Improved Approach to Low Temperature and Fatigue Fracture Performance Grading of Asphalt Cements

Serban Iliuta, Adrian Andriescu, and Simon A. M. Hesp
Department of Chemistry, Queen's University
Kingston, Ontario K7L 3N6

Kai K. Tam
Ministry of Transportation of Ontario
Downsview, Ontario M3M 1J8

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ABSTRACT

Asphalt binders used in three different pavement trials in northern and eastern Ontario have been investigated by using the Bending Beam Rheometer (BBR) and various fracture tests. It was found that binders chemically harden considerably faster in the field than what is currently achieved through laboratory ageing protocols. Regular grading according to the Association of State Highway and Transportation Officials (AASHTO) M320 specification and grading after different conditioning times was performed in order to get an understanding of the physical and steric ageing effects as they occur upon cold storage. All binders were found to show an increase in the stiffness and a decrease in m-value with conditioning time. After only three days of cold storage, the investigated binders lost anywhere from 1.7 to 9.5°C from their low-temperature grade depending on binder type and conditioning temperature. It is concluded that chemical, physical and steric ageing, beyond what is currently achieved in the laboratory grading methods, are likely contributors to early distress and that they can explain, to a large extent, vast performance differences between binders of the same AASHTO M320 grade. In addition, large variations in fracture properties provide a further explanation of the observed performance differences.

RÉSUMÉ

Les liants bitumineux utilisés dans trois essais routiers différents au nord et à l'est de l'Ontario ont été étudiés avec le rhéomètre à poutre de flexion et divers essais de rupture. On a trouvé que les liants durcissent beaucoup plus vite en chantier que ce que l'on obtient couramment avec les protocoles de vieillissement en laboratoire. La classification régulière selon la spécification AASHTO M320 (Association of State Highway and Transportation Officials) et la classification après divers temps de conditionnement a été faite afin d'avoir une compréhension des effets physiques et stériques du vieillissement tels qu'ils se produisent lors de l'entreposage à froid. On a trouvé que tous les liants montraient un accroissement de la rigidité et une diminution de la valeur-m avec la durée de conditionnement. Après seulement trois jours d'entreposage à froid, les liants évalués ont perdu entre 1,7 et 9,5°C de leur classe à basse température selon le type de liant et la température de conditionnement. On conclut que le vieillissement chimique, physique et stérique, au-delà de ce qui se produit couramment dans les méthodes de classification en laboratoire, contribue vraisemblablement à la détérioration précoce et qu'il peut expliquer en grande partie les grosses différences de performance entre les liants de même classification AASHTO M320. En outre, les grandes variations dans les propriétés de rupture fournissent une explication supplémentaire aux différences de performance observées.

1.0 INTRODUCTION AND SCOPE

Low-temperature cracking of flexible asphalt pavements is a major form of distress in much of the northern United States and Canada. Premature failures are a major concern due to the high costs of repair and the eventual need for early rehabilitation and reconstruction. The currently used American Association of State Highway and Transportation Officials (AASHTO) M320 asphalt binder grading method has shown, on more than one occasion, its inability to predict the low-temperature and fatigue fracture performance in practice [1-6].

Soon after the United States' Strategic Highway Research Program (SHRP) ended in the early 1990s, it was realized that the binder grading methods developed with straight asphalt may not always be sufficient for the performance prediction of modified and specialty asphalt binders. With respect to the low-temperature grading test development effort, SHRP researchers recognized that "Ideally, it is necessary to determine the fracture mechanics parameters for neat asphalt cement as well as for hot-mix asphalt concrete" [7]; however, due to a lack of time and resources, in the end only the Bending Beam Rheometer (BBR) and the Direct Tension Test (DTT) were developed and no direct comparison was ever made with fracture mechanics–based failure tests.

At about the same time as the final SHRP reports were released, research at Queen's University had started on the fracture mechanics testing of binders in their brittle state [8-10]. This early work recognized that binders of approximately the same SHRP performance grade could show enormous differences in their low-temperature fracture toughness and fracture energy. Furthermore, it was discovered that notched binder specimens sometimes fail in a brittle fashion at much warmer temperatures than unnotched specimens [11]. Hence, since these results were published, some have suggested that these differences in brittle fracture properties could perhaps explain the many anomalous results reported in the literature [12-16]. Recognizing the fact that pavements spend most of their service life in the ductile state, the focus has been broadened by also assessing the ductile fracture properties of the binder [17, 18]. It appears that binders are best tested in both brittle and ductile states to more accurately predict and control fracture performance in the field.

This study provides a more definitive answer to the question of which approach more accurately predicts field performance. To this end, AASHTO M320 and fracture properties of recovered and laboratory-aged binders were compared with the performance of corresponding test sections on Highway 631 near Hearst, Ontario; on Highway 17 in Petawawa, Ontario; and on Highway 655 near Cochrane, Ontario.

2.0 BACKGROUND

2.1 Early Investigations of Failure in Asphalt Binders and Mixtures

Seminal investigations into the failure of asphalt binders and mixtures at Koninklijke-Shell Laboratories in Amsterdam focused on stiffness as the cardinal property in rutting, fatigue, as well as in low-temperature fracture [19-23]. In one of these early papers on rheology and fracture, Heukelom concluded from his experimental data [21]: (1) "That Van der Poel's stiffness concept has provided a valuable means of simplifying the description of, not only rheological, but also fracture properties of asphalt cements and mixes."; (2) "The modulus of asphalt cement is a measure of the rheological condition of the bitumen, on which the fracture properties depend. The effect of temperature and loading time on the fracture properties of road bitumens of various grade and origin is thus condensed in the stiffness as a single parameter."; and (3) "Parameters for the fracture properties of mixes can be separated into the stiffness of the asphalt cement and a 'mix factor' which is independent of the above-mentioned variables, but dependent on the proportion of asphalt cement, grading of the minerals, and compaction of the mix. A further study of these variables can be simplified by determining the value of the mix factor only, so that much superfluous laboratory effort can be saved."

Following these far-reaching suggestions, a large number of researchers including those involved in the SHRP program [7, 23-27] have thus focused their attention on stiffness as a binder specification parameter for failure at low temperatures. It is generally assumed that if the stiffness at a somewhat arbitrary loading time exceeds a limiting value, transverse cracking will occur in the road. However, it is not widely recognized that the correlation made by Heukelom is only valid for unmodified binders. While modifiers were used only sparingly in the late 1960s, today the situation is different in that in some areas nearly half of all binders are modified (air blown, polymer-modified, gelled, "engineered," etc.) [28]. This development, which has slowly evolved over the last 30 to 40 years, was the main reason for the existence of the SHRP program whose ultimate aim is to find better ways to evaluate asphalt materials by using properties that are more accurate and reliable for performance prediction.

2.2 Development of the AASHTO M320 Low-Temperature Binder Specification

During the SHRP program, a large amount of resources were devoted to identifying those properties of asphalt cements that were directly responsible for performance of asphalt pavements and further incorporation of these into a more reliable binder purchase specification.

Researchers involved in SHRP Contract A-002A followed closely the work of Readshaw [26] of British Columbia in their investigation [7]. Analyzing McLeod's [24] results from a survey of existing pavements in three Canadian provinces, Readshaw concluded that, with few exceptions, cracking did not occur if the stiffness of the binder (as obtained from Van der Poel's nomograph) did not exceed 200 MPa at a loading time of two hours. However, several issues that were not considered may have confounded this study. First, no field temperature data were reported but a single value of –40°C was used for calculating the stiffness with the nomograph, therefore making it difficult to assess the pavement performance relative to field conditions. Second, for the actual binders that were investigated, the majority were recovered from mix samples. However, extraction of binders is not always a simple task, and it is an unknown factor what effect, if any, the extraction procedure had on the properties measured and therefore on the conclusions of the study. Notwithstanding these uncertainties, SHRP researchers used the 200 MPa limit reported by Readshaw in their initial low-temperature binder specification. In order to simplify the testing procedure, the time–temperature superposition principle was applied and the stiffness after two hours of loading at the grade temperature was replaced with the stiffness obtained after 60 seconds of loading (S(60)) at a temperature 10°C warmer.

In addition to an upper limit on binder stiffness, the SHRP program also introduced a lower limit on the slope of the creep curve (the so-called m-value) as measured in the BBR. The introduction of the "m-value" was rationalized through its involvement in the development of thermal stresses in a sample. In addition, it would preclude the use of highly oxidized binders with low m-values and known poor field performance [29]. Thus, the specification limits originally proposed under draft specification 7G were S(60) = 200 MPa and m(60) = 0.35.

The issue of what limits should be placed on stiffness and m-value was further addressed by a team working under SHRP Contract A-005A [30]. After comparing the limiting parameters as proposed by

SHRP Contract A-002A with the actual field performance of a large number of general pavement sites, it was concluded that the values were too conservative and new values of S(60) = 300 MPa and m(60) = 0.3 were proposed [30]: “The defined limits on S and m (S < 29,000 psi and m > 0.35) are too restrictive. Based on the data obtained in this study, the following limits are recommended: S > 45,000 psi and m < 0.3.” The authors expressed the opinion that “Further investigations beyond the scope of this project are recommended to verify that S and m are appropriate specification parameters” and “The most one can expect from a binder specification is to identify only those binders that will not perform well regardless of the mixture in which they are used. Conversely, the specification should accept binders that should perform well with a suitable mixture design.”

How good or bad the current specification is can now be determined, since long–term performance data for many of the Canadian Strategic Highway Research Program (C-SHRP) and Long–Term Pavement Performance Program Special Pavement Sites (LTPP SPS-9) test sections (which were constructed to validate the SHRP low-temperature specification) have gradually become available. Sections constructed under C-SHRP and SPS-9 contain well-controlled mixture designs and pavement structures and are therefore, ideally suited for validation of any binder specification.

The choice of a limiting stiffness of 300 MPa after 60 s of loading was further justified by Bahia and Anderson [31] based on their observation that the BBR stiffness was on average 50 percent larger than the estimated stiffness from the Van der Poel nomograph. However, an earlier publication by the same authors shows that differences of up to 400 percent for short loading times (15 s) and up to 800 percent for longer loading times (240 s) were in fact prevalent for the group of just eight unmodified SHRP core asphalts as tested at four different temperatures (see Figure 13 in [32]).

Despite this, based on the discussions in the early 1990s, the S(60) = 300 MPa and m(60) = 0.3 criteria were first approved in a provisional standard under the AASHTO designation MP1 [33] and subsequently as a standard specification under AASHTO designation M320 [34]. The development of the BBR under the SHRP program offered the means of direct measurement of creep stiffness of asphalt binders in a fast, reproducible, and convenient manner. Hence, it has been accepted for specification testing in most of the United States and in eastern Canada. However, based on the above review of the literature, it becomes apparent that the development of the specification contained a significant degree of empiricism. There is no scientific justification for why the m-value limit settled at 0.3 and the S-value limit at 300 MPa and not, for instance, the 200 MPa as proposed by Readshaw [26], or why these properties and not others should be selected for specification testing.

The introduction of an additional parameter, strain at failure, was also proposed by the SHRP A-002A team. They noticed that the equivalence of the strain at failure and the stiffness, as first described by Heukelom [21], is not valid in the case of certain modified binders and hypothesized that this could result in an underestimation of the binder performance. Thus, it was proposed that the limiting stiffness could be increased to 600 MPa, provided the m-value criterion is satisfied and the elongation at break as obtained from DTT is above 1 percent. The criterion has never found widespread use and at this moment is all but abandoned. However, DTT has found a new use in the recently proposed MP1a specification which will be discussed next.

2.3 Development of the AASHTO MP1a Low-Temperature Specification

Bouldin and coworkers [35] more recently proposed a new approach for grading binders according to their performance. The original idea behind the development of MP1a can be traced back to the work on mixtures by Hills and Brien [36] at the Koninklijke/Shell Laboratories in Amsterdam, which was further developed by Hills some years later [23]. Bouldin et al. [35] started from observations that the AASHTO M320 grading system failed to accurately predict the performance of various binders. They hypothesized that such binders (polymer modified, oxidized, and chemically modified) that do not obey the stiffness/strain at failure relationship could be better graded by calculating the thermal stresses developed in a restrained binder beam and comparing them with the tensile strength of the binder obtained in DTT. In this way, materials that were over-predicted by the present specification should be identified and avoided [37]. The stiffness and relaxation data from the BBR are used to calculate the thermal stress build up in the binder, which is then compared with the tensile strength as measured in DTT, in order to get a critical temperature at which the binder would be expected to fail. The so-called pavement constant is introduced in the method in an attempt to scale the thermal stress developed in the binder to that in the asphalt pavement. The authors used the cracking onset temperatures from the C-SHRP test road in Lamont, Alberta, to calibrate their model and to arrive at a fitting pavement constant of 24 [35]. However, it should be noted that at other times the pavement constant was adjusted to 16 or 18 to bring MP1a in line with M320 data [37].

In a study on two unmodified and six modified asphalt materials, it was found that this pavement constant could vary between 3.4 and 16.7 [38]. This was revealed through comparing the true stress build up in a restrained cooling test on the binders with that in the corresponding mixtures. Such a large variation for only eight systems in what is supposed to be a constant naturally casts much doubt on the validity of this concept. The approach has nevertheless been adopted as a provisional standard under the MP1a designation by AASHTO [39].

It should be noted that this approach only attempts to predict the onset of cracking and that it does not allow the user to say much about the severity of low-temperature cracking over longer periods of time. As will be discussed later, the philosophies embodied in both AASHTO M320 and MP1a have the limitation that they are empirical rather than mechanistic. The actual mechanism of low-temperature cracking involves crack initiation and propagation phases in both the ductile and brittle regimes that are not easily captured by a single limiting or critical temperature. Furthermore, additional factors such as physical ageing, which is believed to play an important role in low-temperature failure, were omitted. Hence, a new approach is needed to come to a more comprehensive assessment of a binder’s ability to resist low-temperature and fatigue failure. Before the current efforts to develop such method are discussed, it is worthwhile to first review performance data from the Lamont, Alberta and Hearst, Ontario test roads.

2.4 Recent Performance Data for C-SHRP Test Roads

C-SHRP was originally designed to construct and monitor three test roads in Canada to validate the Canadian General Standards Board (CGSB) asphalt binder specification but was later also used with regards to the validation of Superpave™ (Superpave) low-temperature specification [1, 4, 5, 35, 37, 40]. The primary test road was constructed in 1991 on Secondary Highway 637:02, east of Lamont, Alberta, some 80 km northeast of Edmonton. The satellite test road in Hearst was also constructed in 1991 and is located on the first two kilometres of Highway 631, south of the Highway 11 junction some 65 km west of Hearst, Ontario. Publications with 12-year performance data for both sites have appeared recently and are briefly reviewed next [6, 40]. A second satellite test road was constructed in Sherbrooke, Quebec, but it was reconstructed after a relatively short life and hence, will not be discussed here.

For a detailed description of the asphalt binders used in the test sections for both Lamont and Hearst, the pavement and subgrade structure, and the instrumentation for performance monitoring, the reader is referred to the original construction report by EBA Engineering Consultants Ltd. [41] and the final project report by Anderson [5]. This paper only provides the pertinent asphalt binder data and discusses how these relate to transverse cracking performance results for both sites.

2.4.1 Lamont Test Road Performance

The Lamont test road contained seven test sections ranging from 417 to 500 m in length EBA [41]. The different binders used in each section and their respective properties are listed in Table 1. The cracking onset temperatures for the seven sections as reported by Anderson et al. [5] are provided in Figure 1, while the 12-year cracking severity as correlated with various binder properties is provided in Figure 2. The data provide several interesting points of discussion.

First, the cracking appears to start in all sections in a narrow range between –30°C and –32°C at 12 mm depth. The exception is Section 4 for which it was suspected that the thermocouple was incorrectly calibrated. This appears counter-intuitive given the fact that the binders that cracked varied by more than 9°C in their SHRP performance grades. Possible reasons for this could be related to flaws in the design of the crack detection system, with perhaps the aluminum foil breaking in all sections at about the same temperature. An alternative explanation could relate this to a failure at the binder-aggregate interface, which could have occurred at about the same temperature in all sections causing transverse cracking (at about the same temperature). However, this is only speculation, and the exact reason for this observation is probably difficult if not impossible to determine at this moment.

Table 1. Pertinent Binder Properties for Lamont Test Road Sections

Section	Grade Temperature, °C		Penetration, dmm	PVN_{60}	CGSB Grade	Crude Sources
	M320	MP1a				
1	-26.1	-34.5	100	-0.81	80-100B	Boundary Lake/Air Blown
2	-29.9	-24.1	150	-0.67	150-200B	Montana/Bow River
3	-38.4	-39.3	333	0.05	300-400A	Cold Lake
4	-24.9	-24.6	93	-1.19	80-100C	Redwater/Gulf
5	-33.8	-31.4	88	0.27	80-100A	Lloydminster/Air Blown
6	-33.8	-35.1	176	0.01	150-200A	Lloydminster
7	-35.8	-35.3	241	-0.08	200-300A	Edmonton

Reproduced from: EBA [41], Robertson [1], and Gavin et al. [40]. Note: M320 is AASHTO M320 specification method; MP1a is AASHTO MP1a specification method; penetration was measured at 25°C and penetration-viscosity number: PVN_{60} was measured with absolute viscosity at 60°C.

Second, the correlation between the limiting temperatures as measured by BBR and DTT (M320 and MP1a) and the 12-year cracking severity appears to be reasonable, but the M320 approach still misses the poor performance of the air-blown binder of Section 5 and the B grade binder from Section 2. The MP1a method misses the poor performance of the two air-blown binders in Sections 1 and 5, which are the very binders which it was designed to detect and exclude [35]. A third observation relates to the predictive ability of the penetration grades at 25°C and 5°C. These appear to predict the cracking severity with similar if not better accuracy as the AASHTO M320 and MP1a methods with correlation coefficients of 0.62 (Pen at 25°C) and 0.81 (Pen at 5°C) compared to 0.61 (M320) and 0.62 (MP1a).

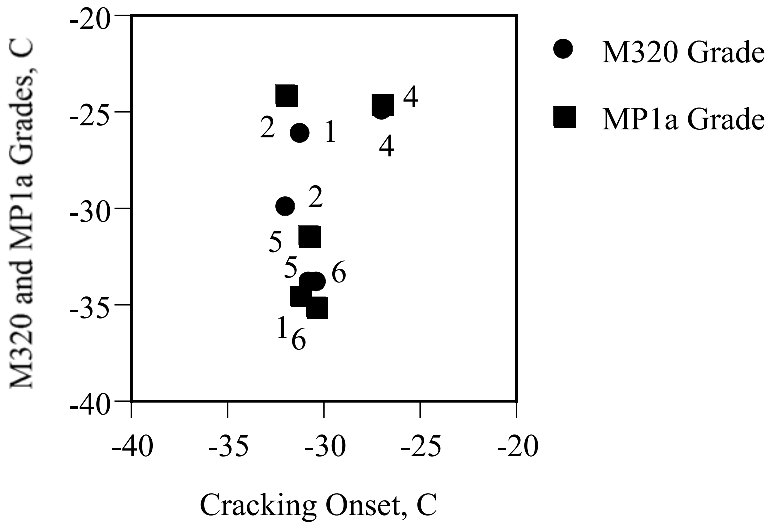

Figure 1. Comparison Between AASHTO M320 and MP1a Grade Temperatures and Cracking Onset Temperatures as Measured at 12-mm Depth

Note: Labels at each symbol relate to test sections for which cracking onset was determined. AASHTO M320 and MP1a temperatures as reported by Robertson [1] and Bouldin et al. [35] and cracking onset temperatures as reported by Anderson et al. [5] and Gavin et al. [40].

Gavin et al. [40] remark in the conclusions of their paper that both the CGSB and the Superpave asphalt cement specifications do a reasonable job at performance prediction. However, the comparisons in Figure 2 show that improvement is needed in their ability to distinguish the performance of oxidized binders from straight run and other modified binders. A close scrutiny of the correlation presented in Figure 2 shows that binders of nearly the same M320 grade can show differences in cracking severity of a factor of three (compare Sections 5 and 6). For the MP1a specification, binders of nearly the same grade can show differences of up to a factor of ten (compare Sections 6 and 7) or 35 (compare Sections 1 and 7).

It should be noted that the Lamont test road has and continues to service a relatively low level of traffic for a road that is designed at 100 mm thickness. In 1990, the Average Annual Daily Traffic (AADT) was approximately 260 or 9 ESAL. In 2000, this had increased to approximately 980 with 13.6 percent truck traffic for a total 89.4 ESAL per day. Hence, the fatigue fracture properties of the binders are probably less important for predicting the fracture performance except where these are extremely poor such as what may have been the case for the oxidized binders.

Before the details of an improved approach to binder grading are discussed, a brief review of the Hearst test road performance data and the physical ageing phenomenon is given.

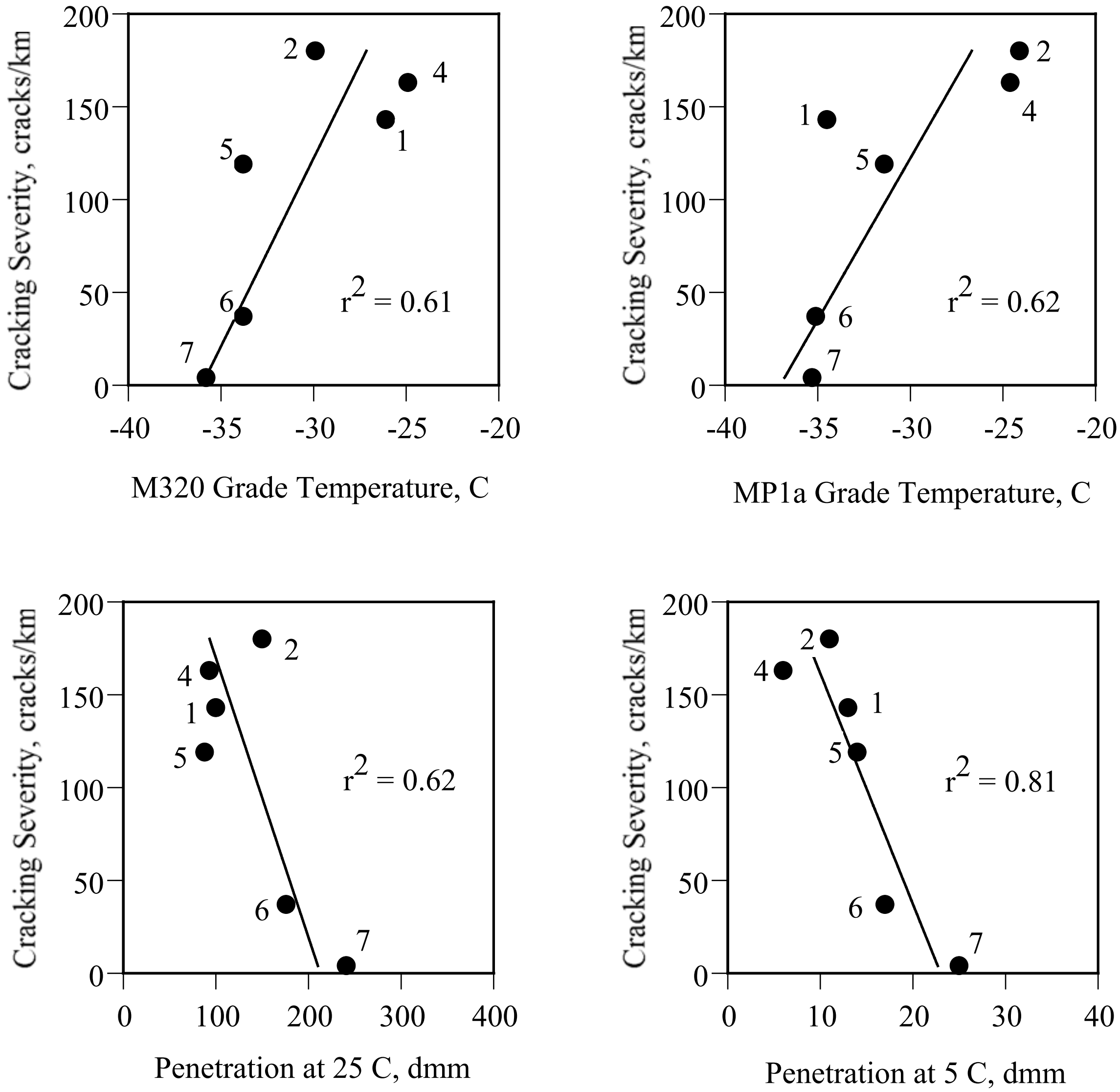

Figure 2. Correlations Between Cracking Severity and AASHTO M320, MP1a and Penetration Values at 25°C and 5°C for Lamont, Alberta Test Sections

Note: Labels at each symbol relate to test section number. Binder properties as reported by EBA [41] and Gavin et al. [40], severity as reported by Gavin et al. [40].

2.4.2 Hearst Test Road Performance

A review of the performance of all existing test roads in Ontario including the C-SHRP site west of Hearst was recently published by Iliuta et al. [6]. The site had last been visited in 1997 until it was revisited in 2003. A detailed discussion is provided in [6] of which only the most important observations are reviewed herein. Later in this paper an attempt is made to explain, with an improved approach to binder grading, the large performance differences for the sections near Hearst.

Highway 631 is located 63 km west of Hearst in northern Ontario and was part of the C-SHRP program. Four sections, containing three different binders of two grades, were constructed in 1991 on new granular base. Section lengths varied between 360 and 590 m. Three sections had a design thickness of 50 mm while a fourth was designed at 100 mm thick. In 1990, the AADT on these sections was 300 with 25 percent truck traffic. In 1999, AADT had increased to 600 of which now 29 percent was truck traffic. Two unmodified binders were graded as a PG 52-33.1 and 52-33.2, respectively, while the third was oxidized and graded at PG 46-36.8. The LTPPBIND® grade required for the location is a PG 52-40 at 98 percent confidence, which reduces to a PG 46-34 by accepting 50 percent confidence. Hence, at –33°C it is likely that the pavement will experience what are supposed to be damaging temperatures regularly. The three binders used for this trial were no longer available but data from EBA [41], Robertson [1], and Roque and coworkers [42] are listed in Table 2. For details on the design and early performance of this test road refer to Robertson [1] and Anderson et al. [5].

Table 2. Pertinent Binder Properties for Hearst Test Road Sections

Section	CGSB Grade	M320 Grade	Penetration, dmm		BBR Data for Recovered Binder		
			5°C	25°C	T, °C	S(MPa)	m-value
631-1	150-200A	-33.1	13	154	-18	175	0.37
631-2	150-200A	-33.2	14	160	-18	104	0.39
631-3	200-300B	-36.8	28	224	-18	133	0.36
631-4	150-200A	-33.1	13	154	-18	-	-

Reproduced from : EBA [41], Roque et al. [42], and Robertson [1]. Note: CGSB grades are as reported by EBA by using the absolute viscosity at 60°C.

The cracking severity data for 1992, 1997, and 2003 are given in Figure 3. These data are interesting in several respects. First, binders in Sections 1 and 2, graded according to CGSB, AASHTO M320 and penetration test methods at a nearly identical level of performance, show enormous transverse cracking differences after 12 years in service. Section 1 has more than ten times as many cracks as Section 2. If crack length were to be used as a measure of severity the difference is nearly a factor of 20.

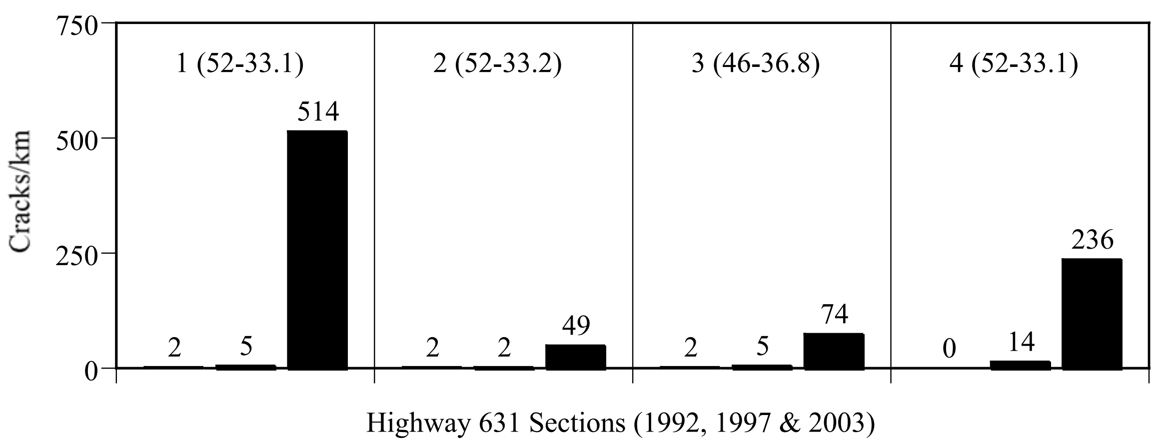

Figure 3. Cracking History for the C-SHRP Test Road on Highway 631 near Hearst, Ontario
Note: SHRP performance grades are given in brackets after the section number and calculated from the data presented by Robertson [1].

The data also show that the oxidized binder in Section 3 with an M320 performance grade that is 3.6°C lower than the one that is used in Section 2 yields 50 percent more cracks after 12 years in service. Section 4 has double the thickness of Section 1 but is made with the same binder and shows only 46 percent of the cracking severity, suggesting that fatigue is an important factor in transverse cracking.

The results from Hearst show, once more, that there is a need for improvement. It is believed that these differences are due to the fact that the current specifications fail to relate to the events that precede and occur during transverse cracking in the pavement. The detrimental effects of steric and physical ageing can be large and are currently ignored in the M320 and MP1a specifications. Furthermore, the effects of large variations in brittle and ductile fracture energies can also have an impact on how well a binder test method can predict the fracture performance in the field. Later in this paper the large performance differences for the sections near Hearst will be explained through variations in chemical, steric and physical ageing and fracture properties.

2.5 Physical and Steric Ageing in Asphalt Binders During Cold Storage

When asphalt binders are rapidly cooled from the molten state and stored at low temperatures, the non-equilibrium position of the molecules will drive the system to undergo internal changes leading to a slow deterioration of rheological and fracture properties. The relaxation ability, as measured by the BBR, slowly decreases depending on the nature of the binder, the thermal history, and the degree of cooling. Some asphalt binders change very little upon cooling while others change a great deal. For some the rate of change peaks at some low temperature while for others the rate of change continues to increase with a continuing decrease in temperature. This ageing process is complex and at present poorly understood. However, that it has significance for the development of low-temperature performance grading tests for asphalt binders will be shown later with field data.

Struik [43] was the first to do extensive research on *physical ageing* in polymers and a range of other similar substances such as Dutch cheese, metals, and asphalt binders, to name a few. Physical ageing is thought to occur only at or below the glass transition temperature of a material and is associated with the collapse of free volume [43]. The phenomenon was rediscovered in asphalt binders during the SHRP program by Bahia [44] who somehow changed the name to *physical hardening* which is the term most commonly used today in the asphalt literature [45-49].

There are a number of reports in the literature that suggest physical ageing is not an issue at low temperatures [48, 49] or that it is only an issue for some binders in combination with certain aggregates [46]. However, all these describe only laboratory studies which are inherently weak at replicating failure mechanisms that occur in the pavement.

In addition to the physical ageing phenomenon there is a somewhat different process called *steric hardening* which was described as far back as 1937 [50]. It is generally believed to occur only at higher temperatures and is associated with molecular association [7].

Initially the AASHTO MP1 (now M320) specification made a provision for testing of the binder after one hour and 24 hours of isothermal conditioning at the grading test temperature, $T_{design} + 10$ [29]. At the time of the implementation of the BBR specification, there was no general consensus on how the effect could be included in a specification criterion so it was recognized that further research was needed. However, the provision never appears to have found wide acceptance, for reasons that are not well documented. It is believed that this type of ageing is a serious unknown factor in low-temperature specification grading and for that reason an assessment of its effect should be included in an improved specification method.

2.6 Development of an Improved Fracture Mechanics-Based Asphalt Binder Grading Method

In light of the observed deficiencies in AASHTO M320 and MP1a methods, an effort to develop an improved binder specification method was started. The method is being developed with performance data from test roads in northern Ontario. The short-term goal of the work is to be able to explain the performance differences as, for instance, those observed for the test sections on the Hearst test road. Longer term it is hoped that the method will find wide acceptance in the paving industry.

The standard test method *LS 296 – Laboratory Standard for the Fracture Performance Grading of Asphalt Binders* from the Laboratory Testing Manual of the Ministry of Transportation of Ontario (MTO) [51] allows for the determination of a grade temperature above which transverse cracking is largely prevented in pavements that are expected to fail from distress other than fracture (Method A). Or, as an alternative, it allows for a more comprehensive grading, involving a full set of fracture mechanics-based property evaluations below and above the brittle-to-ductile transition temperature (Method B).

Method A involves the loading of a notched binder specimen in Single-Edge-Notched three-point Bending (SENB), as indicated in Figure 4 or alternatively loaded in notched Compact Tension (CT) geometry, while monitoring the Load (P) and Load-Line Displacement (v). The load versus the displacement data is used to determine the Plastic Component of the Displacement (v_p). The temperature at which this plastic component nearly disappears i.e., purely brittle failure is approached, is used to determine a grade temperature for asphalt binders that can be used in pavements which are primarily expected to fail through forms of distress other than fracture. Due to the high rate of loading employed in the grading test method, as compared to the rate at which the pavement contracts during a typical cold winter night, the temperature at which the binder becomes brittle is shifted to colder temperatures by 10°C to obtain the grade temperature for the binder, i.e., $T_{grade} = T_{brittle} -10$. If the grade temperature falls below the minimum design temperature for the pavement, then the binder passes the grading test. Conversely, if the grade temperature falls above the minimum design temperature, then the binder fails the grading test. The low-temperature risk levels for the minimum design temperature are obtained from the LTTPBIND® temperature database as available from the United States Federal Highway Administration. Alternatively, the grade temperature may be set at the temperature where the Fracture Energy (G_f) reaches 100 $J.m^{-2}$ where it has been found that the majority of binders become ductile [52].

Method B involves the loading of sharply notched binder samples in either SENB or CT, as well as the loading of samples in Double-Edge-Notched Tension (DENT), as indicated in Figures 4 and 5. In the ductile-to-brittle regime, the Load-Line Displacement at Failure (v) or the Crack Opening Displacement at Failure (COD) is used to calculate the Plastic Component of the Load-Line Displacement (v_p) or the Plastic Component of the Crack Opening Displacement (COD_p). The temperature at which either the v_p or the COD_p nearly disappears i.e., purely brittle failure is approached, is used as the ductile-to-brittle transition temperature. In the same way as for Method A, due to the relatively high rate of loading employed, the ductile-to-brittle temperature in Method B is shifted to colder temperatures by 10°C to obtain the grade temperature. The low-temperature risk levels for the minimum design temperature are also obtained from the LTTPBIND® temperature database. If the grade temperature falls below the design temperature, then the binder passes the grading test. Conversely, if the grade temperature falls above the design temperature, then the binder fails the grading test. In addition, Method B also evaluates material properties in the brittle and ductile regimes. In the brittle temperature regime, the load at failure is used to calculate the Fracture Toughness (K_{Ic}) and the area under the load versus load-line displacement diagram is used to calculate the Fracture Energy (G_{Ic}). Finally, in the ductile regime, the Total Work of Fracture (W_f) in a DENT test is used to determine the Specific Essential Work of Fracture (w_e) and the Specific Plastic Work of Fracture (w_p).

In both Methods A and B, testing is conducted before and after thermal conditioning i.e., physical and steric ageing, to determine load versus load-line displacement data, crack tip opening displacements, fracture toughness, fracture energies, essential works of fracture, and plastic works of fracture, at specified temperatures, and rates of loading. If significant losses occur in the grading properties after thermal conditioning, then the binder must be classified as a lower grade.

A lower limit on the Fracture Energy (G_{Ic}) in the brittle state may be used to control thermal and traffic-induced fatigue cracking (Method B). It is believed that cracks that form through fatigue-type distress can add to the severity of transverse stress cracking at lower temperatures in subsequent winters. Lower limits on the Specific Essential Work of Fracture (w_e) and the Specific Plastic Work of Fracture (w_p) at close to 0°C may be used to control load-induced cracking caused by spring-thaw-related distress (Method B). It is likely that cracks initiated by this fatigue-type distress mechanism can also add to the severity of transverse stress cracking at lower temperatures in subsequent winters. Lower limits on the Specific Essential Work of Fracture (w_e) and the Specific Plastic Work of Fracture (w_p) at a temperature close to the expected average daily summer pavement temperature, may be used to control load-induced cracking caused by traffic (Method B). It is likely that cracks initiated by this distress mechanism can also add to the severity of transverse stress cracking at lower temperatures in subsequent winters. This distress can be particularly severe for thin pavements that take a significant amount of heavy traffic.

The remainder of this paper discusses the application of some of the newly developed test methods to the recovered binders from the C-SHRP test site on Highway 631, the SPS-9 site on Highway 17 near Petawawa, and the recently constructed pavement trial on Highway 655 near Timmins. Early indications are that the fracture mechanics-based method can explain performance differences where the CGSB, AASHTO M320 and MP1a methods fail.

3. EXPERIMENTAL

3.1 Materials

3.1.1 Highway 631 near Hearst

Binders from the C-SHRP test near Hearst were no longer available, so small samples of asphalt were obtained from the road and sent back to Queen's University. The asphalt cement was extracted with tetrahydrofuran (THF) in four washings. For a batch of approximately 6 kg of asphalt concrete a total of approximately 6 litres THF was used to extract approximately 250 to 300 g of asphalt cement. The THF was subsequently removed by gentle rotary evaporation until no further solvent could be removed. During the final stage of the evaporation process, the vacuum increased to 680 mm Hg (close to the maximum the pump could deliver), no further THF could be condensed, and the process was continued for an additional two hours at between 150°C and 160°C. THF was chosen as the desired solvent, since it has a very low boiling point, and it is excellent at dissolving both asphalt, as well as polymer modifiers such as Styrene-Butadiene-Styrene (SBS) and Reactive Ethylene-butylacrylate Terpolymers (RET) as used in other test sections. Trichloroethylene or other commonly used solvents do not remove polymers completely and hence, are avoided. During the spring of 2004, an additional 30 cores were drilled at regular intervals along the entire 2 km stretch of Highway 631. These cores were taken to obtain additional asphalt for extraction of the binder. An additional purpose of the coring was to obtain information about how the pavement thickness, void, and asphalt cement contents varied within and between the four test sections.

3.1.2 Highway 17 near Petawawa

Original binders from the SPS-9 site in Petawawa were obtained from the Ministry of Transportation of Ontario and were aged according to standard protocol of the Rolling Thin Film Oven Test (RTFOT) for 85 minutes followed by the Pressure Ageing Vessel (PAV) for 20 hours.

During the summer of 2002, a total of 60 cores were drilled at regular intervals along the entire stretch of Highway 17. The coring was done to obtain information about how the pavement thickness varied within and between the six test sections.

3.1.3 Highway 655 near Cochrane

Original binder samples from the new pavement trial on Highway 655 were obtained prior to construction of each section. Five 20 litre samples of the asphalt cement used in the binder course were collected and ten 20 litre samples for the surface course of each section were collected. The materials were all aged according to standard RTFOT and PAV protocol. In this paper only the properties of the asphalt cements from the surface courses is discussed.

3.2 Experimental Procedures

3.2.1 AASHTO M320 and MP1a Specification Grading

The AASHTO M320 and MP1a specification testing was conducted according to standard procedures with the BBR [34, 39]. Samples were tested at 4°C and 10°C above their predicted grade temperature after one hour of conditioning. The effect of physical ageing was studied by conditioning of the samples for periods of up to 72 hours at various temperatures.

3.2.2 Fracture Testing in the Brittle State

The fracture properties of the binders in their brittle state were determined according to procedures outlined in *Test Method LS 296 – Laboratory Standard for the Fracture Performance Grading of Asphalt Binders* from the Laboratory Testing Manual of the Ministry of Transportation of Ontario [51].

In brief, samples were cast with the aid of Teflon® film in between inserts that were held in place by support spacers. A 5 mm deep and 25 μm wide notch was introduced in the asphalt part of the beam by inserting a straight Teflon® sheet across the mold prior to pouring the binder in the cavity. Schematics of the mold and sample are provided in Figures 4 (a) and 4 (b), respectively. The binder was slowly cooled over a one-hour period after which the mold was placed in a freezer for one hour. Molds were then disassembled and the specimens stored for three days at low temperatures prior to testing. Conditioned samples were tested at a range of temperatures. A schematic of the setup is provided in Figure 4 (c). The test software was used to accurately integrate the load versus displacement curves to provide a measure of fracture energy according to standard procedures [51].

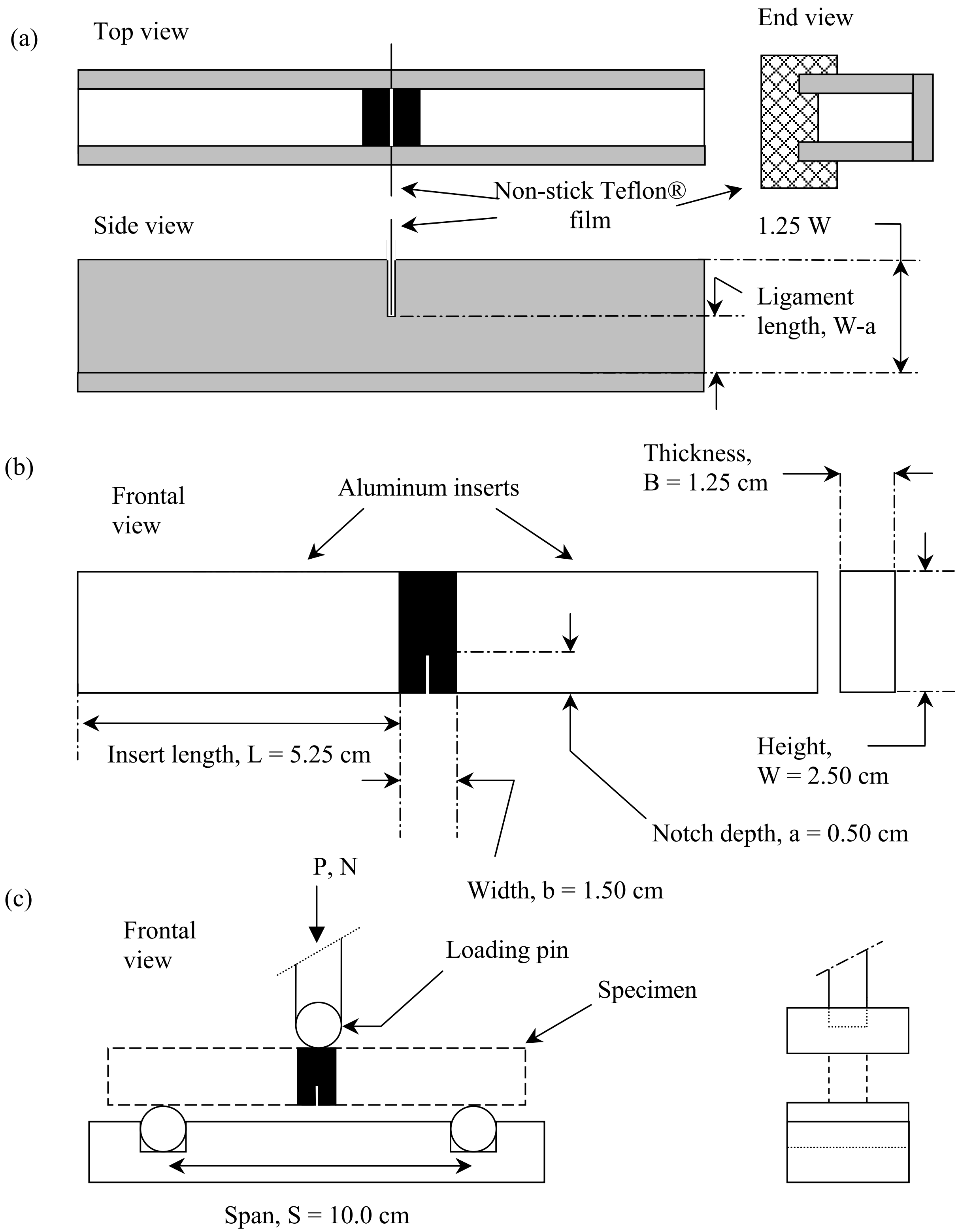

Figure 4. Fracture Test for Asphalt Binders: (a) Mold Design; (b) Sample Dimensions; and (c) Three-Point Bend Test Setup

3.2.3 Fracture Testing in the Ductile State

Fracture properties of the binders in their ductile state were determined according to procedures also described in *Test Method LS 296 – Laboratory Standard for the Fracture Performance Grading of Asphalt Binders* [51].

Ductile fracture properties were determined according to the essential work of fracture method as recently developed for asphalt binders by Andriescu et al. [17, 18]. In brief, samples with varying notch depths were cast with the aid of silicone rubber molds in between aluminum end pieces. The samples were conditioned at 25°C or 0°C over a period of 24 hours before being tested in direct tension. The notch depth in the samples was varied in order to provide ligament lengths ranging from 5 to 25 mm. The energy under each force-displacement curve was determined and the total specific work of fracture was plotted against the ligament length. The intercept of this regression provides the Essential Work of Fracture (w_e) while the slope of the line provides the Plastic Work of Fracture times a Scaling Constant (βw_p). Good binders will possess both high essential and plastic works of fracture while inferior ones will show one or both of these as low. Although experience with this test method is limited, it appears that large variations exist in both works of fracture for binders of the same grade [17, 18].

A schematic of a DENT specimen and actual photographs of samples after failure at 25°C and 0°C are provided in Figure 5. The arrows indicate the clearly visible elliptical shape of the yield zone for the sample that failed in a ductile manner at 25°C and the shiny fracture surface on the sample that failed through stable crack growth at 0°C.

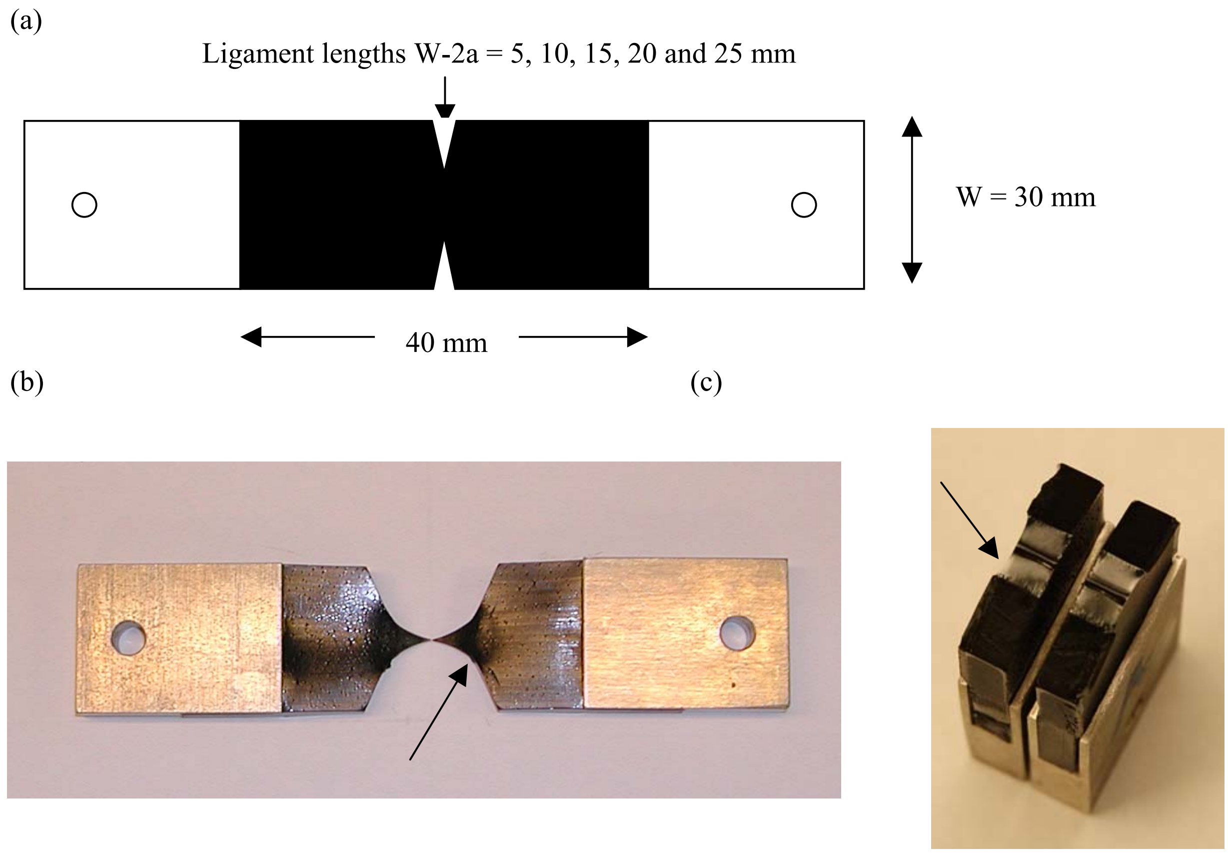

Figure 5. (a) Schematic of Double-Edge-Notched Tension (DENT) Specimen Design, (b) Photograph of Sample after Ductile Failure, and (c) Photograph of Sample after Failure at 0°C

Note: Arrow in (b) indicates the ductile fracture process zone with characteristic elliptical shape. Arrow in (c) shows the shiny fracture surface of a sample that failed through stable crack growth at 0°C.

3.2.4 Crack Surveying

Highway 613, due to its remote location, was only visited during June 2003, while Highway 17 was visited on four occasions during 2000, 2001, 2002, and 2003. Highway 655 was visited right after construction and during April 2004.

3.2.5 Temperature Data Collection

Minimum and maximum daily air temperatures were obtained from Environment Canada for weather stations located on Kapuskasing Airport, as well as the one located within the confines of the Canadian Forces Base (CFB) in Petawawa. Kapuskasing Airport is located some 160 km east of the test site on Highway 631. Hence, it is likely that the site temperatures were somewhat different from those obtained in Kapuskasing. The weather station on CFB Petawawa is located within close proximity of the test site since that part of Highway 17 is bordered on both sides by the base.

Minimum surface temperatures were calculated according to the algorithm used in the LTPPBIND® software program [53] and were considered accurate within one or two degrees for both Highway 17, with a lower accuracy for Highway 631 due to its great distance from the weather station.

The air and pavement temperatures on the Highway 655 site were measured by calibrated temperature loggers purchased from OMEGA Canada. Two OM-CP-TEMP1000P loggers were positioned on either side of the test road to record an air temperature reading every 30 minutes for up to a year. An additional two OM-CP-QUADTEMP four-channel loggers were buried on both sides of the road to measure pavement surface temperatures using thermocouples that were glued into the surface of the road. The thermocouples were positioned in the centre of both the southbound and northbound lanes at a depth of approximately 5 mm.

3.3 Test Road Designs

3.3.1 Highway 631 near Hearst

For a detailed description of the C-SHRP test road west of Hearst, the reader is referred to Anderson et al. [5] and Section 2.3.1. In brief, the road consisted of four sections constructed with three different binders. The binders were originally supposed to cover the A, B, and C classes of the CGSB grading system, but due to various scheduling problems they ended up all being A grades (as measured with the penetration at 25°C and the kinematic viscosity at 135°C) except for Section 3 which graded as an A with the kinematic viscosity at 135°C and Section B with the absolute viscosity at 60°C. The binder in Section 1 was manufactured from a Venezuelan crude source while the binder from Section 2 was manufactured from a Lloydminster crude. The binder in Section 3 had been chemically modified (oxidized). Some test data for the binders used in the Highway 631 trial are provided in Table 2.

3.3.2 Highway 17 near Petawawa

The Highway 17 trial, near Petawawa in northeastern Ontario, was part of the SPS-9 program. Six sections, containing five different binders of four SHRP grades, as well as an 85/100 penetration-graded binder, were constructed in 1996 starting some 5.4 km west of the Petawawa River on Highway 17 which is part of the Trans-Canada Highway. The 65 mm thick binder course for this trial was constructed in late 1996 while a surface course of equal thickness was placed in June 1997. What effect the delay in the construction of the surface course has had on the performance of the test sections is an unknown for this trial. The winter of 1997 was cold which may have inflicted some damage to the binder course.

In 1994, AADT was approximately 5,670 with 12 percent truck traffic. In 2000, this had increased to approximately 6,000 with 14 percent truck traffic.

Asphalt binder samples were kept for each of the PG-graded, as well as the penetration graded materials. SHRP grades used included PG 58-40P (polymer-modified binder for both Marshall and Superpave designs), PG 58-34 (unmodified, as well as polymer-modified binder for Superpave designs), and PG 58-28 (unmodified binder for Superpave design). The modifier used in the PG 58-34P and PG 58-40P grades was a styrene-butadiene copolymer. Straight PG 58-34 was obtained from a western Canadian source. Pertinent properties are listed in Table 3.

Table 3. Pertinent Binder Properties for Trial Sections on Highway 17

Section	Grade	AASHTO M320 on PAV Residue	
		T(S = 300 MPa)	T(m-value = 0.3)
17-1	85/100	-30.3	-29.7
17-2	58-40P	-41.0	-41.0
17-3	58-34P	-36.0	-35.0
17-4	58-28	-28.0	-28.0
17-5	58-34	-36.0	-35.0
17-6	58-40P	-41.0	-41.0

Note: A letter P behind the grade indicates that the asphalt contained polymer modifier.

3.3.3 Highway 655 near Cochrane

The most recent pavement trial discussed in this paper was constructed during the summer of 2003 on Highway 655 some 60 km north of Timmins near Cochrane, Ontario. The test sections formed part of a contract for the total reconstruction of 13.4 km of Highway 655.

The binders chosen for the seven test sections were nearly identical in terms of current AASHTO M320 low-temperature specification (PG xx-34 grades) and comprised polymer modified, air blown, and one unmodified binder. The intent of this project is to identify other issues, if any, that need to be considered in order to provide a better selection tool for paving grade binders. One of the reasons this is believed to be necessary is because of previous experience with the test roads near Lamont and Hearst, in which asphalt cements of nearly the same grades showed large variations in fracture performance. It is presumed that more insight into the low temperature and fatigue failure mechanisms might be gained from comparing a larger number of binders of nearly identical BBR grades, perhaps pointing out why the current AASHTO M320 and MP1a specification methods fail to correctly predict the performance of some binders.

Each test section comprised a 50 m long transition area followed by a 100 m sampling area, 200 m monitoring portion, and another 150 m sampling area. The same binder from the same supplier was used to pave both the binder and surface course on both lanes of a test section. However, it should be noted that some suppliers switched lots between the binder and surface course because of scheduling delays.

The most recent traffic data for Highway 655 are dated 2001. At that time, AADT was 1,250 with 10.2 percent truck traffic. The intensity was rather constant throughout the year, Summer Average Daily Traffic (SADT) being only slightly higher at 1,350 vehicles per day. The traffic split was 62 percent on the northbound lane, although it is known that the southbound lane is subjected to much heavier loads due to logging trucks traveling loaded to Timmins and empty in the other direction.

Since the Lamont and Hearst test roads contained only straight run and oxidized binders, it was decided to include one unmodified, two oxidized, and four polymer-modified binders in the new trial. All binders graded according to AASHTO M320 between 64°C and 70°C in the dynamic shear rheometer and within a narrow range below -34°C in the BBR. Limiting temperatures of the binders are given in Table 4.

Table 4. AASHTO M320 Limiting Temperatures for Trial Sections on Highway 655

Section	Type	AASHTO M320					
		T(S = 300 MPa)			T(m-value = 0.3)		
		Contractor	MTO	Queen's	Contractor	MTO	Queen's
655-1	RET	-27.5	**-26.4**	**-25.7**	**-26.0**	-27.1	-27.3
655-2	Ox/SBS	-27.5	-28.3	-27.9	**-24.0**	**-24.2**	**-25.7**
655-3	SBS	-28.8	-29.5	-29.4	**-25.1**	**-26.6**	**-27.0**
655-4	SBS	-28.7	-28.5	-28.8	**-25.0**	**-25.3**	**-26.0**
655-5	SBS	-26.9	-27.2	-27.2	**-24.5**	**-24.0**	**-25.4**
655-6	Ox	-28.7	-28.7	-28.6	**-24.7**	**-25.3**	**-24.6**
655-7	Control	-24.9	-29.2	-28.3	**-24.8**	**-26.2**	**-24.7**
Ranges:		3.9	3.1	3.2	2.0	2.6	2.4

Note: Numbers in **bold** are those that determine the AASHTO M320 performance grade. RET = Reactive Ethylene-butyl acrylate-glycidyl methacrylate Terpolymer-modified asphalt, Ox = Oxidized, SBS = Styrene-Butadiene-Styrene triblock copolymer-modified asphalt.

4. RESULTS AND DISCUSSION

4.1 Hearst Test Road

The C-SHRP test road in Hearst was constructed in 1991 and hence, it provides the most reliable transverse and fatigue cracking data. As shown in Figure 3, early rankings in terms of low-temperature cracking performance often change but this should no longer be a concern today after 12 years of service. Over the period prior to the visit in the summer of 2003, it has likely been exposed to temperatures close to its design temperature on many occasions, while it also has gone through 12 of what were likely extended periods of freeze–thaw distress.

The main issue is what factor or factors are responsible for the large performance differences as reported in Figure 3 for binders of nearly the same SHRP and CGSB performance grades. The difference in cracking severity between Sections 1 and 2, which have identical performance grades, is a factor of 20 as measured by total crack length. Furthermore, Section 3 has 50 percent more cracks compared to Section 2, despite the fact that its performance grade is as much as 3.6°C lower than that of section 2. Finally, comparing the performance of Sections 1 and 4, that were constructed with the same binder, shows that doubling the asphalt pavement thickness, as was done in Section 4, reduces the cracking severity for this particular binder by as much as 54 percent.

The following discussion will focus on all possible and the few probable factors that can explain the performance differences in this test road. The same analysis will be given for test sections on Highways 17 and 655, after which a brief discussion follows on where the authors think that major improvements can be made in low-temperature and fatigue fracture specification testing of asphalt binders.

4.1.1 Structural and Design Issues

The first obvious place to look for possible causes for the observed performance differences in the Highway 631 test road is in the structural and design aspects of the test sections. Given the fact that this was one of the original C-SHRP test sites, it was unlikely that structural and/or design factors confound the findings. However, it was considered prudent to determine the variations in pavement thickness, void content, and asphalt cement content for the four test sections. Table 5 shows the results for this analysis on a total of 30 core samples taken at 50 m intervals all through the site. The average core thickness and void content for the sections were determined from measurements on eight cores for each of Sections 1, 2, and 3 and six cores for Section 4. Asphalt cement contents were determined by ignition. The average is for two cores and the error is the difference between the two determinations divided by two.

Table 5. Structural and Design Parameters for C-SHRP Test Sections on Highway 631

Section	Thickness, mm	Voids, %	Asphalt Cement, wt %
631-1	45.0 ± 5.1 (40-52)	6.9 ± 0.09 (5.0-8.2)	5.36 ± 0.22
631-2	46.3 ± 3.6 (42-51)	6.7 ± 0.06 (5.5-7.9)	5.82 ± 0.01
631-3	46.3 ± 6.4 (37-54)	7.3 ± 0.06 (5.8-8.2)	5.73 ± 0.14
631-4	88.8 ± 5.0 (83-95)	-	-

Note: Averages are given with standard deviations while numbers in brackets give the range as found for the core thickness and void contents. Void and binder contents for section 4 were not determined but were considered to be close to those of section 1.

The data in Table 5 clearly show that this was a carefully constructed test site and that structural and design factors (other than the double thickness in Section 4) are unlikely factors to explain the large differences in performance between sections. The slightly low asphalt cement content in Section 1 may be representative or it may just be because only two cores were taken for ignition from each section. However, if correct then it would probably be able to explain a small part of the 20 fold difference in cracking severity between Sections 1 and 2.

4.1.2 Chemical Ageing Effects

If large differences in the tendency of each binder to chemically harden were present then this would explain to some extent the observed performance differences. The binders used in Hearst were from three different sources and hence, are likely to show variations in their loss of grade temperature after 12 years. Table 6 shows the comparison between the original binder grading temperatures with those determined on extracted materials.

Table 6. PAV-aged versus Field Aged AASHTO M320 Properties for Highway 631 Binders

Section	Original PAV Material		12 Years Aged In Service		Loss in Grade Temperature, °C
	T(S=300 MPa)	T(m=0.3)	T(S=300 MPa)	T(m=0.3)	
631-1	**-23.1**	-25.0	-21.1	**-15.5**	7.6
631-2	**-23.2**	-26.4	**-19.6**	-19.7	3.6
631-3	**-26.8**	-28.2	-23.6	**-21.5**	5.3

Note: Numbers in **bold** are those that determine the performance grade which is set at a temperature 10°C below the measured limiting temperature. Original binder properties as reported by Robertson [1].

The results show that there are significant differences in chemical ageing behaviour among the three binders (as measured by BBR on extracted samples). Binder 2, with the least amount of cracking, shows a significantly lower tendency for chemical ageing, gaining an advantage of 4.1°C after 12 years of service compared to Binder 1. Given the fact that the above properties were determined on binder extracted from whole asphalt core samples, the question is raised of how much the ageing varied with depth in the pavement. Perhaps Binder 1, with the highest number of cracks, has aged significantly on the top surface thus making it more prone to crack initiation and therefore large-scale transverse cracking.

The loss in grade temperatures of between 3.6°C and 7.6°C over and above the PAV aged materials is a serious cause for concern. The RTFOT/PAV ageing protocol was supposed to age binders to the equivalent of about 7 to 8 years of field service. Recent research suggests that this is equivalent to only 2 to 3 years at best, suggesting that an improved laboratory ageing method would go a long way to reduce the widespread occurrence of premature low-temperature distress [52].

Although the differences among the three sections is significant, it was considered that this may not be the only factor to explain the differences in cracking severity of a factor of 20. Hence, other factors including differences in physical ageing and in brittle and ductile fracture performance were also investigated.

4.1.3 Physical and Steric Ageing Effects

A second likely contributing factor for the large differences in performance as observed on Highway 631 comes from the tendency of different binders to suffer to different degrees from physical and steric ageing. The binder used in Sections 1 and 4 would only have to age by a few degrees more than the one from Section 2 to explain a large part of the difference in cracking severity. Temperature statistics are such that if the "real" performance grade would have been reached for Section 2 on only a couple of occasions, then Section 1 would only have to age by a few degrees more to have been exposed to numerous more days of damaging temperatures.

Figure 6 provides the rates of physical ageing for the three binders, as extracted from the asphalt mixture, after 12 years of service. The data show that all three binders physically age to a significant degree. For Sections 1 and 2 the maximum (worst) rate was found to occur for conditioning temperatures of –10°C and –16°C, respectively, while for Section 3 it happened at –22°C. The results show that the physical ageing mechanism is a contributing factor that can explain a large portion of the severe cracking in Section 1 and a small portion of the factor of 20 difference in cracking severity between Sections 1 and 2. While after one hour of conditioning the difference in grade temperature between Sections 1 and 2 is already a significant 4.1°C, as shown in Table 6, this worsens to 6.4°C after 72 hours of conditioning.

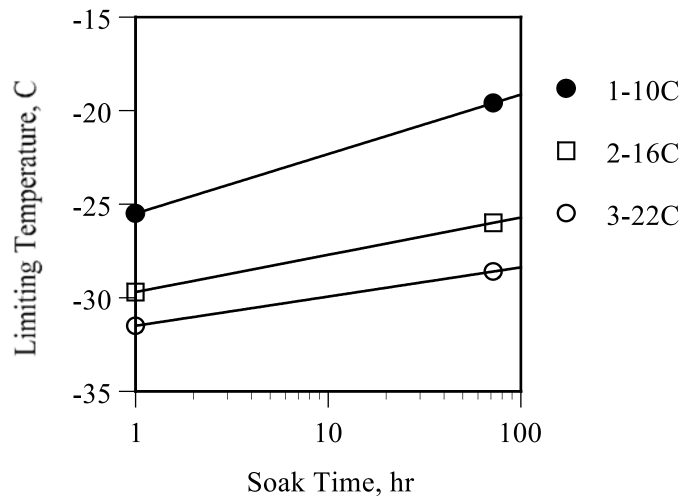

Figure 6. Physical Ageing Rates for Binders Used on C-SHRP Test Sections on Highway 631
Note: Section numbers in legend are followed by soak temperature where maximum physical ageing occurred.

Figure 7 illustrates the possible consequences of this difference in grade temperature in light of the past 12 years of temperature records available for Kapuskasing Airport. The graph shows that if the grade of Section 2 caused it to be damaged for only a couple of days then the binder in Section 1 with a limiting grade temperature of around 6.4°C warmer would have sustained damage for many more days. If the true grades were those as measured after 72 hours of conditioning, then Section 1 would have sustained damage for approximately 595 days over the life of the road versus approximately 154 days for Section 2. This difference of nearly a factor of 4 is likely responsible for a large part of the 20 fold difference in performance between Sections 1 and 2.

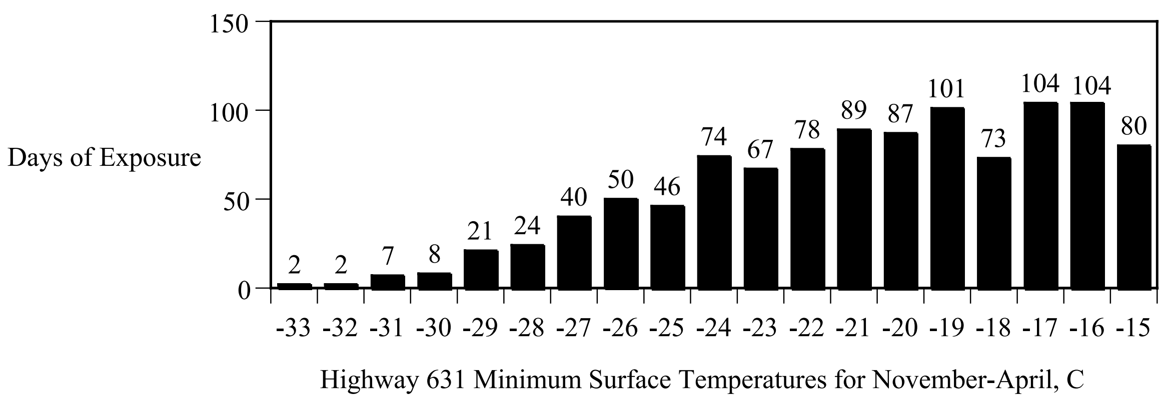

Figure 7. Minimum Pavement Surface Temperatures for Highway 631 Test Sections as Calculated from Air Temperatures as Recorded at Nearest Weather Station on Kapuskasing Airport

That there is more to this issue becomes clear from a comparison between Sections 1 and 4 which were made with the same binder from Venezuela but which had different thicknesses. The fact that Section 4 has only 46 percent of the cracking severity when compared with Section 1 suggests that more than half of the cracks in Section 1 are related to some form of fatigue distress (thermal, spring-thaw or traffic induced). If some of the cracks in Section 4 are also related to fatigue then the proportion of fatigue related cracks in Section 1 would even be higher than 46 percent. Further, the effects of chemical and physical ageing also fail to explain the difference in cracking severity between Sections 2 and 3 with 49 and 74 cracks, at performance grades of –26.0°C and –28.6°C, respectively, after three days of conditioning. Both binders suffer equally from physical ageing so there must be other factors to explain the significant difference in performance. This analysis suggests that the fatigue properties of the binders must have some impact on the level of transverse cracking. Hence, brittle fracture energies at –30°C and –24°C as well as ductile fracture energies at 0°C and 25°C were determined in order to further understand the observed distress levels.

4.1.4 Brittle and Ductile State Fracture Properties

The final piece of the puzzle for Highway 631 comes from an analysis of the fracture properties of the recovered binders in both their brittle and ductile states. It is only reasonable to assume that transverse cracks are, to some degree, promoted by the presence of fatigue distress given the fact that trial studies have repeatedly shown that thicker sections or lanes with less traffic show fewer transverse cracks.

Figures 8 and 9 show the fracture energies of the binders as determined at –30°C and –24°C in the brittle state and at 0°C and 25°C in the ductile state. The brittle properties were determined according to the test configuration in Figure 4 (c) while the ductile properties were determined according to the test configuration given in Figure 5 (a). In order to assure good fatigue and transverse cracking resistance it is desirable to have both high brittle (G_{Ic}) and ductile (w_e and w_p) state fracture energies. The data show that the best performing binder is the one used in Section 2, which also has the lowest number of transverse cracks. The worst performing binder is the one used in Section 3. This section has 50 percent more cracks than Section 2 despite the fact that it has a performance grade temperature that is a few degrees lower than that of the Section 2 binder. The binder used in Sections 1 and 4 has fracture properties in between those for Sections 1 and 3. It also has the highest number of transverse cracks which is likely due, in large part, to the significant degree of physical ageing upon cold storage (see Section 4.1.3).

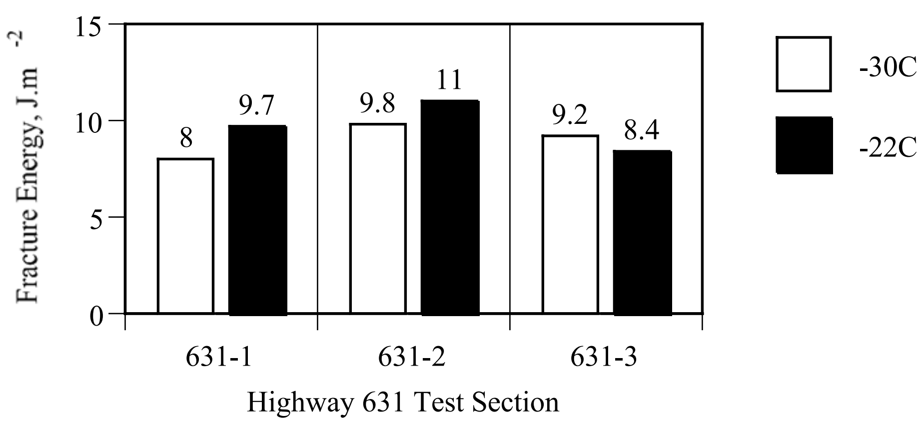

Figure 8. Fracture Energies in Brittle State for Recovered Binders from Highway 631

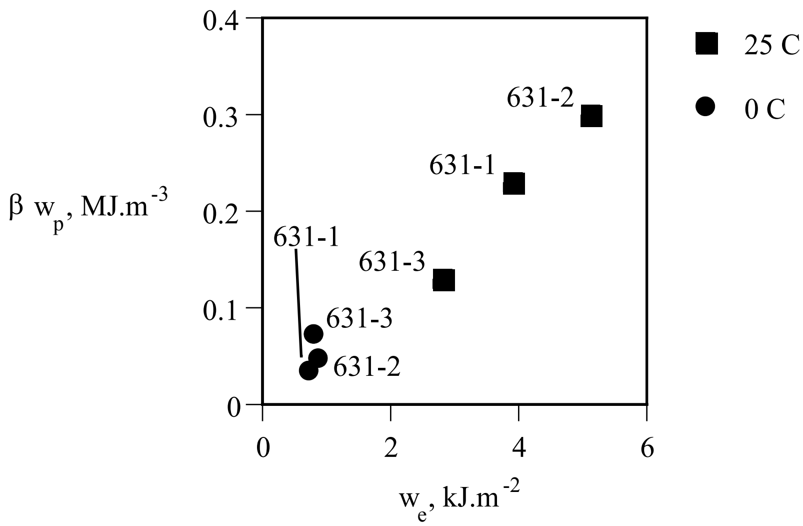

Figure 9. Fracture Energies in Ductile State for Recovered Binders from Highway 631
Note: Labels relate to section numbers. Scaling factor β accounts for the shape of the plastic fracture process zone. (For further details see Andriescu et al. [17, 18].) Experiments at 25°C were done at 100 mm/min while experiments at 0°C were done at 0.1 mm/min.

The advantage in brittle fracture energy for Section 2 is 22.5 percent at –30°C. At ambient temperatures the differences among the fracture properties for Sections 1 to 3 are more significant. If it is assumed that the essential work and plastic work of fracture terms have equal importance in the failure process then their product $w_e \times \beta w_p$ gives an indication of the binder's performance with respect to fracture in the ductile state. In such analysis the binder in Section 2 has a significant advantage over the one in Section 1. The oxidized binder in Section 3 does most poorly at 25°C while it does slightly better at 0°C. Figure 10 shows the relative performance of the three binders at both temperatures.

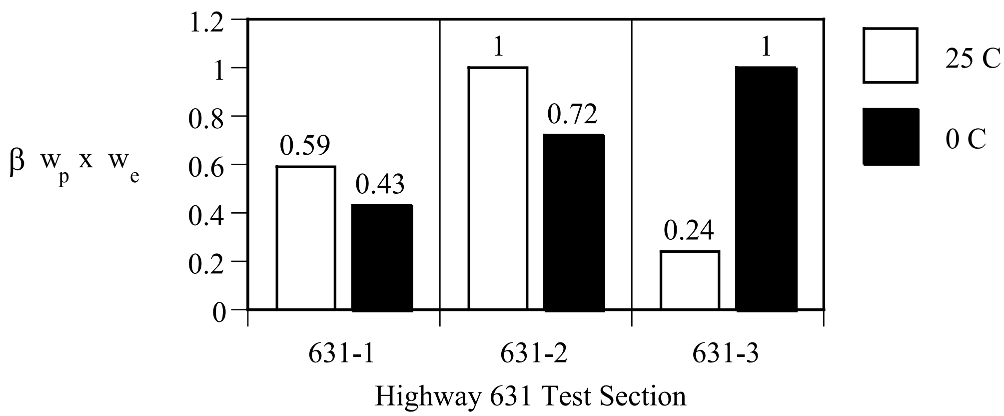

Figure 10. Relative Ductile Fracture Performance of C-SHRP Binders from Highway 631
Note: Bar heights reflect relative rankings of $w_e \times \beta w_p$ at both test temperatures.

The low fracture energies for Section 3 at 25°C are likely a contributing factor to the relatively high level of transverse cracking in this section as compared to Section 2 (see Figure 3). It should be noted that the fracture process at 0°C proceeded through a stable crack growth phase whereas at 25°C the process was one of yielding, necking and tearing (see Figures 5 (b) and (c)).

4.2 Petawawa Test Road

The results from the trial on the Trans-Canada Highway in Petawawa are discussed in detail in Iliuta et al. [6] and Hesp [52]. A short summary of the findings, as well as a slightly different analysis is presented herein.

Perhaps the most useful yet disconcerting results were obtained from the sections on Highway 17. At an AADT of 6,000, it is the busiest of the three test roads discussed in this paper. The SPS-9A sections on Highway 17 were exposed to significant low-temperature distress in their first winter when the pavement surface temperature dropped below –25°C on three occasions. It should be noted that the binder course for this site was constructed in late 1996 while the surface course was constructed in June 1997. In early 1999, the temperature dropped to –24.7°C. The lowest pavement surface temperature prior to the last survey of this site was reached in 2003 when the temperature fluctuated between –26.5°C and –27.2°C for about a week. A complete account of all exposures is provided in Figure 11. It should be noted that in the first seven years of service, at no time did the pavement surface temperature fall below the –28.0°C continuous AASHTO M320 grade temperature for the PG 58-28 section (see Table 3).

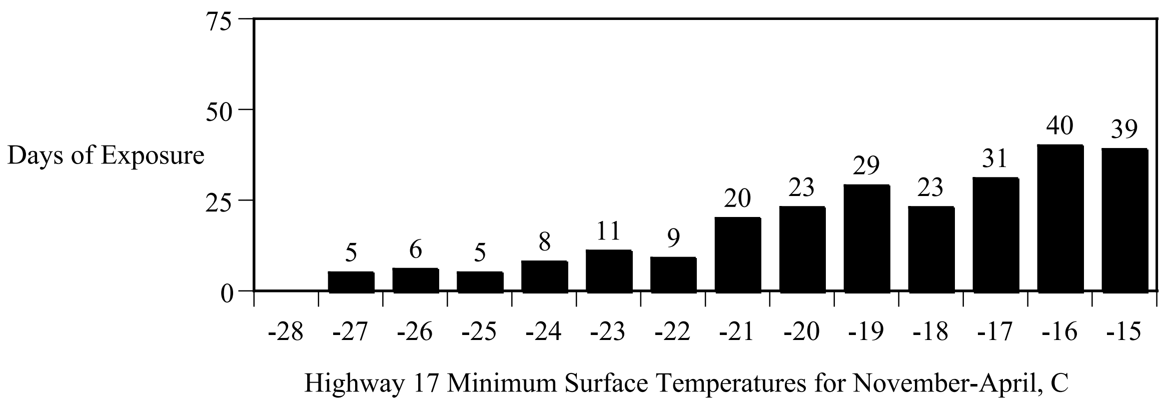

Figure 11. Minimum Pavement Surface Temperatures for Highway 17 SPS-9 Test Sections as Calculated from Air Temperatures as Recorded at CFB Petawawa

Figure 12 shows that the AASHTO M320 method fails to predict the cracking onset for the PG 85/100, 58-28, 58-34, and 58-34 PMA sections by a large margin. These cracked at temperatures above what was predicted by the BBR limits. The 85/100 (graded at –28°C) and the PG 58-28 section started to crack after the first winter but since no direct measurements were made it is impossible to determine exactly when it started. For the PG 58-28 section the continuous grade temperature after three days of conditioning shows a loss of around 6°C compared to the regular AASHTO M320 temperature determined after 1 hour. Hence, cracking likely started anywhere above –28°C but likely below –22°C.

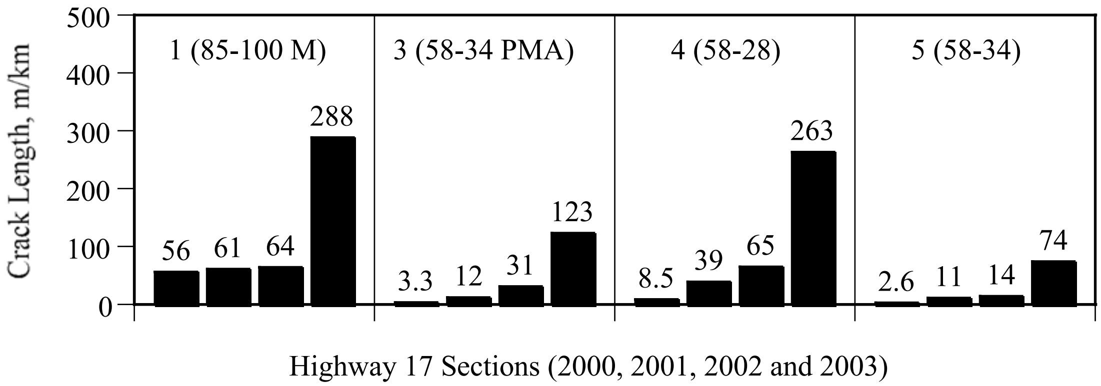

Figure 12. Cracking History for SPS-9 Site on Highway 17 in Petawawa, Ontario
Note: 85/100 M = 85/100 penetration grade used in Marshall mixture design and PMA = polymer-modified asphalt cement. Neither section 2 or 6 made with PG 58-40 binders showed cracking in 2003.

It is perhaps more revealing to consider the two PG 58-34 sections. These should not have failed until the pavement surface temperature dropped to at least –34°C, which is more than six degrees colder than what had been recorded at the time of the last survey in 2003. However, Figure 12 shows that this is clearly not what happened, since a significant 123 m/km (65 cracks/km) and 74 m/km (52 cracks/km) of transverse cracking had occurred, at a relatively early stage in the design life for this pavement. In early 2004, the temperature dropped to even lower values; hence it is expected that the distress is now much worse.

Reviewing the grading temperatures for both binders reveals that the limiting m-value predicts a cracking onset at –35°C after one hour conditioning and –29°C after three day conditioning. The limiting fracture energy temperatures as determined after three day conditioning predict the onset of cracking at –23°C and at –25°C, which are somewhat more discriminating temperatures. The difference in cracking severity between the polymer-modified and unmodified PG 58-34 sections is significant and deserves further investigation. The brittle state fracture energy of the straight binder was approximately 20 percent higher than what it was for the polymer-modified binder. Pavement thickness did not vary by more than 8 percent between the two sections; hence, it was not considered a significant factor. However, other factors such as variations in binder and void contents, and/or differences in essential and plastic works of fracture were not investigated for this site but would probably have accounted for most of the difference.

The Petawawa test road has shown that physical ageing is a likely contributor to early pavement distress and that an improved binder specification test method should account for the different tendencies of different binders to deteriorate in terms of their low temperature performance.

4.3 Cochrane Test Road

The newest of the test roads in Ontario experienced severely cold weather just three months after the end of construction. Whereas the road was designed to withstand a minimum surface temperature of –34°C after some seven to eight years of service, the pavement temperature hit an all-time low for the area in early January 2004 when the air temperature reached to approximately –47°C. The two thermocouple loggers recorded minimum air temperatures of –46.1°C and –48.2°C. The measured pavement temperature at approximately 5 mm depth was –34°C and a surface temperature calculated according to the latest LTTPBIND® algorithm would have been around –37°C. Figure 13 provides a plot of the average air and 5 mm deep pavement temperatures recorded. The graph shows that for eight days in January 2004 the air temperature dropped below –40°C and the pavement temperature as measured at 5 mm depth dropped below –30°C. The record low pavement temperatures were reached on January 9 and 15 both at 9:30 AM when the average read –34.0 and –34.1°C, respectively. Given the fact that this road was designed to withstand a pavement surface temperature of –34°C after some seven to eight years of service, this was a near-perfect accelerated pavement test!

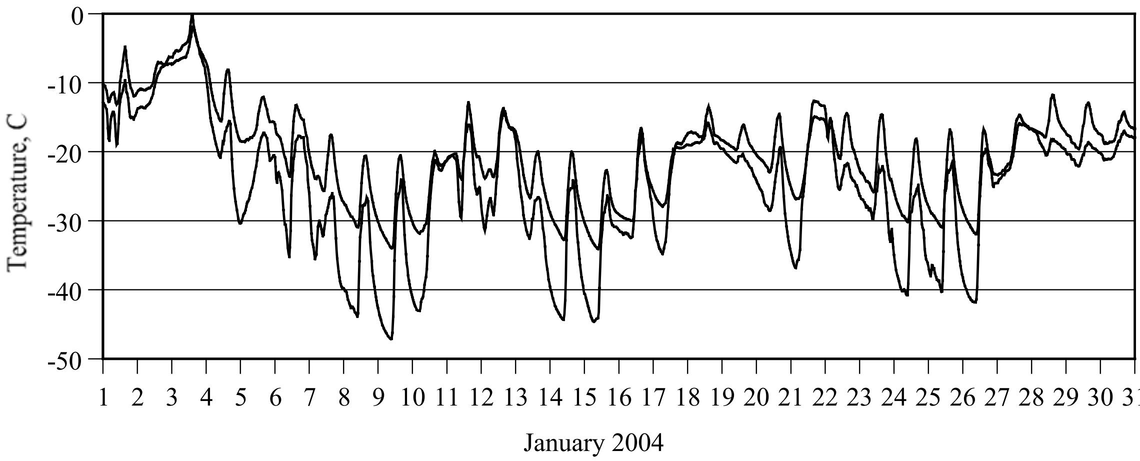

Figure 13. Average Air and Pavement Temperatures for January 2004 Cold Spell on Highway 655
Note: Air temperature (lower curve) is average of two thermocouple loggers whereas the pavement temperature (upper curve) is an average of four readings. Air readings varied by no more than 2°C between thermocouples whereas pavement readings varied by no more than 1.5°C between thermocouples.

On January 31, 2004, the site was visited to look for possible distress and to switch one of the two air thermocouple loggers. During this initial assessment only two transverse cracks were found in Section 2 (one had started at the joint with Section 1 while the other was located near the joint with Section 3) and one in Section 3 (located close to the joint with Section 4).

On April 24, 2004, the site was again visited for a second look for distress and to download the pavement temperature data. Surprisingly, a large amount of wheel path cracking was observed in Sections 2, 3, and 4 and a lesser amount in Section 6. Section 2 with 60 m of wheel path cracking also showed six transverse cracks, although these were all located either at the beginning or end of the section. Section 3 with 40 m of wheel path cracking showed only a single transverse crack that was less than half a lane wide towards the end of the section. Section 4 also had about 40 m of wheel path cracking and only a single transverse crack about half a lane wide approximately 180 m into the section. Section 6 showed approximately 5 m of wheel path cracking about 325 m into the section. Section 7 had only a single transverse crack about half a lane wide approximately 230 m into the section. Finally, only Sections 1 and 5 showed no detectable damage.

Figure 14 shows a picture of (a) a transverse crack that has occurred in Section 2 and (b) the typical wheel path cracking that occurred in Sections 2, 3, and 4.

(a) (b)

Figure 14. (a) Transverse Crack in Section 2 and (b) Representative Wheel Path Crack in Section 3

The other noteworthy aspect of the cracking within the site is that all of the wheel path distress occurred on the left side of the southbound lane. This can be partially explained by the fact that loaded logging trucks go south to Timmins and return empty. However, it is more challenging to explain why the distress was all confined to the *left* wheel path. The exact reason for this is difficult to determine but might have to do with the way in which the pavement thaws during spring or how it builds up thermal stress during cold spells. In northern Ontario salt is deposited only in the middle of the road and left to be dispersed by the passing traffic. Hence, the left wheel path may have been subjected to more spring thaw type distress. A second contributing factor may have been the higher level of *transverse* restraining stress in the left wheel path compared to the right wheel path (which is closer to the unrestrained edge of the pavement). Such higher level of thermal stress would cause more damage initiation in the left wheel path during the early hours of January 8-10, 14-15 and 24-26, which only became visible during the spring thaw. However, there may be other reasons for the fact that only the left wheel path was affected. It is expected that significant damage will show in all wheel paths for most sections. This study will have to show in years to come if some of the tougher modified binders can reduce or completely mitigate this distress and thereby produce a benefit to the user agency.

At this moment, only the physical and steric ageing phenomena and the ductile fracture energies at 25°C have been determined for the seven binders in this trial. It is interesting to note that Section 2, an oxidized binder with the most severe wheel path and transverse cracking, also was the binder that was the most susceptible to physical and steric ageing. Figure 15 shows the continuous grade temperatures after three days of conditioning at various temperatures.

Section 3 with some 40 m of wheel path cracking has one of the lowest plastic works of fracture at 25°C and 100 mm/min but Sections 6 and 7 are not far behind. Figure 16 shows the ductile works of fracture for all binders. The only two binders that show no detectable cracking are those used in Sections 1 and 5 which also happen to show the highest essential and plastic works of fracture in this set of data. It would be premature to draw any firm conclusions from the limited distress data and binder test results since early rankings often change due to small variations in pavement thickness, void, and asphalt binder contents.

However, these preliminary results support the general thrust of the work that a new binder specification test is not only needed but also feasible. The effects of physical and steric ageing and large variations in brittle and ductile fracture energies is relatively easily incorporated and would probably produce substantial savings for user agencies.

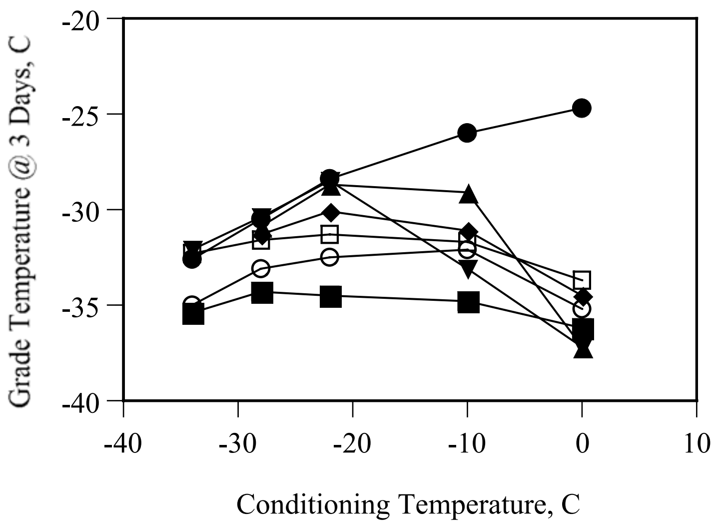

Figure 15. Continuous Grade Temperatures after 3 Days Conditioning for Highway 655 Binders
Note: ■ – 655-1; ● – 655-2; ○ – 655-3; ♦ – 655-4; □ – 655-5; ▲ – 655-6; and ▼– 655-7.

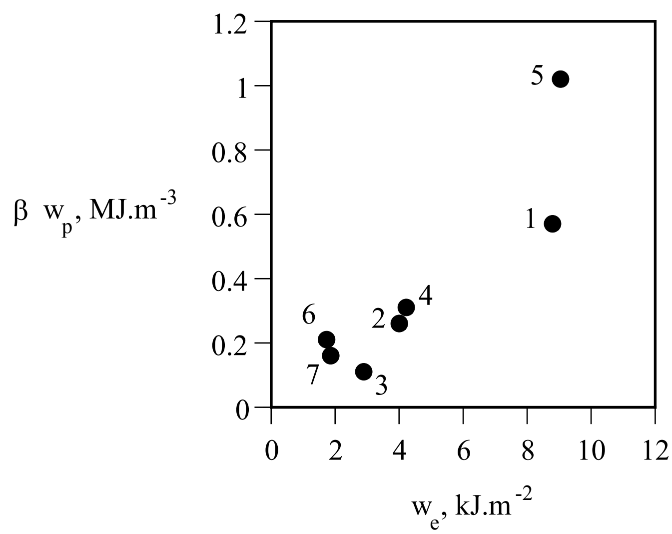

Figure 16. Fracture Energies in Ductile State for PAV-aged Highway 655 Binders
Note: Labels relate to section numbers. Scaling factor β accounts for the shape of the plastic fracture process zone. For further details see Andriescu et al. [17, 18]. Samples were conditioned at room temperature for 24 hr prior to testing. Double-edge-notched tension tests were done at 25°C and 100 mm/min.

5. SUMMARY AND CONCLUSIONS

Considering the results presented in this paper, the following summary and conclusions may be given:

1. A review of the field performance data for asphalt binders that were used in the Lamont, Alberta and Hearst, Ontario C-SHRP trials shows that binders of nearly the same low-temperature CGSB or AASHTO grades can show differences in transverse cracking severity of up to a factor of 35. A review of the field performance data for binders used in the Petawawa, Ontario SPS-9 pavement trial shows that binders graded at PG –34°C can show significant cracking onset at temperatures around –27°C. These findings underscore the need for an improved low-temperature asphalt binder specification method.

2. Chemical, physical and steric ageing of asphalt binders, beyond what happens under standard laboratory grading protocols, are likely important factors in the explanation of early low-temperature failures, durability problems with pavements that have fractured, and vast performance differences between binders of the same CGSB or AASHTO grade. Hence, the effects are addressed in a newly developed performance grading test method by testing asphalt binders after a sufficient degree of chemical ageing and after a sufficient period of cold storage to account for differences in steric and physical ageing [51].

3. Differences in brittle and ductile fracture properties of binders of the same CGSB or Superpave grade can be very significant. Hence, the newly developed performance grading test method allows user agencies to include lower limits on both the fracture energy in the brittle state, G_{Ic}, and the essential and plastic fracture energies in the ductile states, w_e and w_p. Above these limits the binders are able to withstand a significant amount of low, intermediate, and high-temperature load and/or thermal distress without showing signs of premature cracking.

The relevance and accuracy of some of these findings needs to be tested with the performance data to be obtained from the well-designed pavement test sections on Highway 655 and from other contracts.

DISCLAIMER

None of the sponsoring agencies necessarily concur with, endorse, or adopt the findings, conclusions or recommendations either inferred or expressly stated in subject data developed in this study.

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