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Assessing the fatigue performance of buton rock asphalt modified mixtures

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Abstract

The main objective of this research was to assess the effect of granular Buton Rock Asphalt (BRA) modifier binder on the fatigue life of asphalt mixtures. This involved the laboratory evaluation of fatigue strength on four-point bending equipment under repeated flexural bending test in accordance with Austroads AG: PT/T233. 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 20°C test temperature, three different peak tensile strain including 400, 600, and 800 με, 10 Hz loading frequency, continuous haversine mode of loading, and 5% air void content. The results showed the number of cycles for BRA modified asphalt mixtures increased from 2.0 to 2.4 and 1.6 to 2.1 as observed with the classical and energy stiffness ratio approaches, respectively. According to the strain approach, the number of cycles to failure for BRA modified asphalt mixtures was higher when compared with unmodified ones. Meanwhile, the strain-stiffness approach showed the initial flexural stiffness (S_0) affected the fatigue life for both mixtures. Furthermore, the regression equations model to predict the fatigue life based on the strain and strain-mix stiffness approaches were developed. It was discovered that using BRA modifier binder in asphalt mixtures had a significant effect on the relationship between intercept (k_1) and slope (k_2) variables while the damage rate was lower compared to the unmodified asphalt mixtures.

Keywords: Asphalt mixtures; Granular Buton rock asphalt; Fatigue performance

1. Introduction

According to studies [1-3], fatigue cracking is produced by fatigue failure due to repetitive stress and strains caused by load and environmental factors and has also been considered as a major form of distress in asphalt mixtures. It occurs when the tensile stress of the materials exceeds the tensile strength due to repetitive stress and strain. The cracks formed to decrease the structural capacity of the pavement to cause an increment in the maintenance costs. It also provides the pathways for water to penetrate the pavement layer and greatly accelerate the deterioration process. Therefore, the fatigue resistance of asphalt mixtures is defined as the capability to withstand repetitive loading without any significant failure such as cracking or premature failure developed under other circumstances such as environmental conditions.

The cracking starts with a microcrack at the points where critical tensile strain/stress occurs, and later develops to form a macrocrack in the form of an alligator cracking pattern, and finally penetrate the pavement surface [4]. According to Di Benedetto et al. [5], the degradation process during fatigue cracking is divided into two phases. Firstly, there is an initiation and propagation of a microcrack network which causes a decrease in the modulus and also related to the degradation caused by the damage uniformly spread in the asphalt mixtures. This stage is known as the initiation phase. Secondly, a macrocrack appears from the combination of microcracks which propagates further within the materials. The second phase is called the propagation phase.

Several concepts have been developed by researchers to evaluate the fatigue resistance of mixtures. They are grouped into three including linear viscoelastic properties E_0 and φ_0 (determined at the beginning of each test at N = 100), life duration (number of cycles to specified failure criterion), and the fatigue damage characteristics in the crack initiation phase. These three concepts are widely used to study fatigue failure criteria of asphalt mixtures, including the classical or traditional, fracture mechanics, and the damage-energy or dissipated energy approaches [1,3,6,7]. Baburamani [8] reported the fracture mechanics and dissipated energy were included as a mechanistic approach while the classical criterion was categorized as a phenomenological approach. In the mechanistic approaches, the damage process in the fatigue occurs in two distinct stages which are the crack initiation and crack propagation or growth. Generally, fracture mechanics are used to characterize the crack propagation in asphalt mixtures while the classical and damage-energy are applied to develop the fatigue prediction models for the crack initiation.

Materials characteristics are one of the factors influencing fatigue cracking. This has led to the development and use of modified binders in bituminous mixtures to increase the fatigue

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performance. Many studies have been conducted to improve the resistance of asphalt mixtures to fatigue failure, for example, the use of polymer-modified binder [9-11] and crumb rubber [12] have been reported to have a significant effect on improving the fatigue life of asphalt mixtures. Limited studies were found on fatigue performance of asphalt mixtures using BRA, for example, the feasibility of using BRA filler for fatigue performance was researched and the results showed the number of cycles to failure (N_f) for Hot Rolled Asphalt (HRA) mixtures containing BRA filler were higher compared to unmodified mixtures at all level of stress [13]. Moreover, Lv et al. [14] conducted a low-temperature bending beam test at -10°C and found the addition of BRA was able to improve mixtures' resistance to low-temperature cracks. Li et al. [15] also conducted fracture and fatigue damage tests on cylindrical specimens and found rock asphalt to have increased the fracture energy and had a longer fatigue life compared to the control mixtures. This research was, however, conducted to study the potential use of granular BRA modifier binder to increase the resistance of asphalt mixtures to fatigue cracking by investigating the fatigue life, fatigue life prediction, and dissipated energy of Buton rock asphalt mixtures. The raw materials were found traditionally on Buton Island in Indonesia.

2. Materials and methods

2.1. Materials

Class-170 (Pen 60/80) base asphalt binder was used for the unmodified asphalt mixtures and classified according to the Australian Standard AS-2008 [16]. The BRA modified binder used was made by replacing 20% base asphalt binder by total weight in the mixtures with BRA natural binder and the specifications are presented in Table 1.

Fig. 1 shows the granular BRA modifier binder was in pellet form with the diameter ranging between 7 mm and 10 mm and triplicate portions were subjected to an extraction process [17]. Moreover, the bitumen content test showed it averagely consisted of 70% mineral and 30% binder by total weight of materials. The mean particle size distributions for each sieve include 2.36 mm (100%), 1.18 mm (97%), 0.6 mm (92%), 0.3 mm (81%), 0.15 mm (61%), and 0.075 mm (36%).

Crushed granite aggregates were used such that both the unmodified and BRA-modified asphalt mixtures applied a typical dense graded aggregate with a maximum size of 9.5 mm (DG 10) based on Specification 504 [18]. Moreover, the coarse and fine aggregates required were obtained using each sieve diameter with the aggregate properties determined summarized in Table 2.

Table 1
Properties of bitumen.

Bitumen property	Standard	Value	
		Base binder	BRA modified binder
Penetration (25°C; 0.1 mm)	AS 2341.12-1993	67	59
Softening points (°C)	AS 2341.18-1992	48	52.8
Ductility (25°C), cm	AS 2341.11-1980	>100	>100
Mass loss (%)	AS 2341.10-2015	0.19	0.09
Ductility after TFOT (25°C), cm	AS 2341.11-1980	>100	>100

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

Table 2
Properties of aggregates.

Properties	Standard	Value
Los Angeles value (%)	AS 1141.23-1995	21.8
Flakiness index (%)	AS 1141.15-1999	14.14
Apparent coarse aggregate density (t/m^3)	AS 1141.5-2000	2.63
Apparent fine aggregate density (t/m^3)	AS 1141.5-2000	2.60

2.2. Mix design and specimen preparation

The Marshall mix design was used to determine the optimum bitumen content (OBC) for the unmodified asphalt mixtures based on specification 504 [18]. Specimens in triplicate with dimensions 101 mm diameter and 63.5 mm height were compacted by applying 75 blows to each side of the specimen using an automatic Marshall compactor for different contents of asphalt binder. The result showed a void of 5% was selected to produce a binder content of 5.4% as the optimum bitumen content (OBC) (by mass of asphalt mixture). Moreover, the BRA-modified asphalt mixtures were designed to exhibit the benefits attached to mixing granular BRA modifier binder into the asphalt mixtures. Therefore, the same binder content used in unmodified asphalt mixtures was also applied in BRA-modified asphalt mixtures in order to maintain consistency for comparison purposes. According to Walubita et al. [19] and Walubita et al. [20], the cracking resistance potential of mixtures increases with the content of the asphalt binder. Therefore, the 5.4% OBC by weight of the total mixtures was also used for the BRA-modified asphalt mixtures. Furthermore, in order to focus on the contribution of the granular BRA modifier binder, 20 percent of the material was used in a single mix composition to produce a nominally identical mixture. Table 3 shows the substitution of base asphalt binder allowed the proportion of fines passing 2.36 to be adjusted. Therefore, the total mass of crushed fine aggregate was decreased and replaced with the mineral contained in the granular BRA modifier to minimize the variance in the aggregate gradation.

Table 4 shows the proportion of the base asphalt and BRA modifier binders in unmodified and BRA-modified asphalt mixtures, as well as the proportion of granular BRA (pellets) mixed. Furthermore, after each size of crushed aggregates had

Table 3
Final crushed aggregate gradation used (percent passing).

Sieve size (mm)	BRA modifier content				Limit values
	0%	20%			
	Crushed aggregate	Crushed aggregate	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

Table 4
Proportion of materials used in asphalt mixtures (percentage by total weight of mixtures (%)).

Materials	BRA modifier content	
	0%	20%
1. Total binder content	5.40	5.40
Base binder	5.40	4.30
BRA-modified binder	0.00	1.10
2. Total aggregate content	94.60	94.60
Crushed rock	94.60	92.10
BRA mineral	0.00	2.50
3. Granular BRA (pellets) *)	0.00	3.60
Number of samples (specimen)	9	9

*) Granular BRA (pellets) = 1b. BRA-modified binder + 2b. BRA mineral.

been weighed, they were blended manually in a pan and heated in an oven at a temperature of 105ºC for 24 hours. This was followed by heating the blend aggregates in the same oven at 150ºC before mixing with asphalt binder for approximately two hours. Meanwhile, the base asphalt and granular BRA modifier binders (pellets) were placed in the same bowl and the mixture was later placed in an oven at a temperature of 150±5 ºC for 30 to 60 minutes with frequent manual stirring to blend and incorporate the two as BRA modified binder. This was followed by the placement of the previously blended aggregates in the same bowl with the BRA modified binder and mixed using mixing equipment for 1.5 minutes.

Laboratory mixtures were prepared based on the procedure for sampling loose asphalt described in AS 2891.1.1-2008 [21], and immediately compacted in a slab mould using a roller compactor as seen in Fig. 2 in accordance with Austroads standard AG: PT/T220 [22]. The specimens consisted of slabs with 400 mm length, 305 mm width, and 75 mm height prepared according to Austroads standard AG: PT/T233 [23]. The compaction process was aimed at optimizing the packing of the aggregates and to uniformly distribute the bitumen and air voids. According to Hartman et al. [24], rolling wheel compaction appears to be the most appealing compared to other devices such as Marshall, kneading, and gyratory due to its similarities with field compaction, especially the action and air void distribution. After 24 hours the compaction process, the slabs were cut to produce the final beam specimens with 390±5 mm height, 50±5 mm depth, and 63.5±5 mm width as illustrated in Fig. 3. Therefore, nine specimens for

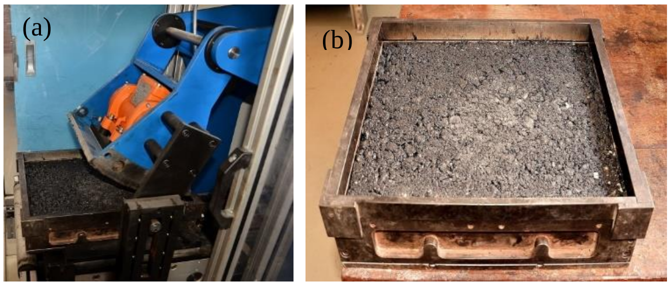

Fig. 2. Compaction process for the slabs in the flexural bending test.

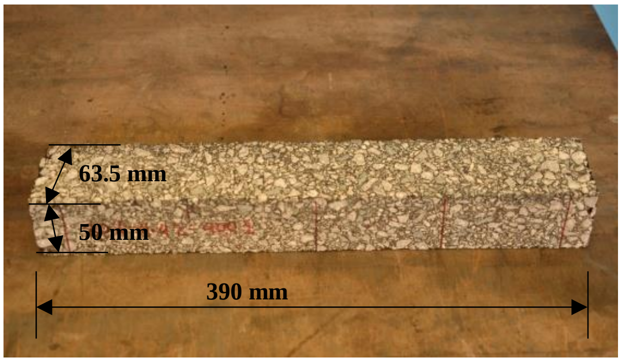

Fig. 3. Final beam specimen.

unmodified asphalt mixtures and nine for BRA-modified asphalt mixtures were prepared in this study due to the triplication of the specimens for each initial tensile strain.

2.3. Fatigue performance characterization

The laboratory fatigue investigations on bituminous mixtures were based on several types of tests including repeated indirect tensile strength on cylindrical specimens, two-point bending on trapezoidal and prismatic specimens, and three- and four-point bending on prismatic beam specimens [25]. The similarity of these methods is subjecting the sample to a bending load. For example, a beam used in the four-point fatigue tests simulates traffic loading by a vertical deflection while flexure beam fatigue tests were conducted in the laboratory to simulate the flexural stress pattern found in situ with the results reflecting the possible weakness in the asphalt mixtures. However, according to Hakimelahi et al. [2], the beam fatigue test requires a long testing period and its high variability makes it impractical for QC/QA testing.

The laboratory evaluation methods for fatigue strength on four-point bending equipment under repeated flexural bending test was conducted in accordance with Austroads AG: PT/T233 [23]. It was established on the Strategic Highway Research Program (SHRP) Designation M-009, “Standard Method of Test for Determining the Fatigue Life of Compacted Bituminous Mixtures Subjected to Repeated Flexural Bending”. The tests were performed under the strain-controlled mode of loading and the standard reference conditions include test temperature of 20±0.5°C, three different peak tensile strain at 400, 600, and 800 $\mu\varepsilon$, loading frequency of 10±0.1 Hz, continuous haversine mode of loading, and air void content of 5±0.5%. The unmodified and BRA modified asphalt mixtures used a 5±0.5% air void content based on the standards instituted by Austroads AG: PT/T220 [22]. The target density was 95 percent theoretical maximum which corresponds to 5 percent air voids. Meanwhile, any specimen with a significant difference (>0.5 percent) in air voids from the desired value was rejected.

Before each test started, the specimen was placed inside the device cabinet for two hours to ensure its temperature is maintained during the test after which it is transferred to the loading frame cradle and clamped at four points as shown in Fig. 4. After the clamping and before the test was run, the specimen was left for a minimum of 30 minutes to relieve its clamping stress. Initially, 50 load cycles were applied to determine and calculate the initial flexure stiffness and cumulative initial dissipated energy.

3. Results and discussion

3.1. Fatigue life

Previous studies have shown that density or air voids affect the fatigue life of asphalt mixtures. Vazquez et al. [26] included air void content as one of the factors affecting the resistance to fatigue cracking of asphalt mixtures and Nejad et al. [27] also stated that density should be used in asphalt mixtures to increase the resistance to fatigue cracking. Mogawer et al. [28] focused on the density of asphalt mixtures as a factor related to fatigue cracking potential while Baburamani [8] and Hartman et al. [24] described the degree of compaction (air void content) achieved in the mix to have affected the flexural stiffness and fatigue life of asphalt mixtures. Walubita et al. [29] also suggested variables and test parameters such as air voids need to be considered to improve the robustness and repeatability of the test.

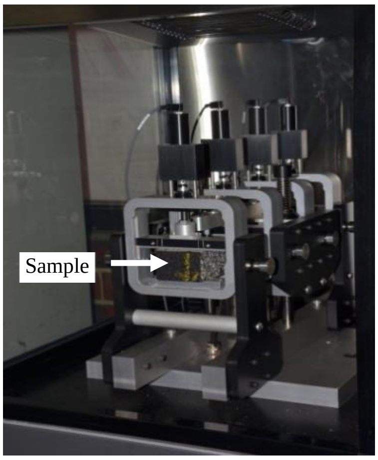

Fig. 4. Set-up of the repeated flexural bending test.

The beam fatigue samples in this study were provided with statistically the same density in order to limit its effect on flexural stiffness while the mean air voids were within the range of 5±0.5%. The mean, standard deviation (SD), and coefficient of variation (CoV) for air void and initial flexural stiffness for both unmodified and BRA-modified asphalt mixtures are presented in Table 5. The repeatability of initial flexural stiffness was considered to be excellent while the coefficients of variation were 10 percent maximum. Moreover, a paired sample t-test method with a 95% confidence level was used to statistically analyze both air void and initial flexural stiffness with the results summarized in Table 6.

Table 6 shows there was no significant difference in voids and the variations observed with the initial tensile strain had no substantial effect on the initial flexural stiffness values for both unmodified and BRA-modified asphalt mixtures (significance > 0.05). The tensile strain occurring at the bottom of the asphalt layer was considered to be a parameter controlling fatigue cracking.

However, comparing the flexural stiffness at the same strain level indicates the initial flexural stiffness for BRA-modified mixtures was statistically significantly higher than unmodified ones (significance < 0.05). This was evident in the BRA-modified asphalt mixtures values which were 71%, 61%, and 67% at 400, 600, and 800 με respectively which are higher than unmodified ones as presented in Table 5. It can, therefore, be concluded that the behavior of the BRA modified mixtures at a given initial

Table 5
Average air void content and flexural stiffness.

Asphalt mixtures	Initial tensile strain (με)	Air void (%)			Initial flexural stiffness (MPa)		
		Mean	*SD*	*CoV*	*Mean*	*SD*	*CoV*
Unmodified	400	5.1	0.06	1.1	5044	509	10
Unmodified	600	5.0	0.06	1.1	5155	174	3.4
Unmodified	800	5.1	0.10	2.0	5365	309	5.7
BRA modified	400	4.9	0.06	1.2	8639	197	2.3
BRA modified	600	5.0	0.00	0.0	8562	396	4.6
BRA modified	800	5.0	0.10	2.0	9002	135	1.5

Table 6
The summary of paired sample t-test of air voids and initial flexural stiffness.

Asphalt mixtures	Initial tensile strain ($\mu\varepsilon$)	Air void				Initial flexural stiffness			
		t	*df*	*Sig.*	*d*	*t*	*df*	*Sig.*	*d*
Unmodified	400 – 600	1.7	2	0.225	1.75	-0.3	2	0.779	-0.18
	400 – 800	0.5	2	0.667	0.49	-2.6	2	0.118	-1.53
	600 – 800	-0.8	2	0.529	-0.75	-0.8	2	0.484	0.49
BRA-modified	400 – 600	-2.0	2	0.184	-2.03	0.2	2	0.831	0.14
	400 – 800	-2.0	2	0.184	-2.03	-2.8	2	0.106	-1.63
	600 – 800	0.0	2	1.000	0.00	2.3	2	0.151	1.31
Unmodified – BRA-modified	400	3.4	2	0.074	3.46	-8.8	2	0.013	-5.09
	600	1.0	2	0.423	1.00	-16.3	2	0.004	-9.42
	800	1.7	2	0.225	1.73	-16.6	2	0.004	-9.58

tensile strain and measured at a loading frequency of 10 Hz was more elastic.

The flexural strain of BRA-modified asphalt mixtures was generated by the cohesive strength of the BRA modifier binder and the bond strength of its combination with the aggregate interface. Meanwhile, asphalt mixture with higher tensile strength provides better resistance to fatigue and this further implies modified mixtures appear to be capable of withstanding larger tensile strain before cracking. Therefore, the BRA modifier binder added in the asphalt mixtures has the ability to generate a higher tensile strength in the modified mixtures to improve the long-term performance of the asphalt pavement.

Furthermore, the classical approach and energy stiffness ratio (ER) method developed by Abojaradeh [30] were used to analyze the flexure fatigue test results as shown in Fig. 5. In the classical, the failure point (N_{f50}) was assumed to be the number of loading corresponding to the 50% reduction of initial flexural stiffness recorded at 50, 200, and 500 cycles [1,31]. The Australian standard Austroads AG: PT/T233 [23] also adopted this model as the failure criterion. Meanwhile, the ER method 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) was determined by plotting the peak value of the energy stiffness ratio to the number of cycles. During the test, the ER value improves as the number of cycles (N_i) increases until it reaches a peak value where it suddenly decreases. In the same period, the stiffness of materials (S_i) also suddenly decreases even though the number of cycles (N_i) increases.

The results for the mean fatigue life for three specimens tested at each set of initial tensile strain for each mixture are shown in Table 7 and a decrease in fatigue life for both asphalt mixtures was observed due to the increase in initial tensile strain. The same conclusion was reported by Wang and Zhang [32]. The results in Table 7 were used to develop graphs in Fig. 6 and the repeatability of the fatigue life was considered reasonable. Meanwhile, the coefficients of variation for the fatigue life were less than 20 percent, except for unmodified asphalt mixtures tested at an initial tensile strain of 600 $\mu\varepsilon$.

Comparing the fatigue life for the two mixtures at the same initial tensile strain indicates the fatigue life of BRA-modified was about 2.0 to 2.4 and 1.6 to 2.1 using classical and ER approaches, respectively and these were higher compared to unmodified asphalt mixtures as shown in Table 7. The stiffness and nature of asphalt mixtures have been reported to affect the magnitude of the strain [33] and the same conclusion was established by Some et al. [34]. The better fatigue life performance of BRA-modified asphalt mixtures is attributed to the unique combination of base asphalt and BRA modifier binders compared to the unmodified mixtures.

De Mello et al. [35] indicated materials properties affect fatigue life and binder is an important factor for higher fatigue life in BRA-modified asphalt mixtures. Razmi and Mirsayar [36] performed a fracture test on modified asphalt concrete and observed the asphalt binder was far more temperature-sensitive than aggregates. According to Karami and Hamid [37], BRA-modified asphalt mixtures are stiffer, thereby, contributing to greater strength within the mix to increase the fatigue life. Moreover, Vazquez et al. [26] and Tarefder et al. [38] included asphalt binder grade and content as the factors with the most effect on resistance to fatigue cracking. As Guler [39] and Khiavi and Ameri [1] argued, binder type is considered to be an important factor among the other variables such as aggregate gradation, aggregate type, binder content, compaction temperature and traffic loading with a significant effect on the fatigue life of the bituminous mixes.

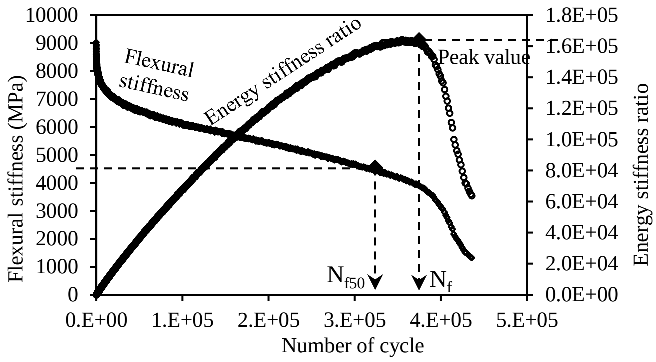

Fig. 5. Energy stiffness ratio versus loading cycles (for specimen tested at 20°C and 400με).

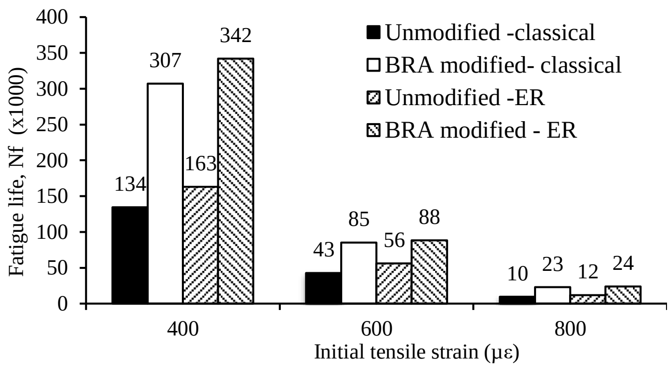

Fig. 6. Mean fatigue life of asphalt mixtures.

Table 7
Determination of fatigue life (N_f).

Initial tensile strain ($\mu\varepsilon$)		Classical 0%	Classical 20%	ER 0%	ER 20%	Fatigue life ratio Classical	Fatigue life ratio ER
400	Average (cycle)	134,487	306,933	163,113	341,953	2.3	2.1
	SD (cycle)	10,618	16,008	12,296	8,380		
	CoV (%)	7.9	5.2	7.5	2.5		
600	Average (cycle)	42,663	85,283	56,247	88,283	2.0	1.6
	SD (cycle)	9,472	11,713	7,494	11,815		
	CoV (%)	22.2	13.7	13.3	13.4		
800	Average (cycle)	9,617	22,933	11,990	24,050	2.4	2.0
	SD (cycle)	1,562	1,255	1,176	1,293		
	CoV (%)	16.2	5.4	9.8	5.4		

The flexural stiffness ratio was calculated by dividing the flexural stiffness for a given fatigue failure (S_f) by the initial flexural stiffness (S_0) and the mean values obtained were 0.42 – 0.44 for unmodified and 0.48 for BRA-modified asphalt mixtures. These results confirmed the fatigue life of both mixtures developed using the ER method is longer compared to those analyzed with the classical method. Similar observations have been reported by other researchers. Khiavi and Ameri [1] showed the fatigue life based on the RDEC and DER criteria produced 65% and 55% initial stiffness reduction while Maggiore [7] reported it was longer than 50%. Other researchers, Rowe [40] and Walubita [41], stated that this phenomenon usually occurs in a range of 40-50%.

Fig. 7 shows the sensitivity of the ER to the change in strain. The ER value at failure corresponded to the number of cycles at failure for all specimens on the log-log scale. The results showed a straight line with a much higher coefficient of determination R^2 (0.996). Moreover, the energy stiffness ratio at failure exhibited a good correlation with the load cycles to failure. The regression equation was then changed to become Eq. (1) and as the power of 1.028 was moving close to 1.0, the (S_f/S_0) value was 0.332. This is consistent with the well-established concept of the strain criterion. According to Abojaradeh [30], the (S_f/S_0) value was 0.3512 for controlled strain.

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

3.2. *Fatigue Life Prediction*

The magnitude of tensile strain at the bottom of the asphalt layer was used as a criterion to determine the occurrence of microcracking, crack initiation, and failure. Accordingly, the relationship between the number of cycles to failure and strain was used to assess fatigue performance and also enabled the determination of asphalt layer thickness in structural pavement design. Moreover, Shen and Carpenter [42] showed tensile strain as the most important parameter for fatigue cracking. It is also possible to explain the controlled displacement results using the relationship between the initial strain and load repetition as shown in Eq. (2) [8,43,44] which represents the relation between the radial strain at the bottom of the asphalt mixture layer and the number of load applications up to the occurrence of cracking in the pavement. The fatigue coefficient k_1 and k_2 may vary between models but k_2 is usually in a range between 3 and 6, while k_1 is affected by several magnitudes. They are both, however, specific to the asphalt binder type, asphalt mixture type, volumetric composition, and the test parameters used in the laboratory characterization [8].

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

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

Fig. 8 shows the regression analysis developed to determine the phenomenological and ER approach for both mixtures. The number of cycles to failure was found to have exhibited a good correlation with the initial tensile strain for both methods as shown in the summarized statistical analysis provided in Table 8. A large correlation coefficient ranging between 0.95 and 0.98 was obtained between the independent and dependent variables while the linear regression between laboratory fatigue life (N_f) and tensile strain indicated an increase in the fatigue life of BRA modified mixtures. Furthermore, a comparison between the regression equations of the two methods of analysis shown in Fig. 8 indicates very similar exponents and this means the slopes of the lines are similar. The slope of the fatigue line (k_2) reflects the binder contribution to the mixtures' fatigue. However, the intercepts for the ER approach (7.534×10^{14} for unmodified and 2.777×10^{15} for modified mixtures) were closer to each other than the classical approach. Therefore, when N_f was measured using the ER approach the two regression lines became closer to each other.

The intercept and slope, k_1 and k_2 respectively, are the most important variables obtained from the test. Under the strain-controlled mode of loading, they represent the properties of the materials used and are also typical values for asphalt mixtures. According to Ghuzlan and Carpenter [45], it is possible to use the variables in the fatigue-based mechanism design procedures with the typical range of k_2 being between 3 and 6. All the slopes of

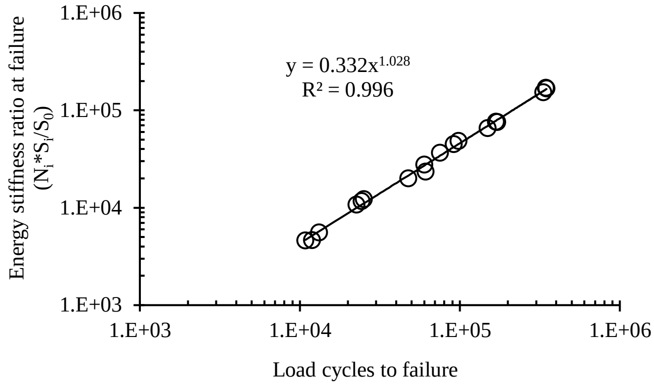

Fig. 7. Energy stiffness ratio at failure vs load cycles at failure for controlled strain.

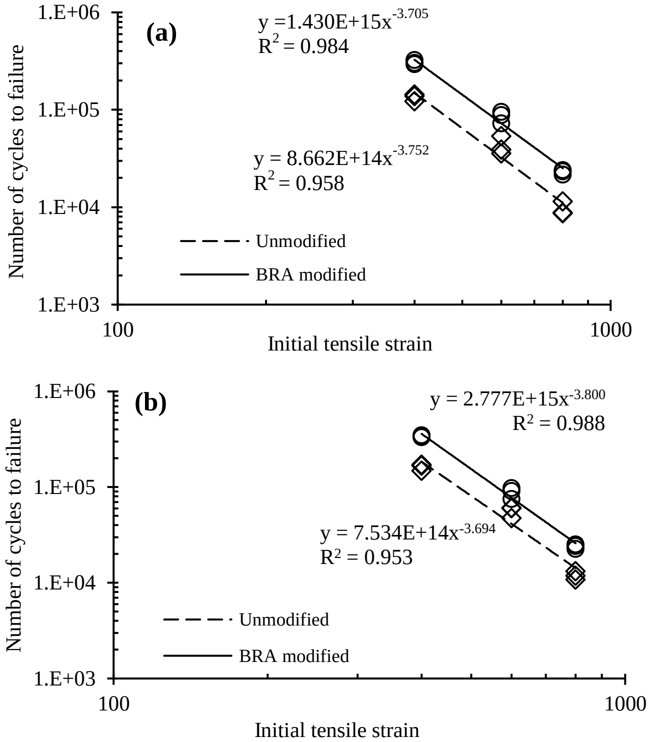

Fig. 8. Fatigue characteristics of unmodified and BRA modified asphalt mixtures using (a) classical approach and (b) ER approach.

Table 8
Equations developed for fatigue life-initial strain.

Asphalt mixtures	Classical approach			ER approach		
	k_1	k_2	R^2	k_1	k_2	R^2
Unmodified	8.662×10^{14}	3.752	0.958	7.534×10^{14}	3.694	0.953
BRA-modified	1.430×10^{15}	3.705	0.984	2.777×10^{15}	3.800	0.988

the fatigue curve (k_2) in this study were observed to be within this range even though in some models, k_2 was fixed to a specific number, as in the Asphalt Institute and Illinois fatigue equations, where it was defined as 3.29 and 3.0 respectively. Moreover, Ghuzlan and Carpenter [46] argued that k_1 and k_2 are fundamental values of asphalt mixtures.

Fig. 9 shows the good correlation between k_1 and k_2 as evident in R^2 values of 0.895 and 0.927 respectively. The values of the variables are presented in one line despite the different mixture properties and this relationship is consistent with the findings of other researchers [45].

Fig. 10 shows the effect of using the BRA modifier binder on k_1-k_2 relation. Both lines are not close and this shows there is a significant difference between them at a 95% significance level. Therefore, the use of the BRA modifier binder in asphalt mixtures was concluded to have a significant effect on the k_1-k_2 relation.

A common perception observed from fatigue life models with strain and mix stiffness as independent variables is that greater stiffness reduces fatigue life for controlled-strain testing. This is in line with the findings of Shen and Carpenter [42] that the stiffness of asphalt mixtures affects the fatigue life. Therefore, Eq. (2) was modified to produce Eq. (3) which defines the mixture's stiffness dependent behavior [8,42,47].

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

where, N_f is the number of loading application to failure, ε_t is the tensile strain, S_0 is mixture stiffness while a, b, and c are material coefficients, derived from fitting the data.

The test results of individual mixtures were used to determine the constants a, b, and c in the classical and ER approaches in accordance with Eq. (3). Statistical analysis was conducted 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 initial flexural stiffness (S_0, in MPa) as independent variables for both mixtures as presented in Table 9. The results showed a much higher coefficient of determination R^2 ranging from 0.937 to 0.996.

Fig. 11 shows the effect of initial flexural stiffness on the fatigue life of unmodified and BRA-modified asphalt mixtures according to the Equations in Table 9 which were plotted at different values of initial tensile strain between 200 με and 1200 με and initial flexural stiffness from1000 MPa to 20,000 MPa. The cycle to failure (N_f) was observed to have decreased as initial flexural stiffness (S_0) increases for both mixtures and this is in agreement

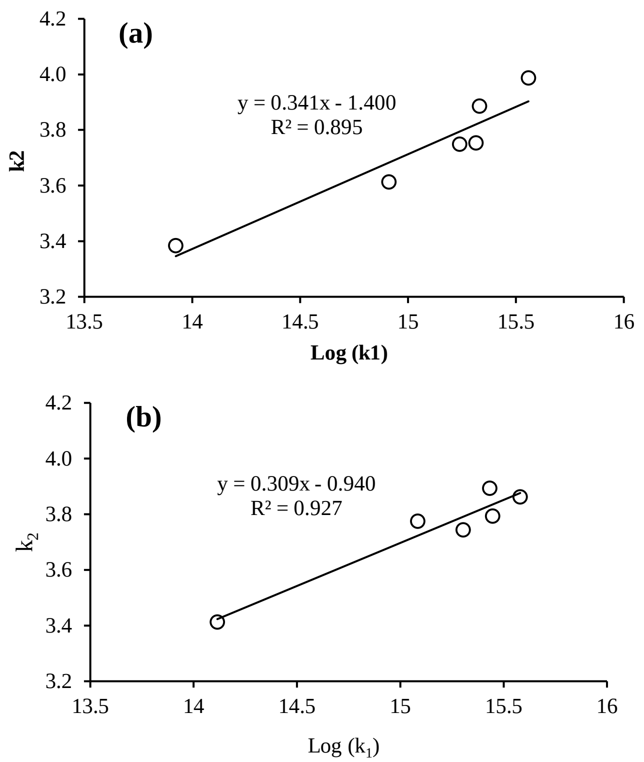

Fig. 9. The relationship between k_1-k_2 for all asphalt mixtures using (a) classical approach and (b) ER approach.

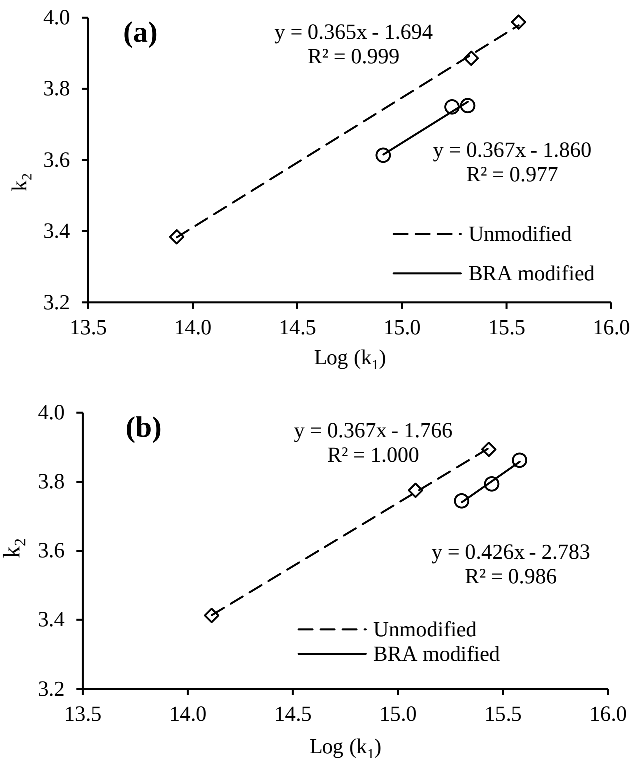

Fig. 10 The relationship between k_1-k_2 for unmodified and BRA-modified asphalt mixtures using (a) classical approach and (b) energy-stiffness ratio approach.

Table 9
Prediction models for the number of cycles (N_f).

Asphalt mixtures	Classical approach		ER approach	
	Equation	R^2	Equation	R^2
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

with the findings of de Mello et al. [35]. The comparison between the cycle to failure of the two mixtures indicates BRA-modified mixture has a higher value until the initial flexural stiffness (S_0) reached about 13,030 MPa and 11,500 MPa for the classical and ER approaches, respectively. Therefore, the BRA-modified asphalt mixtures with an initial flexural stiffness greater than 11,500 MPa are predicted to have a shorter fatigue life than unmodified mixtures. This also means the addition of BRA modifier binder in asphalt mixtures at more than 20% content has the potential to produce greater stiffness modulus but also has a negative impact on the fatigue life performance of the modified mixtures. Similar observations have been reported by other researchers, for example, Monismith et al. [47] showed fatigue life was influenced by the flexural stiffness of asphalt mixtures. Accordingly, the substitution of base asphalt binder with BRA modifier binder to produce an initial flexural stiffness of BRA-modified asphalt mixtures greater than 13,030 MPa using the classical approach or 11,500 MPa with the ER approach has a cycle to failure lower than unmodified ones.

3.3. *Dissipated Energy*

The damage-energy approach was used to analyze the dissipated energy concept which is mostly used as a great indicator of fatigue response during each loading cycle due to its ability to capture both elastic and viscous effects. The concept states that fatigue life is a function of accumulated dissipated energy on each loading cycle affected by those in the previous cycles.

According to Baburamani [8], as viscoelastic materials, it is possible to analyze asphalt mixtures in terms of the energy dissipated in the specimen during the testing process. It was also argued that the rheology of the mixtures as a function of temperature, loading frequency, and strain/stress level influences the energy dissipated. Van Dijk et al. cited in Baburamani [8] also reported it is possible to control the total energy dissipated during a fatigue test with the fatigue life and change in the mechanical properties of asphalt mixtures. Furthermore, the energy dissipated can be used to explain the decrease in mechanical properties such as flexural stiffness loss during the test. Hopman et al. [48] noted

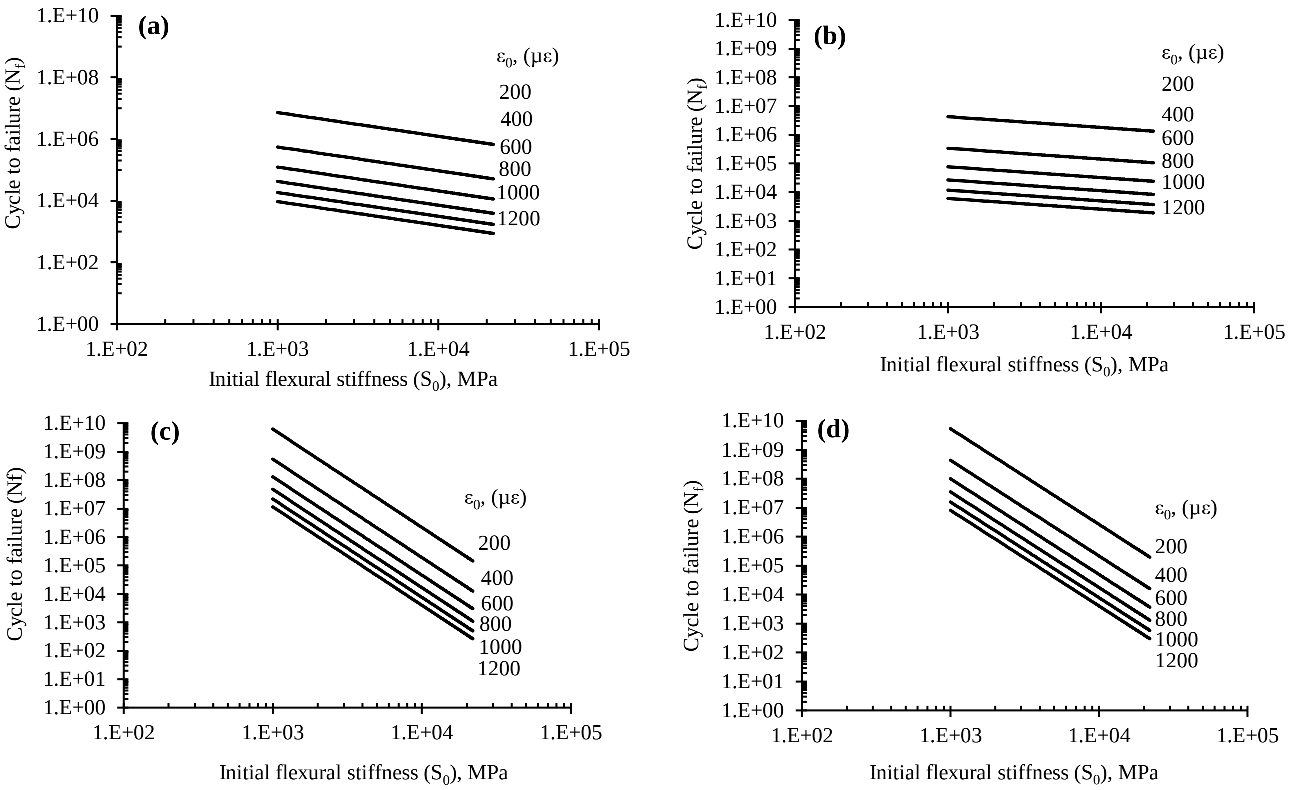

Fig. 11. Strain-stiffness relationship between N_f, ε_0, and S_0: (a) Unmodified (classical), (b) Unmodified (ER), (c) BRA-modified (classical), and (d) BRA-modified (ER).

the energy dissipated per cycle controls the fatigue damage of asphalt mixtures while Hassan and Khalid [49] reported it is the difference between the induced and released energy due to load application and relief. Therefore, energy dissipated in each pulse of a loading cycle causes incremental damage to the asphalt mixtures and this leads to crack extension, plastic deformation, and thermal energy [31].

In viscoelastic materials, deformation and strain increase over time as long as a constant load is applied, and after it has been removed, some deformation is recoverable while others are unrecoverable. 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 Eq. (4) [1,8,31,49,50], however, an unloaded material has a different path. This, therefore, usually leads to the recording of the phase lag between the applied stress and the measured strain. Furthermore, the energy is dissipated in the form of mechanical work, heat generation, or damage. In the strain-controlled fatigue test, the energy decreases with an increment in the number of load cycles and a reduction in the stress but increases for the stress-controlled fatigue test [30,51].

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

where, DE_i is the 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. [7,46] presented Eq. (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 the cumulative dissipated energy to failure, N_f is the number of load cycle to failure, and A, z are mixture dependent constants.

Pronk and Hopman [52] proposed an energy-ratio (ER) concept to define fatigue life of asphalt mixtures in the strain-controlled tests. The energy-ratio was defined as the ratio of the initial dissipated energy to the dissipated energy at the i_{th} cycle multiplied by the load cycle n. Furthermore, Pronk [53] also proposed the concept of energy ratio to define the failure as the ratio of cumulative dissipated energy at cycle n to the dissipated energy for cycle n. Rowe and Bouldin [54] introduced a new definition for failure as the load cycle multiplied by the stiffness at the cycle. Meanwhile, Abojaradeh [30] proposed a fatigue failure criterion based on the energy stiffness ratio while Ghuzlan and Carpenter [46,55] and Carpenter et al. [43] developed and suggested the dissipated energy ratio (DER) method in defining a failure point in asphalt mixtures fatigue life.

The examples of dissipated energy evolution with the number of cycles and typical results for flexural stiffness and cumulative dissipated energy in this research are presented in Fig. 12. The stiffness of asphalt mixtures is showing to have reduced during a fatigue test to produce microcracks in the materials due to the repeated application of stresses to the specimen below the failure stress. This, therefore, means the dissipated energy varies per each loading cycle and decreases for controlled strain tests and this is in line with the opinion of Carpenter and Shen [56] that the energy dissipated in loading cycles is affected by those applied in previous cycles. Baburamani [8] suggested the rate of dissipated energy change per cycle is a better indicator of the initiation and growth of damage or cracking. The dissipated energy in this study was obtained for each cycle in both unmodified and BRA-modified asphalt mixtures.

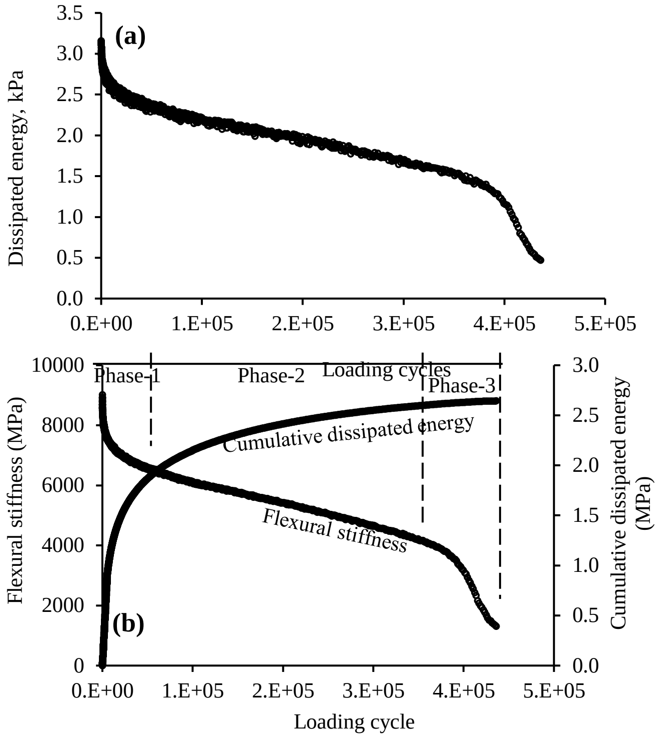

Fig. 12 Typical results of fatigue test (a) evolution of dissipated energy per cycle, (b) progression curve of flexural stiffness and cumulative dissipated energy.

Fig. 12(b) plotted the relationship between flexural stiffness and number of cycles with three phases observed for the flexural stiffness is similar to those presented by Hassan and Khalid [49], Di Benedetto et al. [57], and Maggiore et al. [58,59]. Phase 1 is characterized by a rapid reduction in flexural stiffness due to repetitive excitation, which is followed by Phase 2, where the reduction in the stiffness modulus is approximately linear.

According to Di Benedetto et al., in phase 1, the decrease in flexural stiffness is not only associated with fatigue damage but also heating as well as a third phenomenon which was considered to be playing an important role. However, stiffness loss is totally recoverable. In phase-2, the role of fatigue on flexural stiffness reduction is predominant while thermal heating has an insignificant effect but has to be considered. Phases 1 and 2, therefore, correspond to the crack initiation process in the asphalt mixtures. Finally, the flexural stiffness exhibits a marked drop in phase-3 after passing through an inflection point with load cycles culminating in failure. This further leads to a local crack propagation which later develops to global failure at the end of the phase [57].

Fig. 12 shows an increase in the number of loading cycles leads to a reduction in flexural stiffness and an increase in cumulative dissipated energy with the test conducted in constant strain mode. Meanwhile, a rapid reduction in flexural stiffness at phase-1 was observed to have caused a rapid increase in cumulative dissipated energy and this was observed to have increased linearly at phase-2. According to Di Benedetto et al. [57], the fatigue damage was characterized only by phase-2, and Eq. (6) [49] was used to obtain the damage rate, dD/dN, as a function of the slope of the line in the phase.

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

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

The results of damage parameters obtained according to the progression curve of flexural stiffness and cumulative dissipated energy are presented in Table 10 and 11, respectively and they show the BRA modifier binder influenced the damage parameters of asphalt mixtures. This confirms the findings of Lytton et al. [60] that the rate of change of dissipated energy per cycle is a better indicator of the initiation and growth of damage or cracking. The comparison of the parameters showed the slopes and damage rate values for unmodified asphalt mixtures in both flexural stiffness and the cumulative dissipated energy progression curve at the same initial tensile strain were higher than modified ones. This, therefore, indicates the BRA-modified asphalt mixtures have much better resistance to fatigue failure.

Fig. 13 shows the relationship between the number of cycles to failure and cumulative dissipated energy to the failure point for both mixtures using classical and ER approaches. The figure was developed for all the specimens tested at 400 με, 600 με, and 800 με due to the fact that some factors such as temperature, loading frequency, and mode of loading seemed not to affect the relationship. The slopes (k_2) of the curves for BRA-modified asphalt mixtures were found to be higher and this means the sensitivity of cumulative dissipated energy to the number of cycles to failure was higher than for the unmodified ones.

Fig. 14 also shows the cumulative initial dissipated energy values recorded at 50 cycles for both mixtures. The initial tensile strain was observed to have a significant effect such that at higher values, more cumulative initial dissipated energy is produced. Moreover, BRA-modified asphalt mixtures were found to have higher values with an increase of 1.5 to 1.8 times.

Table 10
The summary of the damage parameter of unmodified and BRA-modified asphalt mixtures based on the flexural stiffness progression curve.

Initial tensile strain (με)	Unmodified			BRA-modified		
	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 11
The summary of the damage parameter of unmodified and BRA-modified asphalt mixtures based on the cumulative dissipated energy

Initial tensile strain (με)	Unmodified			BRA-modified		
	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

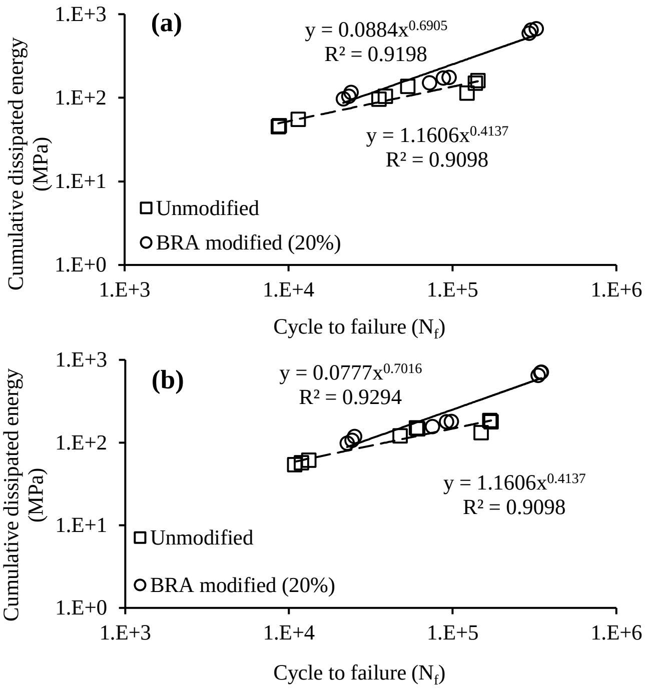

Fig. 13. Relationship between cycle to failure and cumulative dissipated energy using (a) classical approach and (b) ER approach.

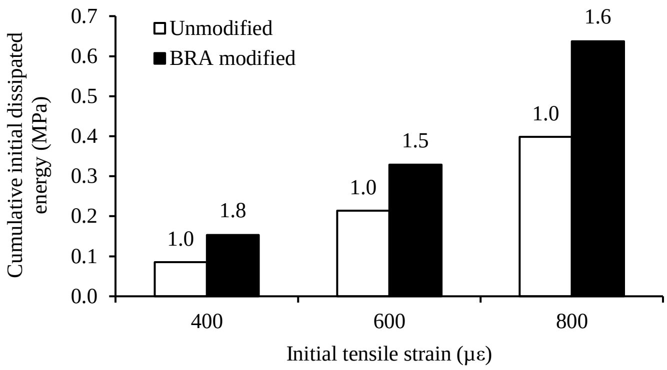

Fig. 14. Cumulative initial dissipated energy for asphalt mixtures.

4. Conclusion

Repeated flexural bending tests were conducted to determine the effect of the BRA modifier binder on the fatigue strength of BRA-modified asphalt mixtures. The results showed the use of a 20% BRA modifier binder led to an increment in the number of cycles for BRA-modified asphalt mixtures by 2.0-2.4 and 1.6-2.1 using classical and energy stiffness ratio approaches, respectively. According to the strain approach, the number of cycles to failure for the modified mixtures was also higher compared to the unmodified ones. Furthermore, the strain-stiffness approach showed the initial flexural stiffness (S_0) affected the fatigue life of both mixtures. Meanwhile, the regression equation model was developed to predict the fatigue life of the mixtures based on the strain and strain-mix stiffness approaches and the use of BRA modifier binder in asphalt mixtures was discovered to have a significant effect on the relationship between intercept (k_1) and slope (k_2) variables. The damage to asphalt mixtures caused by the fatigue response was observed using an energy dissipated and flexural stiffness progression curve and the rate for BRA-modified asphalt mixtures was found to be lower. The results also showed BRA-modified asphalt mixtures have better resistance to fatigue failure. However, a comprehensive evaluation of the mechanical properties of the modified asphalt mixtures with different testing parameters was necessary to illustrate the effect of loading and environment such as the variations in temperature and loading frequency on the repeated flexural bending test.

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