LOW TEMPERATURE INVESTIGATIONS ON ASPHALT BINDER PERFORMANCE – A CASE STUDY ON HIGHWAY 417 TRIAL SECTIONS

by

OLURANTI PAUL TOGUNDE

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Abstract

This thesis investigates and documents fundamental studies of highway materials (asphalt engineering properties) especially on different modified asphalt binders and mixtures in order to understand failure mechanisms at low temperature and superior performance of such asphalt binders with the aim of preventing premature cracking on Ontario highways. In addition, seven asphalt binders of different compositions were used as a template for study and this research work is tailored towards Superpave® performance-based specification testing with the aim of improving asphalt pavement performance under various conditions and consequently reducing premature cracking in order to achieve long lasting highways.

Based on the actual applied pattern of Superpave® specification criteria, the mechanical responses of the binders are analyzed by extended bending beam rheometer (eBBR), tensile stress ductilometer (Petrotest DDA3®), compact tension test (Instron AsphaltPro®), double-edge-notched tension and single-edge-notched tension (MTS 810 universal testing machine) protocols. The objective of this study entails establishing and developing of a proper procedure for the testing of binders with the aim of ranking (grading) the performance after validation of laboratory and field experiments.

Analysis of the results appears to show that the premature distress on the Highway 417 trial sections can be attributed to reversible aging tendency (wax crystallization) at low temperatures coupled with low fatigue resistance of the binders. The results suggest that different polymer modifications had significant influence on the performance of asphalt mix as demonstrated

from the results obtained from essential and plastic work of fracture using double-edge-notch-tension test (DENT). Crack tip opening displacement (CTOD) parameter consistently show the performing grading of asphalt binder while compact tension test protocol provides plane strain fracture toughness (K_{1c}) which could be used to rank binders with respect to fracture resistance at low temperature. Hence, CTOD is a promising parameter which can be used to establish performance ranking of the binders.

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List of Symbols and Abbreviations

Abbreviations

AASHTO American Association of State Highway and Transportation Officials

ASTM American Society for Testing and Materials

BBR Bending Beam Rheometer

DENT Double-Egde-Notched Tension

EWF Essential Work of Fracture

MTO Ministry of Transportation of Ontario

PAV Pressure Aging Vessel

PG Performance Grade

RTFO Rolling Thin Film Oven

SENT Single-Edge-Notched Tension Test

SHRP Strategic Highway Research Program

SUPERPAVE® SUperior PERforming Asphalt PAVEment

CTOD Crack Tip Opening Displacement

CMOD Crack Mouth Opening Displacement

HMA Hot-mix Asphalt

Symbols

a	length of a sharp crack, m
b	Beam width, 12.5mm
B	Specimen Thickness, m
G_{1c}	Plane-Strain fracture Energy, $J.m^{-2}$
h	Beam thickness, 6.25mm
K_{1c}	Fracture Toughness, $N.m^{-3/2}$
m(t)	m-value
P	Load applied, N
S(t)	Time-dependent flexural creep stiffness, MPa
t	Loading time, s
W_e	Essential fracture energy, J
W_e	Specific essential work of fracture, $J.m^{-2}$
W_p	Plastic or non essential work of fracture, J
w_p	Specific plastic work of fracture, $J.m^{-2}$
W_t	Total energy, J
w_t	Specific total work of fracture, Jm^{-2}

Chapter 1

Introduction

1.1 General Comments on Asphalt Binder Performance

Low temperature performance of asphalt binders is one of the major concerns of many researchers considering resistance of such asphalt pavement to various forms of cracking particularly at low temperature. Hence, cracking is one of the main modes of road deterioration caused by traffic and environmental factors. Invariably, the behavior of asphalt binders can be traced to its composition and modifications. Asphalt binders can simply be defined as "dark brown to black cementitious material in which the predominating constituents are bitumen which occur in nature or are obtained in petroleum processing" according to American Society of Testing and Materials (ASTM)[1]. These materials are complex mixtures of organic compounds (mainly aliphatic and aromatic hydrocarbons) which are commonly modeled as colloids, with asphaltenes as the dispersed phase and maltenes as the continuous phase. It is also described as polycondensed aromatic and heteroaromatic clusters with methyl and long-chain hydrocarbon groups attached and surrounded by less polar aromatic and aliphatic components.[2] The general uses of asphalt binders fall under road paving and as roofing material because of the valuable engineering properties they possess. As a result of its good adhesion to mineral aggregates and viscoelastic properties, it has found a wider application in the construction of roads since it functions as a thermoplastic adhesive (glue) that holds the road together.[3] In other words, roads and highways constitute the largest single use of asphalt binders. In view of the cost of rehabilitation of asphalt pavements and construction of new roads, prevention of premature failure at low temperature has been the major goal of pavement researchers since premature cracking lowers the overall pavement performance and increases long-term maintenance costs for reconstruction.

Moreover, fundamental studies of the highway material have shown that thorough knowledge of failure mechanisms and superior performance of asphalt binders are crucial in order to achieve necessary properties for paving purposes. Hence, asphalt binders are carefully produced from chosen crude oil blends and processed to an appropriate grade. More importantly, the desired properties of the asphalt binder can be improved by introducing additives (usually polymers but sometimes also less desirable chemicals such as acids, bases and other gelling agents) which are blended or reacted with the binder with the aim to enhance the mechanical properties of such asphalt binders. The addition of polymers to asphalt binders (polymer-modified asphalts) has been shown to reduce thermal susceptibility and permanent deformation of roads paved with asphalt binders.[3] Also, this process of modification of asphalt binders has enhanced resistance to low-temperature cracking, thereby improving the general performance of roads over a long period of time.

In addition, the mechanical properties of the asphalt binders play an important role in the development and improvement of performance-based specifications, as a result of this fact, asphalt binders are graded according to the methods developed under the United States' Strategic Highway Research Program (SHRP) which is used throughout most of North America. The grading approach for the asphalt binder was meant to deliver superior performing pavements otherwise known as Superpave® (Superior Performing Asphalt Pavement) pavements. The developed specification was to improve the performance of asphalt binders in terms of their durability, longevity, and quality as well as the life cycle cost of asphalt pavements. This new grading system (Superpave®) became necessary when the conventional penetration and viscosity grading system encountered a lot of difficulties due to the introduction of significant modification techniques adopted for asphalt binders. These frequent modification techniques include the following: polymer-modification, air-blowing, acid-modification and others.[4] In addition, the conventional grading approach of asphalt appears to be empirical in nature since fundamental properties of binders with respect to pavement performance cannot be deduced from the method.

1.2 Deterioration of Asphalt Pavements

Asphalt pavement performance rests mostly on the interactions between asphalt binders and the mineral aggregate surface since the overall performance of the pavement depends on the properties of the mixture of the binder and mineral aggregate.[5] The cause of premature asphalt failures through fatigue cracking, moisture damage or other failure mechanisms have been attributed significantly to the extent of interaction between asphalt binders and mineral aggregate.[7] Furthermore, the presence of air voids has been recognized as one of the potential factors responsible for the pavement failures/distress due to the inhomogeneity between aggregate and asphalt binders, this in turn causes adverse effects on hot-mix asphalt (HMA) due to significant stress and strain concentration at the interface between the two phases, hence resulting in pavement deterioration.[6] In addition, asphalt pavement degradation (road damage) can be aggravated by the following: pavement quality, moisture damage, increased traffic loads, extreme weather conditions (low temperatures particularly common in large parts of North America), pavement design and construction.

One of the major causes of asphalt pavement distress/failure is rutting. This is a longitudinal surface depression running along a pavement's wheel path which may be due to deformation in an asphalt pavement layer or underlying layer. Also, the distress on the asphalt pavement can be the result of fatigue cracking - repeated traffic load flexing the pavement layer which result in jagged cracks that eventually interconnect to form "alligator cracking". Further still, transverse and longitudinal cracks caused by the contraction and accumulation of stresses on asphalt pavement at low temperature are prominent, hence leading to low temperature cracking.[7,8] Considerable improvement has been achieved on the properties of asphalt mastic (composite of mineral fillers and asphalt binder) through the addition of mineral fillers (calcite, quartz). Analysis of the role of mineral fillers in this respect revealed significant improvement on the grading performance of asphalt binders particularly on the resistance to rutting due to considerable increase in the moduli of the mixture of asphalt and the mineral fillers.[9, 10]

Reflective cracking is also considered a main type of distress by which asphalt pavements can deteriorate. This form of pavement distress usually occurs when hot-mix asphalt overlays are used to rehabilitate severely distressed Portland cement concrete pavement, though this could nevertheless be an economical solution to pavement distress. Such pavement systems are prone to develop reflective cracking distress in the hot-mix asphalt because of uneven deformation caused by traffic loads and temperature changes resulting in cracks being initiated in overlays. The cracks will ultimately propagate from the bottom to the surface with time which will cause excessive damage to the pavement if left unchecked. [11]

1.3 Viscoelastic Properties of Asphalt Binders

Asphalt binders are known to show viscous as well as elastic properties depending on climatic conditions such as temperature and loading time. Asphalt binder behaves like a purely elastic material at very low temperatures whereas at intermediate temperatures the behavior tends towards a more viscous material. However, at high temperatures, it can flow completely like water. Hence, materials such as asphalt binders are referred to as viscoelastic materials because they exhibit combined behavior (properties) of elastic and viscous material as shown in Figure 1. In this case, when a constant load is applied to an elastic material, the strain of the material is proportional to the applied stress and when the applied stress is removed from the material, there is a complete recovery to the original position. Figure 1.1(b) describes the behavior of a viscous material in which the strain of the material increases over time under constant stress. Figure 1.1(c) demonstrates the behavior of a viscoelastic material in which a constant stress increases the strain over a long time and when the applied stress is removed, the material fails to attain its original position leading to permanent deformation.

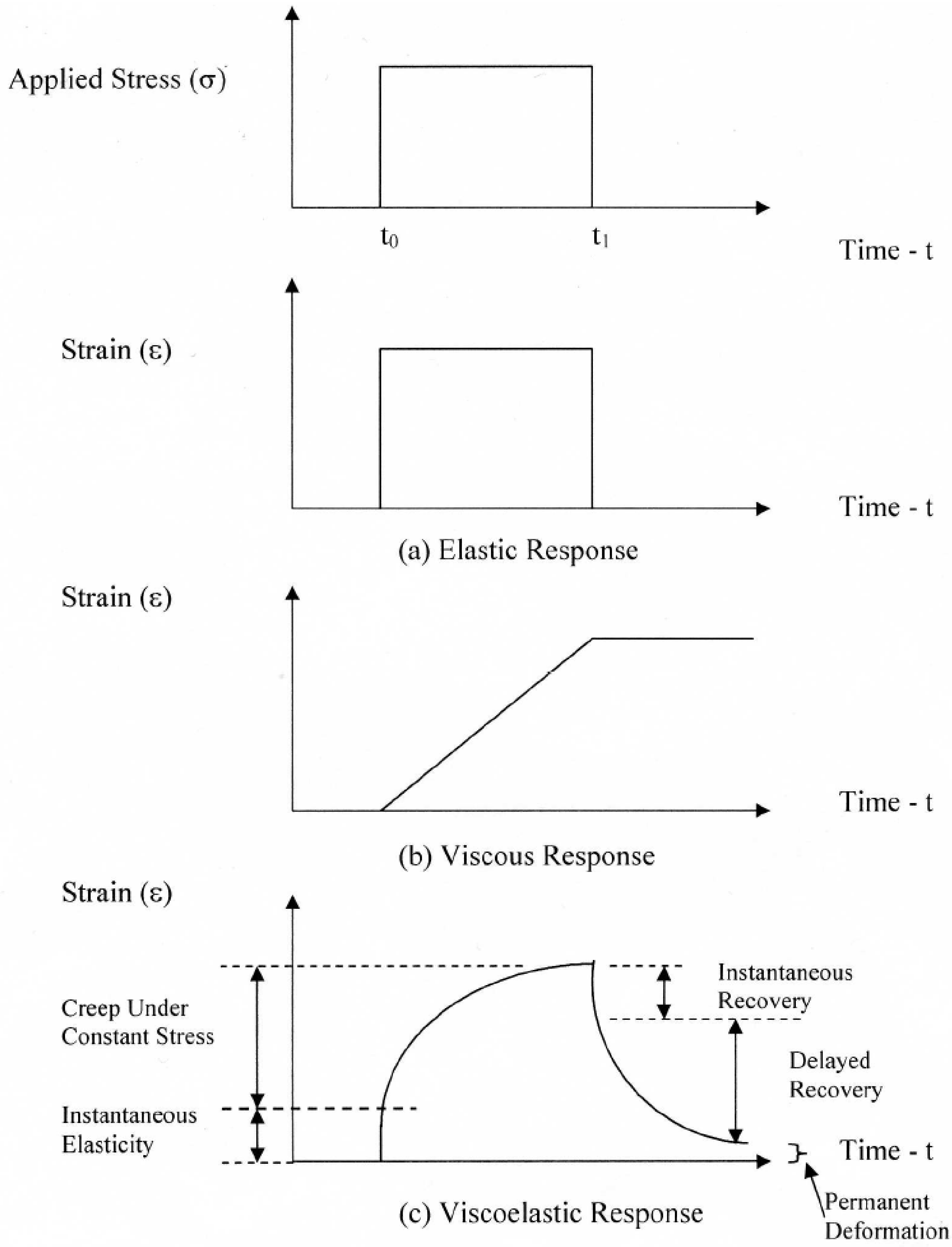

Figure 1.1 Idealized response of elastic, viscous and viscoelastic material under constant stress loading.[12]

As a result of these properties, rheological behavior of asphalt binders via mechanical response to loading conditions is a function of time and temperature. In other words, the properties exhibited by asphalt binders to various loading conditions are time and temperature dependent.

Numerous studies have been conducted to characterize the rheological behavior of asphalt binders and mixtures particularly the viscoelastic properties since the stress-strain behavior of this material in response to load is time and temperature dependent. In addition, this form of behavior is prominent under creep load which causes increased deformation over time. Also, it can be in the form of stress relaxation under constant strain. However, the lag between stress and strain under dynamic loading shows the ability of asphalt binders to store and dissipate energy upon loading. One of the ways of characterizing viscoelastic behavior of asphalt binders is through the stress-strain response under a constant load for a specified time; this is otherwise called a creep test. In this case, when a load is applied on a material like an asphalt binder, there is an immediate deformation which corresponds to the elastic response and then followed by a time-dependent deformation. This form of deformation corresponds to the viscous component of the material - asphalt.[12]

In general, the stiffness modulus of asphalt binders can be defined as

$$S(t) = \sigma/\varepsilon(t)$$

where:

$S(t)$ = time –dependent stiffness modulus (Pa)

t = loading time (s)

σ = applied constant uniaxial stress (Pa)

$\varepsilon(t)$ = uniaxial strain at time t, (m/m)

1.4 Asphalt Aggregate Interactions

A significant number of studies have shown that the interaction between asphalt binders and aggregates in pavements directly affect the adhesion and bond strength which consequently affect the level of crack formation upon traffic loading. In other words, the chemical composition of asphalt as well as aggregates plays important roles in bond formation at the interface between the asphalt and aggregates. Essentially, the ability of asphalt pavements to resist permanent deformation and cracking upon traffic loading at low temperatures depends on the interaction among the asphalt cement, aggregates and air voids. In addition, the degree of interaction between asphalt and aggregates determines the resistance of a pavement to moisture damage (attack), which can lead to cracking. Pavement deterioration can be worsened due to lack of asphalt-aggregate interactions which can easily be caused by action of water (moisture damage or water stripping).[13]

1.5 Performance Grading of Asphalt Binders

Asphalt binders are categorized on the basis of their performance using various methods ranging from simple to complex. These systems of grading asphalt binders have been used to classify the performance and physical properties particularly for paving purposes. Essentially, one of the traditional methods of grading the asphalt performance is by their penetration and viscosity. In the penetration test, a 100 g is weight placed on top of a needle which is allowed to penetrate the asphalt binder for 5 seconds at 25ºC. The depth of penetration of the needle is used as a yardstick to characterize the performance of binders. Likewise, the viscosity of binders is measured through a capillary tube at 60ºC and 135ºC in order to show the fundamental flow of binders (viscous behavior). This measurement can be used to predict the performance of binders although their low temperature behavior is not inclusive. Invariably, this method appears to be empirical since fundamental parameters and properties that are related to pavement performance are not measured.

Further still, the inability of conventional methods to correlate binder properties with pavement performance has led to the development of a new specification called Superpave® (Superior Performing Asphalt Pavements) performance grade in the early 1990's. Indeed, the tests and specification were designed to correlate the binder properties with pavement performance especially with respect to pavement distresses such as rutting, fatigue cracking, low temperature cracking, and other forms of pavement deterioration. Hence, the Superpave® performance grading system appears to be a better tool than conventional methods since binder properties and behavior are being compared at the condition under which it is being used. In other words, various factors such as climatic conditions and aging behavior that are responsible for the performance of asphalt binders were considered in this grading system. Therefore, physical properties of binders were compared directly with field performance under various conditions using the principles of engineering to correlate the results. Besides, two numbers are reported to indicate average seven-day maximum pavement temperature and the second number to indicate minimum pavement design temperature in which binders are likely to experience. Thus, this performance grading (PG) system attempts to properly classify asphalt binders based on their performance.

Recently, a number of studies have shown that binders of the same performance grading based on the currently used Superpave® specification (PG grades) tend to show serious differences regarding their performance at low temperatures. As a result, it becomes necessary to develop a test or specification such as fracture properties of the binders to differentiate fracture performance of asphalt binders using fundamental engineering principles. This approach is meant to enhance pavement performance whereby pavement distresses can be reduced under various climatic conditions through an appropriate choice of appropriately modified asphalt binders.[14, 15]

1.6 Aging Behavior in Asphalt Binders

Aging of asphalt has received extensive attention in recent times in order to investigate such effects on the general properties of binders particularly at low temperatures. Since the behavior of asphalt binders is dependent on temperature and time of loading such that time and temperature can be used interchangeably, the behavior of asphalt at high temperature within short times appears to be equivalent to lower temperatures and longer times (time-temperature superposition principle).[16]

In this case, asphalt aging refers to oxidation reaction which occurs in asphalt binders particularly during hot mixing and construction. This form of oxidation takes place in the presence of organic molecules present in asphalt which leads to an increase in viscosity and hence the binder becomes more brittle in nature. As a result, the asphalt mixture is susceptible to pavement distress. Also, an improperly compacted asphalt pavement has a greater tendency to exhibit oxidative hardening which consequently leads to premature pavement failure. The penetration of air in asphalt mixtures having a higher percentage of interconnected air voids enhances oxidative hardening. In other words, the chemical composition of asphalt binders plays an essential role in oxidative hardening and also affects the level of adhesion (interaction) between aggregate and modifiers of binders.[17] In addition, studies have shown that aging behavior of asphalt can be related to the volatilization of its light components, particularly when heat is applied and progressive oxidation takes place. Hence, repeated heating of asphalt binders during hot mixing with aggregates and construction has a greater tendency to lead to age hardening which consequently leads to premature failure of asphalt pavement. Essentially, aging behavior of asphalt binders can be simulated in the laboratory in order to evaluate the effect of such aging on short and long-term service conditions of pavement. Invariably, the effect of aging is evaluated on the mechanical and rheological properties on the binders. Short-term aging refers to the type of aging that occur in asphalt binders when mixed with hot aggregate (hot-mixed asphalt), while long-term aging refers to aging that occur after HMA pavement is exposed to environmental condition and traffic loadings for a long period of time.[18]

Chapter 2

Background and Literature Review

2.1 Introduction on Materials and Design of Asphalt Pavement

Asphalt pavement performance is dependent on its susceptibility to physical and chemical property changes particularly with respect to time, temperature and other environmental factors such as moisture, oxidation and volatilization. Fundamentally, "the rheological response of asphalt pavement is governed by changes in its rheological properties caused by variation in temperatures and time."[19] Hence, the construction of asphalt pavements involves the use of materials such as modified asphalt binders (polymer-, acid-, or base-modified), crushed mineral aggregates and other additives such as anti-stripping agents. In essence, this form of pavement could be either referred to as rigid or flexible pavement depending on the composition of the materials utilized particularly the sizes of aggregates used and the foundation overlay of the pavement.

Generally, pavements are divided into 3 layers namely: bituminous surfacing (surface course), road base (base course) and sub-base. "Surface course can simply be defined as the top layer of the pavement structure which is in direct contact with traffic loads while base course is defined as the layer of selected material in a pavement structure placed between a sub-base and a surface course."[20] The load bearing layer of the pavement structure is the sub-base layer. Therefore, design of asphalt pavement is considered to be crucial particularly on the performance at different climatic conditions. As a result, special consideration is given to main input during construction of pavement such as choice of soil parameters, thickness and stiffness of the foundation materials. In other words, the design entails use of engineering techniques with principles to construct and maintain flexible asphalt pavement having the ultimate goal of reducing cracking phenomenon on the roads.[19]

Further, the actual material used in paving roads is termed hot mix asphalt (HMA), which is composed of aggregates bound together into a solid mass by asphalt cement (binder). In this case, a properly and well mixed asphalt binder and aggregates are produced by heating to a temperature of approximately 180^{o}C in a central mixing plant. The essence of heating the asphalt is to decrease its viscosity and enhance the formation of a homogeneous mixture. Paving and compaction is done while the asphalt mixture is still hot through the use of a mechanical spreader and rollers, respectively. Hence, construction of the asphalt pavement involves the use of surface and binder courses while other additives (fibres) may be added to reinforce the strength of the pavement.

According to Superpave® specification parameters, as developed in the early 1990's with funding from the U.S. Federal Highway Administration, the design of a HMA pavement requires the knowledge of the traffic load, the drainage system and the sub-grade soil support. This becomes necessary since pavement life can significantly be altered by different traffic loads. Also, poor drainage systems and the load bearing ability of the underlying soil can have significant effects on the general performance of the asphalt pavement. Consequently, the following factors are taken into consideration for a good design of an asphalt pavement:[20]

(i) performance grading of asphalt binders;

(ii) compaction level of the asphalt mixture (air voids);

(iii) aggregate sizes; and

(iv) foundation thickness.

Studies have shown that poor quality mixes have been identified as one of the factors responsible for premature cracking.[21]

2.2 Low Temperature and Fatigue Cracking in Asphalt Pavement

Low temperature cracking, also known as thermal- or cold-cracking, is one of the major distresses that is observed on asphalt pavements especially in North America due to unusually large drops in temperature that can occur at regular intervals during cold winter months. However, this type of distress is also observed in other parts of the world where the temperature not often falls below freezing, such as in desert climates due to large, absolute, daily temperature fluctuations.

This form of pavement distress has been recognized as a commonly existing problem which is induced by the effects of temperature and the quality of the asphalt pavement. Fatigue cracking is closely associated with repeated traffic loads flexing the asphalt pavement layer which is capable of causing jagged cracks that eventually interconnect to form a pattern often referred to as “alligator cracking”. This is illustrated in figure 2.2. In other words, “pavements fail at points well below the ultimate tensile strength or yield strength due to progressive, localized fluctuating strain at nominal stresses”. [22]

Figure 2.1 Longitudinal and transverse cracks typically due to thermal effects.

Figure 2.2 Alligator-type fatigue cracks typically due to load (traffic) distress.

For the past two decades, significant research has shown that the mechanism of this type of pavement failure (that is thermal cracking) occurs when "internal stress is built up and exceeds the strength of the mix which is induced by contraction at low temperature or rate of cooling increases significantly."[23] Essentially, as the temperature drops, this leads to stress growth and consequently results in stress relaxation which ultimately causes a microcrack (small crack) to develop at the edge of the surface of the pavement. The propagation of the microcrack is enhanced by penetration of water on the pavement which eventually leads to moisture damage due to stripping in the asphalt-aggregate mixture.[23] Other potential factors responsible for aggravating the low temperature cracking are pavement age, wax crystallization (reversible aging), additives (certain polymers), pavement design and structure. More importantly, repetitive thermal stress and impact of recurrent loading have a significant effect on the asphalt pavement because of the fact that the mixture of aggregate-asphalt is weakened due to the traffic loads which eventually cause the pavement to fail.[24]

The susceptibility of asphalt pavement to low temperature cracking has received greater attention in recent years which has led to the modification of asphalt binders with different polymers, fillers, and fibers with the aim to develop pavements having improved resistance to pavement distresses (especially thermal cracking) on the asphalt roads. Generally, different types of polymers are used to modify the performance of asphalt binders, these include: styrene-butadiene-styrene (SBS), functionalized SBS, polyphosphoric acids (PPA, H_3PO_4) and others are among the vast number of polymers showing significant impact on the performance of asphalt pavement. "Polymer-modified asphalt mixtures have shown significant improvement on the low temperature performance of asphalt pavement while resistance to thermal and fatigue cracking has been successfully achieved and the pavement life is being elongated through the use of polymers."[25,26] It is interesting to note that a large number of studies have shown that lime treatment of asphalt binders usually decreases the asphalt age hardening thereby improving fatigue resistance in aged pavements and the resistance to low temperature transverse cracking is also improved.[27]

In addition, it is worth mentioning that aggregate fillers (such as hydrated lime) are essentially useful in the improvement of asphalt mixture to withstand premature cracking on the roads thereby extending the fatigue life and tensile strength of asphalt pavement.[28, 29] According to Anderson,[30] tensile strength and fracture properties of asphalt binders are used to rank the rheological properties with respect to low-temperature (thermal) cracking. In this case, the results shown by Anderson appeared to be significant compared to the original Superpave® low-temperature specification for asphalt binders due to the limit on the low-temperature stiffness (S) and m-values obtained for the performance grading of asphalt binders. The concept of fracture mechanics has been introduced in the past years to understand the failure mechanism of asphalt pavement at low temperature which is meant to investigate the fracture properties of asphalt mixture. Also, the various models developed are meant to improve the rheological properties of the binder in which the fatigue life and permanent deformation resistance are aimed to be improved. The ultimate goal of various forms of modification processes of asphalt is to focus on long-lasting pavements with improved performance particularly at low temperature.

Not only that, several test methods that can simulate field conditions and provide reasonable results applicable to mechanistic models have incorporated the concept of fracture in order to accommodate different types of variables such as aging, moisture conditioning and so on. Some of the test methods to evaluate the material properties on low-temperature cracking in asphalt binder and its mixture include the following: indirect tensile testing (IDT), bending beam rheometer (BBR), and thermal stress restrained specimen test (TSRST), single and double edge notched tension specimen tests (SENT and DENT). According to Epps,[31] comparing the results of the test methods on the evaluation of low temperature cracking susceptibility of asphalt mixture, TSRST seems to be more advantageous over others since it simulates field conditions and the responses are applicable to mechanistic models.[31]

A significant number of researchers have studied fatigue properties of asphalt mixture (concrete) based on fracture mechanics approach which is essentially one of the major methods to investigate and study asphalt fatigue cracking through the use of small specimen (beam bending or cylindrical specimen) to determine crack growth parameters.[32] In this case, the research effort is directed towards developing asphalt pavement design to minimize or avoid fatigue cracking. "The fundamental approach on the fatigue cracking entails that one of the performance-based parameter which is used for evaluation of stress concentration condition is stress intensity factor (K_c)."[33] Invariably, this parameter (K_c) can be used to characterize crack propagation in the form of Paris' law as shown below:[33-35]

$$\frac{dc}{dN} = AK_c^n \quad \text{……………..………………………...} \; (1)$$

or

$$\mathbf{N}_f = \int_{Co}^{Cf} \frac{dc}{AK_c^n} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

where

dc/dN = rate of crack propagation (mm/s).

K_c = stress intensity factor ($N/mm^{3/2}$)

C = crack size (mm)

N_f = fatigue life (Number of road repetition)

C_0 = initial crack length (mm)

C_f = critical length (mm)

A and n are material constants - fundamental properties of asphalt concrete.

Also, the presence of air voids in asphalt concrete has been a recognized initial crack (flaw) under which repeated loadings cause initial microcracks to gradually develop to form large dominant cracks. Consequently, the macrocracks formed would eventually lead to failure of the asphalt pavement or permanent deformation. Hence, fatigue cracking is considered to be a two-stage process which can be categorized as follow: crack initiation and crack propagation.

According to Lytton et al.,[36] the first stage of the cracking process is called crack initiation, "it can be described as a process by which initial cracks such as air voids develop and coalesce to form a macrocrack on the asphalt pavement where the shape and the size of the cracks become visible."[36] Not only that, crack propagation is followed also under repeated stress. Therefore, "crack propagation is described as the period for the dominant cracks to grow and further develop to form a critical size leading to pavement distress."[36] In a nutshell, asphalt *fatigue* testing could be used to determine crack initiation life based on a micromechanical model. In addition, the difference between crack initiation and crack propagation is figured out from the analysis on the rate of dissipation of energy or reduction in stiffness during asphalt fatigue testing.[37, 38]

Considerable research has revealed that extensive beam fatigue testing could be conducted under controlled stress and controlled strain where variables were investigated such as asphalt type, asphalt content, aggregate stripping potential, degree of compaction, temperature, and stress/strain level.[39] Under controlled strain specimen tests, the amplitude of deformation or displacement is kept constant under controlled strain test and the specimen is considered to fail once the stiffness is reduced to half of its original value while in the case of controlled stress test, the applied force to the specimen is maintained and the specimen is considered to have failed once it breaks.[39]

2.3 Reversible Aging in Asphalt Pavements

Reversible aging is a common phenomenon that is observed in asphalt cement and it can be used to describe the processes that asphalt binder undergoes when it is stored at low temperature for an extended period of time. This process entails “subtle yet important structural changes that asphalt binders undergo at cold temperature storage such as wax crystallization, free volume collapse, and asphaltene aggregation.”[40] Hence, the phenomenon was thought to be caused by the existence of a thermally unstable structure within the binder.[40] Consequently, this process of reversible aging becomes more pronounced as the temperature falls significantly, asphalt binder shrinks in volume and this leads to increase in asphalt hardness even though the temperature stabilizes at a constant low value. As a result, the rheological and fracture performance of asphalt pavement is reduced during low temperature conditioning.

In addition, it has been recognized from early work on reversible aging that air-blown binders reversibly age more significantly due to the reaction with oxygen (oxidation) which depletes the resins fraction to form more asphaltenes with associated changes in morphology and consequently a more brittle structure. As a result of the process of reversible hardening, asphalt pavements are more susceptible to cracking. Further, a newly constructed asphalt pavement may be excessively prone to oxidative hardening if it is not compacted properly such that excessive air voids could allow greater amount of air to penetrate the asphalt mixture leading to premature failure of the pavement. However, the morphological and chemical features responsible for reversible aging are still quite uncertain in the literature.[41] In recent times there is an indication that the wax content of asphalt binders is one of the major contributing factors for reversible aging which presumably causes premature and excessive low temperature cracking.[40]

According to Struik,[41] a qualitative free volume collapse theory was proposed in order to give explanation on reversible aging particularly with respect to some of the observations of asphalt binders at low temperature storage. This extensive study on physical hardening was further strengthened by the Strategic Highway Research Program in which a conclusion was drawn on the fact that high wax content in asphalt binders is most likely responsible for the susceptibility of asphalt to reversible aging.

The low temperature performance grade of asphalt binders can be measured by the extended bending beam rheometer (eBBR) test in which the stiffness and relaxation ability of each asphalt binder is evaluated after going through a physical aging (reversible aging) process. Invariably, the effect of extended exposure of asphalt pavement to cold temperature can be simulated by conditioning the physically aged sample of asphalt binders at low temperature for periods of 1, 24 and 72 h and subsequently, extended BBR testing is carried out on the samples. In this case, the samples of asphalt binder are aged in the pressure aging vessel (PAV) system where they are heated under pressure (2.0 MPa) for extended time (20 h) to simulate long term field aging.[42]

Further, the aged samples (PAV residues) are tested at two different temperatures corresponding to those for which asphalt binders are expected to pass the limiting BBR criteria (stiffness less than or equal to 300 MPa and m-value (slope of the creep stiffness master curve) greater than or equal to

0.3) and when asphalt binder just reaches or fails the limiting BBR criteria respectively. The limiting temperatures of the asphalt binder are measured for 1, 24 and 72 hours where the stiffness (S) at 60 seconds of loading reaches 300 MPa or the slope of creep stiffness (m-value) reaches 0.3 based on the specification criteria.[40] Hence, the asphalt binders are ranked for their reversible aging tendency from a determination of the worst grade loss based on the warmest limiting temperatures measured.

Significant and in-depth studies on reversible aging have been conducted in the 1930s by Traxler and Schweyer in which the process was initially described as age hardening, steric hardening, physical aging and more recently, it is called physical hardening which is completely distinguished from oxidative aging.[44,45] As a result of research findings, a conclusion was drawn in which the cause of physical hardening was presumably based on free volume collapse by the formation of crystalline wax and that asphalt binders with higher content of wax were likely to show stronger physical hardening both below and above their glass transition temperature.[46] However, there is still much uncertainty in the literature on the correlation of reversible aging to the performance of asphalt binders with respect to asphalt mixture and pavement.

2.4 Low Temperature Fracture Testing of Straight and Polymer Modified Binders

Early investigations on failure in both modified and unmodified asphalt binders including their asphalt mixtures suggested that the stiffness and modulus of asphalt binders are measures or yardsticks that reveal the rheological conditions, and that fracture properties of binders depend on these parameters (that is stiffness, modulus, toughness, and strength). Hence, "the effect of traffic loading and temperature can be quantified on the fracture properties of the binders through changes in stiffness and the modulus parameters of such binders." [47] In the case of asphalt mixtures, it was revealed that the stiffness parameter only reveals the fracture properties of the asphalt mixture but also that "mix factors" play an essential role in the failure of asphalt pavement. In other words, the so-called "mix factor" depends on things such as the proportion of asphalt cement, the grading of the minerals, and the compaction of the mix (air voids ratio).[48]

Further studies on low temperature fracture testing of asphalt binders have shown that the concept of stiffness parameter to understand the fracture properties at low temperature or as a binder's specification parameter for the failure at low temperature is inadequate since it was assumed to be valid for unmodified asphalt binders.[48] Although, the concept of stiffness as a parameter for grading the performance of asphalt binders was initially accepted until different modification methods were introduced to produce modified asphalt binders, as a result, an improved method was developed by researchers under the Strategic Highway Research Program (SHRP) in the early 1990s.[48,49] The new method developed under SHRP was meant to address the problem of fracture properties at low temperature for asphalt binders with a special consideration that stiffness concept initially proposed might not be sufficient for the characterization. Thus, the new method involves the use of a direct tension test (DTT) which was supposed to complement the 3-point bending beam rheometer test (BBR).

Since asphalt binder is a viscoelastic material, the BBR developed can be used to show creep stiffness as a function of time (S(t)) and relaxation ability (stress relaxation or m-value) under traffic loading. “Creep stiffness of binders is defined to be the stiffness at a constant loading for a specified time.”[48,50] Invariably, this m-value reflects the ability of binders to reduce thermal stress through viscous flow at long loading times, hence, relaxation ability (m-value) could show rheological and failure characteristics of the binders. Recently, there have been a lot of concerns as to the ability of the bending beam rheometer and direct tension test to predict the performance of binders at low temperature.[50,51]

2.5 Essential Work of Fracture on Asphalt Binders

The concept of essential work of fracture has been used in recent times to determine the failure resistance of asphalt binders in their ductile state in order to evaluate the performance grading both in the field and the laboratory. In other words, the ability of asphalt binder and their corresponding mixtures to resist failure in the ductile state can be estimated through this concept. Since many ductile materials are in their elasto-plastic state at ambient temperatures, the concept of essential work of fracture theory was proposed by Cotterell and Reddel for assessment of fracture toughness of such materials.[53, 54]

Strictly speaking, the concept of essential work of fracture in ductile materials (polymers) is based on thermodynamic method which states that "the work necessary to fracture a pre-notched elasto-plastic specimen can be divided into two parts: the essential work performed in the end region and non essential work performed in the screening plastic region."[53] Hence, the total work of fracture is the sum of an essential work of fracture (w_e) spent in the end regions ahead of a crack tip (i.e. the fracture zone and the non-essential plastic work (w_p) dissipated in the outer region).[52-54]

Significant studies on this concept revealed that the essential work of fracture (w_e) is proportional to the fracture area (i.e. ligament length (l) multiplied by sample thickness (B) while non essential work is proportional to the volume of the plastic zone). Plastic zone simply refers to the fracture area (lB) multiplied by ligament length and β-factor which depends on the shape of the plastic zone. Mathematically, the total work of fracture (w_f) can be expressed as:

$$W_f = W_e + W_p \quad \text{.................................. (3)}$$

$$W_f = lBw_e + \beta l^2 Bw_p \quad \text{......................... (4)}$$

$$w_f = W_f/lB = w_e + \beta l^2 w_p \quad \text{...................... (5)}$$

where

w_e = specific essential work of fracture (J/m^2)

w_p = specific plastic work of fracture (J/m^3)

l = ligament length in the DENT specimen (m)

B = thickness of the sample (m)

β = shape factor of the plastic zone.

In a nutshell, the specific terms w_e and βw_p determined in plane stress or mixed mode (plane stress/plane strain) have been shown to be material properties, since these parameters are independent of the thickness of the materials tested.[55,56]

Fundamental research on this concept revealed that the plotting of total work of fracture (w_f) against ligament length (l) should give a linear relationship where the specific essential work of fracture is extrapolated when the ligament length is zero. In most cases, when the ligament length falls within a plane strain/plane stress condition, extrapolation to zero ligament length is uncertain while the linear and power curve fitting have been proposed for the extrapolation of specific essential work of fracture.[53,57] In addition, several studies have shown that specific essential work of fracture (w_e) can be estimated reasonably well via crack tip opening displacement (CTOD) using the relationship below:

$$w_e = M\,\sigma_y\,\mathrm{CTOD} \quad \text{.......................................} \quad (6)$$

M = plastic constraint factor for DENT = 1.15;

CTOD (δ_t) = crack tip opening displacement (m);

σ_y = net section stress or yield stress (N/m^2).

The parameter CTOD (δ_t) indicates crack initiation/propagation via specific essential work.[58] Further still, the concept of essential work of fracture has been applied to a wide range of ductile materials such as pure and blended polymers, polymeric films, metals, and glass-reinforced polymers. Based on this concept, fracture behavior of asphalt binders can be determined via crack tip opening displacement (CTOD) as shown in equation (6). Recently, double–edge-notch tension test has been adopted for asphalt binders in order to determine master curve of ductile failure where essential work and plastic work of fracture on asphalt binders and mixtures are determined through approximate critical crack tip opening displacement.[58]

In addition, the concept of essential work of fracture was validated for the fatigue grading of asphalt binders after a series of asphalt binder samples were tested both in the laboratory and field (test roads) through U.S. Federal Highway Administration's pavement testing facility.[59] It was revealed that the crack tip opening displacement parameter could be used as a property that correctly ranks performance and provides a high correlation with cracking distress. In other words, CTOD appears to be a proven fracture mechanics parameter and provides a measure of strain tolerance in the presence of cracks or flaws in the brittle-to-ductile transition and ductile state. Hence, the actual failure processes in the pavement usually occur around stress concentration (notches through broken aggregate particles and voids provide crack initiation sites). It is believed that the differences in the performance grading of binders can be explained using the concept of essential work of fracture and CTOD parameters.[59]

2.6 Asphalt Mixture Testing

A series of studies have been going on for some time on asphalt mixture testing in order to correlate the performance of asphalt binder and its corresponding mixture with the aim to establish a parameter (performance-based property or material property) for low temperature cracking on the roads (asphalt mixture). It appears that the binder content, binder type, aggregate type and shape are primarily responsible for the various form of cracking (reflection, fatigue and low temperature cracking) in the sense that these factors indirectly affect the performance of asphalt mixture.[60] As a result, it becomes necessary to evaluate the effects of aggregate shapes and type on the performance of asphalt mixture. In fact, it was observed that mixtures with lower stiffness have greater tendency to resist fatigue cracking under controlled-strain loading when compared with asphalt mixture having higher stiffness and rough textured aggregates.[60,61]

According to some recent research, the essential work of fracture concept was proposed for the evaluation of failure properties of asphalt mixture at ambient temperature since the performance based parameter such as specific essential work of fracture (w_e) appeared to show a correlation between the asphalt mixture and field distress. In addition, the thermal stress restrained specimen test (TSRST) is particularly used for evaluation of low temperature cracking resistance and aging performance of modified asphalt mixture specimens.[62]

Another fracture mechanics-based test for fracture testing of asphalt mixtures is crack mouth opening displacement (CMOD). The development of this test was made after ASTM standard test method E 1290-93 for CTOD fracture toughness measurement. In this case, it is believed that CMOD is meant to investigate the possible differences in the performance of asphalt mixtures. The measurement of CMOD of a notched sample of asphalt mixture is carried out using a clip-on gauge (MTS model) and the crack opening approach can be validated by comparing the mixture sample results with corresponding results of the binders crack tip opening displacements. It is generally believed that this concept can show differences in the performing grading of asphalt mixtures.[63,64]

Chapter 3

Experimental

3.1 Materials

3.1.1 Highway 417 Pavement Trial Materials

Asphalt binders investigated for this study were obtained during the construction of a major pavement trial on Highway 417 just east of Ottawa, Ontario. Seven different straight and modified binders were stored in sealed cans at ambient temperature from their sampling during late summer 2006 until their testing in summer 2007. All tested samples were aged in the rolling thin film oven (RTFO) and pressure aging vessel (PAV) (100^0C for 20 hours) according to American Association of State Highway and Transportation Officials (AASHTO) methods at the Imperial Oil Research Laboratory in Sarnia, Ontario prior to testing at Queen's University.[58, 66] A comprehensive list of pertinent properties of the samples used in this study is given in table 3.1. Samples are labeled from 417-1 to 417-7 and numbers correspond to the respective section in the pavement trial.

Table 3.1 Pertinent asphalt binder properties[1]

Asphalt Binder (Sections)	Sources	Modification Type	PG Grade
417-1	Lloydminster	3 % radial SBS	64-34
417-2	Lloydminster	7 % radial SBS	70-34
417-3	Cold Lake	7 % D-1101 linear SBS	82-40
417-4	Prairie Blend	3.3 % Elvaloy 1052 RET and 0.4 % PPA	70-34
417-5	Prairie Blend	2.1 % Elvaloy 4170 RET and 0.4 % PPA	64-40
417-6	Unknown base Asphalt	SBS (unknown grade and concentration)	76-34
417-7	Unknown Control	Control - SB diblock copolymer and PPA (unknown grade and concentration)	64-34

[1]**Note:** SBS = Styrene-Butadiene-Styrene Copolymer, PPA = Polyphosporic Acid. SB = Styrene–Butadiene Copolymer, RET = Reactive Ethylene Terpolymer, PG = performance grade of asphalt binders.

Binders for sections 417-1 and 417-2 were prepared with base asphalt from Lloydminster, Alberta. Binder for section 417-3 was made with base asphalt from Cold Lake, Alberta. The origin of the base asphalts from sections 417-4 to 417-7 is unknown.

3.1.2 Recovered Asphalt Cement from Highway 62 Contract

In addition to the binders tested for the Highway 417 trial (table 3.1) two recovered asphalt cements from large eastern Ontario contracts were also tested in the extended BBR method. Both contracts were located on Highway 62 ((1) north of Bloomfield and (2) around Bannockburn). The difference between the contracts was their performance. The Bloomfield site was almost free of any distress after 14 years of service while the Bannockburn site had cracked prematurely and excessively in early life.

In order to recover the asphalt binders from field samples, about 4 kg of core samples were cut into smaller pieces and these were soaked in tetrahydrofuran (THF) for about 10-12 hours. Subsequently, a graduated cylinder was used for the sedimentation of aggregate fines before the evaporation of the solvent using a rotatory evaporator at high vacuum and heat. This procedure was repeated several times in order to recover the asphalt binder completely from the core samples. At the end of the recovery process, asphalt binders were heated for 1.5 h at 150°C and the aspirator pressure was at 20 mm Hg so as to remove the solvent completely from the recovered binders and to prevent oxidation or hardening. In a nutshell, approximately 200 g of asphalt binder was recovered from each of the samples (cores from pavement contracts) and these samples were used as a template to understand the physical hardening in the extended BBR test and the ductile failure of the binders in the double-edge-notch tension (DENT) test.[70]

3.2 Methodology

3.2.1 Binders Aging

- RTFO Samples

According to the Strategic Highway Research Project (SHRP) which was concluded in the early 1990s, an aging procedure was proposed to simulate short and long term field oxidative aging of asphalt binders in order to understand the failure mechanism of asphalt pavement. This proposal involves development of the Rolling Thin Film Oven (RTFO) and the

Pressure Aging Vessel (PAV) for simulation of short and long term aging, respectively. While RTFO samples of asphalt binder is used to simulate short term aging (hot mixing of asphalt and aggregate), PAV asphalt sample is used to simulate “in service” aging of binders as a result of the effect of time, temperature, traffic load and environment.[74] In other words, these procedures are meant to mimic field aging of binders in order to understand changes in rheological properties of the binders.[74]

Short-term aging of asphalt binders in the RTFO entails the following: First, about 35g of asphalt sample is measured into a transparent glass tube while the temperature of the RTFO machine is set to 163^{o}C (325^{o}F) and the air flow adjusted to 4 L/min. Second, samples of asphalt binders in the tubes are arranged on the RTFO machine as shown in Figure 3.1

Figure 3.1 Typical rolling thin film oven (RTFO) used for short-time aging of asphalt binders.[69]

Meanwhile, the sample is allowed to rotate continuously for 85 minutes in order to induce oxidative hardening (age hardening). This is typical of the aging that occurs during the hot mixing process or the early stages of the pavement life and it is expected that the aged samples would reflect the rheological changes in the properties of the asphalt binders.[75]

This test method (RTFO test) is designed to determine the effect of heat and air on the moving film of asphalt. Invariably, this test serves as an indicator of the approximate change in properties during conventional hot mixing. Figures 3.2 and 3.3 show the process of short term aging of binders.

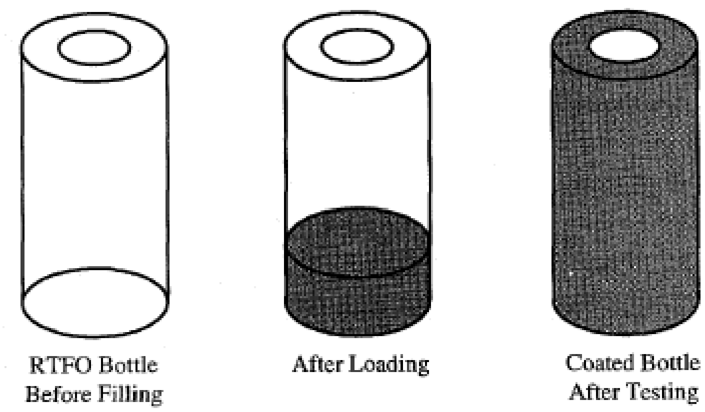

Figure 3.2 RTFO bottle, sample before and after aging.

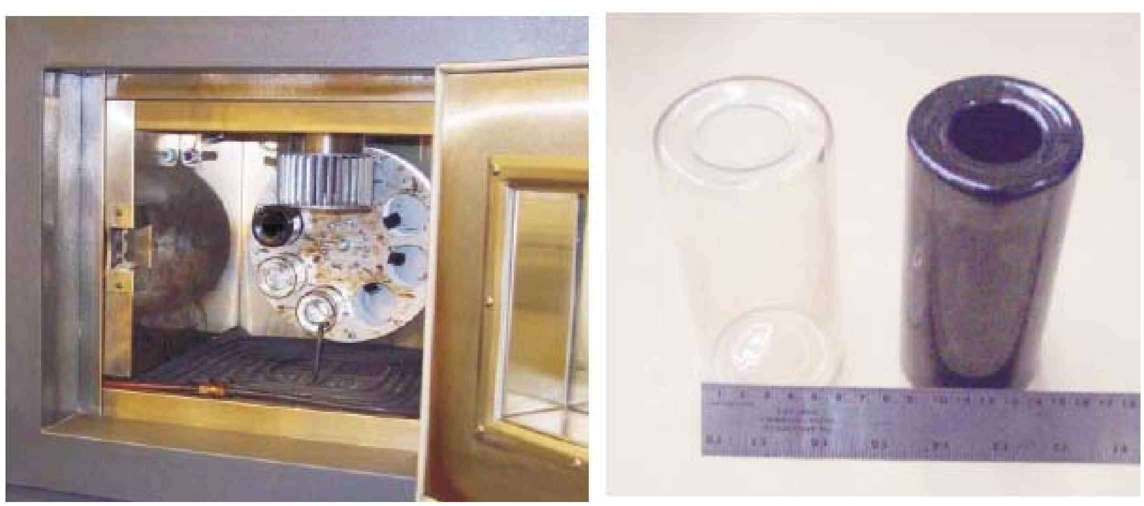

Figure 3.3 RTFO with array of transparent bottles used for aging purpose.

- PAV Samples

On the other hand, the pressure aging vessel is used to simulate long-term aging of pavement. The procedure of the test is explained as follows: Samples of asphalt binder are first of all aged by the RTFO method as explained above and these samples were further aged in a standard steel pan for 20 hours in a vessel (PAV) pressured with air to 2.1 MPa at a temperature of 100^{o}C (212^{o}F). Since the pressure aging vessel system is considered to simulate the aging caused by oxidation during 7-10 years of pavement service, it is expected that PAV-oxidised asphalt binders could be used to estimate the rheological properties of binders after significant aging on the road. A schematic diagram of the PAV system is shown in Figure 3.4 below.

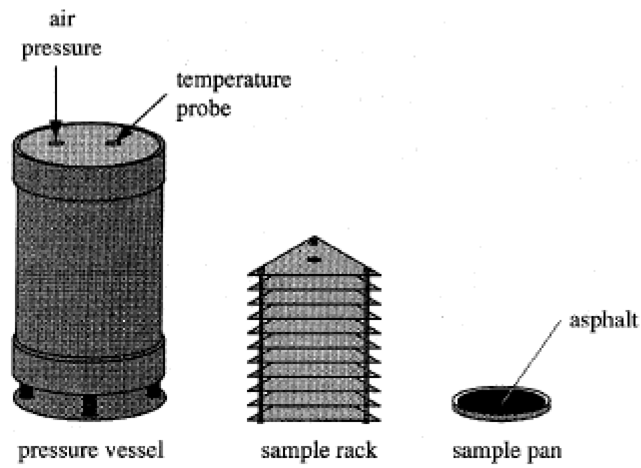

Figure 3.4 Pressure aging components used for long term aging.[69]

3.2.2 Extended Bending Beam Rheometer (eBBR) Protocol

Samples of seven asphalt binders were prepared and tested using the extended bending beam rheometer (eBBR) protocol in order to measure the stiffness and creep rate accurately at various temperatures representative of the lowest pavement temperatures. The aim of this testing is to evaluate the susceptibility of each of the binders to reversible aging/physical hardening particularly at low temperature based on the new method developed in Ontario. The result of the testing is meant to provide vital information on the critical cracking temperature which is considered to be the point at which the maximum stress at a given temperature exceeds the strength of the material.[24,66] Figure 3.5 shows a schematic diagram of the BBR test.

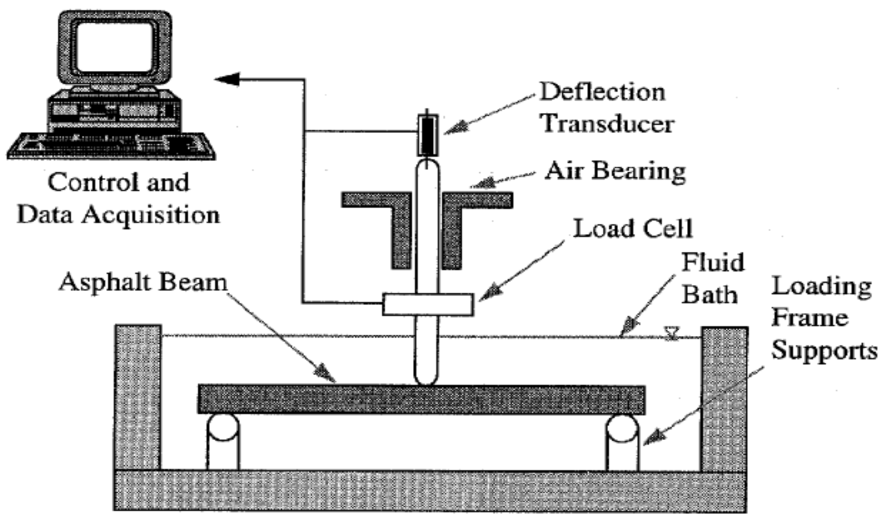

Figure 3.5 Schematic diagram of Bending Beam Rheometer connected to computer output for control and data acquisition.[72]

According to the American Association of State Transportation and Highway Officials Standard M320 on testing asphalt binders,[67] samples were heated in the oven at 160°C for approximately one hour in order to reduce their viscosity and improve homogeneity by stirring. Subsequently, these samples were poured into aluminum molds greased with petroleum jelly while acetate strips were placed against the greased faces of dimension specified by the standard (length = 101.60 mm, width = 6.35 mm, height = 12.70 mm). In addition, the end pieces of the mold were treated with release agent composed of glycerin and talc mixed to achieve paste-like consistency. A typical asphalt beam made in this process described above is shown in Figure 3.6.[69]

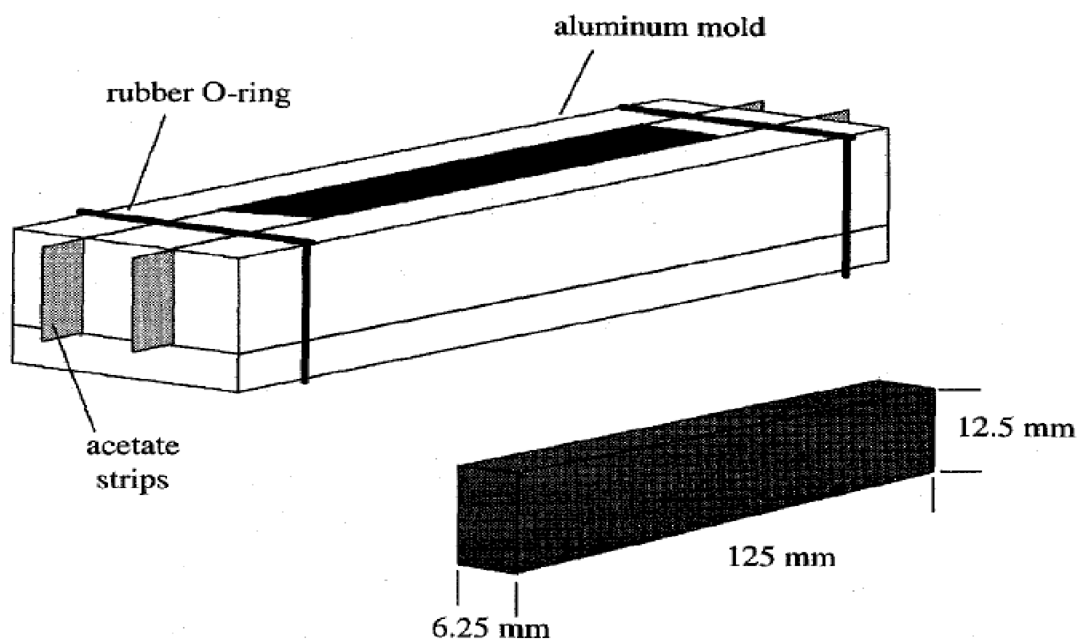

Figure 3.6 Asphalt mold assembly and dimensions of asphalt beam.

Thereafter, samples of asphalt binder were carefully demolded while the excess asphalt was trimmed from the upper surface using a hot spatula after cooling. Asphalt beams were subsequently conditioned at the following temperatures: -10^{o}C, -20^{o}C, -30^{o}C. Samples were conditioned for periods of 1, 24 and 72 hours prior to testing for their critical properties (S = 300 MPa and m-value = 0.3). The samples were tested in the BBR at different temperatures in which a constant load was applied continuously at the centre of the asphalt sample beam for approximately four minutes.

Creep load (100 g) was used to simulate the stress that gradually builds up in a pavement when the temperature drops. Two important parameters were evaluated from the BBR test on asphalt samples so as to determine the limiting temperature of the binders, and consequently provide an indication of the low temperature stiffness and cracking potential of each of the asphalt binders (grade loss). Firstly, the so-called m-value is obtained which is a measure of how asphalt stiffness changes with loading time as load is applied at a loading time of 60 seconds. Secondly, the stiffness at 60 seconds loading is also obtained as a secondary specification criterion.[67] When the m-value reaches 0.3 or the creep stiffness reaches 300 MPa the limiting temperature is reached under the current AASHTO M320 specification. Creep stiffness and m-value measurements are shown in Figure 3.7 and 3.8, respectively. Fundamentally, the basic beam theory on which the BBR calculates beam stiffness (S (t)) and the rate of change of the stiffness (m-value) as the load is applied is stated below[67]:

$$S(t) = \frac{PL^3}{4bh^3\delta(t)}$$

where: S (t) = creep stiffness at time, t = 60 seconds;

P = applied constant load (980 ±20 mN) obtained using a 100g load. (Note that 100 g multiplied by force of gravity (9.8 m/s^2) equals 0.98 N or 980 mN;

L = distance between beam supports, 102 mm;

b = beam width, 12.5 mm;

h = beam thickness, 6.25 mm; and

δ (t) = deflection at time, t = 60 seconds.

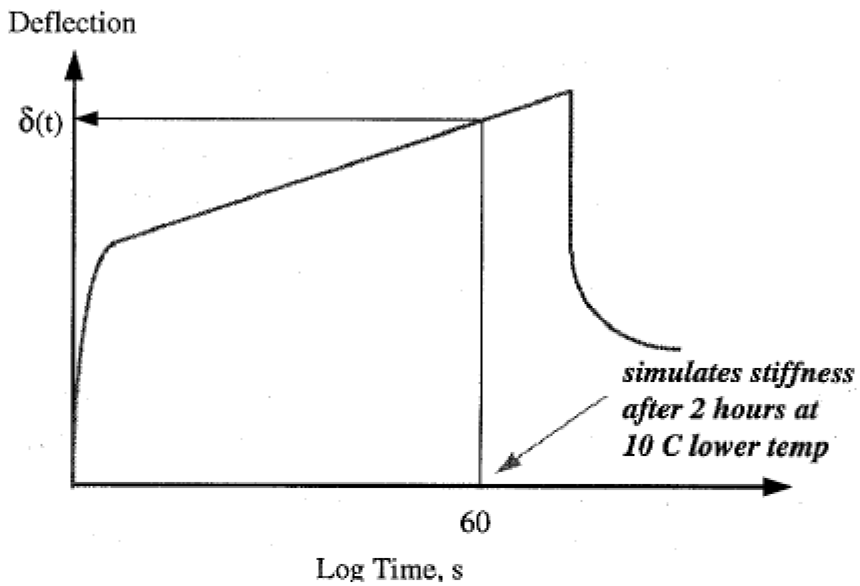

Figure 3.7 Creep stiffness of asphalt binders based on change in deflection with time.[69]

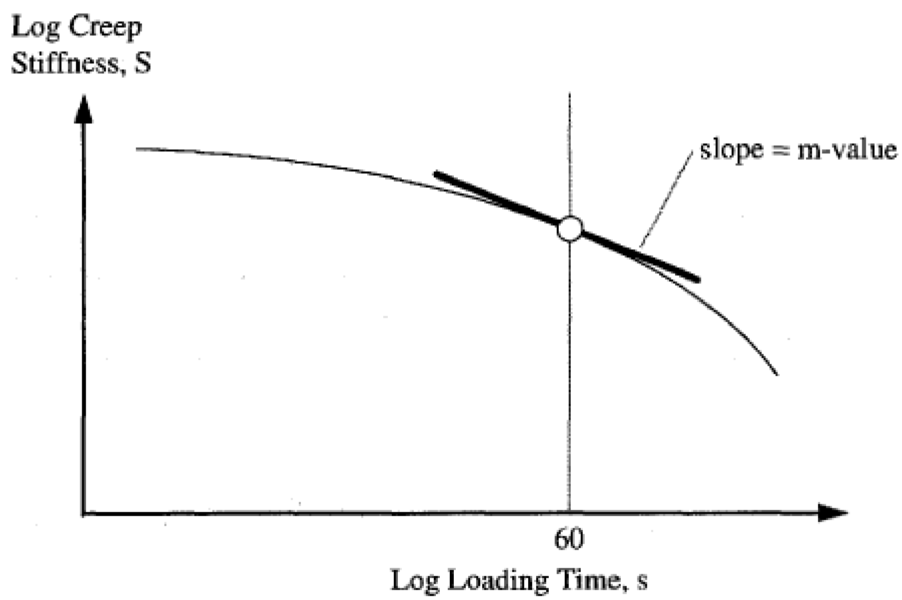

Figure 3.8 Evaluation of m-values of asphalt binders.[69]

The m-value is determined as the slope of the logarithm of stiffness versus the logarithm of time curve at a specified time of 60 seconds of loading as shown in Figure 3.8. Secondly, the creep stiffness is evaluated which is a measure of how asphalt binders resist constant loading typical of thermal loads. The deflection of the asphalt beam under the BBR test is shown in Figure 3.9

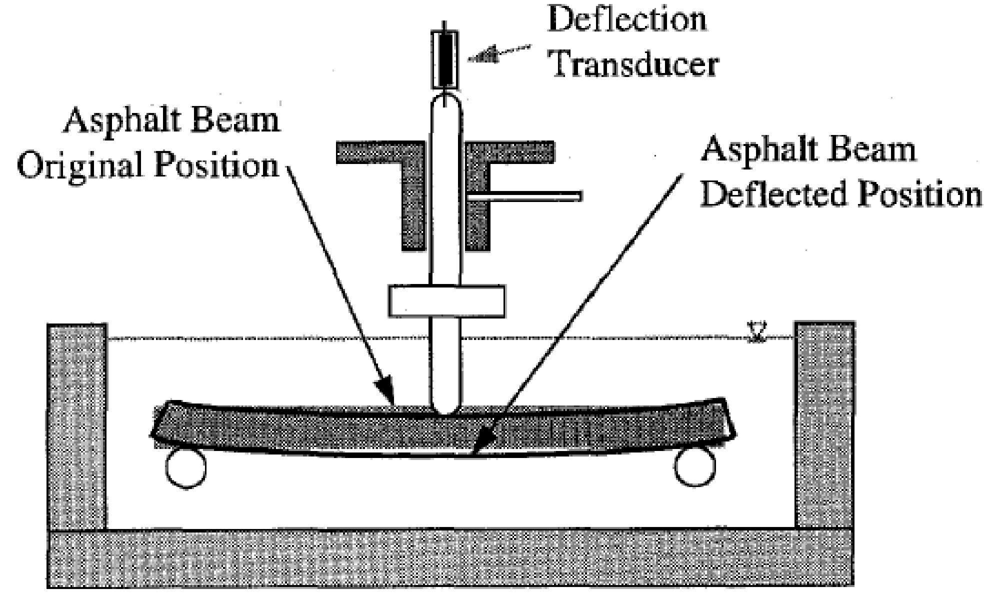

Figure 3.9 Deflected asphalt beam on bending beam test.[69]

The argument for including an extended BBR test rather than one that solely relies on the test after a single hour of conditioning at the test temperature is based on the fact that an asphalt pavement layer usually shrinks in cold weather over extended time periods (days, weeks and months), which eventually leads to low temperature cracking. Consequently, pavement tensile stress will exceed tensile strength resulting in cracking under low temperature conditions. Hence, the determination of the worst grade loss for the asphalt binders' samples after three days of conditioning compared to the 1 h values enables one to make a more accurate assessment of the true performance properties for such material.[40]

In a nutshell, the extended creep test helps to determine the critical cracking temperature of the samples of asphalt binders where the limiting temperature is figured out from the analysis of the data obtained from the testing after various conditioning times. Hence, the asphalt binders have time to come closer to thermodynamic equilibrium and a state that better reflects their performance in the field where samples are conditioned for weeks and months prior to the arrival of a serious cold spell.[68]

3.2.3 Double-Edge-Notched Tension (DENT) Testing Protocol

Two sets of asphalt binders were tested in the double-edge-notched tension (DENT) test: (1) two recovered asphalt binders from pavement contracts (field cores) on Highway 62 under investigation and (2) those asphalt binders aged in the RTFO/PAV as listed in Table 3.1.

Furthermore, ductile failure of Highway 417 binders aged in pressure aging vessel (PAV) according to AASHTO standards were tested at different temperatures in a force-ductilometer with the displacement rate at 50 mm/min. In other words, asphalt binders were heated for about one hour in the oven at 160-165^{o}C to ensure that sample (asphalt binders) readily flow when dispensed from the container and then poured into prepared silicone (DENT) molds having two aluminum end pieces inserted into each of the molds as shown in Figure 3.10

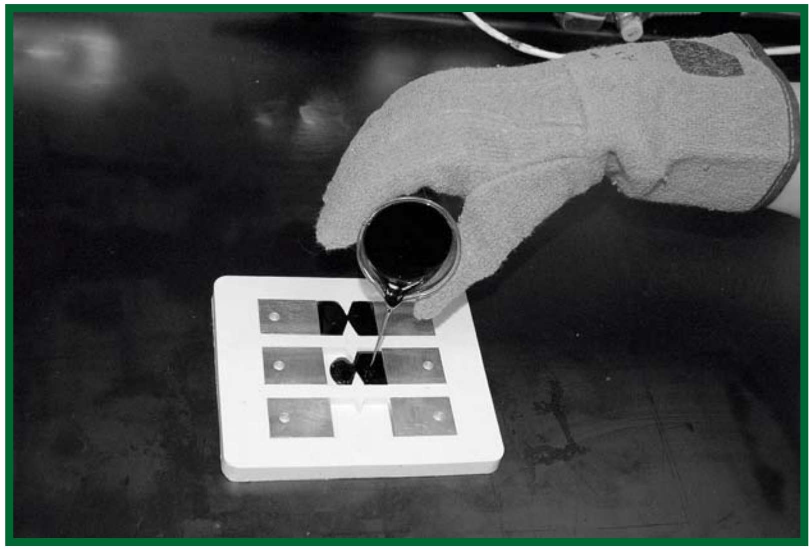

Figure 3.10 Pouring of asphalt binder in the silicone molds.

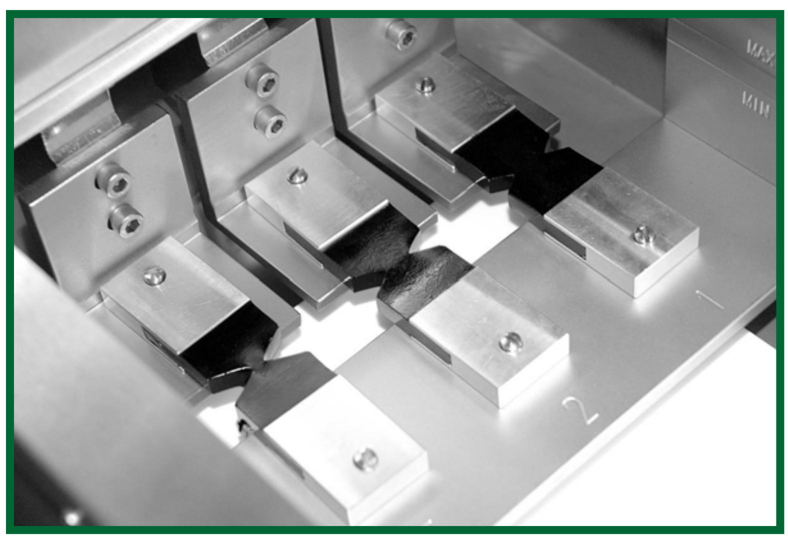

Figure 3.11 Double-edge-notched tension test set-up.

Moreover, samples were conditioned at ambient temperature for 24 hours before they were removed from the silicone mold and were loaded into the force-ductilometer apparatus where asphalt binder samples were re-conditioned in the water bath for about 30 minutes at different temperatures (4, 10, 16, 22, 28, 34, and 40^{o}C) before the test was carried out as shown in Figure 3.11

The recovered samples from the pavement contract (cores samples) were tested at 15^{o}C and 50 mm/min in order to evaluate the ductile failure properties of asphalt binders.

3.2.4 Compact Tension Testing Protocol

The compact tension test is commonly used to determine the fracture toughness of polymers using fracture mechanics concepts. This concept is used to determine the fracture toughness of asphalt binders in order to establish performance grading of asphalt binders at low temperatures in their brittle states. The test procedure entails the following: First, asphalt binder is heated in an oven at approximately 165^{o}C for about one hour in order to reduce the viscosity and increase the ability to fill a silicone mold. The heated asphalt binder is poured into the silicone molds which already contain aluminium end pieces (inserts), this ensures that the required geometry for the asphalt sample was obtained before the test was carried out. The asphalt sample in the silicone mold was allowed to cool for about 10 minutes before a sharp knife was used to scrap the excess asphalt from the silicone mold so that the appropriate size and geometry of the asphalt sample is consistent and accurate. After about one hour, the asphalt sample is removed from the silicone mold and was stored in the refrigerator (freezer) for 20 hours at a temperature of -20^{o}C. Furthermore, after conditioning the asphalt sample at -20^{o}C, it was reconditioned for 10 minutes in the AsphaltPro® 5525 machine containing a coolant (potassium acetate or antifreeze solution) and asphalt samples were tested at the following temperatures: -22^{o}C, -28^{o}C, -32^{o}C, while the respective fracture toughness (K_{1c}) was determined in accordance to the procedure of LS-296.[72,73]

Essentially, compact tension (CT) testing of the asphalt sample was carried out with Instron's AsphaltPro® machine where tensile tester having two special grips with integral knife-edges that fit into the aluminums end pieces of the asphalt specimen were used to carry out the test. Usually, coolant solution in the machine ensured that the test temperatures were held within approximately (± 0.1^{o}C) while the loading rate was kept constant at

0.01 mm/s for each of the test. A schematic diagram of the test set up is shown in Figure 3.12.

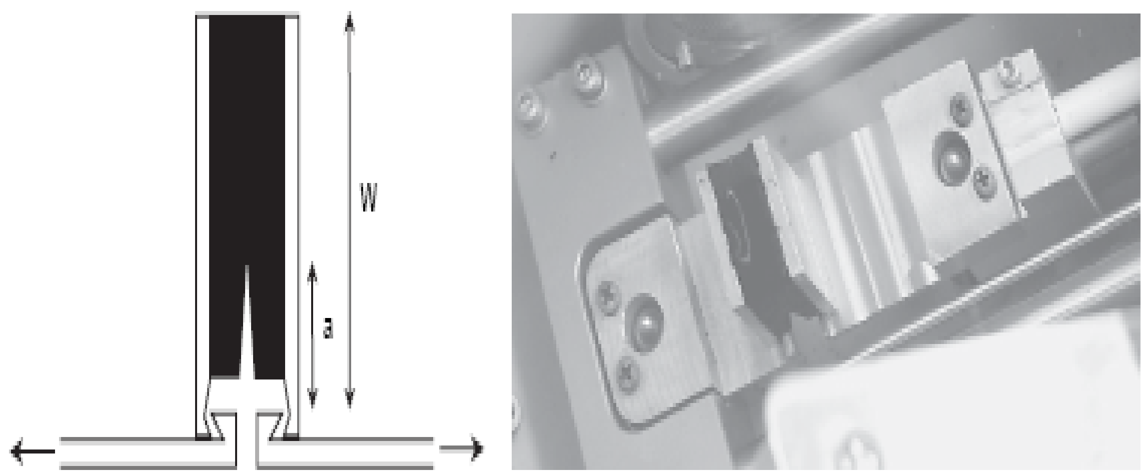

Figure 3.12 Compact tension test set-up.[73]

Fracture toughness and generic fracture energy were obtained from load versus displacement record where the peak load and failure energy were extrapolated from the curve.

3.3 Asphalt Mixture Testing

3.3.1 Preparation of Asphalt Mixture Slabs

Asphalt-aggregate mixtures used for the construction of the pavement trial sections on Highway 417 were collected and processed into asphalt slabs for the test purpose. Essentially, about 72 kg of asphalt-aggregate mixture collected from each trial section was weighed and heated in an oven at 180^{o}C for about 3 hours. Then, a mixer was used to mix the sample in order to enhance homogeneous mixture of approximately 6-7 % air void and this exercise is meant to prevent segregation of mixture sample as well. A plate compactor was eventually used to compact the mixture while the required percentage air void is still maintained. After the sample is left for 24 hours at ambient temperature, it was then cut into smaller pieces of required dimensions (l = 154 mm (± 1), b = 130 mm (± 1), w = 48 mm (± 1)) for testing purposes. Subsequently, the sample was air dried after cutting it into smaller pieces and the void content was measured by weighing the sample above and under water.

3.3.2 Single-Edge-Notched Tension (SENT) Testing

Asphalt concretes (slabs) were cut into smaller pieces of dimension as mentioned above while a notch of 40 mm (± 1) was introduced to one of the edges of the sample with a diamond saw blade while an a/W (notch depth over sample width) ratio is maintained between 0.45 and 0.55 as required in the ASTM E1290-93 standard. A photograph of a single SENT sample is shown below in Figure 3.13

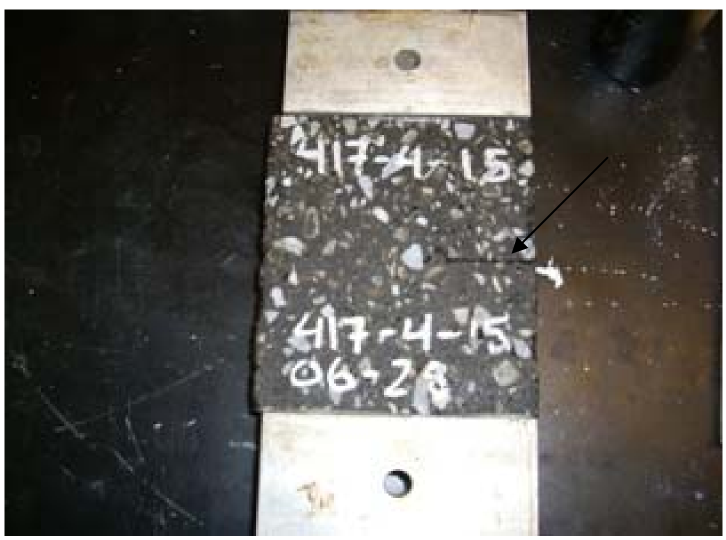

Figure 3.13 Single-Edge-Notched Tension (SENT) specimen.

Figure 3.14 Gradual crack propagation after the SENT test.

Samples were conditioned at -10°C for 1 hour, 24 hours and 72 hours in order to understand the fracture behaviour of the pavement under investigation. The sample was reconditioned in the environmental chamber at -22°C for 45 minutes before the start of each test. The study of the crack propagation was enhanced by the use of a crack mouth opening displacement (CMOD) clip-on gauge mounted at the crack tip of the sample. Hence, the CMOD was measured using an MTS clip-on gauge placed at the crack tip. Further, the load applied to the specimen is measured and a constant rate of 20 μm/min CMOD was applied through active feedback between the clip-on gauge and test frame software. A typical test conducted in this manner usually takes about five hours to complete while the corresponding fracture properties were measured at the same time.[71]

3.4 End Result Specification Data for Highway 417 Trial Sections

As part of the quality control/quality assurance protocol that exists for all Ontario paving contracts, a large number of asphalt binder and mixture properties were determined for all the Highway 417 pavement trial sections.

This series of field test sections were constructed in order to understand the rheological performance of asphalt concrete pavement and correlate its properties with laboratory investigations of the binders and mixtures, hence seven test sections of the asphalt pavements were built where real life traffic loading was performed so as to further understand the failure mechanism of the pavement. These test sections were made of different polymer modifications as shown in Table 3.1. Basically, a Superpave® mixture design was employed by the construction company for the construction of test sections - both surface and binder courses. The Superpave® design involves careful selection of aggregates and optimum asphalt binder content determination. Invariably, this method has been proven to produce quality hot-mix asphalt from which long-lasting pavements could be constructed.

Considering the compaction level of asphalt pavement in both surface and binder course, it is obvious that the level of compaction has a great effect on the performance of the pavement especially the percentage of air voids. Since air voids greatly influence the pavement performance and stability, hence, it is an important requirement for effective performance of the binders. According to Mohammad,[76] the presence of voids in asphalt mixture affects the porosity and water permeability consequently affecting its rheological behaviour and performance under traffic loading. Therefore, careful consideration of air voids in compacted bituminous mixture is examined as summarised in Table 3.2 below.

Table 3.2 Air voids and compaction level in Hwy 417 test sections

Test Sections		Surface Course		Binder Course	
		Air voids	Compaction	Air voids	Compaction
1	Sub-Lot 1	3.6	93.3	3.8	94.5
	Sub-Lot 2	3.9	94.3	4.5	94.4
	Sub-Lot 3	3.8	93.6	3.7	91.7
	Average	3.8	93.7	4.0	93.5
2	Sub-Lot 1	4.4	92.8	3.7	94.1
	Sub-Lot 2	4.8	93.9	3.9	94.0
	Sub-Lot 3	3.9	92.8	4.1	92.9
	Average	4.4	93.2	3.9	93.7
3	Sub-Lot 1	4.7	93.8	4.2	93.6
	Sub-Lot 2	3.3	96.2	3.9	92.4
	Sub-Lot 3	5.2	94.1	4.6	92.7
	Average	4.4	94.7	4.2	92.9
4	Sub-Lot 1	5.3	95.4	5.0	94.7
	Sub-Lot 2	3.3	94.5	5.3	93.1
	Sub-Lot 3	4.5	94.0	4.0	92.7
	Average	4.4	94.6	4.8	93.5
5	Sub-Lot 1	4.2	95.3	4.3	93.4
	Sub-Lot 2	4.9	93.5	4.5	94.0
	Sub-Lot 3	4.7	94.1	3.8	93.1
	Average	4.6	94.3	4.2	93.5
6	Sub-Lot 1	3.7	93.3	4.1	94.8
	Sub-Lot 2	4.3	92.3	4.8	93.9
	Sub-Lot 3	4.2	92.9	4.6	94.0
	Average	4.1	92.8	4.5	94.2

7	Sub-Lot 1	3.6	93.5	3.7	93.0
	Sub-Lot 2	3.9	92.5	3.8	93.8
	Sub-Lot 3	3.7	92.1	4.5	92.9
	Average	3.7	92.7	4.0	93.2

Chapter 4

Results and Discussion

4.1 Reversible Aging Test Results for Asphalt Binders

According to the newly developed Ontario laboratory standard test method, LS-308, the results obtained from reversible aging study for various types of straight and polymer-modified asphalt binders and their respective grade losses are shown in Figure 4.1. The error bars given are pooled errors based on the entire data set obtained in our laboratory. The performance ranking is based on grade loss which indicates the reversible aging tendency of binders upon which cracking could be predicted especially at low temperature.

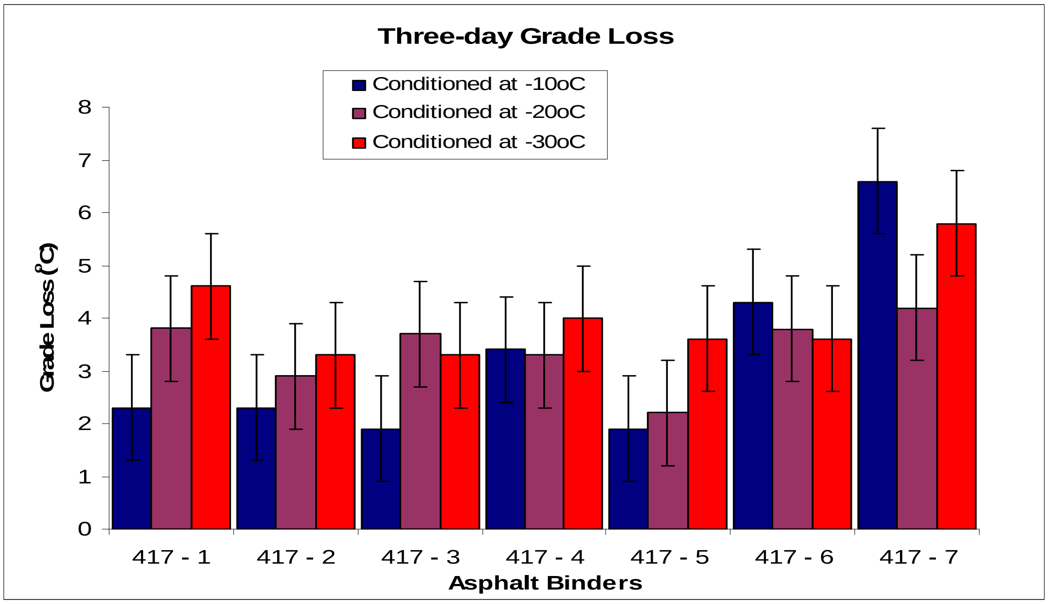

Figure 4.1 Three-day grade losses of asphalt binders from highway 417. (Note: Error bars provide ± 1 standard deviation of the mean)

From the extended BBR test on the binders, limiting temperatures for stiffness and m-values respectively (T_s, T_m) are determined at each conditioning time. The warmest limiting temperature values are used to determine the performance grade of the binders. Hence, these values can be used to rank the binders and, if the proposed approach in LS-308 is correct, eventually determine cracking severity of the asphalt pavement. In other words, the warmer limiting temperature values between T_s and T_m are used to determine the worst 3-day grade loss while the sample of asphalt binders are stored at 10°C or 20°C above their Superpave® low temperature grades.

The extended BBR results for 417-6 and 417-7 show that there are considerable grade losses (4.0 to 6.0^{o}C) for the binders used in these sections and that the largest losses are at the warmest conditioning temperatures where the road spends most of its life. Hence, it appears that the asphalt binders used for these sections are inferior and such large losses are likely to promote low temperature cracking (thermal cracking) under traffic loading. Also, Figure 4.1 shows that binders 1-5 are not very good when stored at the lowest conditioning temperature (-30^{o}C) and therefore may exhibit low resistance to low temperature cracking (thermal cracking) but only when it gets very cold. The rheological or morphological properties responsible for relatively low performance of binders could be traced to wax crystallization and free volume collapse in the binders' reversible aging process at low temperature.[40]

Table 4.1 summarises the limiting performance grading temperatures and the grade loss of the binders conditioned for 1, 24, 72 hours at -10^{o}C. Also, the conditioning temperature was changed to -20^{o}C and -30^{o}C above the predicted performance grade as shown in Tables 4.2 and 4.3. The trend of results suggested that generally the grade loss increases at colder temperatures; this indicates the temperature susceptibility to cracking at low temperatures. Not only that, performance grade temperature shows maximum allowable pavement design temperatures in order to avoid premature cracking or catastrophic cracking at low temperature. Therefore, a great deal of pavement cracking could be avoided if the right pavement design temperature is chosen along with appropriate binders (rightly graded performance) during the construction of asphalt pavement.

The data in Tables 4.1- 4.3 also show that the binder of section 417-3 grades the lowest at -41.6^{o}C when stored for three days at -30^{o}C. This should benefit this test section and hence it is expected to show better long term performance.

Table 4.1 Limiting temperatures and grade loss (°C) of binders at -10°C

Conditioning Time (Hrs)	417 - 1	417 - 2	417 - 3	417 - 4	417 - 5	417 - 6	417 - 7
1	-37.8	-39.4	-46.2	-39.7	-40.3	-37.5	-36.4
24	-36.0	-36.9	-44.5	-37.2	-38.7	-34.3	-30.7
72	-35.5	-37.1	-44.3	-36.3	-38.4	-33.2	-29.8
Grade Loss	2.3	2.3	1.9	3.4	1.9	4.3	6.6

Table 4.2 Limiting temperatures and grade loss (°C) of binders at -20°C

Conditioning Time (Hrs)	417 - 1	417 - 2	417 - 3	417 – 4	417 - 5	417 - 6	417 - 7
1	-38.1	-39.4	-47.9	-39.5	-39.7	-36.9	-35.8
24	-37.9	-36.6	-45.1	-37.8	-38.4	-32.9	-32.5
72	-34.3	-36.5	-44.2	-36.2	-37.5	-33.1	-31.6
Grade Loss	3.8	2.9	3.7	3.3	2.2	3.8	4.2

Table 4.3 Limiting temperatures and grade loss (^oC) of binders at -30^oC

Conditioning Time (Hrs)	417 - 1	417 - 2	417 - 3	417 – 4	417 - 5	417 - 6	417 - 7
1	-37.9	-38.6	-44.9	-39.0	-39.5	-36.4	-36.3
24	-34.6	-36.1	-42.7	-36.3	-37.3	-33.9	-31.5
72	-33.3	-35.3	-41.6	-35	-35.9	-32.8	-30.5
Grade Loss	4.6	3.3	3.3	4.0	3.6	3.6	5.8

Note: Limiting temperatures were the warmest of the limiting stiffness and m-value temperatures.

The estimate of the errors on these data is based generally on the previous experiments and experience on Bending Beam Rheometer (BBR) which is assumed to be ±1 as a pooled standard deviation from mean. Invariably, the errors on the performance of BBR for the low temperature prediction of asphalt binders are usually in the range of ±1 generally. Hence, this error is considered to be fair considering the grade losses obtained in this experiment.

In addition to the Highway 417 trial binders a number of materials from other contracts on Highway 417 and 62 were recovered and tested according to the newly developed grading method (eBBR). Analysis of the results in terms of grade losses due to isothermal conditioning (LS-308) is shown in Figure 4.2.

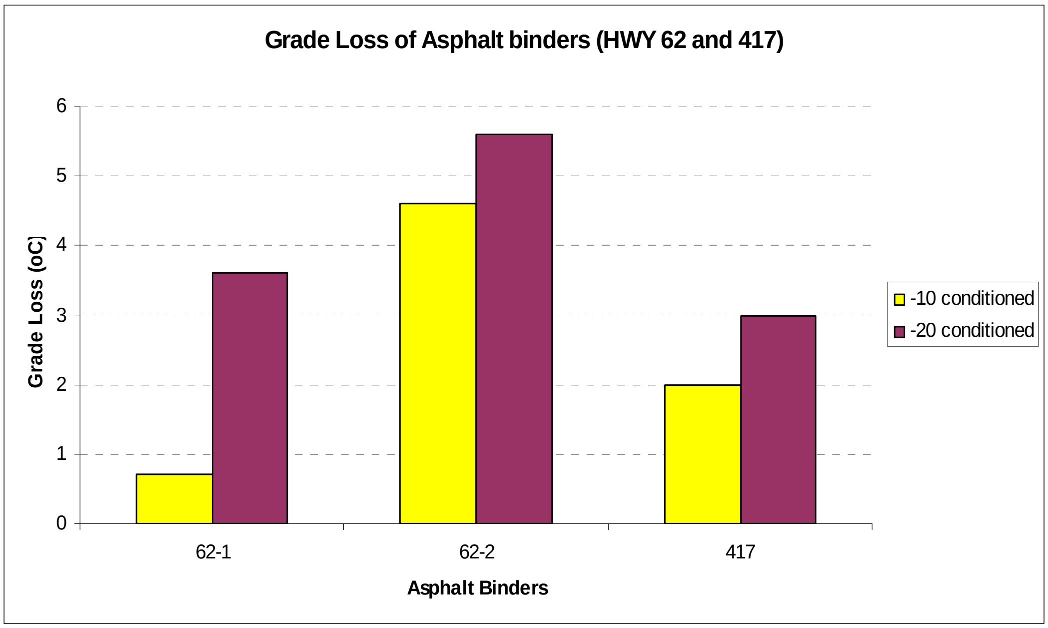

Figure 4.2 Typical grade loss in asphalt binders recovered from pavement contracts on highways 62 and 417.

The results suggest that the binder from highway 62, site 1 (62-1) near Bloomfield has superior performance considering the fact that its grade loss due to isothermal conditioning is negligible at -10°C. This is corroborated by the fact that this pavement contract has shown virtually no distress for 15 years since its construction in 1993. In contrast, it appears that there are significant grade losses in materials recovered from highway 62, site 2 near Bannockburn (62-2) and highway 417 pavement contracts both of which showed large scale cracking in early life. Therefore, the newly developed grading method is a promising tool which has a good parameter to distinguish relatively poor performing binders from the rest as indicated in Figures 4.1and 4.2.

4.2 Ductile Failure Properties of Binders

Fractures testing of the asphalt binders were carried out where the failure energy is determined for each section of the binders. Hence, the concept of essential and plastic works of fracture is used to determine failure energy at various temperatures from (4 to 40^{o}C) typical of the ductile state while the rate of loading is maintained at 50 mm/min. The concept of essential and plastic work of fracture for asphalt binders was recently developed in our laboratory which involves the use of a double-edge-notched tension test (DENT) to determine the fracture properties of binders and asphalt mixtures in order to understand performance ranking of the binders. It appears that this is a promising method which can significantly differentiate superior performance of binders based on strain tolerance of binders at each test temperature.

From the results obtained on force versus displacement plot for each binder tested using the DENT test, it was observed that there was a similar pattern of deformation at different ligament lengths (5 mm to 15 mm), this indicates that all the specimens had a similar sequence of stretching, yielding and tearing. Hence, this shows that the essential work of fracture extrapolated from the plot is valid since requirements for valid essential work of fracture entails complete ligament yielding prior to failure which was obtained in the experiment. Figure 4.3 shows the essential and plastic zones in a binder specimen undergoing a ductile failure test.

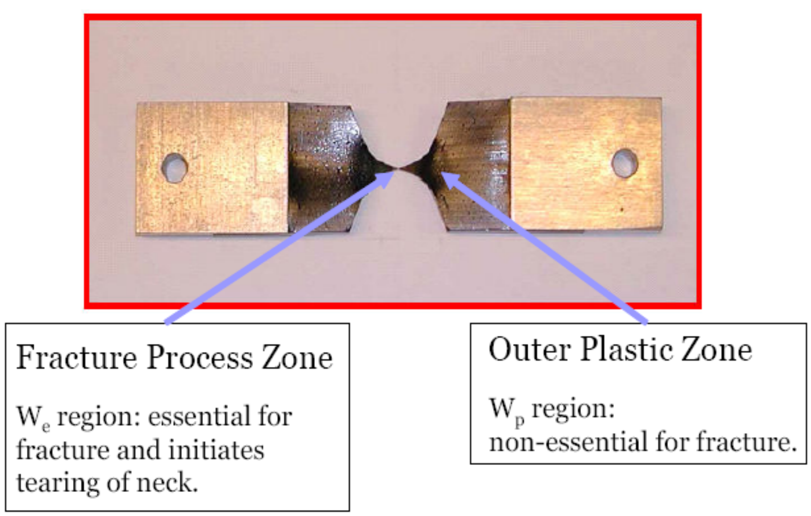

Figure 4.3 Diagram of fracture and plastic zone of asphalt binders

Furthermore, analysis of the results obtained from the DENT test shows that superior performances possess high strain tolerance as compared to poor performing binders having low strain tolerance as shown in Figure 4.4. Also, this test method is highly reproducible with duplicate measures generally providing excellent agreement .

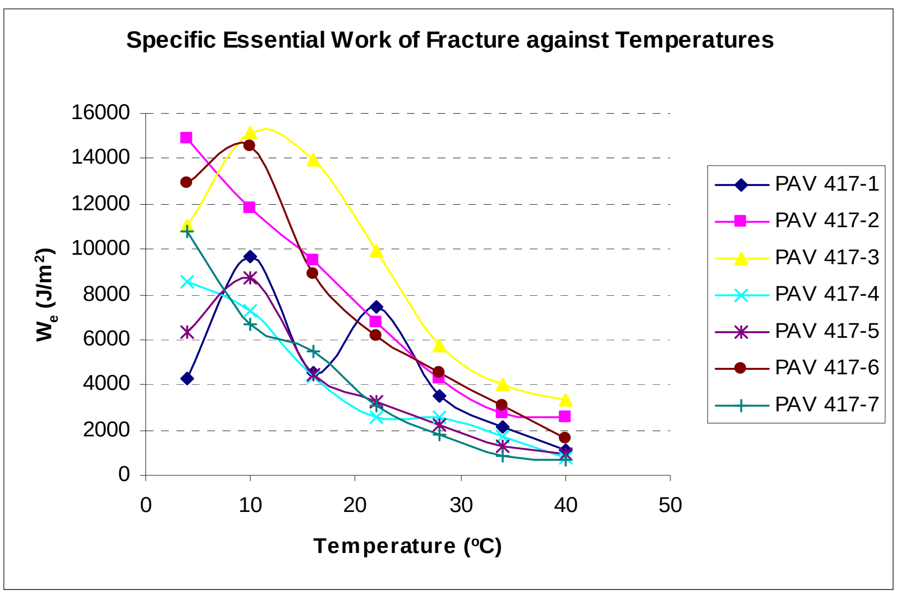

Figure 4.4 Specific essential work of fracture of highway 417 asphalt trial binders at different temperatures.

The concept of essential and plastic work of fracture for performance grading of asphalt binders has been proven to be a material property which is independent of specimen geometry. Figures 4.5 to 4.7 show clear differences between the performance of binders based on the essential and plastic works of fracture. In fact, Andriescu et al. establish the fact that essential work of fracture is closely linked with performance of the binders.[71]

From Figure 4.4, Sections 2, 3, 6 of Highway 417 exhibited high essential work of fracture which indicates superior performance of binders. The performances of sections 2, 3 and 6 are further complemented using plastic work of fracture at various temperatures as shown in Figure 4.5. Hence, inferior binders are much more likely to provide pavements that crack prematurely and/or excessively. Therefore, this concept of performance grading of binders could be used to differentiate superior performance from the relatively poor performing binders.

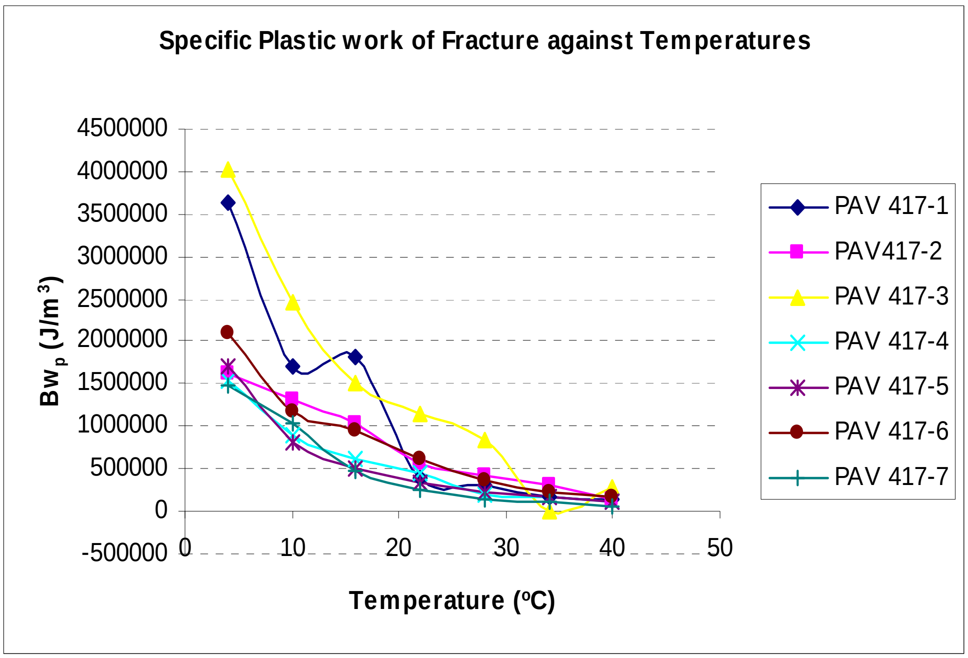

Figure 4.5 Specific plastic work of fracture of highway 417 trial asphalt binders.

Further, from Figures 4.4 and 4.5, there is a general trend of a decrease in both essential work and plastic work of fracture. This indicates that at high temperature, there may be a considerable decrease in performance of the binders in terms of failure energy. Then again, the peak stress in the DENT test decreases with temperature thus resulting in an increase in strain tolerance (CTOD). Therefore, optimum performing temperature could be used to understand the rheological properties as well as the performance grading.

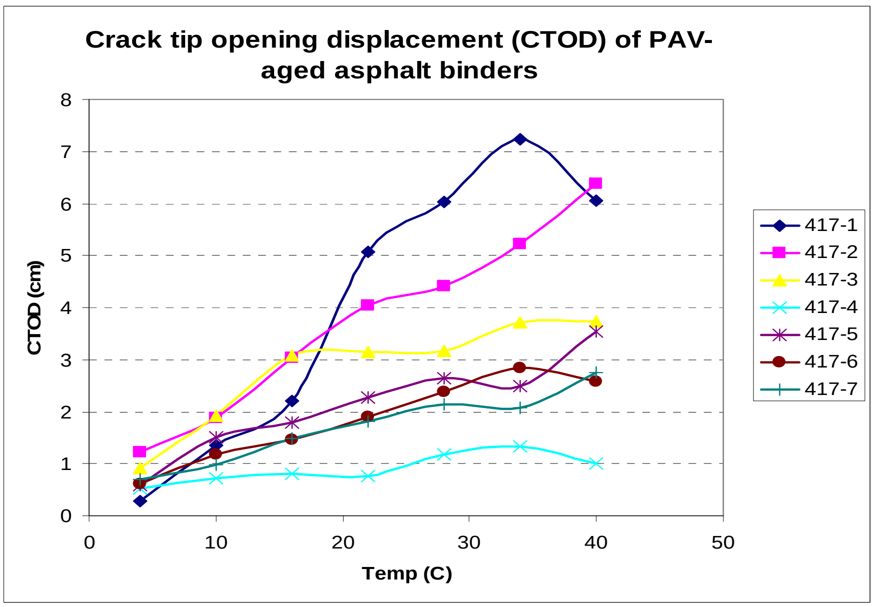

Figure 4.6 Crack tip opening displacement of PAV-aged asphalt binders of highway 417 at different temperatures

As shown in Figure 4.6, the crack tip opening displacement (CTOD) further explains the strain tolerance of PAV-aged asphalt binders at different temperatures. It can be deduced that the performance of various sections of tested samples is clearly shown with section 4 having worst performance considering its strain tolerance. Generally, the performance of the asphalt binders is based on the temperature at which the sample is tested. Hence, it appears that the optimum performing temperatures lie between 30 - 40^{o}C and the performance of sections 1 and 2 is ranked better than the other sections based on the parameter CTOD.

Also, essential work of fracture was used to determine the performance grading of the rolling thin film oven-aged (RTFO) samples in order to correlate its performance to that of PAV samples. Indeed, the specific essential and plastic work of fracture of RTFO samples appear to be significantly larger compared to corresponding PAV samples as shown in Figure 4.7 and 4.8 except at low temperatures where they are similar.

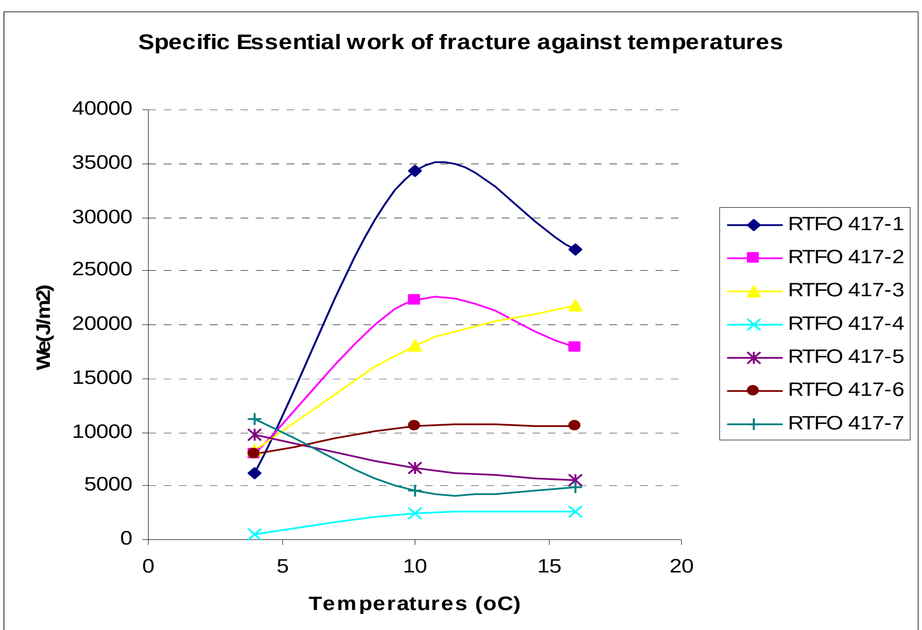

Figure 4.7 Specific essential work of fracture of asphalt binders (RTFO samples)

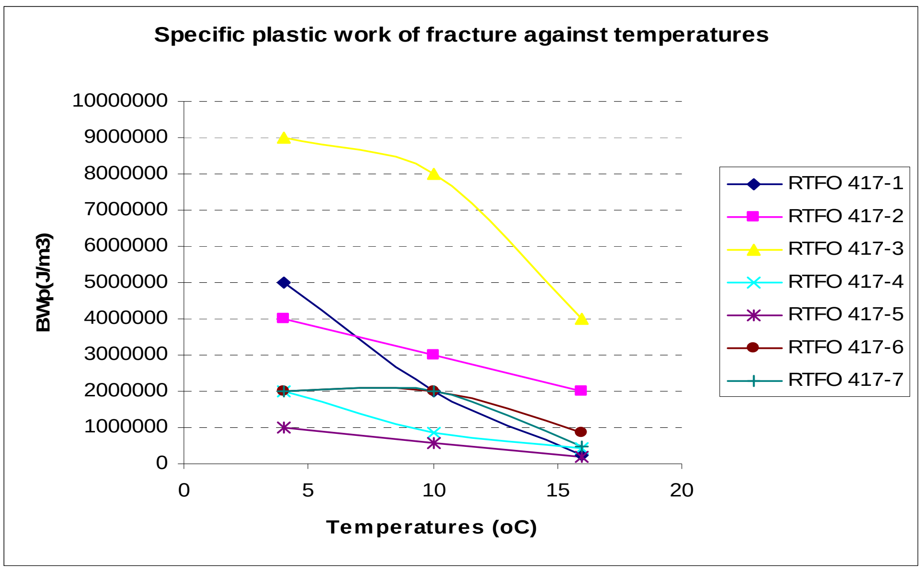

Figure 4.8 Specific plastic work of fracture of asphalt binders (RTFO samples).

Figure 4.9 shows the relative performance of the RTFO samples using a parameter called crack tip opening displacement to establish the ranking of the binders. It appears that sections 1 and 2 of the samples have fairly large

CTOD while the rest are relatively small. This indicates that sections 1 and 2 have high strain tolerance compared to others.

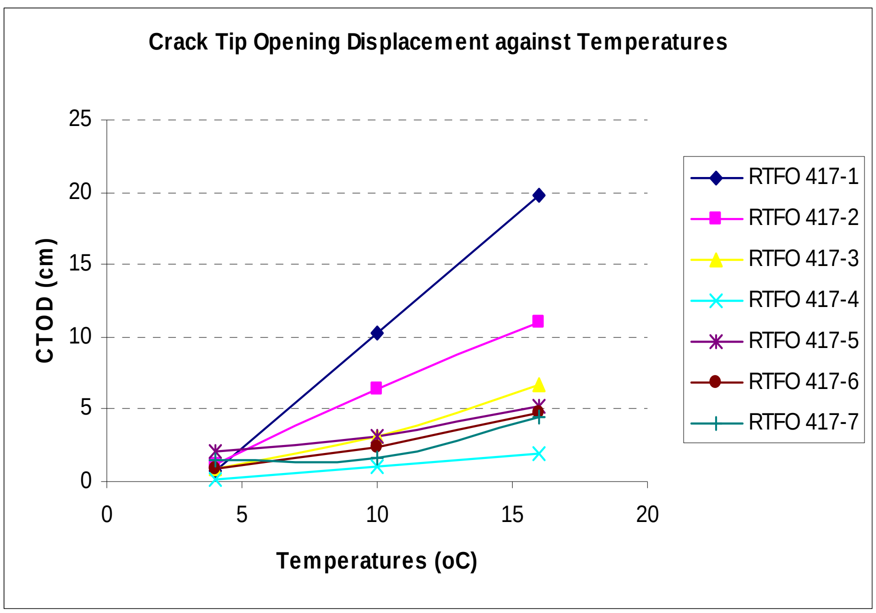

Figure 4.9 Crack tip opening displacement of asphalt binders (RTFO samples).

4.3 Brittle State Fracture Toughness of Binders by Compact Tension Method

Low-temperature thermal cracking of asphalt binders is ranked by their fracture properties. In this case, fracture mechanics approach which is adopted for asphalt binders is based on performance ranking at low temperatures. The fracture toughness, (K_{1C} - mode1, crack opening) and the fracture energy are used as a parameter to evaluate the performance of the binders at low temperatures. As shown in Figure 4.10, it is clear that fracture toughness, K_{1c} is successfully used as a reliable performance indicator particularly at -22^{o}C and -28^{o}C. From analysis of the results, Section 3 of Highway 417 consistently showed high fracture toughness and fracture energy which indicates superior performance as compared to other tested sections. Apart from the fact mentioned above, the parameters K_{1c}, and the fracture energy clearly distinguish the performance ranking of the binders depending on the type of polymer modifiers used. Since the reproducibility of force against displacement is fairly good, then it could be deduced that fracture performance of the binders could easily be identified.

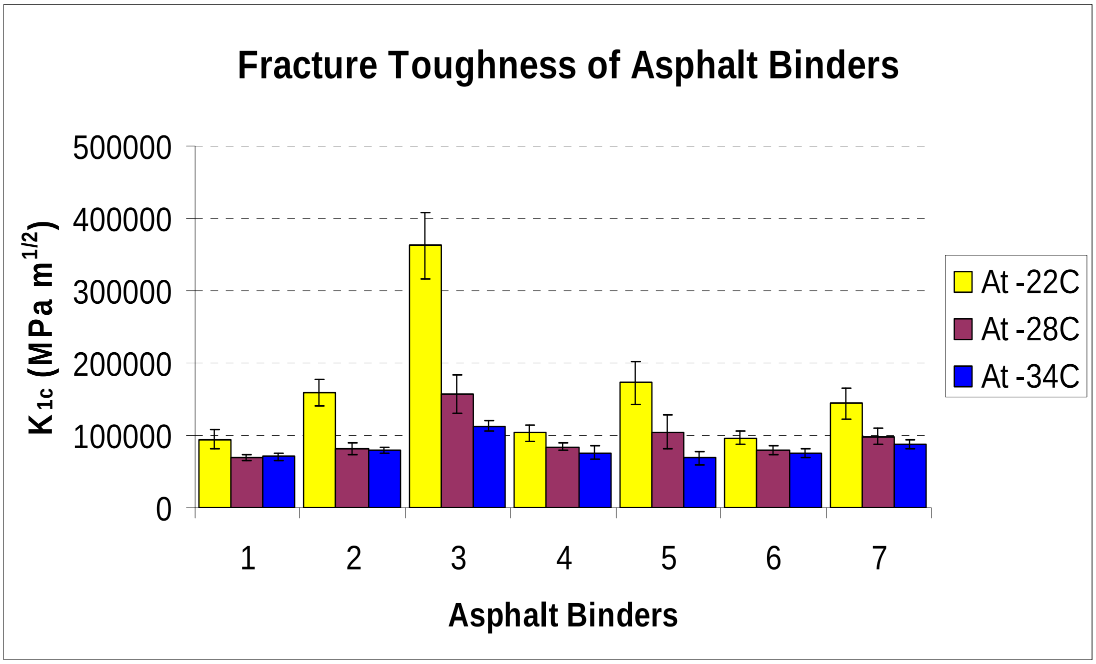

Figure 4.10 Fracture toughness of asphalt binders (PAV samples). (Note: error bars obtained from standard deviation of mean)

Tables 4.4 to 4.6 summarise the fracture properties in terms of fracture toughness and energy at low temperatures where the asphalt binders are tested. Also, the error bars shown in figure 4.10 are obtained from the standard deviations of the samples tested which explain the accuracy and consistency of the test method as shown in the tables. It appears the error bars are reasonably small except for trial section 3 with a fairly large error bar at -22°C and -28°C.The large error bar for section 3 may be caused by the nature of the source of the binders.

Table 4.4 Fracture properties of binders at -22^{o}C at the strain rate of 0.01mm/s

Binders	Generic Fracture Energy, G_{ic} (J/m^2)	Standard Deviation for Fracture Energy (±)	Fracture Toughness, K_{ic} (MPa.m^1/2)	Standard Deviation for Fracture Toughness (±)
417-1	30.63	11.26	94826	12773
417-2	92.48	26.00	159773	18789
417-3	1745.15	292.00	363014	46046
417-4	40.79	10.23	103251	10419
417-5	103.47	38.45	173035	29766
417-6	35.92	8.90	96785	9010
417-7	66.32	16.14	143954	20640

Table 4.5 Fracture properties of binders at -28^{o}C at strain rate of 0.01mm/s

Binders	Generic Fracture Energy, G_{ic} (J/m^2)	Standard Deviation for Fracture Energy (±)	Fracture Toughness, K_{ic} (MPa.m^1/2)	Standard Deviation for Fracture Toughness (±)
417-1	16.40	5.65	69150	4532
417-2	17.10	4.38	81009	8304
417-3	64.29	24.84	157230	26272.6
417-4	23.24	8.36	84161	5008
417-5	27.33	8.30	104456	23431
417-6	17.42	3.52	79871	6380
417-7	24.41	4.30	98896	10755

Table 4.6 Fracture properties of binders at -34^{o}C at strain rate of 0.01mm/s

Binders	Generic Fracture Energy, G_{ic} (J/m^2)	Standard Deviation for Fracture Energy (±)	Fracture Toughness, K_{ic} (MPa.m^1/2)	Standard Deviation for Fracture Toughness (±)
417-1	18.91	4.64	70661	5299.83
417-2	24.10	5.23	79283	4559.07
417-3	36.33	4.25	113050	6794.62
417-4	16.75	4.82	75910	9131.69
417-5	17.36	6.28	68438	8896.84
417-6	18.78	3.91	75227	5810.67
417-7	22.93	3.65	87636	6136.09

In this study, fracture toughness indicates the level of fracture resistance to cracking at low temperature. This concept can also be used to evaluate the driving force on a crack. In order words, "the assumption is that crack growth requires creation of surface energy, which is supplied by the loss of strain energy accompanying the relaxation of local stresses as the crack advances."[77] Therefore, failure occurs when the loss of strain energy is sufficient to provide the increase in surface energy. Figures 4.11 and 4.12 show the generic fracture energies of the binders tested at -28^{o}C and -34^{o}C.

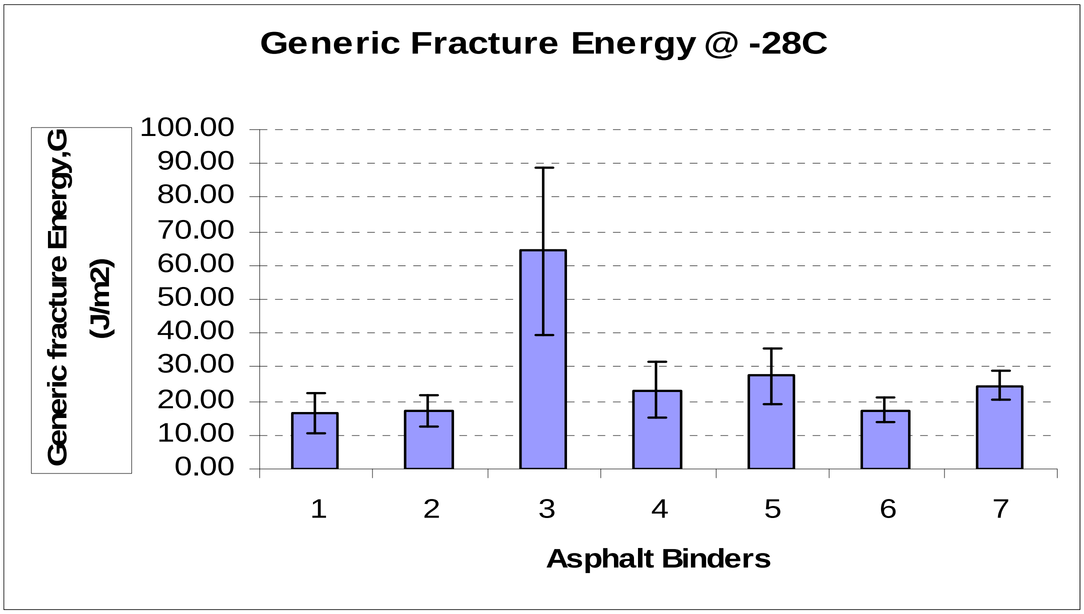

Figure 4.11 Fracture energy of asphalt binders at -28°C (PAV samples).

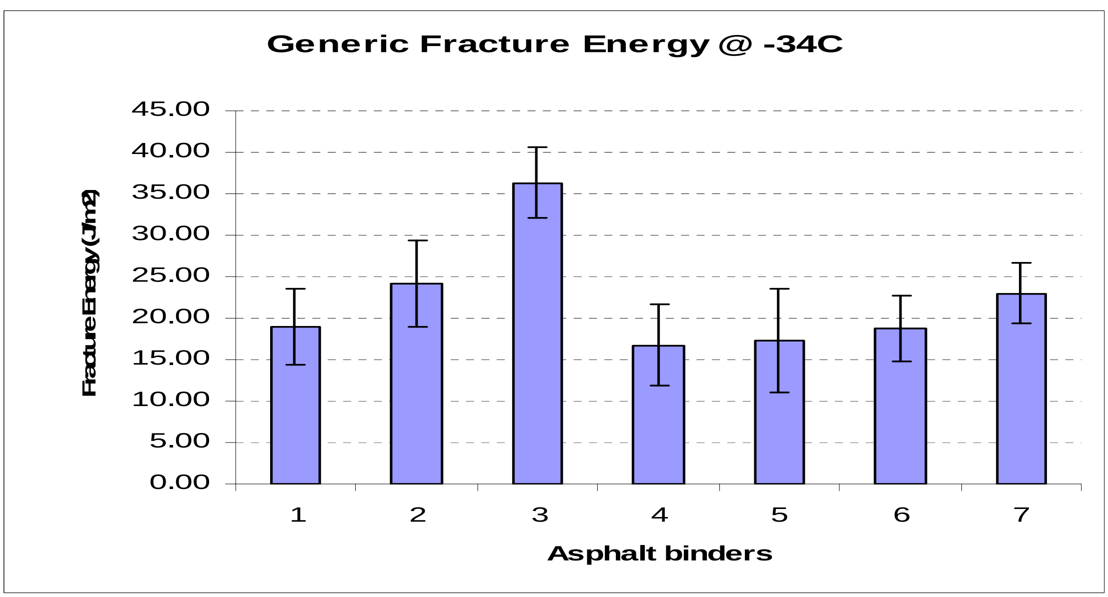

Figure 4.12 Fracture energy of asphalt binders at -34°C (PAV samples).

4.4 Mixture Testing

4.4.1 Single-Edge-Notched Tension Test

Considering the analysis of the results obtained on single-edge-notched tension (SENT) test performed on mixture samples, it appears that the crack opening displacements (CTOD) at the peak load of the samples are comparatively low for the 72 hours test as shown in Figure 4.13, this indicates that strain tolerance of the mixture samples is small and consequently leading to low resistance to cracking upon loading. However, the crosshead displacements at the peak load of the same mixture samples are fairly high for 72 hour test which contradicts the results obtained using CTOD parameter since the strain tolerance is small. The high value of crosshead displacement at peak load could likely have been caused by microcracks in the mixture samples that would have allowed the crosshead to move but the critical crack mouth opening displacement to deteriorate with storage time at low temperatures. This observation deserves further investigation with samples stored for longer periods and with other methods such as digital image correlation which measures the entire strain field across the sample. Such studies are already ongoing in our lab.

From Figure 4.14, for one-hour test of mixture samples, there appears to be no appreciable difference in terms of their performances based on cross head displacement results at peak load for each sample tested. Also, the error bars are fairly large which indicate the fact that the reproducibility of the mixture sample is not very good to conclude the field performance correlations of the mixture samples. Based on Figure 4.13 and 4.14, it appears that there is no significant correlation between the binders' performance as shown by compact tension results (Figure 4.10) when compared to the one obtained in this experiment. Therefore, the field performance correlation with mixture test is not yet conclusive because of the variant results between the binders and the mixture sample based on these tested specimens. However, this could possibly be explained by the different aging protocols used between PAV binder aging and oven aging for loose mixture prior to compaction of laboratory slabs. Yet another explanation could be that the mechanisms is just different between binder failure in the compact tension test and mixture failure where there is a mastic-aggregate interface through which cracks may prefer to start and propagate.

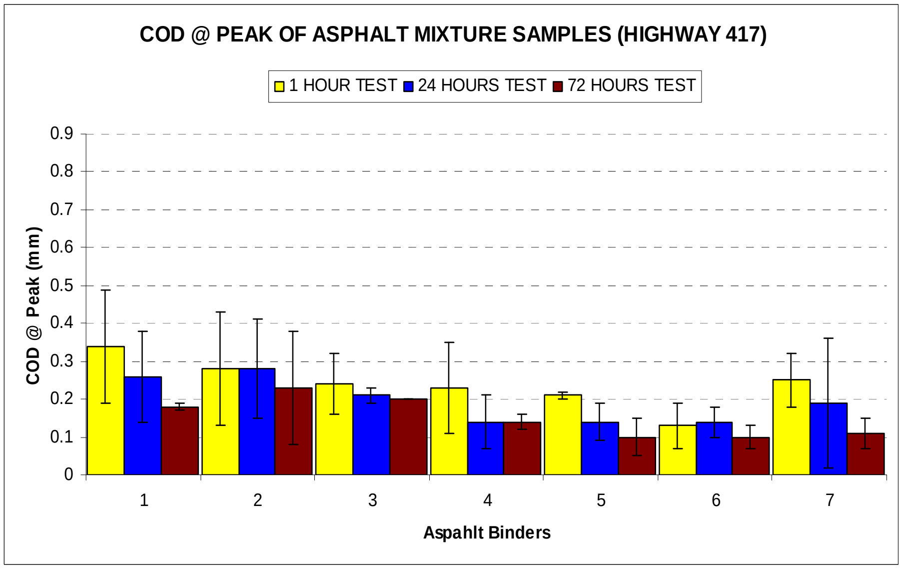

Figure 4.13 Crack opening displacements of mixture samples tested at -22°C.

The error bars in figures 4.13 and 4.14 are obtained from the values of range divided by the square root of the number of samples tested. Although the error bars appear to be fairly large, the clear performance of the tested sections is obvious. These large error bars may have been caused by the microcracks or flaws (air voids) present in the samples and hence indicate the fact that the reproducibility of the tested samples is fairly poor. There is a greater possibility of the error bars to be reduced to a reasonable level if the number of samples tested is greatly increased and this would enhance the accuracy of the results obtained.

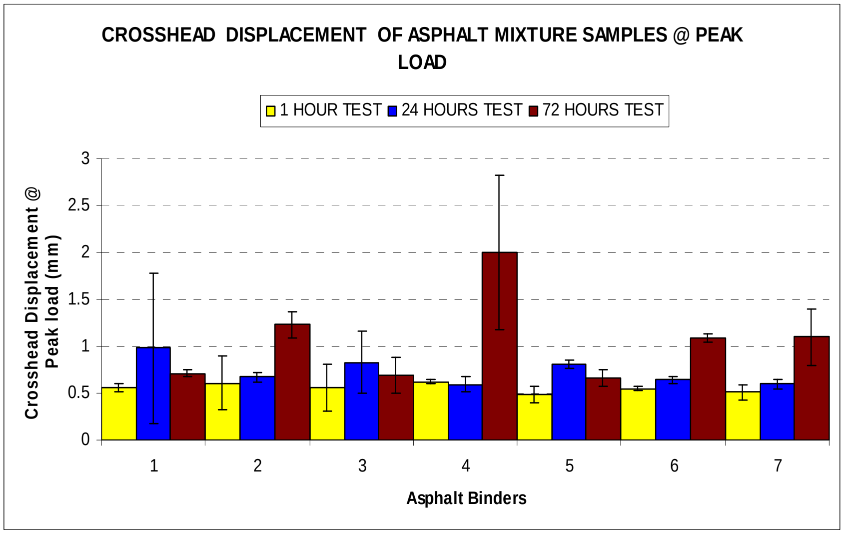

Figure 4.14 Cross head displacement of mixture samples tested at -22°C.

4.4.2 Double-Edge-Notched Tension (DENT) Test

The concept of essential work of fracture was suggested as a failure criterion for a mix sample of asphalt in order to understand the failure mechanism and possibly rank the performance of mix samples compared to corresponding binders. Using the concept of failure energy, the results obtained showed that the general requirements for validity of EWF tests are met for the mixture tests in which the shape of the load-displacement graph was found to be self similar for different ligament tested. Also, it was noticed that microcracks resulted after the peak load was reached which is an indication of surface separation after the ligament has fully yielded.

Considering the results obtained as shown in Figure 4.15, this indicates that in the regressions of different sections tested, the performance of sections tested can be summarised as follows: the performance of Sections 3-5 is the worst followed by Section 6 and then 7, it appears that the performance of 1 and 2 are better when compared to other tested sections since there is considerable (high) specific energy obtained. Hence, this shows that 3.5% SBS from Husky appears to perform better than the 7% SBS from Husky based on the compositions of the binders. Apparently, polymer modifiers have significant impact on the performance of the mix samples. It should be noted though that the variability of these ductile tests is rather high and that some changes in properties may have occurred during the heating of the samples prior to slab compaction.

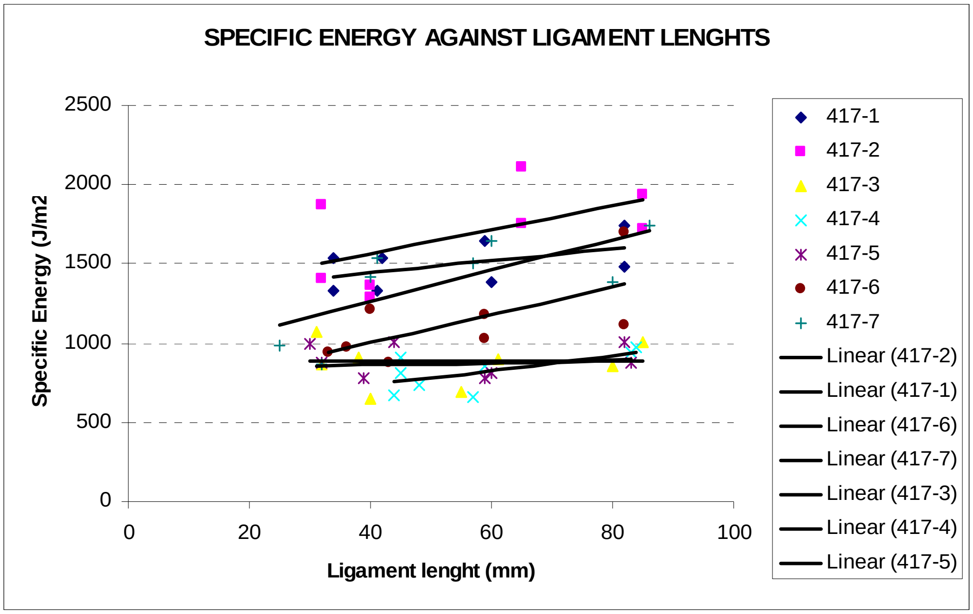

Figure 4.15 Specific energy of the mixture samples tested at ambient temperatures (25°C).

Also, the 7% SBS from Kraton (section 3) has consistently performed best in all previous tests and now performs quite a bit worse in the mixture sample. This poor performance is at present unexplained but further tests with samples from the field or with a greater number of samples made in the laboratory may shed light on this issue. It is expected that an increase in the number of samples tested would lead to a higher accuracy in the EWF evaluation. Finally, field samples that have not been exposed to a rather severe temperature treatment during the re-heating of the samples for slab compaction may provide a more accurate reflection of performance.

In addition, Figure 4.16 shows the crack tip opening displacement (CTOD) of the asphalt mixture which clearly distinguishes the performance of the various sections tested while section 1 and 2 appear to show superior performance since the strain tolerance is significantly higher than the rest.

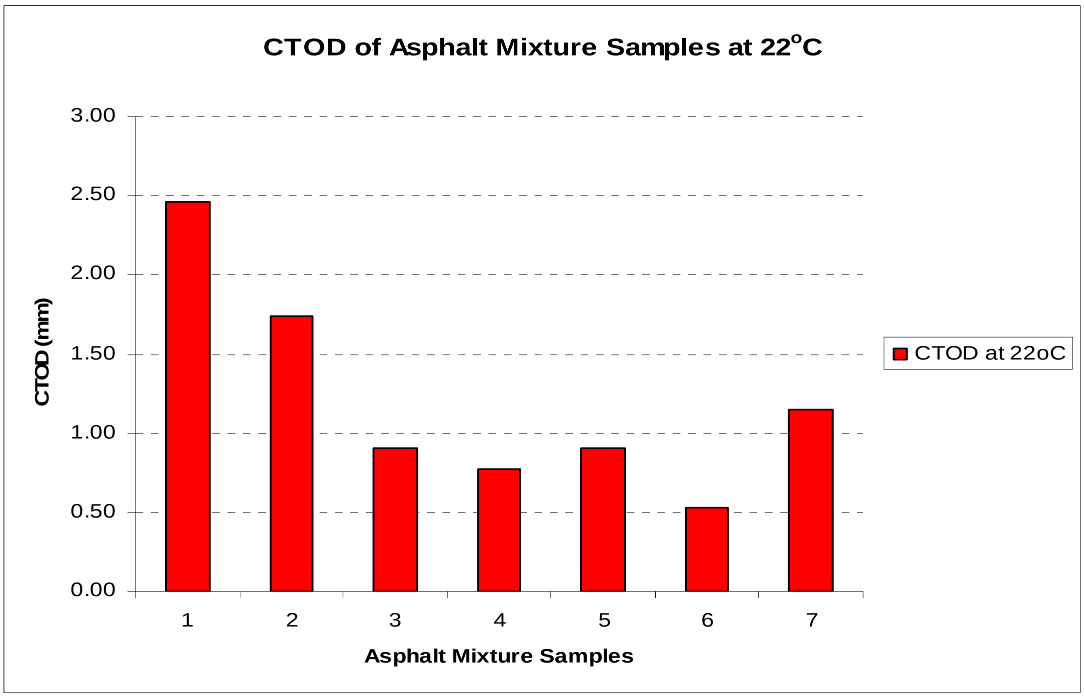

Figure 4.16 Crack tip opening displacement (CTOD) of asphalt binders at 22^0C

Invariably, these results seem to be consistent with the corresponding binders as shown in figure 4.5. Therefore, superior performance of asphalt binders and corresponding mixture samples can be ranked through the concept of Crack tip opening displacement (CTOD) where the strain tolerance is used to determine the fracture resistance of samples tested. Error bars are not indicated in figure 4.16 since it is a measure of strain tolerance when a constant load is placed on the sample tested.

Chapter 5

Summary and Conclusions

The following broad summary and conclusions may be reached from the literature review and discussions presented in the previous chapter based on the findings on the experiments conducted in the laboratory.

- The results obtained in this study indicate that fracture properties of asphalt binders are different from those of asphalt mixtures.
- Concept of CTOD and fracture properties can be used as low temperature failure criteria for asphalt binders and its corresponding pavement at different temperature.
- Premature cracking in asphalt pavement can be attributed to the effect of reversible aging (wax crystallization, asphaltene precipitation, free volume collapse) and low fracture resistance of the binders. A significant number of studies have shown that high fracture energy values, specific essential and plastic work of fracture can be related to superior low temperature performance and fatigue resistance.

- Fracture performances of asphalt binders of the same Superpave® grade can be differentiated through the binders test methods which are highly reproducible. Hence, the differences observed in the fracture energies in both brittle and ductile states indicate the fact that different modification methods lead to different performance at various conditions.
- Critical stress intensity factor (K_{1c}), which is the stress intensity factor for an effective crack length, explains how unstable crack growth starts at peak load.
- CTOD is invariably the crack opening at which unstable crack growth starts and hence should also provide a measure of performance. Therefore, if these parameters are included in the binder's specification, it is very possible that premature cracking would be avoided particularly at low temperature.

Although the proposed test method for grading performance of asphalt is sound, the major setback is the inability of the test method to effectively separate the energy (essential work) involved in the process zone to produce a new surface and that of energy involve in the plastic deformation away from the crack zone. Defects which act as crack initiator are always bound to be present in asphalt mixture consequently leading to premature cracking. Therefore, a mixture sample having higher a value of CMOD appears to have a gradual mode of failure (slow stable crack growth). However, an asphalt mixture is more likely to fail catastrophically when the value of CMOD is very small (fast unstable crack growth).

Future research intends to address the performance of asphalt binders with similar physical (rheological) properties but dissimilar chemistries under various conditions particularly with respect to the interaction of asphalt binder and aggregate on notched mixture samples. Moreover, evaluation and understanding the level of interaction between asphalt and mineral aggregates in the mixture (pavement) with a goal of enhancing the pavement performance will be the ultimate focus of the research particularly with respect to fatigue cracking and low temperature performance.

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