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Fracture resistance characterization of chemically modified crumb rubber asphalt pavement

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The fracture resistance of chemically modified crumb rubber asphalt (CMCRA) pavement was evaluated based on the *J*-integral concept. The chemical modification process used was developed by the Federal Highway Administration and patented in 1998. The results were compared to that of crumb rubber asphalt (CRA) and control asphalt pavement. Four semi-circular core specimens (76 mm radius and 57 mm thickness) were cut from each gyratory compacted cylinder (GCC) for the fracture resistance tests. Notches with different depth to radius ratios were introduced at the middle of the flat surface of each specimen. Three point bend loading was used to allow the separation of the two surfaces due to tensile stresses at the crack tip. It was found that the CMCRA pavement, had the highest residual strength, at all notch depths tested. The fracture resistance of the CMCRA pavement, based on J_c was found to be about twice that of the CRA and control pavements. The CRA pavement was found to have a slightly higher fracture resistance than that of the control pavement. Scanning Electron Microscopic examination of the fracture surface of each mixture revealed the microstructural origin of the improved fracture resistance of the CMCRA pavement in comparison with the control pavement.
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1. Introduction
The nation's infrastructure of asphalt roads is becoming increasingly costly to maintain and any improvement in service life, which can be gained, is a great economical advantage. The introduction of the mix design procedures known as Superpave^{TM} is an attempt to extend the service life of asphalt pavements. Superpave mix design was introduced through the Strategic Highway Research Program as established by Congress in 1987. The Superpave system is aimed at improving three areas of pavement performance, namely controlling rutting, low temperature cracking and fatigue cracking, through the use of performance based material characterization on both unaged and aged asphalt samples [1, 2]. This method specifies certain criteria that an asphalt binder must meet and establishes a procedure to measure the working temperature range over which the binder is able to meet the prescribed specifications, resulting in the performance grade (PG) of the binder. This method provides a quantitative means to judge the effectiveness of one binder compared to another over an actual working temperature range, rather than base the grade of a binder on a single test at one temperature. Thus, pavement engineers can choose a system that more closely matches the overall working temperature range of their location, hopefully extending the service life of the pavement.

The polymer modification of asphalt concrete with synthetic polymers began with the introduction of polyacrylonitrile, polystyrene, polyvinyl chloride and styrene-butadiene-styrene [3]. By 1980, over 1000 papers had been presented or published on the utilization of polymers in asphalt modification, and yet due to increased cost and a lack of understanding of the technology, until the past two decades polymer modification received limited use. Polymer modifiers are introduced in an attempt to increase the high temperature stiffness and low-temperature flexibility of asphalt pavements [4, 5]. These two properties are mostly responsible for decreasing a pavement's susceptibility to environmental challenges and increasing its lifetime. Superpave has presented means to determine the effectiveness and

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need for polymer modification of pavement, by quantitatively defining the ability of the modifiers to extend the working temperature range of a binder to meet a certain PG.

Crumb rubber (CR) has been suggested as an asphalt modifier for pavement applications for quite some time [6]. Originally Crumb rubber (CR) modifier was used as an additive in an attempt to improve the performance and extend the lifetime of asphalt pavement mixtures. In the early 1960's, several State Highway Agencies tried using SBR and neoprene latex rubbers in asphalt concrete. The mixtures at that time were too expensive compared to the benefits. More recently, the addition of crumb rubber to pavement has been seen as a means to alleviate the environmental problem of scrap tire storage and disposal. In 1991, the Intermodal Surface Transportation Efficiency Act (ISTEA), section 1038(d) was passed in the US, which mandated the use of recycled rubber in a certain quantity of the asphalt pavement laid. Unfortunately, the technical basis for the use of crumb rubber in pavement did not exist, and because of the many technical issues that presented themselves and the additional cost of laying pavement with crumb rubber, in 1995, the National Highway Safety Designation Act section 1038 struck down the 1991 ISTEA section 1038(d) mandate. In spite of this, various states and institutions are still conducting research on the use of CR as a modifier for asphalt pavement. A variety of studies, both laboratory and field, with mixed results on the effectiveness of crumb rubber as an asphalt pavement modifier have been presented [7–11]. A summary of the work done through 1993 has been reported in a Federal Highway Administration/EPA report [12]. The basic conclusions of this report were that when properly constructed, there is no evidence that shows that pavements containing recycled rubber will not perform adequately. Although it is commonly noted that CR produces asphalt binders and mixtures with improved low and high temperature properties, one common problem mentioned in several studies [11, 13–15] is seperation between the crumb rubber and asphalt binder during storage. This is in part due to the crumb rubber remaining in particle form after mixing, not reacting chemically or dissolving in the asphalt binder [14].

Recently, the US Federal Highway Administration (FHWA) has developed a chemically modified crumb rubber asphalt (CMCRA) [16–19] in an attempt to alleviate the problem of seperation. The proprietary process developed at the FHWA, has produced a chemically modified crumb rubber with superior seperation characteristics during hot storage compared to standard crumb rubber asphalt. This is accomplished by treating the crumb rubber with certain chemicals in order to generate free radicals on the surface. This allows the CR to better interact with the asphalt. It was also shown that the continuous PG grading of a variety of asphalts tested were increased at both the high and low temperature by the addition of both crumb rubber and chemically modified crumb rubber. Although the two types of crumb rubber appeared to increase the PG approximately the same amount, further study of the solubility data of the two types of crumb rubber revealed that the chemical modification of the crumb rubber drastically improved the binder solubility and thus the homogeneity of the binder [20].

Although the work done on the chemically modified crumb rubber asphalt binders reveals promising results, performance related properties of the CMCR asphalt mixture have not been investigated.

Standard tests for the evaluation of ultimate strength and elastic modulus can only reveal the behavior of a homogeneous material with no inherent defects. Heterogeneous materials such as asphalt pavement do not fit this description. Fracture resistance characterization is a more relevant approach since it accounts for the flaws as represented by a notch, this in turn reveals the resistance of the material to crack propagation (fracture resistance). Asphalt pavements are considered elasto-plastic/visco-plastic materials and therefore linear elastic fracture mechanics theory is inadequate to evaluate their fracture resistance. The J-integral concept [21] is more appropriate to describe the fracture resistance of asphalt mixtures. In order to determine J_c using specimens with various notch-to- depth ratios, the following equation is used:

$$J_c = -\left(\frac{1}{b}\right)\frac{dU}{da} \tag{1}$$

where b is the specimen thickness, a is the notch depth and U is the total strain energy to failure, i.e. the area up to fracture under the load-deflection plot.

The J-integral approach was used instead of the stress intensity factor, K_I to study the fatigue crack growth kinetics of various asphalt mixtures. [22, 23]. These studies employed three point bend beam and Marshall-type cylindrical samples respectively. Both studies concluded that J_c could be used as a fracture mechanics characterization parameter of asphalt mixtures. Later, the concept of J_c was also applied to characterize the fracture resistance of asphalt mixtures at low temperatures [24]. In this study two methods were used; the calibration curve method with multiple notch depths and a single specimen method. Dongre *et al.* found that J_c is sensitive to asphalt mix properties and concluded that its use warranted further study. In 1997, Bhurke *et al.* studied polymer modified asphalt concrete using the J_c fracture resistance approach [25]. Four different polymer additives, including styrene-butadiene-styrene, an epoxy based system and styrene-butadiene rubber were studied as modifiers in an viscosity graded AC-5 asphalt. They concluded that the tests were repeatable and were sensitive to material differences due to polymer modification.

The J_c characterization of asphalt pavement so far has mostly used, three point bend beam specimens of substantial span length. Difficulties can arise during testing with this specimen due to the sagging of the beam under its own weight, especially at elevated temperatures. This deflection of the bending beam specimens (under their own weight) will lead to considerable error in the calcuclation of J_c. It would be more convenient if a specimen can easily be made from the gyratory compacted cylindrical specimens already produced for other tests specified by the Superpave method or as a core obtained from the field.

In the present work, a semi-circular core specimen is introduced for the evaluation of the fracture resistance, J_c, of various asphalt mixtures based on Equation 1. It has the advantages of being compact and stable so that there is minimal deformation due to its own weight. It can also be obtained from standard cores prepared in the gyratory compactor or taken from the field. In addition, multiple specimens can be obtained from one core, reducing the error caused by heterogeneities from one core to the next. This specimen was used by Chong *et al.* to evaluate the critical value of the J-integral of rock and cementous materials [26, 27].

The emphasis of this work is placed on the fracture resistance characterization of the CMCRA mixtures using the semi-circular specimen. This important performance related property reveals the degree of interaction between the binder and the aggregate in the mixture, as well as the cohesion of the binder itself. Comparison will be made with a crumb rubber asphalt mixture, as well as a control mixture. Scanning electron microscopy is used to elucidate the micromechanical behavior responsible for the observed fracture resistances.

2. Materials and test methods

2.1. Binders

Three different binders were used for this study; one control binder produced from air-blown asphalt with no catalyst, one asphalt modified with plain crumb rubber and one chemically modified crumb rubber asphalt. The CRA and CMCRA contained the same rubber content. The binders were designed according to the Superpave specifications and were prepared by the FHWA Turner-Fairbank Highway Research Center. The source for all of the binders was nearly 100-percent Boscan crude from Venezuela. Table I presents the specifications for the binders used. The asphalt binder content of the mixture was 4.85 percent by total mass of the mixture.

TABLE II Aggregate gradation

Sieve size (mm)	Percent passing
25.0	100.0
19.0	98.7
12.5	76.0
9.5	62.0
4.75	44.0
2.36	32.1
1.18	23.8
0.600	16.9
0.300	11.3
0.150	7.9
0.075	5.5

2.2. Aggregates

The aggregate gradation, which is shown in Table II, met the requirements for a Virginia Department of Transportation (VDOT) SM-3 surface mixture gradation for high traffic areas. The aggregate consisted of 61% No. 68 diabase, 31% No. 10 diabase, and 8% natural sand. It had a nominal maximum aggregate size of 19.0 mm. The specific gravities of the aggregate blend are: Bulk Dry = 2.892, Bulk SSD = 2.916, Apparent = 2.961. The percent absorption was 0.8 and the L. A. Abrasion of the No. 68 aggregate was 14. The % flat & elongated particles in the No. 68 diabase was 21. The fine aggregate angularity of the No. 10 diabase was 49. and the fine aggregate angularity of the natural sand is 45. This aggregate was used in the three asphalt mixtures under consideration. These are control, CR and CMCR asphalt mixtures. The same aggregate and the gradation shown in Table II are also being used in NCHRP Project 09-17 "Accelerated Laboratory Rutting Tests: Asphalt Pavement Analyzer," and NCHRP 09-19 "Superpave Support and Performance Models Management," in order to tie the projects together.

2.3. Specimen geometry and loading configuration

The specimen shown in Fig. 1, is obtained by slicing a cylindrical core along the central axis to obtain two half

TABLE I Superpave rheological properties of asphalt binders

Binder type / Properties	Air-blown asphalt PG 70-28	Unmodifed crumb asphalt rubber (CRA) PG 70-22	Chemically modifed crumb asphalt rubber (CMCRA) PG 76-28
Tenderness		Original asphalt binder	
Temperature at a $G^*/\sin\delta$ of 1.00 kPa and 10 rad/s, °C	74	73	79
Rutting		RTFO residue (short-term aging)	
Temperature at a $G^*/\sin\delta$ of 2.20 kPa and 10 rad/s, °C	75	72	80
Fatigue cracking		RTFO residue (short-term aging)	
Temperature at a $G^*/\sin\delta$ of 5000 kPa and 10 rad/s, °C	16	14	15
Low-temp cracking		RTFO/PAV residue (long-term aging)	
Temperature at a creep stiffness of 300 MPa and 60 s, °C	−20	−23	−25
Temperature at an m-value of 0.30 and 60 s, °C	−25	−17	−18

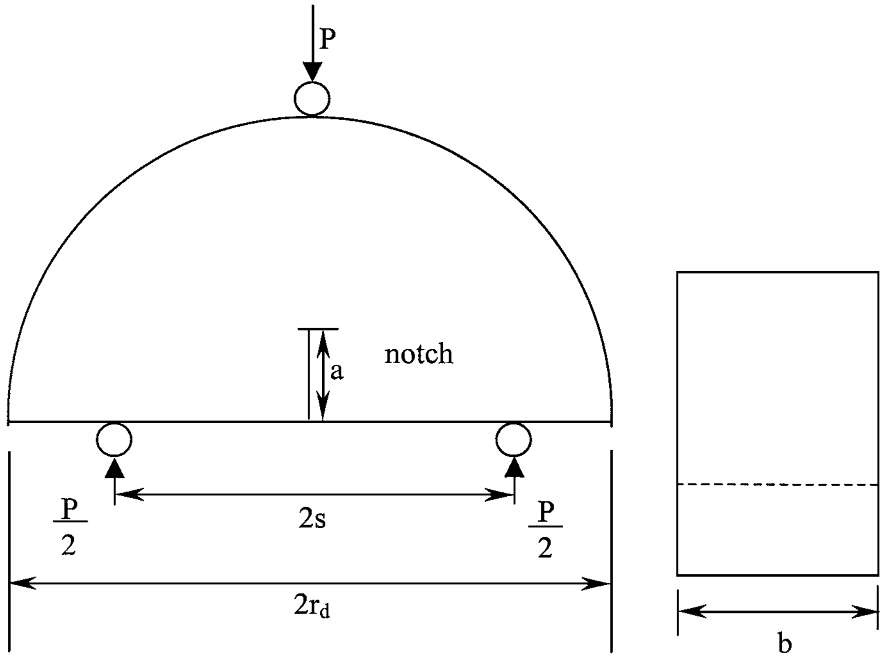

Figure 1 Semi-circular core specimen.

cylinders. These half cylinders are then sliced perpendicular to the axis to obtain the semi-circular specimens. In the case of asphalt concrete, standard gyratory compacted 152 mm by 152 mm cores produced in accord with Superpave procedures were used. The distance between the bottom points of load application, 2 s, was 127 mm. The advantage of obtaining four test specimens from one core is that it reduces the scatter in data associated with different cores of heterogeneous materials such as asphalt concrete. Three specimens from each material were tested at each notch depth. Notch depths of 25.4 mm, 31.8 mm and 38 mm were used. The notches were introduced using a tungsten carbide saw blade of 1.6 mm thickness.

2.4. Mechanical testing

The semi-circular core specimens were loaded monotonically on an MTS machine at a cross-head speed of 0.5 mm/min in a three-point bend load configuration, as shown in Fig. 2. The load–deflection curves were recorded on a standard X-Y recorder. The tests were conducted at ambient temperature ($\sim$24°C).

2.5. Scanning electron microscopy

Post failure analysis was performed on the fracture surface of tested samples using a Scanning Electron Microscopy. The SEM samples (about 25 mm $\times$ 35 mm) were cut from the fracture surface adjacent to the crack

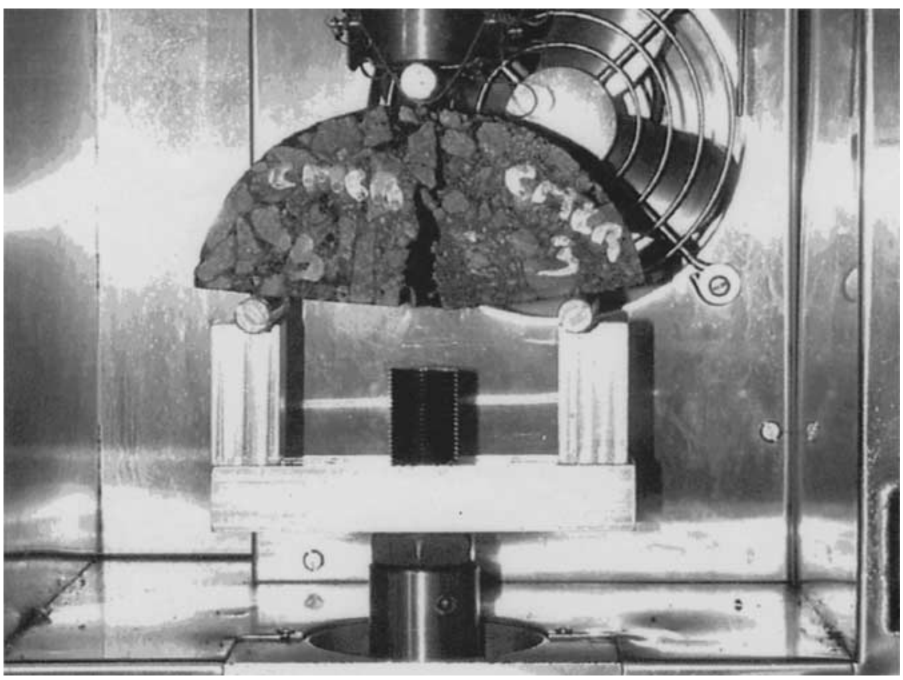

Figure 2 Loading configuration.

tip. The samples were coated with a thin platinum layer prior to fractographic analysis.

3. Results and discussion

3.1. Load–deflection behavior

The load deflection behavior of notched specimens, from the three asphalt mixtures under investigation, with different notch depths are shown in Figs 3–5. These curves represent the average values for three samples tested from each mixture at each notch depth. The relationships presented in Fig. 3 are for the control asphalt mixture. It can be seen from Fig. 3 that the initial load deflection response is almost linear, for all specimens at various notch depths. A non-linear response is displayed immediately before the maximum sustainable load is reached. This is the case for all notch depths

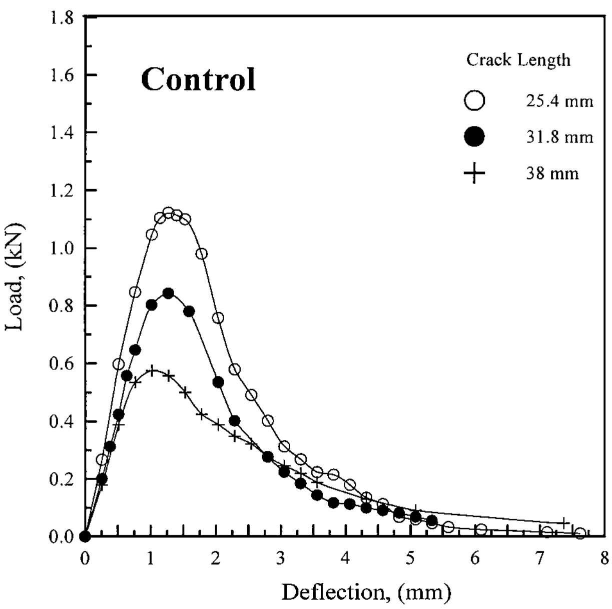

Figure 3 Load-deflection behavior for the control asphalt mixture tested statically in three-point bending.

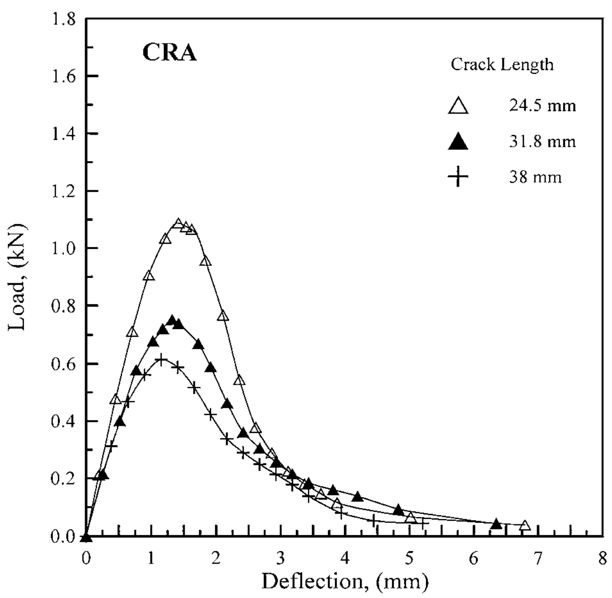

Figure 4 Load-deflection behavior for the CRA mixture tested statically in three-point bending.

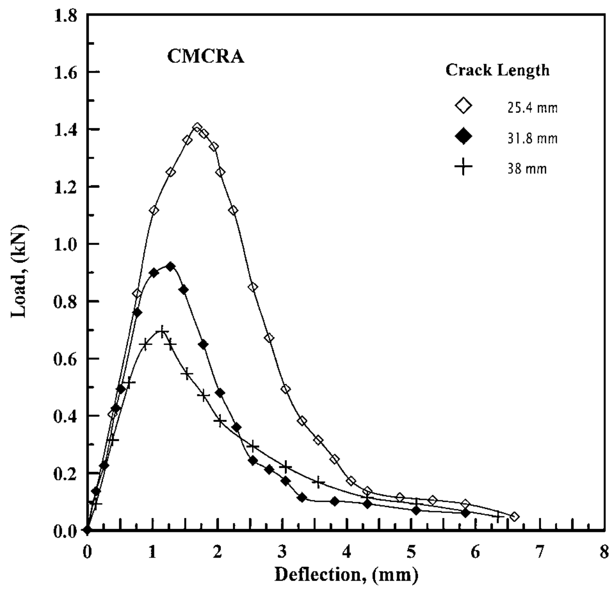

Figure 5 Load-deflection behavior of the CMCRA mixture tested statically in three-point bending.

tested for the control mixture. The crack then propagated from the notch tip after the maximum load was reached, and the fracture occurred in the Mode I; opening mode. The load deflection curve decayed slowly due to the mechanical interlock between the aggregate and the asphalt binder.

The load deflection behavior of the CR asphalt mixture, shown in Fig. 4, resembles that of the control mixture. There is a small increase in nonlinearity after the first linear portion of the load deflection curve, associated with all notch depths tested. The CR asphalt mixture displays a decaying load–deflection curve similar to the control asphalt mixture after the maximum load was reached. There is no significant difference in the maximum load sustained by the CR and control mixture at the same notch depth. There is however, a slight increase in the deflection at the maximum load for the CR asphalt mixture. This can be attributed to the capability of the CR modified mixture to store more energy than the control mixture.

The load deflection behavior of CMCR asphalt mixture at various notch depths is shown in Fig. 5. It should be noted that each curve in Fig. 5 represents the average of three identical specimens having the same notch depth. The scatter in the data for all of the materials tested was 15% or less based on twice the standard deviation at three different points on each load-displacement curve. An interesting behavior emerges in Fig. 5. The chemical modification of the CR has produced a more cohesive binder with a better adhesion to the aggregate. This is manifested in the higher residual strength (maximum load for notched specimens) and the deflection at maximum load in comparison with the control and the CR mixture at the same notch depth. The prominent nonlinearity, which can be seen in Fig. 5, particularly with a 25.4 mm notch depth is indicative of a toughening mechanism. This can be attributed to the activation of the surface of CMCRA resulting in a strong interaction between the rubber and asphalt and the binder and the aggregate. This will be investigated next through the evaluation of J_c for the CMCRA mixture in comparison with the control and CR asphalt mixtures.

The crack trajectory in a typical specimen of the CMCR mixture is shown in Fig. 2. The crack always initiated from the notch tip and traveled vertically upwards through the path of the least resistance. This is exemplified in Fig. 2 by the crack going around a large piece of aggregate. It is also noticed that this specimen gives reproducible results. That is specimens always fracture along a similar path and the load displacement curves as discussed earlier are within 15% of each other based on twice the standard deviation. In addition, fracture propagation is always due to tensile bending stresses. As the crack advances the specimen hinges under the top loading roller and the two 'halves' of the specimen rotate outward. This failure mechanism, associated with this simple test specimen, is appropriate in testing the fracture resistance of heterogeneous materials such as asphalt. It allows for examination of the cohesive strength of the binder as well as the interfacial strength between the binder and the aggregate.

3.2. Fracture resistance, J_c

In order to obtain the critical value of the fracture resistance, the area under the loading portion of the load deflection curves, up to the maximum load, was measured from the curves presented in Figs 3–5 for the three asphalt mixtures. These values were then plotted as a function of notch depth as shown in Fig. 6 for the

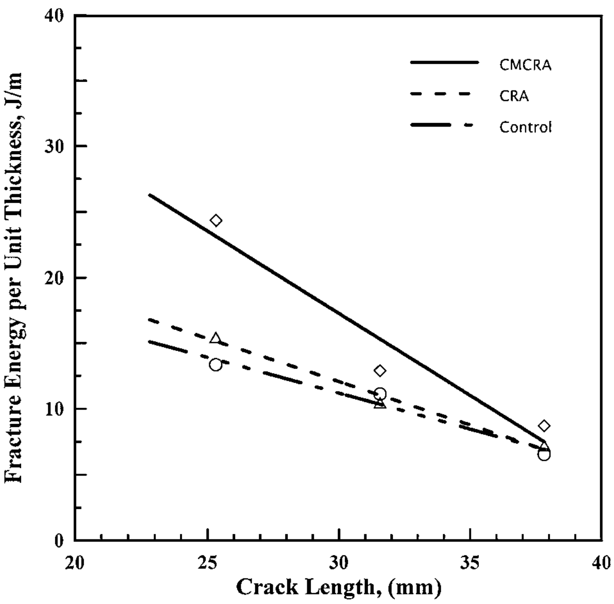

Figure 6 The fracture energy per unit thickness for the three asphalt concrete mixtures as a function of crack length.

Control, CRA and CMCRA mixtures. As can be seen the relationship between the total strain energy to failure U and the notch depth for all three of the asphalt mixtures are very linear. The correlation coefficients for the three straight lines are 0.93, 0.98, and 0.96 for the CMCRA, CRA and the Control mixture, respectively. It should be noted that three different notch depths for each material were used and that three samples were tested at each notch depth. The utilization of the semi-circular specimen to determine the critical value of the J-integral intended to use only two notch depths [28]. On this basis, Equation 1 can be written as:

$$J_c = \left(\frac{U_1}{b_1} - \frac{U_2}{b_2}\right)\frac{1}{a_2 - a_1} \tag{2}$$

where U is the strain energy to failure obtained from the load deflection behavior, b is the specimen thickness and a is the notch depth. The subscripts 1 and 2 refer to notch depths 1 and 2 respectively. In the current study, however, three notch depths were used. This will increase the accuracy of the value of J_c for these asphalt mixtures.

The slope of the lines presented in Fig. 6, is the critical fracture resistance, J_c, for the three mixtures as obtained from Equation 1 instead of Equation 2. These values are given in Table III. As can be seen, the CMCR asphalt has a value of J_c which is almost twice that of the other two mixtures. The CR and control asphalt mixtures have similar values of J_c, with CR being 20% higher. The higher fracture resistance of the CMCR asphalt mixture is attributed to the chemical modification, which has caused the mixture to become more resistant to crack tip separation. An attempt to explain this mechanism will be discussed in the next section, through studying the fracture surface morphology of the three mixtures.

Table III presents the values of J_c obtained in two other studies using three point bend beam specimens, as well as the J_c data obtained for the mixtures tested here. The AR-4000 and AC-20 mixtures were tested at 16°C, the AC-5 polymer modified mixtures were tested at −10°C. The data from the current study were obtained using semi-circular core specimens at 24°C. As can be seen, the data are on the same order of magnitude and mostly all within one fold of each other, lending credibility to the J-integral method of fracture resistance characterization. If one accepts J_c as a material parameter, characteristic of the resistance to fracture, it should not depend on temperature or specimen geometry. This is true for linear elastic materials with a small degree of yielding or elastic-plastic materials. There is a challenge however, with asphalts and polymer modified asphalts, which become visco-elastic and/or visco-plastic when they under go climatic changes, i.e. winter

TABLE III Comparison of the critical fracture resistance for various asphalt mixtures

	Control	CRA	CMCRA	AR-4000	AC-20	AC-5 + 5% SEBS	AC-5 + 2% Elvaloy
Critical fracture resistance, J_c (kJ/m^2)	0.54	0.65	1.23	0.63	1.03	0.42	0.48
		Current study		Ref. 24		Ref. 25	

to summer. Nevertheless, given the consistency of the data presented for these three studies, J_c has the potential to be a material parameter. Therefore, it is important to further the research in this area and develop a standardized technique for J_c determination of unmodified and modified asphalts.

It should be noticed that the value of J_c represents the fracture resistance of a material under monotonic loading (load excursion). It does not reflect the durability or fatigue lifetime. Nevertheless it can be a valuable correlative tool in fatigue crack growth studies. Thus, it is equally important to study the fatigue crack propagation behavior of the asphalt mixtures under consideration using the proposed semicircular specimen and the J-integral concept. This will provide a sound evaluation of the behavior of the chemically modified mixture under cyclic loading.

3.3. Fracture surface morphology

Scanning electron microscopy (SEM) was used to examine the fracture surface of typical specimens from the three mixtures. Samples were cut from the fracture surface in an area just ahead of the crack tip for SEM examination. This is the region exposed to tensile stress during the static-bend testing of the semicircular specimens. It is this area which reveals the micromechanism associated with the fracture resistance of the mixture. Figs 7–9 show micrographs taken at 500×

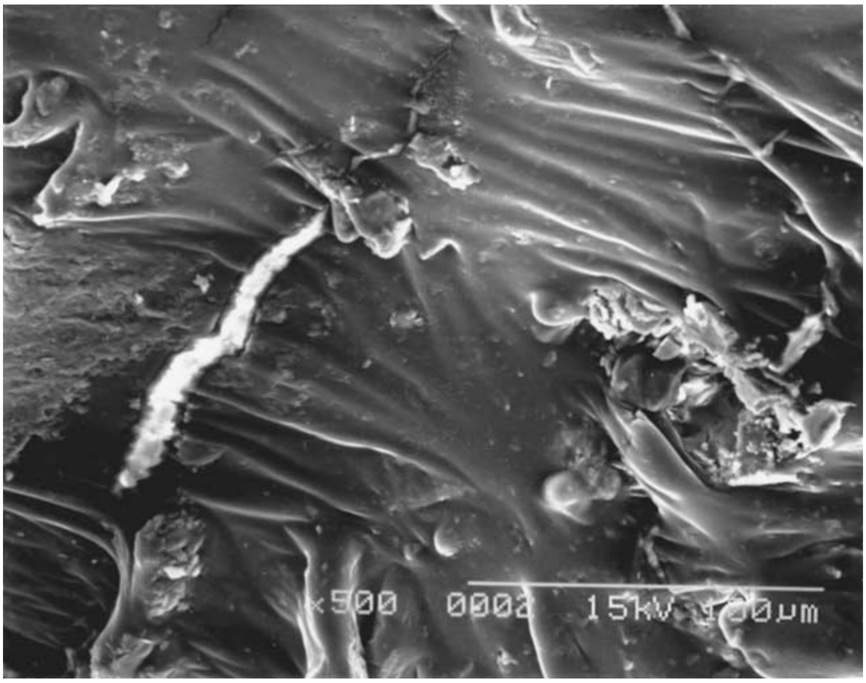

Figure 7 Fracture surface of the statically failed control asphalt mixture at 500× magnification.

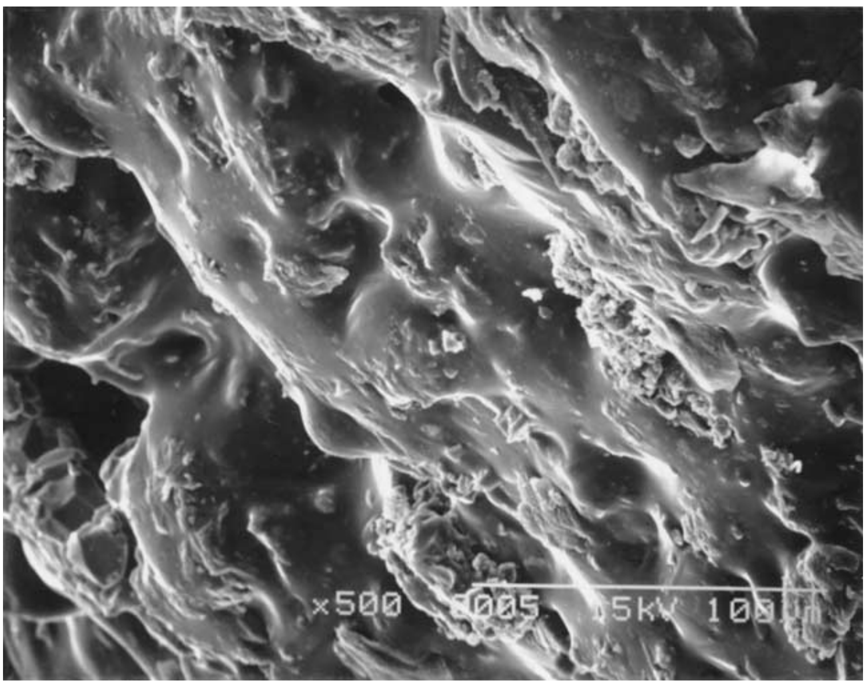

Figure 8 Fracture surface of a statically failed CRA mixture at 500× magnification.

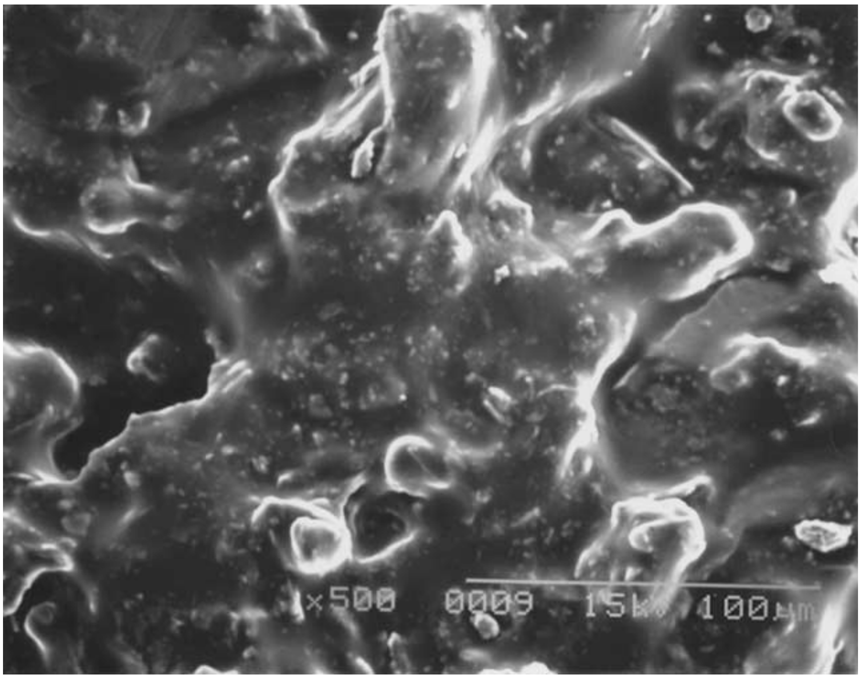

Figure 9 Fracture surface of a statically failed CMCRA mixture at 500× magnification.

magnification from the Control, CRA and CMCRA mixtures, respectively.

The fracture surface in Fig. 7 for the control specimen appears to be smooth with some ridges, indicative of a slight resistance of the binder to fracture. A few pieces of uncoated aggregate are seen in the middle of the right side of the micrograph. A couple of mini-cracks can also be seen in the binder. In Fig. 8, a micrograph of the fracture surface of the CRA mixture, the surface appears smooth with larger ridges and well pronounced dimples. The small particles, which are seen all over the surface appear to be partially coated. Fig. 9 shows the CMCRA mixture at 500×. The CMCRA mixture has a more tortuous appearance with voids and larger extrusions as opposed to the gradual dimples in the CRA mixture. Many finer fracture events are seen. The small underlying particles are totally covered in binder, revealing good interfacial adhesion between the binder and the aggregate/rubber particles. This increase in interfacial adhesion can explain the increased fracture resistance of the CMCRA mixture.

In Figs 10–12 the three asphalt mixtures, control, CRA and CMCRA are shown at 2000× magnification, respectively. Fig. 10 shows an area of exposed aggregate in the control specimen. It is clear in Fig. 10 that the surface of the fine particle aggregate is clean, indicative of a lack of adhesion, although there is some coherence within the binder as shown earlier in Fig. 7,

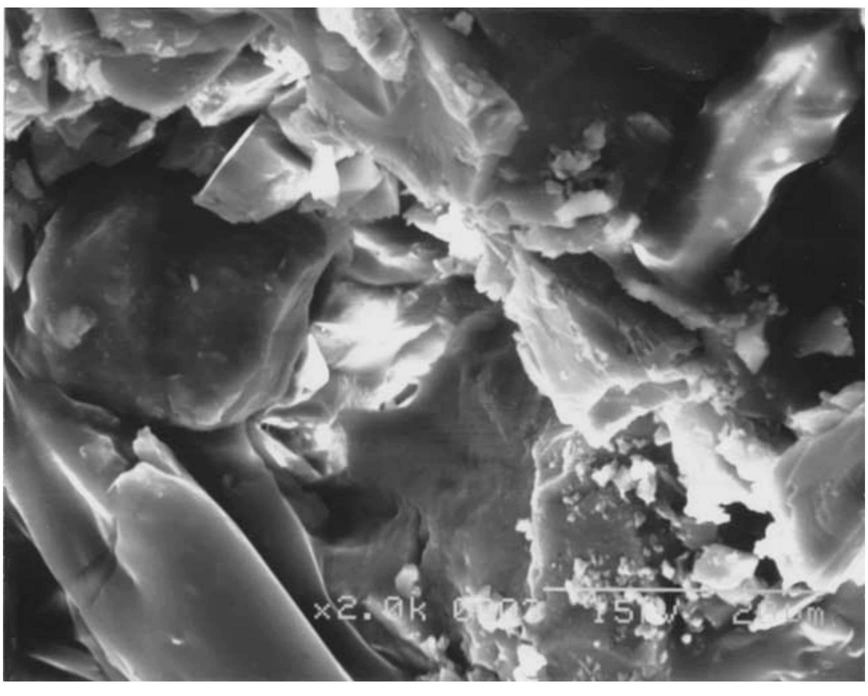

Figure 10 Fracture Surface of the statically failed Control asphalt mixture at 2000× magnification.

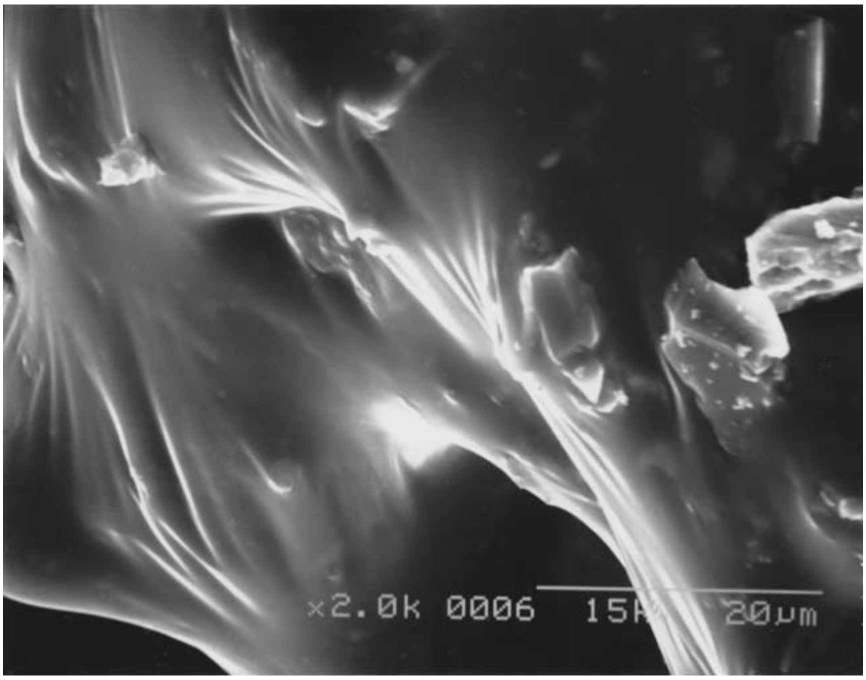

Figure 11 Fracture surface of the statically failed CRA at 2000× magnification.

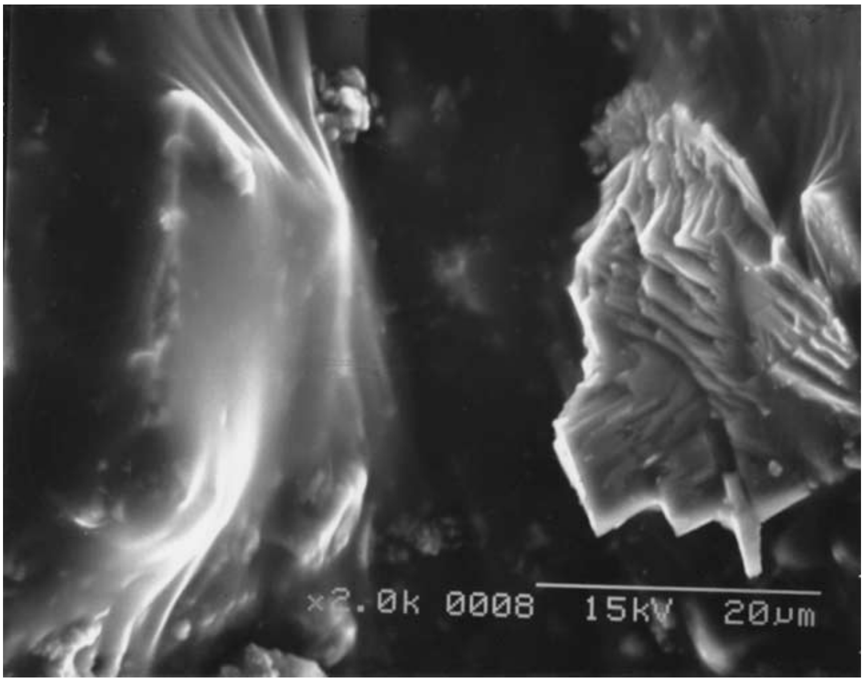

Figure 12 Fracture surface of a statically failed CMCRA mixture at 2000× magnification.

at 500× magnification for the control mixture. There is however discontinuity of the binder, which prevents it from completely coating and wetting the aggregate. In Fig. 11, the CR alone appears to have enhanced the consistency of the binder, creating finer ridges and better wetting of the particles. In Fig. 12, the CMCRA mixture binder appears more continuous and coherent without discontinuity. Several micro and mini aggregate fracture surfaces such as on the right side of Fig. 12 are also seen. These fractured aggregates can be seen all over the surface, even at low magnification. This is clear evidence of strong interfacial adhesion between the CMCRA binder and the aggregate. Thus, the chemical modification of the asphalt has enhanced the cohesiveness of the binder as well as the interfacial adhesion between the aggregate and the binder. These are the mechanisms, by which the chemically modified crumb rubber asphalt mixture, acquired its fracture resistance.

4. Conclusions

- Semi-circular core specimens tested in three point bending provided a stable testing configuration for stress-strain measurements of asphalt mixtures, which could then be used for fracture resistance determination.

- The critical fracture resistance, as determined from the J-integral approach, for Chemically Modified Crumb Rubber Asphalt was found to be twice that of Crumb Rubber Asphalt with the same rubber content and a control asphalt mixture.
- Fracture surface examination of the three asphalt mixtures reveals that the Chemically Modified Crumb Rubber asphalt has better cohesion within the binder and better adhesion to the aggregate surface, which may be the mechanisms responsible for its increased fracture resistance, compared to Crumb Rubber and a control asphalt.
- Comparison of the fracture resistance, based on the J-integral approach, between three studies reveals the consistency of this method. Based on this and the importance of the value of fracture resistance in fatigue models, future effort should be placed on developing a uniform method to obtain J_c and investigate further the effects of environmental conditions and specimen geometry on its value.

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