Development of Fracture Criterion for Asphalt Mixes at Low Temperatures

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This paper describes the development and verification of a fracture criterion sensitive to the elastic-plastic behavior of asphalt concrete mixes. This method is based on the elastic-plastic fracture mechanics (EPFM) concept that leads to the laboratory determination of the critical strain energy release rate, also called the critical value of J integral, J_{1c} . Also employed was the linear elastic fracture mechanics approach (LEFM), which leads to the determination of critical stress intensity factor, also called fracture toughness, K_{1c} . Asphalt concrete beams 3 in. by 3 in. by 16 in. were tested under three-point bending. Notches 1/16 in. wide and long enough so that the ratio of notch depth to beam depth always remained between 0.5 and 0.7 were sawed at the midpoint of the beams to give a single-edge notch beam geometry. A locally used mix design called ID-2 was employed. Twelve asphalt cements were selected from around the country for making the mixes. Testing was conducted at four temperatures below 60°F (16°C): -5°F (-21°C), 10°F (-12°C), 25°F (-4°C), and 40°F (4.0°C). Both EPFM and LEFM approaches were used to analyze the laboratory data. The results were compared with the more routine tests, such as indirect tensile strength for asphalt mixes, as well as the Fraass brittle point test, T_{PEN1.2}, and T_g from DSC and DMA for asphalt cement. It was found that at low temperatures (below 60°F (16°C)) the EPFM approach with J_{1c} as a fracture characterization parameter is sensitive to asphalt mix properties. J_{1c} also seems to be related to the routine parameters. LEFM with K_{1c} as a fracture characterization parameter is not sensitive to asphalt grade or source.

Pavement designers in recent years have expressed concern over the cracking of flexible pavements under various traffic loading and environmental conditions. One type of distress that manifests itself as alligator or map cracking is initially confined to localized zones and spreads at an increasing rate. Another type of pavement distress appears as transverse cracking-that is, nearly straight cracks across the pavement or perpendicular to the direction of traffic. Transverse cracks are caused by stresses produced by temperature excursion (thermal stresses). One of the causes for temperature-induced transverse cracking in asphalt pavements is temperature cycling that eventually results in an exceeding of the fatigue resistance of the asphalt concrete. Temperature-induced transverse cracking can also result from low temperatures' causing shrinkage stresses greater than the tensile strength of the pavement material, a mechanism termed "low temperature cracking."

Thermal cracking is predicted using various computer-based

models. The earlier models are based on empirical or statistical relationships that relate cracking to asphalt specification data and environmental parameters. More recently, under the assumption that an asphalt concrete pavement is linear-elastic, Lytton used linear elastic fracture mechanics theory to develop a computer-based model to predict thermal fatigue cracking (1). As shown by Anderson et al., neither the earlier statistical-empirical models nor the more recent fracture mechanics model can be used reliably to relate fundamental asphalt or mixture properties to the incidence of thermal cracking (2). One of the reasons the fracture mechanics model is not realistic is that it is based on the assumption that asphalt concrete pavement behaves in a linear elastic manner. Numerous other studies have established that asphalt concrete shows viscoelastic and viscoplastic behavior. Another shortcoming of the existing fracture-mechanics-based model is that fracture parameters used in this model are determined from statistical regression equations rather than from an incorporation of fundamental asphalt or mixture fracture properties.

Until recently only linear elastic fracture mechanics was used to study the fracture characteristics of asphalt concretes (3-10). Little and Mahboub (11) studied the fracture mechanics properties of first-generation, plasticized sulfur binders. Notched beams under three-point loading were tested to determine the critical value of J integral, J_{1c} . They recommend using J_{1c} as a fracture mechanics characterization parameter for sulfur binders. Abdulshafi and Majidzadeh (12) have also applied J_{1c} criteria to asphaltic mixtures. They used Marshall-type disk samples instead of three-point-bend samples based on the ASTM standard (13). The Marshall-type samples were notched at one diametral extremity and loaded on the other extremity in a way similar to indirect tensile strength testing. The notched end had simple supports on both sides of the notch.

In this investigation a procedure to determine J_{1c} for asphalt concrete using a three-point-bend specimen is established. Trends showing a correlation between J_{1c} and asphalt cement routine test data are established; finally, J_{1c} approach and K_{1c} approach were compared to show that J_{1c} is a more realistic fracture characterization approach.

MATERIALS

A single asphalt concrete gradation and aggregate source meeting the requirements of the PennDOT ID-2 specification was used in this study. Mix design parameters are given elsewhere (2). Crushed limestone from a local quarry was used.

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Twelve asphalt cements ranging from AC-5 to AC-20 (soft to hard), representing nine different sources, were used to make a series of mixes. These asphalt cements were selected from varieties used in different parts of the country and include those known to show good as well as poor resistance to thermal cracking. This selection process is explained in more detail elsewhere (2).

Because of their number and variety of grade and source, the asphalts in this study were divided into three groups for convenience in providing good contrasts: Group A, consisting of asphalt source numbers 2, 4, 5, and 7; Group B, numbers 1, 8, 11, and 12; and Group C, numbers 13, 14, 16, and 17. This arrangement (Table 1) gave the greatest contrast with respect to grade and source among all the groups, as well as within a particular group. Group A contained asphalts from two different sources and two grades, AC-5 and AC-20, from each of the two sources. Group B contained four different sources and three different grades. Group C contained four asphalts used in Canada; two of these asphalts are from the same source but with different refining techniques.

SPECIMEN

Beam specimens 3 in. by 3 in. by 16 in. were compacted with a Cox model CS-1000 kneading compactor. The compaction procedure followed ASTM D3202-83 and ASTM D1561(25) except for the kneading pressure, and the number of tamping blows was modified to obtain air voids of between 4.5 to 5.5 percent. Compaction trials showed that 48 tamping blows, equivalent to approximately three complete passes, at a foot pressure of 90 lb/sq in. followed by 48 blows at 126 lb/sq in. followed by 48 blows at 250 lb/sq in. gave air voids of between 4 and 6 percent. The specimens were compacted using two lifts 1.5 in. thick, and the compaction sequence was followed for each lift. An additional 48 blows were given to the second lift at 350 lb/sq in. foot pressure. Immediately after compaction, the specimen was subjected to a leveling load, which produced a pressure of 400 lb/sq in., using a leveling bar. The static load was applied by Forney model CA-103 compression tester. The compaction sequence used for compacting beams is given in more detail in elsewhere (2).

TESTING APPARATUS AND EXPERIMENTAL PROCEDURE

Apparatus

The loading frame used to conduct the fracture test is shown in Figure 1. Loading of the specimens was accomplished with an MTS model 810-14.2 closed-loop electrohydraulic testing machine. Loads were measured with a Lebow model 3169 load cell mounted between the hydraulic actuator and the specimen loading disk. Signal conditioning equipment for the load cell is contained within the MTS testing machine.

The deformation measuring equipment consisted of two RDP model d5/100 linear variable differential transducers (LVDTs) each supported by a rod fastened to the base plate. The tests were performed at and below 60°F (16°C), and there was no permanent deformation observed under the loading strip. Therefore, deformation measurements were made on the underside of the beam by attaching aluminium strips and referencing the LVDTs to the strips. The deformation measurement arrangement is shown in Figure 2. Two LVDTs were used, one on each side of the beam, to obtain an accurate measurement of deformation. Two Data Translation model



FIGURE 1 Loading frame.

Group Number	Asphalt	Grade (Source	Crude
A	2	AC-5	Venezuela Crude
A	4	AC-20	Venezuela Crude
А	5	AC - 5	Cosden, Texas
А	7	AC-20	Cosden, Texas
В	1	85/100	Conoco, Montana
В	8	AR-4000) Edgington, CA
В	11	AC-20	Diamond Shamrock, OK
В	12	AR-4000) Santa Maria, CA
С	13	200/300) Cold Lake, Canada
С	14	150/200) Cold Lake, Canada
С	16	AC-8	Red Water, Canada
С	17	AC - 8	Red Water, Canada (Oxidized)

TABLE 1 ASPHALT GROUPS ACCORDING TO GRADE AND SOURCE



FIGURE 2 Deflection measurement arrangement.

3100 LVDT conditioners were used to provide signal excitation and conditioning for the LVDTs.

The load and deformation signals were sampled with a Data Translation Model DT-2801A analog-to-digital I/O board mounted in an IBM PC. ASYST, a scientific programming environment, was used as an interface between the A/D board and the computer. An ASYST program was written for online data acquisition and analysis (14).

Experimental Procedure

There is no standard procedure for determining the fracture mechanics properties of notched asphalt concrete beams. Therefore, the following procedures and recommendations were adopted for this investigation:

1. Because no standard exists for asphalt concrete, the portland cement concrete standard ASTM C31-69 was used as a guide. A maximum aggregate size of 3/8 in. (9.15 mm) was selected for this study. Note that the maximum aggregate size (3/8 in.) is one-third of the maximum unfractured ligament size of 1.5 in.

2. ASTM E813-81, the standard for J_{1c} testing for metals, was used to select the span-to-depth ratio for beam specimens used in this study. Consequently, an overall length of 16 in. (406 mm) and a span length of 12 in. (305 mm), giving a ratio of 4, were adopted.

3. To limit extensive plastic flow in the unfractured ligament in the notched beams, the notch-length-to-beam-depth ratio (a/d) was chosen at between 0.5 and 0.7. This ratio was determined from laboratory trial tests and is based on the ASTM E813-81 requirement. Three notch depths (a) were used: 1.5 in., 1.7 in., and 1.9 in.

The fracture tests were conducted at five temperatures: 60°F (16°C), 40°F (4°C), 25°F (-4°C), 10°F (-12°C), and -5°F $(-20^{\circ}C)$. Prior to being tested notches were sawed at the midpoint on the underside of each beam (see Figure 1). A small diamond-blade saw left a very smooth surface and a crack width of approximately 1/8 in. (3 mm). The notch depths (a) were such that the notch-depth-to-beam-depth ratio (a/d)varied between 0.5 and 0.7. Typical notch depths used were 1.5 in., 1.7 in., and 1.9 in. The tip of the notch was sharpened using a 24-tooth/in. (1-tooth/mm) hacksaw blade just before testing. A typical test procedure for a specimen proceeded as follows. The specimen, MTS loading frame, and the threepoint loading frame were all kept in an environment chamber at the test temperature for 18-24 hr; two 1.2-in. half-rounds were glued to the underside of the beam at both span ends; the simply supported beam was then placed on the threepoint-bend loading frame and monotonically loaded (load control) to failure at a loading rate of 1.5 lb/sec; the load and deflection measurements were automatically recorded by the data acquisition program.

THEORETICAL CONSIDERATIONS

Among the various parameters used to characterize elasticplastic fracture, the J integral proposed by Rice in 1968 has found wide acceptance (15). The J integral is defined as a path-independent contour integral representing a nonlinear elastic energy release rate. Under certain restrictions J integral can be used as an elastic-plastic energy release rate. Two different methods were used to evaluate the J integral in this investigation. In the first method, called Method 1, J_{1c} was obtained by computing the area under the load displacement curves (i.e., the total energy to failure, U_T) for different transverse crack lengths. These cracks were introduced at the midsection of the beam specimen. The following formula was used to compute J_{1c} from the total energy to failure and the corresponding crack length.

$$J_{1c} = -(1/b) (dU_T/da)$$
(1)

where

b = beam width (mm) and

 U_T = total strain energy, that is, area under load displacement plot (lb-in.) (J).

Rice (15) suggested a method for the evaluation of J integral that requires only one specimen and one crack length. The formula based on this method is (16)

$$J_{1c} = \eta U_{Tb}(d-a) \tag{2}$$

Sumpter and Turner have discussed Equation 2 in detail (16). A detailed derivation of Equation 2 is also given elsewhere (17). For length-to-depth ratios equal to 4 and notch-to-depth ratios between 0.5 and 0.7, the constant, η , in Equation 2 is equal to 2 (16). Therefore, Equation 2 can be stated as

$$J_{1c} = 2U_{T/b}(d-a)$$
(3)

Equation 3 is valid only for the length-to-depth and notchto-depth ratios just given. Because the specimens used in this study satisfied these requirements, Equation 3 was used to calculate J_{1c} by Method 2.

Because at low temperatures asphalt concretes display almost elastic behavior, the fracture criterion is based on the linear elastic fracture mechanics (LEFM). It is generally specified in terms of the fracture toughness K_{1c} . In this investigation the K_{1c} values were computed using two equations: the Winnie and Wundt equation and the stress analysis equation.

Winnie and Wundt developed the following equation based on Griffith theory to determine the plane strain critical stress intensity factor, K_{1c} (18).

$$K_{1c} = 0.521 \ \sigma_c^2 B \tag{4}$$

where

- K_{1c} = critical stress intensity factor for plane strain conditions [lb/sq in. ($\sqrt{$ in.) (Pa- \sqrt{m})];
- σ_c^2 = applied critical remote bending stress (lb/sq in.) (Pa); B = (d-a) (mm);
- d = depth of the beam (mm); and
- $a = \operatorname{crack}$ (notch) depth (mm).

Ewalds and Wanhill give the following formula derived using the collocation method for determining fracture toughness, K_{1c} (19):

$$K_{1c} = \left[\frac{Pl}{b}(d)^{3/2} \right] f(a/d) \tag{5}$$

where

- P =maximum load on the load deflection curve [lb (N)];
- l = span of the beam (mm);
- b = width of the beam (mm);
- d = depth of the beam (mm);
- f(a/d) = function of crack geometry = A/B;
- $A = 3(a/d)^{1/2} [1.99 (a/d) (1-a/d) (2.15-3.93(a/d) + 2.7((a/d)^2)]; \text{ and}$
 - $B = 2(1 + 2(a/d)) (1-a/d)^{3/2}.$

EXPERIMENTAL RESULTS AND INTERPRETATION

Critical values of the J integral in plane strain, J_{1c} , were determined from laboratory load deflection data as follows:

• Total energy, U_T , under the load deflection diagram was computed by summing the area under the curve up to the point of failure, as shown in Figure 3. Failure was defined at the maximum load. The area was computed using the acquired data and the trapezoidal rule of summation of area under a curve.

• $U_{T/b}$, the total energy per unit thickness, was then plotted against notch depth, as shown in Figure 4. The slope, $(1/b)(dU_T/da)$, was obtained through regression. This procedure was repeated for each test temperature and mixture.

• The critical J integral, J_{1c} , was then determined for each test temperature from the slope of the energy versus crack length plots (Figure 4) using the following equation:

$$J_{1c} = -(1/b)(dU_T/da)$$
(6)

The results of the J_{1c} calculations for the individual mixes are summarized in Table 2. Figures 5 through 7 show plots of J_{1c} versus temperature.

DISCUSSION OF RESULTS

 J_{1c} values were calculated using Equations 1 and 3. To compare the values of J_{1c} obtained from Methods 1 and 2 (see Table 2), a statistical regression was conducted. For most of the asphalts used in the study, R^2 was greater than 0.8, which shows that J_{1c} values obtained from the two methods are well correlated. A close examination of Table 2 indicates that Method 2 gives more consistent results. Method 1 requires several specimens to determine J_{1c} . As a result, Method 1 involves variabilities between one sample and another, producing scatter in the data.



FIGURE 3 Load-deflection diagram.



FIGURE 4 Energy per unit thickness versus notch (crack) length.



FIGURE 5 J_{1c} versus temperature, Group A.

J_{1c} Versus Asphalt Grade and Source

 J_{1c} values were plotted versus temperature using the same grouping used earlier (see Figures 5 through 7). These plots indicate a shift in position on the temperature axis according to asphalt grade. All of the mixes tend to approach values between 0.3 to 1.0 lb in./sq in. at low temperatures (at and below -5° F (-21° C); at the higher temperature the softer asphalts generally showed larger J_{1c} values. It was also observed that asphalt cements of the same grade obtained from differ-

ent sources showed differences in J_{1c} . In Figure 5, for example, asphalt numbers 2 and 5 are AC-5 viscosity grade from two different sources, and asphalt numbers 4 and 7 are of AC-20 viscosity grade from the same sources as the AC-5s. Note the difference in curvatures between asphalt numbers 2 and 5 and asphalt numbers 4 and 7, which are due to the difference in source. Also note the difference in curvature between asphalt numbers 2 and 4 because of a difference in viscosity grades and a similar difference between asphalt numbers 5 and 7. Differences are observed between and within the three groups,



FIGURE 6 J_{1c} versus temperature, Group B.



FIGURE 7 J_{1c} versus temperature, Group C.

indicating that the asphalt cements show sensitivity to fracture behavior with regard to both grade and source.

A rather interesting difference is observed in Figure 7 with mixes made with asphalt numbers 16 and 17. Both are waxy asphalts of the same grade (medium AC-8) and same source, except that asphalt number 17 was oxidized during its manufacture. However, asphalt number 16 (AC-8) shows behavior similar to that of asphalt number 7 in Figure 5, which is a hard asphalt (AC-20); whereas asphalt number 17 (oxidized AC-8) shows behavior similar to that of asphalt number 5 in Figure 5, which is a soft asphalt (AC-5). Rheological results (2) also show a similar difference between asphalts 16 and 17. The reason for the difference in behavior is that the wax in asphalt 16 crystallizes at temperatures below 60°F (16°C), giving it pseudohardness. In summary, results indicate that J_{1c} is asphalt grade and source sensitive.

	J1c, lb-in/in ²										
Asphalt Number	Method	hod -5 ^o F		10 ⁰ F	15 ^o f	25 ⁰ F	40 ⁰ F	60 ⁰ F	R ²		
1	Method 1	,66			1.50		1.67	7.67			
	Method 2	.67			1.11		1.47	8.31	0.98		
2	Method 1	.66		.81		.49	8.64				
	Method 2	.76		1.08		1.11	11.23		0.85		
4	Method 1	.45		1.22		1.80	1.90				
	Method 2	.54		.62		1.10	1.54		0.75		
5	Method 1	.64		1.14		1.56	6.20				
	Method 2	.55		.84		1.16	4.85		0.9		
7	Method 1	.82			.40		.90	5.85			
	Method 2	.58			.65		1.30	7.20	0.89		
8	Method 1	.28	.39				1.45	3.57			
	Method 2	.29	.38				1.10	4.56	0.84		
11	Method 1	.10		N/A		1.51	.81				
	Method 2	.49		.36		1.15	.99				
12	Method 1	.20		.21		.76	2.09				
	Method 2	.93		.90		1.68	7.47		0.09		
13	Method 1	.13	.66			5.71	7.74				
	Method 2	.85	1.26			5.10	5.79		0.88		
14	Method 1	.20		2.24		1.77	2.58				
	Method 2	.90		1.00		1.34	9.40		0.06		
16	Method 1	.28			.95		1.10	8.58			
	Method 2	.39			.69		1.36	8.05	0.99		
17	Method 1	.15		1.37		1.91	6.39				
0.00	Method 2	.55		1.05		1.22	4.10		0.73		

TABLE 2	SUMMA	RY OF	J_{1c} DATA
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Comparison of J_{1c} Transition Temperature to Other Variables

Because J_{1c} versus temperature curves showed a positional relationship between the different grades and sources, a quantitative method for determining their relative position on the temperature axis was developed. This position was characterized by a temperature at which the J_{1c} versus temperature curve became asymptotic; hereafter, it is called the transition temperature (Figure 8). A transition temperature can be defined as a temperature at which asphalt concrete shows a transition from viscoelastic behavior to elastic behavior. To determine the transition temperature, a straight line was drawn parallel to the temperature axis, the x-axis, and asymptotic to the J_{1c} versus temperature plot. A tangent was then drawn from the largest measured J_{1c} value on the same plot to intersect the parallel straight line drawn earlier. The temperature value at the point of intersection gave the estimated transition temperature. Finally, the transition temperature was expressed as a ranking, with the highest ranking indicating the highest sensitivity to cracking.

The J_{1c} transition temperatures for asphalt concrete are given in Table 3 along with temperature shifts that were obtained from tensile strength and static modulus results, Fraass brittle point temperature, and the temperature where the asphalt penetration is 1.2, $T_{pen1.2}$. The latter temperatures are for neat asphalt coments and were obtained elsewhere (2). Because each of these temperatures was obtained under very different test conditions (rate of loading), it is not surprising that they are not equal for a particular asphalt. However, they should correlate with each other if they are measures of the same property, in this case, the sensitivity to cracking. To verify the levels of correlation between the J_{1c} transition temperatures and the other variables, the temperature shift for tensile strength and modulus are plotted versus the J_{1c} transition temperatures in Figure 9, where the R^2 values are 0.76 and 0.82, respectively. In Figure 10 the Fraass brittle point temperature and $T_{pen1,2}$ are plotted versus the J_{1c} transition temperature, yielding R^2 values of 0.60 and 0.75, respectively. These R^2 values indicate that the J_{1e} transition temperature is correlated with other characteristic temperatures, although the correlation is not strong enough to warrant use of the J_{1c}



FIGURE 8 Procedure for determining J_{1c} transition temperature.

Asphalt No.		1,	Temp Shi	erature ft, °F	Fraass	Temperature	Tempera	ture, Tg
	Source	Transition Temperature F	Tensile Strength	Static Modulus	Point Temperature °F	at Penetration 1.2, °F	From DSC °F	From DSC °F
1	A	43	12	14	12.8	15.5	-27	55.4
2 4	B B	29 40	-4 8	-18 8	4.3 6.8	2.0 14.2	-18 -27	42.4 46.0
5 7	c c	29 45	0 15	5 18	0.1 21.7	12.0 21.1	-34 -25	50.6 48.3
8	D	43	19	14	23.9	24.2	18	68.3
11	E	>40	4	13	4.6	16.6	-28	44.7
12	F	26	0	-9	7.9	-5.6	-27	42.3
13 14	G G	9 31	-14 1	-26 -4	-5.8 -0.2	-6.6 7.3	-17 -25	42.0 42.3
16 17	I I	49 32	5 -1	11 -3	10.0 3.2	21.1 -2.6	-26 -40	55.0 37.9
intercept slope, bj coefficie determi R ²	i, b _o ent of ination,		-21 0.72 0.76	-38 1.13 0.82	-13 0.62 0.60	-20 0.85 0.75	-28 0.15 0.01	33 0.46 0.36

TABLE 3 J_{1c} TRANSITION TEMPERATURE AND OTHER TRANSITION TEMPERATURES

transition temperature as a surrogate for other variables. The J_{1c} transition temperature was also compared to the DSC and DMA glass transition temperatures (Figure 11), and the resulting correlations were very poor, giving R^2 values of 0.01 and 0.36, respectively. Because of the poor correlation, regression lines are not shown in Figure 11. No specific explanation can be given for the poor correlation, especially with the DSC data, except that the testing conditions differ greatly for the DSC and DMA measurements.

A comparison of the ranking of other characteristic temperatures versus the ranking of the J_{1c} transition temperature



FIGURE 9 Plot of temperature shift versus J_{1c} transition temperature, tensile strength, and static modulus.

was made by summing the absolute values of the difference between the individual ranking for the J_{1c} transition temperatures and the respective rankings for each of the other temperatures. The result, along with the rankings, is shown in Table 4. The result shows the same general trends evidenced with the temperature correlations given before. However, a close examination of the data in Table 4 indicates that the methods are not direct surrogates of each other.

The correlations between the J_{1c} transition temperatures and the mixture temperature shifts, $T_{pen1,2}$, and the Fraass brittle point temperature do verify the dependency of J integral on asphalt source and grade and warrant further development of J integral as a material fracture characterization variable and its development as a tool in fracture analyses. Successful use of the J integral has also been reported by Little for use with sulfur-modified asphalt concrete mixes (3). Further study is needed to refine the measurement of J_{1c} and to integrate the J integral into a computer model. The use of J_{1c} and any associated computer model must be verified with a full-scale research study in the field.

Table 5 presents fracture toughness, K_{1c} , values for all mixes as determined with Equations 4 and 5; they are identified as Method 1 and Method 2, respectively. A quick examination of the data in Table 5 will show that the K_{1c} values determined from Equations 4 and 5 are very similar. Therefore, K_{1c} calculated with Winnie-Wundt formula (Equation 1, Method 1 in Table 5) was arbitrarily chosen for use in the comparisons that follow (Figure 12). Because K_{1c} versus temperature for all asphalts showed a straight-line relationship, only a typical Figure 12 is presented. Figures for other asphalts are given elsewhere (2). From Figure 12 it appears that K_{1c} varies in a linear fashion with temperature. Although a linear relationship between K_{1c} and temperature may be expected for brittle materials, a different relationship for hot-mix asphalt concrete is expected as asphalt concrete is not brittle at all temperatures.



FIGURE 10 Plot of Fraass temperature and $T_{\text{PEN1.2}}$ versus J_{1c} transition temperature.



J1C TRANSITION TEMPERATURE, OF

FIGURE 11 Plot of Tg determined by DSC and by DMA versus J_{1c} transition temperature.

TABLE 4 RANKINGS OBTAINED FROM $J_{\rm lc}$ TRANSITION TEMPERATURE AND OTHER TRANSITION TEMPERATURES

Asphalt No.	Source	J _{1c} Transi	; ition	Tens Stren Shi	ile gth ft	Tens Modu Shi	ile lus ft	Fraa Britt Poir	iss le nt	T _{pen1}	.2	T _g from DS		T _g from DM/	n N
	and Grade	Rank	٥F	Rank	٩F	Rank	٩F	Rank	٩F	Rank	٩F	Rank	٩F	Rank	٥F
1	A: 85/100	8	43	9	12	9	14	9	13	8	16	3	-27	9	55
2 4	B: AC-5 B: AC-20	3 7	29 40	2 8	-4 8	2 7	-18 8	5 6	4 7	4 7	2 14	9 4	-18 -27	2 6	42 46
5 7	C: AC-5 C: AC-20	4 10	29 45	4 10	0 15	6 11	5 18	2 10	0 22	6 9	12 21	2 7	-34 -25	8 7	51 48
8	D: AR-4000	9	43	11	19	10	14	11	24	11	24	11	18	11	68
12	F: AR-4000	2	26	5	0	3	-9	7	8	2	-6	5	-27	3	42
13 14	G: 200/300 G: 150/300	1 5	9 31	1 6	-14 1	1 4	-26 -4	1 3	-6 0	1 5	-7 7	10 8	-17 -25	4 5	42 41
16 17	1: AC-20 1: 85/100	11 6	49 32	7 3	5 -1	8 5	11 -3	8 4	10 3	10 3	21 -3	6 1	-26 -40	10 1	55 38
Sum o ence tempe other	f absolute val in ranking of rature and ran characteristi	ue of dif J _{1c} trans king for c tempera	fer- ition the tures		16		12		20		10		46		22

Comparison of J_{1c} Versus K_{1c} as Fracture Parameter

The critical stress intensity factor, K_{1c} , was derived for linear elastic materials and is based on the assumption that the stress strain relationship of the material being characterized is lin-

ear. In other words, K_{1c} is directly proportional to stress and reflects any changes in stress with regard to temperature, stress rate, or time. Because the stress strain curves are linear, behavior of a linear elastic material can be characterized by stress alone. Therefore, K_{1c} can successfully be applied. On the other hand, for nonlinear or elastic-plastic materials, the

TABLE 5 SUMMARY OF K1c DATA

		K _{1c} , lb-in/sqrt in									
Number	Method ¹	-5 ⁰ F	8 ⁰ F	10 ⁰ F	15 ⁰ F	25 ^o F	40 ⁰ F	60 °F			
1	Method 1	871.5	NA	NA	896.4	NA	711.9	602.8			
	Method 2	763.0	NA	NA	785.7	NA	623.3	527.7			
2	Method 1	863.9	NA	845.3	NA	780.3	793.4	NA			
	Method 2	757.5	NA	740.8	NA	681.7	695.4	NA			
4	Method 1	894.9	NA	720.88	NA	757.1	764.7	NA			
	Method 2	784.9	NA	638.1	NA	662.5	669.8	NA			
5	Method 1	816.1	NA	772.6	NA	821.5	936.8	NA			
	Method 2	714.1	NA	674.9	NA	719.2	821.2	NA			
7	Method 1	792.1	NA	NA	841.02	NA	735.2	790.0			
	Method 2	694.0	NA	NA	736.9	NA	644.3	693.2			
8	Method 1	852.7	883.8	NA	NA	NA	749.0	840.8			
	Method 2	751.7	774.6	NA	NA	NA	655.7	736.6			
11	Method 1	800.6	NA	772.5	NA	827.9	664.5	NA			
	Method 2	700.9	NA	680.0	NA	724.6	579.03	NA			
12	Hethod 1	933.4	NA	851.6	NA	1022.5	867.52	NA			
	Method 2	816.4	NA	747.0	NA	896.5	761.9	NA			
					NA			NA			
13	Method 1	859.31	845.9	NA	NA	1054.5	662.8	NA			
	Method 2	755.0	139.1	NA	NA	824.6	581.5	NA			
1/	Nothed 1	1148 0		8/3 05	NA	9// 1	870 5	NA			
14	Method 2	980 8	NA	737 2	NA	757 8	763 3	NA			
	nethod L	/00.0	10	131.6		151.0	105.5	114			
16	Method 1	767.8	NA	NA	732.1	NA	692.6	608.7			
	Method 2	672.6	NA	NA	642.3	NA	607.7	532.9			
17	Method 1	736.775	NA	767.2	NA	771.9	806.4	NA			
	Method 2	648.4	NA	672.6	NA	675.0	705.4	NA			

Note:

Method 1: K1c determined from Winnie and Wundt equation.

Method 2: K_{1c} determined from Stress Analysis equation.

stress strain curves are not linear; therefore, stress alone is not sufficient to characterize their behavior.

 J_{1c} , which is determined on the basis of the strain energy release rate, is related to the variation of stress and strain, not to stress or strain alone. Therefore, J_{1c} is sensitive to both linear or nonlinear material behavior and can be used as a fracture characterization parameter. As a matter of fact, for a material such as asphalt concrete, which shows a transition from nonlinear to linear behavior as the temperature decreases, J_{1c} is a promising fracture characterization parameter.

To summarize, it has been shown that linear elastic fracture mechanics (LEFM) is not as sensitive to changes in asphalt concrete fracture properties as the elastic-plastic fracture mechanics (EPFM). The critical values of J integral, J_{1c} , determined using EPFM, do discriminate between asphalt concrete mixes made using asphalt cements of different grades and

sources. The rankings obtained from J_{1c} transition temperatures are similar to those obtained from other parameters and can be used in a mechanistic analysis. Consequently, this approach warrants further study.

CONCLUSIONS

1. J_{1c} is sensitive to asphalt concrete stiffness, asphalt cement grade, and source. J_{1c} is a promising fracture characterization parameter for asphalt concrete at low temperatures (60°F (16°C) and below.

2. K_{1c} is not sensitive to asphalt concrete variables.

3. Although Equation 1 is a more fundamentally correct method of determining J_{1c} in the laboratory, Equation 3 appears to be a reasonable approximation.



FIGURE 12 Typical plot of K_{1c} versus temperature (Group A).

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Publication of this paper sponsored by Committee on Characteristics of Bituminous Paving Mixtures to Meet Structural Requirements.