C-LTPP GRADUATE RESEARCH REPORT

AN EXAMINATION OF HOW ASPHALT CEMENT QUALITY INFLUENCES LOW TEMPERATURE CRACKING AND PERFORMANCE

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ABSTRACT

The performance of an asphalt pavement is strongly affected by the response of the asphalt cement in the mix formulation to the in-service condition. In Canada, one of the major obstacles to achieving long term pavement performance is low temperature cracking. The Strategic Highway Research Program (SHRP) Superpave methodology incorporates design requirement based on the low and high in-service temperatures. This research examines how existing practice can be incorporated into the requirements of the SHRP Superpave Performance Graded (PG) asphalts. C-LTPP material, cracking and construction data was used in this research project. This report summarizes only the results where C-LTPP or the C-SHRP test data was used. An integrated model which relates material properties to pavement performance and life-cycle cost was developed and full details are available in the author's doctoral thesis [Tighe 99].

The first part of the report focuses on the material characterization of the asphalt cement. This analysis examines the McLeod Penetration Viscosity Number (PVN) as a low temperature susceptibility indicator. The results show that PVN is a fingerprint as it remains constant over time. PVN also relates to the SHRP Superpave Performance Graded (PG) asphalts. A decrease in the minimum temperature corresponding to the PG asphalt is consistent with an increase in the PVN. As well, the PVN shows variation within a crude source as would be expected and it shows PVN is constant regardless of the calculation method.

The material properties where then used to predict low temperature cracking. The Canadian Airport Model and Hajek model is used to predict cracking on the C-LTPP and C-SHRP test sites. The predicted cracking is compared to the observed cracking. The analysis indicates that the Hajek model and Canadian Airport model show good correlation to the observed cracking. The thermal contraction coefficient is examined and found to be a very good indicator of low temperature cracking based on observed cracking. The pavement performance was calculated using the cracking prediction. Using roughness as the measure of pavement performance, the performance of the pavement is predicted. Low temperature cracking is related to roughness in terms of Riding Comfort Index (RCI). Various relationships which relate RCI to the International Roughness Index (IRI) is examined. The predicted IRI values is compared to the observed values using the C-LTPP test sites. The Canadian Airport Model is considered to be used as a starting point with recognition that it is conservative. Construction data from the C-LTPP sites was examined. A lognormal distribution is determined to be the most appropriate distribution for pavement lift thickness

A framework for predicting the technical and performance of a pavement is presented. Overall the thesis illustrates how these variables can be used to provide pavement designers with a methodology for predicting low temperature cracking. The model is intended to compliment the SHRP Superpave methodology and ultimately result in the selection of the most appropriate design based on it's technical and economic merits.

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CHAPTER ONE: INTRODUCTION

1.1 BACKGROUND

Transport Canada indicates that approximately 12 billion dollars is spent on roads annually in Canada [Richardson 96]. Based on geography, Canada's road system plays a particularly important role with regard to both economic and social development. Overall, the total length of Canada's roads and streets is about 840,000 km of which 37% are paved or surface treated [TAC 92]. However, the paved portion is the most intensely used with about 55% of the vehicle–km of travel occurring in urban areas even though these areas only account for about 15% of the total km [RTAC 88]. Based on the high usage of these paved roads, pavement engineers attempt to design, construct and maintain pavements that are safe, long lasting and cost effective.

In Canada, one of the major obstacles to achieving long-term performance is low temperature cracking of asphalt pavements. Low temperature cracking is a multimillion dollar pavement distress problem in Canada [AI 98]. Cracking due to climatic effects occurs when: ambient temperatures reach significant low values, when shrinkage cracks occur due to volatilization and oxidation, and when thermal cycling effects combine with increasing stiffness with age [Seddick 95]. Canadian studies have confirmed that thermal cracking is a single event phenomenon [Anderson 99]. Research has shown that excessive thermal stresses are prevalent when temperatures decrease. This low temperature cracking results from the initiation of a microcrack. The studies show the internal damage to the pavement may be due to the formation of the micro cracks within the asphalt mix (aggregate and asphalt) when it is cooled. At a certain point the mix loses it's ability to "flow" within the mix matrix. The thermal contraction coefficient of the binder is approximately 15 to 20 times greater than aggregate [AI 98]. When the induced stresses exceed the tensile strength, the crack propagates to the full depth of the layer upon one or more thermal cycles. The rate and progression of this cracking is largely dependent on the in-service temperature and the stiffness of the asphalt binder.

Low temperature cracks also form when an existing cracked pavement is resurfaced. The underlying crack is thought by some to propagate up through the new layer under one or more thermal cycles (commonly referred to as "reflection cracking"). Once a crack forms, the pavement structure is vulnerable to the infiltration of water and other particles. It is imperative that once a crack has initiated, it needs to be sealed. Otherwise, the newly formed crack will result in the premature deterioration of the pavement structure.

In addition to the aforementioned climatic effects, asphalt properties (i.e. penetration, viscosity, and stiffness as a function of temperature) have a huge impact on low temperature cracking [Haas 87]. The St. Anne's field study confirmed that pavements cracked at low temperature extremes and high asphalt stiffness [Anderson 99]. Additional field data also indicated that when an asphalt cement stiffens in excess of the cracking stiffness, the severity of cracking is affected by the mix properties and mix design, pavement design and pavement age [Haas 87].

In an attempt to quantify the environmental effect on pavements, the United States (U.S.) Strategic Highway Research Program (SHRP) has developed Superpave (Superior Performing Pavements) binder specifications for Performance Graded (PG) binders. This program proposes that structural design should be based on low and high in-service temperatures rather than on traditional empirical relationships. While the U.S. SHRP program was developing Superpave, the Canadian General Standards Board (CGSB) introduced performance based asphalt binder specifications [AI 98]. CGSB introduced Group A, B, and C quality grade asphalt specifications that closely relate pavement performance to asphalt cement performance [NSB 90]. The Canadian Strategic Highway Research Program (C-SHRP) initiated a full scale test road to enhance understanding of the low temperature asphalt binder characteristics and correlate CGSB asphalt specifications to field performance across Canada [Anderson 99]. These results support the use of CGSB asphalt specifications for selecting asphalts best suited for cold climates.

Based on the in-service temperatures and the pavement factors (i.e. traffic level, functional classification, etc.) the asphalt cement grade is selected. According to the U.S. SHRP Superpave design methodology, a large number of pavements will require an asphalt modification or enhancement to meet the design specification. Suppliers and manufacturers of enhanced asphalt have their own proprietary technology for formulations and processes. Sometimes a process is used to change the asphalt properties, while in other cases, modifiers are combined with the base asphalt through high shear rate mixing and dispersing agents to maintain stability in the storage tanks. [Tighe 97].

To select the most appropriate pavement design for a given situation, pavement specialists need to understand how the asphalt material properties relate to cracking and life cycle cost. A given design may be most appropriate on facility while it may be least appropriate on another. Different asphalt cements dramatically affect the long-term performance. With the advent of new design methodologies, there is a tremendous need to develop performance and economic models which examine low temperature cracking and examine how an asphalt cement influences pavement performance and life cycle economic cost.

1.2 SUMMARY OF RESEARCH APPROACH

The research described herein predicts the performance of various asphalt pavements. Within this research methodology, various laboratory test results have been examined and assessed in terms of how they relate to field performance. Low temperature susceptibility indicators are analyzed extensively in terms of relevance and uncertainty with regard to field performance. Once the low temperature performance is determined, the roughness progression is estimated. Information is combined to produce a mathematical model for use in Canada. The research extracts material characterization data (laboratory and field), cracking data, pavement factors such as subgrade and traffic levels, performance data such as roughness and cracking. Some of the data has been used for model development while other data was used for verification. Probabilistic simulation is used to determine uncertainty associated with performance of various asphalt pavements.

1.3 RESEARCH PROBLEM: LOW TEMPERATURE CRACKING

Fracture markedly accelerates the loss of serviceability in a pavement, thus resulting in bumps, dips and potholes. In 1965, behavior of flexible pavements and their components at low temperatures was judged by the Canadian Good Roads Association to be the highest priority road research need in Canada. It is still today one of the first priority road research needs, as evidenced by the Canadian Strategic Highway Research Program (C-SHRP). To properly manage a pavement network in Canada it is important that low temperature cracking can be predicted. Laboratory testing must indicate low temperature susceptibility so that proper materials and designs are selected for long term performance. The laboratory tests should accurately predict in-service cracking and in a timely manner (i.e. testing should be relatively accessible and not overly time consuming).

1.4 ASPHALT CEMENT CHARACTERIZATION

Asphalt cement used for roadway construction is tested at the refinery, pre-construction, at the asphalt plant and post-construction. The material characterization tests generally fall into the following basic categories: consistency testing, static testing and cyclic (dynamic) testing. These tests are outlined in the following subsections as some of the results are variables in the integrated model subsequently described.

1.4.1 Consistency Testing

Consistency tests are most widely used as they measure the degree of fluidity at any particular temperature. Thus, it is important to measure consistency at the same temperature and shear loading for a number of conditions. The most important consistency tests are penetration, viscosity, softening point and ductility [Soleymani 97].

The penetration test is an empirical test that measures the “stiffness” at a loading time of approximately 0.4 seconds [Van Der Poel 54]. The test is usually performed at 25°C to approximate the average in service pavement temperature. A needle with 100g weight is allowed to penetrate the asphalt binder for 5 seconds. The depth of the penetration is measured in units of 0.1mm and is reported as a penetration number. While the test is considered to be empirical in that only depth of penetration is reported, it is actually also a shear test. However, little if any analytical work has been carried out on the actual (time dependent) shear response in penetration testing.

Viscosity is the ratio of shear stress to shear rate at any given temperature. Asphalt viscosity is a fundamental measure of the asphalt flowability that is not affected by changes in testing conditions (i.e., configuration of test instruments, geometry of sample). Viscosity is typically performed at 60°C, which is denoted as the absolute viscosity, and 135°C, which is denoted as the kinematic viscosity. The absolute viscosity is defined as the resistance to flow of a fluid at 60°C and it approximates the maximum in-service summer pavement temperature. To determine absolute viscosity, a viscometer is mounted in a thermostatically controlled temperature and is charged with asphalt cement. A partial vacuum is applied and the flow is timed in seconds and then multiplied by a calibration factor (unit in poise) [Roberts 96]. The kinematic viscosity describes high temperature behavior and essentially characterizes the flow behavior under gravity. The asphalt cement flows through a capillary tube under gravitational force and approximates the mixing and lay down temperatures during pavement construction. The kinematic viscosity is measured in centistokes because the gravitational forces induce flow and density of the material affects the rate of flow [Roberts 96].

The Ring and Ball (R+B) test measures the softening point. The purpose of this test is to determine the temperature at which a phase change occurs in the asphalt cement. Ductility tests are also commonly performed at specific loading rates and temperatures in order to determine the relationship between shear susceptibility at the various in service temperatures. Some researchers believe there is correlation between ductility and low temperature cracking while others have debated that ductility tests have little value based on their empirical nature and their poor reproducibility.

1.4.2 Static Load Test

Static tests have been used for the characterization of time dependency. The three most commonly used loading modes are: creep, which applies a constant load and measures deformation with time; relaxation, where constant deformation is applied while the load change is measured with time and constant deformation where the material is exposed to a constant rate of deformation while the load required to keep that rate constant is measured.

The moduli from the loading modes are calculated to characterize the material behavior as a function of loading time [Soleymani 97]. The creep mode is the simplest and most convenient. Schweyer [Schweyer 74] introduced various types of viscometers to examine creep behavior. These include: rotational type (utilizes coaxial cylinders or cone and plate) and a specialized capillary type in which a piston is used to drive the asphalt cement through a capillary tube [Soleymani 97]. The bending beam rheometer developed by SHRP is a creep loading test [Bahia 92].

A constant stress creep test result exhibits three stages [Tia 87]. In the first stage, the initial elastic response can be used to calculate the elastic modulus E_0. The second stage is denoted as the creep portion which becomes linear, resulting in a constant slope (ε_{cr}) or velocity. This slope is the strain or shear rate (ε or γ) corresponding to a given stress and test temperature. Once the applied stress is released, an immediate recovery of elastic strain occurs followed by the gradual, time dependent recovery of elastic strain ($\varepsilon_{recovered}$, delayed elastic strain). The residual strain that exists after complete elastic recovery is the nonrecoverable strain or permanent deformation (creep strain) [Tia 87, Ruth 96]. To analyze the creep test data, the stress and strain development at a specified temperature are examined. Note that at the lowest temperature T1, the viscosity is very high resulting in low creep rates (ε_{cr}) and high stress (σ). Overall, the creep test can be used to effectively calculate the stresses and strains developed during cooling of a bituminous pavement [Ruth 96].

1.4.3 Cyclic (Dynamic) Testing

In cyclic (dynamic) testing, a sinusoidal stress is applied to a specimen and the resulting strain is monitored as a function of frequency. Strain controlled testing (sinusoidal varying stress is applied and strain is measured) is more common than stress controlled testing. The primary responses in dynamic testing are the complex modulus (G*) and the phase angle (δ). G* is a measure of the asphalt cement resistance to deformation while δ is the time delay or phase angle between the applied stress/strain, and the response stress/strain. [SHRP 95].

1.5 ASPHALT TEMPERATURE SUSCEPTIBILITY

Low temperature cracking has been shown to occur when thermal stresses in the bituminous layer exceed fracture strength [Haas 87]. Excessive thermal stresses are most prevalent under decreasing temperatures when a microcrack initiates. In terms of practical implications this means that one of two things happen:

1. Either a microcrack initiates at the surface when the temperature induced stresses exceed the tensile strength (due to decreasing temperature) and the crack propagates only to some limited depth because of varying stiffness gradient.
2. Or the crack propagates to full depth of the surface upon one or more cycles of sudden warming due to the creation of stress imbalances. Continued warming (in spring) leads to firstly open, visually apparent cracks which then close with further warming (late spring) [Haas 87].

The most critical factors influencing low temperature cracking have been identified as follows [Haas 87]:

1) Climatic Effects (ambient temperature and rate of temperature decrease)
2) Asphalt Properties (penetration, viscosity, stiffness as a function of temperature)
3) Mix Properties and Mix Design (stiffness as a function of temperature)
4) Pavement Design (subgrade type, asphalt thickness)
5) Pavement Age (increase in stiffness with age)
6) Other Factors such as ductility of binder, voids and type and amount of mineral filler.

Based on some consistency tests as outlined earlier, researchers have tried to develop temperature susceptibility indices. Three important temperature susceptibility indices are: Penetration Viscosity Number (PVN), Penetration Index (PI) and the Viscosity Temperature Susceptibility (VTS).

McLeod [McLeod 76] proposed the use of PVN to determine the temperature susceptibility of asphalt cements. PVN is based on the penetration at 25°C and the viscosity at 135°C. It is calculated as:

$$PVN = -1.5 \frac{[L - \log X]}{[L - M]} \tag{1.1}$$

where:
X = viscosity at 135°C of the asphalt (centistoke)
L = 4.258 - 0.79674 log P (centistoke)
M = 3.46289 - 0.61094 log P (centistoke)
P = penetration at 25°C of the asphalt

Most paving asphalts have a PVN between -1 and 1. Lower PVN values are associated with asphalts that are more temperature susceptible. Based on McLeod's field studies [McLeod 78, 87], he concluded that the PVN of asphalt cement was associated with low temperature transverse cracking and that it remained constant over time. This was strongly supported by a field study of twenty six Canadian airport pavements [Haas 87]. The PVN was believed to be a "fingerprint" of the asphalt that remained constant over time.

The PI [Pfeiffer 50] uses the penetration and the ring and ball softening point temperature. The PI calculation assumes that the relationship between log penetration under the same loading and temperature conditions is linear and that penetration of asphalt at it's softening point is about 800 [Lefebvre 70]. It is calculated as:

$$PI = [30/(1+90B)] - 10 \quad (1.2)$$

where: $B = [2.9031 - \log(Pen25)]/(T_{R+B} - 77)$ (1.3)

Pen 25 = the penetration at 25°C

T_{R+B} = the ring and ball softening point temperature, °C

Research showed that the PI did not work well on waxy asphalts, so Heukelom [Heukelom 73] developed the following revised PI calculation:

$$PI = (20 - 500A)/(1 + 50A) \quad (1.4)$$

where $A = (\log \text{Pen at } T_1 - \log \text{Pen at } T_2)/(T_1 - T_2)$ (1.5)

T_1, T_2 = temperatures in °C.

The PI calculated using the Pfeiffer and Heukelom equations are slightly different than PVN. However, the range of values is very similar [Soleymani 97].

The VTS is another type of susceptibility index as defined below [Roberts 96]:

$$VTS = \frac{\log\log(\text{viscosity at } T_2) - \log\log(\text{viscosity at } T_1)}{\log T_1 - \log T_2} \quad (1.6)$$

Larger VTS's denote higher temperature susceptibility. Most asphalt cement VTS's range from 3.36 to 3.98 [Roberts 96].

Soleymani summarized that many investigators have tried to correlate relationships between consistency tests and temperature susceptibility indices [Soleymani 97]. Some have shown that there is poor correlation between viscosity, penetration and ductility values. This poor correlation has been attributed to unknown and variable shear rates at low temperatures. Consequently, attention must be given to stress levels or shear rates to attain a better relationship between penetration and viscosity. In terms of the temperature susceptibility indices, good correlation is found between the PVN and VTS indices while poor correlation is found between PI and PVN and PI and VTS.

1.5.1 Characterization of Low Temperature Susceptibility of Asphalt Mixes

In order to assess the potential of low temperature cracking it is important to determine the stiffness of the asphalt at low temperatures (asphalts which have a high stiffness modulus at low temperatures are very prone to cracking) [Roberts 96].

Stiffness is a fundamental property in the evaluation of low temperature response. Indirect methods are termed so because they estimate stiffness without direct laboratory measurements (using routine index test data and transforming it into stiffness values) while direct methods for measuring stiffness include uni-axial tension testing [Haas 73]. The two methods commonly used to estimate thermal fracture in an asphalt layer are described herein. The first method calculates stress in a long completely restrained strip or an extension which incorporates both temperature and stiffness gradients through the depth of the bituminous layer. Both of these methods assume the bituminous layer is completely restrained and free of cracks. The data acquired from the constant rate of extension tests is than used to estimate the fracture temperature, T_{FR}, as follows [Seddick 95]:

1) Values of stiffness modulus are determined at each temperature at a certain strain rate.
2) Relationships of stiffness modulus (secant based) versus loading time are plotted for different temperatures and the reference temperature, T_o, is selected (i.e. -15°C) [Seddick 95a].

Master curves are then constructed. These curves characterize the behavior of the asphalt at any combination of loading times and temperatures to provide information needed to predict cracking. Ultimately, if the asphalt stiffness (S) increases too much or too quickly as the temperature drops then cracking can be induced. Once the master curve is established with its associated shift factor a_T. The use of time temperature superposition allows the stiffness modulus to be derived for any failure stress at a specified loading time and temperature. The tensile strength is then adjusted according to the new estimated values for stiffness. In turn, thermally induced stress, σ_T, for a certain cooling time (T), can be estimated. The fracture temperature T_{FR} can then be obtained from the combination of the adjusted tensile strength and the calculated induced stress.

The second method utilized to determine fracture temperature, T_{FR}, is based on a strain criterion. The stress-strain values are calculated at each temperature and at certain strain rate for the mixes. The mean values of failure strain and stiffness are calculated in the same manner as above with the stress criteria. The average strain to failure at four temperatures is plotted versus temperature together with induced strain. Induced strain is determined using the coefficient of thermal contraction α. The point of intersection of average strain and thermally induced strains should represent the point of fracture (T_{FR}) [Seddick 95].

Seddick suggests that thermally induced stress methods seem to give better values than the strain methods and that the results indicate the α is different based on the different test methods and the different T_{FR} relating to different test methods [Seddick 95]. Overall, results suggest that cracking frequency of asphalt pavements in most cases is related to low temperature stiffness of the mix as reflected by the T_{FR}. In a study of twenty-two airport pavements [Haas 87] it was shown that sections with low cracking had low T_{FR} except for one section. Low crack frequencies are generally associated with mixtures of lower stiffness at low temperature regardless of other factors such as freezing index and subgrade type. In addition, sections with higher T_{FR} had more cracking with much stiffer mixes at low temperatures. This indicates cracking frequency is largely associated with mix stiffness rather than freezing index per se or subgrade soil type. However, Deme has shown that subgrade type on the St. Anne's Road was dependent on subgrade type [Anderson 99].

1.5.2 Low Temperature Cracking Models

A considerable amount of research has been directed toward the prediction of low temperature cracking in flexible pavements. In essence, the following are design considerations for prevention of low temperature cracking [Haas 94]:

1. Select an appropriate grade of asphalt cement for the design temperature conditions within certain specifications.
2. Design a limited stiffness for the asphalt mix for the design temperature conditions.
3. Predict cracking temperature by using the estimated or measured stiffness values of the mix and expected field temperature conditions.
4. Predict the frequency of cracking at various ages for the design under consideration based on empirical relationships and measuring asphalt binder and mix properties.

1.5.3 PVN and Cracking

In the mid 1980's, a comprehensive study was carried out which examined cold climate performance of Canadian airport pavements [Haas 87]. It developed cracking prediction models for design and it assessed the feasibility of using a new asphalt specification for controlling cracking. The study involved taking four replicate core samples from each of the twenty six airport pavements. Tests such as penetration and viscosity on the extracted binders, and residual asphalt content were performed. In addition, low temperature tests were performed on beams sawn from the cores and the stiffness modulus was calculated for various stress, strain and loading time values. Penetration-viscosity number (PVN) values were calculated from viscosity and penetration test results on the extracted binders. Regression equations were also developed which related the observed transverse cracking in the field to several independent variables, including Riding Comfort Index (RCI). Based on these regression equations, a life-cycle cost analysis was performed which quantified the dollar savings possible with reduced levels of cracking. A number of equations (see chapter six for details) were developed for transverse cracking, incorporating one or more independent variables, as follows:

Transverse cracking frequency = f(asphalt concrete layer thickness, minimum design temperature, stiffness modulus increase between 0^{o}C and -17^{o}C) and PVN) (1.7)

Overall, the finding indicated that low temperature cracking is strongly affected by PVN, as well as the coefficient of thermal contraction, asphalt layer thickness and subgrade soil type to some degree. It was also suggested that the aged binder (i.e., recovered) PVN values were similar to the original, unaged binders [Haas 87].

1.5.4 Hajek Model

Hajek [Hajek 71] identified five independent variables which influence low temperature cracking as follows: stiffness of original asphalt cement, in kg/cm^2, according to McLeod's method, winter design temperature, ^{o}C; age of pavement, in years; thickness of bituminous layer, in inches and subgrade soil type. This is summarized in functional form as follows:

$$I = f(S, t, a, m, d) \tag{1.8}$$

where
- I = cracking index (transverse cracks per 500/ft)
- S = Stiffness of original asphalt cement, kg/cm^2 determined for loading time of 20, 000 seconds, winter design temperature (^{o}C)
- t = combined thickness of bituminous layers, inches
- a = age of pavement, years
- m = winter design temperature, ^{o}C
- d = subgrade type (dimensionless code 1-sand 2-loam 3-clay)

Limits: Modulus of stiffness equals zero; then function = 0

To use the model the following are required: average freezing index at the site of the project, bituminous layer thickness, soil type, and the penetration and viscosity of the binder. The modulus of stiffness is then estimated by calculating the winter design temperature, the PI or PVN, and the Ring and Ball Temperature T_{R+B} at the base temperature [Hajek 71, Haas 73].

1.5.5 Superpave Low Temperature Models

The non load related material characterization SUPERPAVE model utilizes laboratory test data on asphalt mix samples to determine tensile strength and master relaxation curves [Witczak 97]. All other properties are either assumed (default) or estimated using empirical relations. The tensile strength obtained from the low temperature indirect tension test is defined as the stress at which the difference between the vertical and horizontal diametric deviation reach a maximum. The relaxation master curve is obtained from the low temperature indirect tensile creep test assuming viscoelastic material behavior. Bending beam rheometer (BBR) data for the binder is used to supplement the mixture requirements. Prony series expansion are fitted to all compliance curves and then shifted to form a master curve at a reference temperature of $-20^{o}C$. The master compliance curve is converted to a master relaxation curve via a Laplace transformation [Witczak 97]. The crack extension is modeled using the Paris law as expressed as:

$$\Delta C = A(\Delta K_1)^n \tag{1.9}$$

where
- ΔC = the increment of crack extension
- ΔK_1 = the increment in the stress intensity factor
- A = fracture parameter of the mix which is estimated from an empirical expression developed by Molenaar [Molenaar 83]
- n = fracture parameter of the mix which is obtained from slope m of the log creep compliance curve ($n = 0.8 * (1 + 1/m)$) (1.10)

$$\text{Log } A = 4.389 - 2.52 * \log(E * \sigma_m * n) \tag{1.11}$$

- E = relaxation modulus of the mixture (kPa)
- σ_m = mixture tensile strength (kPa)

In the Superpave system, Molenaar's empirical relationship for A is modified by replacing the modulus E with a field derived calibration coefficient k so that:

where $A = 4.389 - 2.52 * \log (k * \sigma_m * n)$ (1.12)

k = field calibration set to 10,000

σ_m = mixture tensile strength (kPa)

The thermal cracking model consists of three parts: the stress intensity factor; the crack depth (or fracture) model and the crack extent model. The stress intensity factor k_1 is computed from a regression equation as a function of the far field tensile stress and the crack length. The regression equation is based upon two-dimensional linearly elastic finite element analysis that simulates the conditions at the tip of a local vertical crack as functions of loading/stress, layer thickness and material properties. Crack extension for a given cooling cycle is estimated from ΔK_1 value and the incremental form of the Paris law for crack propagation. The crack extent model predicts the number (or frequency) of thermal cracks per unit length of pavement as a function of the probability that the crack depth is equal to or greater than the thickness of the surface layer.

The Superpave low temperature model was extensively reviewed and considered to be basically sound; however the following concerns were noted [Witczak 97]:

1. The coefficients of thermal expansion α calculated in the Superpave program were found to be too high by approximately one order of magnitude.
2. Variation of α with temperature is ignored, which contrasts with known test results that show it is not constant at low temperatures.
3. Although tensile strength is measured at -20^{o}C, -10^{o}C and 0^{o}C the thermal cracking model to estimate the A and n fracture parameters, uses –10^{o}C.
4. The empirically derived A and n parameters in the Paris crack extension law are assumed constant with respect to temperature. In reality, they are expected to vary and these empirical relationships may not be applicable for modified mixes.

Additionally, a separate crack initiation phase is not included in the low temperature cracking model; and field calibration was based on material properties measured from field aged cores. Based on this field aged core measurement, the calibration constants could change if material properties were measured from laboratory specimens and or production cores [Witczak 97].

A study of the two models [Seddick 95a] showed that the Superpave model predicted less cracking than Hajek's model [Hajek 71] at an asphaltic concrete thickness of 280 mm for both mixes. However, at an asphaltic concrete thickness of 140 mm and 70 mm, including Petro Canada's premium asphalt mix, the Hajek model predicted much lower cracking than Superpave. Based on this study, Hajek's model was found to be quick and simple and predicted results comparable if not better than the complex Superpave model. In addition, as cited in the Superpave concerns, Seddick also notes that Superpave utilizes a calculated α. However, a more precise, direct measurement method should be used [Seddick 95a].

1.6 PAVEMENT PERFORMANCE

The basic objective of evaluating pavement performance is to quantitatively establish how well a pavement is serving the users. To assess performance, the functional behavior of a section of pavement is intended to provide a smooth, comfortable and safe ride. Based on this definition, the user's opinion must be measured in order to rate the serviceability of the pavement. The user's opinion is quantified by the response to motion as characterized by the particular pavement-vehicle-human interaction for a given speed. The other major measure of pavement performance or deterioration is surface distress [Haas 94]. Structural adequacy, while not a measure of deterioration per se, is related to the rate of deterioration.

Pavement roughness characterizes the pavement-vehicle-human interaction. Roughness is used as a measure of pavement serviceability and is defined as " a distortion of the pavement surface that contributes to an undesirable or uncomfortable ride" [TAC 97]. The degree of roughness is dependent on the amplitude and frequency or wave lengths of the pavement distortions. The three most common types of devices used to quantify roughness include: profiling measuring devices; response measuring devices, and subjective rating [TAC 97].

The second measure, surface distress, is often expressed in terms of a Surface Distress Index (SDI), or incorporated in a Pavement Condition Index (PCI) [MTO 90]. The PCI combines the type of distress with the extent and severity. The principle causes of pavement distress include: construction deficiencies, material deficiencies, traffic loading and environmental and climatic conditions. The rating classifies the type of density or extent of the distress (few, intermittent, frequent, extensive, very extensive). The severity and density of the particular distresses are then converted to a Distress Manifestation Index (DMI). An equation has been developed to combine the roughness and the DMI to determine a PCI value, according to Ontario practice [MTO 90].

These performance indicators enable pavement engineers to monitor pavements, provide information for priority programming and in general are an important part of managing a pavement network [Tighe 97].

1.7 DATA SOURCES

The databases used in the thesis research to develop the integrated model have come from a number of sources. The three primary sources include: data obtained through research grants, data obtained through professional contacts and data obtained from the literature review. Some of the data was used to develop the model while some was used to verify the model. If the data was used for development, it was not used for verification and vice versa. This report focuses on data obtained from C-LTPP however, it is important to comment on other sources of data.

Two of the primary databases used to develop and verify the model came from research grants specifically related to this research project. The Canadian Long-Term Pavement Performance (C-LTPP) database through a Graduate Research Grant. The Canadian Strategic Highway Research Program (C-SHRP) initiated this C-LTPP project in 1989. It is a national full scale field experiment involving the design and construction of 24 test sites located on the major highway system in all ten provinces, each with two to four adjacent test sections for a total of 65 test sections. Each test site consists of an asphalt overlay placed over an existing asphalt concrete pavement with a granular base course. Adjacent sections were constructed to allow a comparison of the performance of different overlay strategies under identical traffic, climatic conditions and the same underlying pavement structure. The experiment was designed so that across the 24 test sites, asphalt overlay factors, traffic, environment, subgrade type and freezing index conditions could be compared [C-SHRP 95, 96].

Although the C-LTPP database contains various data on the experimental factors, the following data relevant to the integrated model developed herein was obtained: material data, pavement design information, pre-construction and as-built data, pavement distress data, roughness data, traffic data, subgrade data (bore hole logs), weather data and moisture data. The data has been analyzed and incorporated into the modules as appropriate.

Additional data was obtained from C-SHRP however, it was not part of the aforementioned grant. These data came from three other test roads located in Lamont (Alberta), Hearst (Ontario) and Sherbrooke (Quebec). These sites were constructed in 1991 and 1992 to validate or suggest possible changes to incorporating binder and mixture specifications appropriate for Canadian climatic conditions. As well the intent was to investigate existing fracture temperature models for Canadian use [Anderson 99]. Material characterization and cracking data from this C-SHRP study have been used in this research.

The second database obtained through a grant with Imperial Oil Limited (IOL) in the form of a University Research Grant (URG). Through this grant an extensive database on material characterization was obtained. The database includes asphalt properties from around the world. It is extensive in terms of both the number of asphalt cement samples and the testing data made available. Both chemical and physical test results are available for various asphalt cements. This database was used primarily for examining material characterization. Due to the proprietary nature of the data, actual crude sources have not been identified. Additional cost data and cracking performance data from test sites was obtained from IOL

The second category of data has been obtained through professional contacts. Cost data from the Ontario Ministry of Transportation (MTO) was obtained from their Project Value System (PVS). This system provides item prices for capital construction projects and maintenance contracts. The individual costs per contract or project are provided as an average of the three lowest bid prices. The data includes prices from contracts/projects from 1993 to 1997 (most recent available). Additional cost data was obtained for various conventional and enhanced asphalt cements from asphalt suppliers in Canada.

Another source of data provided by MTO were action plan fact sheets which determine performance, typical pavement designs and maintenance schedules for respective pavement sections located throughout Ontario. This data was collected as part of a previous study which examined the Rehabilitation and Reconstruction Candidates in Ontario for the Canadian Portland Cement Association [Tighe 98]. The action plan fact sheets combined with data obtained from another study [Tighe 97] have been used to develop “best practice” pavement sections used in the integrated model.

The third source of data has been obtained through the literature review and has been referenced in the respective modules.

1.8 ANALYSIS OF VARIANCE

Analysis of Variance (ANOVA) is used in this research to identify variation associated with experimental data. It has been used to examine various parameter relationships related to asphalt cements. ANOVA is a statistical technique for analyzing measurements depending on several kinds of effects operating simultaneously. The measured observations may be part of experimental or non-experimental science [Scheffe 59].

ANOVA procedures separate a portion of the variation observable in a response variable into two basic components: variations due to assignable causes and variation due to uncontrolled or random causes [Mason 89]. Assignable causes are generally known sources of variation that can be controlled or measured during an experiment. Random sources are all other sources of variation not controlled or measured. The ANOVA variability is measured as sum of squares. The total variability is referred to as the total sum of squares (TSS). For a single factor experiment, the variability is partitioned as an assignable cause (A) or as an error cause (E) as depicted below:

$$TSS = SS_A + SS_E \tag{1.13}$$

where: TSS = Total sum of squares
SS_A = Sum of squares due to assignable causes
SS_E = Sum of squares due to random causes

The degrees of freedom (sample size – dependent variables) refers to the number of statistically independent response variables or functions of the response variables which comprise a sum of squares. The mean sum of squares (MSS) are the respective sums of squares divided by their number of degrees of freedom. F is used to determine overall compliance with the null hypothesis. The F-statistic is the mean square due to the experimental factor divided by the mean square due to error [Mason 89].

In most cases, the experimenter wants to determine if the variation between the "treatment" means is significantly larger than the variation that occurs within treatments [Duever 98]. The Null Hypothesis (H_o) generally tests if the parameters corresponding to the main effects and interactions are zero. The complement, the Alternative Hypothesis (H_a) would suggest that there were differences in the parameters (they are not equal) for at least one pair of treatments. The $F_{calculated}$ in the ANOVA is compared to the tabulated value of F, or $F_{critical}$ given by the degrees of freedom between and within the experiment at the pre-selected significance level (α). The significance level of the test represents the Type I error. Type I error is the chance of rejecting the null hypothesis when it is true. A 95% confidence level or $\alpha = 0.05$ is commonly specified.

If the $F_{calculated} > F_{critical}$, the H_o is rejected, therefore concluding that there are differences between the experimental treatments or that at least one pair of treatments are different. Alternatively, if the $F_{calculated} < F_{critical}$, the response variability attributable to assignable causes is not significantly greater than the variability from random causes [Mason 89]. Overall, the F-test indicates whether or not the means are equal. However, it does not tell us which particular means are unequal [Christensen 96].

Another very useful parameter associated with ANOVA is the significance probability (p). The p value is the probability of obtaining a value for a test statistic that is as extreme or more extreme than the observed value assuming the H_0 is true [Mason 89]. The calculated p value can be compared to the significance level, similar to the aforementioned F-test procedure. Overall, it is the level at which the test would just barely be rejected. The smaller the p value, the more evidence against the H_0. For instance, if the p = 0.06, then at α = 0.10 (90% confidence level), the H_0 would be rejected, and there is a 6% probability the conclusion is wrong. However, at α = 0.05 (95% confidence level), the H_0 would not be rejected (Type II error). The p value in this case indicates how consistent the data is with the H_0. Thus if a large p value is obtained, it indicates great consistency with the H_0 and conversely a small p value in this situation would indicate inconsistency with H_0 [Christensen 96].

Type II error or β, is the probability of incorrectly rejecting H_0 when the null hypothesis is false. This risk is almost never known to the experimenter [Mason 89]. Type II error, can be calculated for specific values of interest. This calculation is known as Power (1 - β), or the probability of correctly rejecting H_0 when H_0 is false. Type II error depends on the size of the sample and the degree of difference between the hypothesized and true populations at the given significance level. The larger the sample size, the more easier a difference between the two populations can be detected [Scheffe 59]. This calculation is important when the experimenter must know if the true population mean is different than hypothesized as it increases the information in the sample that is incorporated into the test statistic [Mason 89]. If the H_0 is true, the power of a test equals α. If a very small α is given, the statistical significance occurs only for a large difference between the hypothesis and the true population. To realize good power, the largest possible sample sizes are to be employed. If the sample size is small, the significance level must not be too small because it reduces the power of the test [Mason 89, Scheffe 59].

1.9 PROBABILITY DISTRIBUTIONS

Results from an experiment can be described as discrete or continuous. Discrete variables are those that are finite while continuous variables can take on an infinite number of possible values. For a discrete variable, the probability of outcome is described as a summation over all possible values, while the relative frequency behaviour of continuous variables is called a probability density function [Scheaffer 82]. For continuous variables, once the random variable are obtained, probability distributions can be assigned to serve as models for various observed phenomena. If a distribution is continuous, it is then important to check symmetry and skewness of the distribution [Button 99].

For continuous variables the bounds of the distribution are extremely important. Most of the variables in this research are bounded by zero but have no upper limit. In addition they generally have one mode and are positively skewed with most values closest to the lower limit. When there is no sharp upper bound, a skew and a single mode, then a lognormal or gamma distribution are good candidate distributions [Button 99].

When a distribution function and its parameters of a random variable are known, the exact outcome cannot necessarily be predicted due to the inherent randomness of the natural phenomenon. This uncertainty or error of parameter estimation can be reduced by increasing the sample size [Ang 75]. To solve most engineering problems, it is preferable to obtain as large a sample size as possible for predicting future behaviour. However, time and money often limit the ability to collect a large sample size.

CHAPTER TWO: MATERIAL CHARACTERIZATION

This chapter describes material characterization. The intent is to examine how McLeod's Penetration Viscosity Number (PVN) relates to the Superpave low temperature parameters, how PVN varies over time and what type of variability is associated with PVN values for different crude sources. The statistical significance of PVN is assessed and analyzed. The results of this analysis are then used to predict cracking in the next chapter.

2.1 ASPHALT BINDER SELECTION

The behavior of the asphalt pavement at low temperature conditions is affected by the properties of the asphalt cement. Various researchers [McLeod 76, Haas 87, Roberts 96] have agreed that the asphalt cement is a dominant component in low temperature performance. Thus, it is important to study the low temperature behaviour of the asphalt cement to obtain a better understanding of low temperature performance. The main objective of this analysis is therefore to characterize the asphalt cement in terms of low temperature susceptibility.

This susceptibility is the rate at which the consistency of the asphalt cements changes with a change in temperature. In general, the refining process has a large impact on the asphalt cement as it influences the chemical structure [McLeod 75]. However, regardless of the refining process, variations in crude source combined with proprietary refining differences yield variations in the asphalt cements [Marks 85]. Much of the unsatisfactory performance of asphalt pavements occurs due to the failure to select the proper asphalt cements.

The intent of asphalt specifications is to identify which asphalts should be used given a particular design situation. In 1990, the Canadian General Standards Board (CGSB) through the committee on Road Materials, produced revised specifications [NSB 90]. The specification uses penetration at 25^{o}C and either absolute viscosity at 60^{o}C or kinematic viscosity at 135^{o}C. Temperature susceptibility was incorporated into the specification and defined at 60^{o}C absolute viscosity and penetration at 25^{o}C. Three asphalt groups were specified, Group A for low temperature susceptibility, Group B for medium temperature susceptibility and Group C for high temperature susceptibility [NSB 90, EBA 93]. The PVN values can be easily computed from the penetration and viscosity values. Some of the concerns associated with the CGSB specification include: penetration does not describe the mechanical properties related to performance of a binder and the penetration and viscosity are not determined at low temperatures or for long term aging.

The Superpave test methods have been designed to measure mechanical properties at various loading rates and in-service temperatures. The asphalt classification is based on the average seven day maximum and one day minimum pavement design temperature [SHRP 95a]. Overall both systems classify the asphalt binders for use in pavements with the intent of reducing and/or eliminating permanent deformation, fatigue cracking and low temperature cracking for the given facility.

This analysis focuses on temperature susceptibility. The penetration at 25^{o}C and the viscosity either at 60^{o}C or at 135^{o}C have been traditionally used to assess material properties. However, with SHRP Superpave design methodology, penetration would no longer be used and would be replaced with alternative measures. In any case, viscosity and penetration continue to be used as in-line tests in asphalt plants as a quality control indicator[1]. From these two material tests, the PVN can be calculated. If an asphalt cement has a PVN = 0.0, then it has a low temperature susceptibility while a PVN = -1.5 would mean the asphalt cement is highly temperature susceptible (i.e. high probability of low temperature cracking). Based on studies performed by McLeod [McLeod 75, 76], and an Iowa Department of Transportation (DOT) study [Marks 85], it appears that PVN is a good indicator of susceptibility. PVN has also shown to be a statistically significant variable in the progression of low temperature cracking on 22 Canadian Airports [Haas 87].

Based on this information, and the fact that many databases including the C-SHRP C-LTPP database do not include the SHRP test parameters, PVN was used to examine low temperature susceptibility. PVN, which utilizes penetration, is considered an empirical measure. However, given that long-term performance data are not available for many in-service roads, and PVN appears to be a good measure, it was used to assess the material properties of the asphalt cements. One of the important assumptions is that PVN was developed using asphalts manufactured by steam and vacuum reduction processes and thus its applicability to all asphalt cements needed to be examined.

Data obtained from IOL, C-SHRP C-LTPP, C-SHRP test roads and the literature were used in this analysis. In some cases both the CGSB and Superpave test results were available while in other cases only the CGSB results were available.

The purpose of this analysis is to determine:

1. If the PVN calculated using the absolute viscosity is not statistically different from the PVN calculated using the kinematic viscosity
2. To assess the uncertainty associated with PVN calculations.
3. If PVN remains constant with time, for both short term and long term aging.
4. How the PVN relates to the Superpave PG specifications.
5. How the PVN varies within a crude source.

Initially all the asphalts in the databases were included. However because PVN was developed based on asphalts produced by steam and vacuum reduction processes, some of the results appear to be inconsistent. Consequently, when an inconsistency arose in the analysis, the crude source was examined. In some cases, it was subsequently omitted if it was felt to be outside the "valid PVN

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[1] The alternative would be Dynamic Shear Rheometer (DSR) test values. However, the cost of the equipment can be prohibitive and the speed and practicality of regular production applications can be questioned.

2.2 PVN CALCULATIONS

This analysis examines whether the PVN values calculated on original or aged asphalt using either the kinematic viscosity (@ 135°C) and/or absolute viscosity (@ 60°C) are statistically different. For most of the asphalt test results available, both the kinematic and absolute viscosities on original asphalt cements were available. If an aged sample was available, then in most cases only the absolute viscosity was available. Six ANOVA's have been calculated for the initial PVN examination as shown in Figure 2.1. This figure details the data that were used to calculate the PVN's using a 95% confidence level. When the absolute viscosity was available, it was used to determine the PVN.

To calculate the PVN using the kinematic viscosity, the following equation was used:

$$PVN = -1.5 \frac{[L - \log X]}{[L - M]} \qquad (2.1)$$

where:
- X = viscosity at 135°C of the asphalt (centistoke)
- L = 4.258 - 0.79674 log P (centistoke)
- M = 3.46289 - 0.61094 log P (centistoke)
- P = penetration at 25°C of the asphalt

Initially 45 asphalt cement samples[2] from the C-SHRP test roads and IOL database were used to test if the PVN calculated for original samples using kinematic viscosity and absolute viscosity and the PVN calculated using aged material (aged penetration and absolute viscosity of the sample) were statistically the same. Although, there are more than 45 samples in the combined database, there were only 45 samples where these three PVN values could be calculated. ANOVA A, was used to determine if the PVN values calculated using the asphalt data from the various laboratory tests at various times would be significantly different statistically. The null hypothesis for between asphalt cements was established as follows:

$$H_o: \; PVN_{Asphalt\ 1} = PVN_{Asphalt\ 2} = \ldots = PVN_{Asphalt\ N} \qquad (2.2)$$

$$H_a: \; PVN_{Asphalt\ i} \neq PVN_{Asphaltj} \qquad (2.3)$$

Where: H_o = Null Hypothesis
H_a = Alternative Hypothesis
N = number of samples (N = 45)
$PVN_{Asphalt\ 1}$, $PVN_{Asphalt\ i}$, $PVN_{Asphaltj}$ = Calculated PVN values

The null hypothesis for variation within an asphalt cement was as follows

$$H_o: \; PVN_{Asphalt\ 1} = PVN_{Asphalt\ 2} = PVN_{Asphalt\ n} \qquad (2.4)$$

$$H_a: \; PVN_{Asphalt\ i} \neq PVN_{Asphaltj} \qquad (2.5)$$

Where: H_o = Null Hypothesis
H_a = Alternative Hypothesis
n = number of asphalt cement samples (n = 3)
$PVN_{Asphalt\ 1}$ = PVN(original penetration, original viscosity @135°C)
$PVN_{Asphalt\ 2}$ = PVN(original penetration, original viscosity @60°C)

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[2] A database of detailed material properties, not included herein, has been kept on file in the writer's office at the University of Waterloo.

$PVN_{Asphalt\ 3}$ = PVN(aged penetration, aged viscosity @60°C)

Complete ANOVA results are summarized in [Tighe 99] and are summarized in Table 2.1.

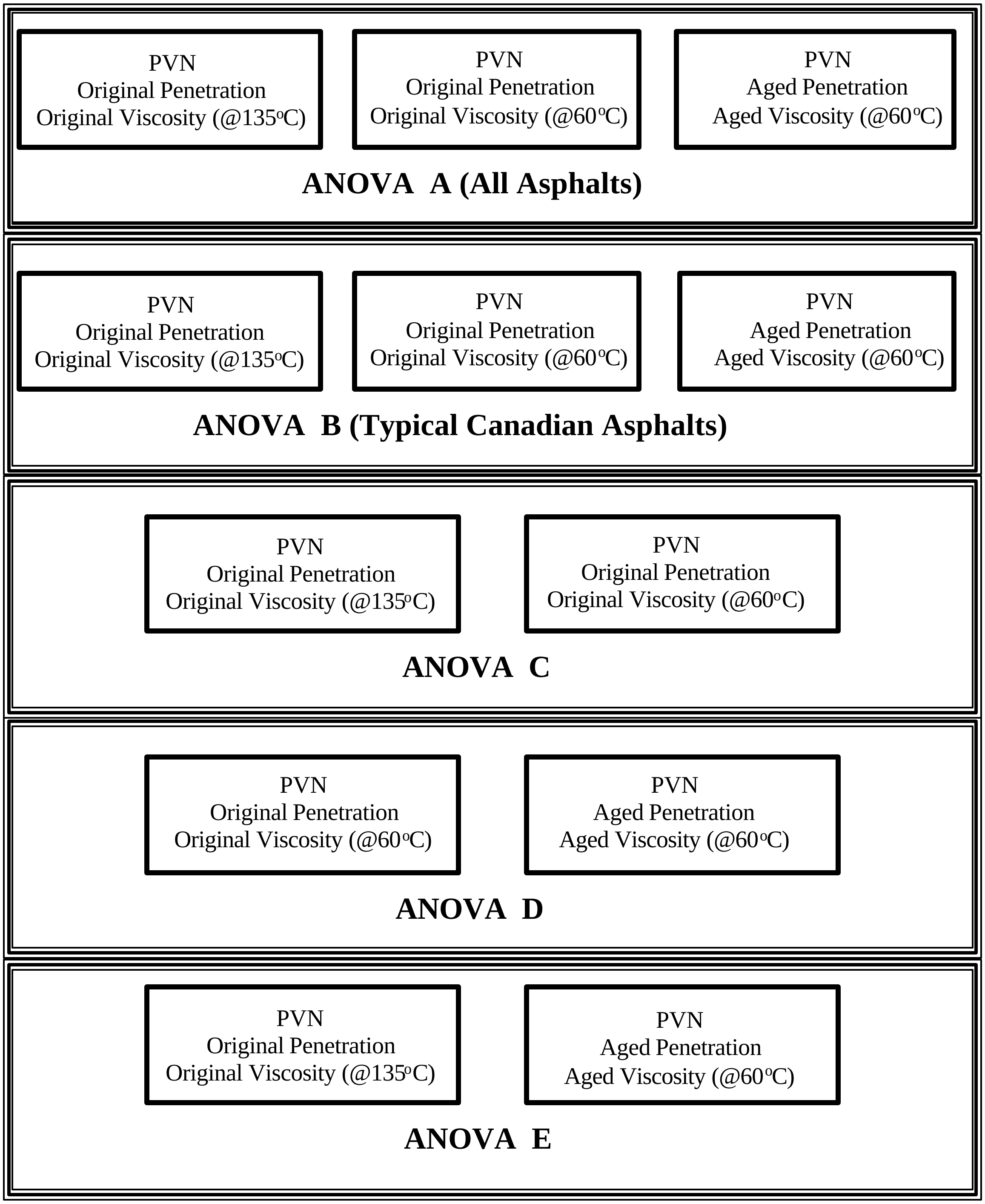


Figure 2.1 Summary of Data Needs For ANOVA

Table 2.1 PVN Calculation Method Comparison Using All Asphalts (ANOVA A)

Comparison	Test	$F_{CALCULATED}$	$F_{CRITICAL}$	P-value	Degrees of Freedom
$PVN_{Asphalt\ 1}$ To $PVN_{Asphalt\ 45}$	Between[1] Asphalt	3.9650	1.5147	1.9539E-8	44
$PVN_{Asphalt\ 1}$ $PVN_{Asphalt2}$ $PVN_{Asphalt\ 3}$	Within[2] Asphalt 1, 2,3	4.2366	3.1001	0.0175	2

Notes: 1) Represents the difference between asphalt samples where the null hypothesis states all asphalts are equal.
2) Represents the differences in PVN values calculated for the same asphalt where the null hypothesis states regardless of how PVN is calculated the values will be equal.

The results indicate that the PVN variation between the true value of the asphalt cement, or at least one of values are different ($F_{CALCULATED} > F_{CRITICAL}$). The low p-value indicates there is a small probability under the null hypothesis of having an F value more extreme than 3.9650 for comparison one and 4.2366 for comparison two. This result is consistent with the engineering experience that different asphalts would have different low temperature susceptibilities. The within asphalt result indicates that there is a significant difference between the PVN calculation methods. Upon further examination of the PVN values for the 45 asphalt cements, using the three laboratory samples, it would seem reasonable that the differences can be explained. The laboratory tests performed on the various asphalt cements were not necessarily tested or performed by the same person. Thus, some differences or errors due to random causes may not be properly assigned. However, the repeatability of penetration and viscosity testing is very good. In addition, some of the asphalts analyzed were not typical asphalts used in North America. Based on these findings, the analysis was repeated using "typical" asphalt cements used in Canada (ANOVA B). The data was examined to check for unusual results that may be attributed to testing errors and/or data input errors. The results are presented in Table 2.2.

Table 2.2 PVN Calculation Method Comparison Using Typical Canadian Asphalt (ANOVA B)

Comparison	Test	$F_{CALCULATED}$	$F_{CRITICAL}$	P-value	Degrees of Freedom
$PVN_{Asphalt\ 1}$ To $PVN_{Asphalt\ 29}$	Between[1] Asphalt	55.4561	1.6775	1.6614E-31	28
$PVN_{Asphalt\ 1}$ $PVN_{Asphalt2}$ $PVN_{Asphalt\ 3}$	Within[2] Asphalt 1, 2,3	0.3643	3.1619	0.6963	2

Notes: 1) Represents the difference between asphalt samples where the null hypothesis states all asphalts are equal.
2) Represents the differences in PVN values calculated for the same asphalt where the null hypothesis states regardless of how PVN is calculated the values will be the same.

In total, 29 asphalt samples were included in the ANOVA B analysis where the three calculated PVN values were compared. The first null hypothesis was rejected as the $F_{CALCULATED} > F_{CRITICAL}$ (55.4561 > 1.6775). This result was consistent with ANOVA A and is consistent with the theory of differences in low temperature susceptibility. Again, the p-value is very small indicating consistency. The ANOVA for within variation associated with the three values of PVN calculation results in the acceptance of the null hypothesis as the $F_{CALCULATED} < F_{CRITICAL}$ (0.3643 < 3.1619). This indicates that it is not possible to find a difference which is significant at the 95% confidence level. Consequently, the PVN calculated using PVN (original penetration, original viscosity @135°C), PVN (original penetration, original viscosity @60°C) and PVN (aged penetration, aged viscosity @60°C) are statistically equal, for these 29 asphalts.

Based on the high p-value obtained in ANOVA A for within group variation, three additional ANOVA's were carried out, ANOVA C, ANOVA D and ANOVA E. Data from the Canadian asphalts analysis, ANOVA B plus additional points for asphalt samples where only two PVN values were available were included in the subsequent analysis. ANOVA C compares the PVN calculated on original binder using only original asphalt data. Table 2.3 summarizes the ANOVA C for the original asphalts. Tables 2.4 and 2.5 summarize the difference between PVN calculations using only absolute viscosities (ANOVA D) and PVN calculations using kinematic viscosities on original and absolute viscosities on aged asphalts (ANOVA E). Detailed results for these two evaluations are in [Tighe 99].

Table 2.3 PVN Calculation Method Comparison Using Original Asphalt Only (ANOVA C)

Comparison	Test	$F_{CALCULATED}$	$F_{CRITICAL}$	P-value	Degrees of Freedom
$PVN_{Asphalt\ 1}$ To $PVN_{Asphalt\ 49}$	Between[1)] Asphalt	37.8962	1.6154	6.31E-26	48
PVN@135C PVN@60C	Within[2)] Asphalt	0.7667	4.0426	0.3856	1

Table 2.4 PVN Calculation Method Comparison Using Absolute Viscosity (ANOVA D)

Comparison	Test	$F_{CALCULATED}$	$F_{CRITICAL}$	P-value	Degrees of Freedom
$PVN_{Asphalt\ 1}$ To $PVN_{Asphalt\ 48}$	Between[1)] Asphalt	7.3516	1.6238	1.15E-10	47
$PVN_{Original}$ PVN_{Aged}	Within[2)] Asphalt	3.6138	4.0471	0.0634	1

Table 2.5 PVN Comparison Using Original (@135°C) and Aged (@60°C) (ANOVA E)

Comparison	Test	$F_{CALCULATED}$	$F_{CRITICAL}$	P-value	Degrees of Freedom
$PVN_{Asphalt\ 1}$ To	Between[1)] Asphalt	1.6124	1.5579	0.0383	56

$PVN_{Asphalt\ 57}$					
$PVN_{Original}$ PVN_{Aged}	Within[2)] Asphalt	1.0365	4.013	0.3130	1

Notes: 1) Represents the difference between asphalt samples where the null hypothesis states all asphalts are equal.
2) Represents the differences in PVN values calculated for the same asphalt where the null hypothesis states the true values of the PVN are the same.

In all three cases where two PVN values are compared, the null hypothesis is rejected for the between asphalt analysis. This is consistent with ANOVA A and B. With regard to the within asphalt test, in these three cases the null hypothesis is accepted, as the $F_{CALCULATED} < F_{CRITICAL}$. This indicates that the differences between the PVN calculations are not statistically significant or that regardless of what is used for the PVN calculation, the overall result is that the PVN would be statistically the same. Given that the analysis is carried out for original binder and binder which has been aged, there are two pertinent implications to this finding. First, it indicates that PVN can be calculated with either absolute viscosity or kinematic viscosity and the results will be the same. The second implication is that the PVN remains constant over short term aging as simulated in the laboratory, as the differences between the original asphalt and aged sample are not statistically significant. Consequently, in terms of short term aging the PVN remains constant.

2.3 IS PVN A LONG TERM FINGERPRINT?

In 1996, a study was published which presented penetration, viscosity, BBR data and cracking data on six samples at four different stages [Kandahl 96]. The test data were compiled on the original asphalt cement, short term aging, after construction and seven years in-service. Analysis of the data as part of this thesis research, indicates that the PVN value remains constant over time for these six asphalt cements. Although the PVN values have a limited range, it does indicate temporal stability, which supports the McLeod theory that PVN is a "fingerprint" [McLeod 76]. The concept of a fingerprint is very powerful as PVN can be used to determine low temperature susceptibility in a quick and simple manner. Table 2.6 summarizes ANOVA F. The ANOVA indicates that between asphalts, the calculated PVN values are statistically significant. With regard to the within asphalt analysis, the $F_{CALCULATED} < F_{CRITICAL}$, which means the null hypothesis is accepted and the PVN remains constant with time. This result is very important as it confirms that PVN does remain constant with time.

Table 2.6 PVN Over Time (ANOVA F)

Comparison[1)]	Test	$F_{CALCULATED}$	$F_{CRITICAL}$	P-value	Degrees of Freedom
$PVN_{Asphalt\ 1}$ To $PVN_{Asphalt\ 6}$	Between[2)] Asphalt	67.241	2.901	9.84E-10	5
$PVN_{Original}$ & PVN_{Aged} & $PVN_{Construction}$ & $PVN_{In\text{-}Service}$	Between Points In Time[3)]	3.034	3.287	0.0620	3

Notes: 1) PVN calculated using data from [Kandahl 96].
2) Represents the difference between asphalt samples where the null hypothesis states the

true PVN values for the asphalt are equal.

3) Represents the differences in PVN values calculated for the same asphalt where the null hypothesis states the PVN values calculated will be equal.

2.4 DETERMINING POWER OF THE ANOVA

The ANOVA results indicate that PVN is constant over time, it is repeatable and it is an indicator of low temperature cracking. These conclusions are based on the ANOVA results where the $F_{CALCULATED} < F_{CRITICAL}$ at the 95% confidence level. However, there is Type II error (β) or a buyer's risk associated with this result. It is the risk of accepting the conclusion when it is wrong. In other words, even though the ANOVA states the true PVN values are equal, there is a risk that they are not.

This risk is almost never known to the experimentor, although, it can be calculated for specific values of interest. The calculation is known as the power calculation or the probability of correctly rejecting the H_o when the H_o is false [Mason 89]. Power depends on the sample size and the degree of difference between the hypothesized and true populations [Scheffe 59]. If the H_o is true, the maximum power of a test equals α. Based on data available, this power test would be a two-way layout with equal numbers of observations in the cells and would be described in equation 2.6. The power calculation can be simulated using this equation with a Monte Carlo analysis. The probability of rejecting the null hypothesis is determined by dividing the number of simulations where the H_o is rejected by the total number of simulations. Alternatively, the power can be calculated by calculating φ for the Pearson and Hartley tables provided in Scheffe and described in equation 2.7 [Scheffe 59].

$$Y_{ij} = \mu_{ij} + \alpha_i + \beta_j + \varepsilon_{ij} \qquad (2.6)$$

Where: μ_{ij} = mean row value

α_i = a x [row averages matrix] i.e. .01 x [1 0 –1]

β_j = [column averages matrix] i.e. [1 2 –1 –2]

ε_{ij} = iid N (μ, σ^2)

a = percent difference in the true populations

$$\varphi = [(1 / (\upsilon_1 + 1).5) + 1 / \sigma_R \text{ x } ((\upsilon_2) \Sigma (\alpha_i - \alpha_m)^2)^{.5}] / (\sigma^2) \qquad (2.7)$$

Where: φ = value required for Pearson and Hartley Table [Scheffe 59]

υ_1 = rows degree of freedom

υ_2 = column degree of freedom

σ_R = standard deviation of rows

σ^2 = variance entire sample

α_i = mean for the i^{th} row under the alternative hypothesis Ha

α_m = a [row average matrix]

a = percent difference in the true populations

The degrees of freedom (υ_1 and υ_2) are used to select the appropriate Pearson and Hartley table [Scheffe 59]. The probability of rejecting the null hypothesis (power) can then be determined from the charts. When the degree of difference in true values is zero, the probability of rejecting the null hypothesis is 5%. This would be expected as the F test is being conducted at the 95 % confidence level and the power would be 5%. In other words, when two values are equal, there is a 5% probability that the null hypothesis would be rejected. As the percent difference between values increases, the power increases indicating a better probability of detecting differences.

To determine the power of ANOVA's B, C, D, E and F, the value was calculated according to equation 2.7. The probability of rejecting H_o was then determined using the Pearson and Hartley tables in Scheffe [Scheffe 59]. The limitations of these calculations were that the tables provided values for degrees of freedom of rows (υ_1) between one and six and degrees of freedom for columns (υ_2) between six and sixty.

To assess the power, the 95% probability of rejecting the null hypothesis was examined on the operating curves. The 95% was selected for consistency with the ANOVA. Table 2.7 summarizes the percent difference between the two values when the power is 95 %. The percent difference in the true values at a 95% probability of rejection ranges from 3.3% to 45%. Based on this finding, it would indicate that ANOVA F, which tests the temporal nature of PVN is an extremely good test. At a 95% rejection of the null hypothesis, the difference in values would be very small for the null hypothesis to be rejected. In fact for ANOVA B, C, and D, the results are also very good as they are all under 10 %.

Table 2.7 PVN Power Summary for ANOVA's

ANOVA	**% Difference @ 95%**	$\mathbf{u_1}$	$\mathbf{u_2}$	**Purpose of ANOVA Test**
B	7.9 %	2	28	Repeatability of PVN and PVN over time
C	6.7%	1	48	Repeatability of PVN only
D	8.0%	1	47	Repeatability of PVN and PVN over time
E	45%	1	56	PVN over time
F	3.3%	3	5	PVN over time

For ANOVA E, which compares the PVN calculated on original and aged asphalt, it shows the true values would need to differ by 45% for rejection. This result is concerning and contradicts ANOVA F. Although, there are more samples with ANOVA E, the ANOVA F has four performance points, including a long term performance point.

2.5 PVN AND SUPERPAVE PG COMPARISON

The relationship between the CGSB and SHRP design methodologies is examined in terms of low temperature susceptibility. If the PVN is consistent with the Superpave low temperature grades and if a comparison shows that a higher PVN corresponds to a lower PG, then it would reinforce the idea that PVN is a low temperature susceptibility parameter. This also has implications for assessing long term performance as the SHRP Superpave performance grades are not available for the C-LTPP database; however, PVN's can be calculated.

CGSB specification bands have been shown to be much narrower at high and low in-service temperatures [Jhanwar 98]. In general the band is usually 1°C to 2°C as compared to the 6°C for the

Superpave increments. Thus, to apply a transformation between systems is technically not advisable. For example a CGSB 150 – 200 A, which meets a PG 58-28, does not mean that a PG 58-28 meets a CGSB 150 – 200 A [Jhanwar 98]. As well, variability associated with intra-laboratory, "within site" reproducibility and inter-laboratory, "different sites" could result in misclassification of a PG asphalt [Puzic 96]. To avoid such misclassifications, the actual grades should be used as opposed to the official grades with the 6°C band.

One major advantage to the Superpave PG methodology is that it subdivides the asphalt cements by providing the maximum and minimum temperatures. For example a PG 58 – 28 would have better low temperature performance compared to a PG 58 – 22. The following analysis examines whether the PVN value within a CGSB grade provides similar information in terms of low temperature susceptibility as the Superpave PG asphalt grades.

2.5.1 PVN and Superpave PG Minimum Temperature Comparison Results

The asphalt cements included in this analysis are those samples for which both the PG grade was available and PVN could be calculated. The PVN values for the PG asphalts were compared both within groups and between groups at the 95% confidence level. Four PVN comparisons were made for each of the following : PG 58 – 22 and PG 58 – 28, PG 52 – 28 and PG 52 – 34, PG 64 – 22 and PG 64 – 28 and PG 46 – 28 and PG 46 –34. The null hypothesis is rejected for the first three comparisons (i.e. $F_{CRITICAL} < F_{CALCULATED}$) as shown in Table 2.8. This means that for these comparisons, there are statistically significant differences for the PVN's in these groups. However, in the last case, the PG 46 – 28 and PG 46 – 34, the difference is not statistically significant. In all cases, as one moves from a less conservative low temperature grade to a more conservative grades, for example from PG 46 – 28 to PG 46 – 34, or from PG 52 – 28 to PG 52 – 34, the PVN value increases. This finding has various implications, as it indicates that PVN does relate to low temperature susceptibility because the comparison has essentially isolated the PG low temperature influence.

Table 2.8 ANOVA Summary for Superpave Comparison

Comparison[1)]	Test	$F_{CALCULATED}$	$F_{CRITICAL}$	P-value	Degrees of Freedom
$PVN_{PG\ 58-22}$ AND $PVN_{PG\ 58-28}$	Between[2)] Grade	12.184	4.279	0.002	1
$PVN_{PG\ 52-28}$ AND $PVN_{PG\ 52-34}$	Between[2)] Grade	20.928	6.608	0.006	1
$PVN_{PG\ 64-22}$ AND $PVN_{PG\ 64-28}$	Between[2)] Grade	51.287	6.608	0.0008	1
$PVN_{PG\ 46-28}$ AND $PVN_{PG\ 46-34}$	Between[2)] Grade	5.636	10.128	0.098	1

Notes: 1) PVN calculated and compared according to the PG low temperature grade.
2) Represents the difference between asphalt samples where the null hypothesis states all asphalts are equal.

CHAPTER THREE: LOW TEMPERATURE CRACKING

3.1 PREDICTION OF CRACKING

The objective of this analysis is to relate the characteristics of the asphalt binder (stiffness, PVN, etc.) to thermal cracking. Stiffness is a fundamental parameter. It has been shown that limiting stiffness is related to thermal cracking. In mechanical terms, if the asphalt stiffness, S, increases too much or too quickly as the temperature drops, then cracking can be induced. Limiting stiffness specifications, as reflected in SHRP's Superpave PG binder specifications, is really an indirect means of controlling low temperature cracking. Another method is to select a design temperature for a certain project around which the cracking temperature, T_{FR}, for a particular asphalt cement can be estimated. In effect, the design approach based on estimated T_{FR} for expected in-service temperature conditions is a variation on the limiting stiffness approach [Haas 94]. However, neither approach estimates the frequency of cracking, which is needed to relate to performance.

If the premise holds that cracking is related to stiffness, and that stiffness is related to PVN, then a cracking frequency model using stiffness directly as an independent variable would be very useful for predicting performance. It is desirable for a pavement designer to estimate cracking frequency as in Canada low temperature cracking can substantially affect the long term performance and life-cycle cost of a pavement. It may even be possible to design a pavement without temperature related cracks. However, due to structural requirements for fatigue and rutting this may not be technically or economically feasible. Hence a trade-off may be necessary. However, regardless of the pavement design factors, if the designer can predict low temperature performance, particularly in Canada, and optimize the design by relating the cracking to performance and life cycle cost, it could result in tremendous cost savings. The Hajek model [Hajek 71] and the Canadian Airport Model [Haas 87] are used to predict low temperature cracking frequency for the C-LTPP and the C-SHRP test sections [Anderson 99] in this analysis.

3.2 MODEL ONE: HAJEK MODEL PREDICITON

The Hajek model was developed based on 42 observations from a number of pavement sections in Ontario and Manitoba [Hajek 71]. Five independent variables are related to a cracking index. The cracking index is based on the MTO definition is the number of full plus one half of the half transverse cracks per 500 foot section of two lane roadway. Cracks less than one half of the roadway width are not included, the assumption being that they form subsequent to low temperature cracking and are therefore not a primary manifestation of the phenomena [Haas 73]. The Hajek model (R^2 = 0.82) predicts cracking as:

$$\log CI = 30.3974 + 0.6026*S*\log(d) - 12.4958*m + 1.3388\log(S) - 2.1316*d - 0.874*t*\log(S) + 6.7977*\log(S) \quad (3.1)$$

where: CI = Cracking Index

S = Stiffness of original asphalt cement in kg/cm^2*10^{-1} as determined by McLeod's method for loading time 20,000 seconds and for winter design temperature

a = Age of the pavement, for the time of prediction
m = Winter design temperature in (^{o}C x -10^{-1})
d = Subgrade type (dimensionless code 1-sand 2-loam 3-clay)
t = Combined thickness of the bituminous layers in inches

The Hajek model was selected based on Seddick's work [Seddick 95a] and the concerns raised by the SHRP review committee with regard to the Superpave low temperature cracking models. [Witczak 97]. Overall, based on the available data, Hajek's model seemed well suited to this research. However, it must be recognized that the stiffness used in this model, S, is not determined by test but rather by an indirect (nomograph) means developed by McLeod and that it was only developed for conventional binders [Haas 73].

3.3 MODEL TWO: CANADIAN AIRPORT MODEL PREDICITON

The second model used in the research is the Canadian Airport model [Haas 87] which predicts transverse cracking. This model is based on data obtained from 22 airports across Canada. Data includes cracking surveys, laboratory test data, climatic data as well as design and construction information. Thirty-two variables were examined during the development of this model as detailed in [Haas 87]. Regression models were used to relate the independent variables to cracking. The best-fit model (R^2 = 0.70) and the model used herein is defined as:

TRANCRAK = 218 + 1.28 ACTHICK + 2.52 MINTEMP + 30 PVN – 60 COEFFX (3.2)

where:TRANCRAK = Transverse crack average spacing in metres
MINTEMP = Minimum temperature recorded on site in ^{o}C
PVN = Penetration Viscosity Number
COEFFX = Coefficient of thermal contraction in mm/1000mm/^{o}C
ACTHICK = Thickness of the asphalt concrete layer in centimetres

One limitation associated with this model is related to the fact that only viscosities and penetrations on recovered asphalts were available for the 22 airports and PVN calculations were for the recovered asphalts. Original asphalt properties would also have been desirable or even preferable. However, the model was developed based on observed performance and it seems reasonable to examine it in this research.

3.4 HAJEK MODEL PREDICTIONS FOR THE C-LTPP SITES

Cracking frequency was estimated for those sections where the five independent variables as outlined in equation 3.1 were available. The stiffness value was determined based on the penetration and PVN for the respective test sections while the thickness value was obtained from the available core data. Both the total thickness and overlay thickness were examined. The overlay thickness or new thickness for the C-LTPP sites was used as it was found to give more reliable cracking results. The total thicknesses of the asphalt for the three G-SHRP test roads (new construction) were used. The subgrade and minimum temperature were categorized according to the data provided. Table 3.1 summarizes the results for the sections where cracks could be estimated using the Hajek model and where the observed transverse crack data was available.

Table 3.1 Low Temperature Cracking Using Hajek Prediction (Page 1 of 2)

Data	Test Site	Section	Age Last Evaluation (years)	Cracking Index Predicted (/150m)	Cracking Index Observed (/150m)
C-LTPP	810404	1	7	13.3	14.3
		2	7	6.8	4.4
		3	7	12.7	1.7
		4	7	12.8	1.9
	820205	1	4	12.8	0.5
		2	4	9.7	0.0
	820502	1	8	10.0	0.0
		2	8	8.8	0.0
	820605	1	8	9.2	0.0
		2	8	4.7	0.0
	830403	1	8	53.9	71.5
		2	8	32.7	29.0
		3	8	59.9	30.9
	830801	1	7	60.2	92.6
		2	7	11.9	25.4
		3	7	95.8	96.6
		4	7	45.8	78.2
	840101	1	7	9.8	0.0
		2	7	9.5	0.0
		3	7	12.7	0.7
	840204	1	7	33.9	11.8
		2	7	45.2	22.8
	840604	1	7	49.3	69.9
		2	7	26.2	58.6
		3	7	NA	64.3
		4	7	54.6	75.2
	850201	1	7	NA	0.0
		2	7	NA	0.0
	850206	1	7	5.5	5.2
		2	7	7.2	19.8
	850601	1	7	NA	0.0
		2	7	NA	0.0
	860501	1	7	4.1	0.0
		2	7	7.2	0.0
		3	7	8.2	0.0
	860603	1	7	11.2	0.0
		2	7	11.4	0.0
		3	7	12.5	0.0
	870102	1	6	13.4	40.4
		2	6	21.9	66.8
	870504	1	6	41.9	33.8
		2	6	70.0	54.0

Table 3.1 Low Temperature Cracking Using Hajek Prediction (Page 2 of 2)

Data	Test Site	Section	Age Last Evaluation (years)	Cracking Index Predicted (/150m)	Cracking Index Observed (/150m)
C-LTPP	870505	1	6	9.0	0.0
		2	6	NA	49.0
		3	6	11.8	5.5
		4	6	27.1	23.8
	870701	1	6	27.7	60.5
		2	6	17.8	54.6
	880203	1	8	13.0	0.8
		2	8	11.7	0.0
		3	8	13.0	2.5
		4	8	11.9	0.0
	890503	1	7	41.4	32.2
		2	7	14.7	9.7
		3	7	17.3	14.0
		4	7	17.0	15.0
	890702	1	7	33.5	10.9
		2	7	40.4	22.7
	900402	1	8	48.3	37.1
		2	8	NA	12.9
	900802	1	8	60.6	38.3
		2	8	NA	32.1
		3	8	NA	26.2
	900803	1	8	39.8	26.4
		2	8	NA	55.2
C-SHRP	Lamont	1	6	67.2	12.0
		2	6	45.5	17.9
		3	6	1.1	0.0
		4	6	92.7	18.1
		5	6	16.5	7.1
		6	6	1.7	2.9
		7	6	8.3	1.0
	Hearst	AA	6	0.0	0.6
		A	6	0.9	0.3
		B	6	2.1	0.6
		BB	6	2.6	1.6
	Sherbrooke	A	5	5.8	1.2
		B	5	57.8	6.8
		C	5	25.6	4.9
		D	5	32.2	1.6

The purpose of the analysis was to examine how close the Hajek results were to the observed transverse cracking. The predicted results are those results corresponding to the age the pavement section was when it was last evaluated. The observed values are those values provided in the databases. It appears that in some cases, the observed cracking is very similar to the predicted cracking. However, in some cases, particularly when there are more than 100 cracks observed over 150m, the model under predicts cracking. It is not readily apparent why the prediction is so different from the observed. Some difference may be attributed to the subjective nature in counting cracks and the variability associated with the other independent variables.

An ANOVA was performed only on those sections where both Hajek predictions and observed were available. The results are summarized in Table 3.2. The results indicate that the differences between the predicted cracking and the observed are statistically insignificant ($F_{CALCULATED}$ 0.4407 < $F_{CRITICAL}$ 3.9798). Thus it would indicate that the Hajek model is a good predictor of cracking frequency.

Table 3.2 ANOVA Summary for Hajek Prediction and Observed Cracking

Comparison[1)]	Test	$F_{CALCULATED}$	$F_{CRITICAL}$	P-value	Degrees of Freedom
Section 1 To Section 70	Between[2)] Sections	2.3621	1.4900	0.0002	69
Hajek Cracking Prediction AND Observed Cracking	Between [3)] Model and Observed	0.4407	3.9798	0.5096	1

Notes: 1) Uses C-LTPP and C-SHRP test data [Anderson 99].
2) Represents the difference between test sections where the null hypothesis states all sections are equal.
3) Represents the differences in Hajek prediction and observed cracking where the null hypothesis states regardless of how cracking is determined (observed or predicted), the values will be the same.

A power test was performed on the ANOVA results for the between model and observed cracking. The results identify the percent difference between the true values based on the Hajek model and observed model comparison. When the probability of rejecting the null hypothesis is 95%, the percent difference in true values would be 3.4. This small difference indicates the Hajek cracking prediction is a very good model and the buyers risk is low.

3.5 AIRPORT MODEL PREDICTIONS AND OBSERVED DATA

Similar to the Hajek model evaluation, a comparison of the Canadian Airport Model [Haas 87] and observed data was performed. Thickness, PVN and minimum temperatures were the same values used as in the Hajek predictions for the various sections. The fourth independent variable in this model is the thermal contraction coefficient (α). This value refers to the asphalt mix volume that decreases when the temperature decreases. Ideally a direct measurement is preferable. However, when a direct measurement cannot be taken, the contraction coefficient can be estimated using prediction methods [Anderson 99]. The predictive equations for thermal contraction used by the Superpave system at the time of this study were not validated and overall there are various methods of estimating thermal contraction coefficients as outlined in [Anderson 99]. However, the use of typical coefficients is often a reasonable procedure when direct measurements are not available [Anderson 99].

For this comparison analysis, two approaches were taken based on the available data. The crack frequency estimates for the C-LTPP sections were calculated using five thermal contraction coefficients considered to be in a reasonable range based on the literature (1.2 x 10^{-6}, 1.5 x 10^{-6}, 2.0 x 10^{-6}, 2.2 x 10^{-6}, and 2.5 x 10^{-6}). The closest cracking estimate for the five coefficients was selected and this value was compared to the observed cracking. The second approach involved calculating the cracking frequency for the C-SHRP sections using the measured values from Anderson [Anderson 99]. The predicted cracking was then compared to the observed cracking. The thermal coefficients for all the sections were calculated based on the observed cracking and compared to the measured values.

3.5.1 Thermal Contraction Coefficient Analysis Using C-LTPP Data

A comparison between the transverse cracks predicted for the C-LTPP test sections and the observed cracks was carried out. When the observed cracking on the various test sections is greater than 70 cracks per 150m it indicates that the cracking prediction for the airport model for the various thermal coefficients is not very close to the observed. One possible explanation for this occurrence is that when more than 70 cracks occur, this is outside the range of the data used to develop the model. Based on this discrepancy, two ANOVA's were run to compare the airport cracking prediction to the observed cracking as summarized in Tables 3.3 and 3.4.

Table 3.3 ANOVA Summary for Airport Prediction and Observed Cracking

Comparison 1)	Test	$F_{CALCULATED}$	$F_{CRITICAL}$	P-value	Degrees of Freedom
Section 1 To Section 65	Between 2) Sections	2.8267	1.5133	2.53E-5	64
Airport Cracking Prediction AND Observed Cracking	Between 3) Model and Observed	9.3084	3.9909	0.0033	1

Notes: 1) Uses C-LTPP test data.
2) Represents the difference between test sections where the null hypothesis states all section are equal.
3) Represents the differences in Airport prediction and observed cracking where the null hypothesis states regardless of how cracking is determined (observed or predicted), the true values will be equal.

Table 3.3 summarizes the ANOVA when all sections are included where both the airport prediction can be made and observed values are available. The differences between the airport model prediction and the observed are shown to be statistically significant. Thus the model does not appear to predict results close to those observed using typical thermal contraction coefficients. When the ANOVA is rerun (Table 3.4) excluding the sections where more than 70 cracks have been observed, the differences between the model and observed are shown to be not statistically significant, with a p value of 0.0687 or 6.87% chance that the result will be greater than the F calculated value. Consequently, it reinforces the fact that the Airport model appears to be conservative in predicting cracking. The Type II error was assessed on this result.

Table 3.4 ANOVA for Airport Prediction and Observed Cracking (< 70 Cracks)

Comparison[1)]	Test	$F_{CALCULATED}$	$F_{CRITICAL}$	P-value	Degrees of Freedom
Section 1 To Section 60	Between[2)] Sections	4.3175	1.5400	3.9E-8	59
Airport Cracking Prediction AND Observed Cracking	Between[3)] Model and Observed	3.4385	4.0040	0.0687	1

Notes: 1) Uses C-LTPP test data.
2) Represents the difference between test sections where the null hypothesis states all section are equal.
3) Represents the differences in Airport prediction and observed cracking where the null hypothesis states regardless of how cracking is determined (observed or predicted), the true values will be equal.

The power test based on the ANOVA in Table 3.4 was performed. At the 95% rejection rate of the null hypothesis, the difference between the true values would only be 7.5%. This difference is small and it reinforces that the Airport Model [Haas 87] is a good model for predicting cracking when less than 70 cracks occur. However, as pointed out in Table 3.3, when more than 70 cracks occur it is not a good predictor.

3.5.2 Thermal Contraction Coefficient Analysis Using C-SHRP Test Sites

The measured thermal coefficients, as provided by Anderson [Anderson 99] for the three C-SHRP test roads located in Lamont Alberta, Hearst Ontario and Sherbrooke Quebec, were used to predict cracking at the minimum temperature. The thermal coefficient based on the observed cracking was also calculated. Only those test roads where greater than 1 crack per 150 m was included. The results, in Table 3.5 indicate that the differences between the measured thermal contraction coefficient and the calculated thermal contraction coefficient based on the observed cracking are not statistically significant. Additionally, the high p value (.8838) indicates that there is a strong indication that the measured value is consistent with the value based on the observed cracking on the various test sections.

Table 3.5 ANOVA for Airport Prediction and Observed Cracking (< 100 Cracks)

Comparison[1)]	Test	$F_{CALCULATED}$	$F_{CRITICAL}$	P-value	Degrees of Freedom
Section 1 To Section 10	Between[2)] Sections	0.8722	3.1789	0.5791	9
Measured α AND α Based on Observed Cracking	Between α [3)]	2.1393	5.1174	0.1776	1

Notes: 1) Uses C-SHRP test sections.
2) Represents the difference between test sections where the null hypothesis states all section are statistically equal.

3) Represents the differences in α measured and α based on observed cracking where the null hypothesis states regardless of how α is determined (observed or measured), the values will be the same.

3.6 THREE WAY COMPARISON

Table 3.6 summarizes the cracking predictions for the Hajek model, Canadian Airport model and the observed cracking on the various test sections. The thermal coefficient calculated based on the observed cracking is summarized in the last column. For the C-LTPP numbered sites (test site 810404 through 900803), the thermal coefficients range between 0.1 to 2.78. The 0.1 to .6 range is based on a very small amount of observed cracking. The majority of the values appear reasonable based on the literature [Anderson 99].

Figures 3.1 and 3.2 summarize the cracking difference for all sections and those sections with less than 70 cracks for the C-LTPP sections. This further emphasizes that when a small amount of cracking is observed, the airport model is closest. However, if the amount of cracking observed increases, the Hajek model gives a closer prediction.

Table 3.6 Comparison of Transverse Crack Prediction (Page 1 of 2)

Test Site	Section	Minimum Temperature (°C)	Airport Model		Hajek Prediction (/150m)	Actual Cracks (/150m)	Thermal 2) Coefficient
			Thermal 1) Coefficient	Crack Prediction (/150m)			
810404	1	-30	2.20	12.9	13.3	14.3	2.22
	2	-30	2.00	5.6	6.8	4.4	1.88
	3	-30	1.50	2.5	12.7	1.7	1.04
	4	-30	1.50	2.7	12.8	1.9	1.12
820205	1	-15	1.20	1.9	12.8	0.5	0 3)
	2	-15	1.20	1.7	9.7	0.0	0.00
820502	1	-15	1.20	1.6	10.0	0.0	0.00
	2	-15	1.20	1.5	8.8	0.0	0.00
820605	1	-20	1.20	2.2	9.2	0.0	0.00
	2	-20	1.20	2.0	4.7	0.0	0.00
830403	1	-35	2.00	12.3	53.9	71.5	2.17
	2	-35	2.20	78.9	32.7	29.0	2.15
	3	-35	2.20	26.9	59.9	30.9	2.21
830801	1	-35	2.20	14.8	60.2	92.6	2.34
	2	-35	2.00	14.1	11.9	25.4	2.08
	3	-35	2.00	33.1	95.8	96.6	2.05
	4	-35	2.18	17.1	45.8	78.2	2.31
840101	1	-23	2.20	6.1	9.8	0.0	0.00
	2	-23	NA	NA	9.5	0.0	NA
	3	-23	1.20	2.2	12.7	0.7	0 3)
840204	1	-24	2.20	8.4	33.9	11.8	2.29
	2	-24	2.20	16.9	45.2	22.8	2.24
840604	1	-25	2.20	10.2	49.3	69.9	2.41
	2	-25	2.20	29.8	26.2	58.6	2.24
	3	-25	NA	NA	NA	64.3	NA
	4	-25	2.20	11.1	54.6	75.2	2.39
850201	1	-30	NA	NA	NA	0.0	NA
	2	-30	NA	NA	NA	0.0	NA
850206	1	-30	2.00	6.5	5.5	5.2	1.90
	2	-30	NA	NA	NA	19.8	NA
850601	1	-30	NA	NA	NA	0.0	NA
	2	-30	NA	NA	NA	0.0	NA
860501	1	-22	1.20	2.0	4.1	0.0	0.00
	2	-22	1.20	1.9	7.2	0.0	0.00
	3	-22	1.20	2.0	8.2	0.0	0.00
860603	1	-20	1.20	1.8	11.2	0.0	0.00
	2	-20	NA	NA	11.4	0.0	0.00
	3	-20	1.20	1.9	12.5	0.0	0.00
870102	1	-20	NA	NA	13.4	40.4	2.73
	2	-20	2.50	50.1	21.9	66.8	2.66
870504	1	-25	2.18	17.3	41.9	33.8	2.27
	2	-25	2.20	15.1	70.0	54.0	2.32
870505	1	-21	1.23	1.2	9.0	0.0	0.00

Table 3.6 Comparison of Transverse Crack Prediction (Page 2 of 2)

Test Site	Section	Minimum Temperature (°C)	Airport Model Thermal[1)] Coefficient	Airport Model Crack Prediction (/150m)	Hajek Prediction (/150m)	Actual Cracks (/150m)	Thermal[2)] Coefficient
870505	2	-21	NA	NA	NA	49.0	NA
	3	-21	2.20	4.7	11.8	5.5	2.27
	4	-21	2.50	67.8	27.1	23.8	2.43
870701	1	-25	NA	NA	NA	60.5	NA
	2	-25	2.50	120.7	17.8	54.6	2.47
880203	1	-20	1.20	2.2	13.0	0.8	0[3)]
	2	-20	1.20	1.6	11.7	0.0	0.00
	3	-20	2.00	4.2	13.0	2.5	1.60
	4	-20	1.20	1.7	11.9	0.0	0.00
890503	1	-27	2.20	18.6	41.4	32.2	2.26
	2	-27	2.50	6.5	14.7	9.7	2.62
	3	-27	2.20	9.1	17.3	14.0	2.30
	4	-27	2.50	5.5	17.0	15.0	2.78
890702	1	-28	2.22	19.6	33.5	10.9	2.10
	2	-28	2.20	11.6	40.4	22.7	2.31
900402	1	-35	3.28	5.4	48.3	37.1	1.90
	2	-35	NA	NA	NA	12.9	NA
900802	1	-35	1.50	4.9	60.6	38.3	1.94
	2	-35	NA	NA	NA	32.1	NA
	3	-35	NA	NA	NA	26.2	NA
900803	1	-35	2.20	34.2	39.8	26.4	2.18
	2	-35	NA	NA	NA	55.2	NA
Lamont	1	-40	1.60	10.4	67.2	12.0	1.63
	2	-40	1.60	13.1	45.5	17.9	1.65
	3	-40	1.60	4.7	1.1	0.0	0.10
	4	-40	1.20	6.8	92.7	18.1	1.43
	5	-40	1.60	4.2	16.5	7.1	1.85
	6	-40	2.70	-4.6	1.7	2.9	1.30
	7	-40	1.60	3.2	8.3	1.0	0.10
Hearst	AA	-35	1.80	7.3	0.0	0.6	0.10
	A	-35	1.80	7.5	0.9	0.3	0.10
	B	-35	1.80	9.0	2.1	0.6	0.10
	BB	-35	1.80	6.8	2.6	1.6	0.60
Sherbrooke	A	-35	1.50	2.7	5.8	1.2	0.50
	B	-35	1.50	4.4	57.8	6.8	1.70
	C	-35	1.50	2.6	25.6	4.9	2.00
	D	-35	1.50	4.5	32.2	1.6	0.50

Notes: NA means not available

1) Indicates closest cracking prediction to observed using coefficients (1.2, 1.5, 2.0, 2.2, and 2.5)

2) Refers to the thermal coefficient that matches the observed cracking

3) Refers to a negative calculated value which is not possible

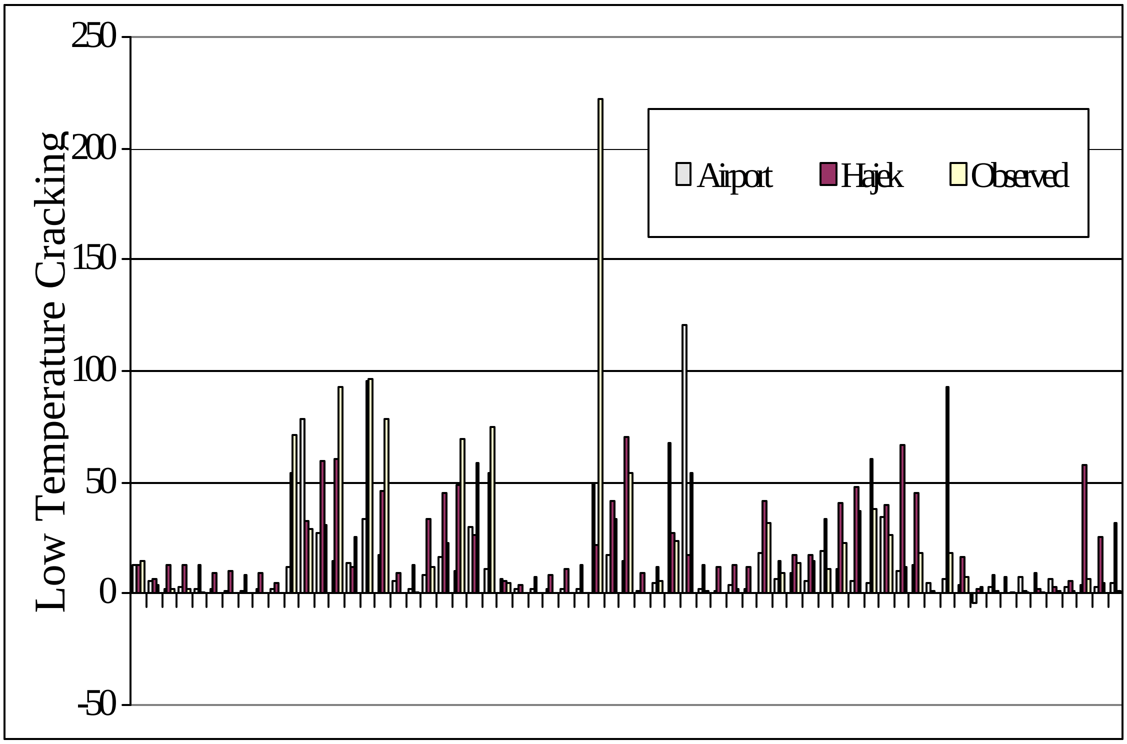


Figure 3.1 Comparison of Cracking Between Airport and Hajek Model For C-LTPP Sections

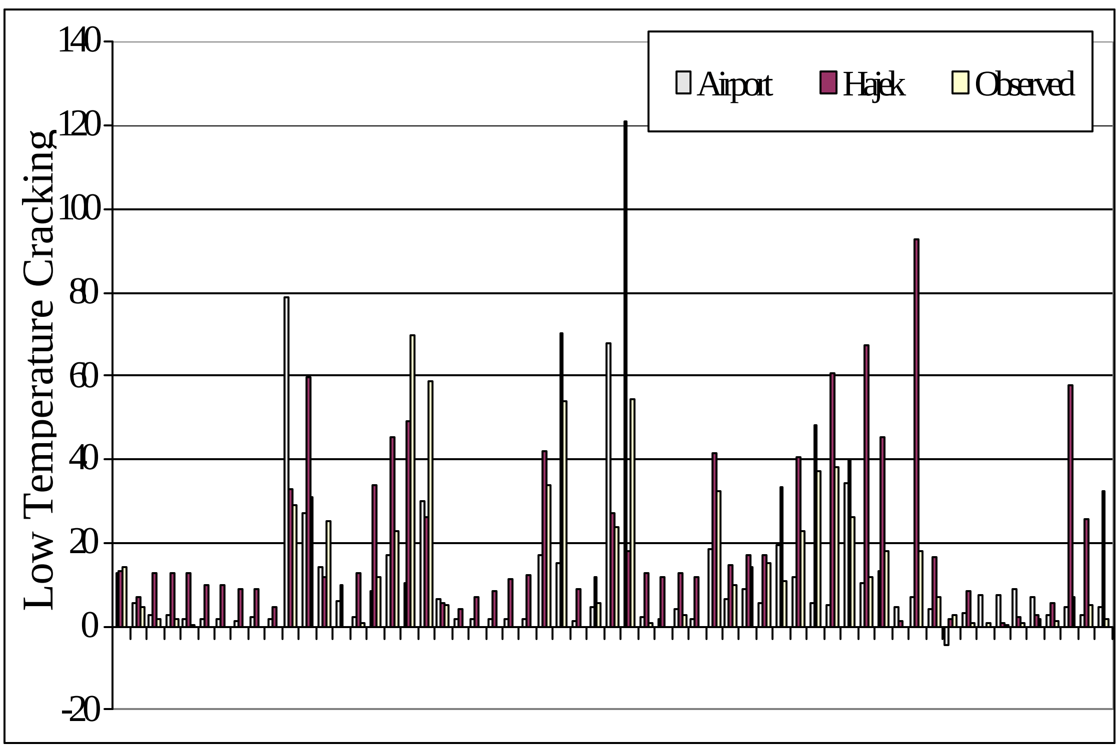


Figure 3.2 Comparison of Cracking (< 70 Cracks) Between Airport and Hajek Model

CHAPTER FOUR: PERFORMANCE PREDICTION

The objective of this chapter is to relate the thermal cracking obtained from the previous analysis to pavement performance. The chapter describes how low temperature cracking is related to roughness in terms of the Riding Comfort Index (RCI). Various relationships, which relate RCI to International Roughness Index (IRI), are examined. As well, measured IRI values on the C-LTPP sections are compared to the predicted IRI values. The roughness information is then used to predict performance.

4.1 PAVEMENT DETERIORATION

The purpose of a performance model is to estimate the future deterioration of the various pavement sections and ultimately determine when the section will need to be rehabilitated [TAC 97]. Traditionally, the Riding Comfort Index (RCI) has been the selected method for characterizing roughness in Canada. This is based on a subjective measure which describes the pavement-vehicle-human interaction. Table 4.1 outlines the typical range of RCI values and their practical significance [MTO 89]. Using the RCI value, the remaining life can be calculated based on the functional classification (i.e., freeway, principal arterial, etc.) and the pavement factors (i.e. traffic level, subgrade, environment, etc.).

Table 4.1 Riding Condition Rating [MTO 89]

RCR/RCI	Ride Condition	Guidelines
8 – 10	Excellent	Very smooth ride.
6 – 8	Good	Smooth ride with just a few bumps or depressions.
4 – 6	Fair	Still comfortable ride with intermittent bumps or depressions.
2 – 4	Poor	Uncomfortable ride with frequent bumps or depressions.
0 – 2	Very Poor	Uncomfortable ride with constant bumps or depressions resulting in rattle and hake of rating vehicle. Cannot maintain posted speed and must steer constantly to avoid bumps or depressions. Dangerous at 80km/hr.

Many agencies use their own roughness measures. In an attempt to standardize roughness, the International Roughness Index (IRI) has been developed [Sayers 86]. It is the measure used for the C-LTPP sections. The IRI is a measurement scale which simulates a standardized response type road roughness measuring system in terms of a quarter car. IRI uses the longitudinal road profile by accumulating the output from a quarter car model and dividing by the profile length to yield a summary roughness index with units of slope such as metres per kilometre (m/km). It has become recognized as a general purpose roughness index that is strongly correlated to most kinds of vehicle response type measuring devices. The IRI is linearly proportional to roughness and it describes profile roughness that causes vehicle vibrations. An IRI of 0.0 m/km means the profile is perfectly flat or smooth. An IRI of 8m/km would be nearly impassible to drive except at reduced speeds [Sayers 86].

4.2 RELATING LOW TEMPERATURE CRACKING TO ROUGHNESS

The results in the previous chapter focus on predicting low temperature cracking in a pavement structure. The intent of this chapter is to determine roughness based on cracking. This is a very difficult and challenging engineering task, because of all the confounding factors. In other words, the problem is to isolate the contribution of cracking to roughness progression or loss of performance. It appears that the only project able to do this involved the Canadian Airport Study [Haas 87], which developed the model of equation 4.1. The equation relates the transverse cracking to the RCI.

$$RCI = 5.4 + 0.02\ TRANCRAK - 11.6 / (TRANCRAK)^2 \quad (4.1)$$

where: RCI = Riding Comfort Index (Between 0 and 10)

TRANCRAK = Transverse crack spacing, m

Cracking represents damage to the pavement. This damage is often associated with reduced in-service life. However, very little data is available which isolates the effect of cracking on performance. One of the major problems in isolating this effect, is that there are so many other factors which can influence pavement performance. Pavement life is increased with increased crack spacing (i.e. lower frequency of cracking). For example, five years longer life is obtained if the crack spacing increases from 5m to 20m, where the 5m spacing corresponds to the normal initial design life of 15 years for these pavements. This increased life has significant implications with regards to the life cycle cost of the pavement.

It is recognized that having only one such model currently available is certainly a weakness in the overall integrated model. Nevertheless, it was also considered appropriate to use it in any case, with a strong recommendation that this should be a priority research item, particularly in view of the adverse effect of cracking on performance.

4.3 RELATING RCI/RCR TO IRI

Based on the subjective nature of RCI, advances in automated roughness measuring devices and the fact that IRI is becoming a standard measure for pavement roughness, various relationships have been developed which relate RCI or RCR to IRI. Based on a review provided in the Transportation Association of Canada Pavement Design and Management Guide [TAC 97], the following five equations were used to relate the RCI value obtained using the Canadian Airport Model to the observed IRI value.

$IRI = 5.588 - 0.578 * RCI$	[Hein 89]	(4.2)
$RCI = 10 * e^{-0.18IRI}$	[Paterson 86]	(4.3)
$RCI = 10 * e^{-0.26IRI}$	[Al-Omari 84]	(4.4)
$RCR_{carusers} = 9.11 - 1.39 * IRI$	[Hajek 95]	(4.5)
$RCR_{truckusers} = 9.37 - 1.71 * IRI$	[Hajek 95]	(4.6)

Where: IRI = International Roughness Index

RCI = Riding Comfort Index

RCR = Riding Comfort Rating

Only equation 4.2 uses RCI as an independent variable to estimate IRI. Thus, it is the only “valid” one in a statistical sense. However, the others, equations 4.3 to 4.6 were also used, with full recognition of this caveat.

4.4 DATA USED FOR PERFORMANCE PREDICTION

The C-LTPP test sections were used in this module as both the thermal cracking and roughness data was available. The roughness data was recently validated and examined under a research contract carried out by the University of Waterloo for the Transportation Association of Canada [Haas 99]. For the purpose of this analysis, the roughness values for each of the C-LTPP sections were compared to the thermal cracking provided. It is important to recognize that thermal or transverse cracking is one of seven types of cracking data (block, centreline, edge, meander, midlane, transverse and wheel crack) available in the C-LTPP database. It is assumed in this work, that the Canadian Airport Model accounts for these other distresses in the roughness relationship although the only independent variable is the transverse cracking.

It is also notable that the IRI values for the C-LTPP test sections are extremely smooth based on engineering experience [Haas 99]. In fact, the IRI values tend to range between 1.0 and 2.0 for most sections at 8 years in service. One reason may be that these are test sections and extra care was taken during the construction.

4.5 ANALYSIS OF C-LTPP SECTIONS

The analysis procedure for determining pavement performance is outlined in Figure 4.1. The RCI values for the C-LTPP sections were calculated using observed thermal cracking. IRI’s were then calculated using equations 4.2 through 4.6 for the various sections. The observed IRI’s for the C-LTPP sections are then compared to the predicted IRI values using the equations.

Table 4.2 summarizes the RCI and IRI calculations using the aforementioned equations. The observed IRI values are also included. The sections included in this summary are only those with less than 100 observed cracks, with RCI values between 0 and 10 and sections where all the IRI values calculated using the equations were positive. It is apparent that the observed IRI values on the C-LTPP sections are consistently less than predicted.

Figure 4.2 indicates that the observed cracking is not well correlated to the observed IRI. It would be expected that as the amount of observed cracking increases, the observed IRI would increase. However, based on this figure there does not seem to be any relationship between the observed cracking and observed IRI. The reason is not apparent, other then it is early in the life of the sections and the IRI’s all are “clustered” in a fairly narrow range. Figure 4.3 shows that the calculated RCI using the Canadian Airport Model decreases with increased cracking (as would be expected) in a form which appears to be sinusoidal. Based on these initial findings, two subsequent analyses were performed. Five ANOVA’s were performed and a cracking grouping based on the observed cracks were performed. An ANOVA for the calculated IRI, using equations 4.2 through 4.6, versus the observed IRI values was performed. The complete summary is found in [Tighe 99] while the summarized results are in Table 4.3. All five of the IRI predictions are shown to be statistically different as compared to the observed IRI values ($F_{CALCULATED} > F_{CRITICAL}$).

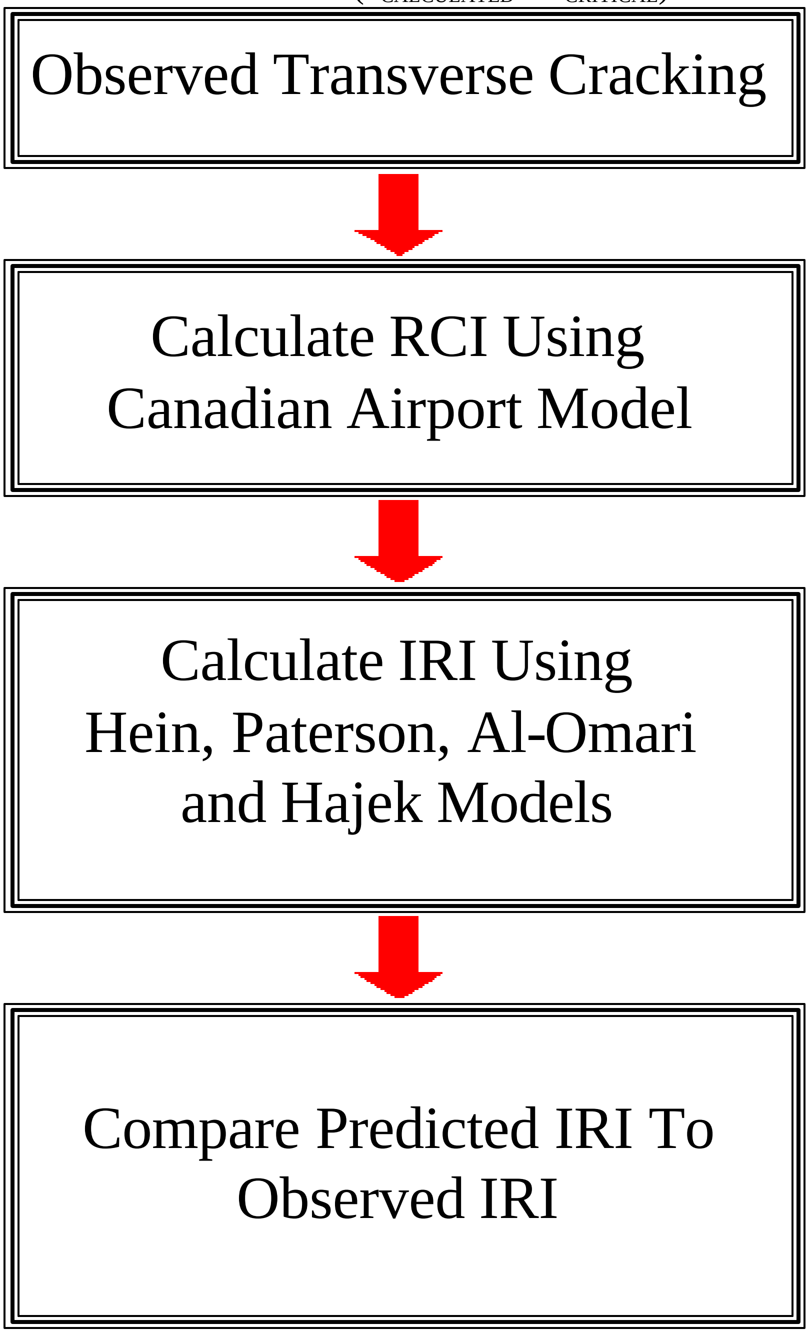


Figure 4.1 Analysis Procedure Pavement Performance Prediction

Test Site	Section	Actual Cracks (/150m)	Airport[1)] Model RCI	IRI[2)]	IRI[3)]	IRI[4)]	IRI[5)]	IRI[6)]	Observed Age	Observed IRI
810404	1	14.3	5.54	2.39	3.26	2.27	2.57	2.24	5	1.452
810404	2	4.4	6.10	2.06	2.73	1.90	2.17	1.91	5	1.372
810404	3	1.7	7.17	1.44	1.84	1.28	1.40	1.29	5	1.335
810404	4	1.9	6.99	1.55	1.98	1.38	1.53	1.39	5	1.174
830403	1	71.5	2.84	3.95	6.95	4.84	4.51	3.82	8	1.266
830403	2	29.0	5.11	2.64	3.71	2.58	2.88	2.49	8	1.497
830403	3	30.9	5.04	2.67	3.78	2.63	2.93	2.53	8	1.573
830801	1	92.6	1.04	4.98	12.49	8.69	5.80	4.87	8	0.851
830801	2	25.4	5.22	2.57	3.59	2.50	2.80	2.43	8	1.146
830801	3	96.6	0.65	5.21	15.09	10.51	6.09	5.10	8	1.465
830801	4	78.2	2.32	4.25	8.07	5.62	4.89	4.12	8	0.862
840204	1	11.8	5.62	2.34	3.19	2.22	2.51	2.19	8	1.838
840204	2	22.8	5.30	2.52	3.51	2.44	2.74	2.38	8	1.798
840604	1	69.9	2.96	3.88	6.73	4.68	4.43	3.75	8	1.584
840604	2	58.6	3.72	3.44	5.47	3.81	3.88	3.31	8	1.66
840604	3	64.3	3.35	3.65	6.04	4.21	4.14	3.52	8	1.42
840604	4	75.2	2.56	4.11	7.53	5.24	4.71	3.98	8	2.319
850206	1	5.2	5.99	2.12	2.83	1.97	2.24	1.98	5	1.409
850206	2	19.8	5.39	2.47	3.42	2.38	2.68	2.33	5	1.34
870102	1	40.4	4.67	2.89	4.20	2.93	3.19	2.75	8	1.59
870102	2	66.8	3.18	3.75	6.32	4.40	4.26	3.62	8	2.633
870504	1	33.8	4.94	2.73	3.90	2.71	3.00	2.59	8	1.295
870504	2	54.0	3.99	3.28	5.08	3.54	3.68	3.15	8	1.134
870505	2	49.0	4.26	3.13	4.71	3.28	3.49	2.99	8	0.986
870505	3	5.5	5.96	2.14	2.86	1.99	2.27	1.99	8	0.765
870505	4	23.8	5.27	2.54	3.54	2.46	2.76	2.40	8	1.356
870701	1	60.5	3.60	3.51	5.65	3.93	3.97	3.38	8	1.485
870701	2	54.6	3.95	3.30	5.13	3.57	3.71	3.17	8	1.017
880203	1	0.8	9.12	0.32	0.51	0.35	-0.01	0.15	8	1.42
880203	3	2.5	6.61	1.76	2.28	1.59	1.80	1.61	8	1.712
890503	1	32.2	5.00	2.70	3.83	2.67	2.96	2.56	8	1.557
890503	2	9.7	5.69	2.30	3.11	2.17	2.46	2.15	8	1.421
890503	3	14.0	5.55	2.38	3.25	2.27	2.56	2.23	8	1.171
890503	4	15.0	5.52	2.40	3.28	2.29	2.58	2.25	8	1.183
890702	1	10.9	5.65	2.32	3.16	2.20	2.49	2.18	5	1.78
890702	2	22.7	5.30	2.52	3.50	2.44	2.74	2.38	5	1.558
900402	1	37.1	4.81	2.81	4.05	2.82	3.09	2.67	8	1.148
900402	2	12.9	5.58	2.36	3.22	2.24	2.54	2.22	8	1.069
900802	1	38.3	4.76	2.84	4.10	2.86	3.13	2.70	8	0.92
900802	2	32.1	5.00	2.70	3.83	2.67	2.96	2.56	8	1.028
900802	3	26.2	5.20	2.58	3.62	2.52	2.81	2.44	8	0.929
900803	1	26.4	5.19	2.59	3.62	2.52	2.82	2.44	8	1.231
900803	2	55.2	3.92	3.32	5.17	3.60	3.73	3.19	8	1.806

Notes: 1) RCI = 5.444+0.019624TRANCRAK - 11.62/(TRANCRAK)2 [Haas 87]

2) IRI = 5.588 - 0.578*RCI [Hein 89]

3) RCI = 10 *e $^{-0.18IRI}$ [Paterson 86]

4) RCI = 10 * e$^{-0.26IRI}$ [Al-Omari 94]

5) $RCR_{car\ users}$ = 9.11 - 1.39 IRI [Hajek 95]

6) $RCR_{truck\ users}$ = 9.37 - 1.71 IRI [Hajek 95]

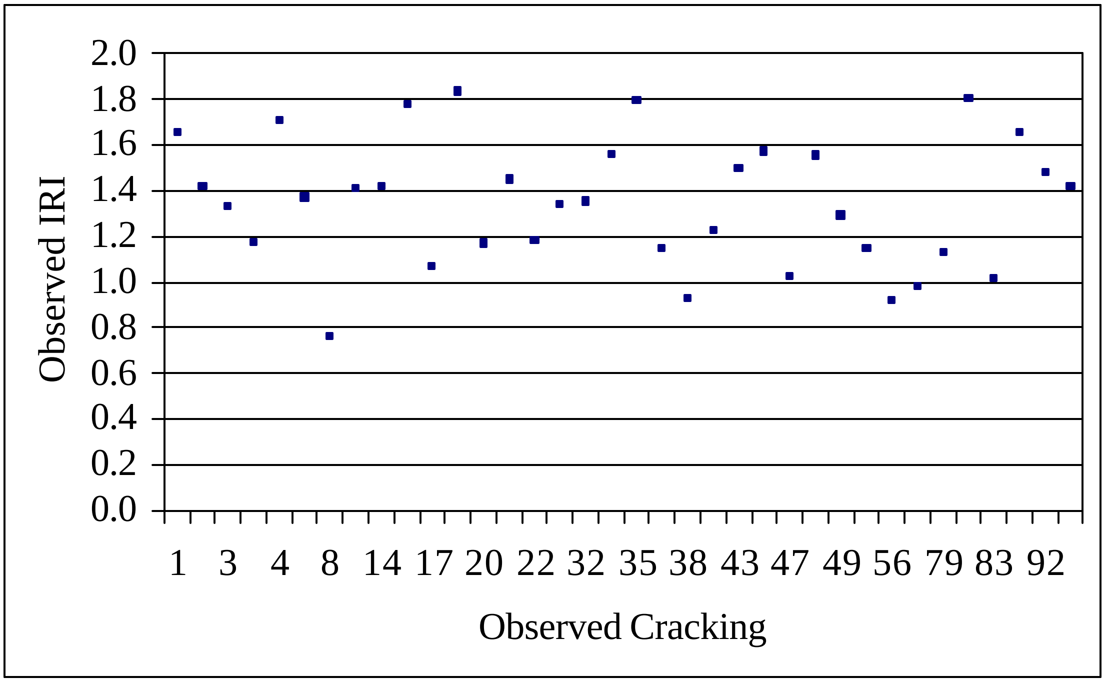


Figure 4.2 Relationship Between IRI and Observed Cracking

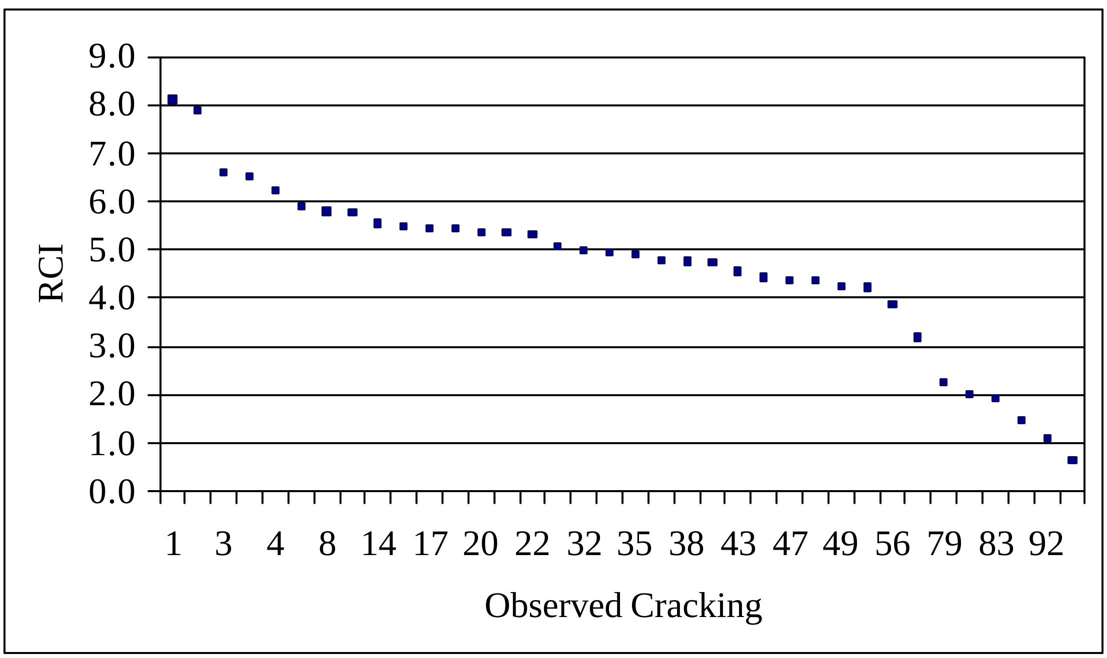


Figure 4.3 Relationship Between RCI and Observed Cracking

Table 4.3 ANOVA for IRI Prediction and Observed IRI

Comparison[1)]	Test	$F_{CALCULATED}$	$F_{CRITICAL}$	P-value	Degrees of Freedom
Hein IRI AND Observed IRI	Between[2)] Observed & Predicted	98.3751	4.0727	1.4E-12	1
Paterson IRI AND Observed IRI	Between[2)] Observed & Predicted	61.9870	4.0727	8.4E-10	1
Al-Omari IRI AND Observed IRI	Between[2)] Observed & Predicted	39.6384	4.0727	1.4E-7	1
$\text{Hajek}_{\text{carusers}}$ IRI AND Observed IRI	Between[2)] Observed & Predicted	95.7969	4.0727	2.1E-12	1
$\text{Hajek}_{\text{truckusers}}$ IRI AND Observed IRI	Between[2)] Observed & Predicted	78.4219	4.0727	3.7E-11	1

Notes: 1) Uses C-LTPP test sections.
2) Represents the differences in the predicted IRI and the observed cracking where the null hypothesis states regardless of how IRI is determined (observed or predicted), the values will be equal.

Table 4.4 ANOVA for IRI Prediction and Observed IRI Less Than 10 Cracks

Comparison[1)]	Test	$F_{CALCULATED}$	$F_{CRITICAL}$	P-value	Degrees of Freedom
Hein IRI AND Observed IRI	Between[2)] Observed & Predicted	2.6991	5.5915	0.1444	1
Paterson IRI AND Observed IRI	Between[2)] Observed & Predicted	9.3023	5.5915	0.0186	1
Al-Omari IRI AND Observed IRI	Between[2)] Observed & Predicted	1.1980	5.5915	.3099	1
$\text{Hajek}_{\text{carusers}}$ IRI AND Observed IRI	Between[2)] Observed & Predicted	2.2790	5.5915	0.1749	1
$\text{Hajek}_{\text{truckusers}}$ IRI AND Observed IRI	Between[2)] Observed & Predicted	1.0518	5.5915	0.3392	1

Notes: 1) Uses C-SHRP test sections.
2) Represents the differences in the predicted IRI and the observed cracking where the

null hypothesis states regardless of how IRI is determined (observed or predicted), the values will be equal.

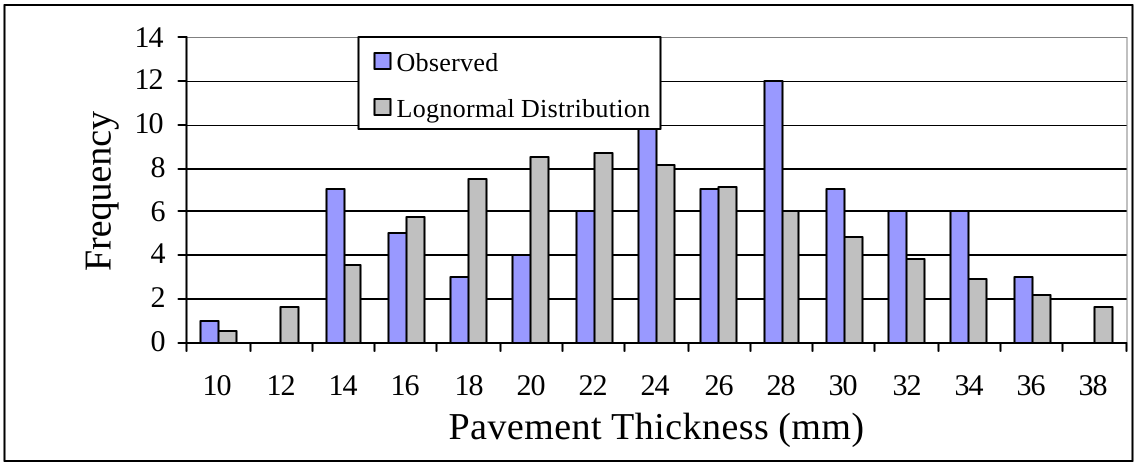
Based on the findings, another ANOVA was performed. This ANOVA focused on the situation where less than 10 cracks were observed. The observed IRI was then compared to the predicted. Based on the results presented in Table 4.4, when there were less than 10 cracks the IRI predicted using the RCI based on the Canadian Airport Model [Haas 87] was statistically not different from the observed IRI with the exception of the Paterson equation.

4.6 THICKNESS VARIATION

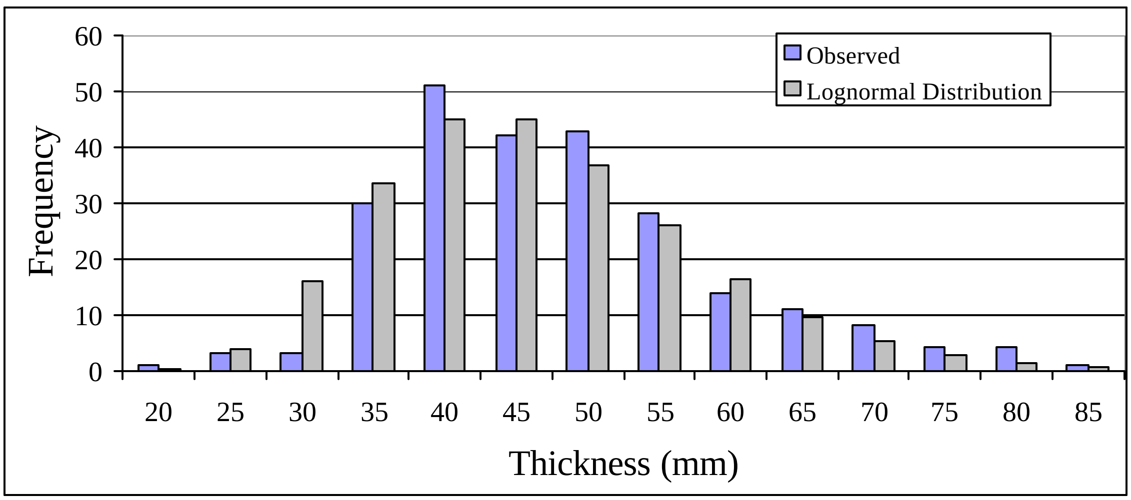
A pavement designer will develop a design based on the circumstances (i.e. the in-service conditions) in which it must perform. Pavement thickness is generally specified based on the structure requirements and is rounded off to the nearest ten mm (i.e. 50mm, 100mm) for practical purposes. There is little to no information in the literature about variation associated with lift thickness and there was no information available on the types of probability distributions associated with asphalt lift thickness. The distributions are examined in addition to examining the mean and standard deviation values relative to design.

The C-LTPP test sections are used in this analysis. The design overlay thickness for each test section was categorized as thin, medium and thick. The sections were categorized based on the average thickness according to the asphalt cores taken after construction. The thin overlay thicknesses were those which fell between 10 mm and 38 mm, medium overlay thicknesses ranged from 25 mm to 80 mm and the thick overlay thicknesses were those with more than 80 mm thickness. Table 4.5 summarizes the mean and standard deviation for the three pavement overlay thicknesses. Although most of the averages appear to be close to the designs, the standard deviation associated with the designs is very high. Figure 4.4 shows the best fit distributions for the three pavement thicknesses. The lognormal distribution is found to have the best ι^2 and be the closest distribution. The chi-square (ι^2) is a goodness of fit test for the distribution models, which is used to indicate the relative degree of validity of the different distributions [Ang 75]. The lower the ι^2, the better the fit.

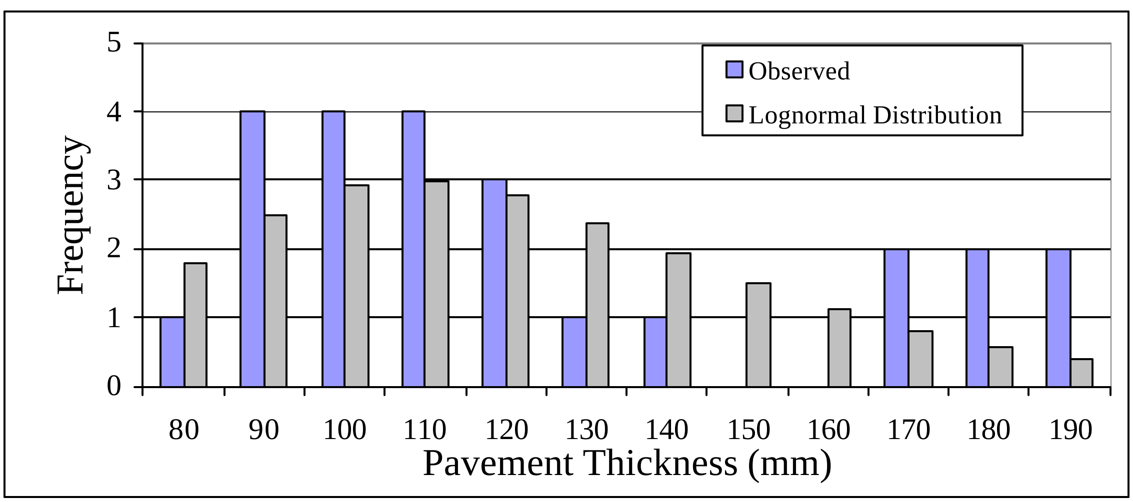
For thin overlay pavement thicknesses, the best fit distributions are not commonly used (i.e. extreme value, weibull, triangular, beta). However, the lognormal is the most common distribution out of the five. For medium overlay thickness, which is the most common thickness in Ontario and in fact throughout Canada, the lognormal distribution is also selected as the best fit. The higher ι^2 for this analysis can be explained by the split between the typcial lift thickness of 40 and/or 50mm. Based on the ι^2 analysis, the lowest ι^2 are shown with the thick overlay thicknesses. The lognormal best fit would be the most reasonable choice as it is more common and appears to be the most reasonable choice as it is more common and seems to be appropriate. It should be noted that although the thick pavement overlay is showing 100mm, it is reasonable



Thickness Distribution for Thin Pavement Overlay Thickness



Thickness Distribution for Medium Pavement Overlay Thickness



Thickness Distribution for Thick Pavement Overlay Thickness

Figure 4.4 Overlay Thickness Distribution Curves Based on C-LTPP Core Thickness

to presume that this was placed as two medium lifts (i.e. two at 50mm). However, this breakdown was not available.

Table 4.5 Best Fit Distributions For Thickness

Overlay Thickness	Average Thickness	Standard Deviation	Best Fit Distribution (c^2)
Thin	25.0 mm	6.8 mm	1. Extreme Value (8.59) 2. Beta (9.44) 3. Triangular (10.00) 4. Weibull (11.13) 5. Lognormal (21.0)
Medium	46.0 mm	11.3 mm	1. Gamma (39.4) 2. Lognormal (51.89) 3. Extreme Value (55.15) 4. Beta (53.52) 5. Logistic (75.00)
Thick	121.0 mm	36.8 mm	1. Extreme Value (3.08) 2. Lognormal (3.08) 3. Pareto (3.92) 4. Normal (5.58) 5. Gamma (3.912)

The lognormal distribution was selected as an overall guide for overlay thickness contrary to the standard belief which assumes it is best described by a normal distribution. In addition, the use of a lognormal distribution would appear to be most appropriate as the values of the random variable with the lognormal variates are always positive.

CHAPTER FIVE: CONCLUSIONS AND RECOMMENDATIONS

The purpose of this chapter is to summarize the major finding of the research and provide recommendations on future direction. The results presented provide a methodology for predicting performance. Pavement designers can use PVN as a tool for predicting field performance. The models proposed predict cracking which is similar to that observed in the field. However an update is recommended to reflect current as-built and in-service conditions. Roughness trends are predicted and compared to those observed. Recommendations are proposed to updating roughness prediction based on in-service conditions.

5.1 MATERIAL CHARACTERIZATION

The following conclusions can be drawn from the material characterization module that examined PVN as a low temperature susceptibility variable.

- PVN is an indicator of low temperature susceptibility.
- PVN is related to the minimum temperature specified in the Superpave design methodology.
- PVN can be calculated with either absolute viscosity (@ 60^{o}C) or kinematic viscosity (@ 135^{o}C) and they are equal.

5.2 LOW TEMPERATURE CRACKING PREDICTION

The conclusions based on the analysis predicted low temperature cracking using the Canadian Airport model and the Hajek model for the C-LTPP test sections is presented.

- The Hajek model is good for predicting low temperature cracking based on the C-LTPP and C-SHRP test sections. It is also easy for designers to determine the variables necessary for pavement life-cycle performance prediction.
- The Canadian Airport model is a good model for predicting thermal cracking. However, it does not perform well in cases where a large amount of thermal cracking (more than 100 cracks per 150m) was observed.
- In cases where the thermal coefficient was measured and used to predict observed cracking, the Canadian Airport Model provided a good prediction.
- The thermal contraction coefficient and the minimum observed temperature is an important variable for predicting thermal cracking.

5.3 PAVEMENT PERFORMANCE

The Canadian Airport model that relates transverse cracking to RCI was assessed. The RCI was then related to IRI using the five equations [Hein 89, Paterson 86, Al-Omari 94 and Hajek 95].

- The RCI predicted using the Canadian Airport Model [Haas 87] predicted values which would be consistent with engineering judgement.

- The observed IRI on the C-LTPP sites was in a very smooth range. This could be related to new and improved construction methods which improve the initial roughness and ultimately increase the service life of the pavement.
- When the predicted IRI values based on the equations were compared to the observed roughness on the C-LTPP sites, the values were not very close. This in part could be due to the fact the IRI values on the C-LTPP sites are based on "Dipstick" measurements and the other relationships were based on different measurements.
- When there were less than 10 thermal cracks per 150m, the IRI predictions were close to observed.

5.4 THICKNESS ANALYSIS

- Thickness of pavement layers is best described by a lognormal distribution based on the C-LTPP test sections for thin, medium and thick lift thickness.
- The probabilistic analysis should include thickness and cost distributions to simulate in-service pavement variation.

5.5 RECOMMENDATIONS

The following recommendations are presented based on the research summarized in this report.

- It is extremely important to continue monitoring the C-LTPP sections as they provide very valuable information into pavement performance in Canada.
- As pavement technology moves into the next century, there is a concerted movement away from conventional testing methods and movement towards the SHRP Superpave methodology. The results presented here indicate that PVN is a good indicator of low temperature susceptibility. PVN also relates to the Superpave PG minimum service temperature and can be easily calculated in a timely manner (i.e. the penetration test and viscosity tests are relatively simple with good repeatability and require a finite sample size). PVN can be used by the designer to predict low temperature cracking, pavement performance and life-cycle cost.
- Use the Hajek model for predicting low temperature cracking as a design tool. Carry out a Bayesian update on the Canadian Airport Model on both the cracking prediction and RCI model to reflect current values. It appears that changes to construction practices have changed the RCI range and how it relates to low temperature cracking.
- Examine thermal contraction coefficients and how they relate to PVN. Carry out a sensitivity analysis to examine how variability influences low temperature cracking.
- Examine as-built roughness and examine the possibility of updating the values provided by Sayers [Sayers 86]. It appears the Sayers values are not reflective of the IRI values being measured on the C-LTPP test sections.
- Examine how as-built roughness influences crack progression and long term performance.
- Develop a comprehensive life-cycle cost package which incorporates PVN, thermal cracking prediction and life-cycle cost.

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