr>
<tr>
<td>SA</td>
<td>SA02 ALF Callington</td>
<td></td>
<td>South Eastern Fwy</td>
<td>asphalt overlay</td>
<td>1/W</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SA03 ALF Callington</td>
<td></td>
<td>South Eastern Fwy</td>
<td>asphalt overlay</td>
<td>1/W</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SA05 ALF Callington</td>
<td></td>
<td>South Eastern Fwy</td>
<td>asphalt overlay</td>
<td>1/W</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SA10 ALF Callington</td>
<td></td>
<td>South Eastern Fwy</td>
<td>asphalt overlay</td>
<td>1/W</td>
<td></td>
</tr>
<tr>
<td>Vic.</td>
<td>VC01 SHRP Tullamarine</td>
<td></td>
<td>Western Ring Rd</td>
<td>asphalt/CTCR</td>
<td>1/E</td>
<td></td>
</tr>
<tr>
<td></td>
<td>VC02 SHRP Jacana</td>
<td></td>
<td>Western Ring Rd</td>
<td>asphalt/CTCR</td>
<td>1/E</td>
<td></td>
</tr>
<tr>
<td></td>
<td>VC03 SHRP Broadmeadows</td>
<td></td>
<td>Western Ring Rd</td>
<td>asphalt/CTCR</td>
<td>1/E</td>
<td></td>
</tr>
<tr>
<td></td>
<td>VC04 SHRP Glenroy</td>
<td></td>
<td>Western Ring Rd</td>
<td>asphalt/CTCR</td>
<td>1/E</td>
<td></td>
</tr>
<tr>
<td></td>
<td>VC05 SHRP Fawkner</td>
<td></td>
<td>Western Ring Rd</td>
<td>asphalt CTCR</td>
<td>1/E</td>
<td></td>
</tr>
</tbody>
</table>

1. Location: nearest town or suburb.
2. Lane/Direction: 1 slow or outer lane; S southbound; N northbound; W westbound; E eastbound.

Based on this finding, Kruize et al. (1990) concluded that, “the Shell fatigue relationship is conservative for predicting the fatigue life of asphalt-surfaced pavements in the Melbourne metropolitan area.” If 10% ESAs is considered to be the observed fatigue life of these pavements (5-10% length cracked), then this study suggested that an asphalt fatigue shift factor of about 2 to 5 was appropriate.

Paul (1997) reviewed the performance of deep strength asphalt pavements comprising cement-treated sub-bases and thick asphalt bases constructed in urban areas of Victoria over a 25 year period. In all, 14 deep strength asphalt pavements, constructed between 1971 and 1995, were analysed.

Figure 6: VicRoads performance data on deep strength asphalt pavements (Paul 1997)

The allowable loading of each as-constructed pavement was predicted using the current (1997) VicRoads mechanistic pavement design procedures, but using a full
Standard Axle rather than a half Standard Axle loading. For each site, the damage ratio was calculated, this being the past traffic (up to 1997) divided by the allowable loading. Figure 6 compares these damage ratios with the percentage of the length of the outer wheelpath that was cracked in 1997. It would be expected that, if the asphalt fatigue shift factor were greater than 1.0, then no cracking would be observed for sites where the damage ratio was less than 1.0. This is not apparent from Figure 6, as two sites where the damage ratio was less than 1.0 were substantially cracked and two sites where the ratio exceeded 1.0 were not cracked. In other words, this VicRoads data does not suggest the Austroads Guide procedures need to be amended to include an asphalt fatigue shift factor.

AAPA Heavy Duty Flexible Pavement Performance Study
The Australian asphalt industry has nominated the evaluation of the performance of heavy duty flexible pavements in the field as its highest priority in its R&D strategic planning. As a result, AAPA has provided funding for the commencement of a study involving the monitoring of selected sites in Victoria, New South Wales and Queensland. Emphasis in the project is being placed on existing full depth and deep-strength asphalt pavements which were constructed during the 1970s and early 1980s and which have not been reconstructed or substantially rehabilitated in the intervening period. A further requirement is that there be a reasonable level of historical data available, including pavement age, composition and periodic condition monitoring reports. It is also imperative that the traffic that has used these sections since construction is known to a reasonable degree of accuracy in order that the life predicted using current design recommendations can be compared with the observed life. Typically, the pavements are subject to medium to heavy traffic (>10^6 ESAs) and have a thickness of asphalt of at least 100 mm.

The main perceived outputs of the project are a database incorporating the structural condition, and material properties, of a range of heavy duty flexible pavements, comparisons of observed performance with predicted performance and recommendations regarding improvements to materials specifications and pavement design models which will assist in the optimisation of design reliability. As a result, it is anticipated that this project will provide data on appropriate shift factors for use in the Austroads Pavement Design Guide.

Further details of the study, and progress to date, are reported in Sharp et al. (1999).

Required Modification to Austroads Pavement Design Guide to Incorporate Shift Factors
If it were decided to adopt an asphalt fatigue shift factor in the Austroads Guide, then the following additional development work would be required before the publication of the 2000 Austroads Guide:

If a shift factor is adopted presumably it will be a factor that results in the Austroads design method having an average reliability of 50% because, currently the average reliability of asphalt pavements is 85-95% (Moffatt et al. 1998). Given this, consideration would need to be given to adjusting the subgrade failure and cemented materials fatigue relationships so that they also result in the design procedures for these two distress modes having average design reliabilities of 50%. VicRoads (1993) Technical Bulletin 37 eqn (5.2) is an example of such a 50% design reliability relationship for subgrade failure.

The incorporation of shift factors will require the design reliability traffic multipliers (Moffatt et al. 1998) to be recalculated because, as discussed by Moffatt et al. (1999), the use of shift factors to relate laboratory or predicted performance to field performance should not be considered in isolation to other pavement design issues. The next edition of the Austroads Pavement Design Guide will contain methods which allow cognisance to be taken, during the design stage, of the relative reliabilities of
various pavement configurations and the reliability level desired for a given road project. These assumptions are based upon the use of the current asphalt fatigue relationship (Shell) without the use of a shift factor. As such, the reliability procedures already have in place a means of allowing for the difference between predicted and field performance. Any future work addressing the issue of shift factors will therefore require a revision of the design reliability work, lest allowance for differences between estimated and field performance are made twice within the overall design process.

The Chapter 8 example design charts (Moffatt and Jameson 1998) and the design examples in Appendices G and H will also need to be revised.

The design reliability issue is of particular interest. The average design reliability of asphalt on unbound granular pavements is considered to be 85% (Moffatt et al. 1998). For traffic loadings exceeding $10^6$ ESAs, the Austroads (1992) procedures indicate that life is limited by asphalt fatigue rather than by subgrade deformation. This suggests that the Shell laboratory asphalt fatigue relationship used to calculate the asphalt fatigue lives is conservative.

Jameson and Robinson (1999) estimated that, by increasing the Shell laboratory fatigue life predictions by a factor of 3.3, the average reliability of the Austroads design process would be reduced from 85% to 50%. This suggests that, for asphalt pavements on granular based, the use of a shift factor of 3.3 with the Shell laboratory fatigue relationship would result in an average design reliability of 50% for the Austroads pavement design procedure. (Full depth asphalt pavements have a design reliability of 95%, and the effective shift factor for these pavements is about 4.0.)

In revising the Austroads Guide two possible options are:

- use the proposed reliability procedures which are based on the current asphalt fatigue relationship which effectively includes an asphalt fatigue shift factor of 3-4, or
- based on findings of the Jameson (1999) study, explicitly provide an asphalt fatigue shift factor and revise the design reliability procedures.

CONCLUSIONS

This paper has summarised the findings of an Austroads/AAPA-sponsored accelerated loading trial conducted to examine the fatigue performance of asphalt mixes. Although over 500,000 cycles of the ALF 80 kN dual-wheel load were applied to four test sites during two stages of testing, no fatigue failure could be induced in any section, with the only distress observed being surface deformation related to shear failure of the underlying unbound base and sub-base layers. The results of an extensive series of response-to-load testing using H-bar strain gauges were also inconclusive because, whilst the effect of temperature on asphalt strain was clearly demonstrated, the results were counter intuitive in terms of the effect of load magnitude and type, and loading rate, on pavement response. Unfortunately the usefulness of the strain gauge data was severely limited because of the large corrections that needed to be applied to take account of variations in pavement temperature.

An extensive series of laboratory testing was also undertaken on both laboratory-prepared samples and field specimens. The fatigue performance of the Class 600 mix was significantly superior to that of the Class 320 mix at $10^\circ$C, particularly at the lower initial strain level ($\leq 400 \mu$). The fatigue performance of the Class 320 mix tested in Stage I was also superior to that of the Class 320 mix tested in Stage II by at least by a factor of two. As observed with the fatigue results obtained at $20^\circ$C, the fatigue performance of the Class 320 mix tested in Stage I and the Class 600 mix at $10^\circ$C was also similar. The reasons for the anomalous stiffness and fatigue characteristics of the
Class 600 mix are not clear, and detailed characterisation of the Class 600 binder may be required.

Following the completion of the trial, the performance data were evaluated, along with data obtained from other accelerated loading trials and in-service field trials, to assess whether an asphalt fatigue shift factor should be adopted in the Austroads pavement design system. A number of trials were evaluated, including a recent trial conducted in the USA and the OECD DIVINE Project. The results were inconclusive in that some studies supported the introduction of shift factors whilst others did not.

If it were decided to adopt an asphalt fatigue shift factor in the revised Austroads Pavement Design Guide, then presumably it will be a factor that results in the Austroads design method having an average reliability of 50%, rather than the current situation, where the average reliability of asphalt pavements is 85-95%. Given this, consideration would need to be given to adjusting the subgrade failure and cemented materials fatigue relationships so that they also result in the design procedures for these two distress modes having average design reliabilities of 50%. The incorporation of shift factors will also require the design reliability traffic multipliers to be recalculated because the use of shift factors to relate laboratory or predicted performance to field performance should not be considered in isolation from other pavement design issues.

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