THE DEVELOPMENT OF A FATIGUE TRANSFER FUNCTION FOR *IN SITU* FOAMED BITUMEN STABILISED PAVEMENTS

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SUMMARY

This report reviews the results of research undertaken into the stiffness and fatigue performance of *in situ* foamed bitumen stabilised pavement materials at various sites in the Cities of Canning and Gosnells in Western Australia. The aim of the research was to assess if a design relationship could be developed to predict the fatigue life of *in situ* foamed bitumen stabilised pavements and if the visco elastic properties of the bitumen binder were reflected in the stiffness and fatigue performance.

1 INTRODUCTION

The City of Canning has some experience with the *in situ* foamed bitumen stabilisation process. In January 1999, four pavement sections were rehabilitated using this process and following the success of these projects, this process has become the major method for the rehabilitation of moderate to heavy trafficked roads throughout the City.

Over subsequent years, slabs from completed pavements were extracted and sent to the ARRB Transport Research laboratory in Melbourne for flexural fatigue testing. Fatigue testing for Orrong Road and Willeri Drive was undertaken by SAMI and Pioneer Road Surfaces respectively in 2005. In addition, extensive MATTA testing was undertaken on cores cut from the pavement. The results of these early investigations have been previously reported at the 20th ARRB conference in 2001 (Leek 2001)

The aim of the research was to assess:

- A approach to predict the fatigue life of *in situ* foamed bitumen stabilised pavements
- If the flexural modulus was temperature dependent
- If the fatigue life was temperature dependent
- If the asphalt fatigue transfer function was applicable to *in situ* foamed bitumen.

Fatigue life due to applied loading rate was not part of this study.

The slabs were cut into beams for fatigue testing in the IPC asphalt beam fatigue test machine. The aim was generally to test beams from each site at three different strain levels at two different temperatures (20^{0}C and 30^{0}C). In all cases beams were recovered from within the middle of the upper 100 mm layer of stabilised material with the exception of one site, where testing was undertaken at 20^{0}C only, but at three different levels in the pavement. In some instances however beams were tested at 20^{0}C only.

The results of the original investigation indicated that the asphalt fatigue equation (AUSTROADS 1992 Pavement design guide) used during the design process was probably not applicable to Foamed Bitumen Stabilised pavements, but owing to the small sample size, no definitive trend could be established.

The results of the second round of testing were used to supplement previous testing and develop a proposed equation to predict pavement fatigue life based on applied strain. This was reported at the IPWEA State conference in Perth 2002 (Leek 2002). This equation, developed such that 95% of all test results exceeded the predicted fatigue life was:

$$N = (649/\mu\varepsilon)^{8.5} \quad (1)$$

The 95th percentile was selected as that is the value recommended in VICROADS Technical Bulletin 37 (Sept 93) as being applicable to main roads. However it was recognised that this equation would need to be refined as further testing was undertaken.

Following the success of the early works, *in situ* foamed bitumen stabilisation has continued to play an important role in pavement rehabilitation in the City of Canning, where at December 2012, 33 sections totalling 166,000 m² had been rehabilitated using this method. A list of pavement sections is shown in Appendix A.

Over subsequent years, slabs have been extracted from completed road pavements and tested for fatigue performance to add to the data base. In 2008 after 9 years of service, two pavement sections, both of which demonstrated early transverse cracking, were sampled for repeat fatigue testing, and three sections were sampled for repeat Indirect Tensile Modulus Testing.

This repeat testing has influenced previous predictions regarding the design life of *in situ* foamed bitumen stabilised pavements, and further knowledge regarding the effect of shift factors between laboratory testing and field performance has been considered.

As will be discussed, only for the original projects was a laboratory characterisation to predict the properties of the material and to optimise binder content undertaken. Following this, it was decided that due to pavement variability and cost, characterisation and optimisation of binder content was not to be undertaken on future works and that performance would be based on past experience.

2 DESIGN AND CONSTRUCTION OF TEST PAVEMENTS

2.1 MATERIAL CHARACTERISATION

In 1998 when the first projects were proposed, samples were collected from High Road and sent to Mobil in Melbourne for a mix design to be undertaken.

A mix design should include laboratory trial mixes at different bitumen contents to determine the optimum bitumen content that will be used in the pavement construction. However what was received was a design modulus value at 4% bitumen and 2% hydrated lime. Only one sample was prepared, The testing indicated an indirect tensile modulus of 4,730 MPa dry and 3,080 MPa wet at 4% bitumen content and 2% hydrated lime content. Testing of a similar pavement from another location resulted in values of 3,130 MPa and 2050 MPa. A conservative presumptive flexural modulus of 2,400 MPa for the stabilised layer was adopted, and used as a seed value for pavement thickness design using CIRCLY, with the asphalt fatigue equation used to determine the allowable repetitions.

The characterisation of pavements is difficult due to the extremely variable nature of the existing pavements as demonstrated by the pavement profiles of High Road and Nicholson Road as shown in Table 2.1 and Table 2.2. In this table, the term roadbase is applied to a crushed granite base quality material with a maximum size of 20 mm. The term limestone refers to "Tamala Limestone", a locally quarried material from naturally occurring limestone with a $CaCO_3$ content of greater than 60%. This material generally has a maximum size of 75 mm, but is gap graded and contains mostly sand sized particles. Gravel is laterite gravel sourced from the Darling Scarp and is composed of rounded uncrushed aggregates.

Table 2.1: Existing pavement profile High Road

Chainage (m)	Surfacing (mm)	Base Layer (mm)	Subbase Layer (mm)	Subbase Layer (mm)	Total (mm)
East Bound Carriageway					
20	30 AC	30 Roadbase	280 Gravel		340
120	25 AC	65 Roadbase	195 Limestone		285
220	15 AC	65 Roadbase	230 Limestone		310
320	25 AC	75 Roadbase	220 Limestone	160 Gravel	480
420	30 AC	65 Roadbase	215 Limestone		310
520	25 AC	70 Roadbase	105 Limestone		200
620	25 AC	85 Roadbase	155 Limestone	225 Gravel	490
720	15 AC	25 Roadbase	270 Limestone		450
820	30 AC	60 Roadbase	180 Limestone	140 Old mix	410
920	15 AC	85 Roadbase	230 Limestone		330
1020	35 AC	85 Roadbase	370 Limestone		490

For all subsequent pavements, a bitumen content of between 3.5% and 4.0% has been generally adopted, but trials of 3.0% have been undertaken. After observation of transverse cracking on the initial projects, it was decided to reduce the lime from an initial 2% to 1% hydrated lime or 0.8% quicklime. Since this change was made, no further cracking has been observed, but it is stressed that this is the case for the pavement materials and lime used applicable to these specific sites where low plasticity materials are found. This may not be the case for all pavement materials.

Table 2.2: Existing pavement profile Nicholson Road.

Chainage	Surfacing	Base Layer	Subbase Layer	Subbase Layer (mm)	Total (mm)

(m)	(mm)	(mm)	(mm)		
82	70 AC	120 Limestone	100 Stabilised	210+ Limestone	500+
108	75 AC	25 Roadbase	230 Limestone		330
128	25 AC	80 Roadbase	70 Stabilised	280+ Limestone	455+
152	110 AC	210 Limestone			320
178	20 AC	100 Roadbase	240 Limestone		360
198	80 AC	300 Limestone			380
218	50 AC	35 Roadbase	30 Old Mix	335 Limestone	450
267	40 AC	150 Ferricrete	50 Roadbase	165 Limestone	405
270	35 AC	150 Ferricrete	310 Limestone		495
310	40 AC	150 Ferricrete	145 Roadbase	320+ Limestone	655+

3 DESIGN METHOD

3.1 BACKGROUND

The design of an *in situ* foamed bitumen stabilised pavement requires the following inputs:

- Subgrade modulus (or CBR)
- Flexural modulus and Poissons Ratio for stabilised layer
- Flexural modulus and Poissons Ratio of asphalt surface
- Fatigue performance criteria for stabilised layer
- Subgrade performance criteria
- Modelling assumptions for the layer interfaces (i.e. rough or smooth)
- Design traffic loading and heavy vehicle annual growth
- Traffic constant
- Design Reliability Factor

A fatigue performance criterion for asphalt surface is not listed, as this will not be the primary failure mode. These parameters are input into the CIRCLY program, which calculates the strain in each pavement layer and the compressive strain on the subgrade when the pavement is subjected to the design standard axle for an assumed pavement thickness. From the computed results the level of strain and the performance fatigue criteria one can estimate the allowable number of load repetitions, which should exceed the actual traffic loading.

In addition, some judgement is required to allow for a shift factor to account for the difference between the number of repetitions of a load under accelerated laboratory testing and the allowable number of repetitions in service where considerable rest periods exist between load applications.

The required modulus required for the CIRCLY program is not stated either in AUSTROADS or the CIRCLY manual. It is considered that as the pavement is acting as a beam, the flexural modulus should be adopted.

The 2001 AUSTROADS Pavement Design Guide (Final Daft) for Public Comment (AP-T10) Section 6.4.3 makes specific reference to input modulus being the resilient modulus, but Section 6.4.5.2 states that the flexural fatigue testing of asphalt beams is preferred as the test reproduces actual behaviour of an asphalt layer under wheel loading more closely than any other method. Thus the actual modulus value to be used is not clearly stated.

The AUSTROADS 2008 Pavement Guide to Pavement Technology Part 2: Pavement Structural Design incudes in the constants, a conversion from resilient modulus to flexural modulus (personal communication Geoff Jameson, Senior Research Scientist AARB Group).

For bituminous materials, the modulus is not a single value, but varies with traffic speed (strain rate loading) and pavement temperature. The modulus values used therefore need to be specified at a specific or reference speed (rise time) and temperature. The Weighted Mean Annual Pavement Temperature (WMAPT) for Perth (30^0C) was used for the design of the pavement sections using the *in situ* foamed bitumen stabilisation method. Therefore a knowledge of the variation of properties with temperature was considered important.

The transfer function for fatigue performance criteria adopted by most designers is the AUSTROADS asphalt model developed by Shell in 1978, and is based on work by Maccarrone, Holleran and Leonard. (Maccarrone *et al.*, 1993). The original pavement design method used by the City of Canning adopted this procedure. The asphalt fatigue relation is as follows:

$$N = \left[\frac{6918(0.856V_B + 1.08)}{S_{mix}^{0.36} \mu\varepsilon} \right]^5 \qquad (2)$$

where:
N = allowable number of standard load repetitions
V_B = Bitumen percentage by volume in mix
S_{mix} = mix stiffness (modulus) MPa
$\mu\varepsilon$= tensile strain induced by load (microstrains).

3.2 MODIFIED FATIGUE EQUATION

Testing undertaken by the City of Canning has shown that this model may be optimistic in the prediction of fatigue performance. In 2002, the City commenced using the following equation as a 95th percentile equation with a reliability factor of one to design *in situ* foamed bitumen stabilised pavements.

$$N = (649/\mu\varepsilon)^{8.5} \qquad (3)$$

After further testing as more data became available, the fatigue relation was again modified and the equation currently used is:

$$N = (1734/\mu\varepsilon)^{6.2} \qquad (4)$$

4 LABORATORY TESTING OF FIELD SAMPLES FOR FATIGUE, FLEXURAL AND RESILIENT MODULUS

4.1 METHOD FOR FATIGUE AND FLEXURAL MODULUS

Slabs were cut from a single location in a selection of roads treated and sent to the ARRB Transport Research laboratory in Melbourne for fatigue testing. Four slabs of 0.5 m square were cut in each location, and from three of these three beams were cut to 400 mm x 64 mm x 50 mm, the size required for the fatigue tests. The testing apparatus showing one of the beams is shown in Figure 4.1.

For all but the Bannister Rd (Forum Ave) site, the beams were cut at a depth of 64 mm to 114 mm from the upper surface, the top 40 mm of which was the asphalt wearing course. Thus the beam location in relation to the top of the stabilised layer extended approximately 24 mm to 74 mm from the surface of the layer. These beams were tested in triplicates at three strain levels and at temperatures of 20^{0}C and 30^{0}C to determine the relationship between fatigue life and strain levels, and the relationship between fatigue life and temperature.

In the case of the Bannister Rd (Forum Ave) site, the beams were cut at 25 mm to 75 mm, 125 mm to 175 mm and 225 mm to 275 mm from the top of the stabilised layer. The purpose of this test was to determine if the fatigue life decreased with depth, as it was previously shown that both compacted density and modulus decreased with depth (Leek, 2001).

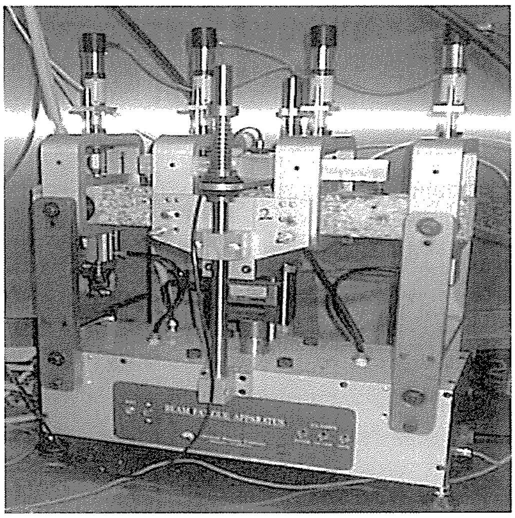

Figure 4.1: Beam testing apparatus showing a foamed bitumen beam undergoing fatigue testing.

This test was undertaken only at 20^{0}C and at one strain level. Two additional beams were cut at 125 mm to 175 mm position and tested at 20^{0}C at three strain levels to increase the data for the determination of the fatigue/strain relationship.

The fatigue testing was undertaken at 20^{0}C, using draft test currently under development as an Australian Standard. The test value is obtained by determining the initial flexural modulus and applying a cyclic load to achieve given strain level until the flexural modulus reduced to half its initial value. The number of load repetitions to achieve a reduction of 50% of the original flexural stiffness is taken as the fatigue life of that sample at the particular strain level.

In addition to this value, it was desired to understand the behaviour of the material with regard to the onset of cracking. It has been postulated (Pronk, 1995) that the energy dissipated during each loading cycle will remain more or less constant until the onset of cracking. As a secondary analysis, the point at which cracking began was sought by plotting the energy dissipated per cycle and fitting a linear regression to the data either side of the discontinuity.

4.2 TESTING PROGRAM FOR FATIGUE AND FLEXURAL MODULUS

The test method employed to determine the fatigue properties is as follows:

- Continuous haversine loading at a frequency of 10 Hz.
- Controlled strain (strain level was varied so as to span failure at one million cycles).
- Third point loading which theoretically induced a constant strain in the mid third of the beam.
- A test temperature of either 20°C or 30°C, controlled to within 0.5°C.
- Beam samples of 50 mm deep by 64 mm wide and between 400 and 450 mm long.

Testing was to continue until failure in the beam occurred. Failure in this instance is defined as the point when the flexural stiffness of the beam decreases to half the initial flexural stiffness measured at the 50th cycle. The initial modulus is taken as the modulus at the 50th cycle, and the failure point is the number of cycles recorded when the stiffness reached 50% of the initial value (at 50 cycles). In some instances, failure was not achieved as the test was discontinued by the operator due to the excessive duration of the test. The apparatus is able to apply 864,000 loading cycles per day and a number of the tests continued past 5 days.

Some tests were terminated before failure was reached, as the decrease in stiffness indicated that the test would proceed for an excessively long time. In these cases the results were obtained by extrapolation.

From the three basic measurements, (displacement, load and time) a number of other variables can be estimated and these include:

- Peak stress per loading cycle.
- Peak strain per loading cycle.
- Flexural stiffness (a function of the peak stress divided by the peak strain).
- Phase angle (a measure of the viscoelastic nature of the material and is a function of the time delay between the occurrence of the peak load and the occurrence of the peak displacement).
- Dissipated energy (a measure of the energy losses in the system due to viscous behaviour and believed to be related to crack initiation).

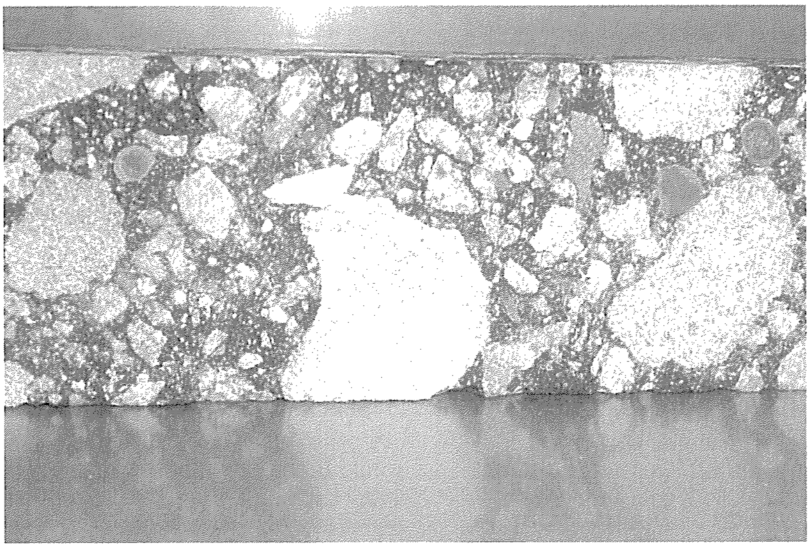

Figure 4.2: Foamed bitumen beam trimmed for fatigue.

In this testing programme, a set of three specimens was used. It is recognised in the testing of beams for flexure or fatigue that large variations are encountered, and for statistical analysis, some authorities recommend testing eight replicate specimens. However this is extremely costly, time consuming and was not possible due to budget constraints.

A further cause for imprecision in the analysis of foamed bitumen beams is the presence of large (70 mm) aggregate particles in the mix. When a beam is cut (50 mm x 60 mm) a single aggregate piece can make up almost the entire cross section of the beam, resulting in stress concentrations as shown in Figure 4.2.

4.3 RESULTS OF TESTING FOR FATIGUE AND FLEXURAL MODULUS

Details of the test results are given in the individual ARRB Transport Research Contract Reports Fatigue Properties of Bitumen/Lime Stabilised Materials – Parts A, B, C, D & E and Stage 3 and Stage 4 produced in stages between September 2000 and August 2004. Fatigue testing for Orrong Road and Willeri Drive was undertaken by SAMI and Pioneer Road Surfaces respectively in 2005.

As the traditional method of interpreting failure for bound materials is to consider fatigue failure as the point where modulus at failure is 50% of the initial modulus, only those values are repeated here.

Testing was undertaken at two different temperatures. The results of testing at 20^0C are given in Table 4.1 and at 30^0C are given in Table 4.2. The cycles to failure in Table 4.1 and Table 4.2 are either the mean of the actual or extrapolated cycles to failure of the triplicate specimens unless otherwise noted. Three beams, two from the latest testing round and one from the 1999 testing round exhibited vastly reduced initial modulus, possibly due to damage in preparation and these beams were excluded from the results.

Included in the Table 5.1 and Table 5.2 are the estimated cycles to failure as predicted by the asphalt fatigue equation. (AUSTROADS 1992). The asphalt fatigue equation uses the initial modulus and the bitumen content of the mix by volume to determine the fatigue life according to the following relationship used in AUSTROADS 2008 Pavement Guide to Pavement Technology Part 2: Pavement Structural Design, but originally developed by Shell in 1978:

$$N = RF\left[\frac{6918(0.856V_B + 1.08)}{S_{mix}^{0.36}\mu\varepsilon}\right]^5 \tag{5}$$

where:
N = allowable number of standard load repetitions
V_B = Bitumen percentage by volume in mix
S_{mix} = mix stiffness (modulus) MPa
RF= Reliability factor
$\mu\varepsilon$= tensile strain induced by load (microstrains).

The bitumen contents given in the Table 4.1 and Table 4.2 used to calculate the allowable repetitions have been calculated from the as constructed bitumen application rate as, due to the presence of bitumen in the original asphalt surface, conventional extraction gives an inflated indication of binder added.

In order to simplify comparisons, the Log of the cycles is used. As testing was undertaken at 20^0C in 1999 on High Rd and John Street (Leek 2001), these results are included in Table 4.1. A third road was also tested in 1999 (Nicholson Rd) but this sample was found to have been located on a continuous construction joint and was not reliable.

A value of greater than 1 for the ratio N/Np indicates that the measured laboratory fatigue life of the stabilised material is greater than that predicted by the asphalt fatigue equation.

In order to ascertain if there was a reduction in fatigue life with depth, the samples cut from Bannister Rd (Forum Ave) were tested at three different levels, but at one strain level and one temperature only. These results are shown in Table 4.3.

In considering the results, it must be noted that there are two distinct pavement materials represented. Bannister Rd, Orchard Rd, Vulcan Rd and High Rd are composed of a mixture of asphalt, roadbase and limestone, where all of the component materials are crushed quarried materials.

John Street and Railway Parade are composed of a mixture of asphalt and natural rounded lateritic gravels, where the crushed quarried component is made up only of the material contained in the original weathered thin asphalt overlay. The flexural modulus for these different broad material types would be expected to show major differences.

In 2008, two sections of pavement were selected for a repeat round of fatigue tests, and three sections for repeat Indirect Tensile Modulus Testing. The fatigue testing was undertaken by ARRB Transport Research and the Indirect Tensile Modulus Testing by ASLAB.

The results of the repeat fatigue tests undertaken on Bannister Road, Magnet to Baile and Vulcan Road, Magnet to Coulson and those of the original investigation for the same road sections are shown in Table 4.4.

The results of the repeat testing indicate that the flexural modulus may have increased marginally over the 9 year period, and this is considered a reliable result. There appears also to have been an increase in remaining fatigue life, but given the variability of the fatigue part of the test, the variation may not be significant.

Table 4.1: Results of fatigue testing on beams and comparison to asphalt fatigue equation at 20^{0}C (Ei is the mean of the individual test beams).

Road section	Testing at 20^{0}C					
	Test Results			Asphalt Fatigue Comparison		
	Test Strain (με)	E_i Initial Flexural Modulus (MPa)	Cycles to Failure Ei/2 Log (N)	Volume Bitumen in Mix (%)	Predicted Cycles to Failure Log (Np)	Ratio N/Np
Bannister Road (Baile Road)	100	8440	9.88	7.8	6.58	1991
	150	8360	6.17	7.8	5.71	2.90
	175	8550	7.05	7.8	5.36	49.48
	200	7790	5.57	7.8	5.14	2.70
Railway Parade	125	5830	6.47	7.2	6.24	1.71
	175	5380	5.26	7.2	5.57	0.49
	225	4600	5.43	7.2	5.15	1.92
Vulcan Road	150	7980	6.49	7.9	5.77	5.27
	175	7580	5.88	7.9	5.47	2.55
	200	7670	5.58	7.9	5.17	2.54
	250	7680	4.77	7.9	4.69	1.21
Orchard Road	150	8060	8.75	5.8	5.19	3590
	175	6790	6.86	5.8	4.99	73.41
	200	8170	4.61	5.8	4.56	1.12
Bannister Rd (Forum Avenue)	125	8580	12.05	7.8	6.08	925788
	175	9388	5.60	7.8	5.27	2.14
	225	7790	5.61	7.8	4.88	5.34
High Road	126	6330	6.02	7.8	6.30	0.52
	190	8060	6.52	7.8	5.22	19.81
	251	5140	3.81	7.8	4.97	0.07
John Street	126	2480	7.12	6.9	6.81	2.05
	252	1820	6.33	6.9	5.55	6.08
	400	1380	4.19	6.9	4.76	0.27
Bannister Road (South Street)	125	5707	8.34	8.5	6.57	58.11
	175	4880	5.58	8.5	5.97	0.41
	225	5670	6.21	8.5	5.30	8.06
Nicholson Road (Woodloes Street)	125	3990	7.28	8.5	6.85	2.69
	175	4230	4.96	8.5	6.08	0.08
	225	4230	4.96	8.5	5.53	0.27
Nicholson Road (Spencer Road)	125	6275	8.51	6.6	6.02	308
	225	10287	6.09	6.6	4.36	53.95
High Road (Velgrove Avenue)	100	1950	6.59	7.8	7.72	0.07
	150	3038	4.75	7.8	6.49	0.02
	175	3010	5.33	7.8	6.16	0.15
High Road (Velgrove Avenue) Class 50 bitumen no foaming agent	100	5060	6.53	7.8	6.97	0.36
	150	2943	5.69	7.8	6.52	0.15
	175	3483	4.53	7.8	6.05	0.03
Orrong Road	100	7536	6.83	6.4	6.29	3.44
	150	7397	5.32	6.4	5.43	0.78
	175	8093	4.82	6.4	5.02	0.64
Willeri Drive	100	5343	6.81	8.8	7.18	0.43
	150	6314	6.31	8.8	6.16	1.39

	175	5778	6.22	8.8	5.90	2.08

Table 4.2: Results of fatigue testing on beams and comparison to asphalt fatigue equation at 30^{0}C.

Road section	Testing at 30^{0}C					
	Test Results			Asphalt Fatigue Comparison		
	Test Strain (με)	Ei Initial Flexural Modulus (MPa)	Cycles to Failure Ei/2 Log (N)	Volume Bitumen in Mix (%)	Predicted Cycles to Failure Log (Np)	Ratio N/Np
Bannister Road (Baile Road)	125	6880	6.23	7.8	6.26	0.94
	175	6560	4.48	7.8	5.56	0.08
	225	7420	5.19	7.8	4.92	1.86
Railway Parade	125	4400	6.94	7.2	6.46	3.04
	175	4260	5.35	7.2	5.75	0.40
	225	4340	4.36	7.2	5.19	0.15
Vulcan Road	125	6310	6.11	7.9	6.35	0.58
	175	7820	5.87	7.9	5.45	2.64
	225	6990	5.16	7.9	4.99	1.48
Orchard Road	125	7310	8.67	5.8	5.67	1006
	175	5283	8.07	5.8	5.19	758
	225	4540	6.85	5.8	4.76	122
Orrong Road	100	6223	6.60	6.4	6.44	1.44

Table 4.3: Results of fatigue testing on beams and comparison to asphalt fatigue equation at 20^{0}C at varying levels in pavement.

Road section	Testing at Different Levels in Pavement at 20^{0}C						
	Test Results				Asphalt Fatigue Comparison		
	Test Strain (με)	Level in Pavement from Top (mm)	E_i Initial Flexural Modulus (MPa)	Cycles to Failure Ei/2 Log (N)	Volume Bitumen in Mix (%)	Predicted Cycles to Failure Log (N_p)	Ratio N/Np
Bannister Road (Forum Avenue)	175	25-75	11020	4.76	7.8	5.16	0.40
	175	125-175	7760	5.60	7.8	5.43	1.47
	175	225-275	5290	5.01	7.8	5.73	0.19

Table 4.4: Original and repeat fatigue test results for Vulcan and Bannister Road.

Road section	Testing at 20^{0}C					
	Test Results (1999)			Test Results (2008)		
	Test Strain (με)	E_i Initial Flexural Modulus (MPa)	Cycles to Failure Ei/2 Log (N)	Test Strain (με)	E_i Initial Flexural Modulus (MPa)	Cycles to Failure Ei/2 Log (N)
Bannister Road. (Baile Road)	100	8440	9.88			
	150	8360	6.17	150	9024	6.50
	175	8550	7.05			
	200	7790	5.57	200	8976	6.18
				250	8390	4.69
Vulcan Road	150	7980	6.49	150	9069	6.47
	175	7580	5.88			
	200	7670	5.58	200	8375	5.26
	250	7680	4.77	250	9931	5.43

Note: The results for E_i and log (N) are triplicate mean values

4.4 TEST RESULTS FOR RESILIENT MODULUS

In addition to undertaking additional fatigue testing, additional cores have been extracted from the pavement to add to the available data on resilient modulus and any relationship between this value and flexural modulus. Resilient or

Indirect Tensile Modulus is a simple test that can be performed by a number of laboratories. Details of the numbers of cores tested are given in Table 4.5.

The results of resilient modulus testing of 178 cores extracted from the 0-100 mm level over 25 separate sites are shown in Table 4.6. Not all cores were tested at all temperatures. Where enough core could be extracted, cores were tested at 0-100 mm, 100-200 mm and greater than 200 mm from the surface of the stabilised layer.

Table 4.5: Details of cores tested for ITM.

Temperature	Number of Cores Tested Depth below surface of finished stabilised pavement		
	0 – 100mm	100 – 200mm	200 – 300mm
20^0C	72	57	14
25^0C	104	74	18
30^0C	87	62	17
35^0C	19	15	1

Table 4.6: Indirect Tensile Modulus for pavements originally comprising crushed granular materials.

Depth from Surface			0-100		100-200	200+
Temp ^{0}C	Rise Time	Mean Modulus (MPa)	Standard Deviation	90th Percentile Value	%age of Top Layer Modulus	%age of Top Layer Modulus
20	25	10562	4262	5525	80.60%	51.50%
	50	9540	3893	4903		
	100	8519	3542	4406		
25	25	9544	3737	5454	74.40%	53.70%
	50	8505	3476	4453		
	100	7465	3271	3740		
30	25	7656	3508	3852	75.00%	50.20%
	50	6710	3064	3266		
	100	5764	2659	2623		
35	25	9403	3598	6064	97.70%	25.50%
	50	8208	3322	5118		
	100	6945	3121	4086		

Table 4.7: Results of Indirect Tensile Modulus tests at 9 Years.

Test temperature	Rise Time	1999			2009			9 Year Difference
		Mean	Std Dev	95 % ile	Mean	Std Dev	95 %ile	
	ms	MPa	MPa	MPa	MPa	MPa	MPa	%
Testing at 20^0 C	25	13880	4099	9439	13722	4428	8871	94.0%
	30	13518	4090	9085	13487	4375	8705	95.8%
	50	12504	4070	8095	12829	4231	8242	101.8%
	75	11699	4056	7309	12307	4122	7874	107.7%
	100	11128	4048	6751	11936	4048	7612	112.8%
Testing at 25^0 C	25	13115	3987	9174	11919	4208	7507	81.8%
	30	12850	4096	8849	11712	4150	7376	83.4%
	50	12109	4445	7917	11132	3992	7005	88.5%
	75	11520	4995	7169	10672	3870	6687	93.3%
	100	11102	4995	6639	10346	3785	6461	97.3%

5 ANALYSIS OF RESULTS

5.1 FATIGUE RELATIONSHIP

In the analysis of the results, it is important to recognise the limitations of the fatigue test. As mentioned previously, large aggregate pieces can be included in a test beam, which represents only a small cross section of the total pavement. Stress concentrations will develop when this condition occurs, which can, and most probably will, reduce the actual fatigue life of the beam in the test.

Many beams did exhibit failure less than that predicted by using the AUSTROADS asphalt equation, but some beams defied this trend and exhibit an enormous increase in the fatigue life over that expected from an asphalt beam of similar bitumen content and modulus.

Thus the beams which exhibited shortened life may have done so for two possible reasons. The first possibility is that the beam had a weakness in manufacture or transport, or developed a stress concentration at the point where a single large aggregate piece was cut through. The second possibility was that the beam is truly representative of the material under the test conditions.

In the case of beams exhibiting significantly increased fatigue life, no explanation could be found by the authors other than that beam was resistant to fatigue failure. It should be noted here that the applied minimum test strain is higher that the design strain in the actual pavement designs (typically around 80 $\mu\varepsilon$) and thus equations require extrapolation outside the test limits. In those cases where the test results indicate that the fatigue life is shorter than would be predicted by the asphalt equation, the beams in the triplicate were reasonably consistent.

Table 4.1 shows the results of fatigue testing undertaken at 20^0C. Each entry represents the mean value of the beams tested at that strain level although it should be noted that there was often considerable variation between individual test results. In most cases beams are tested in triplicate, but in the case of Bannister Road, an additional strain level was included and samples were tested in pairs, in Bannister Road (Baile Road) an additional beam was tested at a lower strain level and in High Road (Velgrove Avenue) one sample at 100 $\mu\varepsilon$ broke. Sometimes if additional samples could be cut and an additional beam was tested.

In 27 triplicates tested, the test results indicated better performance than predicted by the asphalt model. The fatigue life in these cases ranged from 1.12 to 925,000 times that predicted by the asphalt model.

In 16 cases however the triplicate means were less than that predicted by the asphalt model, ranging from a factor of 0.02 to 0.78 times that of the asphalt model.

Table 4.2 shows the results of fatigue testing undertaken at 30^0C. Again generally triplicates were tested, but in one case one beam broke and in another an additional beam was added. In this case, 8 triplicates performed better that the asphalt model, ranging from 1.44 to 1006 of the fatigue life predicted by the asphalt model, and 5 triplicates performed worse ranging from 0.08 to 0.94 of the fatigue life predicted by the asphalt model.

Table 4.3 shows the results of testing one site (Bannister Rodd at Forum Avenue) at one strain level at three different levels in the pavement. Here 2 of the 3 results indicated that the asphalt model over–predicted fatigue life.

Whilst it may be justifiably argued that the low results could result from stress concentrations in those particular beams, in 6 cases, the triplicate samples were such that picking the most optimistic results from the triplicates would not result in the test result exceeding the asphalt prediction.

Overall of the 56 triplicate samples analysed, 35 triplicates performed better than the asphalt model, and 21 performed worse than the asphalt model.

It was apparent during the testing that the stabilised material behaved in two different manners. Of the beams tested, 25% behaved as a highly fatigue-resistant cement treated crushed rock, and 75% behaved as would be expected of an asphalt beam. No property could be determined that would explain this apparent difference in performance. However those beams exhibiting the properties of highly fatigue resistant cement treated crushed rock withstand strain levels that CTCR could not.

In analysing the results, there is considerable scatter of data, as would be expected considering the nature of the stabilised materials with the inherent variability encountered in existing pavements. The number of sample sites is small, and the failure modes of the beams during test were not consistent.

Due to the high cost of testing and limited funds, only one site was tested at multiple depths to determine if the reduction of density with depth effects fatigue performance. This is shown in Table 4.3. This test was not conclusive, and the fatigue performance of the bottom layer, whilst well short of that predicted by the asphalt model, is not the worst performing case compared to some other locations.

The testing showed that the performance of individual beams varied widely. Whilst bitumen content and stiffness would be considered to contribute to fatigue life, and test temperature would be expected to affect stiffness, due to the scatter of individual results, there was no significant relationship between modulus and fatigue life or bitumen content and fatigue life. Therefore a simplified equation is proposed in line with that used for cemented materials, subgrades and indeed bituminous materials, but excluding specific reference to bitumen content and stiffness.

The results of the individual beam tests were plotted and using the inbuilt curve fitting functions in EXCEL, a best fit using the power function was determined. The power function was selected as this is the basis of the relationship used for predicting the performance of other pavement materials. Due to the statistical insignificance of test temperature, it was considered reasonable to bulk all results together to maximise the data available for determination of a fatigue relationship. The following relationships were generated by the EXCEL curve fitting function:

$N = (1743/\mu\varepsilon)^{5.7}$ where individual beam results were analysed

$N = (1734/\mu\varepsilon)^{6.2}$ where mean of triplicate beam results were analysed

where:

N = allowable number of load repetitions of strain level με

με= tensile strain induced in bottom of stabilised layer pavement by applied load

The R^2 of these equations is <0.2, and thus the equations are not presented as in any form being reliable, but based on the assumption that fatigue life will decrease with increasing strain, with the variability of material, designers need some method to select a pavement thickness.

This trend line is a best fit relationship, such that many individual results fall either side of the equation, and thus if this equation were applied, 50% of the pavement would be likely to fail by fatigue cracking.

However consideration needs to be given to the inherent difference between accelerated loading test conditions, and that loading regime that actually occurs in the pavement. Healing of bituminous pavements is thought to occur between repeated loads, and this does not occur under accelerated loading conditions. A shift factor of at least 10 may be applicable to convert to field life, but in the thicker layer high temperature situation applicable to Australian foamed bitumen pavements, a shift factor of up to 20 is more applicable. (Claessen, 1997)

Given the size of the aggregate and associated stress concentrations when cut to a such a small size as in the beam fatigue test, in the case of foamed bitumen pavements, the best fit curve of Figure 5.2 would be sufficiently conservative to use as a design model, allowing no shift factor for the conservative nature of the test regime.

Also shown are the results generated by each of the beams if the modulus and bitumen volume were entered into the asphalt fatigue equation (blue line). The application of triplicate means gives more conservative results, due to the moderating effect this has on extreme values.

Although field data in Queensland was limited when the design equation was suggested, there was enough data to indicate that the primary distress mechanism of foamed bitumen stabilised pavements was fatigue failure of the stabilised layer (Jones and Ramanujam 2004).

The individual results and triplicate mean results of the fatigue beam tests undertaken on the samples extracted from the 9 year old pavements were plotted on the same chart as the original tests results. These are also shown in shown in Figure 5.1 and Figure 5.2 in green. The results for the 9 year tests seem to compare reasonably with those at year zero, that is, there does not appear to be a significant shift in remaining fatigue life.

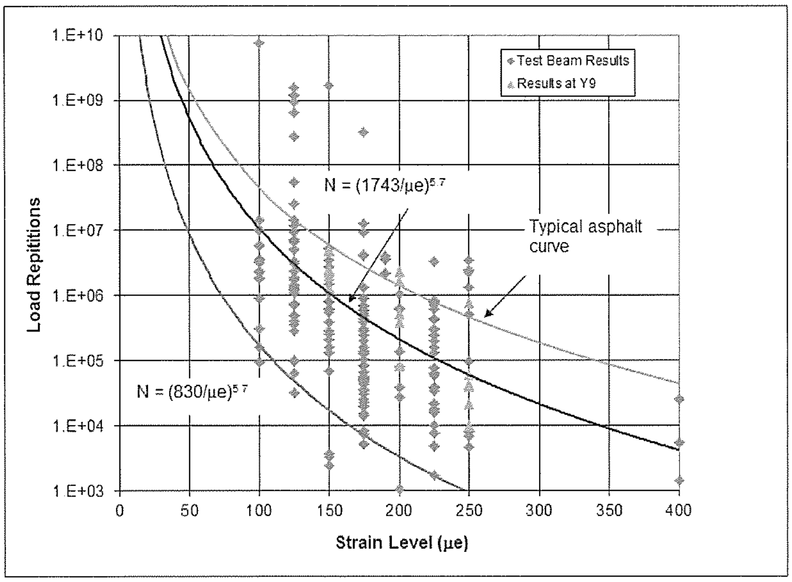

Figure 5.1: Graphical representation of individual beam test results.

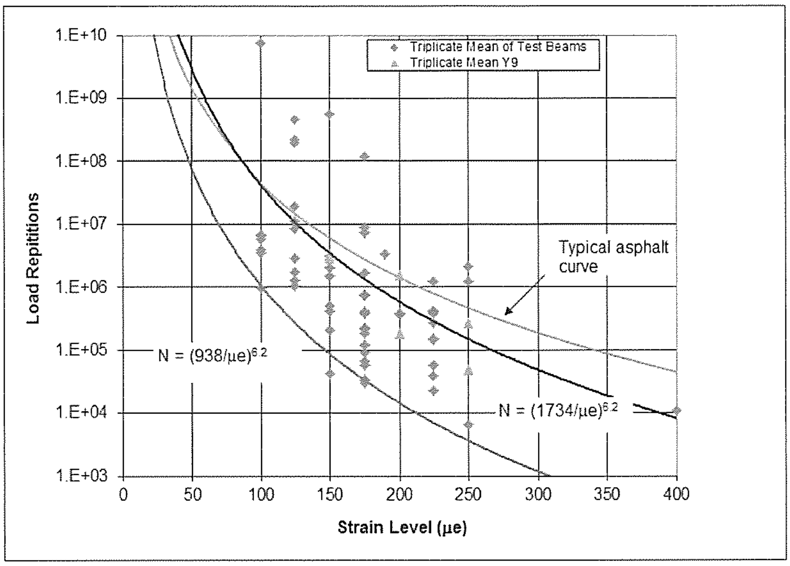

Figure 5.2: Graphical representation of triplicate mean test results.

In the early designs adopted by the City of Canning, the asphalt fatigue equation was used, but a design reliability factor of 4.5 was used in line with the VICROADS Supplement to the 1992 AUSTROADS Pavement Design Guide. Thus if the infield performance of the beams does truly reflect pavement performance, then the initial pavements will be conservatively designed.

5.2 FLEXURAL MODULUS VALUES

The analysis of modulus results needs to be undertaken according to broad material types depending on existing pavement materials. There were two broad material types, the majority type being pavements originally composed of all crushed materials, and pavements composed of rounded lateritic gravels. John Street and Railway Parade are from the latter type. All other pavements were composed of crushed quarried products.

Several pavement sections were analysed at both 20^0C and 30^0C. In order to compare the difference in flexural modulus, only those beams tested at both locations were used. The results showed a mean 9.4% decrease in modulus between 20^0C and 30^0C. The summary of the results is shown in Table 5.1

In comparison, previous testing by ARRB Transport Research Alderson (2001) on asphalt produced with Class 320 bitumen showed a reduction of 29% in stiffness when tested at 20^0C and 25^0C, thus the foamed bitumen stabilised material appears considerably less temperature sensitive than asphalt, as well as being considerably stiffer than asphalt.

Pavements composed of natural lateritic gravels and crushed granite/limestone mixtures were tested. Those pavements containing natural lateritic gravels demonstrated a lower flexural modulus than those containing all crushed gravels.

As only two pavements containing natural gravels were tested and one of these (John Street) was constructed differently to all others (a single mix with bitumen and lime added in one pass) showed very untypical stiffness, the pavements containing natural gravels have been excluded from the analysis of flexural stiffness shown in Table 5.2.

Table 5.1: Effect of temperature on modulus.

Location	Ei at 20^0C (MPa)	Ei at 30^0C (MPa)	Reduction from 20^0C to 30^0C
Bannister Road (Baile Road)	7539	7359	2.4%
Vulcan Road	7726	7401	4.2%
Orchard Road	6970	5872	15.7%
Railway Parade	5271	4853	7.9%
Orrong Road	7536	6223	17.4%
Mean %age Reduction from 20^0C to 30^0C			9.5%

Table 5.2: Flexural modulus (top layer).

Details	All results		Outliers Excluded	
	Mean Modulus (MPa)	95th %ile Modulus (MPa)	Mean Modulus (MPa)	95th %ile Modulus (MPa)
Granular Pavements all temperatures	6494	2618	6629	3282
Granular Pavements at 20°C	6459	2608	6532	2986

Only two sets of results were available for modulus at different levels in the pavement, this being from Bannister Road (Forum Avenue) and Orrong Road. These results are shown in Table 5.3. Analysing these results gives a mean modulus at 100 to 200 mm depth from the top of the stabilised layer of 70% of the top layer and at 200 to 300 mm depth of 48% of the top layer at 175mm at Bannister Road, but the modulus results at Orrong Road showed little difference between the 0-100 mm layer and the 100-200 mm layer.

This compares well to that previously determined using the comparison of a larger number of Indirect Tensile Modulus results of 74% and 46% respectively for the middle 100mm and bottom 100 mm layers compared to the top 100 mm layer. (Leek, 2001)

Table 5.3: Variation of modulus with depth at Bannister Road (Forum Avenue) and Orrong Road.

Road	Test Strain ($\mu\varepsilon$)	Position in Pavement Layer	Mean E_i for Layer (MPa)	Percentage of Top Layer
Bannister Road (Baile Road)	175	Top	11020	N/A
		Middle	7757	70%
		Bottom	5290	48%
Orrong Road	100	Top	7536	N/A
		Bottom	7153	95%
	150	Top	7397	N/A
		Bottom	8033	109%
	175	Top	8093	N/A
		Bottom	7363	91%

In the analysis of pavements composed of rounded lateritic gravels with a small contribution of crushed materials via the original asphalt surface, it is notable that there is considerable difference in flexural modulus between Railway Parade and John Street, so that combining the results of these two pavements together for analysis would not be considered.

5.3 INDIRECT TENSILE MODULUS VALUES

When examining the results in Table 4.6, it is noted that the modulus values are generally very high. FWD testing also indicated a very stiff pavement, indicated by typical deflection and curvature values of 0.1 mm and 0.01 mm respectively.

The flexural modulus determined by the beam testing apparatus is undertaken at 20°C and 10 Hz. The beam fatigue apparatus roughly approximates to a rise time of 25 ms. Thus any relationship between the flexural modulus determined by the beam fatigue apparatus and the resilient modulus determined by the MATTA should compare MATTA results determined at 25 ms and 20°C.

If the values for flexural modulus in Table 5.2 are compared to the resilient modulus values at 25 ms and 20°C in Table 4.6, it can be seen that the flexural modulus is approximately 60% of the resilient modulus.

As flexural modulus is the value required for pavement design, the resilient modulus for the Weighted Mean Annual Pavement Temperature and at the appropriate traffic speed should be factored by 0.6 to obtain the design value at that temperature and speed regime applicable to the pavement section.

As demonstrated in Table 5.3, there is an apparent decrease in modulus with depth, and this should be accounted for in the pavement design process, as decreasing modulus will result in increased strain in the pavement and subgrade layers. Based on the values given in Table 5.3, the modulus at 100-200 mm below the stabilised layer surface is approximately 85% of the top layer and for depths greater than 200 mm below the stabilised surface, the modulus is approximately 60% of the top layer.

The bitumen content in all cases was determined by the actual application rate, the depth of stabilisation and the field dry density results, and ranged between 6.2% and 8.8% by volume of mix. There was no obvious relationship between bitumen content and modulus or bitumen content and fatigue life.

5.4 PROPOSED DESIGN MODULUS VALUES

Design modulus values will vary according to materials, and FWD testing undertaken on High Road at 2 days, two months and four months after testing indicated that the pavement had further stiffened between two and four months after construction. Therefore any laboratory samples used for design purposes should be cured for a reasonable time, but how long this time should be the subject of further investigation.

In the absence of detailed site or laboratory testing, the values in Table 5.4 have been determined by the City of Canning when stabilising existing pavements composed of crushed limestone sub-base, crushed granite base course and asphalt surfacing. Note that due to the sandy soil conditions on which these pavements are constructed, using soaked values is not considered justified.

Similar testing of a much smaller sample size has been undertaken on natural gravel type pavements. A value of 80% of values shown in Table 5.4 is assumed for pavements composed principally of natural laterite gravels.

Table 5.4: Adopted modulus for crushed granular pavements in City of Canning.

Depth below stabilised surface	Design Modulus
0 – 100 mm	4300 MPa
100 – 200 mm	3600 MPa
>200 mm	2600 MPa

5.5 VARIATION OF FATIGUE LIFE WITH DEPTH

One of the aims of the research programme was to determine the variation of fatigue life with depth in the pavement layer. Due to the limitations of cutting equipment, it was difficult to extract slabs from the pavement without causing damage to the bottom layer. In addition, due to the cost of fatigue testing, a compromise in the amount of testing was necessary. Due to these factors, only two sites were analysed in detail to determine the variation of fatigue life with depth in the pavement layer.

At one site, Bannister Road near Forum Avenue, there was a definite drop in flexural modulus with depth in the pavement, but a more recent analysis of results from Orrong Road did not support this trend. However based on the larger number of cores tested for resilient modulus, the Orrong Road results are not typical. The results of the fatigue tests were widely scattered varying from 8% to 1440% and no trend could be determined.

This does not imply that there is not a variation with depth, as it is considered that the fatigue life should decrease with lower compaction and higher voids. However the scatter of results meant that with the small sample size, a trend was not observed and a conclusion could not be determined.

5.6 VARIATION OF FATIGUE LIFE WITH TEMPERATURE

An investigation to determine if the fatigue life of *in situ* foamed bitumen stabilised materials was temperature dependent was also one of the aims of the investigation. Here the results over a larger number of samples indicate that whilst modulus is temperature dependent, the variation of fatigue life is probably far more dependent on the inherent variation found in the pavement materials than in variation with temperature. Thus it is considered that the fatigue predictions used at 20^0C are equally valid for use at 30^0C.

6 CONCLUSIONS

Considerable research effort has gone into developing a fatigue transfer function for *in situ* foamed bitumen stabilised pavements, but as would be expected, there is a large degree of variability in the testing of these materials.

There was no apparent relationship between fatigue life and temperature over the ranges tested. It is considered that fatigue life should improve with temperature, but inherent variations within the pavement will have a greater effect on fatigue life than temperature.

There is little evidence that bitumen volume within the range applied in Australia stiffness has any statistical relation with fatigue performance over the ranges found in foamed bitumen stabilised pavements and the following more simplified approximation to fatigue life could be used:

$$N = (1734/\mu\varepsilon)^{6.2} \quad (6)$$

where N = allowable number of load repetitions
$\mu\varepsilon$= tensile strain induced in bottom of stabilised layer pavement by applied 80kN axle load.

Whilst it is noted that this is a non-conservative approach, as 50% of the fatigue lives are less that that determined, the fatigue testing is undertaken at accelerated frequency and high strain, and does not allow for a shift factor previously discussed. Thus it is considered by the authors as a conservative approach. However an alternative equation that provides a fit for 95% of all test results is:

$$N = (1734/\mu\varepsilon)^{6.2} \tag{7}$$

It was not possible to determine a relationship between fatigue life and depth. A trend was not evident, but this is considered to be due to the small sample size. Intuitively it is considered that there should be a reduction, although it is possible that again variations within the pavement materials may be more significant.

Testing at pavement age of 9 years for laboratory fatigue, indirect tensile modulus and deflection testing all indicate that the pavements have retained the high stiffness recorded at 6 months age, and no evidence of a reduction in modulus by fatiguing mechanisms were evidenced. This trend may indicate that a shift factor does apply in the relationship between laboratory predictions of field life and those evidenced in the field.

Care is recommended with the use of foamed bitumen stabilisation in very heavy truck traffic situations where pavements are constructed of rounded aggregate particles with no crushed faces, as these pavements may rut early in their service life.

This report of the investigation into *in situ* foamed bitumen stabilisation is only a summary of the extensive work undertaken by ARRB Transport Research. The testing programme has shown many variables in the performance of the material, some of which cannot be explained with the limited data base available at present.

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