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EXPERIMENTAL METHOD OF FATIGUE PERFORMANCE OF MASTIC ASPHALT FOR BRIDGE DECK PAVEMENT

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Abstract. Mastic asphalt is a type of pavement material that has good fluidity and is self-levelling at construction temperature for the bridge deck. There are highly accurate methods and indexes for evaluating fluidity and high-temperature deformation resistance for mastic asphalt-design and construction-control systems. The fatigue cracking is one of the main failure forms of bridge deck pavement. Therefore, the method used to evaluate the fatigue properties of pavement materials is also essential. The anti-deformation capability of the mastic asphalt must be increased, that results in poor fatigue performance and consequent failure of the bridge deck pavement to avoid the rutting of bridge deck pavement. In this study, a simple method is put forward for evaluating mastic asphalt fatigue performance. Impact toughness is defined as the area under the load-displacement curve of a three-point bending beam specimen under impact load to evaluate the fatigue performance

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of mastic asphalt. The four-point bending beam fatigue test is used to verify the rationality of the impact toughness test method. The results showed that there is a good correlation between the impacts toughness index of mastic asphalt produced under different mixing conditions and the accumulative dissipative energy and fatigue life demonstrated by the four-point bending beam test. Therefore, to evaluate the fatigue performance of mastic asphalt by impact toughness test. Fatigue performance and rut resistance are two ways to evaluate road performance of asphalt mixtures, but they are mutually restrictive. The results show that impact toughness and dynamic stability are inversely correlated. As the impact toughness increases, dynamic stability decreases. Therefore, balancing the fatigue performance and high-temperature rutting resistance of mastic asphalt in the design and quality control is very important.

Keywords: bridge deck pavement, dissipative energy, experimental method, fatigue, mastic asphalt, performance balance.

Introduction

The main structures of steel bridges are generally designed to 100 years, but the life of the steel bridge deck pavement is only about 10~15 years. Some bridge deck pavements needed to receive major rehabilitation within five years (Battista, Pfeil, & Carvalho, 2008; Liu, Medani, Scarpas, Huurman, & Molenaar, 2010; Medani, Huurman, Liu, Scarpas, & Molenaar, 2007). One of the main reasons is the considerable deformation due to the flexibility of the steel bridge deck. Higher quality is required for steel deck pavement. Generally, two types of asphalt concrete materials were used in steel deck pavement, gussasphalt (GA), and mastic asphalt (MA). Gussasphalt was originally developed in 1917 in Germany and was called Guβ, meaning river (Wang, Zhang, Zhu, Hao, & Xue, 2011). Originally, it was used as waterproofing material in building construction and for pavement construction (*BS 1447:1998 Specification for Mastic Asphalt (Limestone Fine Aggregate) for Roads, Footways and Pavings in Building, Published under the Authority of the Board of BSI*). Gussasphalt was successfully used in steel deck pavement (such as the Oberkasseler Bridge, Mulheim Bridge, and Zoo Bridge). It was later used in steel bridge construction in Europe (France, Sweden, the Netherlands) with good performance records. It was in the United Kingdom that the asphalt concrete material for steel bridge deck was first called MA, a term based on its characteristics.

The primary difference between GA and MA is in the processes used to produce these two asphalt concrete mixes. In producing MA, the mineral filler, bitumen, and fine aggregate are fed at ambient temperature into a mixer sequentially and are then mixed for 5–6 hours. The product of the mixing, called mastic epuré (ME), is then fed into a mixing truck (called a cooker) and mixed with a predetermined proportion of coarse aggregate to produce the final MA. For GA production, all ingredients are fed into a batch plant, are mixed for only 2 minutes, and then are dumped into the cooker for secondary mixing and transportation.

Other differences between GA and MA materials include the aggregate gradation, the mix design method, and property evaluation indexes. Despite the differences, both GA and MA contain more proportions of asphalt and fine aggregate and fewer amounts of coarse aggregate. Both flow easily and are thus self-levelling at construction temperature. The asphalt concrete mix for bridge deck generally termed MA in the study, as defined in European Union Specification *BS EN 13108-6:2006 Bituminous Mixtures – Material Specifications – Part 6: Mastic Asphalt, Published under the Authority of the Standards Policy and Strategy Committee*.

In warmer regions of China, the summer season is generally long and hot. With high traffic volume and a severe overloading problem, high-temperature performance is significant for highway and bridge pavements. Researchers studied various ways to improve the rutting-resistance property, such as increasing the content of Trinidad lake asphalt (TLA), using a harder grade of asphalt binder, or moderately pre-ageing asphalt mixtures during construction. However, the better an asphalt mixture resists rutting the worse is its ability to resist cracking. Therefore, performance indexes are needed to define the resistance to both ruttings at higher temperatures and fatigue cracking.

Rutting and fatigue cracking was observed on the same steel bridge deck pavements at many different locations in China. With the same source materials, MA mixtures, and construction methods, the question of how the two opposing distresses occur on the same pavement arises. The most likely reason is inadequate quality control during construction; The mixing temperature and time are very necessary to be monitored during MA production. Mixing time in the mixing truck vary within one and eight hours, but a long mixing time in the cooker require the use of additives to rejuvenate the MA mix. A similar situation also occurs in European countries during the construction of the bridge-deck asphalt pavement.

Tensile strain measured at the top of the wearing course of steel-bridge-deck pavements sometimes exceeds 500 με that is much larger than is observed at the bottom of conventional asphalt pavements. Considerable strain explains why most distresses reported for orthotropic steel-bridge pavements were related to fatigue cracking of asphalt mixtures (de Jong, 2006; Jeong, Kainuma, & Ahn, 2013; Kyung, Shin, Lee, & Jeon, 2006; Shirahata, Akasaka, & Iizuka, 2010). Among the many testing methods, the four- and five-point bending beam fatigue tests were frequently used in research to evaluate fatigue properties of asphalt mixtures (Pouget, Sauzéat, Di Benedetto, & Olard, 2011; Way, Kaloush, Sousa, & Zareh, 2009; Wu, 2009).

In 1993, in the United States, the Strategic Highway Research Program recommended the use of the four-point bending beam fatigue test to estimate fatigue cracking of asphalt mixtures (Yu & Zhang, 2011). Since that time, it has become a standard test adopted by the *AASHTO T321-2007 Standard Method of Test for Determining the Fatigue Life of Compacted Hot-Mix Asphalt (HMA) Subjected to Repeated Flexural Bending*. Among its advantages is a high sensitivity to mixture variables, a larger proportion of test specimens were subjected to a uniform maximum stress level, and bending behaviour similar to real pavement deformation. The four-point test was used by various researchers to evaluate the fatigue performance of pavements and has become popular worldwide as a fatigue test for asphalt mixtures.

However, the four-point bending beam fatigue test requires high-performing equipment and a long testing cycle. Construction contractors and supervisory companies often lack the right conditions to carry out this type of testing, so it is necessary to study simple fatigue testing for MA design and construction quality. The impact toughness test is developed in this study, while also adopting the accepted four-point bending-beam fatigue test to verify feasibility to control MA fatigue performance.

1. Objectives

Impact toughness, a concept normally used in metallic materials science (Buirette & Huez, 2010; Dilthey, Lüder, Bleck, Langenberg, & Nagel, 2000; Tamminen, Juutilainen, & Röning, 2010), is defined as the area underneath the load-displacement curve of a three-point bending specimen subjected to impact loading. Theoretical analyses based on fracture mechanics and energy principals along with laboratory testing (Mull, Othman, & Mohammad, 2005; Mull, Stuart, & Yehia, 2002; Pinho, Robinson, & Iannucci, 2006; Zou, Wu, & Xu, 2013) revealed an excellent correlation between impact toughness of asphalt mixtures and their ability to resist fatigue cracking: stated, the higher the impact-toughness value, the stronger the material. During travel on a steel bridge deck overlay, the wheel-to-overlay contact duration is very short, and traffic loading is considered an impact loading. It is therefore theoretically possible that impact toughness is used to evaluate fatigue performance for overlay material of steel bridge deck.

The objectives of this study were fivefold:

- to propose a test method and an index of impact toughness;
- to verify whether the impact toughness index is applied to fatigue performance evaluation of bridge deck pavement materials by using the four-point bending beam test;
- to see whether a relationship between impact toughness and accumulated dissipative energy is established;
- to find the correlation between impact toughness and the dynamic stability (DS) for MA mixture;
- to decide what indexes to use for quality control of MA design and pavement construction to improve the performance-balance requirements for the pavement materials.

2. Materials, sample preparation, and testing

2.1. Materials

Three types of asphalt binders (Table 1) were used in preparing the MA mixtures in this study, including a conventional asphalt binder commonly used in China (Pen 60/70 binder), a natural-material asphalt binder (TLA), and a blended asphalt binder (70% TLA, 30% Pen 60/70). Pen 60/70 indicates a penetration grade of 60/70 at 25°C.

The aggregate gradation used in MA (Standard System of Great Britain) is much finer than that used in GA. Finer aggregate gradation is generally beneficial for construction quality for steel-bridge-deck pavement. Therefore, the MA aggregate gradation and method of mix design were adopted in this research.

Table 1. Properties of asphalt binders

Properties	Unit	Pen 60/70	TLA	70% TLA+ 30% Pen 60/70
Penetration, 25 °C, 100 g, 5 s	0.1 mm	61	3	18
Ring & Ball temperature	°C	49.0	90.0	65.0
Ductility at 15 °C	cm	> 100	–	–
Solubility of TCE*,,	%	99.9	53.0	67.1
Flash point, COC**	°C	> 60	–	> 260
Density	g/cm^3	1.036	1.381	1.277
Ash (mineral matter)	%	–	36.5	–

Note: *TCE – trichloroethene; **COC – Cleveland Open Cup method

Table 2. Specified gradation of limestone fine aggregate

Gradation of Fine Aggregate*	Per cent by mass	
	Minimum	Maximum
Retained on 2.36 mm	–	2.5
Passing 2.36 mm and retained on 600 μm	4	21
Passing 600 μm and retained on 212 μm	8	32
Passing 212 μm and retained on 75 μm	8	25
Passing 75 μm	40	56

Note: *gradation determined by wet-sieving method (*BS 812-103.2 Testing Aggregates – Part 103: Methods for Determination of Particle Size Distribution – Section 103.2: Sedimentation Test*)

In this study, the fine aggregates consisted of natural limestone ground to the required gradations (Table 2) and were required to exhibit a minimum $CaCO_3$ content of 80% by mass. The combined fine aggregate and asphalt binder constitutes a ME mixture.

The coarse aggregates are defined as materials retained on a 2.36 mm sieve in the MA mixtures. The proportion of coarse aggregate was recommended in the British specifications. For heavily stressed areas, the coarse aggregate content in MA mixtures was specified to be 45% ± 10% by weight of the total mixture, and the remaining 55% consisted of ME. The soluble asphalt binder content was expected to be 14%–17% by weight of the ME mix; in the study, the percentage was 14.5%. For this study, the limestone fine aggregate gradation, the asphalt binder content, and the specification requirements are listed in Table 3.

Table 3. Gradation of fine aggregate and the soluble asphalt-binder content

Gradation	Sieve size, mm					Soluble asphalt-binder content, %
	above 2.36	0.6–2.36	0.212–0.6	0.075–0.212	below 0.075	
Percent by weight	0	16.0	20.0	23.0	41.0	14.5
BS 1447:1998*	0–2.5	4.0–21.0	8.0–32.0	8.0–25.0	40.0–56.0	14–17

Note: *BS 1447:1998 Specification for Mastic Asphalt (Limestone Fine Aggregate) for Roads, Footways and Pavings in Building, Published under the Authority of the Board of BSI.*

2.2. Preparation of mastic asphalt mixtures

Mastic asphalt (MA) mixtures used in this project were produced using two different methods. Conventionally, MA mixtures were produced in the laboratory following processes listed below:

- A-70 bitumen and TLA were heated to 160 °C–170 °C in the oven and mineral fillers were prepared at ambient temperature. They were, then weighed and fed into the mixer (Figure 1). The mixer was heated by gas under controlled-temperature conditions. Inside the mixer, the agitating vanes were operated by a diesel engine. The mixtures were allowed to reach 170°C–190°C and were then mixed for an additional 30 min.
- the fine aggregates, at ambient temperature, were fed into the mixer, and the entire mixture was agitated for 30 minutes after the mixing temperature reached 180 °C–195 °C; the product was called ME.
- the coarse aggregates were then added into the mixer at an ambient temperature, and the mixture was agitated until after the temperature reached 190 °C–215 °C. (the mixture was continued mixing 45 min–120 min in this study.)
- the batch of MA produced in the laboratory weighed about 700 kg.

The mixing time generally requires above 6 hours in the conventional mixing method. To meet the needs of large-scale bridge deck pavement, a more efficient automatic batch plant is used to produce MA. For example, the deck pavement for the Hong Kong–Zhuhai–Macao steel bridge has a scale above 500 000 m^2; production of the MA mixture for its pavement uses the following process. In the first step, all ingredients are fed into the

Figure 1. Conventional mixer

Figure 2. Mixing truck (Cooker)

batch plant and are mixed for about 2 min. The mixtures are then dumped into the mixing truck (Figure 2) for secondary mixing and transportation. The MA used in the study was produced by the above two processes to illustrate the universal applicability of the research results.

2.3. Laboratory testing

To evaluate the rutting resistance at higher temperatures and the fatigue characteristics of the steel-bridge-deck asphalt mixtures, the laboratory-based testing program consisted of three types of tests: the Wheel-Tracking Test (WTT), the impact-loading test, and the four-point bending beam fatigue test.

2.3.1. Wheel-Tracking Test

Many test methods were used worldwide to evaluate the rutting resistance of asphalt mixtures (Guo & Prozzi, 2006). In this study, a wheel-tracking device (Figure 3) was used to evaluate the high-temperature performance of various asphalt–concrete specimens, in according to Chinese Standard Test Method T0719 (*JTG E20-2011 Standard Test Method of Bitumen and Bituminous Mixture for Highway Engineering*). The Wheel-tracking test was developed by the Transport and Road Research Laboratory in Great Britain. This test was performed at a temperature of 60 °C, with a loading speed of 42 times/min and a wheel-contact pressure of 0.7 MPa. Total-deformation values were measured on the specimen surface after it was subjected to 60 min of repeated wheel loading. Also recorded was the DS, defined in this test as the number of wheel loadings required to induce a 1 mm deformation during a testing timeframe of 45 min–60 min.

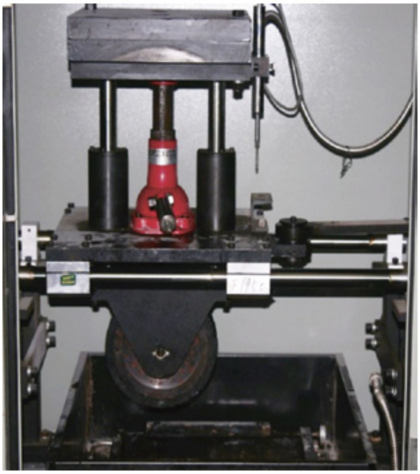

Figure 3. Wheel-tracking device

2.3.2. Impact-Loading Test

The Impact-Loading Test (ILT) was used to evaluate the fatigue characteristics of asphalt mixtures in a pavement system (Yu, 2005). In this test, asphalt concrete specimens made with different materials were compacted to the size 30.0×30.0×5.0 cm, and the specimens were then cut into beams of size 25.0×3.5×3.5 cm. After being cured in a water bath for 4 hours at a constant temperature of 15 °C, the beams were subjected to impact loading at a speed of 50 mm/min (Figure 4).

Derived from a loading-displacement curve (Figure 5), the impact toughness parameter (represented by the area under the curve up to the

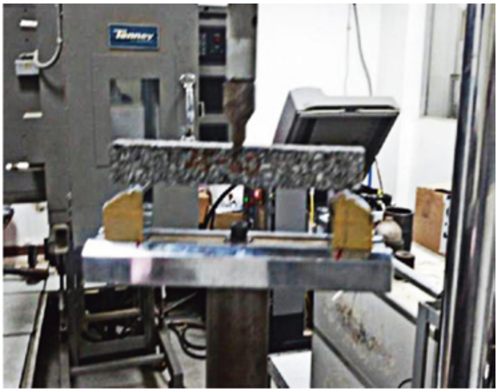

Figure 4. Impact-toughness test

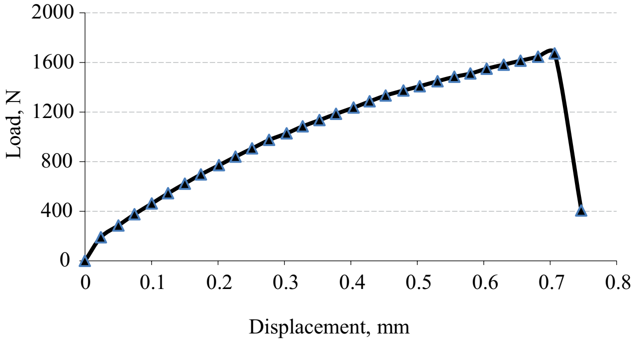

Figure 5. Loading-displacement curve

failure point), was used to characterise the fatigue capacity of the MA mixtures. The higher the impact toughness values of the materials, the better the ability to resist fatigue cracking.

For a single measurement, *a*, if

$$|a - average\ value\ of\ measurements| \le k\ standard\ deviation, \qquad (1)$$

where *k* is 1.15, 1.46, 1.67, or 1.82 if the number of specimens is 3, 4, 5, or 6, respectively.

If the measured data were determined to be valid, then the test result was equal to the average value. Otherwise, if measurement a was considered to be invalid, then it was removed from further analysis, and the test result was equal to the average value of the remaining data except for *a*.

Three kinds of devices for four-point bending fatigue test, provided by James Cox & Sons, Cooper Research Technology, and Industrial Process Controls, were considered. Yu (2005) evaluated these three types of bending devices in a constant-strain test mode, and the fatigue-test results were the same. Figure 6 shows the Cooper Research Technology test apparatus was used in the study.

The size of the specimens was 381.00 ± 6.35 mm (length) by 63.50 ± 6.35 mm (width) by 50.80 ± 6.35 mm (height), and the number of specimens ranged from three to six. Specimens were cured at a constant temperature for 4 hours and then were subjected to sinusoidal loading with a frequency of 10 Hz at 15.0 ± 0.1 °C. The initial stiffness modulus was at the point where the MA modulus started to decrease. The number of loading cycles required to reach the critical damage point, defined as the modulus at 50%, was the fatigue life of the MA mixtures. The same data-processing procedure as used in the ILT, was applied in assessing these fatigue-test data.

Figure 6. Four-point bending-beam fatigue test

3. Results and discussion

Specimens were prepared at various mixing temperatures for various mixing times. The prepared specimens were subjected to the four-point bending-beam fatigue testing at a strain level of 300 με, and a constant temperature of 15 °C. Specimens were also prepared for ILT at various mixing temperatures. The test results and the calculated accumulated dissipative-energy values for the various mixes are presented in Table 4. The experimental results show that the mixing temperature and time affect the accumulated dissipative energy, fatigue life, and impact toughness. The longer the mixing time, the worse the MA fatigue properties. The higher the mixing temperature, the shorter the fatigue life. The short mixing time and low mixing temperature were beneficial to fatigue performance.

Table 4. Results of four-point bending beam fatigue and impact-loading tests

Mixing equipment*	Mixing time and temperature	Accumulated dissipative energy, MJ/m^3	Fatigue life (300 με) cycles	Impact toughness, N·mm
A	45 min, 195°C	286.9	1 546 026	2154
B	2 h, 210°C	197.9	1 329 276	1697
A	2 h, 205°C	211.7	1 074 462	847
B	4 h, 240°C	137.1	831 555	202
B	1.5 h, 220°C	335.8	1 780 029	2040
B	2 h, 230°C	189.9	886 963	785
B	3 h, 230°C	163.8	702 931	543
B	4 h, 230°C	145.9	778 227	441
B	5 h, 230°C	122.4	719 439	375

Note: *A – using conventional mixing simulator, mixing time started when coarse was added to simulator and temperature reached the preset value; B – mixtures were agitated for 2–3 minutes in the batch plant and then put into Cooker truck for additional mixing time.

3.1. Dissipative energy and fatigue life

When viscoelastic materials are subjected to cyclic loadings, the strain–stress curve is different under loading and unloading conditions but mutually connect, forming closed paths termed hysteresis loops. The area within each loop represents the loss of strain energy during each cycle of loading and unloading for a given constant load. For a viscoelastic material, such as the asphalt concrete mixture, the external work applied to the material is consumed in part by inducing cracking on the surface and in part by inducing flow deformation. In fatigue testing, separating the energy applied to cracking from the energy transformed to heat is difficult (in plastic deformation).

A typical strain–stress curve under cyclic loadings is shown in Figure 7.

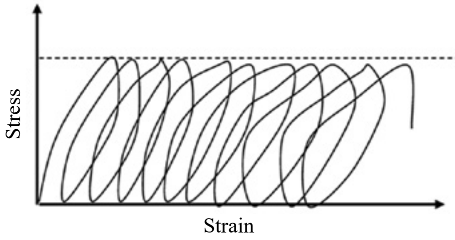

Figure 7. Strain–stress curve sketch

Because of stress-strain hysteresis, when an asphalt-concrete specimen is subjected to cyclic sinusoidal loading

$$\sigma(t) = \sigma_0 \sin(\omega t), \tag{2}$$

and the corresponding strain has the form

$$\varepsilon(t) = \varepsilon_0 \left[\sin(\omega t + \varnothing)\right], \tag{3}$$

where $\varnothing$ is the phase or dispassion angle.

Then, letting

$$x = \sigma_0 \sin(\omega t) \text{ and } y = \varepsilon_0 \left[\sin(\omega t + \varnothing)\right] \tag{4}$$

by mathematical transformation, the following Eq. (5) was derived to represent the stress-strain hysteresis curve under one loading cycle:

$$\frac{\left(x - \frac{\sigma_0}{2}\right)^2}{\left(\frac{\sigma_0}{2}\right)^2} + \frac{\left(y - \frac{\varepsilon_0}{2}\right)^2}{\left(\frac{\varepsilon_0}{2}\right)^2} - \frac{8\cos\varphi}{\sigma_0 \varepsilon_0}\left(x - \frac{\sigma_0}{2}\right)\left(y - \frac{\varepsilon_0}{2}\right) = \sin^2\varphi. \tag{5}$$

For sinusoidal loading, the dispassion energy at the i^{th} loading cycle was represented as

$$\omega_i = \pi \sigma_i \varepsilon_i \sin\varphi_i, \tag{6}$$

where ω_i – dissipative energy in i^{th} loading cycle; σ_i – peak stress in i^{th} loading cycle; ε_i – peak strain in i^{th} loading cycle; φ_i – phase angle in i^{th} loading cycle, for viscoelastic materials, $0 < \varphi < \frac{\pi}{2}$.

During fatigue testing, the accumulated dissipative energy is the sum of all areas enclosed by all hysteresis loops. The total dissipative energy at the damage point was represented as

$$W_f = \sum_{i=1}^{N_f} \omega_i, \tag{7}$$

where N_f – fatigue life, cycle; W_f – accumulated dissipative energy, MJ/m^3.

From their study on the characteristics of energy consumption of pavement specimens under fatigue loading, Van Dijk & Visser (1977) proposed the following Eq. (8):

$$W_f = A \cdot N_f^Z, \tag{8}$$

where A and Z are test constant.

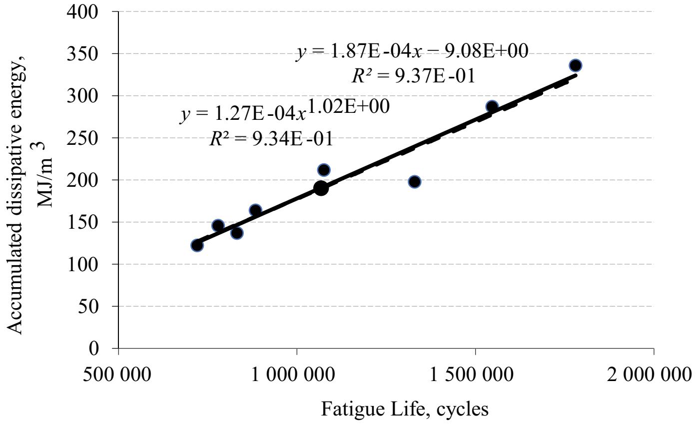

Figure 8. Relationship between fatigue life and accumulated dissipative energy

From Eq. (8), the fatigue damage occurs when cumulative energy consumption reaches a critical value. This phenomenon is known as the energy consumption criterion.

In the research, based on the 1977 study, the correlation between fatigue life and accumulated dissipative energy of the asphalt specimens was analysed (Figure 8).

Figure 8 shows the parameter Z of the Eq. (8) is 1.02, that is close to 1. Therefore, the correlation between fatigue life and the accumulated dissipative energy is considered linear. Linear regression analysis (dashed line in Figure 8; coefficient of correlation $R^2 = 0.937$) indicates that higher required accumulated dissipative energy at the damage point of material corresponds to longer fatigue life.

3.2. Impact toughness and accumulated dissipative energy

On the plot of a typical loading-displacement curve (Figure 5), the impact-toughness parameter is represented by the area under the curve up to the failure point. Impact toughness was interpreted as the energy required fracturing the specimen under impact loading. As shown in Figure 9, the theoretically predicted relationship between impact toughness and accumulated dissipative energy was confirmed in the results of the four-point bending-beam fatigue testing (at 300 με) that reveal the linear correlation ($R^2 = 0.82$) observed for impact toughness (in N·mm):

$$I = 9.54W_f - 890.33, \tag{9}$$

where I – impact toughness, N·mm; W_f – accumulated dissipative energy.

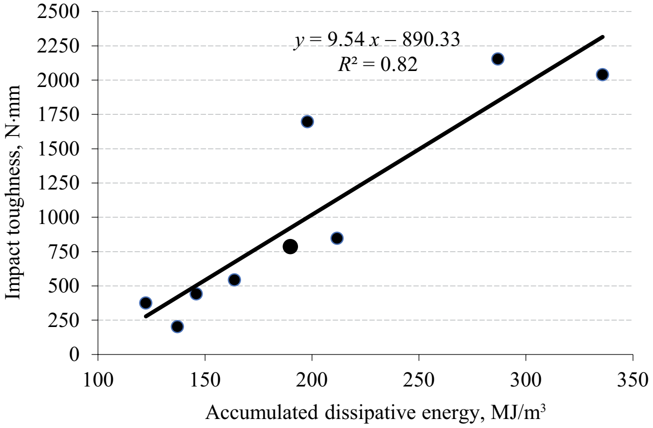

Figure 9. Relationship between impact toughness and accumulated dissipative energy

3.3. Impact toughness and fatigue life

The impact toughness test is a simple test that is easy to perform. As shown in Figure 10, the linear relationship between the impact toughness of the material and its fatigue life (obtained from four-point bending beam fatigue test) takes the following form:

$$N_f = 469.96I + 637833, \tag{10}$$

where N_f – fatigue life, cycle; I – impact toughness, N·mm; correlation coefficient – $R^2 = 0.922$.

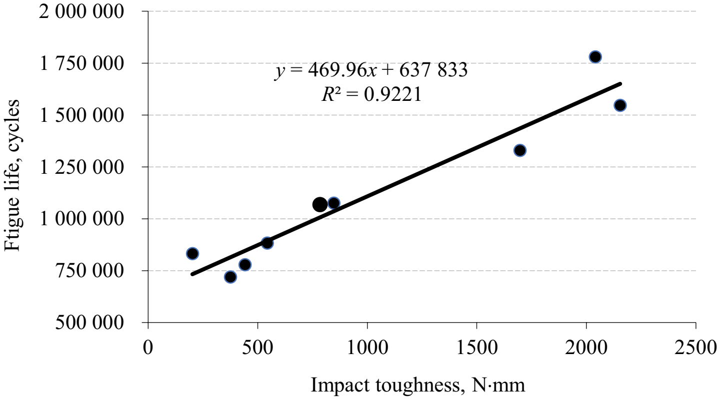

Figure 10. Relationship between impact toughness and fatigue life

Because of the high correlation coefficient, impact toughness was recommended as a quality control measure during bridge deck construction and as a reference index during the mixture design process in the study.

3.4. Rutting resistance and fatigue performance

With increasing mixing time and mixing temperatures, the MA mixture continues to age, and DS increases, thus increasing the ability of the material to resist rutting. However, impact toughness decreases with ageing, thus reducing the fatigue life of the mixture. Therefore, balancing the performance requirements of the MA mixture between the rutting resistance and fatigue cracking is important.

Wheel-Tracking Tests were performed on MA specimens to evaluate the rutting resistance of the material, and DS values were recorded. Regression analysis was performed to evaluate the interrelationship of impact toughness and DS (Figure 11). The exponential function between these two indexes is an inverse relationship with a good correlation coefficient (R^2 = 0.688). As impact toughness decreases, DS increases. Therefore, DS was used as the quality-control index during construction, and both a minimum and a maximum value are specified. When performing MA mixture design, materials selection, and construction control, both the impact toughness and DS were specified. The two indexes were used for quality control of the pavement project for the Hong Kong–Zhuhai–Macao Bridge; to ensure deformation

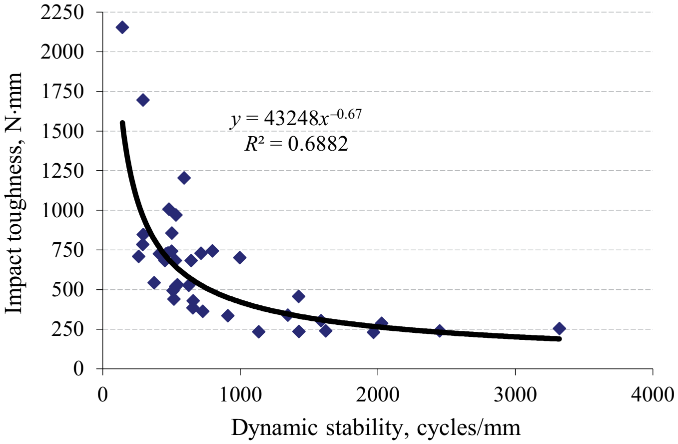

Figure 11. Relationship between impact toughness and dynamic stability for mastic asphalt

resistance and fatigue performance of the bridge-deck pavement, the DS was required to be 300~800 cycles/mm and impact toughness no less than 400 N·mm.

Conclusions

The following conclusions were drawn from this study:

1. The impact toughness parameter is derived based on the energy concept. It was regarded as the energy required to fracture specimens under impact loading. The results show that the relationship between impact toughness and cumulative dissipation energy is linear, the higher the cumulative dissipation energy of mastic asphalt is, the larger the impact toughness is, and correlation coefficient is 0.82.
2. The results of four-point bending beam fatigue tests reveal a linear relationship between accumulated dissipative energy and fatigue life of the mastic asphalt mixtures. The higher the required accumulated dissipative energy is at the damage point, the longer the fatigue life of mastic asphalt mixture.
3. There was a linear relationship between impact toughness and fatigue life obtained by four-point bending beam test, and the correlation degree is very high. Impact toughness test has low requirements for testing equipment and short test period, so it is suitable for fatigue performance index for design and construction of mastic asphalt mixture for bridge deck pavement.
4. Impact toughness and dynamic stability were found to be inversely correlated. As impact toughness increases, dynamic stability decreases. Therefore, fatigue performance and rutting resistance are mutually restrictive properties of mastic asphalt. Based on the required performance balance, impact toughness and dynamic stability are recommend to be used for quality control in mastic asphalt design and pavement construction. It is recommended that the dynamic stability meet the requirement of 300~800 cycles/mm and impact toughness meet the requirements of greater than or equal to 400 N·mm. Of course, it is appropriate to propose targeted index requirements for each specific project.

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