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Laboratory fatigue life of cemented materials in Australia

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In Australia increasing traffic loadings are placing greater pressure on sprayed seal unbound granular pavements, with some non-standard materials no longer being fit-for-purpose. Unfortunately, in many rural areas, the use of high-quality crushed rock is not a cost-effective treatment to improve the structure of these pavements. Consequently, there is growing interest in pavement-strengthening treatments, such as in situ cement stabilisation, that enhance existing non-standard materials. The required thickness of cement stabilisation is commonly governed by the fatigue characteristics of the treated material. This paper presents the results of an on-going Austroads research project to investigate the laboratory fatigue life of a wide range of cemented materials. Flexural beams were manufactured and tested using newly developed laboratory methods. In addition to the fatigue tests, flexural strength and flexural modulus tests were also conducted. Results showed that the fatigue relationship for cemented materials is significantly dependent on breaking strain and not modulus, which suggests that the current Austroads design criterion can be improved. The ratio of initial strain resulting in a fatigue life of 10^6 cycles divided by breaking strain is seen as a potentially superior method of incorporating material quality into the cemented materials' fatigue relationship. Finally, a presumptive laboratory relationship based on strain ratio was developed. The presumptive fatigue relationship is conservatively based on a strain ratio of 0.35 for 10^6 load cycles. A strain damage exponent of 12 is recommended for pavement design.

Keywords: cemented; materials; fatigue; laboratory; damage

1. Introduction and literature review

1.1. *Background*

Over 90% of the Australian and New Zealand sealed road network consists of sprayed, sealed, surfaced granular pavements. Increasing traffic loadings are placing greater pressure on this existing pavement asset, with some non-standard materials no longer being fit-for-purpose. Unfortunately, in many rural areas, the use of asphalt or high-quality crushed rock is not a cost-effective treatment to improve the structure of these pavements. Consequently, there is growing interest in better understanding the performance of treatments that enhance the existing pavement materials by the addition of cementitious and bituminous binders to allow recycling of scarce resources.

Austroads' (the association of Australian and New Zealand road transport and traffic authorities) procedures for the thickness design of alternative structural treatments, such as in situ recycling with cementitious binders, are not as well founded as those for conventional treatments due to a lack of data regarding the performance of these alternative treatments. Improved design procedures

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are required that better reflect the structural contribution of stabilisation treatments as this will lead to more cost-effective rural-road rehabilitation treatments.

In 2007, Austroads commissioned ARRB Group (formerly the Australian Road Research Board) to undertake a research project to develop improved procedures for the design of structural rehabilitation treatments for rural pavements, particularly cementitious and bituminous stabilisation.

The objectives of this project were:

- to develop a presumptive cemented materials' laboratory fatigue relationship applicable to the wide range of granular materials in rural pavements and
- to identify distress modes of bituminous stabilised rural pavements and develop appropriate thickness design procedures.

This paper draws on a laboratory fatigue work undertaken on cemented materials in Australia (Gonzalez, Howard, & de Carteret, 2010). Ten aggregates from different parts of Australia were selected for the study and mixed with cement contents between 3% and 5%. Fourteen cement-treated materials were prepared. The four-point bending flexural beam test method (Yeo, 2008) was adopted for laboratory testing of cemented beam specimens. This testing method is used to estimate the flexural strength, breaking strain, flexural modulus and flexural fatigue characteristics of such cemented specimens.

The main objectives of this research were: (a) to determine fatigue relationships from the laboratory testing, (b) to compare these with the current Austroads fatigue relationship for cemented materials (Austroads, 2012) and, if necessary, (c) to develop a more accurate general presumptive fatigue model that could be incorporated into the Austroads guide.

1.2. *Background to current Austroads fatigue relationship for cemented materials*

The fatigue life (N) of a cemented layer is usually considered to be a function of either the applied tensile stress (σ_t), the applied tensile strain (ε_t) or as a ratio of these responses to the breaking stress (σ_b) or breaking strain (ε_b) of the material (Austroads, 2012; LCPC, 1997; Litwinowicz & Brandon, 1994; Otte, 1978; Theyse, de Beer, & Rust, 1996). The general form of these relationships is typically expressed as

$$\log N = f\left[\frac{\sigma_t}{\sigma_b} \text{ or } \frac{\varepsilon_t}{\varepsilon_b}\right]. \tag{1}$$

Fatigue of cemented materials may also be expressed as a function of the ratio of the breaking strain and the applied tensile strain raised to a load damage exponent (LDE) (Litwinowicz & Brandon, 1994)

$$N = \left[\frac{\varepsilon_b}{\varepsilon_t}\right]^{\text{LDE}}. \tag{2}$$

This equation indicates that, for a cemented material with a specific value of strain at break, increasing the horizontal tensile strain at the bottom of the cemented layer would result in a lower fatigue life (N). The magnitude of the effect on fatigue life is highly influenced by the LDE. A higher LDE indicates that the pavement is more sensitive to tensile strains and therefore more sensitive to traffic loads.

In the NAASRA (1987) and Austroads (1992) versions of the Austroads pavement design guide, the performance relationship for cemented materials was expressed as

$$N = \left[\frac{K}{\mu\varepsilon}\right]^{18}, \tag{3}$$

where N is the number of repetitions of tensile strain at the bottom of the cemented layer before fatigue when the level of this strain is $\mu\varepsilon$ microstrains and K is a value that depends on the elastic modulus of the cemented material. This fatigue relationship and damage exponent were largely based on work by Pretorius (1969). The cemented material used by Pretorius was of a very high strength, closer to a lean mix concrete than a typical Australian cemented material and therefore the equation was not necessarily applicable to Australian pavements.

The dependence of fatigue life on modulus (K value in Equation (3)) was based on results reported by Otte (1978) on the variation of breaking strain with modulus for cement-treated natural weathered materials in South Africa. It should be noted that while these results have been used as a basis for relating breaking strain and modulus in Australia, Otte (1978) concluded that breaking strain was independent of modulus for cement-treated crushed rock as the variation of breaking strain with modulus results was highly variable.

Fatigue life was not observed to be dependent on modulus in the laboratory testing of field beams conducted as part of an accelerated loading trial in Australia (Jameson, Sharp, & Yeo, 1992). Jameson et al. noted that there is considerable doubt about the dependence of fatigue on modulus and suggested that the effect of modulus on fatigue life needed further investigation. A similar conclusion was made by Litwinowicz (1986) in a limited laboratory study of cemented materials.

In 1997, the Austroads fatigue relationship for cemented materials (Equation (3)) was reviewed and subsequently reduced to an LDE of 12 based on a literature review conducted by Jameson, Dash, Tharan, and Vertessy (1995), the findings of an accelerated loading trial on cemented materials, a literature review of overseas laboratory testing and anecdotal evidence from a selection of cement-stabilised roads in Queensland by Angell (1988). The modulus dependency was retained in the 1997 revision due to a lack of sufficient data to support a change in the form of the fatigue

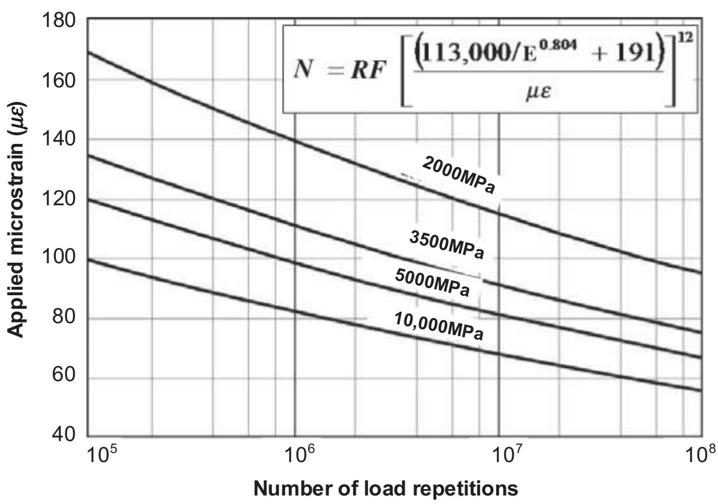

Figure 1. Fatigue relationships for various moduli of cemented materials (expressed in MPa) for a project reliability of 95% (source: Austroads, 2012).

relationship despite reservations regarding the modulus dependency. This general relationship with an LDE of 12 and a modulus dependency remains included in the most recent publication of the Austroads Guide to Pavement Technology – Part 2: Pavement Structural Design (Austroads, 2012), but has been simplified and extended to include an ability to design for project reliability

$$N = \mathrm{RF}\left[\frac{(113,000/E^{0.804} + 191)}{\mu\varepsilon}\right]^{12}, \tag{4}$$

where N is the cemented material fatigue life, $\mu\varepsilon$ the initial tensile strain at the bottom of the cemented layer (microstrain), E the initial flexural modulus of the cemented material (MPa) and RF the reliability factor for cemented materials. The major influence modulus has on fatigue life in the Austroads (2012) relationship is illustrated in Figure 1.

2. Materials and methods

2.1. *Overview of laboratory testing programme*

Fourteen cement-treated materials were prepared in the laboratory study plus two concrete mixes. These concrete mixes were included to allow comparison between the fatigue performance of cemented materials and that of lean mix concrete and base-quality concrete. However, as this work is on-going the results for the concrete mixes are not included in this paper. The flexural beam test method (Yeo, 2008) was adopted for laboratory testing of cemented beam specimens. This testing method is used to estimate the flexural strength, breaking strain, flexural modulus and flexural fatigue characteristics of such cemented specimens. The manufacture and testing of flexural beam specimens were conducted in two stages: 2008/2009 and 2009/2010.

In stage 1 (2008/2009), manufacture of the cementitious-stabilised test samples was conducted using the following approach:

- selection of five granular materials covering a wide range of gradings and plasticities and
- for each granular material, manufacture of test beams with up to two binder contents (e.g. 3% and 5%).

In stage 2 (2009/2010), additional cemented materials were manufactured using the following approach:

- selection of another five granular materials with low to high performing aggregates and
- for each material, manufacture of test beams was conducted at either 3% or 4% binder content.

The objective of manufacturing additional samples in stage 2 was to widen the database of flexural modulus and strength of cemented materials and to verify some of the findings from stage 1.

2.2. *Materials*

2.2.1. *Aggregates*

This research project focused on lower-quality aggregates typically used for rural highway construction in Australia. Candidate pavement materials were selected based on general guidelines detailed in Gonzalez et al. (2010). A total of 10 aggregate materials were selected for testing. Five materials were selected for the first 2008/2009 phase of the study and the remaining five for the second 2009/2010 phase. The aggregates were sourced from four states of Australia: New South Wales (NSW), Victoria (Vic), South Australia (SA) and Queensland (Qld). A summary of the aggregate characteristics and particle size distribution are presented in Table 1 and Figure 2, respectively. The aggregates had a wide range of properties in terms of mineralogy, particle size distribution, maximum particle size and plasticity. The finest material was the prior stream gravel (PSG) with a maximum particle size of 6.7 mm and the coarsest the recycled concrete. The latter did not comply with the aggregate specifications used by road authorities in Victoria (VicRoads). The upper and lower limits shown in Figure 2 are discussed later in this paper.

2.2.2. *Cement*

A general-purpose Portland cement was used in this project. The cement was provided in small quantities during the execution of the project by a local company in Melbourne (Vic). Cement is manufactured from Portland cement clinker and gypsum and complies with AS 3972 (Standards Australia, 2010).

2.3. *Sample preparation*

For each granular material studied, a bulk quantity of 0.5–2.0 t was obtained. The bulk granular materials were mixed then split into 10 kg representative sub-samples using a motorised rotary splitter. To ensure material uniformity, the mixing and splitting procedure was performed twice prior to storage of the sub-samples in sealed containers. Batches of the cemented material mixes were prepared in a motor-driven planetary concrete mixer with the proportions of granular material, cement and water for each batch being monitored and recorded. Following mixing, the cemented material batches were placed in plastic containers, covered with a plastic sheet and left to stand for a period of approximately 30 min to condition prior to compaction.

A segmental slab compactor and a rectangular mould with internal dimensions of 400 mm length × 320 mm width × 145 mm height were used for slab compaction. The cemented mix was placed in the mould in three even layers. Each layer was spread evenly in the mould and tamped manually. Once the total mass of cemented material mix was placed in the mould, the material was compacted as a single layer to the specified height of 100 mm using the slab compactor.

The compacted slab was left in the closed mould and covered with a moist cloth and lid to minimise moisture loss and stored at controlled room temperature ($T = 23°C$) for a minimum of two days before being de-moulded and cured in a fog room ($T = 23°C$). Each slab was subsequently cut into two beams using a diamond tipped saw. The minimum curing period before cutting the slab was 14 days (normally 24 days) to ensure that the strength of the slab enabled cutting without disintegrating. The beams were then cured for a minimum of five months at 23°C either in a fog room or wrapped in moist hessian and sealed in a plastic bag prior to testing.

2.4. *Test methods*

2.4.1. *Flexural test method*

The flexural beam test was used to estimate the flexural strength, flexural modulus and flexural fatigue for each of the cemented materials. The geometry of the flexural beams was based on AS1012.11-2000: methods of testing concrete – Method 11: determination of the modulus of rupture (Standards Australia, 2000).

Given the maximum aggregate size of approximately 20 mm, the nominal dimensions of the beam specimens were 100 mm high × 100 mm wide × 400 mm long. The beam supports were

Table 1. Summary of aggregate properties.

Type	Source	Project stage	MDD (t/m^3)	OMC (%)	PI (%)	LL (%)	Notes
Weathered granite	Cooma, NSW	First 2008/2009	2.04[a]	10.2	14	31	Routine monitoring of associated long-term pavement performance sections has shown minimal distress and indicated that the test sections have performed well
Calcrete limestone	Renmark, SA	First 2008/2009	1.95	13.0	4	23	Well-graded material with relatively low aggregate strength (typical Los Angeles abrasion values of 30)
Basalt	Mount Gambier, SA	First 2008/2009	2.14	12.0	NP	–	Well-graded, high quality, non-plastic material has been used as unbound basecourse in the construction of a number of sections near Mount Gambier
PSG	Hay, NSW	First 2008/2009	2.12	7.2	8	22	The PSG is a fine grained river sand material used in construction of about 300 km of highway and has performed poorly
Modified PSG	Hay, NSW	First 2008/2009	2.27	5.2	8	22	An artificial material comprised of the same PSG presented above, blended with additional larger sized rhyolite aggregates
Recycled concrete[b]	Laverton, Vic	Second 2009/2010	1.96	12.5	6	32	The aggregate tested for this project did not meet the supplier or VicRoads grading envelope
Granite	Oaklands Junction, Vic	Second 2009/2010	2.24	6.5	6	21	A well-graded material complying with VicRoads specifications
Hornfels	Lysterfield, Vic	Second 2009/2010	2.34	6.0	6	21	A well-graded material that has been previously been assessed as a base material under accelerated pavement testing using the ARRB ALF
Basalt	Purga, Qld	Second 2009/2010	2.29	7.8	6	19	This aggregate has been widely used as unbound base material throughout South East Queensland
Siltstone	Parahills, SA	Second 2009/2010	2.09	9.0	8	25	This material has been used extensively in both unbound and cement stabilised forms for highway construction around Adelaide and also tested under ALF

Notes: NSW, New South Wales; SA, South Australia; Vic, Victoria; Qld, Queensland.
[a]Standard compaction was used.
[b]Does not comply with VicRoads specifications.

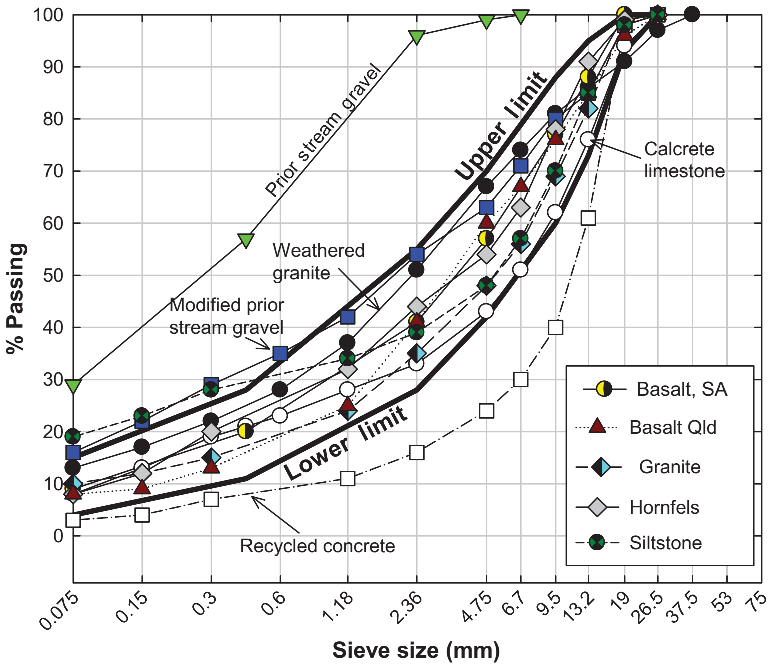

Figure 2. Particle-size distribution for aggregates.

set 300 mm apart to achieve a span to depth ratio of three (Figure 3). The load positions were at third-points along the specimen and the beam displacement was measured at the mid-point. A materials testing apparatus (MATTA) with a 14 kN pneumatic load actuator was used to undertake all the flexural beam tests, with typical loading in the range of 1–6 kN.

To minimise moisture loss during testing, beams were sealed in thin plastic cling wrap prior to testing. Testing was conducted under normal laboratory environment conditions at 23°C. Following testing the beams were weighed, as tested, then dried in an oven to a constant mass for moisture content determination. The density and moisture content at the time of testing were estimated from the measured dry mass and geometry (volume) of the beam.

Each beam specimen was tested for flexural modulus before being subjected to either flexural strength or flexural fatigue testing depending on the required outcomes.

2.4.2. *Flexural strength testing*

For the flexural strength test, the MATTA machine was programmed to apply a monotonic load. The sample was first loaded with a seating force of 50 N for the first 6 s, after which the load was increased at a rate of 3.3 kN/min until the sample failed as described in AS1012.11–2000 (Standards Australia, 2000).

The horizontal tensile strain at the bottom of the flexural specimen was calculated at the breaking load (peak load) and at 95% of the break load. The adoption of the strain at 95% of the breaking load was in line with the approach suggested by Litwinowicz and Brandon (1994) to avoid reporting of excessive variations in the strain at break.

2.4.3. *Flexural modulus testing*

The flexural modulus test involved the application of cyclic haversine load pulses of 250 ms duration. The beam deflection associated with each load pulse was recorded, together with the

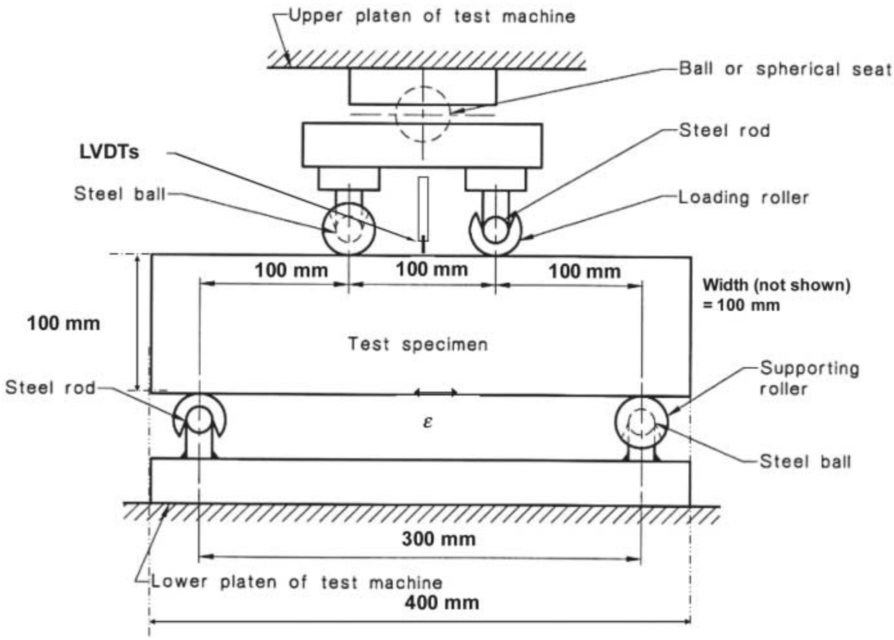

Figure 3. Cross-sectional view of flexural-beam-testing apparatus.

seating load (nominally 50 N) and the peak load. The pulse period was 1 Hz including a 750 ms rest period between load pulses. A minimum of 100 load pulses were applied to the sample. The magnitude of the load pulses was about 40% of the breaking load as determined from the flexural strength tests for the material type. This load was targeted to produce approximately 20–30 microstrains.

2.4.4. *Flexural fatigue testing*

The flexural fatigue test involved the application of cyclic haversine load pulses similar to that described for flexural modulus above. The beam deflection associated with each load pulse was recorded, together with the seating load (nominally 50 N) and the peak load. The shorter pulse period of 2 Hz (twice the rate of flexural modulus testing detailed above) was adopted due to the time-consuming nature of this test procedure. This 2 Hz (500 ms) period included a 250 ms rest period between 250 ms load pulses. The magnitude of the load pulses was generally selected to fall in the range of 40–80% of the breaking load as determined from the strength tests for the material type. This load pulse range was selected to produce a wide range of calculated strain values of 40–100 microstrains corresponding to a range of approximately 10^3–10^6 cycles up to failure, as will be detailed later.

3. Results

3.1. *Flexural modulus*

A summary of flexural modulus results for specimens cured between five and nine months is presented in Table 2. The mean flexural modulus results ranged from 8800 MPa (calcrete limestone 3% cement) to 20,300 MPa (Hornfels). Results of the flexural modulus test at 28 days are not presented in this paper. However, an average increase in the flexural modulus of cemented

Table 2. Summary of flexural modulus test results.

Material	Cement (%)	ID	T	n	Relative density (RD) RD (%)	SD (%)	CV (%)	Flexural modulus (MPa)	SD (MPa)	CV (%)
Weathered granite	3	WG_3	9	26	94.5	0.8	0.8	9200	800	9
Weathered granite	5	WG_5	9	25	95.5	0.8	0.8	14,900	900	6
Calcrete limestone	3	CL_3	9	26	96.4	1.2	1.2	8800	1100	13
Calcrete limestone	5	CL_5	9	26	97.7	1.1	1.1	12,400	800	6
Basalt (Mt. Gambier)	3	BAm_3	9	26	97.7	2.3	2.4	14,700	2000	14
PSG	5	PSG_5	5	20	93.3	1.7	1.8	12,100	1100	9
PSG	5	PSG_5	9	26	95.0	8.3	8.7	12,700	1000	8
Modified PSG	3	MPSG_3	9	28	94.3	1.0	1.1	17,200	1600	9
PSG	3	PSG_3	5	20	93.7	0.9	1.0	9500	700	7
Granite	3	GR_3	5	16	96.3	1.1	1.1	16,500	1700	10
Recycled concrete	3	RC_3	5	15	97.5	1.7	1.7	10,900	1300	12
Hornfels	3	HO_3	5	20	94.0	7.3	7.8	20,300	1500	7
Basalt (Purga)	3	BAp_3	5	16	95.7	1.1	1.1	12,900	1000	8
Siltstone	4	SS_4	5	12	94.5	1.8	1.9	14,300	1600	11

Notes: T, curing time in months; n, number of samples tested; RD, density relative to maximum aggregate dry density determined using modified compaction; SD, standard deviation; CV, coefficient of variation.

materials of 28% was observed between samples tested at 28 days and 9 months. A curing period of five and nine months was chosen to better represent the modulus of the cemented materials in the long term. In addition, five and nine months were a good compromise between curing time and project duration. The range of results is considerably higher than those currently recommended for the current design of cemented-treated pavements in Australia (Figure 1).

3.2. *Flexural strength and flexural breaking strain*

A summary of flexural strength and breaking strain results for specimens cured between five and nine months is presented in Table 3.

Results of flexural strength tests at 28 days are not presented in this paper. However, an average increase in flexural strength of 48% was observed between samples tested at 28 days and 9 months.

Table 3 also compares tensile strains at the point of failure and tensile strains at 95% of the maximum breaking load. It is apparent that the tensile strain at maximum load (failure) varies more than the tensile strain at 95% of breaking load. The average coefficient of variation for the tensile strain at failure is 17% while for the strain at 95% of the breaking load is 10%. This indicates that tensile strain at 95% of the breaking load seems to be a more consistent parameter than tensile strain at failure. Strains at 95% of the breaking load varied from 98 to 286 microstrains. The highest variability was observed on the calcrete limestone 3% (CV = 23%).

3.3. *Flexural fatigue*

3.3.1. *Introduction*

The strain or stress level applied to specimens prepared during the first stage (2008/2009) was applied so that the fatigue life ranged between 10^3 and 10^6 load cycles (i.e. the higher the stress/strain level, the shorter the fatigue life). The number of specimens available for the stage 1 fatigue testing was about 18–20 specimens. This number was sufficient to build a fatigue relationship. Fewer specimens were available for the second stage of fatigue testing (2009/2010). Therefore, the testing was restricted to target a strain that would result in a fatigue life of 10^6 load cycles to verify some of the findings from stage 1.

Table 3. Summary of flexural strength test results (strength and breaking strain).

Material	Cement (%)	ID	T	n	Relative density (RD) RD (%)	SD (%)	CV (%)	Flexural strength (MPa)	SD (MPa)	CV (%)	Breaking strain at 100% of the load ($\mu\varepsilon$)	SD (MPa)	CV (%)	Breaking strain at 95% of the load ($\mu\varepsilon$)	SD (MPa)	CV (%)
Weathered granite	3	WG_3	9	4	95	1	1	0.96	0.08	9	259	2	1	185	2	1
Weathered granite	5	WG_5	9	3	94	1	1	1.51	0.16	11	247	41	17	169	8	5
Calcrete limestone	3	CL_3	9	4	97	0	0	0.97	0.12	12	342	132	39	286	66	23
Calcrete limestone	5	CL_5	9	4	97	0	1	1.50	0.15	10	315	40	13	208	21	10
Basalt (Mt. Gambier)	3	BAm_3	9	4	99	2	2	1.97	0.12	6	355	4	1	216	10	5
PSG	5	PSG_5	5	10	94	2	2	1.02	0.12	12	176	22	13	150	14	9
PSG	5	PSG_5	9	4	93	1	1	1.19	0.13	11	165	12	8	127	11	9
Modified PSG	3	MPSG_3	9	4	94	1	1	1.27	0.16	13	170	15	9	126	16	12
PSG	3	PSG_3	5	10	94	1	1	0.78	0.07	9	164	41	25	186	27	15
Granite	3	GR_3	5	6	96	1	1	1.13	0.10	9	226	37	16	140	12	9
Recycled concrete	3	RC_3	5	6	97	2	2	0.62	0.11	18	112	22	19	98	8	8
Hornfels	3	HO_3	5	7	94	1	1	1.57	0.12	8	227	54	24	130	20	16
Basalt (Purga)	3	BAp_3	5	6	95	1	1	0.98	0.10	10	352	156	44	144	12	8
Siltstone	4	SS_4	5	4	95	2	2	1.41	0.28	20	277	32	12	178	13	7

Notes: T, curing time in months; n, number of samples tested; RD, density relative to maximum dry density determined using modified compaction; SD, standard deviation; CV, coefficient of variation.

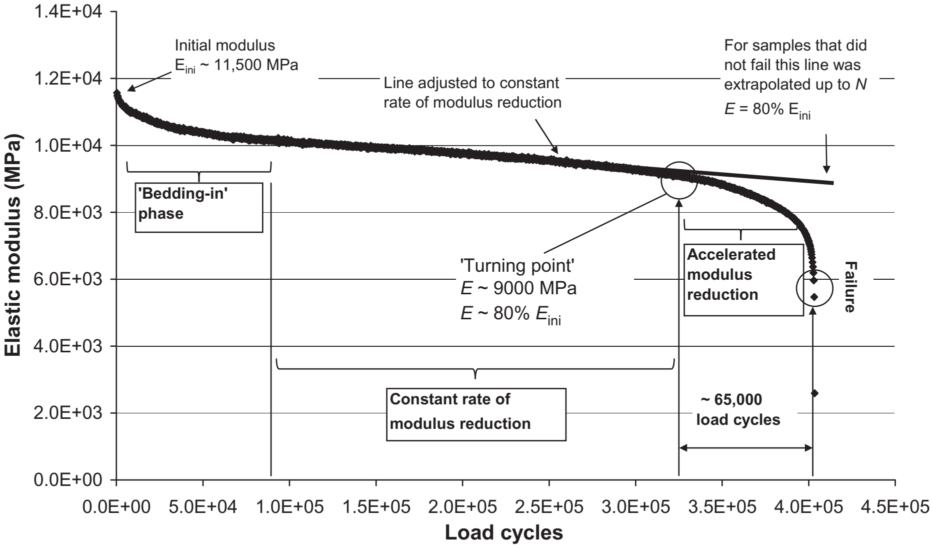

Figure 4. Typical modulus variation during fatigue tests (specimen from siltstone 4% cement).

The failure in fatigue tests was defined as the number of load cycles required to achieve half of the initial modulus. Normally, beam specimens failed (broke) before attaining half modulus or only a few load cycles were applied between half modulus condition and specimen braking. In total, 215 flexural fatigue tests were performed on cemented materials.

It is important to note that the presumptive fatigue relationships were developed using specimens at different cure ages (five and nine months) and therefore the fatigue relationships presented are valid for these curing ages. For example, specimens were not tested for flexural fatigue between 28 days and 5 months of curing.

3.3.2. *Modulus reduction of specimens during fatigue test*

In all the fatigue tests, the modulus decreased rapidly from the start of the test (one load cycle) up to approximately 5×10^3 to 10^4 load cycles (Figure 4). After this initial 'bedding-in' phase, the modulus decreased at a slow, constant rate. For the specimens that failed within the testing range a 'turning point' was observed when the modulus attained approximately 80% of the initial modulus. After this point, an accelerated rate in modulus reduction was observed and the specimen fails after approximately 65×10^3 load cycles.

Of the specimens tested 12% ($n = 27$) did not fail before the maximum number of load cycles applied during the fatigue tests (up to 10^6). In 7 of these 27 specimens, modulus reduction was observed after the initial bedding-in while in the rest of the specimens the modulus remained constant. For the seven specimens where a modulus reduction was observed, the fatigue life was predicted by extrapolating the constant rate of modulus reduction up to 80% of the initial modulus. In addition to the number of load cycles to achieve 80% of the initial modulus, 65×10^3 load cycles were added to account for the accelerated modulus reduction phase (Figure 4).

4. Discussion

4.1. *Form of fatigue relationship*

The form of fatigue relationships in use overseas varies significantly between countries and when compared with the current Austroads relationship. Some cemented materials' fatigue relationships

Table 4. Regression analysis of fatigue data for original cement-treated materials (strain-based equations).

Analysis number	Dependent variable	Independent variable	Independent variable	Independent variable	R^2	Standard error
1	Log (N)	ε_t	–	–	<0.25	0.76
2	Log (N)	log (ε_t)	–	–	<0.25	0.77
3	Log (N)	ε_t	$\mu\varepsilon_b$	–	0.35	0.67
4	Log (N)	log (ε_t)	log (ε_b)	–	0.29	0.70
5	Log (N)	$\varepsilon_t/\varepsilon_b$	–	–	0.33	0.68
6	Log (N)	log $(\varepsilon_t/\varepsilon_b)$	–	–	0.32	0.68
7	Log (N)	$\mu\varepsilon_t$	–	E	<0.25	0.76
8	Log (N)	log (ε_t)	–	E	<0.25	0.77
9	Log (N)	$\mu\varepsilon_t$	$\mu\varepsilon_b$	E	0.37	0.66
10	Log (N)	log (ε_t)	log (ε_b)	E	0.34	0.67
11	Log (N)	log $(\varepsilon_t/\varepsilon_b)$	–	E	0.33	0.68

Notes: General form of equation is $y = ax_1 + bx_2 + cx_3 + d$; N, fatigue life; ε_t, initial strain; ε_b, breaking strain; E, flexural modulus; breaking strain for the purposes of analysis defined as strain at 95% breaking load from nine months flexural strength testing.

in use overseas are of a semi-log form (AASHTO, 2006; LCPC, 1997), where the LDE varies with the applied strain, while the Austroads relationship and its predecessors are of a log–log form. The current Austroads relationship (Equation (4)) also includes modulus as an independent variable while other overseas relationships use breaking strain (Freeme, Maree, & Viljoen, 1982) or breaking stress (LCPC, 1997).

Given the variation in fatigue performance of the materials studied under this project, analysis of aggregated fatigue data from the beams tested in stage 1 (2008/2009) was undertaken to determine the most appropriate form of fatigue relationship. Regression analysis was conducted for a number of scenarios which included altering the form of the relationship, log–log compared with semi-log, and introduction of additional variables such as modulus, stress and strain at 95% breaking load, either singularly or in combination. All the specimens tested in stage 1 were included in the analysis of the aggregated data. Tables 4 and 5 summarise the regression analysis undertaken, the independent variables included and the calculated statistical accuracy of each analysis scenario. For clarity the analyses were separated into strain-based equations (Table 4) and stress-based equations (Table 5). In Table 4, the initial strain (ε_i) is the strain measured after the first 50 load cycles and the breaking strain at 95% of the breaking load is the strain measured at 95% of the peak load. For instance, if a specimen fails at 2.58 kN load with a breaking strain of approximately 165 microstrains, the breaking strain is calculated at 95% of the breaking load ($95\% \times 2.58\,\text{kN} = 2.46\,\text{kN}$). Then, the researcher has to read from the load-deformation experimental data the strain at 2.46 kN (that will be <165 microstrains). Similarly, the initial stress is the stress measured after the first load cycles.

From Tables 4 and 5 it can be seen that the overall strain-based equations are better predictors of the fatigue life of the cemented materials. The best R^2 values for the equations based on strains are about 0.32–0.37, while the best R^2 values for equations based on stress range from 0.26 to 0.29. In addition, when 95% breaking strain or flexural strength is introduced the R^2 values are improved (e.g. in Table 4 analysis 1, 2, 7 and 8 versus 3, 4, 5, 6, 9, 10 and 11 for the strain-based equations). In addition, little statistical difference was found between the semi-log and log–log forms of the various relationships (e.g. analysis 5 versus 6).

It is also apparent from Tables 4 and 5 that incorporating flexural modulus does not improve the fatigue relationship significantly (comparing analysis number 3 and 4 with 9 and 10). This finding,

Table 5. Regression analysis of fatigue data for original cement-treated materials (stress-based equations).

Analysis number	Dependent variable	Independent variable	Independent variable	Independent variable	R^2	Standard error
12	Log (N)	σ_t	–	–	<0.25	0.81
13	Log (N)	log (σ_t)	–	–	<0.25	0.81
14	Log (N)	σ_t	σ_b	–	0.27	0.69
15	Log (N)	log (σ_t)	log (σ_b)	–	0.27	0.70
16	Log (N)	σ_t/σ_b	–	–	0.28	0.69
17	Log (N)	log (σ_t/σ_b)	–	–	0.26	0.70
18	Log (N)	σ_t	–	E	<0.25	0.79
19	Log (N)	log (σ_t)	–	E	<0.25	0.79
20	Log (N)	σ_t	σ_b	E	0.29	0.68
21	Log (N)	log (σ_t)	log (σ_b)	E	0.28	0.68
22	Log (N)	log (σ_t/σ_b)	–	E	0.29	0.68

Notes: General form of equation is $y = ax_1 + bx_2 + cx_3 + d$; N, fatigue life; σ_t, initial stress; σ_b, breaking stress; E, flexural modulus.

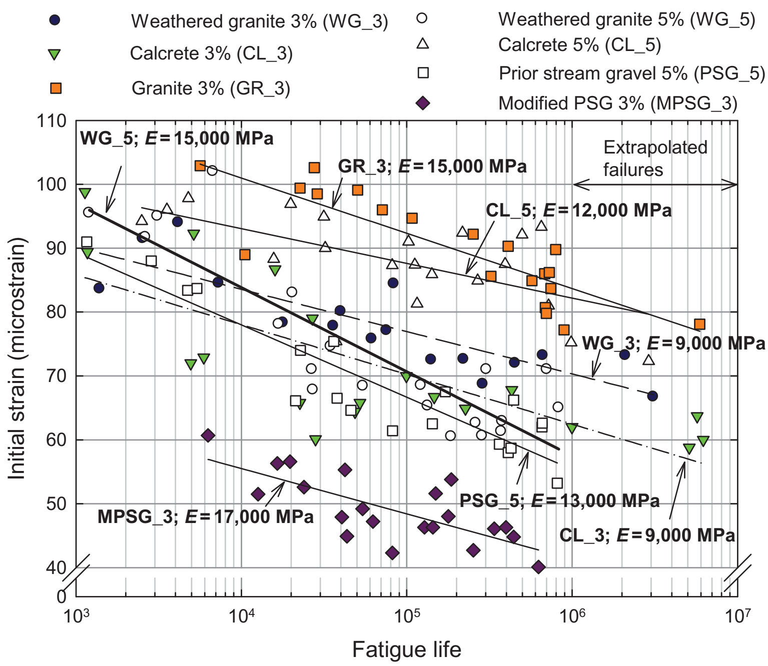

Figure 5. Fatigue life versus initial strain.

while consistent with conclusions from studies conducted both overseas (De Beer, 1990; LCPC, 1997; Litwinowicz & Brandon, 1994; Otte, 1978; Theyse et al., 1996) and locally (Jameson et al., 1995), has implications for pavement design in Australia as modulus is a significant factor in the existing Austroads fatigue relationship for cemented materials.

The stage 1 fatigue test results were plotted in Figure 5 as initial strain versus fatigue life for cemented materials specimens tested during 2008/2009. The fatigue life of the seven specimens that did not fail was predicted via extrapolation and is included in Figure 5. The figure indicates that fatigue relationships vary for each material and that modulus does not appear to influence

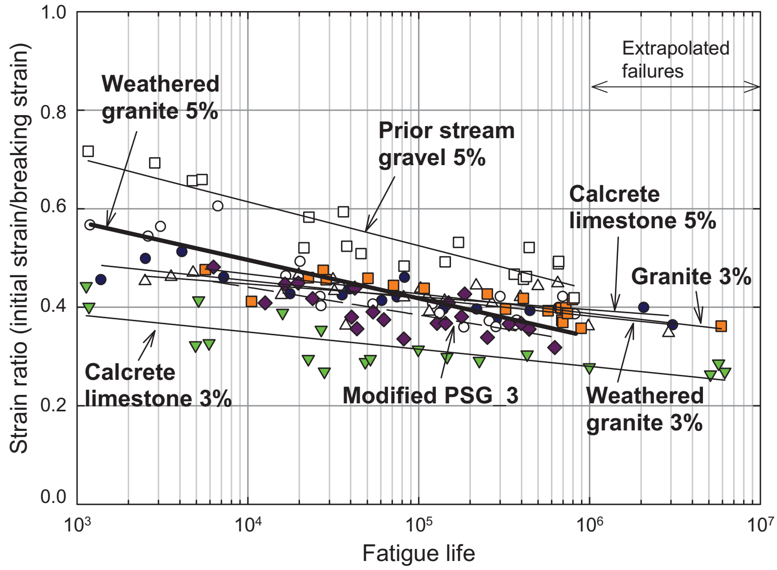

Figure 6. Fatigue life versus ratio of initial strain divided by breaking strain (strain at 95% breaking load).

fatigue life. For example, the cemented material with the highest modulus was the modified PSG with 3% cement but it also showed the poorest fatigue performance in terms of the initial strain. Conversely, the granite with 3% cement from Mt. Gambier had a longer fatigue life although it is the material with the second highest modulus.

To normalise the fatigue results, the strain ratio (i.e. the ratio between the applied initial strain and the breaking strain) was plotted against the fatigue life and is shown in Figure 6. This figure shows that introducing breaking strain into the fatigue relationship better explains the variation in fatigue between the individual materials, confirming the results of the regression analysis.

The fatigue life for the calcrete limestone 3% cement and the PSG 5% cement was significantly different from that of the rest of the materials (Figures 5 and 6). The lower fatigue life found in the calcrete limestone with 3% cement specimens was probably caused by the low cement content not sufficient to produce a bound material with consistent fatigue properties. In contrast, the higher fatigue life observed in the PSG 5% cement could be explained by the very different particle size distribution. When these two cemented materials are excluded from the fatigue analysis the results suggest that strain ratio versus number of load cycles to failure is fairly consistent across different materials.

In summary, it is apparent that fatigue relationships that include strain ratio are reasonable predictors of the fatigue life and that excluding modulus from the fatigue relationship does not significantly compromise the accuracy of these relationships.

4.2. *Stage 2 testing – failure at 10^6 load cycles*

To develop a presumptive fatigue relationship, appropriate values are needed for both the intercept and slope of the $\log(N) - \log(\mu\varepsilon_i/\mu\varepsilon_b)$ fatigue relationship (Equation (5)). Rather than base the intercept on the strain at the first cycle, it is considered that the strain ratio at 10^6 load cycles better reflects material performance for highway pavements since most of the pavements are designed with traffic levels above that number of load repetitions. In the stage 1 fatigue results (from 2008/2009) it was found that the strain ratio resulting in fatigue life of 10^6 cycles was

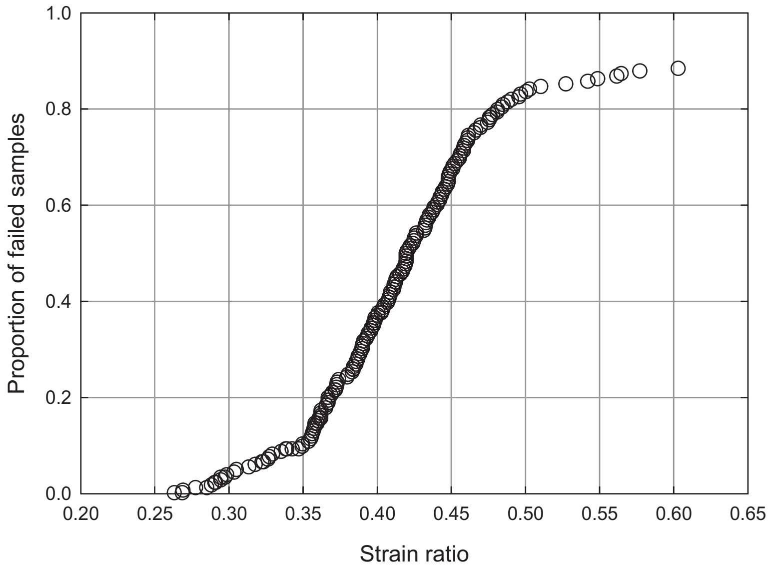

Figure 7. Proportion of failed specimens at different strain ratios.

consistent across the materials tested and within the range 0.32 and 0.40 (excluding the PSG with 5% cement and calcrete limestone with 3% cement). Consequently, in 2009/2010 stage 2 fatigue testing was undertaken to deduce a conservative strain ratio value at 10^6 load cycles for inclusion in the presumptive laboratory fatigue relationship. The presumptive fatigue relationship would be used for projects where it is not feasible to measure fatigue characteristics.

The specimens tested in stage 2 were first subjected to a strain ratio above the range 0.32–0.40, anticipated to result in a fatigue life of $<10^6$ load cycles. Results indicate that fatigue life of failed specimens was within the same range of the previous materials tested. However, increased variability was found in the fatigue life when the strain ratio was reduced to achieve 10^6 load cycles.

Recycled concrete was excluded from the fatigue analysis to develop a presumptive relationship since the particle-size distribution was too coarse (Figure 2). The effect of the poor grading was observed in the flexural strength and breaking strength tests. All the fatigue data (including the additional materials but excluding: PSG with 5% cement, calcrete limestone with 3% cement and recycled concrete with 3% cement) were used to evaluate the fraction of specimens that failed before 10^6 load cycles at different strain ratios (Figure 7). Approximately 60% of the specimens failed before 10^6 load cycles if the strain ratio applied was 0.44, and 10% of specimens failed before 10^6 load cycles if the ratio was 0.35. This distribution will be used to define the presumptive fatigue relationship in the next section.

4.3. *Presumptive laboratory fatigue relationships*

4.3.1. *Grading envelope*

As a step towards the development of a presumptive in-service fatigue relationship, the data presented in the preceding section were analysed to develop a presumptive laboratory fatigue relationship. As fatigue characteristics were found to vary markedly between materials, there was a need to limit the application of the fatigue relationship to a selected grading envelope and plasticity range.

As mentioned in Section 4.1, the fatigue results for the PSG with 5% cement were influenced by its fine grading. In addition, the breaking strains for the recycled concrete were very low, suggesting that its particle-size distribution was too coarse. Based on a review of state road authority specifications for cement-treated crushed rocks, a grading envelope (Figure 2) was selected that encompassed most of the crushed rocks currently being used. It was decided to develop a presumptive fatigue relationship for materials within this envelope. In terms of crushed rock plasticity, a maximum plasticity index (PI) of eight is commonly used. Hence, it was decided that the presumptive relationship also be applicable for materials with up to a maximum PI of 8.

4.3.2. *Development of fatigue relationship*

Based on the previous sections, it was decided that the fatigue relationship used to describe the laboratory fatigue life should have the following characteristics:

- applicable to all the cemented materials with grading and plasticity discussed in the preceding section,
- use breaking strain and initial strain to predict fatigue life (i.e. strain ratio),
- exclude modulus since minor improvements in the R^2 and standard errors were observed from the aggregated analysis,
- be conservative for the range of number of load cycles that will be relevant for design purposes (i.e. most of the pavements with cement-treated materials will be designed for 10^6–10^9 load cycles) and
- for convenience it should have a $\log(N) - \log(\text{strain ratio})$ form.

Based on these characteristics, the form of the fatigue relationship shown in Equation (5) was adopted. Equation (5) could be also expressed as

$$N = B \times \left[\frac{\mu\varepsilon_{\text{break}}}{\mu\varepsilon} \right]^A, \tag{5}$$

where N is the number of load cycles to failure, $\mu\varepsilon$ the initial elastic strain (microstrains), $\mu\varepsilon_{\text{break}}$ the strain at 95% of the breaking load (microstrains), 'A' the strain damage exponent and 'B' a model parameter.

To provide a conservative presumptive relationship it was decided to first select a conservative value for the strain ratio for a fatigue life of 10^6 load cycles (Figure 7). A strain ratio of 0.35 was selected as only 10% of the specimens fail before 10^6 cycles at this strain ratio. Applying this criterion in Equation (6) the fatigue relationship has to satisfy the following condition:

$$10^6 = B \times \left[\frac{1}{0.35} \right]^A. \tag{6}$$

To develop the presumptive fatigue relationship, the fitted lines from Figure 6 were then extrapolated to the range 10^6–10^8 load cycles (Figure 8). The extrapolation shows that fatigue lives for the weathered granite with 5% cement and modified PSG with 3% cement are lower for the 10^6–10^8 range of load cycles and the slope of these lines is higher. The other materials show a similar fatigue life with the calcrete limestone with 5% cement being the material with the highest sensitivity to load (highest damage exponent). It was decided that the presumptive fatigue relationship should be conservative due to the uncertainty of the extrapolations and therefore when plotted in Figure 7 should preferably be located below the extrapolated curves for the range 10^6–10^9 load cycles.

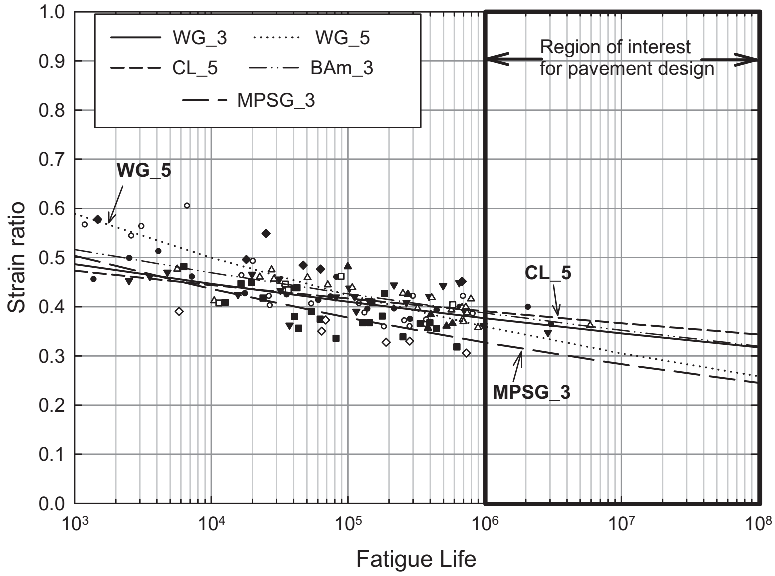

Figure 8. Extrapolated fatigue curves.

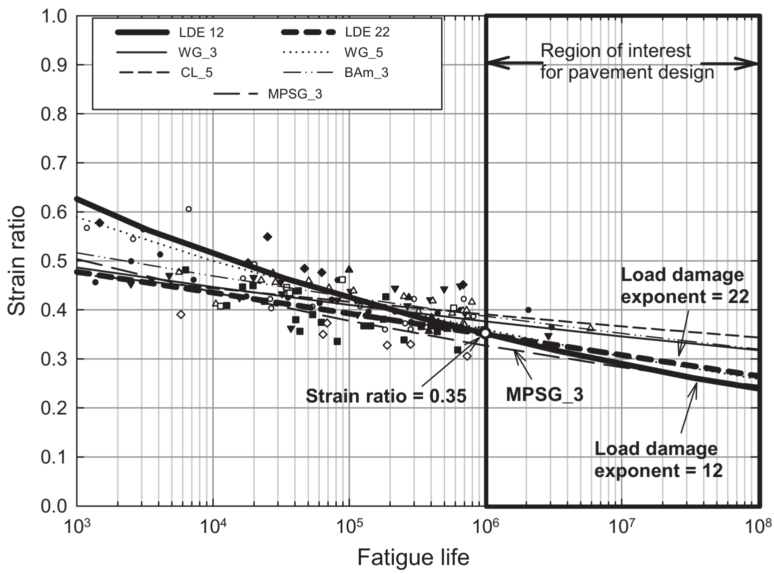

Figure 9. Extrapolated fatigue curves and fatigue relationship with LDE of 12.

The last step to develop the presumptive fatigue relationship was the definition of the strain-damage exponent (A). For this purpose, the line that represents the current damage exponent of 12 is plotted in Figure 9.

Using the condition in Equation (6) the coefficients A and B of the presumptive equation were adjusted for the damage exponent so that a strain ratio of 0.35 yields 10^6 load cycles.

$$N = 3.38 \times \left[\frac{\mu\varepsilon_{\text{break}}}{\mu\varepsilon}\right]^{12}. \tag{7}$$

Figure 9 shows that the fatigue relationship with a damage exponent of 12 (Equation (7)) is below the extrapolated fatigue data for the range of 10^6–10^8 load cycles for all the materials with exception of the modified PSG 3% (only from 10^6 to 10^7 load cycles).

The damage exponent to be adopted for design will be studied in the extended three-year project over the next few years, which will aim to develop a presumptive relationship to predict fatigue life of cemented materials to be included in Section 6.4 of the Austroads Guide to Pavement Technology – Part 2: Pavement Structural Design (Austroads, 2012).

5. Summary and conclusions

This paper presents the laboratory work conducted as part of an on-going research project on cemented materials. The four-point bending flexural beam test was used to characterise 10 different aggregates prepared as 14 cemented materials for flexural strength, breaking strain, modulus and fatigue life. The specimens were prepared in the laboratory under controlled mixing, compaction and curing conditions.

Based on the analysis of laboratory results the following conclusions were drawn:

- Flexural modulus of cement-treated materials tested ranged from 8000 to 20,300 MPa at 5–9 months cure age. This range is higher than presumptive values provided by the currently used Austroads Guide to Pavement Technology – Part 2: Pavement Structural Design.
- Minor statistical differences between the semi-log and log–log forms of the various fatigue relationships were observed.
- A range of forms of fatigue life models were investigated with the finding that initial modulus is not a significant predictor of fatigue life and that the ratio of initial strain to breaking strain provided a superior relationship.
- The fatigue findings were used to develop a presumptive laboratory relationship based on strain ratio. The presumptive fatigue relationship is conservatively based on a strain ratio of 0.35 for 10^6 load cycles. A strain damage exponent of 12 is recommended for pavement design.

Although these testing conditions represent ideal conditions of well-constructed and cured cemented materials pavement layers, the findings need to be verified with field data as other factors, such as micro-cracking, may significantly affect the flexural fatigue or flexural modulus results. In addition, results presented in this paper are valid for specimens cured between five and nine months in the laboratory. If a different curing time is adopted (e.g. 28 days) a shift factor should be assumed to take into account the increase in modulus and strength. Nevertheless, the application of the laboratory-based findings is expected to enhance knowledge of the performance of cemented materials and guide the implementation of a more accurate and reliable design relationship for cemented pavements applicable to the wide range of granular materials used in rural pavements in Australia.

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