**Fatigue performance of Buton rock asphalt modified mixtures**

**Abstract**

The main objective of this research is to assess the effect of granular Buton rock asphalt (BRA) modifier binder on the fatigue life of asphalt mixtures. 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.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: 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%. It is shown that the number of cycles for BRA modified asphalt mixtures increased by 2.0-2.4, and 1.6-2.1 observed by the using 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 asphalt mixtures. Based on the strain-stiffness approach, the initial flexural stiffness (*S0*) affected the fatigue life of unmodified and BRA modified asphalt mixtures. Furthermore, the regression equations model used to predict the fatigue life of unmodified and BRA modified asphalt mixtures based on the strain approach and strain-mix stiffness approach were developed. The use of BRA modifier binder in asphalt mixtures had a significant effect on the relationship between intercept (*k1*) and slope (*k2*) variables. The damage rate for BRA modified asphalt mixtures was lower compared with the unmodified asphalt mixtures.

*Keywords:* asphalt mixtures, granular Buton rock asphalt,fatigue performance

1. **Introduction**

According to studies [1-3], fatigue cracking is a result of fatigue failure due to repetitive stress and strains caused by load and environmental factors, and is considered to be a major form of distress in asphalt mixtures. Fatigue cracking occurs when the tensile stress of materials exceeds the tensile strength due to repetitive stress and strain. Cracks form, resulting in the decreased structural capacity of the pavement and increasing the maintenance costs. 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 repetitive loading without any significant failure such as cracking or premature failure, developed under other circumstances such as environmental conditions.

The cracking begins with a microcrack at the points where critical tensile strain/stress occurs, and then grows to form a macrocrack, usually taking an alligator cracking pattern, and finally penetrating the surface of pavement [4]. According to Di Benedetto *et al*. [5], the process of degradation during fatigue cracking is divided into two phases. Firstly, there is the initiation and propagation of a microcrack network. The decrease in the modulus occurs in this phase. This phase relates to degradation resulting from damage that is uniformly spread in the asphalt mixtures. The first phase is called the initiation phase. Secondly, a microcrack appears as a result of the combination of microcracks. Further macrocracks will propagate within the materials. The second phase is called the propagation phase.

Many researchers have developed various concepts for evaluating the fatigue resistance of mixes. They can be grouped into three types: linier viscoelastic properties *E0* and φ*0* (determined 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. These 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 [1, 3, 6, 7]. Baburamani [8] reported that the fracture mechanics and dissipated energy approaches were included as mechanistic approaches, while the classical criterion was categorized as a phenomenological approach. In the mechanistic approaches (fracture mechanics and dissipated energy), the 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. The use and development of modified binders in bituminous mixes, however, is important with the aim of increasing the fatigue performance. Many studies have been carried out on asphalt mixtures to improve resistance to fatigue failure. Polymer modified binder [9-11] and crumb rubber [12] have a significant effect on improving fatigue life of asphalt mixtures. The main objective of this research however, was to study the potential use of granular BRA modifier binder to increase the resistance of asphalt mixtures to fatigue cracking. The raw materials for natural rock asphalt are found on Buton Island in Indonesia, and this is traditionally known as asphalt Buton (*asbuton*). Limited studies are found on fatigue performance of asphalt mixtures using asbuton.A study on the feasibility of using *asbuton* filler for fatigue performance found that the number of cycles to failure (Nf) for Hot Rolled Asphalt (HRA) mixtures containing *asbuton* filler were higher compared with unmodified HRA mixtures at all level of stress[13].

**2. Materials and methods**

**2.1 Materials**

Class-170 (Pen 60/80) base asphalt binder was used for the unmodified asphalt mixtures. The binder was classified in accordance with the Australian Standard AS-2008 [14]. The BRA modified binder used for BRA modified asphalt mixtures was made by replacing 20% base asphalt binder (by total weight of asphalt binder in the mixtures) with 20% BRA natural binder. Specification of the base asphalt binder and BRA modified binder is given in Table 1.

Figure 1 shows the form of the BRA modifier binder (pellets) with a diameter of 7 mm to 10 mm used in this study. Triplicate portions of granular BRA modifier binder were subjected to an extraction process [15]. The bitumen content test results found that, on average, the granular BRA modifier binder consisted of about 70% mineral and 30% binder by total weight of materials. The mean particle size distributions of BRA mineral for each sieve 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) | 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 |



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

A crushed granite aggregate from a local quarry in Western Australia was used in all of the mixtures. The unmodified and BRA modified asphalt mixtures used a typical dense graded aggregate with a maximum aggregate size of 9.5 mm (DG 10) based on Specification 504 [16]. The coarse and fine aggregates required to prepare the mixes were obtained by sieving on each sieve diameter. The results of determination of aggregates properties are summarized in Table 2.

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/m3) | AS 1141.5-2000 | 2.63 |
| Apparent fine aggregate density (t/m3) | AS 1141.5-2000 | 2.60 |

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 3 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 the mineral contained in the granular BRA modifier with the aim of minimizing the variance in the gradation of aggregates.

Table 3 Final crushed aggregate gradation used in this study (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 |

**2.2 Mix design and specimen preparation**

This study used the Marshall mix design as a method for determining the optimum bitumen content (OBC) of unmodified asphalt mixtures based on specification 504 [16]. Specimens in triplicate with dimension of 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 various contents of asphalt binder. The result shows that a void of 5% was chosen to give binder content of 5.4% as the optimum bitumen content (OBC) (by mass of asphalt mixture). The BRA modified asphalt mixtures were designed to exhibit the benefits arising from mixing granular BRA modifier binder into the asphalt mixtures. Therefore, the same binder content as for unmodified asphalt mixtures was used for the BRA modified asphalt mixtures in order to maintain consistency for comparison purposes. Hence, the OBC of 5.4% by weight of the total mixtures was also used for the BRA modified asphalt mixtures.

Table 4 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, after each size of crushed aggregates had been weighed, they were then blended manually in a pan and then heated in an oven at a temperature of 105ºC for 24 hours. After that, the blend aggregates were heated in the same oven at 150ºC prior to mixing with asphalt binder for approximately two hours. In the other side, the base asphalt binder and granular BRA modifier binder (pellets) were placed in the same bowl. The mixture was then put in an oven at a temperature of 150±5 ºC for 30 to 60 minutes with frequent manual stirring intended to blend and incorporate the two binders as BRA modified binder. Then the previously blended aggregates were put in the same bowl as 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 [17], and then were immediately compacted in a slab mould by using a roller compactor as seen in Figure 2 in accordance with Austroads standard AG:PT/T220 [18]. The specimens consisted of slabs of 400 mm in length, 305 mm in width and 75 mm in height prepared in accordance with Austroads standard AG:PT/T233 [19]. After 24 hours prior to the compaction process, the slabs were then cut to produce the final beam specimens with dimensions of 390±5 mm in height, 50±5 mm in depth and 63.5±5 mm in width as illustrated in Figure 3a. 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 4. 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 |
| 1. Base binder | 5.40 | 4.30 |
| 1. BRA modified binder | 0.00 | 1.10 |
| 2. Total aggregate content | 94.60 | 94.60 |
| 1. Crushed rock | 94.60 | 92.10 |
| 1. BRA mineral | 0.00 | 2.50 |
| 3. Granular BRA (pellets) \*) | 0.00 | 3.60 |

Note:

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

* 1. **Fatigue performance characterization**

Laboratory evaluation methods for fatigue strength on four point bending equipment under repeated flexural bending test was done in accordance with Austroads AG:PT/T233 [19] test method.This test method was based 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 under the test 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 test 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 placed 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 for 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.



(a)

(b)

Figure 2 Compaction process for the slabs for the flexural bending test



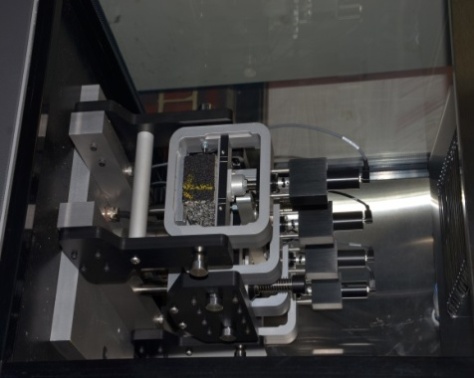
**390 mm**

**63.5 mm**

**50 mm**

**(b)**

**(a)**



Sample

Figure 3. (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 [20] were used to analyse the results of the flexure fatigue tests as shown in Figure 4. In the classical, the failure point (Nf50) 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, 21]. The Australian standard Austroads AG:PT/T233 [19] has also adopted this model as the failure criterion. The ER method defines the energy stiffness ratio as the ratio of the flexural stiffness at cycle-*i* (*Si*) to the initial flexural stiffness (*S0*) multiplied by the load cycle value at cycle-*i* (*Ni*). The fatigue failure (*Nf*) 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 (*Ni*) 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 (*Si*) is suddenly decreases even though the number of cycles (*Ni*) increases.

Figure 4. Energy stiffness ratio versus loading cycles (for specimen tested at 20°C and 400µε)

Figure 5. Mean fatigue life of asphalt mixtures

The results for the mean fatigue life of the asphalt mixtures using both classical and ER approaches are shown in Figure 5. 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 approaches, respectively, higher fatigue lives when compared to unmodified asphalt mixtures. The stiffness and nature of asphalt mixtures affect the magnitude of the strain [22]. 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 [23], BRA modified asphalt mixtures are stiffer which may contribute to greater strength within the mix, thereby increasing the fatigue life. Vazquez *et al*. [24] and Tarefder *et al*. [25] included asphalt binder grade and asphalt binder content as the factors most affecting resistance to fatigue cracking in asphalt mixtures. As Guler [26] 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 (Sf) by the initial flexural stiffness (S0). 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. The results confirmed that the fatigue life of unmodified and BRA modified asphalt mixtures developed by using the ER method 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 [7] found that the fatigue life was longer than the 50% initial stiffness reduction. Other researchers, Rowe [27] and Walubita [28] stated that this phenomenon usually occurs in a range of 40-50%.

Figure 6 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 *R2* (0.996). The regression equation is then changed to become Equation 1. As the power of 1.028 is close to 1.0, the (Sf/S0) value is 0.332. According to Abojaradeh [20]the (Sf/S0) value is 0.3512 for controlled strain.

Figure 6. 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 [29] 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 [8, 30, 31], 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 k1 and k2 may vary between models. Usually, the value of k2 is in a range between 3 and 6, while k1 is affected by several magnitudes. However, k1 and k2 are specific to the asphalt binder type, asphalt mixture type, volumetric composition, and the test parameters used in the laboratory characterization [8]. Furthermore, Shen and Carpenter [29] 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 [8, 29, 32].

where *Nf* is number of loading applications to failure at a particular level of initial strain, *ε0* is initial tensile strain, and *k1* and *k2* are material coefficients derived from fitting the data.

where *Nf* is number of loading application to failure, *εt* is tensile strain, *S0* is mixture stiffness, and *a, b,* and *c* are material coefficients, derived from fitting the data. Figure 7 shows the regression analysis developed to determine the phenomenological and ER approach for unmodified and BRA modified asphalt mixtures.

Figure 7. Fatigue characteristics of unmodified and BRA modified asphalt mixtures, (a) clasical approach, (b) ER approach

The intercept and the slope, *k1* and *k2* 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 [33], *k1* and *k2* can be used in the fatigue based mechanism design procedures and the typical range of *k2* values are between 3 and 6. In this study, it was noted that all of the slopes of the fatigue curve (*k2*) were within this range even though in some fatigue models *k2* was fixed to specific number, as in the Asphalt Institute and Illinois fatigue equation, where the *k2* value is fixed to 3.29 and 3.0 respectively. Ghuzlan and Carpenter [34] argue that *k1* and *k2* is fundamental values of the asphalt mixtures. Figure 8 shows the correlation between the *k1* and *k2* values which gives a good correlation (R2 = 0.895 and 0.927). As shown, the *k1* and *k2* 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 [33].

Figure 8. The relation of *k1-k2* for all of asphalt mixtures using: (a) classical approach, (b) ER approach

In addition, Figure 9 shows the effect of using BRA modifier binder on *k1*-*k2* 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 *k1-k2* relation.

Figure 9. The relation of *k1-k2* for unmodified and BRA modified asphalt mixtures using: (a) classical approach,

(b) 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 (*Nf*, in cycles) as a dependent variable and initial strain (*ε0*, in µε) and the initial flexural stiffness (*S0*, 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 (Nf)

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| Asphalt mixtures | Classical approach | | ER approach | |
| Equation | R2 | Equation | R2 |
| Unmodified |  | 0.937 |  | 0.938 |
| BRA  modified |  | 0.992 |  | 0.996 |

Figure 10 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 (Nf) decreases as initial flexural stiffness (S0) 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 (S0) reaches about 13,030 MPa and 11,500 MPa for the classical and ER approaches, respectively. However, the initial flexural stiffness affects the fatigue life of asphalt mixtures. Hence, 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 asphalt mixtures. In other words, the addition of BRA modifier binder in asphalt mixtures at an amount greater than 20% might potentially result in a greater stiffness modulus but will have a negative impact on the fatigue life performance of BRA modified asphalt mixtures.

Figure 10. Strain-stiffness relationship between *Nf* , *ε0*, and *S0*: (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.* [32] 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 a great indicator of fatigue response during each loading cycles, because it captures both elastic and viscous effects. The dissipated energy (DE) concept states that 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 [8] stated that as viscoelastic materials, asphalt mixtures can be analysed in terms of the energy dissipated in the specimen during testing. He argues that the rheology of asphalt mixtures as a function of temperature, loading frequency and strain/stress level influences the energy dissipated. As Van Dijk *et al.* cited in Baburamani [8] state, the total energy dissipated during a fatigue test may be controlled with the fatigue life and change in the 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 the test. Hopman *et al.* [35] note that the energy dissipated per cycle controls the fatigue damage of asphalt mixtures. According to Hassan and Khalid [36] the dissipated energy is the difference between the induced energy and released energy due to load application and relief. Thus, the energy dissipated in each pulse of a loading cycle causes incremental damage to the asphalt mixtures, then leads to crack extension, plastic deformation and thermal energy [21].

In viscoelastic materials, deformation and strain increase over time as long as a constant load is applied, and when the load is removed, some deformation is recoverable and some is unrecoverable. For 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, 8, 21, 36, 37]. 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 the strain-controlled fatigue test, the energy decreases when the number of load cycles increases as the stress decreases, while for the stress-controlled fatigue test the dissipated energy per cycles increases when the number of cycles increases [20, 38].

where *DEi* is dissipated energy in cycle *i*; *σi*is stress level in cycle *i*; *ε0* is strain level in cycle *i;* and *δi* is phase angle in cycle *i*. Maggiore *et al.* [7, 34] presented Equation 5 to relate the cumulative dissipated energy and the number of cycles to failure as follows:

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

Further, Pronk and Hopman [39] proposed an energy-ratio (ER) concept to define fatigue life of asphalt mixtures in the strain-controlled tests. The energy-ratio is defined as the ratio of the initial dissipated energy to the dissipated energy at the *ith* cycle multiplied by the load cycle *n*. Pronk [40] proposed the 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 [41] developed the definition of failure by introducing a new definition as the load cycle multiplied by the stiffness at that cycle. Abojaradeh [20] introduced a fatigue failure criterion based on the energy stiffness ratio. Ghuzlan and Carpenter [34, 42] and Carpenter *et al.* [30] 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 typical results for flexural stiffness and cumulative dissipated energy in this research are presented in Figure 11. These figures show that during a fatigue test, the stiffness of asphalt mixtures reduces, resulting in microcracks in the materials when repeated stresses are applied to the specimen below the failure stress; therefore the dissipated energy varies per each loading cycle and decreases for controlled strain tests. As Carpenter and Shen [43] have said, the energy dissipated in a loading cycles is affected by the energy applied in the previous cycles. Baburamani [8] suggested that the rate of dissipated energy change per cycle is a better indicator of the 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 11. Typical results of fatigue test: (a) evolution of dissipated energy per cycle, (b) progression curve of flexural stiffness and cumulative dissipated energy

Figure 11(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 [36], Di Benedetto *et al.* [44], and Maggiore *et al.* [45, 46]. 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 [44].

As seen in Figure 11, with the test carried out in constant strain mode, the increase in number of loading cycles resulted in a decrease in flexural stiffness, and increase in cumulative dissipated energy. With a 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*. [44], the fatigue damage could be characterised only by phase-2, and Equation 6 [36] was used to obtained the damage rate *dD/dN* as a function of the slope of the line in phase-2.

where *E00* 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*. [47] suggested that the rate of change of dissipated energy per cycle is a better indicator of the 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 the 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 strain (µε) | Unmodified | | | BRA modified | | |
| *dE\*/dN* | *E00* | *dD/dN* | *dE\*/dN* | *E00* | *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 strain (µε) | Unmodified | | | BRA modified | | |
| *dE\*/dN* | *E00* | *dD/dN* | *dE\*/dN* | *E00* | *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 12 shows a relationship between the number of cycles to failure and cumulative dissipated energy to the failure point for unmodified and BRA modified asphalt mixtures using classical and ER approach. 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 (*k2*) of the curves for BRA modified asphalt mixtures was higher than for unmodified asphalt mixtures. The results reveal that the sensitivity of cumulative dissipated energy to number of cycles to failure was higher than for the unmodified asphalt mixtures.

Figure 12. Relationship between cycle to failure and cumulative dissipated energy: (a) classical approach;

(b) ER approach

Furthermore, Figure 13 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 13. Cumulative initial dissipated energy for asphalt mixtures

**4. Conclusion**

Repeated flexural bending tests were performed on unmodified and BRA modified asphalt mixtures to find out the effect of BRA modifier binder on the fatigue strength of BRA modified asphalt mixtures. The results indicate that when the asphalt mixtures were prepared with 20% BRA modifier binder, the number of cycles for BRA modified asphalt mixtures increased by 2.0-2.4, and 1.6-2.1 observed by the using 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 asphalt mixtures. Furthermore, based on the strain-stiffness approach, the initial flexural stiffness (S0) affected the fatigue life of unmodified and BRA modified asphalt mixtures. In addition, the regression equations model used to predict the fatigue life of unmodified and BRA modified asphalt mixtures based on the strain approach and strain-mix stiffness approach were developed. The use of BRA modifier binder in asphalt mixtures had a significant effect on the relationship between intercept (*k1*) and slope (*k2*) variables. The damage to asphalt mixtures due to their fatigue response was observed by using an energy dissipated and flexural stiffness progression curve. The damage rate for BRA modified asphalt mixtures was lower than for the unmodified asphalt mixtures. The results showed that BRA modified asphalt mixtures have better resistance to fatigue failure than unmodified asphalt mixtures. However, a comprehensive evaluation of the mechanical properties of BRA modified asphalt mixtures with different testing parameters is necessary to illustrate the effect of loading and environment. For example, the effect of variations in testing temperature and loading frequency on the repeated flexural bending test.

**References**

[1] A. K. Khiavi and M. Ameri, "Laboratory evaluation of strain controlled fatigue criteria in hot mix asphalt," *Construction and Building Materials,* vol. 47, pp. 1497-1502, 2013.

[2] H. Hakimelahi, S. Saadeh, and J. Harvey, "Investigation of fracture properties of California asphalt mixtures using semicircular bending and beam fatigue tests," *Road Materials and Pavement Design,* vol. 14, pp. 252-265, 2013.

[3] Q. Li, H. Lee, and T. Kim, "A simple fatigue performance model of asphalt mixtures based on fracture energy," *Construction and Building Materials,* vol. 27, pp. 605-611, 2012.

[4] F. Zhou, X. Hu, S. Hu, L.F. Walubita, and T.Scullion, "Incorporation of Crack Propagation in the M-E Fatigue Cracking Prediction," *Road Materials and Pavement Design,* vol. 9, pp. 433-465, 2008.

[5] H. Di Benedetto, C. De La Roche, H. Baaj, A. Pronk, and R. Lundström, "Fatigue of bituminous mixtures," *Materials and structures,* vol. 37, pp. 202-216, 2004.

[6] M. M. Yuan, Z.X. Ning, C.W. Qiang, and Z.S. Xian, "Ratio of Dissipated Energy Change-based Failure Criteria of Asphalt Mixtures," *Research Journal of Applied Sciences, Engineering and Technology,* vol. 6, pp. 514-2519, 2013.

[7] C. Maggiore, J. Grenfell, G. Airey, and A.C. Collop, "Evaluation of fatigue life using dissipated energy methods," in *7th RILEM International Conference on Cracking in Pavements*, 2012, pp. 643-652.

[8] P. Baburamani, "Asphalt fatigue life prediction model - a literature review," Victoria - Australia1999.

[9] S. S. Awanti, M. S. Amarnath, and A. Veeraragavan, "Influence of rest periods on fatigue characteristics of SBS polymer modified bituminous concrete mixtures," *International Journal of Pavement Engineering,* vol. 8, pp. 177-186, 2007.

[10] C. Brovelli, M. Crispino, J. Pais, and P. Pereira, "Using polymers to improve the rutting resistance of asphalt concrete," *Construction and Building Materials,* vol. 77, pp. 117-123, 2015.

[11] T. W. Kim, J. Baek, H.J. Lee, and J.Y. Choi, "Fatigue performance evaluation of SBS modified mastic asphalt mixtures," *Construction and Building Materials,* vol. 48, pp. 908-916, 2013.

[12] A. Sibal, A. Das, and B. B. Pandey, "Flexural Fatigue Characteristics of Asphalt Concrete with Crumb Rubber," *International Journal of Pavement Engineering,* vol. 1, pp. 119-132, 2000.

[13] B. S. Subagio, J. Adwang, R.H. Karsaman, and I. Fahmi, "Fatigue performance of HRA (hot rolled asphalt) and superpave mixes using Indonesian rock asphalt (asbuton) as fine aggregates and filler," *Journal of the Eastern Asia Society for Transportation Studies* vol. 6, pp. 1207-1216, 2005.

[14] Australian Standard, "Residual bitumen for pavements," in *AS 2008-1997*, ed. New South Wales, Australia, 1997.

[15] *Bitumen content and particle size distribution of asphalt and stabilised soil: centrifuge method,* Main Roads Western Australia, 2011.

[16] Main Road Western Australia, "Asphalt Wearing Course," in *Specification 504*, ed. Perth, 2010, pp. 1-45.

[17] Australian Standard, "Methods of sampling and testing asphalt - Method 1.1: Sampling loose asphalt," in *AS 2891.1.1-2008*, ed. New South Wales, Australia, 2008, pp. 1-12.

[18] Austroads, "Sample preparation - compaction of asphalt slabs suitable for characterisation," in *AG:PT/T220*, ed. Sydney, New South Wales 2005, pp. 1-11.

[19] Austroads, "Fatigue life of compacted bituminous mixes subject to repeated flexural bending," in *AG:PT/T233*, ed. Sydney, New South Wales, 2006, pp. 1-17.

[20] M. Abojaradeh, "Development of fatigue failure criterion for hot-mix asphalt based on dissipated energy and stiffness ratio," *Jordan Journal of Civil Engineering,* vol. 7, pp. 54-69, 2013.

[21] G. Dondi, M. Pettinari, C. Sangiorgi, and S.E. Zoorob, "Traditional and Dissipated Energy approaches to compare the 2PB and 4PB flexural methodologies on a Warm Mix Asphalt," *Construction and Building Materials,* vol. 47, pp. 833-839, 2013.

[22] *The Shell Bitumen Handbook*. Riversdale House, Guildford street, Cherstsey, Surrey - UK: Shell Bitumen UK, 1990.

[23] M. Karami and N. Hamid, "The effect of granular BRA modifier binder on the stiffness modulus of modified asphalt," in *Advances in Civil Engineering and Building Materials IV*, ed: CRC Press, 2015, pp. 345-349.

[24] C. G. Vazquez, J. P. Aguiar-Moya, A. F. Smit, and J. A. Prozzi, "Laboratory Evaluation of Influence of Operational Tolerance (Acceptance Criterion) on Performance of Hot-Mix Asphalt Concrete," Center for Transportation Research The University of Texas, USA, Austin, USA2010.

[25] R. Tarefder, E. Kias, and A. Zaman, "Cracking in asphalt concrete under wet and dry conditions. Pavements and Materials 2008: Modeling, Testing, and Performance," in *Proc. of the Symp. on Pavement Mechanics and Materials at the Inaugural Int. Conf. of the Eng. Mechanics Institute*, 2008, pp. 37-47.

[26] M. Guler, "Effect of mix design variables on mechanical properties of hot mix asphalt," *Journal of Transportation Engineering* vol. 134, pp. 128-136, 2008.

[27] G. M. Rowe, "Performance of asphalt mixtures in the trapezoidal fatigue test," *Asphalt Paving Technology,* vol. 62, pp. 344-344, 1993.

[28] L. F. Walubita, "Comparison of fatigue analysis approaches for predicting fatigue lives of hot-mix asphalt concrete (HMAC) mixtures," Texas A&M University, 2006.

[29] S. Shen and S. H. Carpenter, "Dissipated energy concepts for HMA performance: fatigue and healing," Departmen of Civil and Environmental Engineering - University of Illinois, Urbana, Illinois - USA2007.

[30] S. Carpenter, K. Ghuzlan, and S. Shen, "Fatigue endurance limit for highway and airport pavements," *Transportation Research Record: Journal of the Transportation Research Board,* pp. 131-138, 2003.

[31] S. Shen and S. Carpenter, "Application of the dissipated energy concept in fatigue endurance limit testing," *Transportation Research Record: Journal of the Transportation Research Board,* pp. 165-173, 2005.

[32] C. Monismith, J. Epps, and F. Finn, "Improved asphlt mix design," *Journal of Association of Asphalt Paving Technology,* vol. 55, pp. 347-406, 1985.

[33] Ghuzlan and Carpenter, "Traditional fatigue analysis of asphalt concrete mixtures," *Urbana,* vol. 51, p. 61801, 2002.

[34] K. Ghuzlan and S. Carpenter, "Fatigue damage analysis in asphalt concrete mixtures using the dissipated energy approach," *Canadian Journal of Civil Engineering,* vol. 33, pp. 890-901, 2006.

[35] P. C. Hopman, A.C. Pronk, P.A. Kunst, A.A. Molenaar, and J.M. Molenaar, "Application of the visco-elastic properties of asphalt concrete," in *International Conference on Asphalt Pavements, 7th, 1992, Nottingham, United Kingdom*, 1992.

[36] M. Hassan and H. Khalid, "Fracture characteristics of asphalt mixtures containing incinerator bottom ash aggregate," *Transportation Research Record: Journal of the Transportation Research Board,* vol. 2180, pp. 1-8, 2010.

[37] Q. Li, H. J. Lee, and T. W. Kim, "A simple fatigue performance model of asphalt mixtures based on fracture energy," *Construction and Building Materials,* vol. 27, pp. 605-611, 2012.

[38] G. Al-Khateeb, and A. Shenoy, "A distinctive fatigue failure criterion," *Association of Asphalt Paving Technology,* vol. 73, pp. 585-622, 2004.

[39] A. Pronk and P. Hopman, "Energy dissipation: the leading factor of fatigue," 0727716352, 1991.

[40] A. Pronk, "Comparison of 2 and 4 point fatigue tests and healing in 4 point dynamic bending test based on the dissipated energy concept," in *Eighth International Conference on Asphalt Pavements*, 1997.

[41] G. M. Rowe and M. G. Bouldin, "Improved techniques to evaluate the fatigue resistance of asphaltic mixtures," in *2nd Eurasphalt & Eurobitume Congress Barcelona*, 2000.

[42] K. Ghuzlan and S. Carpenter, "Energy-derived, damage-based failure criterion for fatigue testing," *Transportation Research Record: Journal of the Transportation Research Board,* pp. 141-149, 2000.

[43] S. Carpenter and S. Shen, "Dissipated energy approach to study hot-mix asphalt healing in fatigue," *Transportation Research Record: Journal of the Transportation Research Board,* pp. 178-185, 2006.

[44] H. Di Benedetto, C. De La Roche, H. Baaj, A. Pronk, and R. Lundström, "Fatigue of bituminous mixtures," *Materials and Structures,* vol. 37, pp. 202-216, 2004.

[45] C. Maggiore, J. Grenfell, G. Airey, and A. C. Collop, "Evaluation of fatigue life using dissipated energy methods," in *7th RILEM International Conference on Cracking in Pavements*, 2012, pp. 643-652.

[46] C. Maggiore, G. Airey, and P. Marsac, "A dissipated energy comparison to evaluate fatigue resistance using 2-point bending," *Journal of Traffic and Transportation Engineering (English Edition),* vol. 1, pp. 49-54, 2014.

[47] R. L. Lytton, J. Uzan, E.G. Fernando, R. Roque, D. Hiltunen, and S.M. Stoffels, *Development and validation of performance prediction models and specifications for asphalt binders and paving mixes* vol. 357: Strategic Highway Research Program, 1993.