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Keywords asphalt mixtures, granular Buton rock asphalt, fatigue performance

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Fatigue performance of Buton rock asphalt modified mixtures

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1. Introduction

According to studies, fatigue cracking is a result in fatigue failure due to repetitive stress and strains caused by load and environmental factors which is considered as a major distress happening in asphalt mixtures [1-3]. Fatigue cracking occurred when the tensile stress of materials exceeds the tensile strength due to the repetitive stress and strain. Cracks resulted in decreasing the structural capacity of the pavement and also increase the maintenance cost. Cracks also provide the pathways for water to penetrate the pavement layer and greatly accelerate the process of deterioration. Hence, the fatigue resistance of asphalt mixes is defined as the capability of the mixes to withstand the repetitive loading without any significant failure such as cracking or premature failure, developed under other circumstances such as environmental conditions.

The cracking is initially started from a micro-crack at the points where critical tensile strain/stress occur, and it then grows to form macro-crack, usually to an alligator cracking pattern, and finally penetrate the surface of pavement [4]. According to Di Benedetto *et al.*, the degradation processes during fatigue cracking are usually occurred in two phases. The first phase is manifested by initiation and propagation of a micro-crack network which results in a decrease in the modulus. This phase relates to degradation resulting from damage that is uniformly spread in the asphalt mixtures. In the second phase, from the combination of micro-crack, a macro crack appears which propagate within the materials.

Many researchers have concluded various concepts for evaluating fatigue resistance of mixes into three types: linier viscoelastic properties E_0 and φ_0 (determine at the beginning of each test, N = 100), life duration (number of cycles to specified failure criterion) and fatigue damage characteristics in the crack initiation phase. Hence, three concepts are widely used to study fatigue failure criteria of asphalt mixtures, including the classical (traditional), the fracture mechanics and the damage-energy (dissipated energy) approach [5, 6]. The fracture mechanics and the dissipated energy approach were included as mechanistic approach, while the classical criterion was categorized as phenomenological approach [7]. In the mechanistic approach, damage process in the fatigue occurs in two distinct stages: crack initiation and crack propagation (growth). Generally, fracture mechanics are used to characterize the crack propagation in asphalt mixtures, while the classical and the damage-energy approach are widely used to develop the fatigue prediction models for the crack initiation.

Materials characteristics is one of the factors that influence the fatigue cracking [7]. The use and development of modified binders in bituminous mixes, however, is important with the aim of increasing the fatigue performance. Many studies have carried out on asphalt mixtures to improve the resistance to fatigue failure. Polymer modified binder [8-10] and crumb rubber [11] have been found to improve fatigue life of asphalt mixtures. This research however, dealt with the benefit of using granular Buton Rock Asphalt (BRA) modifier binder with the aim to increase the fatigue life of asphalt mixtures and as a part of the another research presented elsewhere [12]. The raw materials of granular BRA modifier binder are found in Buton Island in Indonesia that is traditionally known as Buton asphalt (*asbuton*). Limited studies are found on fatigue performance of asphalt mixtures using asbuton. Subagio *et al.* [13] conducted a study on fatigue performance of hot rolled asphalt (HRA) by using asbuton mineral as filler. The results showed that the numbers of cycles to failure and initial stiffness for HRA-asbuton modified mixtures were higher compared with the HRA-fly ash modified mixtures. The main objective of this research is to assess the effect of granular BRA modifier binder on the fatigue life of asphalt mixtures.

2. Materials and methods

2.1 Materials

Class-170 (Pen 60/80) base asphalt binder was used for unmodified asphalt mixtures. The binder was classified according to the Australian Standard AS-2008 [14]. The BRA modified binder used for BRA modified asphalt mixtures consist of 80% of the base asphalt binder and 20% of the BRA natural binder (by total weight of asphalt binder in the mixtures) based on previous research [12]. Specification of the base asphalt binder and BRA modified binder are given in Table 1.

Figure 1 shows the form of BRA modifier binder (pellets) with size of 7 mm to 10-mm in diameter used in this study. Triplicate portion of granular BRA modifier binder were subjected to an extraction process [15]. The bitumen content test results found that, on average, about 70% mineral and 30% natural binder by total weight of materials formed the granular BRA modifier binder. The particle size distributions are as follows: 2.36 mm (100%), 1.18 mm (97%), 0.6 mm (92%), 0.3 mm (81%), 0.15 mm (61%), 0.075 mm (36%).

Table 1 Properties of bitumen

Bitumen property	Standard	Value	
		Base binder	BRA modified binder
Penetration (25°C; 0.1 mm)	ASTM-D5	67	59
Softening points (°C)	ASTM-D36	48	52.8
Ductility (25°C), cm	ASTM-D113	>100	>100
Mass loss (%)	ASTM-D1754	0.19	0.09
Ductility after TFOT (25°C), cm	ASTM-D113	>100	>100

Figure 1. The form of granular BRA modifier binder (pellets).

A crushed granite sourced from a local quarry in Western Australia was used in all of the mixtures. One typical dense graded used for unmodified and BRA modified asphalt mixtures with nominal aggregate size of 10 mm (DG 10) was based on Specification 504 [16]. The coarse and fine aggregates required to prepare the mixes were obtained by sieving on each sieve diameter.

As stated in mix design and specimen preparation that the same binder contents as for unmodified asphalt mixtures were used for the BRA modified asphalt mixtures in order to maintain consistency for comparison purposes. To focus on the contribution of granular BRA modifier binder, a 20 percent of granular BRA modifier binder was used in a single mix composition to produce nominally identical mixes. Table 2 shows that in the BRA modified asphalt mixtures, the substitution of the base asphalt binder allowed the proportion of fines passing 2.36 to be adjusted. The total mass of crushed fine aggregate was decreased and replaced with mineral contained in the granular BRA modifier with the aim of minimizing the variance in the gradation of aggregates.

Table 2 Final crushed aggregate gradation used in this study (percent passing)

Sieve size (mm)	BRA modifier content				Limit values
	0%	20%			
	Crushed aggregate	Crushed aggregat	BRA mineral	Final	
13.20	100	100		100	100
9.50	97.5	97.5		97.5	95-100
6.70	83.0	83.0		83.0	78-88
4.75	68.0	68.0		68.0	63-73
2.36	44.0	44.0	100	44.0	40-48
1.18	28.5	28.6	99.9	28.5	25-32
0.60	21.0	21.2	99.8	21.0	18-24
0.30	14.5	15.0	99.5	14.5	12-17
0.15	10.0	11.0	99.0	10.0	8-12
0.075	4.0	5.7	98.3	4.0	3-5

2.2 Mix design and specimen preparation

This study used Marshall mix design method to find out the optimum bitumen content (OBC) of unmodified asphalt mixtures based on specification 504 [16]. Specimen with dimension of 101 mm in diameter and 63.5 mm in height were compacted by applying 75 blows to each side using an automatic Marshall compactor for various contents of asphalt binder. The result shows that the voids of 5% was chosen to give binder content of 5.4% by mass of the asphalt mixtures as optimum bitumen content (OBC) values. To exhibit the benefit of granular BRA modifier binder mix in the asphalt mixtures, the BRA modified asphalt mixtures was designed to use the same binder content as unmodified asphalt mixtures. Hence, the OAC of 5.4% by weight of total mixtures was also used for BRA modified asphalt mixtures.

Table 3 shows the proportion of the base asphalt binder and the BRA modifier binder in unmodified and BRA modified asphalt mixtures, as well as the proportion of granular BRA (pellets) mixed into the mixtures. Furthermore, BRA modified asphalt mixtures specimens were manufactured as presented elsewhere [12]. The final dimension of beam specimens were 390±5 mm in height, 50±5 mm in depth and 63.5±5 mm in width. Nine specimens for unmodified asphalt mixtures and nine specimens for BRA modified asphalt mixtures were prepared in this study (triplicate specimens for each initial tensile strain).

Table 3. Proportion of materials used in asphalt mixtures

Materials	Percentage by total weight of mixtures (%)	
	BRA modifier content	
	0%	20%
1. Total binder content	5.40	5.40
a. Base binder	5.40	4.30
b. BRA modified binder	0.00	1.10
2. Total aggregate content	94.60	94.60
a. Crushed rock	94.60	92.10
b. BRA mineral	0.00	2.50
3. Granular BRA (pellets)	0.00	3.60

2.3 Fatigue performance characterization

Laboratory evaluation method of fatigue strength on four point bending equipment under repeated flexural bending test was done in accordance with Austroads AG:PT/T233 test method. The beam test under the controlled-strain mode of loading was applied for a dense grading of 10 mm of unmodified and BRA modified asphalt mixtures at the conditions as follows: test temperature of 20°C, three different peak tensile strain (400, 600 and 800 με), loading frequency of 10 Hz, mode of loading of continuous haversine, and air void content of 5%.

Before each testing was started, the specimen was placed inside the device cabinet for two hours to ensure that the temperature recorded during the test was the temperature of the test specimen and to maintain the temperature during the test. Then, the specimen was positioned in the loading frame cradle and clamped at the four points to hold the specimen in place as seen in Figure 2. After being clamped and before the test was run, the specimen was left a minimum of 30 minutes to enable the specimen clamping stress to be relieved. Initially, an application of 50 load cycles was used to determine and calculate the initial flexure stiffness and the cumulative initial dissipated energy.

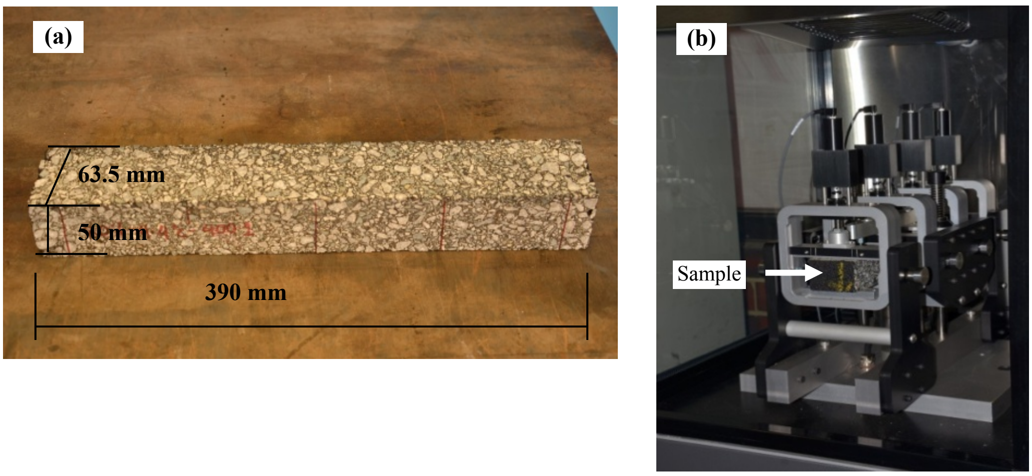

Figure 2. (a) final beam specimen, (b) set-up of the repeated flexural bending test

3. Results and Discussion

3.1 Fatigue life

The classical approach and energy stiffness ratio (ER) method developed by Abojaradeh [17] were used to analyse the results of the flexure fatigue tests as shown in Figure 3. In the classical, the failure point (N_{f50}) is assumed to be the number of loading corresponding to the 50% reduction of initial flexural stiffness recorded at 50, 200 and 500 cycles [1, 18]. The Australian standard Austroads AG:PT/T233 [19] has also adopted this model as the failure criterion. The ER defines the energy stiffness ratio as the ratio of the flexural stiffness at cycle-i (S_i) to the initial flexural stiffness (S_0) multiplied by the load cycle value at cycle-i (N_i). The fatigue failure (N_f) is determined by plotting the peak value of energy stiffness ratio to the number of cycles. During the test, the value of the ER improves as the number of cycles (N_i) increases until it reaches a peak value. Further, the value of the ER decreases suddenly after reaching its peak value. During this time, the stiffness of materials (S_i) is suddenly decreases even though the number of cycles (N_i) increases.

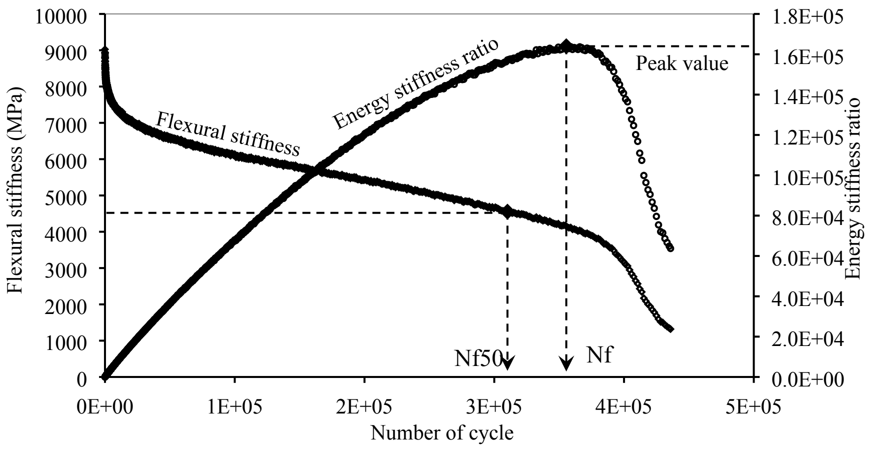

Figure 3. Energy stiffness ratio versus loading cycles (for specimen tested at 20°C and 400με)

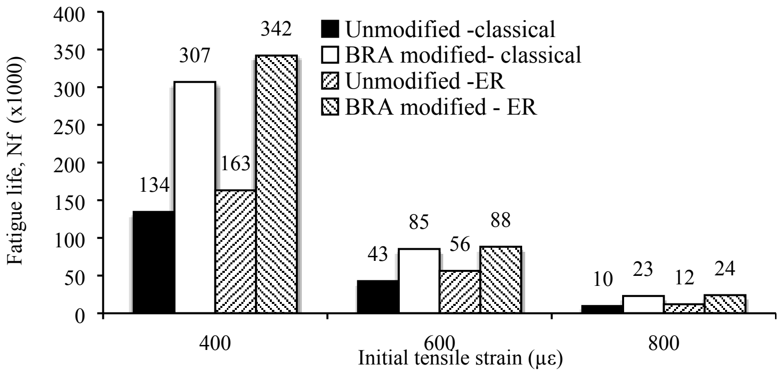

Figure 4. Mean fatigue life of asphalt mixtures

The results for the mean fatigue life of the asphalt mixtures using both classical and ER are shown in Figure 4. Comparison of the fatigue life for unmodified and BRA modified asphalt mixtures at the same initial tensile strain, indicates that the fatigue life of BRA modified was about 2.0 to 2.4 and 1.6 to 2.1 by using classical and ER, respectively, higher fatigue lives when compared to unmodified asphalt mixtures. The stiffness and nature of asphalt mixtures affect the magnitude of the strain [20]. The better performance of BRA modified asphalt mixtures in fatigue life may be attributed to the unique combination of base asphalt binder and BRA modifier binder compared to unmodified asphalt mixtures. Binder can be an important factor for higher fatigue life in BRA modified asphalt mixtures. According to Karami and Hamid [21], BRA modified asphalt mixtures are stiffer which may contribute to increase the fatigue life. Vazquez *et al*. [22] and Tarefder *et al*. [23] included asphalt binder grade and asphalt binder content as the factors most affecting resistance to fatigue cracking in asphalt mixtures. As Guler [24] and Khiavi and Ameri [1] argued, binder type is considered to be an important factor among the other factors in mixture variables such as aggregate gradation, aggregate type, binder content, compaction temperature and traffic loading, which can have a significant effect on the fatigue life of the bituminous mixes.

In addition, the ratio of flexural stiffness was defined by dividing the flexural stiffness for a given fatigue failure (S_f) by the initial flexural stiffness (S_0). The mean ratios of flexural stiffness obtained were 0.42 – 0.44 for unmodified asphalt mixtures and 0.48 for BRA modified asphalt mixtures, respectively as shown in Table 4. The results confirmed that the fatigue life of unmodified and BRA modified asphalt mixtures developed by using the ER are longer compared with the fatigue life of asphalt mixtures analysed by using the classical method. Similar observations have been reported by other researchers. Khiavi and Ameri [1] stated that the fatigue life based on the RDEC and DER criteria corresponds to 65% and 55% initial stiffness reduction. Maggiore [6] found that the fatigue life was longer than the 50% initial stiffness reduction. Other researchers, Rowe [25] and Walubita [26] stated that this phenomenon usually occurs in a range of 40-50%.

Table 4. Flexural stiffness ratio of asphalt mixtures using energy-stiffness ratio method

Initial tensile strain (με)	Unmodified			BRA modified		
	Initial stiffness (S_0) (MPa)	Stiffness at failure (S_f) (MPa)	Ratio (S_f/S_0)	Initial stiffness (S_0) (MPa)	Stiffness at failure (S_f) (MPa)	Ratio (S_f/S_0)
400	5044	2238	0.44	8639	4120	0.48
600	5155	2160	0.42	8562	4153	0.48
800	5365	2242	0.42	9002	4337	0.48

Figure 5 shows the sensitivity of the ER change due to the change in strain. The value of the ER at failure corresponded to the number of cycles at failure for all specimens on the log-log scale. The results show a straight line with a much higher coefficient of determination R^2 (0.996). The regression equation is then changed to become Equation 1. As the power of 1.023 is close to 1.0, the (S_f/S_0) value is 0.332. According to Abojaradeh **[17]** the (S_f/S_0) value is 0.3512 for controlled strain, and there is no difference between the curves for controlled strain and controlled stress.

$$\frac{N_f \text{ x } S_f}{S_o} = 0.332(N_f)^{1.028} \tag{1}$$

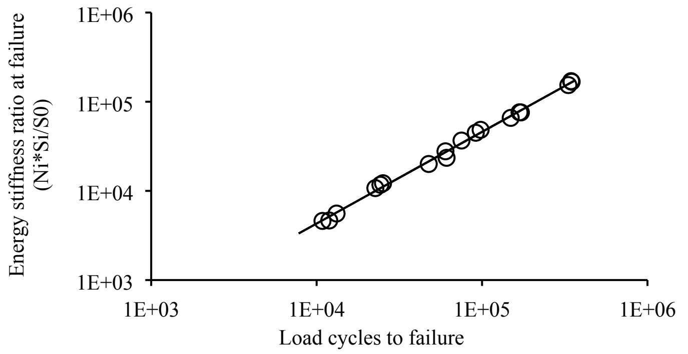

Figure 5. Energy stiffness ratio at failure vs load cycles at failure for controlled strain

3.2 Fatigue Life Prediction

The magnitude of tensile strain at the bottom of the asphalt layer is used as a criterion where the microcracking, crack initiation and failure have occurred. Accordingly, the relationship between the number of cycles to failure and strain is used as the basis for assessing fatigue performance and enables the determination of the thickness of the asphalt layer in structural pavement design. Shen and Carpenter [27] showed that tensile strain is the more important parameter for fatigue cracking. The results of controlled displacement can be explained with regard to the relationship between initial strain and load repetition, as shown in Equation 2 [7, 28, 29], represents the relationship between the radial strain at the bottom of the asphalt mixture layer and the number of load applications until the appearance of cracking in the pavement. The fatigue coefficient k_1 and k_2 may vary between models. Usually, the value of k_2 is in a range between 3 and 6, while k_1 is affected by several magnitudes. However, k_1 and k_2 are specific to the asphalt binder type, asphalt mixture type, volumetric composition, and the test parameters used in the laboratory characterization [7]. Furthermore, Shen and Carpenter [27] stated that the stiffness of asphalt mixtures affects the fatigue life. A modified Equation 2 is given as Equation 3 to define the mixture's stiffness dependent behaviour [7, 27, 30].

$$\mathrm{N_f} = \mathrm{k_1}\left(\frac{1}{\varepsilon_0}\right)^{\mathrm{k_2}} \tag{2}$$

where N_f is number of loading applications to failure at a particular level of initial strain, ε_0 is initial tensile strain, and k_1 and k_2 are material coefficients derived from fitting the data.

$$N_f = a\left(\frac{1}{\varepsilon_t}\right)^b \cdot \left(\frac{1}{S_0}\right)^c \tag{3}$$

where N_f is number of loading application to failure, ε_t is tensile strain, S_0 is mixture stiffness, and *a, b,* and *c* are material coefficients, derived from fitting the data. Figure 6 shows the regression analysis developed to determine the phenomenological and ER approach for unmodified and BRA modified asphalt mixtures.

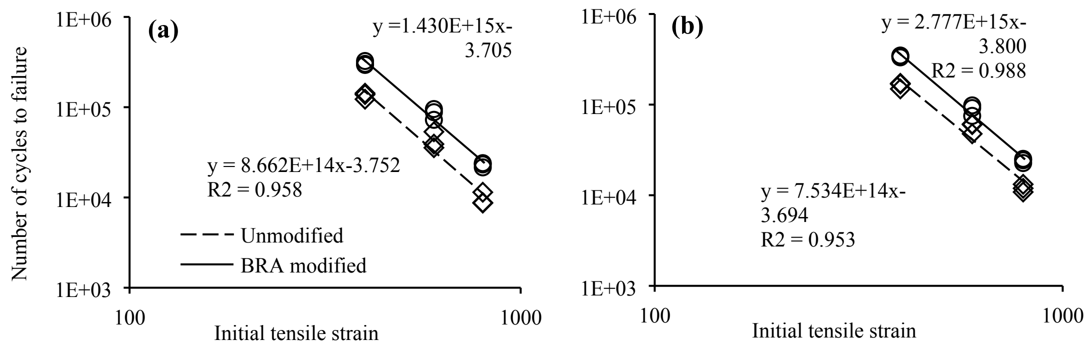

Figure 6. Fatigue characteristics of unmodified and BRA modified asphalt mixtures, (a) clasical approach, (b) ER approach

The intercept and the slope, k_1 and k_2 respectively, are the important variables obtained from the test. Under the strain-controlled mode of loading, these variables represent the properties of materials used in asphalt mixtures and typical values for asphalt mixtures. According to Ghuzlan and Carpenter [31], k_1 and k_2 can be used in the fatigue based mechanism design procedures and the typical range of k_2 values are between 3 and 6. In this study, it was noted that all of the slopes of the fatigue curve (k_2) were within this range even though in some fatigue models k_2 was fixed to specific number, as in the Asphalt Institute and Illinois fatigue equation, where the k_2 value is fixed to 3.29 and 3.0 respectively. Ghuzlan and Carpenter [32] argue that k_1 and k_2 is fundamental values of the asphalt mixtures. Figure 7 shows the correlation between the k_1 and k_2 values which gives a good correlation (R^2 = 0.895 and 0.927). As shown, the k_1 and k_2 values are located in one line in spite of the different mixture properties. The relationship shown here is consistent with the finding of other researchers [31].

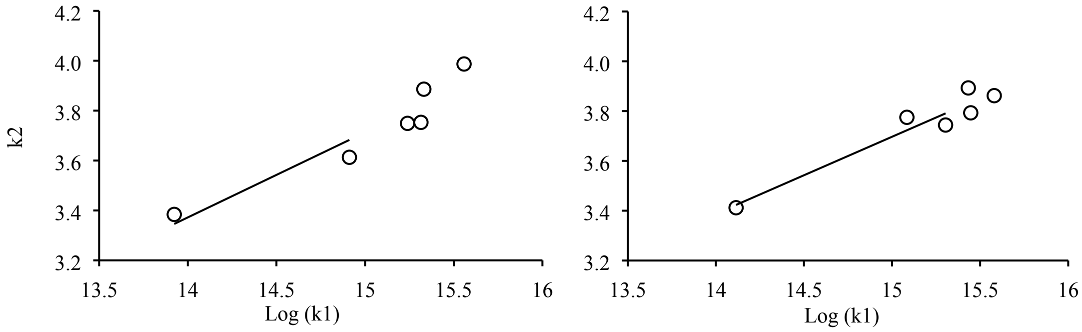

Figure 7. The relation of k_1-k_2 for all of asphalt mixtures using: (a) classical approach, (b) ER approach

In addition, Figure 8 shows the effect of using BRA modifier binder on k_1-k_2 relation. Both lines are not close and it can be proven that there is a significance difference between two lines (at 95% level of significance). Therefore, it is concluded that the use of BRA modified binder in asphalt mixtures has a significant effect on the k_1-k_2 relation.

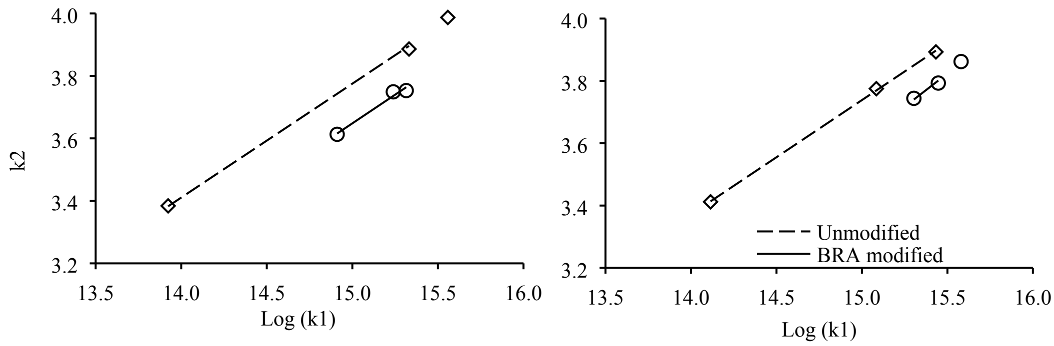

Figure 8. The relation of k_1-k_2 for unmodified and BRA modified asphalt mixtures, (a) using classical approach, (b) using energy-stiffness ratio approach

In accordance with Equation 3, statistical analysis was carried out to develop the strain-stiffness relationship between the number of cycles (N_f, in cycles) as a dependent variable and initial strain (ε_0, in με) and the initial flexural stiffness (S_0, in MPa) as independent variables for the unmodified and BRA modified asphalt mixtures as presented in Table 5.

Table 5. Prediction models for the number of cycles (N_f)

Asphalt mixtures	Classical approach		ER approach	
	Equation	R2	Equation	R2
Unmodified	$N_f = 5.105E + 17\left(\frac{1}{\varepsilon_0}\right)^{3.711}\left(\frac{1}{S_0}\right)^{0.770}$	0.937	$N_f = 1.505E + 16\left(\frac{1}{\varepsilon_0}\right)^{3.660}\left(\frac{1}{S_0}\right)^{0.375}$	0.938
BRA modified	$N_f = 1.832E + 28\left(\frac{1}{\varepsilon_0}\right)^{3.518}\left(\frac{1}{S_0}\right)^{3.457}$	0.992	$N_f = 9.332E + 27\left(\frac{1}{\varepsilon_0}\right)^{3.620}\left(\frac{1}{S_0}\right)^{3.304}$	0.996

Figure 9 shows the effect of initial flexural stiffness on the fatigue life of unmodified and BRA modified asphalt mixtures, in accordance with Equations in Table 5. The equations are plotted at various values of initial tensile strain (200 με to 1200 με) and initial flexural stiffness (1000 MPa to 20,000 MPa). The cycle to failure (N_f) decreases as initial flexural stiffness (S_0) increases for both unmodified and BRA modified asphalt mixtures. The cycle to failure of BRA modified is higher than for the unmodified asphalt mixtures until initial flexural stiffness (S_0) reaches about 13,030 MPa and 11,500 MPa for the classical and ER approaches, respectively. However, the cycle to failure (N_f) values were observed to be lower for the BRA with a higher initial flexural stiffness (S_0) than those mentioned above.

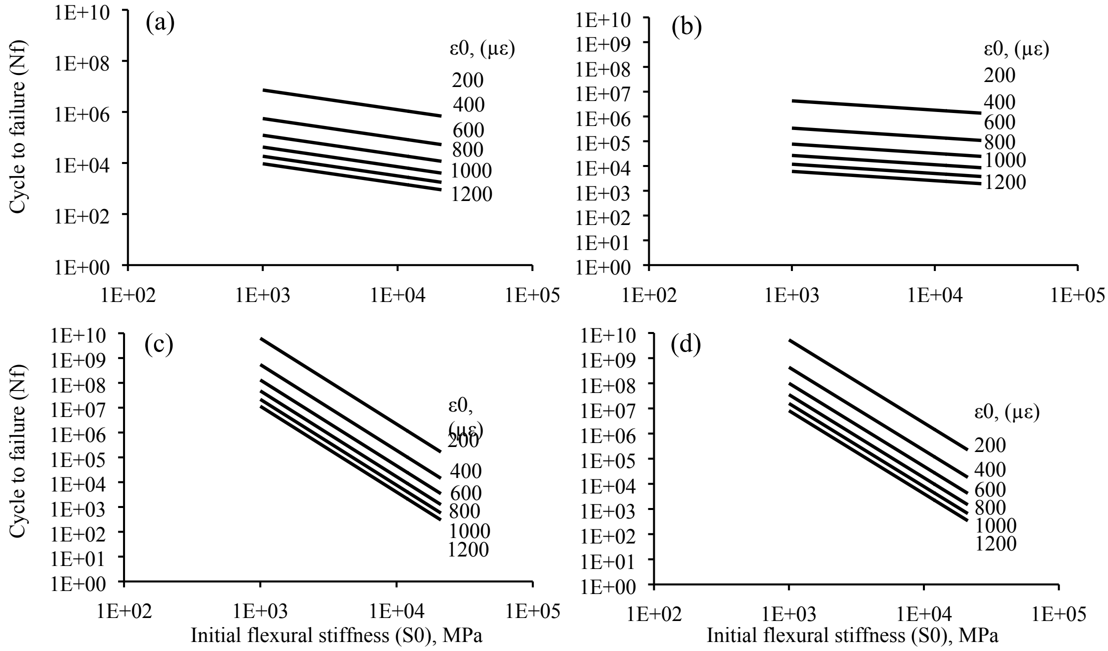

Figure 9. Strain-stiffness relationship between N_f, ε_0, and S_0: (a) Unmodified (classical), (b) Unmodified (ER), (c) BRA modified (classical), and (d) BRA modified (ER)

Similar observations have been reported by other researchers, such as Monismith *et al.* [30] who stated that fatigue life is influenced by the flexural stiffness of asphalt mixtures. Accordingly, the substitution of base asphalt binder with BRA modifier binder which resulted in an initial flexural stiffness of BRA modified asphalt mixtures greater than 13,030 MPa (using the classical approach) or 11,500 MPa (using the ER approach), respectively, will have a cycle to failure lower than for unmodified asphalt mixtures.

3.3 Dissipated Energy

The *damage-energy approach* may be analysed with the dissipated energy concept. The energy dissipated can be used as an excellent indicator of fatigue response during each loading cycles, because it captures both the elastic and viscous effects. The dissipated energy (DE) concept defines that the fatigue life is a function of the accumulation of dissipated energy on each loading cycles where the dissipated energy in a cycle is affected by the energy dissipated in the previous cycles.

Baburamani [7] stated that as viscoelastic materials, asphalt mixtures can be analysed in term of the energy dissipated in the specimen during testing. He argued that the rheology of asphalt mixtures as a function of temperature, loading frequency and strain/stress level influence the energy dissipated. As Van Dijk *et al.* cited Baburamani [7] said, the total energy dissipated during fatigue test may be controlled with the fatigue life and change in mechanical properties of asphalt mixtures. Further, the energy dissipated can be used to explain the decrease in mechanical properties such as flexural stiffness loss during test. Hopman *et al.* [33] said that the energy dissipated per cycle controls the fatigue damage of asphalt mixtures. According to Hassan and Khalid [34] the dissipated energy is a difference between the induced energy and released energy due to load application and relief. Those, the energy dissipated in each pulse of loading cycle to cause incremental damage exist on asphalt mixtures and then will lead to crack extension, plastic deformation and thermal energy [18].

In viscoelastic materials, deformation as well as strain increase over the time as long as a constant load is applied, and when the load is removed, some deformation are recoverable and some of them are unrecoverable. As a viscoelastic material, the dissipated energy in each loading cycle for asphalt mixtures is observed as the area under the stress-strain curve of the hysteresis loop and calculated using the following Equation 4 [1, 3, 7, 18, 34]. The unloaded material has a different path to that when load is applied. Thus, the phase lag is recorded between the applied stress and the measurement strain. Further, the energy is dissipated in the form of mechanical work, heat generation or damage. In strain-controlled fatigue test, the energy are decreased when the number of load cycles increased as the stress decreases, while for stress-controlled fatigue test, the dissipated energy per cycles increase when the number of cycles increased [17, 35].

$$DE_i = \pi.\sigma_i.\varepsilon_i.\sin\left(\delta_i\right) \tag{4}$$

where DE_i is dissipated energy in cycle i; σ_i is stress level in cycle i; ε_0 is strain level in cycle i; and δi is phase angle in cycle i. Maggiore *et al.* [6, 32] presented Equation 5 to relate the cumulative dissipated energy and the number of cycles to failure as follows:

$$W_f = A(N_f)^z \tag{5}$$

where W_f is cumulative dissipated energy to failure; N_f is number of load cycle to failure; and A, z are mixture dependent constants.

Further, Pronk and Hopman [36] proposed an energy-ratio (ER) concept to define fatigue life of asphalt mixtures in the strain-controlled tests as a ratio of the initial dissipated energy to the dissipated energy at the i_{th} cycle multiplied by the load cycle n. Pronk [37] proposed a concept of energy ratio to define failure as the ratio of the cumulative dissipated energy at cycle n to the dissipated energy for cycle n. Rowe and Bouldin [38] developed the definition of failure by introducing a new definition as the load cycle multiplied by the stiffness at that cycle. Abojaradeh [17] introduced a fatigue failure criterion based on the energy stiffness ratio. Ghuzlan and Carpenter [32, 39] and Carpenter *et al.* [28] developed and proposed the dissipated energy ratio (DER) method to define a failure point of fatigue life in asphalt mixtures.

An example of dissipated energy evolution with number of cycles and the typical results of flexural stiffness and cumulative dissipated energy in this research are presented in Figure 10. These figures show that during a fatigue test, the stiffness of asphalt mixtures reduces resulted the micro cracks are induced in the materials when repeated stress are applied to the specimen below the failure stress; therefore the dissipated energy varies per each loading cycle and it decreases for controlled strain tests. As Carpenter and Shen [40] said, energy dissipated in a loading cycles is affected by the energy applied in the previous cycles. Baburamani [7] suggested that the rate of dissipated energy change per cycle is a better indicator of initiation and growth of damage or cracking. In this study, the dissipated energy was obtained for each cycle for both unmodified and BRA modified asphalt mixtures.

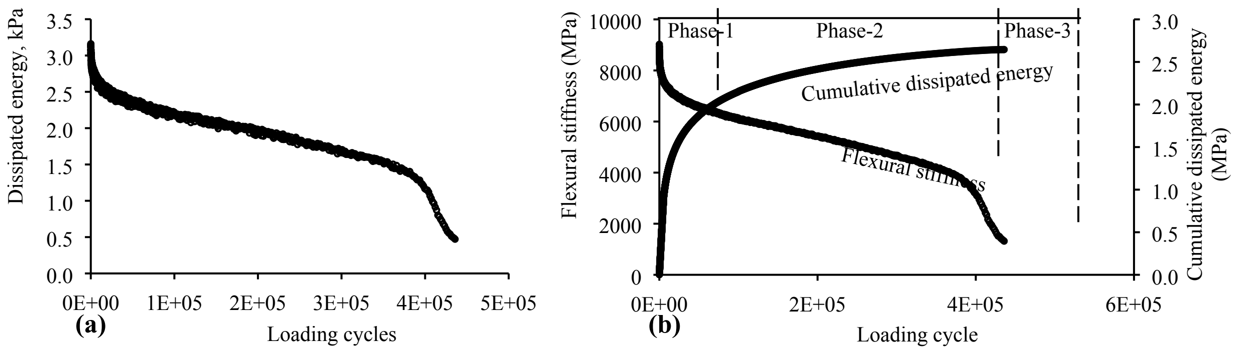

Figure 10. Typical results of fatigue test: (a) evolution of dissipated energy per cycle, (b) progression curve of flexural stiffness and cumulative dissipated energy

Figure 10(b) plotted the relationship between flexural stiffness and number of cycles. Three phase were observed for flexural stiffness, which is similar to those presented by Hassan and Khalid [34], Di Benedetto *et al.* [41], and Maggiore *et al.* [42, 43]. Phase 1 is characterised by a rapid reduction in flexural stiffness due to the repetitive excitation, which is then followed by Phase 2, where the reduction in the stiffness modulus shows as an approximately linear.

Di Benedetto *et al*. discussed that in phase 1, the decrease in flexural stiffness is not only considered by fatigue damage, but heating and a third phenomenon also play important role. In this phase, the stiffness loss is totally recoverable. In phase-2, the role of fatigue on the flexural stiffness decrease is predominant. The effect of thermal heating is small during this phase. However it has still to be considered. Phase-1 and phase-2 correspond to crack initiation process in the asphalt mixtures. Finally, the flexural stiffness is exhibited a marked drop in phase-3, after passing through an inflection point with load cycles culminating in failure. In phase-3, local crack propagation occurs. Further, the macro-crack or cracks starts to develop and global failure is obtained at the end of this phase [41].

As seen in Figure 10, with the test carried out in constant strain mode, the increase in number of loading cycles resulted in the flexural stiffness decreases, and cumulative dissipated energy increases. As rapid reduction in flexural stiffness at phase-1, however, the cumulative dissipated energy increases rapidly. Further, the cumulative dissipated energy increases linearly at phase-2. According to Di Benedetto *et al.* [41], the fatigue damage could be characterised only by phase-2, and Equation 6 [34] was used to obtained the damage rate dD/dN as a function of the slope of the line in phase-2.

$$\frac{dD}{dN} = \left(\frac{1}{E_{00}}\right) * \left(\frac{dE^{*}}{dN}\right) \tag{6}$$

where E_{00} is y-axis intercept of the fitted straight line; dE^*/dN is slope of the fitted straight line of phase-2.

The results of damage parameter obtained in accordance with the progression curve of flexural stiffness and cumulative dissipated energy are presented in Table 6 and Table 7, respectively, from which it can be noticed that BRA modifier binder had an influence on the damage parameters of asphalt mixtures. Lytton *et al.* [44] suggested that the rate of change of dissipated energy per cycle is better indicator of initiation and growth of damage or cracking. Comparing the damage parameter at the same initial tensile strain, the slopes and damage rate values for unmodified asphalt mixtures in both flexural stiffness and cumulative dissipated energy progression curve were higher compared with the BRA modified asphalt mixtures. These results revealed that BRA modified asphalt mixtures have much better resistance to fatigue failure than unmodified asphalt mixtures.

Table 6. The summary of damage parameter of unmodified and BRA modified asphalt mixtures based on the flexural stiffness progression curve

Initial tensile	Unmodified			BRA modified		
strain (με)	*dE*/dN*	E_{00}	*dD/dN*	*dE*/dN*	E_{00}	*dD/dN*
400	0.00775	3641	2.153E-06	0.00768	6885	1.116E-06
600	0.0267	3696	7.210E-06	0.0159	5839	2.701E-06
800	0.1581	4175	3.776E-05	0.0606	5962	9.894E-06

Table 7. The summary of damage parameter of unmodified and BRA modified asphalt mixtures based on the cumulative dissipated energy

Initial tensile	Unmodified			BRA modified		
strain (με)	*dE*/dN*	E_{00}	*dD/dN*	*dE*/dN*	E_{00}	*dD/dN*
400	1.861E-06	1.357	1.368E-06	1.303E-06	2.148	5.616E-07
600	0.0012	59	2.136E-05	0.0006	123	6.109E-06
800	0.0032	21	1.755E-04	0.0016	70	2.281E-05

Figure 11 shows a relationship between the number of cycle to failure and cumulative dissipated energy to the failure point for unmodified and BRA modified asphalt mixtures using classical and ER approach. The failure point for unmodified and BRA modified asphalt mixtures were presented elsewhere [12]. The figure was developed for all specimens tested at 400 με, 600 με, and 800 με as some factors such as temperature, loading frequency, and mode of loading did not seem to have an effect on this relationship. It can be seen that the slopes (k_2) of the curves for BRA modified asphalt mixtures was higher compared with unmodified asphalt mixtures. The results reveal that the sensitivity of cumulative dissipated energy to number of cycle to failure for BRA modified asphalt mixtures was higher compared with the unmodified asphalt mixtures.

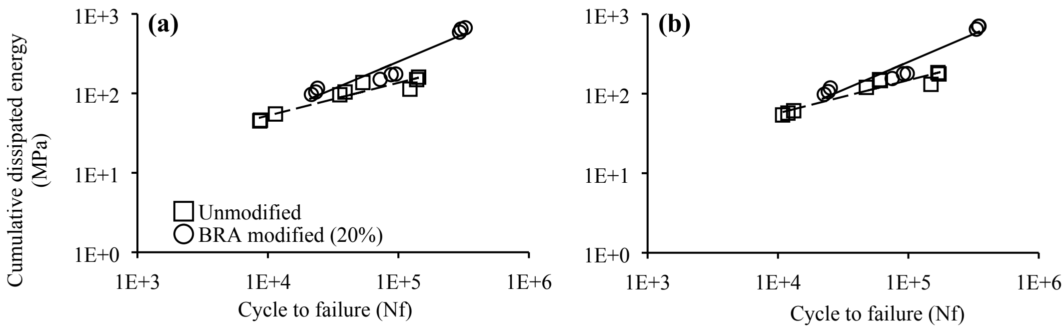

Figure 11. Relationship between cycle to failure and cumulative dissipated energy: (a) classical approach; (b) ER approach

Furthermore, Figure 12 shows the cumulative initial dissipated energy values for unmodified and BRA modified asphalt mixtures recorded at 50 cycles. The initial tensile strain seems to have a significant effect on cumulative initial dissipated energy for both asphalt mixtures. A higher the initial strain resulted in a higher cumulative initial dissipated energy. It can be seen that the cumulative initial dissipated energy values for BRA modified asphalt mixtures are higher compared with unmodified asphalt mixtures. The cumulative initial dissipated energy values for BRA modified asphalt mixtures increase by 1.5 to 1.8 times.

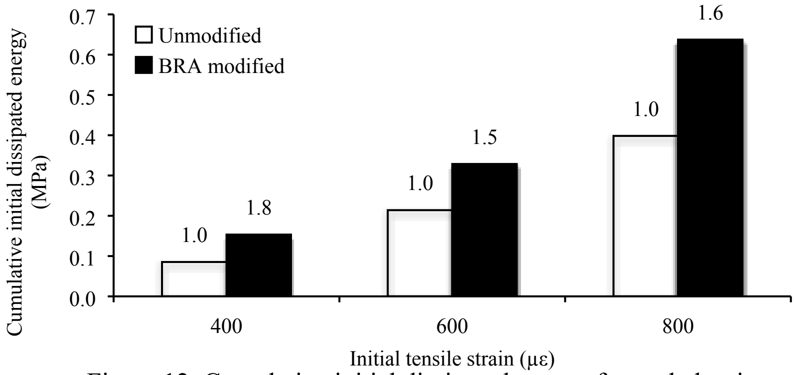

Figure 12. Cumulative initial dissipated energy for asphalt mixtures

4. Conclusion

Several repeated flexural bending tests in laboratory were conducted to evaluate the effect of granular BRA modifier binder on fatigue performance of asphalt mixtures. Substituting 20% BRA modifier binder led to a significant increase in the cumulative initial dissipated energy. The damage on asphalt mixtures due to the fatigue response of asphalt mixtures were observed by using energy dissipated and flexural stiffness progression curve. The damage rate for BRA modified asphalt mixtures was lower compared with the unmodified asphalt mixtures. The results showed that BRA modified asphalt mixtures have better fatigue performance than unmodified asphalt mixtures.

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