VALIDATION OF NEW ASPHALT CEMENT SPECIFICATION TEST METHODS USING EASTERN AND NORTHEASTERN ONTARIO CONTRACTS AND TRIAL SECTIONS

by

SATHISH KUMAR SUBRAMANI

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Abstract

This thesis discusses and documents the validation efforts related to two new test methods developed for the grading of asphalt cement. Asphalt cements that were recovered from a large number of regular paving contracts and trial sections in eastern and northeastern Ontario were tested according to LS-299 and LS-308 test methods. The LS-308 *Extended Bending Beam Rheometer (BBR) Method* involves testing of asphalt cement in a regular BBR after specific times of conditioning at –10ºC and –20ºC. The LS-299 *Double-Edge-Notched Tension (DENT) Test* involves testing of asphalt cement in DENT configuration at 15ºC and 50 mm/min.

Both these methods improved the ranking of asphalt more than that of regular BBR protocol as in AASHTO M320 method. Pavement contracts and trial sections showing little or no distress were made with asphalt cements having low grade losses in LS-308 and high strain tolerances as measured in LS-299. Hence, future implementation of these methods should reduce thermal cracking distress in Ontario roads.

Besides, a number of other properties were investigated. All recovered regular contract materials were tested at –10ºC, –20ºC, and –30ºC after various periods of conditioning. Samples were loaded for 240 s, followed by unloading for 720 s in BBR, allowing the separation of elastic and viscous creep deformations. The regular specification parameters, stiffness (S) and relaxation ability (m-value) and also other performance-related properties were determined. It was found that the confounding effect of simultaneous elastic and viscous deformations and inadequate conditioning prior to testing in the BBR protocol are the most important reasons for the observed inconsistency in grading. These findings are in general agreement to those from the earlier LS-308 tests.

The main reason for the poor performance is asphalt physically age during extended periods of exposure to low temperatures. Hence, those that are graded according to the current AASHTO M320 protocol are often under-designed for thermal cracking. The slow crystallization of waxes and precipitation of asphaltenes from oily phase is the primary cause for deterioration of properties. Besides, waste engine oils with other gelling agents like PPA, increase the chemical ageing tendency of asphalts and hence, thermal cracking distress.

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Abbreviations, Acronyms and Symbols

AADT	Average Annual Daily Traffic Count
AASHTO	American Association of State and Highway Transportation Officials
AC	Asphalt Cement
ASTM	American Society for Testing and Materials
BBR	Bending Beam Rheometer
CTOD	Crack Tip Opening Displacement, m
DENT	Double-Edge-Notched Tension
DSR	Dynamic Shear Rheometer
ER	Elastic Recovery
HMA	Hot Mix Asphalt
LS	Laboratory Standard Test Method
LTPPBind®	Long Term Pavement Performance Binder Selection Software
m(t)	Slope of the Creep Stiffness Master Curve (m-value)
MTO	Ministry of Transportation of Ontario
NCHRP	National Cooperative Highway Research Program
NMR	Nuclear Magnetic Resonance Spectroscopy
NSERC	Natural Sciences and Engineering Research Council of Canada
PG (PGAC)	Performance Grade (Performance Graded Asphalt Cement)
SBS	Styrene-Butadiene-Styrene Block Copolymer
SHRP	Strategic Highway Research Program
S(t)	Creep Stiffness, Pa
XRF	X-Ray Fluorescence Spectroscopy

Chapter 1

Introduction

1.1 Asphalt – Origin, Composition and Need for Testing

Asphalt has several names and forms. The different names by which it is referred to include bitumen, binder, tar, asphalt cement (AC), and performance-graded asphalt cement (PGAC). In Europe the term bitumen is preferred to describe the semi-solid glue used for road paving while in North America the same material is called binder or asphalt cement. The different forms asphalt can take include an elastic solid at low temperatures, a viscous liquid at high temperatures, a semi-solid at intermediate temperatures, or a solid mixture when combined with filler, sand, and aggregates. In this latter state it is often referred to as hot mix asphalt (HMA) or asphalt concrete, the material used for the construction of flexible pavements.

Asphalt cement is defined as a "dark brown or black cementitious material occurring in nature or obtained by crude oil refining", by the American Society for Testing and Materials (ASTM 1998). The major beneficial properties of asphalt come from its adhesive and waterproofing nature. Besides, it is also a strong material under room temperature conditions with a high durability. All of these properties make it a valuable engineering material. Generally, asphalt is a solid or a semi-solid at room temperature and atmospheric pressure, but it can be liquefied by heating or dissolution in petroleum solvents. Also, it can be emulsified in water when appropriate surfactants are added. Naturally occurring asphalts are found in surface deposits in Trinidad and

Venezuela. These natural substances are found within a limestone or sandstone mixture. Though the properties of asphalt are long known, it was not used as a paving material until the early 20th century. This is the period when petroleum refining started and asphalt was obtained as the residue of the refining process. The sticky glue-like material was initially obtained as a waste stream. Hence, this gave rise to a new industry and an extensively used paving material all over the globe.

The use of asphalt has seen a rapid growth since the early 20th century. Starting from a few tonnes, it is predicted to reach a production of about 35 million tonnes by 2020 in North America alone. Currently, there are about 50 major companies that supply asphalt cement in Canada and the USA. About 85% of the total asphalt produced is used in the paving industry. The remaining percentage is used as a roofing material, in adhesives, and in batteries. "About 70 billion lb of asphalt is used annually in the U.S. alone, and asphalt usage will grow dramatically in Asia during the next 10 years," as stated by Arthur M. Usmani, chief scientific officer of Usmani Development Co., Indianapolis, in his book *Asphalt Science and Technology* (Usmani 1997). In all of these major industries, the majority of asphalt has been produced from the residue left after crude oil is refined.

Asphalt has a long history in terms of its applications, dating back to 2000 BC in various ancient civilizations. It has been used as a roofing material and as a binding material to avoid seepage from water storage tanks during that time. Though the application of asphalt has a long history, the testing methods involved to categorize asphalt binders have not evolved with time. Until the recent past, asphalt cements have been graded merely based on a needle penetration test method or based on the viscosity of the binder. These methods may define the physical properties of the asphalt reasonably well, but neither are found to relate to the performance in service. Hence, the most recent testing methods, as developed by the Strategic Highway Research Program (SHRP) funded by the United States government, to grade the asphalt cements were based on testing slender beams in bending at low temperatures (Anderson and Kennedy 1993, Anderson et al. 1994). The test methods developed under SHRP are known as Superpave® test methods, where the name stands for Superior Performing Asphalt Pavements. At first glance, these tests appear to serve the purpose of a clearly-devised method for grading the asphalt. However, the actual road temperature conditions and the duration of cold spells are not taken into consideration, which has become a loophole. Inferior materials are currently being used with often dreadful consequences. Binders that are graded to be the best under Superpave®, can fail miserably with large numbers of cracks showing up within a short period of time (Andriescu et al. 2004, Iliuta et al. 2004, Yee et al. 2006, Bodley et al. 2007). Polymer modifiers are often replaced by cheaper alternatives so that the PGAC passes the test method and fulfills the short-term need for generating profits (Hayner 1999, Collins 2000). However, over the long term, taxpayers could be left to pay for expensive premature failures and excessive cracking.

1.1.1 Composition of Asphalt

Asphalt is composed of ~90-95% crude hydrocarbons containing only carbon and hydrogen, while the remaining ~5-10% is made up of heteroatoms and metals (Romberg et al. 1959). The heteroatoms present are nitrogen, oxygen, and sulphur. These substances interact more with the surroundings resulting in easy oxidation. The oxidation results in hardening of the asphalt, which is commonly known as ageing. The ageing process can also be due to vaporisation of low molecular weight fractions during heating and also due to photodegradation near the surface of the pavement. The metals that are found in asphalt include vanadium and nickel. These metals act as fingerprints sometimes enabling us to identify the source of the asphalt cement. Asphalt can be fractionated and evaluated by two named methods, namely Corbett chromatography, which uses the solubility in asphalt-suitable solvents, and the Rostler method, that uses precipitation techniques (Asphalt Institute 2003).

At a temperature above 60^{o}C, most asphalt behaves like Newtonian liquids, while at room temperature it acts like pseudo-plastics or semi-solids. Asphalt is a visco-elastic material, the behavior of which can be explained by spring and dashpot models. The non-polar high molecular weight components present in asphalt result in viscous behavior, while the polar components are responsible for the elastic behavior of asphalt. High molecular weight content may lead to brittle failure at low temperature conditions, while high polar content gives more flexibility to asphalt (Asphalt Institute 2003). Polymer-modified asphalt also gives better performance at low temperature conditions following a similar trend with polar components. At high temperature conditions such as those found in a desert climate, asphalt acts like a viscous liquid, while at cold freezing temperatures, it undergoes failure like an elastic solid. For better performance of roads, both the polar and non-polar components should be at the right balance (Asphalt Institute 2003).

1.1.2 Need for Testing

Before the Cold War period, and the subsequent rise of the Middle East as a major crude oil producer, the source of crude oil had never changed frequently. However, due to the development of spot market trading, different sources of oil feedstock resulted in the production of asphalt blends with varying properties (Asphalt Institute 2003). This brought about the need for grading and selling the asphalt based on its physical properties and origin, which opened the door for new asphalt testing and evaluation methods. Generally, asphalt is merely a complex mixture and cannot be fractionated as discreet chemical species and, hence, it is difficult to evaluate it chemically, so physical testing methods should be employed.

Conventional testing methods were largely empirical in nature, decided based on the historical field performance of the asphalt. This gave rise to the Superpave® testing system, but it miscalculated the duration of cooling at low temperatures. Hence, an improvement in these methods has become indispensable. This thesis works towards such improvement by validating two recently developed test methods for asphalt cement.

1.2 Design of Pavements

Pavement design can be classified into design of granular bases, binder, and surfaces courses, which are the three layers of the asphalt pavement. The surface course is typically made with a 9 or 12.5 mm maximum aggregate size and relatively high asphalt cement content to provide a smooth ride quality and a surface that is largely impermeable to water (British Standard Institute 2003). The binder course is made with a coarser aggregate of typically 19 or 25 mm and it contains less asphalt cement. The total thickness is constructed to assure that the load is distributed over a large enough area on the base, in order to prevent deformation in the granular material. For heavily trafficked highways the pavement may be constructed with a total of two or more binder courses covered by a single surface course.

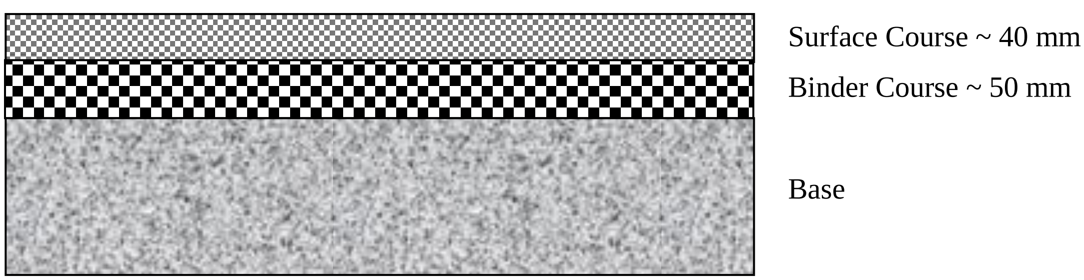


Figure 1. Typical design of three-layered pavement.

In the design, two opposite factors have to be kept at the right balance. The use of large amounts of asphalt cement makes compaction easier but reduces the stiffness, while increasing the aggregate content and size will increase the stiffness but this could lead to segregation. In surface design nowadays, thin surfacing is also tried, but only with the customer's approval. The thickness of each layer is as per the design standards and also depends upon the customer needs. Like in every industry, the pavement standards designed by the research people reaches the common man only through their marketing individuals and intermediate contractors. More than maintenance of standards for lifelong pavements, profit and making pavements to solve the short-term needs become a major concern for these individuals.

1.3 Distress in Pavements

The design of pavements is based on two major properties of materials. The first one is the stress-strain relationship of the material, which decides the critical values of stress and strain to be applied. The different forms of failure modes decide the second one. The two major failure modes of asphalt are fracture and permanent deformation. These types of failure can be caused due to factors like rutting, fatigue, low temperature exposure, and moisture damage. These distress factors are explained in detail below.

1.3.1 Rutting

Rutting is a failure process that occurs in asphalt working under high temperature conditions. It happens due to the strain accumulated in the asphalt pavement. A permanent deformation takes place as a result of this accumulated strain. Rutting occurs primarily under hot climatic conditions where the binder viscosity is reduced and flow can occur under traffic loading, resulting in the formation of ruts (tracks) in the asphalt surface layer(s). The thermal history during storage and transfer and in turn the flow properties of the asphalt play a major role in the rutting of asphalt. Heavy loads with low frequencies are major reasons for the failure of the pavement in this mode. This failure is characterized by complex modulus and phase angle values in the dynamic shear rheometer test (Asphalt Institute 2003). A typical example of a rutted pavement is provided in Figure 2 below.

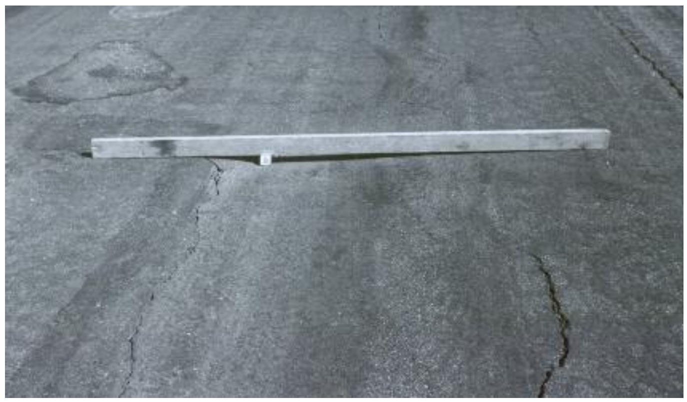


Figure 2. Measurement of rut depth on a severely damaged stretch of pavement.

1.3.2 Fatigue

Fatigue failure occurs in asphalt due to cycling loading by vehicles at moderate and low temperature conditions. While fatigue cracking could be a major mode of failure in thin asphalt pavements, it is hard to quantify easily, as asphalt is a complex substance with different types of aggregates, asphalt cements, modifiers, and air voids. Further, the fatigue testing method itself is expensive and time consuming. Also, the assumption about the loss of modulus from the original is a bit confusing factor which determines the fatigue failure. DSR is used extensively to measure fatigue failure in asphalt. Fatigue failure can be measured from the loss modulus obtained through this experiment. A typical example of a pavement area with fatigue cracking distress is provided in Figure 3 below. However, it should be noted that this type of cracking is not often seen due to early repair strategies and thick pavement designs thus minimizing the severe manifestation of this type of distress.

Figure 3. Area of asphalt pavement suffering from severe fatigue cracking distress.

1.3.3 Low Temperature Cracking

Low temperature cracking is a mode of failure occurring in asphalt pavement due to the stresses induced in the layers of asphalt under low temperature conditions. Low temperature cracking can be due to small fatigue cracks that sprout transverse and longitudinal cracks during subsequent low temperature exposures or due to the hardening of asphalt at low temperatures providing so-called single event thermal cracks. The surface layer and also the layers below them, shrink causing transverse cracking of the asphalt. It is not merely a surface phenomenon and hence merely sealing of the cracks won't help. Shrinkage and cracking of the layers beneath can result in a complete destruction of the road surface. The failure modes can sometimes be brittle. It is due to various factors like weather conditions, and aggregate properties. The properties of the binder play a major role in causing this failure. A thick and strong pavement will not solve the problem of thermal cracking but will lessen the consequences for cracks that are already present. Thick pavements allow less movement of the base and hence the cracked road surface will deteriorate less compared to a thinner pavement.

Thermal cracking occurs as a result of three different phenomena (Yee et al. 2006):

- A sudden drop in temperature below the limit that causes stress build-up above the strength of the mix. This is referred to as single-event cracking.
- The loss of strength of asphalt mix as a result of repeated thermal stresses below the temperature limit. This is known as thermal fatigue cracking.

- Freeze and spring thawing cycle with addition to heavy traffic and repetitive loading. This is known as mixed load/thermal distress cracking.

Typical examples of thermal cracks are provided in Figure 4 below. This thesis is mainly concerned with the validation of tests methods that prevent the occurrence of thermal cracks. It is obvious from the image in Figure 4 that thermal cracks, once formed, can have a serious negative effect on the pavement life-cycle and that they are difficult to repair. It has been found that for pavements of apparently the same design (asphalt cement grade, thickness, traffic level, and weather conditions) the cracking distress can vary a great deal. Hence, it is important to better grade the asphalt cements in order to control the onset and degree of thermal cracking distress.

Figure 4. Typical example of a severe thermal crack in an asphalt pavement.

1.3.4 Moisture Damage

Moisture damage is caused due to two major phenomena namely the stripping or lack of adhesion and lack of cohesion between the layers. Air voids and lack of proper compaction are major reasons for moisture damage in asphalt. Stripping occurs between the asphalt cement and the aggregate particles in the presence of water. A lack of cohesion between the asphalt layers, allowing water to seep in, is also a major concern for failure. This is because the aggregates are attracted towards water while the asphalt cement is hydrophobic in nature. Thus, the water wets the aggregates forming a layer between the asphalt cement and aggregates which results in stripping. Proper drainage, the use of anti stripping agents, and proper compaction prevents moisture damage.

1.4 Asphalt Cement Specification Grading

Asphalt cement is graded based on the temperature conditions at which it should be able to perform without any failure. The low temperature at which asphalt cement is graded relates to the one-in-fifty-year lowest pavement surface temperature that the pavement is expected to experience (98% confidence needs to be obtained that in a given winter the pavement is not exposed to damage). The person designing a certain road can find these surface temperatures in the Long Term Pavement Performance binder selection software (LTTPBind® (FHWA 1999)), which has a database of weather data from which the surface temperatures are calculated through a calibrated equation that relates the air to surface temperatures based on the latitude of the location. A 98% confidence limit is supposed to be used because thermal cracks are very hard to repair once present. Further, an error of only a few degrees can easily reduce the confidence level to less than 50%. The current specification sets a limit on the creep stiffness and the slope of the creep stiffness master curve to limit damage at the pavement design temperature (Asphalt Institute 2003).

The high temperature grade is based on limiting rheological properties at the average highest 7-day surface temperature for an average summer in the contract location. The high temperature grade is doing a reasonable job for two reasons. The rheological properties relate better to rutting than those of the BBR relate to cracking. Further, the high temperature performance is influenced to a large degree by the properties of the aggregate (size, gradation, content, etc).

The Superpave® grading method is based on the test methods performed on the binders which are supposed to directly relate to the performance of the binders (Anderson and Kennedy 1993). Superpave® was supposed to be an entirely performance-based specification method but recent evidence has shown that it is more like a purchase-based specification (where the supplier and user of the asphalt cement agree that the product meets the specification criteria but there is no guarantee for performance in service) (Robertson 1995, Anderson 1998, Gavin et al. 2003, Yee et al. 2006, Zhao and Hesp 2006, and others).

However, unlike conventional methods the Superpave® tests are *less* empirical in nature, and the properties measured are supposed to be related to the actual performance of the roads. As an example, PGAC 58-28 asphalt cement is expected to perform at a high temperature limit of 58ºC and low temperature limit of -28ºC, without any significant amount of either rutting or thermal cracking. As stated, there are not many problems with the high temperature grading test in that it is reasonably able to predict rutting performance. However, the time given for the samples to condition in the low temperature test is very much inadequate. Consequently, many pavements in northern climates are under-designed for thermal cracking. Hence, this research was conducted to investigate improved asphalt cement grading tests.

Chapter 2

Background on Asphalt Cement Performance Grading

2.1 Conventional Testing Methods

Asphalt has been regarded as a construction material by the civil engineer and not as a macromolecule with visco-elastic properties. Until the early 1990s it has been graded based on needle penetration, viscosity and softening point, which are purely empirical in nature (Anderson et al. 1994). These test methods grade the asphalt cement only based on field performance. Also, these test methods never focus on two distinct properties of asphalt at the same time but rather measure an empirical property of the asphalt cement at a somewhat arbitrary single temperature.

The most common old test methods are the following:

- Needle Penetration (Pen);
- Ring and Ball Softening Point ($T_{R\ \&\ B}$); and
- Viscosity.

There are four common conventional grades of bitumen, namely penetration grade, oxidized grade, hard grade and cutback grade. Of these, only penetration grade is used for paving roads. All these grades are decided by the combination of any of the above two test methods or just one. Penetration grade is decided from the test by both needle penetration and ring and ball softening point method. Oxidized and hard grades are decided by the combination of both viscosity and softening point method, while the cutback bitumen is decided by the viscosity method. Hard and oxidized asphalts are used for roofing and painting purposes while the cut back bitumen is used for blending and also for surface coating applications that are mostly industrial in nature. The procedure for conventional test methods is discussed briefly in the following passages.

2.1.1 Needle Penetration Method

The needle penetration method involves the piercing of a needle at a standard temperature (usually 4ºC or 25ºC), for a standard time (usually 5 s), under a standard load (usually 100 g). The needle penetration is measured in units of decimillmeter (0.1 mm). For instance, 80 Pen asphalt cement will have a needle penetration at 25ºC of 8 mm (80 times 0.1 mm). The reproducibility and repeatability standards are specified in the ASTM standard (ASTM 1998).

The penetration test does not relate to any physical material property of asphalt, but gives a value from which the grade is decided based on historical field performance. The major advantage of this method is that it is inexpensive and gives a better result than the high temperature viscosity method in terms of low temperature performance. Two asphalts of the same penetration grade can have different viscosities at a different temperature, which gives them different viscosity grades (Figure 5) due to the temperature susceptibility of the asphalts.

2.1.2 Ring and Ball Softening Point Method

This test method consists of a steel ball and the asphalt sample is heated and softened in the medium of glycerol or water. The softening point test is a kind of reinforcement for the results obtained by the penetration method. Until before the early 1990s these are the only test methods present to grade the asphalt. In this test method, the hard asphalt sample is converted into a soft substance and temperature is measured at that point. The temperature at which the steel ball in the ring pushes the soft bitumen samples to a standard distance is measured as the softening point.

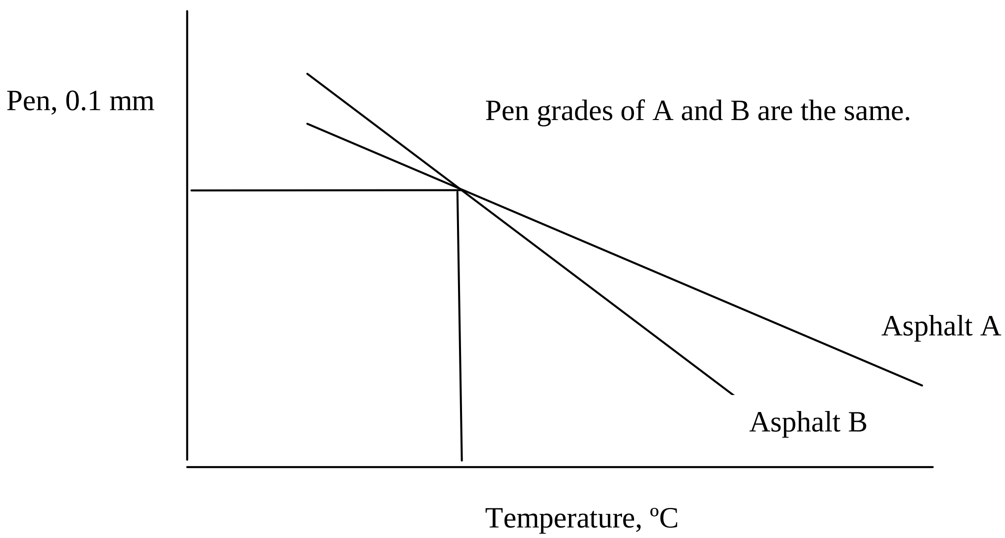


Figure 5. Variation of penetration with temperature.

2.1.3 Viscosity Grading Method

Different types of viscometers are used to measure the viscosity grade of asphalt. Viscosity is measured in two different ways. In the first method, viscosity of asphalt is measured at a temperature of 60ºC from which the grade is decided. In the second method, the asphalt aged in RTFO is graded by absolute viscosity method again at 60ºC. But, the change in properties of asphalt with temperature and its non-Newtonian behavior add up to the viscosity change which is shown in the Figure 6. Temperature susceptibility is the major drawback of the viscosity method even though it measures a material property of asphalt.

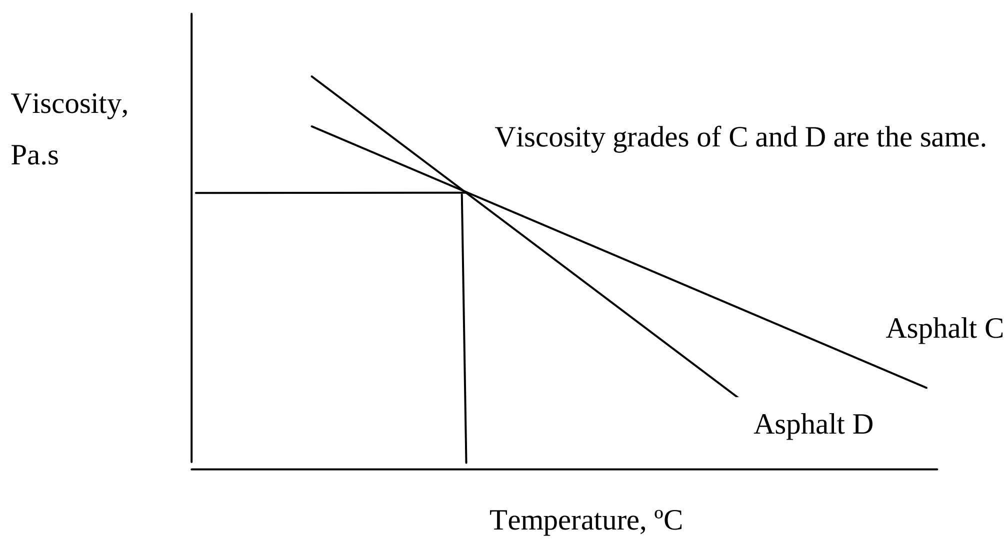


Figure 6. Variation of viscosity for same viscosity grades at different temperatures.

2.2 Superpave® Test Methods

Superpave® test methods were developed by the Strategic Highway Research Program (SHRP) funded by the US government during the late 1980s and early 1990s (Anderson and Kennedy 1993, Anderson et al. 1994). The research team that developed these test methods set out to eliminate the empirical nature of the conventional grading methods such as penetration and ring and ball softening point.

Based on the Superpave® test methods, asphalt is given a grade like PG (or PGAC) XX-YY, where XX represents the high temperature working limit of the asphalt cement and -YY the low temperature limit. The two important test methods applied in Superpave® testing to determine XX and -YY are the following (Asphalt Institute 2003):

- Dynamic Shear Rheometer (DSR) Test for XX, and
- Bending Beam Rheometer (BBR) Test for -YY.

DSR gives reasonably accurate results for failure properties of asphalt like rutting and fatigue at high temperatures and medium temperatures. However, while the BBR test method is assumed to give reasonable results for low temperature cracking, widespread problems with premature and excessive cracking have shown that the method is in need of improvement.

2.2.1 Dynamic Shear Rheometer (DSR) Method

Rutting and fatigue failure in asphalt are characterized by the DSR test method (Asphalt Institute 2003). These distresses mainly relate to the high temperature rheological properties of asphalt. The test method involves measurement of visco-elastic properties in dynamic oscillation of a thin film of asphalt cement between parallel aluminum plates. The binder sample is sheared and the shear angle and torque are measured from which the dynamic properties of asphalt are measured. The dimensions of the circular plates used are 8 mm (for higher stiffness conditions) or 25 mm (for lower stiffness conditions) and the gap between them is 2 mm.

The sample placed between the parallel plates is subjected to a shear force in the form of torsion and the shear modulus is measured at 25ºC and a frequency of 10 rad/s. Three different temperatures are employed for the testing and a master creep curve is typically generated to correlate with theoretical models. The phase angle (δ) or loss tangent ($\tan \delta$) is also measured since it relates to the ratio of the viscous (G”) over the storage or elastic modulus (G’). Phase angle relates to thermal cracking at low temperatures and to rutting at high temperatures.

2.2.2 Bending Beam Rheometer (BBR) Method

Low temperature cracking of asphalt is a major distress form for asphalt pavements that should have been identified by the BBR specification test. The specification criteria for passing/failing the BBR test are given in AASHTO standard M320 (AASHTO 2002). In this test method, both the testing temperature and conditioning temperature are the same. The binder samples are conditioned for one hour at room temperature and then conditioned at -10ºC and -20ºC in refrigerators for one hour before testing. Low temperature grades of asphalt are decided after the one-hour test.

The test involves loading the specimen in three-point bending at a temperature of 10 degrees above the design temperature of the pavement. A photograph of the test set-up is provided in Figure 7. This 10ºC difference is used to reduce the testing duration of the binder sample from 2 hours to 60 seconds, assuming that the time-temperature superposition is valid (Anderson et al. 1994, Basu et al. 2003). The condition for a pass is when the stiffness value is below 300 MPa and the creep rate or slope of the creep stiffness master curve (m-value) is greater than or equal to 0.300. If either the stiffness is above 300 MPa or the m-value is below 0.3 then the material fails the specification and it can only be used in a warmer climate location.

Recent investigations on a large number of pavement trials and regular contracts have shown that the current AASHTO M320 specification is flawed (Andriescu et al. 2004, Iliuta et al. 2004, Zhao and Hesp 2006, Yee et al. 2006, Bodley et al. 2007). Pavements of the exact same low temperature grade can show vast differences in low temperature fracture performance (Anderson et al. 1999, Button and Hasting 1998, Zhao and Hesp 2006, Bodley et al. 2007). Hence, there is a need for improved performance grading methods. Increasing the conditioning time prior to three-point bending tests is one approach taken in Ontario's recently developed LS-308 method on extended BBR testing (MTO 2007a). This thesis investigates the validity of this new method by comparing the field distress of a large number of regular contracts and pavement trial sections with the predicted AASHTO M320 and LS-308 grades.

In addition to the insufficient physical ageing (conditioning) of the asphalt cement, the AASHTO M320 has problems that relate to the absence of a proper chemical ageing method and the absence of true failure tests (BBR and DSR are rheological tests that only measure in the low strain regime whereas thermal cracking is a high strain phenomenon).

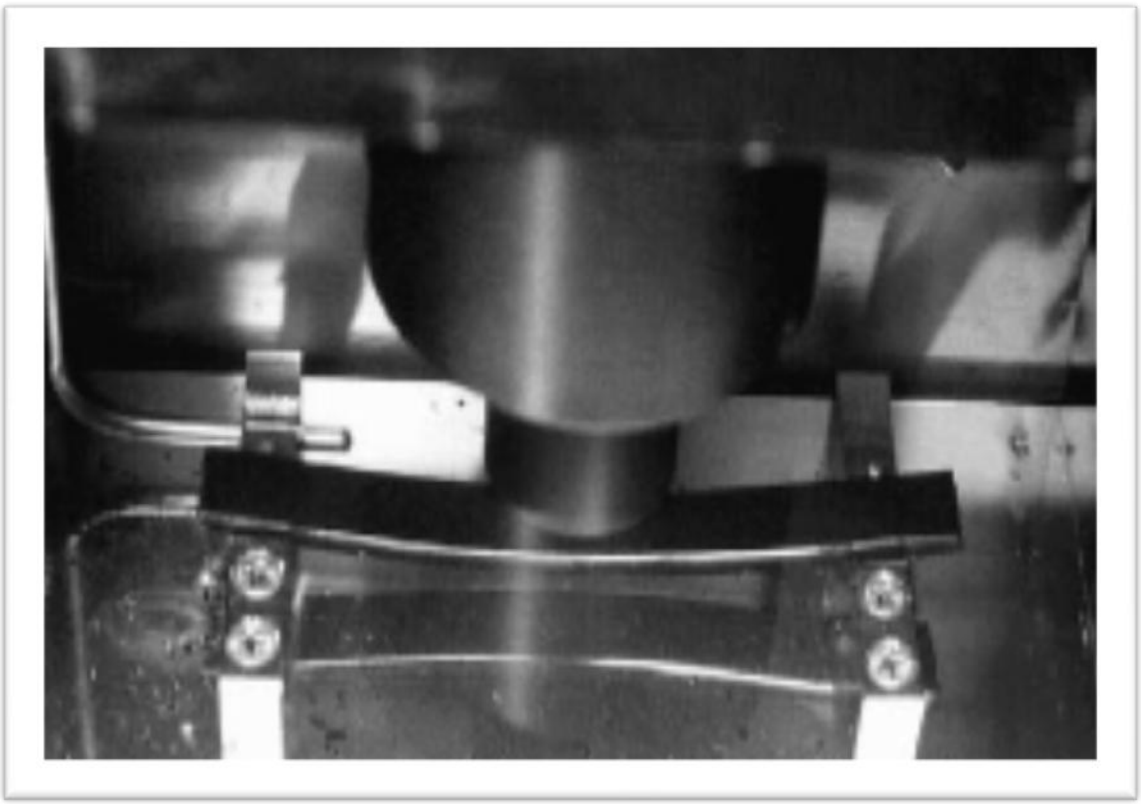


Figure 7. Bending beam rheometer test set-up.

2.3 Improved Low Temperature Specification Testing

Two test methods have recently been developed in Ontario to address the inadequacies of the AASHTO M320 specification (Andriecu et al. 2004 & 2006, Zhao and Hesp 2006, Bodley et al. 2007, Hesp et al. 2009):

- Extended BBR test - LS-308 (MTO 2007a); and
- DENT test - LS-299 (MTO 2007b).

The effects of isothermal conditioning on the low temperature rheological properties of asphalt cement have been studied by a significant number of researchers starting in the earlier part of last century. Traxler (1936, 1937, 1961) cautioned that physical (reversible) hardening mechanisms need to be taken into consideration when measuring rheological properties at low temperatures in asphalt cement. Struik (1970) stated that it is useless to study properties like creep and stress relaxation without considering the physical (reversible) ageing phenomenon. More recently, research at Queen's University has considered the physical ageing phenomenon in great detail and found it to be an important indicator of thermal cracking performance (Zhao and Hesp 2006, Hesp et al. 2007, Bodley et al. 2007, Hesp et al. 2009).

Additional research at Queen's University has produced the double-edge-notched tension (DENT) test to study the ductile failure mechanisms in asphalt cement and hot mix asphalt under high strain conditions (Andriescu et al. 2004, Andriescu 2006, MTO 2007b). It was found that the approximate critical crack tip opening displacement, as determined in the DENT protocol, shows a good correlation with fatigue distress (Andriescu et al. 2004, Andriescu 2006).

The newly developed test methods give a consistently better ability to predict either good or poor performance compared to the regular BBR protocol, LS-299 and LS-308.

2.3.1 Extended BBR Testing According to MTO Method LS-308

A detailed description of the test method will be given in the experimental section. However, the extended BBR test protocol conditions samples at two different temperatures for three different times (MTO 2007a). The two conditioning temperatures related to the design temperature of the pavement: $T_1 = T_{design} + 10$ and $T_2 = T_{design} + 20$. These two temperatures were chosen because over the total 20ºC range it is expected that the physical ageing tendency peaks (at very low temperatures, such as T_{design}, the molecules cannot move while at higher temperatures above $T_{design} + 20$, the thermodynamic tendency for waxes to crystallize and asphaltene molecules to precipitate is absent). The method is not designed to perfectly correlate with low temperature cracking distress but rather to provide a high degree of confidence that thermal cracking is completely avoided. For instance, roads that spend most of their time at a relatively warm $T_{design} + 20$ would need to be protected from a cold spell at $T_{design} + 10$, even though such cold spell may not occur during the early life of the contract.

Regular tests to determine a pass and a fail temperature according to AASHTO M320 criteria (S=300 MPa and m=0.3) are done after one hour of conditioning, one day of conditioning, and three days of conditioning (hence the method is called an extended BBR test). Grade temperature and subsequent grade losses are calculated at the end of each conditioning period. The worst grade loss is the warmest minus the coldest limiting temperature (where S reaches 300 MPa or m reaches 0.3). As will be shown in the results and discussion section, the test method provides 95% accuracy and is also easy to repeat and reproduce, as the method is merely an extension of the BBR test method with slight modifications.

2.3.2 Ductile Failure Testing According to MTO Method LS-299

The procedure for LS-299 will be discussed in detail in the experimental section. The test method involves pulling of a notched binder sample until it fails in a water bath (MTO 2007b). The temperature maintained at the water bath is 15ºC. The ligaments (distances between two opposing notches) are 5 mm, 10 mm and 15 mm in length. Conditioning was done at room temperature for 24 hours and then the binder samples were conditioned in the water bath for half an hour before testing. Second sets of tests were conducted where the samples were conditioned for 24 hours at 15ºC in the water bath prior to testing. A photograph of the test set-up is provided in Figure 8 below.

The test method mainly records the failure energy and the peak force. From the peak force at a 5 mm ligament length, the crack tip opening displacement (CTOD) is calculated. The CTOD value accounts for the strain tolerance in ductile failure which should relate accurately to the tendency for cracking distress. A lower value of CTOD corresponds to a worse condition of the roads. As will be discussed in the results and discussion section, this test method provides an approximate 85% accuracy.

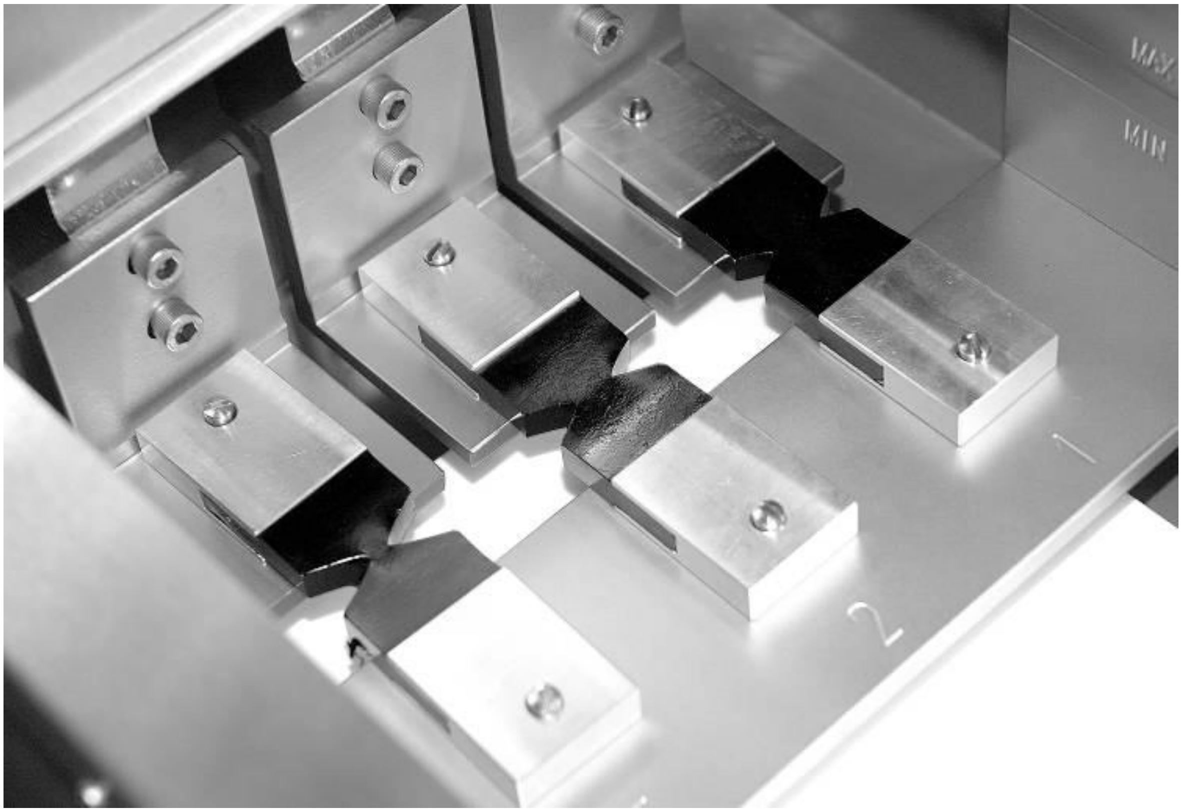


Figure 8. Double-edge-notched tension test set-up.

Chapter 3

Materials and Experimental Procedures

3.1 Materials and Recovery

3.1.1 Eastern and Northeastern Ontario Contracts

The twenty contracts investigated in eastern and northeastern Ontario are listed in Table 1. A map of all contract locations is given in Figure 9. Of these, five were constructed with penetration graded asphalt cements while fourteen were graded based on Superpave® test methods. The grade of site O from Wilno is not specified and could not be identified from the documentation available for this rather old contract. Among these samples, eleven are superior performing ones with the least amount of cracking distress. Nine contracts are in very bad shape with massive cracking requiring sealing and early rehabilitation. Surveys were done for lengths of 250 meter each for long life pavements while for poor ones it is done thrice for 100 meter lengths. Transverse cracking alone is considered while it is likely that longitudinal cracking can also be the result of low temperature exposure. The cracks were categorized as half lane, full lane, and full width (i.e., two lanes). Cracks in half lanes were ignored for the poorly performing contracts while they were taken into account for the good performing contracts. Sealing is also taken into account while considering the cracks in the road.

Table 1. Eastern Ontario Contract Details

Site	Hwy	Location	Age	LTPPBind® temperatures (98% C)	Spec Grades C/Pen	
					Binder course	Surface Course
A	6	Little Current	2000	-29.3	58-34	58-34
B	11	Cochrane	1999	-37.8	52-34	52-34
C	11	Smooth Rock	1998	-36.8	52-34	52-34
D	17	Petawawa	1996	-32.5	85/100	85/100
E	28	Burleigh Falls	1993	-29.2	300/400 (30)	150/200
F	28	Lakefield	97/98	-29.2	52-34(25)+ 64-34	58-28
G	33	Conway	1998	-25.9	-	58-28
H	35	Lindsay	1997	-31.0	85/100	85/100
I	41	Dacre	2000	-31.7	58-34	58-34 PMA
J	41	Denbigh	1996	-33.6	300/400 (35)	150/200
K	41	Kaladar	1999	-33.7	58-34	58-34
L	41	Northbrook	2000	-33.6	58-34 M	58-34 M
M	41	Vennachar	97/98	-33.6	58-34 PMA	58-34 PMA
N	60	Bat lake	1998	-34.8	58-34	58-34
O	60	Wilno	1994	-34.4	-	-
P	62	Bannockburn	1997	-33.6	300/400 (30)	300/400 (30)
Q	62	Bloomfield	1993	-27.7	150/200 (20)	150/200 (20)
R	138	Cornwall	00/01	-28.7	58-34 PMA	58-34 M
S	138	Monkland	1998	-31.5	58-34	58-34
T	416	Spencerville	1999	-29.0	64-34	64-34

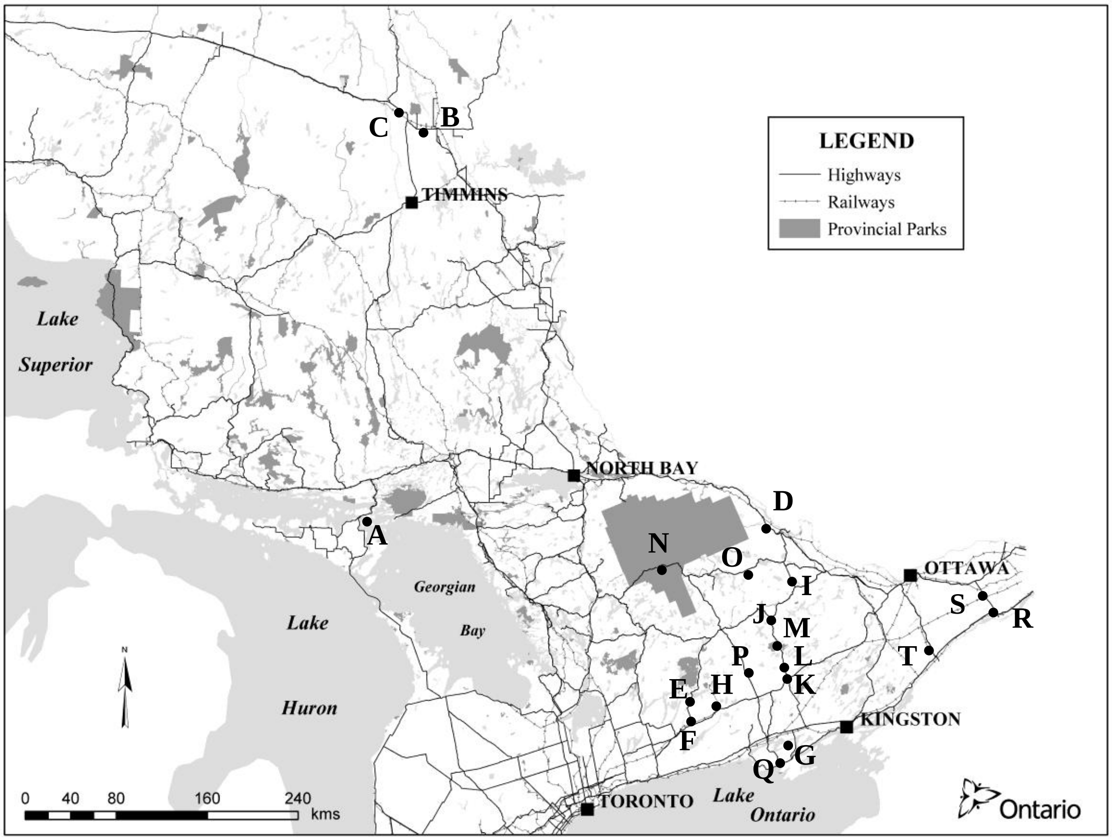


Figure 9. Contract locations for eastern and northeastern Ontario sites investigated.

3.1.2 Highway 655 Trial Sections

The asphalt cement details for the Highway 655 pavement trial are given below in Table 2. SBS stands for styrene-butadiene block copolymer, Ox stands for oxidized asphalt and PPA stands for polyphosphoric acid. RET stands for reactive ethylene terpolymer while P^{31} stands for an additive containing phosphorous (likely PPA and/or zinc dialkyldithiophosphate, which is commonly found in waste engine oils). G* is the complex modulus and δ is the phase lag between stress and strain in a dynamic rheological test (Asphalt Institute 2003).

Table 2. Highway 655 Trial Section Details

Section	Modifier	Base AC	PGAC Grade, ºC	$G^*\sin\delta$ (16ºC), kPa	$T_{G^*\sin\delta = 5\text{ MPa},}$ ºC
1	RET+PPA	Lloydminister	65-36	2218	9.8
2	Ox + SBS	unknown	65-36	2588	10.1
3	SBS	unknown	65-36	1954	7.8
4	SBS + P^{31}	unknown	67-35	2226	9.1
5	SBS	Western Canadian	66-35	2273	9.7
6	Ox + P^{31}	unknown	59-35	1820	7.1
7	P^{31}	unknown	54-35	1542	6.7

3.1.3 Recovery of Asphalt Cements

The surface layers of the asphalt core samples were separated from the bottom layers by a diamond-tipped cutting saw. About 4 kg of each surface layer was soaked in 4-6 L of tetrahydrofuran (THF). The extract was removed periodically and stored in a separate container for particles to settle down at the bottom. The asphalt cement was washed until about 200 g of binder recovered and the solution was relatively clean when compared to the beginning. The solvent was removed by condensation method using a rotary evaporator. The binder sample was then heated at a temperature of 150ºC and a vacuum pressure of about 40 mm Hg to ensure complete removal of solvent from the binder. As much as about 180 g of asphalt binder was collected by this method. This was the standard method of recovery used for all the asphalt binders from regular contracts as well as trial sections.

3.2 Experimental Procedures

3.2.1 Regular BBR Testing According to AASHTO Method M320

Asphalt cement specimens were poured into rectangular beams and left to cool for approximately one hour at room temperature. The beams were then cooled at the testing temperature for an additional one hour prior to testing in three point bending. Figure 11 provides a schematic of the aluminum mold and the dimensions of the asphalt beam.

The beam specimens were loaded in three-point bending with a load of approximately 1000 mN for a period of 240 s. The creep stiffness (S(t)) and the slope of the creep stiffness master curve (m(t)) were calculated at a loading time of 60 s and the warmest temperature at which S(60 s) reached 300 MPa or the m(60 s) reached 0.3 was recorded as the limiting grade temperature. The limiting S and m temperatures were determined by interpolation between a pass and a fail temperature which were chosen at 6ºC intervals. All samples were tested in duplicate and the reproducibility was generally found to be excellent.

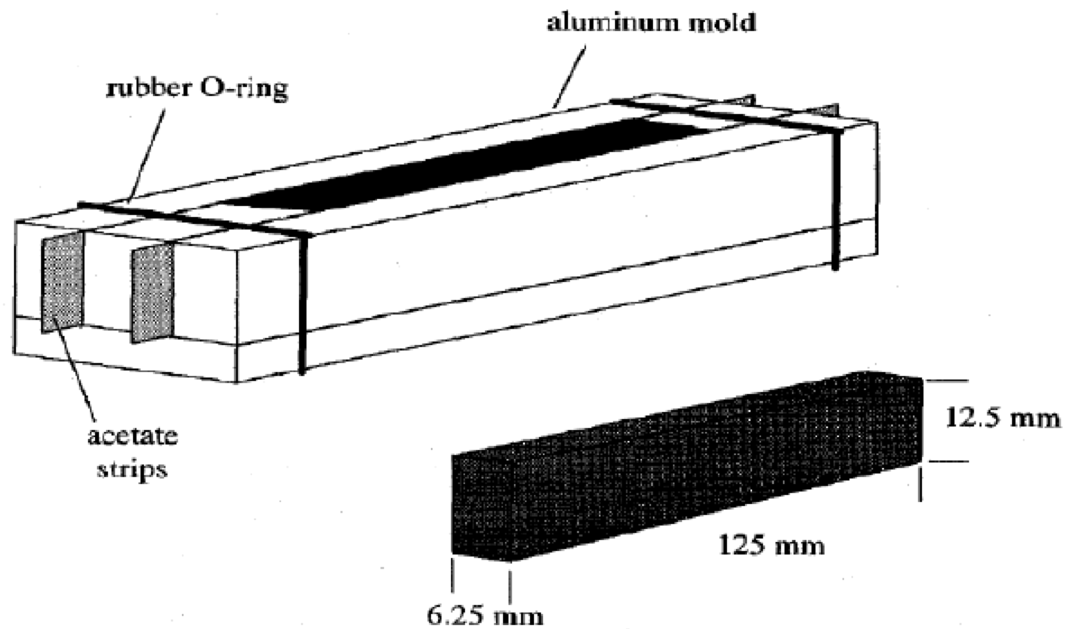


Figure 10. BBR test mould schematic and sample dimensions.

3.2.2 Extended BBR Testing According to MTO Method LS-308

For recovered asphalt, a total of 12 BBR beams were poured at approximately 150ºC. The beams were conditioned for one hour at room temperature followed by one, 24 and 72 hours at either T_{design} + 10 and T_{design} + 20 (six beams for each conditioning temperature (MTO 2007a), where T_{design} is the pavement design low temperature for the contract location according to the Long Term Pavement Performance binder selection software, LTPPBind®, v. 2.1 (LTPPBind® 1999)).

The beams were tested at two different temperatures after each conditioning period. The test temperatures were chosen so as to determine a pass and fail temperature according to regular AASHTO M320 criteria (S (60) = 300 MPa or m (60) = 0.3). The limiting temperatures were recorded in a table from which the maximum (worst) grade loss is calculated. The worst grade after three days of conditioning is then recorded together with the worst grade loss. The worst grade temperature was determined at -10ºC from the warmest limiting temperature where either S(60) = 300 MPa or m(60) = 0.3. The shift of 10ºC accounted for the application of the time-temperature superposition principle.

3.2.3 Ductile Failure Testing According to MTO Method LS-299

The double-edge-notched tension test was developed to control fatigue cracking in asphalt pavements (Andriescu et al. 2004, Andriescu 2006, MTO 2007b). The specimen is poured in between two aluminum inserts with three different notch depths (ligaments between notch tips include 5, 10 and 15 mm). Figure 11 below shows a photograph of how the molds are filled. Samples are slightly over-filled in order to accommodate some minor shrinkage during cooling. Figure 12 shows the specimen dimensions while Figure 13 shows a photograph of a sample after failure. The arrow indicates the ductile zone where most of the non-essential work is dissipated. As many as three sets are poured for each binder sample to ensure a good repeatability.

The method extrapolates the specific total work of fracture (area under the force-displacement curve divided by the cross sectional area of the ligament) versus the ligament length graph to a zero ligament to obtain the essential work of fracture, w_e. The essential work of fracture represents the work done to pull a very thin filament of asphalt cement apart just like it would occur in between two large aggregate particles.

The essential work of fracture is subsequently divided by the net section stress in the smallest ligament (5 mm) in order to arrive at an approximate critical crack tip opening displacement (CTOD). The approximate CTOD provides a measure of strain tolerance in the ductile state in the presence of significant constraint as it would occur during ductile failure in the hot mix asphalt.

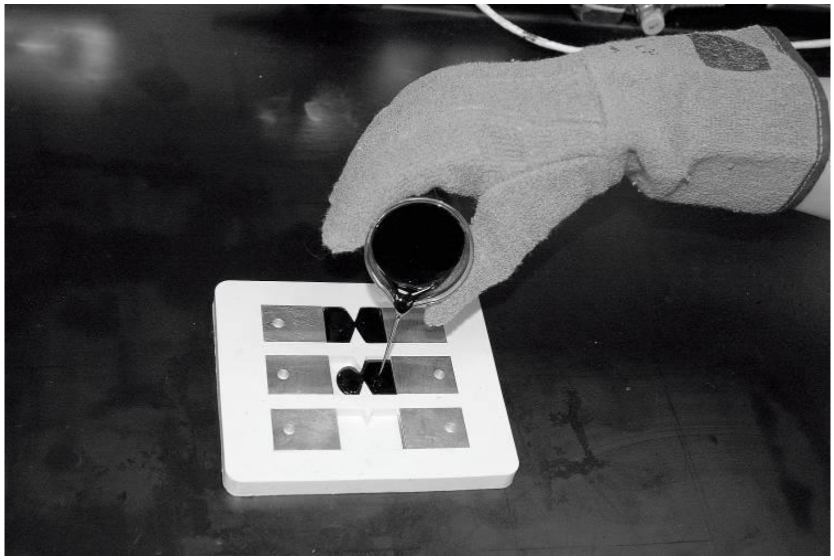


Figure 11. DENT test sample preparation.

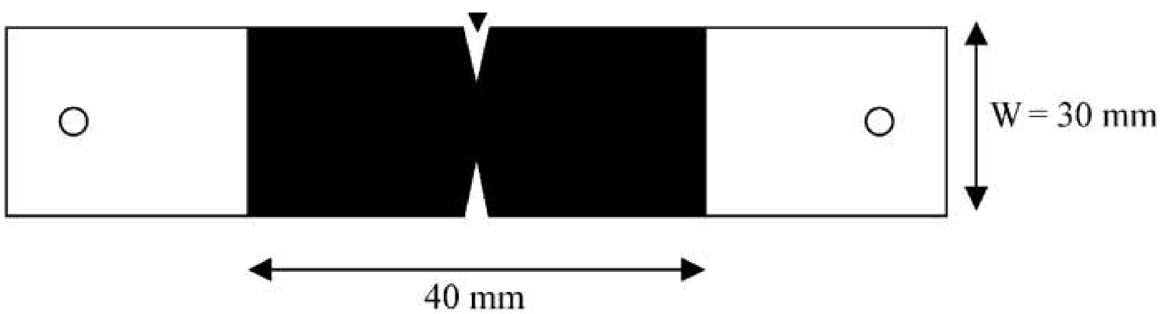


Figure 12. Dimensions of DENT test sample.

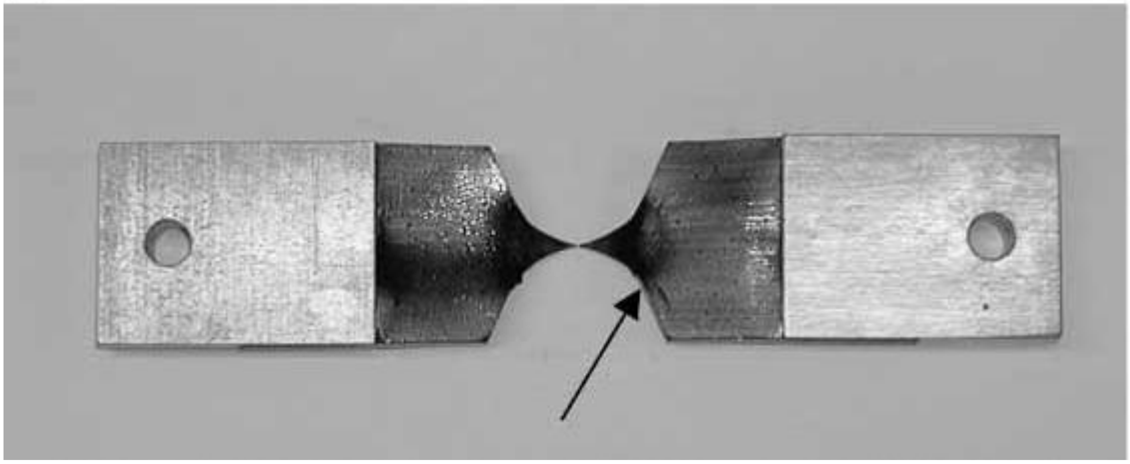


Figure 13. Representation of failed sample in DENT test (Andriescu et al. 2004).

The tensile machine was set at a speed of 50 mm/min for all samples tested in this study. This test speed consisted of a compromise that allowed a test to be finished within a reasonable amount of time since typical ductile failure occurs over much slower speeds and longer time periods (Andriescu and Hesp 2009). For a detailed description of the exact procedures the reader is referred to LS-299 (MTO 2007b).

3.2.4 Elastic Recovery Testing

The elastic recovery test is a simple extension of the regular BBR test. The recovered samples are heated for an hour in hot air oven at 150°C before being poured into BBR molds. The specimens are allowed to sit for one hour at room temperature and are placed in three different refrigerators. Nine specimens are poured for this purpose.

Three specimens are placed with appropriate time gaps at each of -30°C, -20°C, and -10°C refrigerators, respectively, for conditioning. The conditioning is done for three hours. After three hours, samples are placed one by one, in the BBR machine for testing. The test temperature is same as that of the conditioning temperature.

The BBR is set to -30°C prior to pouring of the sample. After calibration, the specimen is conditioned for five minutes prior to testing. A preload is given for 10 seconds for seating of the sample after which the actual load is applied. The bending load of 980 mN is applied for a period of 240 s followed by a 25 mN load during a 720 s recovery period. The recovery of the sample is measured in terms of the displacement curve. The binder sample recovers due to the elastic components (recoverable and delayed elastic) while the non-recoverable part reflects the viscous part of the displacement at 240 s. The 720 s unloading period was selected to make sure that the displacement no longer changes after this period, i.e. all the delayed elasticity has recovered. A typical example of displacement versus time graph is shown in Fig. 14.

The entire data of force and displacement is exported to an Excel file for further processing. The S and m-value at 60 seconds were also noted and used for the purpose of repeatability and reproducibility testing of the previous BBR tests. After testing, the specimens were placed back at -30°C. The BBR machine is set to -20°C to test the next set of three samples. The changing of temperature takes about 20 minutes and the tests were repeated with the same procedure. The samples stored at -10 °C were tested in an identical fashion. The samples are placed back into the respective fridges after testing. After three day conditioning, the test were repeated on all of those samples and the recoveries were noted.

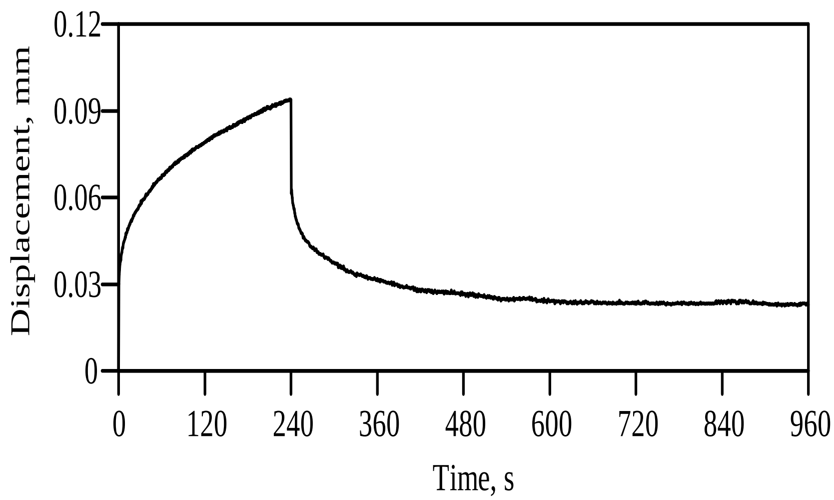


Figure 14. Sample graph of elastic recovery test data.

Chapter 4

Results and Discussion

4.1 Eastern and Northeastern Ontario Contracts

4.1.1 Distress Surveying

The distress surveys were all conducted in the fall of 2007 (Hesp et al. 2009, In Press). Detailed cracking locations were recorded, and photographs were taken of typical cracks. Crack lengths were calculated from the average of three locations (beginning, middle, and end) in each contract. Only the transverse cracking is considered in this thesis. However, many contracts showed significant longitudinal distress which is considered to be due to a combination of thermal and traffic stresses (Iliuta et al. 2004, Wistuba et al. 2006, Bodley et al. 2007).

The cracking severity versus the pavement age for all the 20 contracts is provided in Figure 15. The data indicate a large difference between the better performing contracts and the poor performers. There is bound to be some error in the cracking distress data from Figure 15 since only small sections of the pavement were surveyed, but the general picture is clear. Asphalt pavements of identical designs subjected to similar climatic and traffic conditions can perform on opposite sides of the spectrum – from extremely poor to exceedingly well.

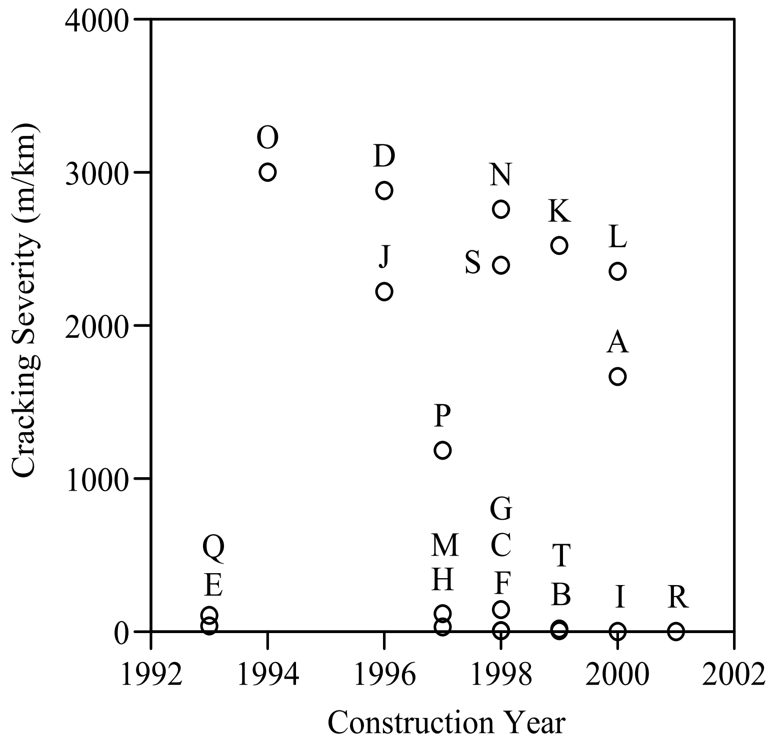


Figure 15. Cracking distress versus construction year for 20 eastern and northeastern Ontario pavement contracts constructed between 1993 and 2000.

(A) Highway 6 north of Little Current. The contract covered 16 km through varied terrain with a relatively low volume of commercial traffic (Annual Average Daily Traffic Count (AADT) = 3560 with 11.2% commercial vehicles for 2007). This stretch of highway was reconstructed in 2000 and cracked prematurely and excessively in the first few years of service. The distress was so pervasive that crack sealing was never an option. The pavement condition index (PCI) had fallen to 79 in 2007 and is expected to deteriorate rapidly in years to come due to the extensive low temperature cracking.

The recovered asphalt cement was tested in a nuclear magnetic resonance (NMR) spectrometer, which hinted at the presence of phosphorous-containing compounds soluble in tetrahydrofuran (THF). The material was also tested with an X-ray fluorescence spectrometer which indicated the presence of zinc. Zinc is never found in straight asphalt cement but does occur in abundance in waste engine oils. The presence of waste engine oils and the likely presence of polyphosphoric acid (PPA) may be partly to blame for the premature and excessive cracking (Kodrat et al. 2007).

(B) Highway 11 west of Cochrane. This 30 km section of Highway 11 was reconstructed in 1999 and has performed remarkably well given that it is one of the most northerly contracts investigated. Joints, shoulders, and areas over culverts have remained free of serious cracking distress. The area is exposed on regular occasions to air temperatures below –40ºC and experienced a record low of nearly –50ºC as logged on a nearby pavement trial in early 2004 (Bodley et al. 2007).

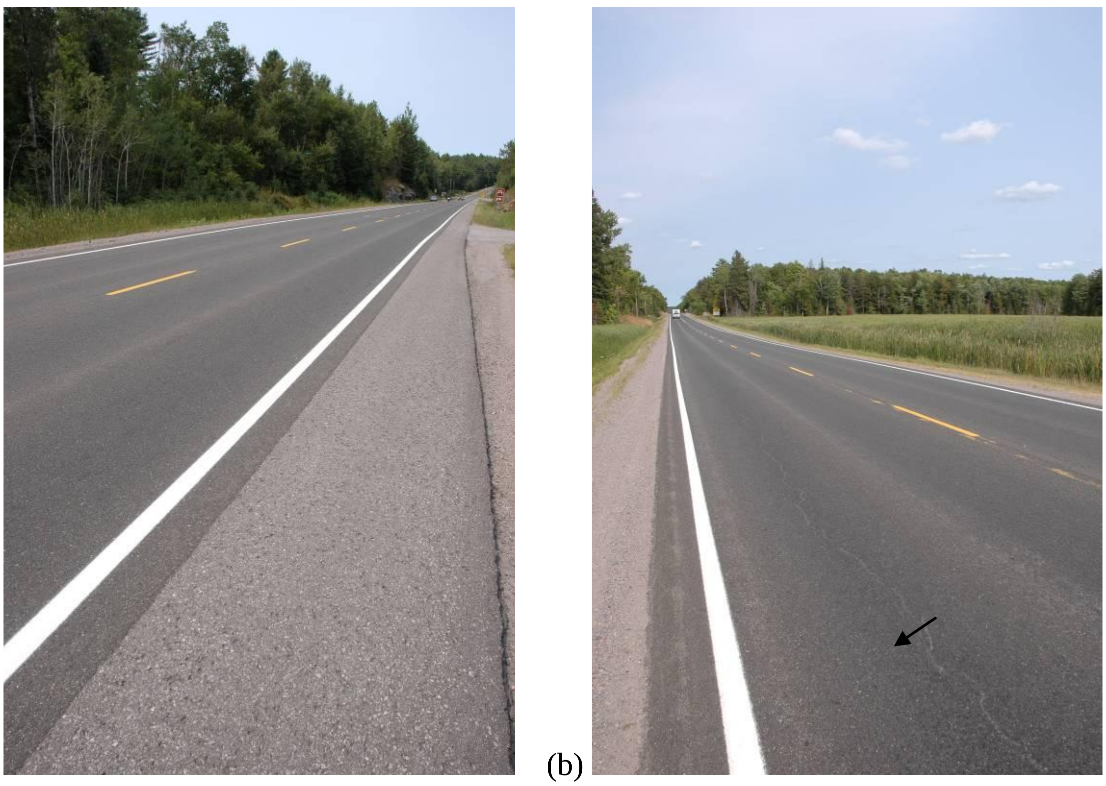
(C) Highway 11 east of Smooth Rock Falls. The 26 km stretch of Highway 11 to the west of the previous contract is also in relatively good condition. It was reconstructed in 1998 and has very good ride and pavement condition ratings in spite of its slightly older age but this may be due to a slightly lower traffic level. The asphalt cement came from the same supplier as that used for Site B, so was likely from the same source.

(D) Highway 17 near Petawawa. This stretch of Highway 17 was reconstructed in 1996 and is now in serious need of rehabilitation. The centreline joint and shoulders are riddled with extensive low temperature cracking distress. Full width transverse cracking is pervasive throughout and numerous cracks are starting to spall. The asphalt cement used for this contract was an 85/100 penetration grade that would likely have graded short of the required –34ºC in this area. However, the site was included in this study since the recovered asphalt cement still grades at a level that is reasonable compared to the lowest calculated pavement temperatures. Weather data from a nearby station (CFB Petawawa) suggests that the pavement surface temperature has never fallen below –27ºC (Iliuta et al. 2004).

(E) Highway 28 north of Burleigh Falls. The stretch of Highway 28 investigated was reconstructed in 1993 and is virtually free of low temperature cracks today. If the mix designs are correct, then both binder courses contained 30% recycled asphalt pavement (RAP), whereas the surface course contained no RAP. Hence, for this study the asphalt cement from both binder courses was recovered for analysis. However, it should be noted that Ontario contractors have an option to use RAP in hot mix with adjustments to the high temperature grade if the RAP content exceeds 20%. There is no indication in the documentation available for this contract whether this actually happened.

The entire 18.5 km section showed only sporadic cracking distress. Hence, only three locations of 250 m were surveyed. The first survey was conducted just south of Eel's Creek at the northerly end of the contract. Three minor transverse cracks were noted (one full lane and two quarter lane width) in addition to approximately 11 m of very minor mid-lane cracking. The second survey was conducted approximately 8 km north of Burleigh Falls near the junction with Highway 56. This 250 m stretch was free of cracking distress. A third survey conducted approximately 2 km north of Burleigh Falls had approximately 10 m of mid-lane cracking in a swampy area. Figures 16(a) and (b) show the general state of the pavement and the distress in the swamp area.

A ~0.5 cm thick microsurfacing overlay was applied in 2004 to correct a problem with inadequate friction. Note that the overlay may have covered up some minor cracking distress but that major transverse cracks would likely have resurfaced after three winters if they had been present.



(a) (b)

Figure 16. Typical surface condition of Highway 28 contract near Burleigh Falls.

Note: Arrow indicates minor wheel path cracking in a swamp area.

(F) Highway 28 south of Lakefield. This stretch of Highway 28 was reconstructed with PG 58-28 asphalt cement in 1997 and 1998. The binder used in this contract fails the required PG –34 for the area, but the surface condition is still excellent today, which may be due to the thick design of the road (120 mm) and moderate traffic volume (AADT = 5740 with only 9.3% commercial vehicles).

(G) Highway 33 east of Conway. This stretch of Highway 33 was reconstructed in 1998 with a PG 58-28 grade asphalt cement, which is appropriate for the area. It is totally free of cracking distress today even though the pavement is only 50 mm in thickness. The ride condition index stands at 7.2 and the pavement condition index at 81, likely due to surface unevenness resulting in part from the thin design.

(H) Highway 35 south of Lindsay. This stretch of Highway 35 was reconstructed in 1997 with an 85/100 penetration grade asphalt cement, which likely would not have made the required –34ºC Superpave® grade for this area. However, the pavement is in very good condition today, which may be due to the design (three lifts totalling 130 mm), the low commercial traffic component (AADT = 8062 with 9% commercial vehicles), or the recent mild weather for the area.

(I) Highway 41 from Dacre to Egansville. This stretch of Highway 41 was reconstructed in 2000 with two different PG 58-34 grade asphalt cements. The material used in the surface course was categorized as “polymer-modified,” but that in the binder course, while of the same grade, was not categorized as such. The pavement surface is in excellent condition today with only minor cold temperature cracking localized in the northern half near Egansville.

(J) Highway 41 near Denbigh. The stretch of Highway 41 investigated was reconstructed in 1996. Total hot mix asphalt for this contract amounted to nearly 24,000 tonnes. An additional 93,000 tonnes of granular material was used for the rehabilitation of the pavement foundation. The binder course contained 35% RAP, whereas the surface course contained none. The asphalt binder from both courses was recovered for analysis since it was difficult to decide which lift was most susceptible to cold temperature cracking. The contract covered 13 km through mainly forested area with good drainage on either side of the pavement.

The contract showed early and extensive cracking distress from beginning to end. Hence, three stretches of only 100 m each were surveyed. The first stretch was severely damaged, with the shoulders and part of the lane showing significant freeze-thaw distress. Figure 17(a) provides a representative image. In addition, 18 additional transverse cracks in total were counted. Ten of these cracks reached the width of a single lane, while the remainder all covered the entire width of the pavement. Figure 17(b) provides a representative image of severe transverse cracking in this contract.

The second and third surveys had 43 and 29 transverse cracks, respectively, most of which covered the full width of the pavement and were of intermediate severity.

Figure 17. Typical surface condition of Highway 41 contract near Denbigh.

Note: Arrows indicate transverse cracks sprouting out from longitudinal distress.

The recovered asphalt cement was tested in NMR and XRF spectrometers, which hinted at the presence of phosphorous and zinc containing compounds soluble in THF, likely originating from the presence of waste engine oils and perhaps polyphosphoric acid (PPA). To what extent the presence of this material in the asphalt cement is to blame for the state of the pavement today deserves to be further investigated.

(K) Highway 41 from Kaladar to Northbrook. The stretch of Highway 41 investigated was reconstructed over 12.4 km in 1998 and 1999. The pavement design consisted of a 40 mm binder course constructed on pulverized grade and a 40 mm surface course. The contract covered 12.4 km through mainly forested areas with reasonable drainage on either side of the pavement. Total hot mix asphalt for this contract amounted to nearly 24,000 tonnes. An additional 52,000 tonnes of granular material was used for the rehabilitation of the pavement foundation. No recycled material was used in this contract, and the binder grade specified was a PG 58-34.

During the 2007 survey the cracking was considered to be severe throughout the contract. Figures 18(a) and (b) provide representative images. Over 40,000 m of cracks were sealed in this contract in 2003, but at the time of the survey in 2007 many additional cracks had already appeared.

The recovered asphalt cement was tested in an NMR spectrometer and with a hand-held XRF spectrometer, which suggested the presence of phosphorous and zinc-containing compounds soluble in THF. To what extent the likely presence of waste engine oils and PPA in the asphalt cement is to blame for the state of the pavement today deserves to be further investigated.

(a)

(b)

Figure 18. Typical surface condition for Highway 41 north of Kaladar. Note: Arrows indicate thermal cracks that have appeared after the original crack sealing.

(L) Highway 41 from Northbrook to Cloyne. The stretch of highway investigated was reconstructed in 2000. The design consisted of a two-lift asphalt concrete pavement on top of a pulverized grade. Total hot mix asphalt for this contract amounted to 13,000 tonnes. Transverse cracking commenced shortly after construction and was considered excessive throughout within a few years, with cracks spaced approximately 3 m apart. The contract covered 8.7 km over mainly forested terrain with good drainage on either side of the pavement. XRF analysis indicated the presence of zinc suggesting the asphalt cement for this contract was modified with waste engine oils.

(M) Highway 41 near Vennachar. The stretch of highway investigated was reconstructed in 1997 and 1998. The design consisted of a two-lift asphalt concrete pavement on top of a pulverized grade. Distress in this contract was judged to be minimal at the time of the survey in 2007, with transverse cracks spaced approximately 43 m apart. The contract covered approximately 20 km over mainly forested terrain with good drainage on either side of the pavement. The asphalt cement used in this contract was listed as "polymer modified", which may explain some of the improved performance.

(N) Highway 60 near Bat Lake. The stretch of Highway 60 investigated was reconstructed in 1998. Total hot mix asphalt for this contract amounted to 21,000 tonnes. The asphalt was placed on a minimum of 150 mm of freshly pulverized and compacted granular material throughout the contract. No recycled materials were used in the hot mix asphalt; hence the binder grade specified was a PG 58-34. The contract covered 13.1 km over mainly forested terrain with good drainage on either side of the pavement.

After only five years of service a total of 29,000 m of cracks had to be sealed in this contract. During the 2007 survey the cracking was considered to be severe throughout the contract, with many areas showing additional deterioration.

(O) Highway 60 east of Wilno. The stretch of Highway 60 investigated was reconstructed in 1994. Little if any information was available about construction details except that the pavement consisted of two lifts on a pulverized grade. The road is now in very poor condition, with pervasive transverse and longitudinal cracking throughout. Cracking started early in the life of the pavement, and total crack sealing in 2004 amounted to 50 km over 8 km of pavement, with approximately half of this for transverse cracking alone.

(P) Highway 62 near Bannockburn. The stretch of Highway 62 investigated was reconstructed in 1997. The pavement design consisted of two lifts of asphalt concrete on top of a pulverized grade. Total hot mix asphalt for this contract amounted to nearly 23,000 tonnes. Nearly 16,000 tonnes of granular material was used for the rehabilitation of the pavement foundation and shoulders. As both binder and surface courses contained 30% RAP the asphalt binder from both courses was recovered for analysis. The contract covered 15 km through mainly swampy and forested area with somewhat poor drainage on either side of the pavement.

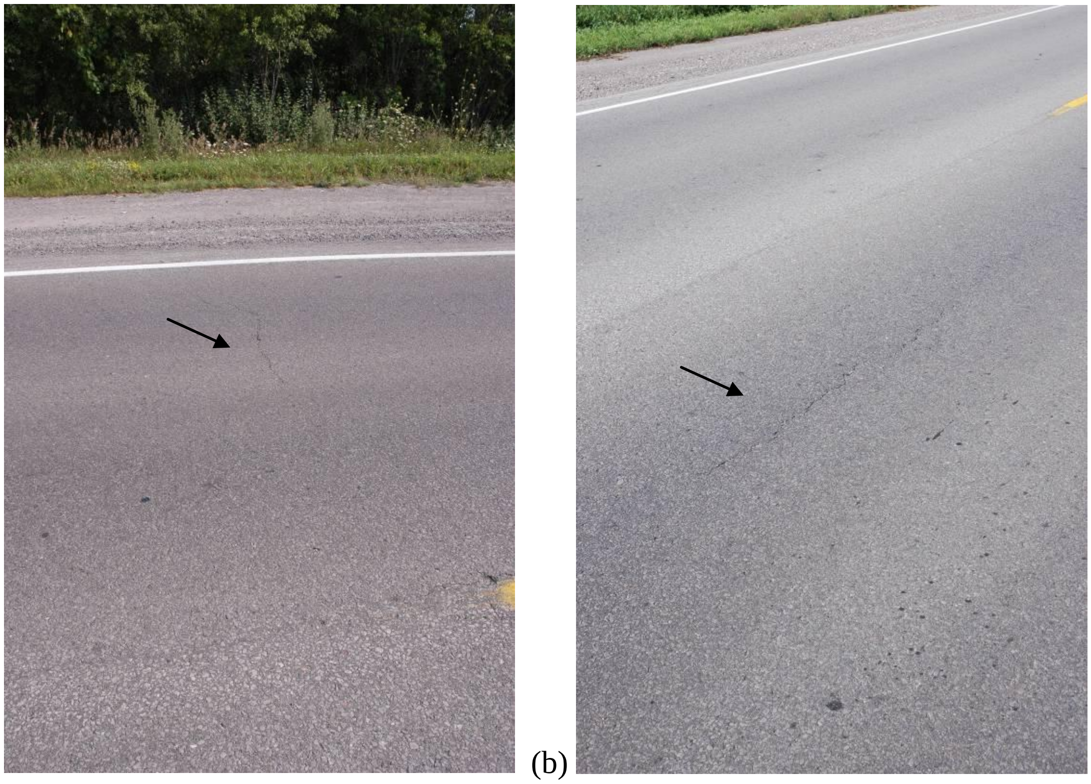
The road was severely cracked from beginning to end after only a few years of service. Hence, three stretches of 100 m were surveyed for cracking distress. The crack counts for the three sections were 57, 86, and 73, respectively, with a mixture of quarter lane, half lane, and full lane, most of minor or very minor severity.

The recovered asphalt cement was tested in an NMR spectrometer, which suggested the presence of phosphorous-containing compounds soluble in THF. To what extent the likely presence of PPA in the asphalt cement is to blame for the state of the pavement today deserves to be further investigated.

(Q) Highway 62 north of Bloomfield. This stretch of Highway 62 was reconstructed in 1993. The pavement design consisted of two lifts of asphalt concrete on top of a pulverized grade. Total hot mix asphalt for this contract amounted to nearly 20,000 tonnes. As both asphalt courses contained 20% RAP the asphalt cement from both was recovered for analysis. The terrain is mainly farmland with good drainage on either side of the pavement.

The entire 11.4 km length of this site showed only sporadic distress. Hence, three stretches of 250 m were surveyed in detail. The first survey was conducted approximately 1 km north of the junction with Highway 33. A total of four very minor transverse cracks were noted in addition to approximately 49 m of very minor inner wheel-path cracking. Figures 19(a) and (b) show typical cracking patterns.

The second survey was conducted approximately 4.6 km north of the junction with Highway 33. It showed only very minor cracking associated with a traffic loop in this area and minor cracking in the shoulders (neither of which was included for this analysis). The third survey was done approximately 9.5 km north of Highway 33. This section had two transverse cracks both a quarter lane wide in addition to some 19.8 m of very minor wheel path cracking.



(a) (b)

Figure 19. Typical surface condition for Highway 62 near Bloomfield. Note: Arrows indicate the presence of minor wheel path cracks.

(R) Highway 138 from Cornwall to Monkland. This stretch of Highway 138 was reconstructed in 2000 and 2001. The pavement design consisted of three lifts of asphalt concrete (130 mm) on top of a pulverized grade. No recycled material was used and the asphalt cement specified was a PG 58-34. Hence, for this study the asphalt cement from all three courses was recovered for analysis. The contract covered 17.0 km through varying terrain (mainly agricultural areas) with good drainage on either side of the pavement. The entire length of the contract was largely free of distress.

(S) Highway 138 north of Monkland. This stretch of Highway 138 was reconstructed in 1998. The pavement design consisted of three lifts of asphalt concrete on top of a pulverized grade. Total hot mix asphalt for this contract amounted to nearly 44,000 tonnes. An additional 45,000 tonnes of granular material was used for the rehabilitation of the pavement foundation. No recycled material was used, and a PG 58-34 grade of asphalt cement was specified. Hence, for this study material from all three courses was recovered for analysis. The contract covered 17.2 km through varying terrain (swamps, forests, and agricultural areas) with good drainage on either side.

The pavement was surveyed in three locations. A total of 66,667 m of cracks in this contract had been sealed after only six years of service. However, at the time of the survey for this study, additional cracks had appeared in many areas that had not yet failed at the time of the repairs three years before. During the 2007 survey the cracking was considered to be severe throughout the contract. Figures 20(a) and (b) show typical distress levels for this location.

The recovered asphalt cement was tested in NMR and XRF spectrometers, which suggested the presence of phosphorous and zinc-containing compounds soluble in THF. To what extent the likely presence of waste engine oils and PPA in the asphalt cement is to blame for the state of the pavement today deserves to be further investigated.

(a)

(b)

Figure 20. Typical surface condition of Highway 138 north of Monkland. <u>Note</u>: Arrows indicate additional cracks that appeared after the initial sealing.

The two Highway 138 contracts show that asphalt pavements of nearly the same designs with the same climate can show vast differences in performance. Both pavements are of approximately the same age while the superior performing contract carries nearly twice the number of cars and truck in a given day.

(T) Highway 416 near Spencerville. This stretch of the southbound lanes of Highway 416 was newly constructed in 1999 as part of the twinning of the highway between Ottawa and Highway 401. The pavement design consisted of three lifts of asphalt concrete (140 mm) on top of a pulverized grade. No recycled asphalt pavement was used in this contract, and the binder specified was a PG 64-34 in both the surface and two heavy-duty binder courses. The asphalt cement from the surface course was recovered for analysis. The contract covered 17.9 km through varying terrain (forests and agricultural areas) with excellent drainage on either side of the pavement. The entire length of the contract was largely free of distress.

The summary of approximate crack lengths for each of the 20 contracts is provided in Figure 15. Please note that the total crack length does not say much about the severity of the distress. The superior pavements also showed the least severe cracking, while the others had a mixture of mild to extremely severe cracking as shown in the various photographs in Figures 16 to 20.

4.1.2 Regular BBR Testing According to AASHTO Method M320

The results of the regular BBR test are shown below in Figure 21. The cracking distress is plotted versus the grade deficit or surplus according to the difference between the required LTPPBind® and obtained AASHTO M320 grade temperatures. The dotted line in the figure represents the 3ºC deficit beyond which the Ministry of Transportation in Ontario (MTO) rejects asphalt cement for a particular climatic location (the three degrees error is allowed to compensate for the variability in the BBR grading test). Hence, the total accuracy of this analysis is just 55%, with the nine failing contracts all passing the current specification limits.

Had the dotted line been drawn to reach the highest possible accuracy then it would have been placed around the zero point, and the accuracy would have been considerably higher at around 80% (16 times out of 20). However, the spread of the grading results remains rather narrow at ~11ºC, from the best to worst performing contracts according to the regular AASHTO M320 criteria.

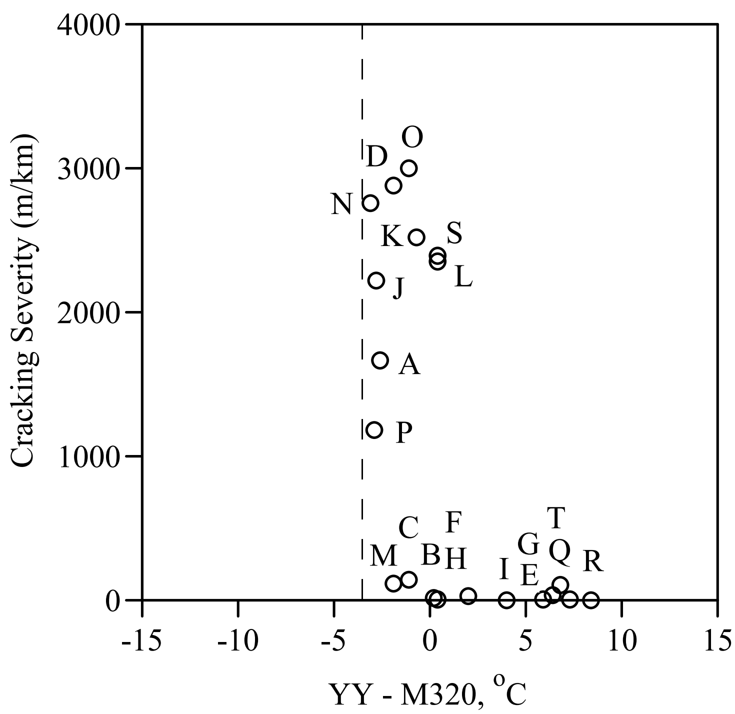


Figure 21. Regular BBR grading results according to AASHTO M320 criteria for 20 eastern and northeastern Ontario contracts.

4.1.3 Extended BBR Testing According to MTO Method LS-308

A graph of three day grade loss at 10°C, according to LS-308 procedures, versus the distress in pavements is shown in Fig. 22. Taking the grade loss greater than three degrees as a limit in the test would provide an accuracy of 95%, which is considerable higher than the 55% for AASHTO M320 as applied in Ontario with a 3ºC deficit (margin of testing error) or the 80% for AASHTO M320 without the margin or error. Of the twenty samples tested, sample M is the only outlier. The reason for the least distress in this sample is quoted as polymer modification which gives it more flexibility at low temperature conditions.

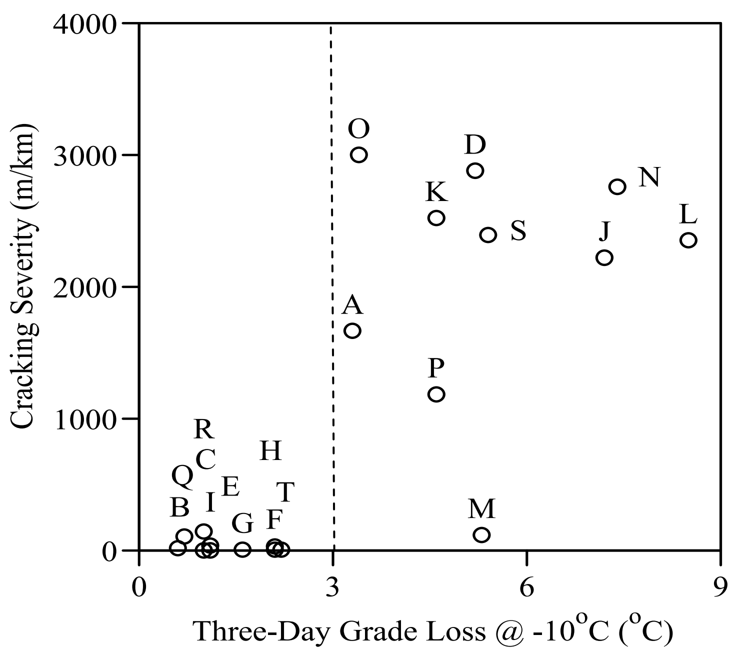


Figure 22. Three-day grade loss in LS-308 versus the cracking severity in 20 eastern and northeastern Ontario contracts.

The grade loss data according to MTO LS-308 also show that the poor performers are all penalized under the extended BBR protocol more than the good performers. Hence, this test method is most desirable from a performance grading point of view in that we would want to promote better performing asphalt cements rather than poor performing materials.

In order to investigate the low temperature grade according to MTO LS-308 the cracking severity was also plotted according to the grade deficit/surplus in Figure 23, analogous to the plot of Figure 21. This graph shows that the accuracy for such analysis is 90% (18 times out of 20) and that the range for worst to best performance is approximately 20ºC, nearly *twice* the range for regular AASHTO M320 grading.

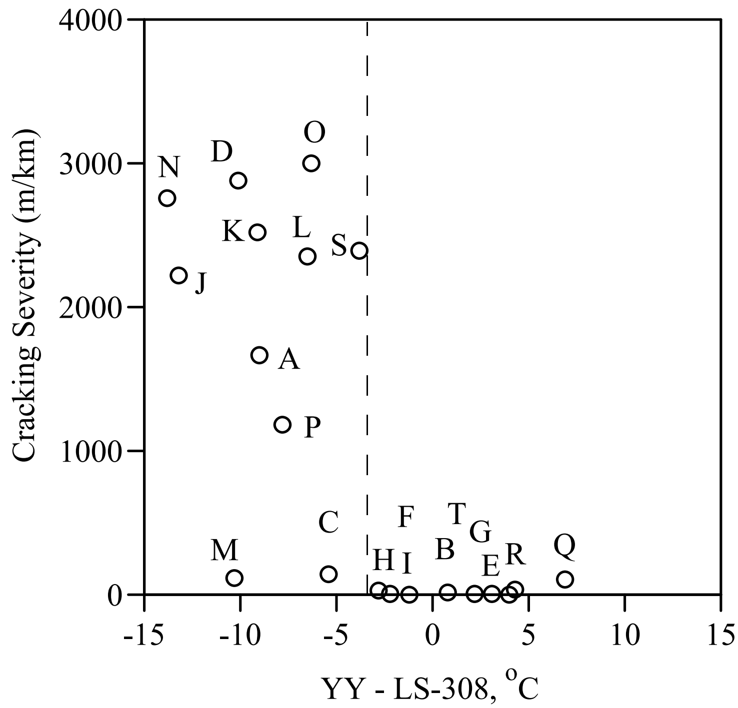


Figure 23. Grading results according to MTO LS-308. Note: The dotted line is placed in a somewhat arbitrary position yet with the aim of obtaining the highest accuracy of 90%.

4.1.4 Ductile Failure Testing According to MTO Method LS-299

In addition to the MTO LS-308 analysis, the ductile failure properties were determined according to MTO LS-299. Asphalt cements that pass an LS-308 *low strain* criterion could still fail in the *high strain* ductile state. Hence, the approximate critical crack tip opening displacement was calculated from the essential work of fracture over the net section stress in the 5 mm ligament specimen. Figure 24 provides the cracking severity as a function of approximate CTOD.

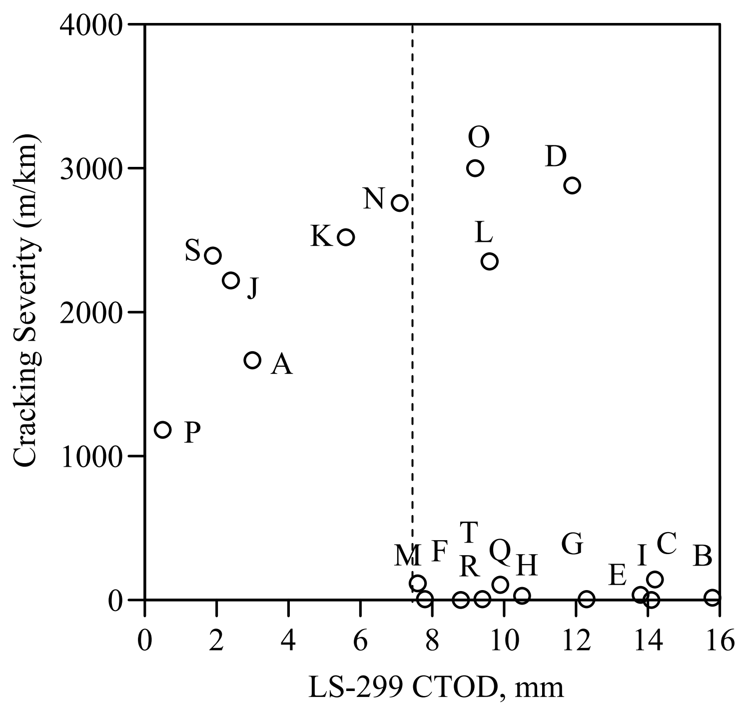


Figure 24. Approximate CTOD in relation to the cracking severity for 20 eastern and northeastern Ontario contract sites.

If a line were to be drawn at around CTOD = 7.5 mm, this method would provide an 85% accuracy for separating the good from the poor performing contracts (17 times out of 20).

That this is not 100% is explained by the fact that binder L failed the LS-308 procedure. (It has poor low temperature properties and likely failed due to low temperature exposure.) Binders O and D were from a very old contract (site O from Wilno was constructed in 1994), and from a rather old contract on the busy Trans-Canada Highway (site D from Petawawa was constructed in 1996). Asphalt cements were recovered from the top 5 cm of the pavement while most of the ageing might have occurred in the top 1 or 2 cm of the surface layer. Hence, the outliers are easily explained by the differences in ageing between the surface layers and the actual sample used for analysis.

The results in Figure 24 show that those asphalt cements from the very poor performing contracts in Little Current (site A), Denbigh (site J), Bannockburn (site P), Kaladar (site K), and Monkland (site S), also performed very poorly in the DENT test, which reassures that the test method correlates with performance in service.

4.1.5 Elastic Recovery Testing

The elastic and viscous components of the creep were separated by loading the beams in the BBR for 240 s followed by unloading for 720 s. The non-recoverable displacement (compliance) reflects the viscous part of the displacement. A low viscosity would indicate that a pavement would have a low tendency to thermally crack. The recoverable displacement reflects the elastic component from the displacement. High elasticity would indicate that the thermal stresses are maintained for long times and do not easily relax. Hence, displacement due to viscous flow is beneficial while displacement due to elastic effects is likely detrimental. Since the current BBR protocol as embodied in AASHTO M320 combines both viscous and elastic displacements to calculate creep stiffness at 60 s loading, the method confounds two opposing effects and therefore provides not a very high degree of accuracy.

Figures 25-27 provide the cracking distress versus the grade deficit/surplus for the total compliance (J_t), the viscous compliance (J_v), and the elastic recovery (ER), respectively. It can be seen from these graphs that the split of viscous and elastic effects increases the accuracy by a considerable amount. While the total displacement provides only an accuracy of 75% (15 times out of 20), the viscous compliance has an accuracy of 95% (19 times out of 20), and the elastic recovery also has an accuracy of 95% (19 times out of 20). It should be noted that these increased accuracies come after only 3 h of conditioning (1 h conditioning could not be done for scheduling reasons).

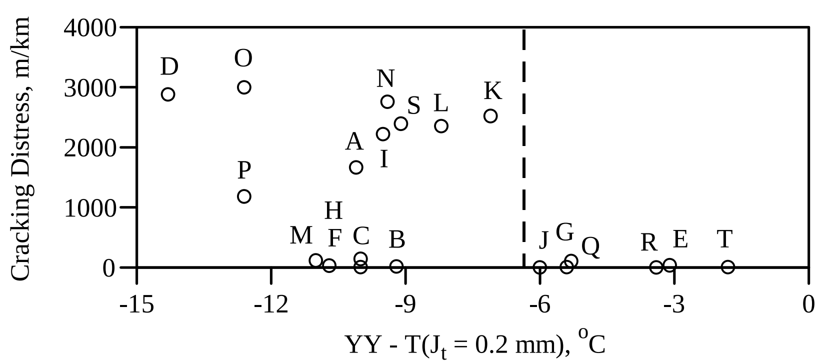


Figure 25. Cracking distress versus the limiting temperature where the total displacement reaches 0.2 mm. Note: Dotted line is placed in order to obtain the highest accuracy.

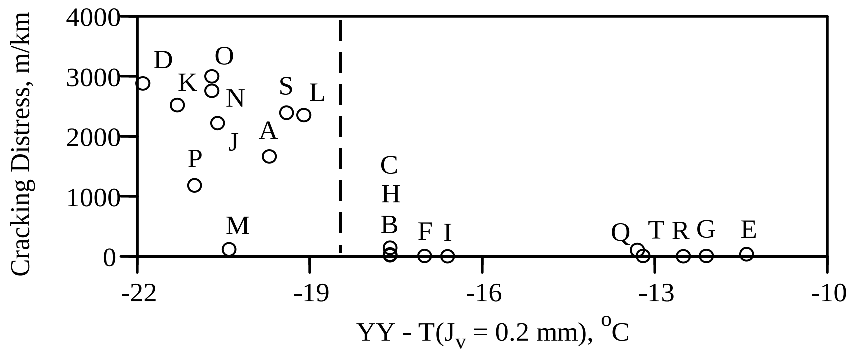


Figure 26. Cracking distress versus the limiting temperature where the viscous displacement reaches 0.2 mm. Note: Dotted line is placed in order to obtain the highest accuracy.

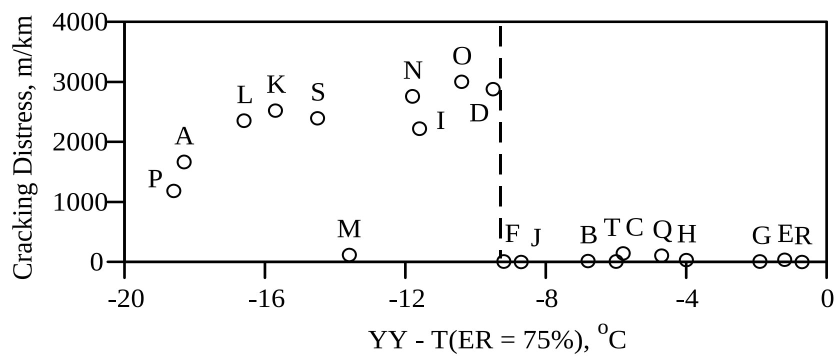


Figure 27. Cracking distress versus the limiting temperature where the elastic recovery reaches 75%. Note: Dotted line is placed in order to obtain the highest accuracy.

The same elastic recovery tests were conducted after three days of conditioning at the test temperature. Figure 28 provides the limiting stiffness and m-value temperatures for one, three, and 72 hours of conditioning. The graphs show that largely those asphalt cements that are m-value controlled lose significant performance, while those that are stiffness controlled lose less. The data also show that the general trend is for all limiting temperatures to move up, with the spread increasing slightly from three to 72 hours of conditioning. For a typical Ontario location, an error of 6ºC is enough to reduce the confidence from the intended 98% to less than 50% that in a given winter no damage will occur. Hence, the 98% confidence limit was chosen by the SHRP researchers for good reasons, since transverse cracks are very detrimental to a pavement's life cycle. All other properties (J_t, J_v, ER) were also determined after 72 hours, and the general trend was that nearly all asphalt cements lose performance, with some deteriorating more than others.

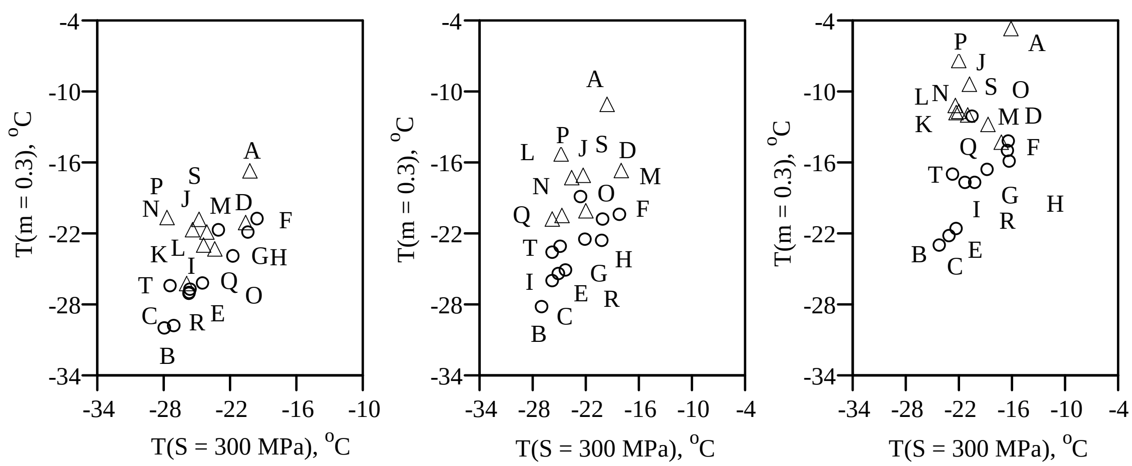


Figure 28. Effect of conditioning time on limiting BBR temperatures: (a) one hour; (b) three hours; and (c) 72 hours. Triangles indicate poor performance and circles indicate good performance.

4.2 Highway 655 Trial Sections

4.2.1 Distress Surveys

The cracking distress was surveyed in the summer of 2008 by MTO staff trained in this exercise (Soleimani et al. 2009). The results for the survey are given in Figure 29 below. The bars in the graph represent the total cracking distress in three 50 m stretches surveyed for each 500 m long test section. The data show that for asphalt cements of nearly the same Superpave® PGAC grade, the variation in cracking distress can be vast. Hence, there is a need for improved low temperature specification grading protocols.

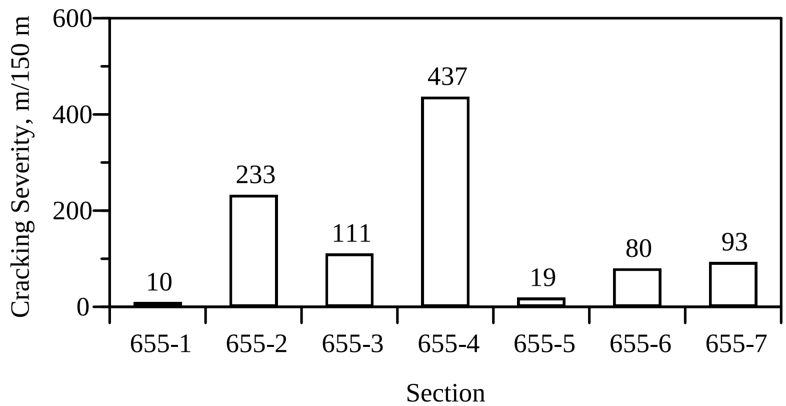


Figure 29. Distress survey for Highway 655 trial sections (Soleimani et al. 2009).

Perhaps a better illustration of the differences in cracking distress is provided by a series of photographs. Figure 30 provides representative images for sections 655-1 through 5, which have nearly identical Superpave® grades. Note that for reasons of time constraints no photos were taken for sections 6 and 7 during the 2008 survey. However, both of these showed similar distress to sections 2-5.

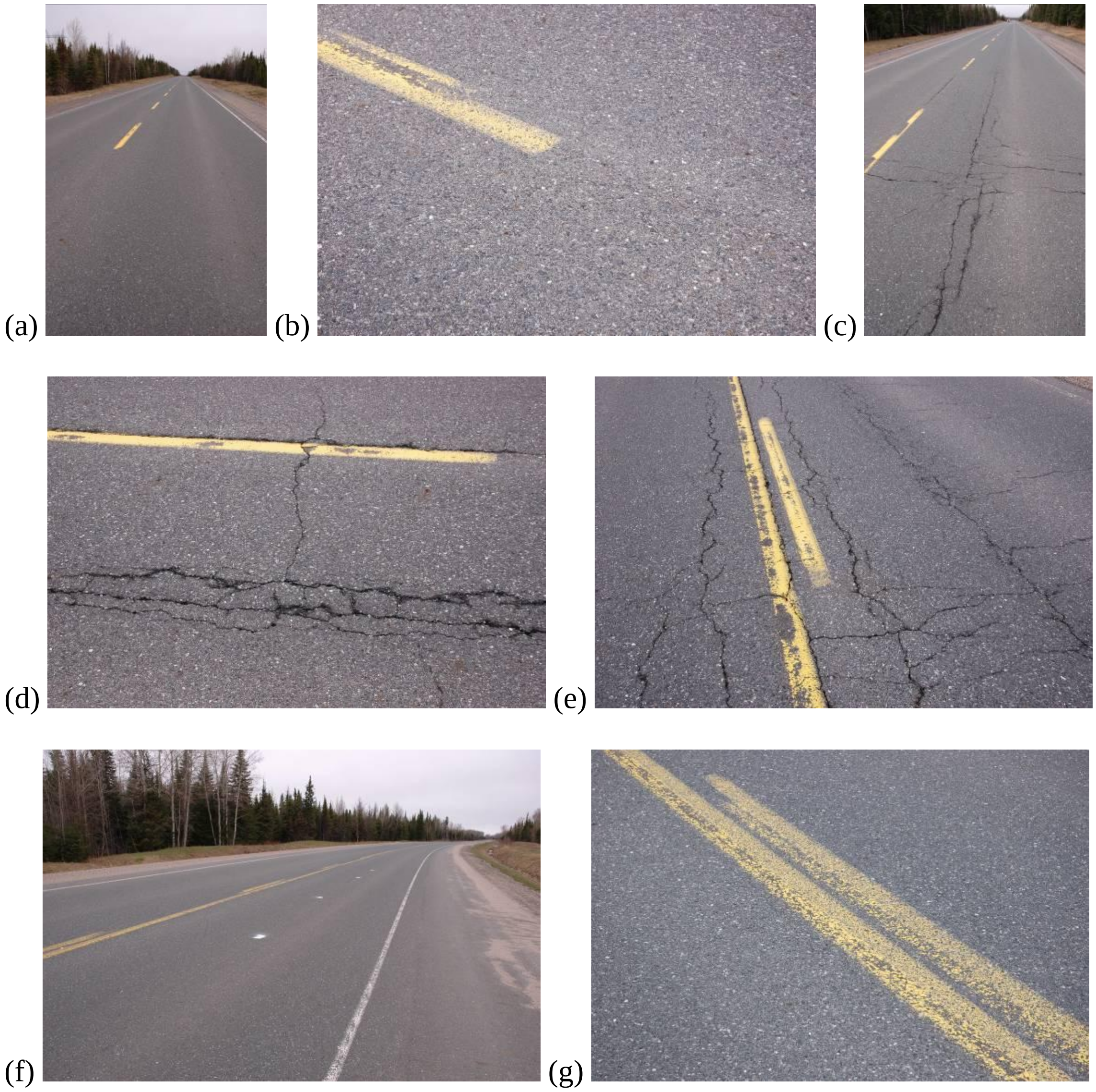


Figure 30. Representative photographs for sections 1-5: (a) & (b) 655-1; (c) 655-2; (d) 655-3; (e) 655-4; (f) & (g) 655-5. Note that no photographs were taken for sections 655-6 and 655-7 in 2008.

4.2.2 Regular BBR Testing According to AASHTO M320

The regular AASHTO M320 specification grades are provided in Figure 31 below. The data are interesting in that the regular AASHTO M320 grade gives somewhat of an indication of the cracking distress in each section. Sections 1 and 5 give the lowest grades and they have largely been left unscathed. Section 4 shows the worst performance in both the AASHTO M320 ranking (-25ºC) and in terms of cracking distress. The fact that the PGAC grade decreased by as much as 10ºC beyond what the PAV residue predicted indicates that an improved laboratory ageing method is required. The RTOF/PAV is supposed to produce a material that reflects properties after 8-10 years in service.

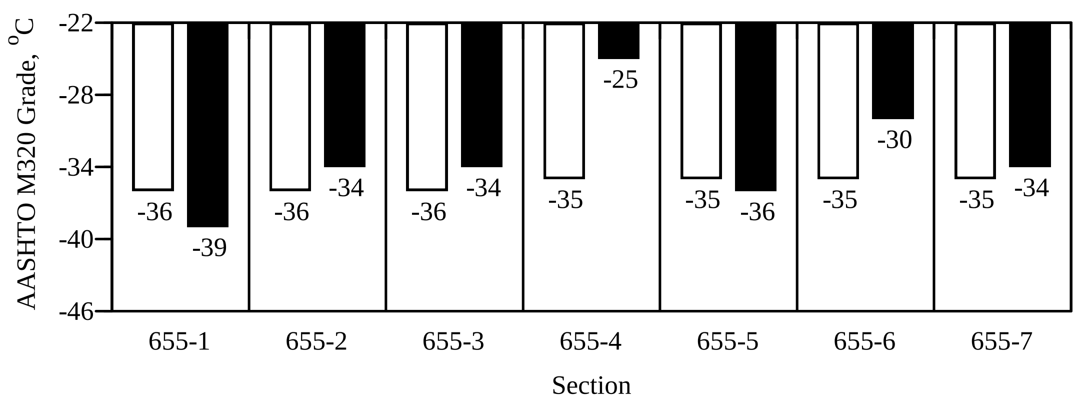


Figure 31. Regular AASHTO M320 specification grades for laboratory-aged (open bars) and recovered (closed bars) asphalt cements.

What the recovered grades in Figure 31 do <u>not</u> explain is the large amount of distress in sections 2, 3 and 7 since they still grade around the -34ºC mark. This pavement only reached its design temperature of -34ºC for two brief periods in early 2004 (Bodley et al. 2007). Hence, these sections should not have cracked but they did crack to a significant degree. To further explain this observation the LS-308 test protocol was used.

4.2.3 Extended BBR Testing According to MTO LS-308

The worst grades after three days of conditioning and the grade losses for each of the recovered asphalt cements are given in Figures 32 and 33 below.

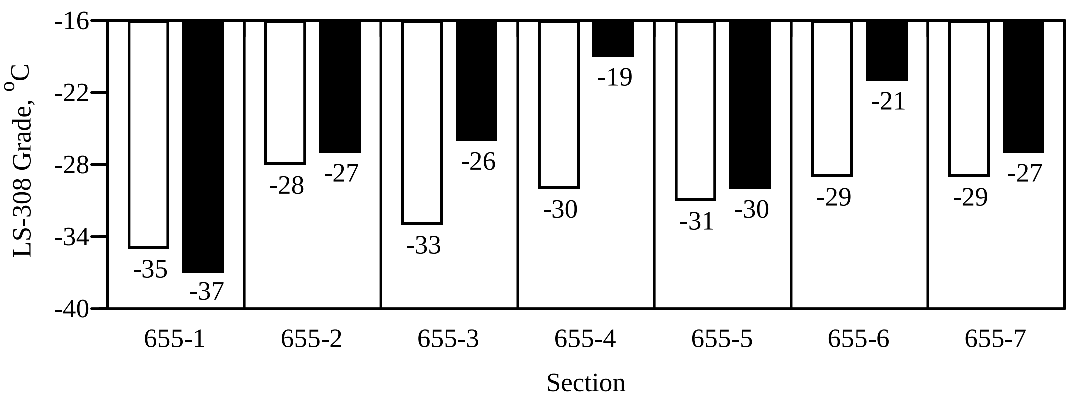


Figure 32. LS-308 specification grades for laboratory-aged (open bars) and recovered (closed bars) asphalt cements.

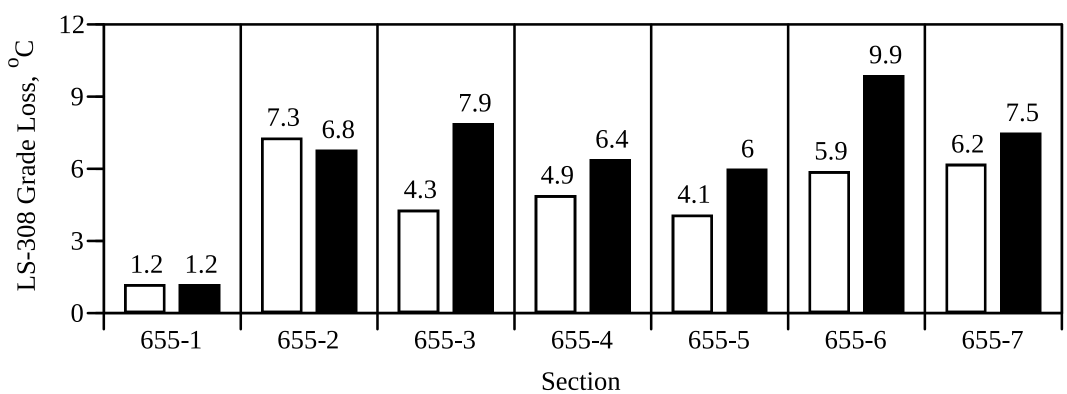


Figure 33. Worst LS-308 grade losses for laboratory-aged (open bars) and recovered (closed bars) asphalt cements.

The results obtained with LS-308 show why the asphalt cements in sections 2, 3 and 7 cracked so significantly. Their worst grades after three days of conditioning at -10ºC and -20ºC are well above the minimum surface temperature recorded in early 2004 (Iliuta et al. 2004, Bodley et al. 2007). While the surface reached to -34ºC for only an hour or two, it has reached to -26ºC and -27ºC for many more hours. Hence, the thermal cracking is largely explained by the grade loss.

It is interesting to note that the worst three day grade losses at a conditioning temperature of -10ºC for the materials recovered from sections 1 and 5 were 0.4ºC and 3.0ºC, respectively. These grade losses were the lowest of all seven materials and hence agree with the findings of Figure 22, which shows that the maximum grade loss at -10ºC for the 20 eastern and northeastern Ontario contracts was highly correlated with the thermal cracking distress. A low grade loss reflects a low tendency to harden at the conditioning temperature. Those asphalt cements that show low grade losses will be able to relax thermal stresses before the pavement reaches the spring thaw cycle. Hence, thermal stresses are relaxed when the pavement base thaws and serious movement occurs. No stress means little or no cracking.

4.2.4 Ductile Failure Testing According to MTO Method LS-299

The final evaluation for the recovered asphalt cement materials was according to the LS-299 double-edge-notched tension test protocol. The approximate critical crack tip opening displacements for all seven binders are given in Figure 34 below.

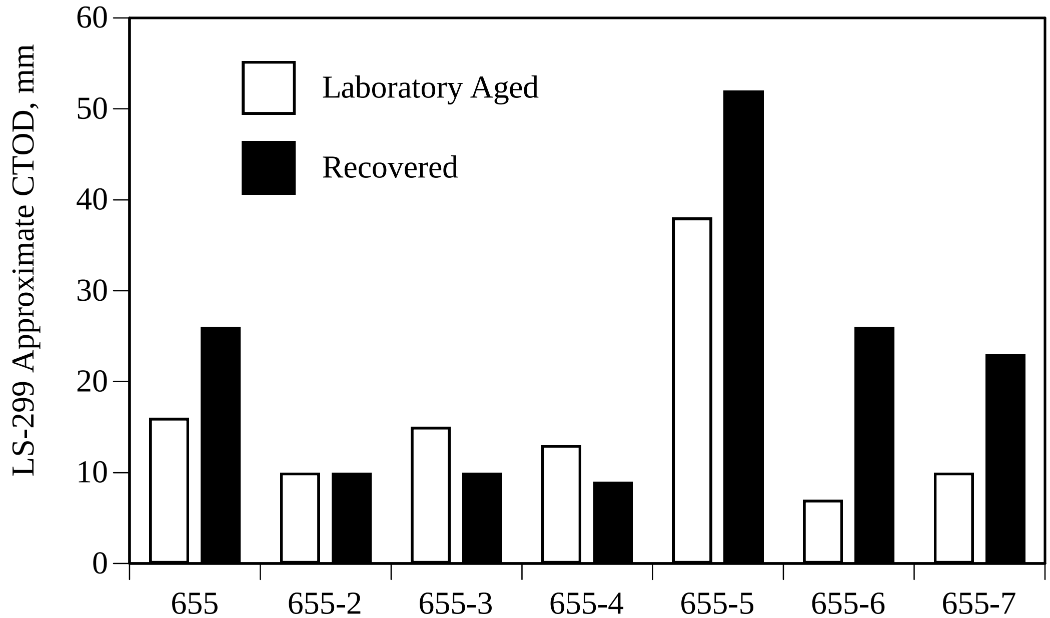


Figure 34. Approximate critical crack tip opening displacements (CTOD) for original asphalt cements (open bars) and recovered materials (closed bars).

This figure shows that the CTOD's for sections 2, 3 and 4 are very low at 9 and 10 mm. Two of these materials were found to contain zinc as detected by X-ray fluorescence (XRF) and traces of phosphorous as detected by nuclear magnetic resonance (NMR). The presence of zinc and phosphorous in both of these, and in five of the poorly performing eastern and northeastern Ontario contracts, suggests that the presence of waste engine oils, with or without the concurrent use of polyphosphoric acid (PPA), is widespread in the asphalt supply. Waste engine oils are thought to weaken the asphalt cement-aggregate interface (Villaneuva et al. 2008), promote the physical hardening process (Hesp et al. 2007), and PPA is found to lower the ductile strain tolerance (Kodrat et al. 2007) and, hence, these additives could therefore be blamed in large part for the poor performance of Ontario roads.

Chapter 5

Summary and Conclusions

Given the data presented and discussed in this thesis, the following summary and conclusions may be presented:

- Asphalt cements of supposedly the exact same AASHTO M320 grade were found to show vast differences in cracking distress for pavements of identical design, traffic level and climate. Hence, the regular BBR specification as embodied in AASHTO M320 needs to be improved.
- The extended BBR protocol, as embodied in MTO method LS-308, provides an improvement over AASHTO M320, since it was found to penalize poorly performing materials more than good performing materials. The grade loss in LS-308 after three days of conditioning at -10ºC ranged from 0.4ºC for the best performing asphalt cement used in trial section 655-1 to nearly 9ºC for what was likely an air blown asphalt cement used in contract site L north of Northbrook. This difference of nearly 9ºC would reduce the confidence that in a given year the pavement is not exposed to damaging temperatures from the intended 98% to less than 10% for a typical Ontario climatic region. Hence, the physical ageing effect as assessed by LS-308 is a likely cause for the large performance variations as found in this study.
- The double-edge-notched tension test results provide a reasonable correlation with cracking distress in service. Those asphalt cements with a high strain

tolerance provide pavements with little or no cracking distress. MTO method LS-299 should provide a second layer of protection to avoid asphalt cements that are highly gelled and thus would perform poorly under high strain conditions.

- The currently-used rolling thin film oven/pressure ageing vessel (RTOF/PAV) protocol to accelerate chemical ageing in the laboratory does not replicate service conditions. In some asphalt cements the ageing in service is significantly more than what the RTFO/PAV predicts. Hence, an improved chemical ageing protocol is urgently needed.
- The current BBR protocol measures creep stiffness and slope of the creep stiffness master curve (m-value) at 60 s loading in three point bending. The total displacement is comprised of both viscous and elastic components. The viscous component lessens thermal stresses through relaxation while the elastic component retains such stresses. Hence, the two factors are confounding the performance grading of the asphalt cement. A separation of these two factors by measuring the loading and unloading behavior increases the accuracy, even after short periods of conditioning.
- Asphalt cements modified with questionable technology (e.g., waste engine oils, air blowing, acid modification, base modification) can pass the current specification methods under AASHTO M320 but these materials fail to provide a good return on investment. Pavements that are modified in the above manners were found to crack prematurely and/or excessively. In large part the failure of the current specification is due to the fact that physical ageing phenomenon is

ignored. However, a second factor, yet to be recognized and dealt with, is the inability of the RTFO/PAV protocol to replicate service conditions.

- The introduction of MTO methods LS-299 and LS-308 would help the Ministry of Transportation of Ontario to better pay for performance and avoid costly pavement failures such as those investigated in this study.

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