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Analytical and Experimental Investigation of Thermal Cracking in Asphalt Pavements

by

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A thesis submitted to
the Faculty of Graduate Studies and Research
in partial fulfillment of
the requirements for the degree of

Doctor of Philosophy

Department of Civil and Environmental Engineering
Carleton University
Ottawa, Ontario, Canada
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Cracking in Asphalt Pavements**

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Abstract

The problem of thermal cracking in asphalt pavements is a serious concern in cold regions in Canada and the northern parts of the United States. This study involves the structural analysis of asphalt pavement systems and the prediction of pavement performance at low temperatures. The Strategic Highway Research Program, SHRP, has proposed new measures and concepts affecting asphalt technology. The shift towards performance based specifications of pavements resulted in a growing necessity for reliable performance models. At the present time, development of performance models for thermal cracking follows one of two approaches. The first is a phenomenological approach, in which the pavement response is represented by a mechanistic or empirical simplified formula, and therefore detailed material properties, the interaction between pavement layers, or existing cracks cannot be simulated. On the other hand, the fracture mechanics approach utilizes a numerical method, such as the finite element method, to examine the propagation of a crack under a set of defined boundary conditions.

In this thesis, a review of the state-of-the-art on prediction of thermal cracking and cracking mechanisms is presented. A novel approach for the analysis of asphalt pavements behavior at low temperatures is demonstrated. Based on the fracture mechanics approach, a finite element analysis for the transient thermal field and corresponding stress field is performed. Different crack configurations and multiple cracks are considered. The results are used to evaluate the cracking potential for a wide range of transverse cracks. An experimental program is implemented to determine the material parameters required for the numerical analysis. The direct tensile test, tensile fatigue and repeated tensile strain test are

used, and damage indicators are obtained at three test temperatures.

Advanced reliability methods are used to extend an existing thermal cracking prediction model. A sensitivity analysis of the improved model is used to determine the effect of each parameter on the extent of cracking. The model is applied to field data in order to verify its predictive abilities. The results of the finite elements analysis, the developed laboratory test methods and the prediction model are useful tools for defining the factors and mechanisms causing thermal cracking.

The thesis presents three methods of examining thermal cracking at three different levels. While, the analytical approach is applicable to the study of specific cracks with known geometry and boundary conditions, the experimental approach is suitable for exploring project-level alternatives. The reliability-based model can be used at network-level for decision making and prediction of thermal cracking.

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List of Symbols

A	fracture parameter
$[B_T]$ and $[B_u]$	derivatives of the shape function matrices
$[C]$	the matrix of mechanical constitutive relations
C_f	the volumetric heat capacity
C_v	aggregate concentration factor
$D(T, t)$	forcing vector
E	modulus of elasticity
F	factor of safety
G	shear modulus
G_1, G_2	general relaxation moduli
H_{max}	maximum tensile strength of the mix
I	cracking index
K	thermal conductivity
K_1, K_2	fatigue coefficients
K_I	stress intensity factor (SIF)
L	lane width
N_f	allowable number of cycles to cause fatigue cracking
N_{ij}	number of cycles at age i and load case j
$[N_T]$ and $[N_u]$	the shape function matrices
P_{FL}	probability of failure in low-temperature
P_{FF}	probability of thermal-fatigue cracking
P_{FLF}	joint failure probability of low-temperature and thermal-fatigue
R	resistance
S_i	stiffness of layer i

$S(T, t)$	binder stiffness
S_{mix}	the mixture stiffness
T and ΔT	absolute and incremental temperatures
$T_{R\&B}$	softening point
T_S	pavement surface temperature ($^{\circ}\text{C}$)
T_Z	temperature at depth Z ($^{\circ}\text{C}$)
U	strain energy
V_A	percent of air voids in the mixture
W	applied work
Z	depth measured from pavement surface (mm)
a, a_c	crack depth and critical crack depth
b, b_c	crack length and critical crack length
e	an element of the discretized domain D
e_{ij}	deviatoric components of the strain tensor
f	coefficient of friction
k	coefficient of thermal conductivity
n_{ij}	number cycles at age i and load case j
s_{ij}	deviatoric components of the stress tensor
t	time
t_i	traction vector
u	displacement
x	crack depth
z	depth
α	coefficient of thermal contraction
β	thermoelastic coefficient

$\Delta K(a)$	change in the stress intensity factor
δ_{ij}	Kronecker's delta function
ϵ and ϵ_{kk}	strain and hydrostatic strain
λ and μ	isothermal values of Lamé's constants
ν	Poisson's ratio
ω	moisture content in percent
ω_{uw}	unfrozen moisture content in percent
ϕ	temperature field
ρ	mass density
ρ_d	dry density
ρ_{df}	dry frozen density
σ and σ_{kk}	stress and hydrostatic stress
σ_{ij}	stress tensor
τ	time variable
$\theta(x_k, t)$	weighted temperature
ξ	reduced time

Chapter 1

Introduction

1.1 Background

Low temperature cracking of asphalt pavements is a serious problem in Canada and the northern parts of the United States. Ultimately, highway pavements should provide a safe, structurally sound and economical surface for the traveling vehicles under the most severe climatic conditions. Over the past three decades, it has been noted that pavements subjected to severely cold temperatures exhibit a high rate of transverse cracking. The cracking usually occurs at an early age in the pavement life [Carpenter and Lytton, 1978, Christensen, 1982]. The uniformity of the cracking patterns has always suggested that uniform loading conditions, such as those induced by low temperatures, are the main contributor to the cracking [Shields, 1964].

Cracking of asphalt pavements does not constitute a serious problem on its own, but the subsequent damage to the pavement and the underlayers is drastically accelerated [Fromm and Phang, 1972]. As a result, cracking effectively reduces the pavement serviceability rating and structural rigidity. Moreover, there is a high probability that cracks would reflect into the surface layer shortly after rehabilitation.

1.2 Definition of Thermal Cracking

Although a comprehensive definition to what constitutes a “thermal crack” is not clearly stated in the literature, thermal cracks are believed to be transverse in general, have a distinctive repetitive pattern, and appear early in the pavement life. The description of transverse cracking in the Pavement Surface Condition Rating System [RTAC, 1987] is as follows,

“Crack which follows a course approximately at right angles to the pavement centre line. Full transverse cracks tend to be regularly spaced along the length of the road, while half transverse and part transverse occur at shorter intermediate distances”.

This definition applies to half, full and multiple cracks. In this research, a thermal crack refers mainly to transverse premature cracks that appear on those pavement surfaces which are subjected to either severe low temperatures or to repeated large temperature fluctuations. Naturally, the two previous cases are termed low-temperature cracking and thermal fatigue cracking.

There are significant differences between fatigue cracks and thermally-induced cracks. A fatigue initiated crack, caused by axle load repetitions, is more likely to occur at the bottom of the asphalt concrete layer where the tensile strain is at its maximum, and propagates towards the surface. On the other hand, a thermal crack is initiated at the pavement surface where the amplitude of the temperature cycle is highest, and propagates downwards. Thermal cracks are more visible and have a pronounced impact on the riding quality and comfort.

1.3 Impact of the Problem

Pavement distress is characterized by three major types; fatigue cracking, rutting, and thermal cracking [Haas et al., 1994]. The first and second types are dependent upon the axle load and the traffic volume composition. Fatigue cracking initiates at the bottom of the surface layer where load-induced tensile strains are highest. Rutting is affected by the shear strength of the asphalt concrete which decreases considerably at high temperatures, and is generally limited to the wheel path. Thermal cracking, however, initiates at the pavement surface where the temperature fluctuation is greatest. It is not governed by traffic volume or limited to the wheel path. The effects of thermal cracks on the riding quality and pavement deterioration are considerable. In many cases, thermally-induced cracks are observed at an early stage of the pavement life leading to higher user costs and earlier-than-scheduled rehabilitation.

The number of temperature cycles which pavements are subjected to are significantly lower than the number of traffic load cycles. The duration of each thermal cycle is much longer to that of a pulsing axle load. In many regions of the Canadian highways network, the combination of cold temperature and relatively low traffic volume, made thermal cracking the primary failure mode that influences pavement design life. A study of over 200 general aviation airports in cold regions in the United States has shown that the most common pavement problems were non-traffic related and are ranked in order of severity as follows: (1) reflective cracking, (2) thermal cracking, and (3) longitudinal cracking at construction joints [Vinson et al., 1986].

Thermal cracks cause the migration of water and fines into and out of the pavement

structure [Janoo, 1993]. If the base becomes saturated, fines may be pumped out due to the confinement of wheel loads, and consequently creating a cavity that reduces the bearing capacity of the pavement structure. During thawing periods, the warm temperature penetrates the pavement from the surface while the underlayers are still frozen which subjects the pavement to a case of thermal stress reversal. Moreover, the water trapped within surface layers effectively reduces the structural strength of the pavement system.

At low temperatures and severe temperature gradients, the cracked pavement is subject to a curling effect raising the free cracked zone with respect to the rest of the pavement surface, and reducing the vertical support of the multi-layered system in this zone. The same effect is also caused by the freezing of moisture underneath the crack. This “lipping” effect causes a ride discomfort, and accelerates the evolution of cracks closer to the readily cracked zone. Thawing in the spring can cause stepping or depression at the crack, also termed “cupping”. The costs of restoring the pavement surface are higher in these two cases than the normal crack sealing treatment [Ponniiah and Kennepohl, 1994].

The Canadian road network consists of 879,530 km of 2-lane highways, of which only 32% are paved [RTAC, 1990]. The growth of the national highway system is stable with a rate less than 0.4% per year. It is estimated that 18% of all paved highways do not provide a minimum operating speed of 90 km/h, and that 16% do not meet a riding comfort index (RCI) of 6.0 or an equivalent on similar measuring systems. Estimated road construction and maintenance expenditures in Canada are presented in Table 1.1. The expenditures are divided between *roads: rehabilitation and maintenance*, and *structures: culverts and bridges*. As shown in the table, the annual needs of the road system alone exceeds \$10.9 billion, and the actual expenditures are 26.2% lower than the needs. Advances in managing

the road network and delaying or eliminating a certain pavement distress will be translated into millions of dollars of savings to the tax payers.

In concluding this section, selected recommendations of researchers representing three Canadian Provinces; Alberta, Quebec and Ontario are quoted [Gavin et al., 1996]:

1. Basic performance criteria are still required to allow the pavement designer to predict cracking frequency and loss in pavement serviceability for controlling and eliminating low-temperature distress.
2. Basic understanding and design concepts for establishing performance specifications for cold climates are necessary, especially in view of new materials, such as modified asphalts.
3. The potential is there to control or even eliminate much of the multi-million dollar losses due to the low-temperature pavement distress problem in Canada.

From this set of recommendations, it is obvious that there is an urgent need for developing the area of thermal cracking prediction and control. The progress in this research area is critical to the maintenance and cost-effectiveness of the national highway system.

1.4 Problem Formulation

The response of pavement structures to various loading and environmental conditions has attracted a lot of attention in the past thirty years. Pavement distress affects the economy in two main areas; the increase in user costs because of deteriorating pavement conditions, and the high maintenance and untimely costs associated with restoring the pavement

surface. Research in the area of prediction models for pavement response and distress is under constant development. Failure to develop mechanistic models that truly describe the elements of material behavior combined with many restrictions on the applicability of purely statistical models has left but one choice; that is to continue on developing the so-called mechanistic-empirical models. The diagram shown in Figure (1.1) illustrates the three main distress types and the reported performance prediction methods for handling each type. As depicted in the figure, performance prediction methods for thermal cracking fall into two categories; low-temperature cracking and thermal fatigue cracking. Most of the Canadian highways network is subjected to extreme weather conditions combined with low traffic volumes particularly in rural areas. Thermal transverse cracking is the dominant distress mode in these conditions.

The Strategic Highway Research Program, SHRP, has promoted the notion of performance based specifications. The new specifications attempt to meet certain performance criteria rather than maintaining superior fundamental material properties. The program proposed new tests that are correlated to the desired level of performance. In such a case, it became important to continually provide comprehensive non-biased assessments of the proposed procedures and update the performance prediction models as being the sole measure for acceptance or rejection. The current models for thermal cracking give some insight to the problem, but often fall short in one way or another to provide a comprehensive and detailed analysis.

1.5 Scope and Objectives

Research in the field of pavement engineering presents some unique difficulties and challenges. The advances in computing and laboratory research may open new areas in this field. Figure (1.2) illustrates the relationship between the cost of research and expected and achievable knowledge in the field of pavement research. It is shown that computer simulation and basic engineering judgement may offer desirable results at a lower cost. Higher reliability can be achieved through pavement performance studies. Shaded areas in Figure (1.2) represent the research tools used in this study. Analytical methods and laboratory tests are within the low to medium costs and offer a medium level of reliability. The research also benefited from the timely release of the Canadian Long Term Pavement Performance database, C-LTPP. The database documents a national pavement performance study. While the high costs of the study are borne by the provincial highway agencies, the database is made available to the public to help advance the research into pavement rehabilitation. The database was first released in February 1996, while annual monitoring data will continue to be collected until the year 2004.

The scope of this study is limited to the thermal cracking problem of asphalt concrete surface layers or overlays and the objectives of the research are:

1. To review the state-of-the-art in pavement thermal cracking prediction and analysis.
2. To classify the types of transverse thermal cracks based on cause, location, geometry or rate of growth.
3. To provide guidelines for assessing the severity and propagation rates of various crack patterns under a given temperature regime.

4. To evaluate the interaction between multiple cracks. The behaviour of crack arrays is critical in crack routing and sealing processes.
5. To establish a testing methodology suitable for determining input parameters specific to thermal cracking numerical analysis, and to demonstrate the effects of several test variables on pavement resistance to cracking.
6. To formulate a reliability-based thermal cracking prediction model that accounts for the variability of input parameters and the correlation between component failure modes, and to quantify the contributions of the two thermal cracking modes.
7. To evaluate the thermal cracking model using the database collected by the Canadian Long Term Pavement Performance program, C-LTPP.

1.6 Research Plan

The purpose of this section is to briefly present the main components included in this study, Figure (1.3). During the planning of this research, it appeared that an analytical study based on a numerical finite element or boundary element analysis can be used to investigate the problem of thermal cracking. However, it became evident that the results of this study are only as reliable as the assumptions they are based on. The variability in pavement material properties and field climatic conditions have to be taken into consideration due to the the nature of the size and location of pavement problems. The study looked into these factors through the modification of an existing performance prediction model to account for variability and interaction between failure modes. The analytical study and

the reliability based model required a large number of input parameters, a typical feature of all similar investigations. Unlike traffic loads and distress condition surveys, most of the specific input parameters to thermal cracking problems are not routinely collected by pavement management agencies, for example; tensile properties of asphalt concretes and aging effects.

To complement the analytical study and the prediction model, an experimental program was carried out. The testing program was designed to provide input values for some of the parameters in the analytical study. In general, the analytical procedure and reliability-based model are easy to follow and have direct practical applications in pavement analysis. The adopted test methods are common laboratory tests and the existing facilities can accommodate the tests without excessive modifications.

1.6.1 Finite Element Analysis

The finite element analysis is used to determine the stress intensity factor; which indicates the crack growth rate. The pavement structure can consist of several layers. Each layer is characterized by thermal properties, tensile properties, and thickness. The applied load is computed from the effect of changes to the ambient air temperature and the corresponding ground thermal regime.

A temperature drop would always place the pavement structure under tension. Although it is numerically inaccurate, a reasonable tensile stiffness is assumed for each layer including unbound layers. The tensile resistance of unbound layers is caused by friction and surcharge effects, and can be represented by a tensile stiffness modulus as long as the tensile resistance

is sufficiently low. It is also known that pavement layers are not fully bonded and a relative slip can develop, however this effect is rarely quantified. In practice, the upper and lower bound of the bond can be supplied to the numerical procedure and the range of the expected crack propagation rates can be obtained. It should be noted that the employed finite element technique is still qualitative at best. The connection to real life situations is established by studying cracks of average representative properties and applying reasonable air temperature cooling rates.

1.6.2 Testing Program

A comprehensive testing program is carried out in this research. Its main aim is to simulate the field thermal conditions leading to cracking in controlled laboratory environment. The two investigated material properties are: a) the tensile strength which is affected by the rate of loading and temperature, and b) the propagation of damage due to cracking with repeated tensile strains from thermal cycling. In an accelerated test environment, it is not practical to reproduce thermal cycling because each loading cycle will require a minimum of two hours to attain isothermal conditions across the tested sample. The lengthy duration of each test negates all practical benefits from an accelerated test method. In addition, the temperature limits and the cooling and warming rates are sensitive parameters that require extreme control. Application of thermal cycling test results is not clearly defined in the literature. In this research, a process based on mechanical cycling at isothermal conditions is adopted. Three test methods are investigated; direct tensile test, tensile fatigue test and a repeated constant rate of extension test. In all tests, the samples consisted of field extracted

slabs of relatively large size and loaded in the horizontal direction. Two different mixes are used in the testing program.

The test results can serve as input parameters to the finite element program such as tensile stiffness, limiting strain, and tensile strength. They can also provide data to replace binder aging models in the reliability-based prediction model.

1.6.3 Mechanistic Thermal Cracking Prediction Models

Despite their wide recognition, thermal cracking prediction models require massive information about the pavement material and environmental conditions that hinders the applicability of these models. A reliability-based model for pavement thermal cracking will be developed in this research to complement similar models developed for rutting and fatigue [Chua et al., 1992].

The prediction model TCRACK [Shalaby and Easa, 1995a, Easa et al., 1996], is founded on an earlier version, known as TC1 [Shahin and McCullough, 1972]. TCRACK reports several enhancements over the earlier version. The model results are evaluated using Monte Carlo simulation and the database of the Canadian Long Term Pavement Performance, C-LTPP.

1.7 Thesis Organization

The thesis is divided into eight chapters and is organized as follows:

Chapter 1: introduction to the research topic, its impact and significance, and the research tools and methodology.

Chapter 2: a literature review that covers pavement cracking mechanisms, structural response, test methods and procedures, performance prediction models, types of models and analysis methods for pavement cracking problems.

Chapter 3: presents a proposed classification of thermal cracks, pavement thermal analysis, and applications of fracture mechanics.

Chapter 4: finite element analysis and fracture analysis of different crack types. The finite element analysis is divided into thermal, stress and fracture components. According to the problem geometry, a two- or three-dimensional mesh may be selected. Results are obtained in terms of fracture parameters; stress intensity factor or J -integral method.

Chapter 5: description of the testing program, test procedures and equipment. Selected trial mixes and sample preparation are presented.

Chapter 6: results of testing program and analysis. Three test methods are considered: direct tensile test, tensile fatigue test, and a repeated constant rate of strain test. The tests are performed at isothermal conditions using three temperatures: +20, 0.0 and -20° C.

Chapter 7: formulation of the reliability-based thermal cracking prediction model, TCRACK . The model determines the probability of cracking for two failure modes and the interaction between them. The model is verified using full distribution Monte Carlo analysis. The application of the thermal cracking prediction model TCRACK to C-SHRP field data is presented and the predicted and observed thermal cracking extents are compared.

Chapter 8: summary of research, conclusions, and recommendations for future research.

Appendix A: listings of 2-D and 3-D finite element input files for ANSYS

Appendix B: data sheets of asphalt mix design for mixes A and B that were used in the testing program.

Table 1.1: Needs versus expenditures of the highway system in Canada in the fiscal year 1988–89 (RTAC 1990).

Jurisdiction	Needs			Actual Expenditures
	Roads	Structures	Total	
Federal	\$185M	\$19M	\$204M	\$145M
Provincial	\$5,627M	\$280M	\$5,907M	\$3,997M
Municipal	\$5,160M	\$150M	\$5,310M	\$4,289M
Total	\$10,972M	\$449M	\$11,421M	\$8,431M

Figure 1.1: Main types of distress and performance prediction methods.

Figure 1.2: Cost and achievable knowledge relationship, after (Hugo et al., 1991), with the areas of interest to the present study being shaded

Figure 1.3: Framework of research plan and anticipated results

Chapter 2

Literature Review

2.1 Introduction

Initiation and propagation of all types of cracks in pavements are attributed to a large number of parameters. These parameters can be categorized as follows:

1. Mix design–related parameters: including pavement thickness, aggregate type and gradation, moisture content, voids in mineral aggregates, binder characteristics and rheological properties, and properties of the subgrade.
2. Construction related parameters: damage caused by compaction, construction joints, curing period, ambient temperature at construction, and use of prime and tack coats.
3. Service conditions: transient and steady state thermal stresses, pattern and magnitude of loads, number of repetitions and the maximum axle load.
4. Environmental conditions: thermal cycling, frost heave, drainage and aging.
5. Reflective cracks from seasonal deformations of cracked subsurface pavement layers.

While these general factors influence and govern all types of cracks, the research presented in this thesis addresses the thermally-induced cracks only. The conditions under which a low temperature crack may occur are summarized as follows [Haas et al., 1987]:

1. Excessive thermal stresses or strains, with and without traffic loads.
2. Freezing shrinkage of the underlayers.
3. Differential frost heave.

2.2 Factors Affecting Thermal Cracking

There are numerous factors that are believed to cause thermal-induced transverse cracking. The factors can be summarized using the following general categories [Vinson et al., 1989].

2.2.1 Material Factors

1. Binder grade: stiffer binders result in higher occurrence of cracking.
2. Aggregate type and gradation: desirable material properties are emphasized by the resistance to tensile stresses, high abrasion resistance and low absorption.
3. Asphalt cement content: increasing the content of asphalt cement has the adverse effect of increasing the thermal contraction of the mix while also lowering the stiffness. The overall effect of reasonable changes of this factor were found to be insignificant.
4. Air voids content: the degree of compaction, air voids and permeability are not directly related to low temperature performance, [Vinson et al., 1986]. However, voids ratio is influential in cyclic loading, and consequently thermal fatigue as the reversal of stresses breaks the bond between the binder and aggregate, resulting in relative displacements in the matrix.

2.2.2 Environmental Factors

1. Temperature: lower temperatures result in higher stresses and stiffening of the material which lead to thermal cracking.
2. Temperature gradient: the difference of top and bottom temperature in a pavement layer can cause curling stresses and increase the tensile stresses leading to a greater incidence of thermal cracking. The gradient is highest in the fall and spring and is highly affected by the rate of cooling, wind speed and solar radiation.
3. Age: older pavements have decaying binder ductility and stiffness and increased hardening which leads to higher cracking incidences.
4. Moisture damage: repeated freeze and thaw cycles and the use of de-icing agents cause accelerated pavement damage and faster crack propagation.

2.2.3 Structural Factors

1. Construction: construction-induced cracks are caused by compaction with steel rollers and act as a starting point for propagating transverse cracks.
2. Pavement width: typical initial crack spacing in roads having 8 m width is about 30 m, while in airports having 16-32 m pavement width the initial spacing is 45 m.
3. Pavement thickness: thicker asphalt pavement has an insulating effect that lowers the temperature gradients. The thicker sections have higher resistance to curling stresses.
4. Bond between layers: increasing the bond between layers using a tack coat helps in reducing thermal cracking.

5. Subgrade type: generally sand subgrade results in a higher incidence of thermal cracking than cohesive subgrade.
6. Present crack spacing: existing cracks tend to relieve the build-up of thermal stresses on the pavement and may prevent future occurrences of cracking. Repair of existing cracks with a stiff sealant may result in opening of new cracks to relax the tensile stresses.

2.3 Mechanisms of Transverse Cracking

There are two theories that explain how thermal cracking occurs [Janoo, 1993]. The first is the “*low-temperature cracking theory*” which assumes that cracking occurs when thermally induced stresses exceed the tensile strength of the pavement. Cracking may take place in a relatively new pavement, and the temperature at which pavement fails is defined as the fracture temperature.

The second theory is the “*thermal-fatigue theory*” which proposes that cracking is caused by temperature cycling between the day-time high and the night-time low temperatures [Shahin and McCullough, 1972]. It is important to note that both theories do not deal with the crack initiation process or the amount of damage in the pavement, except that caused by fatigue or rutting, prior to thermal cracking. The thermal cracking is a distress that takes place as soon as conditions permit.

It has been reported that transverse cracks initiate randomly in the pavement at the edge or center of the lane, and then propagate across the whole width of the asphalt concrete layer [Fromm and Phang, 1971]. Crack spacing decreases from about 30 m at the early stages, to a 3 to 6 m in aged pavements [Haas and Topper, 1969]. In the following sections,

a detailed review of the parameters, models and analytical methods pertaining to thermal cracking is presented.

The conventional approach to cracking mechanisms has been that cracks occur during warming periods that follow cold weathers as the warmer temperatures tend to soften the highly stressed pavement layers. Cracking occurs due to the rapid decrease in the tensile strength of the material, while the restraint of the cold base course to the surface layer would not permit the relaxation of thermal stresses. However, this mechanism could not be replicated in controlled laboratory conditions, where it was shown that relaxation of thermal stresses during warming periods would always prevent the cracking.

2.3.1 Haas-Topper Models

There are two possible mechanisms for cracking [Haas and Topper, 1969]. Both mechanisms assume that a limited thermally induced crack exists at the pavement surface during cold temperatures. Mechanism A, shown in Figure (2.1), theorizes that crack propagation is driven by stress imbalances created by warming at the surface while the rest of the layer is still relatively cool. Cracks would appear visibly opened, and this condition continues until full depth cracking of the layer. When warming of the whole layer occurs, in late spring or summer, cracks tend to close due to expansion.

Mechanism B, shown in Figure (2.2), theorizes that crack propagation is driven by several cycles of night cooling and daytime warming, most probably in regions of relatively moderate climates than those in mechanism A, or during off-winter seasons. Similar to mechanism A, full depth cracks appear visibly open when the layer is cool, and tend to

close when full depth warming occurs. The absence of a crack initiation model was noted by Abd El Halim et al. (1994) and led to the development of a three stage model.

2.3.2 Three-Stage Thermal Cracking Model

A three stage mechanism of thermal cracking is suggested [Abd El Halim et al., 1994]. The model is explained as follows:

Stage 1: Crack initiation

Cracking initiates with construction-induced micro cracks. These cracks are created due to the mismatch in relative rigidities between the roller and the hot asphalt mix. The cracks are formed at high temperatures, 140°C or more, and have a random distribution on the pavement surface.

Stage 2: Crack propagation during the first and second winters

Thermal stresses build-up at the tips of the hairline cracks created in Stage 1, causes a portion of those cracks to propagate and connect leading to macro cracks that may extend to the full depth of the asphalt layer. The macro cracks developed at this stage represent the first set of transverse cracks to appear on the pavement surface.

Stage 3: Evolution of transverse cracks

After the first set of cracks are formed in Stage 2, the pavement can be considered as a set of long slabs separated by transverse cracks at almost uniform spacings. When warming starts at the beginning of the spring, some of the construction micro-cracks closer to the ends of shorter slabs tend to open and those away from the ends tend to close. The overall movement at the cracks will prevent the macro-cracks at the ends of the slabs from closing

completely. At the return of cold temperatures, a portion of the micro-cracks will propagate and form new transverse macro-cracks.

The absence of longitudinal cracks is explained by the effect of current compaction techniques. Abd El Halim (1985,1986) demonstrated that the construction micro-cracks are mainly transverse cracks, because of the nature of the line loading applied by conventional compaction equipment, and the lateral continuous restraint imposed by this loading.

2.4 Rheological Properties of Asphalt Concrete

Extensive research has been performed to determine the rheological properties of asphalt concrete. For a comprehensive study on stress analysis of pavement structures, the time and temperature dependence of the stress-strain relationship on the considered type of loading, has to be determined.

Models consisting of arrays of springs and dash pots can be used to simplify the behavior of asphalt concrete [Monismith et al., 1965]. Previous attempts using three, four and five element models were found inadequate, Figure (2.3). Better representations can be achieved by adding more elements, or even an infinite number of elements, but that leads to a higher complexity of the mathematical manipulations. While models are constructed to simulate the actual behavior of asphalt concrete, their performance is not in fact related to the material physical properties. Therefore, the advantage of mathematical simplicity is nullified by the inadequacy to represent the true behavior.

2.5 Structural Response of Flexible Pavements

Stress analysis of flexible pavements has been performed mainly according to two theories [Ullidtz, 1992]. First, the *Elastic Layer Theory* which is based on the following assumptions:

1. static equilibrium of stresses,
2. compatibility of deformations, and
3. materials are homogeneous and obey Hooke's Law of elasticity.

Applying those assumptions, together with proper representation of load cases and boundary conditions, stresses, strains and deflections are obtained. However, the validity of the above assumptions is questionable. In most cases, structural discontinuities are inevitable and compatibility of deformations cannot be established when the pavement layers are not fully bonded. Moreover, deformations in asphalt concrete are not only elastic but also plastic, viscous and viscoelastic. Paving materials are non-linear and anisotropic. Although, the inaccuracies of some of those assumptions have been treated over the past 30 years, and numerical methods such as the finite element method were introduced to obtain better correlations, nevertheless, data obtained from such an analysis can only be used in a relative sense when compared to actual pavement response.

The second method is the *Multi-Layer Theory*. It is based on *Boussinesq's* equation for computing deflections in elastic media, combined with *Odemark's* assumption that stresses, strains and deflections under a certain layer are only dependent on its stiffness as given by,

$$S_i = \frac{EI}{1 - \nu} \quad (2.1)$$

where,

S_i = stiffness of layer i ,

EI = layer's modulus of elasticity and moment of inertia, and

ν = Poisson's ratio.

The *Elastic Layer Theory* has a broader scope of applications, while the *Multi-Layer Theory* is easier to use and can directly accommodate material non-linearities. Both methods yield comparable results, but unfortunately are not always representative of the true behavior.

Initiation and propagation of thermally-induced cracks is controlled by two types of loading: (1) pseudo-static low temperature tensile stresses; and (2) thermal-fatigue. The first type is manifested by the development of thermal stresses in a restrained pavement section that are in excess of the tensile strength of the material. Catastrophic crack growth may occur during a single cooling cycle and is not affected by the plasticity zone which is formed ahead of the crack tip. Analysis methods for this type include: elastic, elastic-plastic and visco-elastic material models. Numerical methods such as the finite element method have been successfully used to implement those models. The second method, thermal-fatigue, is concerned with crack growth due to repeated thermal cycling. Analysis methods for thermal fatigue include: the fatigue law, Paris' law, blunt crack band theory and crack layer theory. In the next two sections, a detailed review of analytical methods is presented.

2.6 Pseudo-Static Thermal Stress Analysis

The calculation of thermal stresses in pavements has been studied extensively in the past thirty years. One approximate method is to model the pavement as a *pseudo-elastic* beam

subject to discrete temperature intervals. Only the average temperature along the pavement depth is required in the analysis. Alternatively, in a more advanced model, the pavement is characterized as a *linearly viscoelastic* material, and subjected to a time-dependent three dimensional temperature field.

2.6.1 Pseudo-elastic Beam Method

A simple beam analogy considered the pavement surface as an elastic infinitely long beam [Lamb, 1966]. Stresses in the longitudinal direction are computed as,

$$\sigma_x(T) = E\alpha (\Delta T) + \nu fL \quad (2.2)$$

where,

E = modulus of elasticity,

α = average coefficient of thermal contraction (expansion) in the temperature range ΔT ,

T and ΔT = the absolute and incremental temperatures,

ν = Poisson's ratio,

f = coefficient of friction between the asphalt concrete and the base, and

L = one half the width of the pavement section.

This method assumed the coefficient of thermal contraction α to be independent of the temperature, and it does not account for the underlayers. More realistic results are obtained if E is substituted with a stiffness modulus $S(T, t)$, which varies with temperature and time of loading. $S(T, t)$ is evaluated experimentally or from nomographs, and utilized over the entire temperature range [Hills and Brien, 1966]. Lamb concluded that the effect of lateral

restraint is relatively small and can be ignored. Therefore, equation (2.2) is modified to

$$\sigma_x(\dot{T}) = \alpha \sum_{T=T_o}^{T_f} S(T, t) \Delta T \quad (2.3)$$

where

σ_x = the accumulated stress at a cooling rate $\dot{T}$,

α = the coefficient of thermal contraction,

T_o and T_f = the initial and final temperatures,

t = time of loading, and

$S(T, t)$ = stiffness modulus determined experimentally at the midpoint of each temperature interval ΔT .

This approximate analysis provides a one dimensional model of the problem, ignores the restraining action of the base, and cannot accommodate temperature gradients within the pavement thickness and the corresponding stiffness gradients.

2.6.2 Coupled Thermoelastic Stress Analysis

In cases where the interaction between temperature changes and mechanical properties cannot be neglected, a coupled analysis should be considered [Stuzalec, 1992, Hsu, 1986].

The equations of equilibrium are given by,

$$(\lambda + \mu) u_{i,li} + \mu u_{i,ll} - \beta T_{,i} = \rho \ddot{u}_i \quad (2.4)$$

which can be stated in a vector form

$$(\lambda + \mu) \text{grad div } \mathbf{u} + \mu \nabla^2 \mathbf{u} - \beta \text{grad } T = \rho \ddot{\mathbf{u}} \quad (2.5)$$

where

λ and μ = isothermal values of Lamé's constants,

ρ = the mass density,

β = the thermoelastic coefficient, and

u and T = the unknown displacement and temperature fields function of position x_i and time.

The general heat conduction equation can be written as

$$(k T_{,i})_{,i} - \rho c \dot{T} - (T - T_o)\beta \dot{\epsilon}_{ii} + Q = 0 \quad (2.6)$$

where

k = the coefficient of thermal conductivity,

c = the specific heat at constant pressure,

Q = the heat source term, and

T_o = the initial temperature distribution.

The strain tensor is given by

$$\epsilon_{ij} = \frac{1}{2}(u_{i,j} + u_{j,i}) \quad (2.7)$$

and the linear thermoelastic constitutive equation is

$$\sigma_{ij} = C_{ijkl} \epsilon_{kl} - \beta (T - T_o) \delta_{ij} \quad (2.8)$$

where δ_{ij} is the Kronecker's delta function. For isotropic elastic materials, the constants

C_{ijkl} can be defined in terms of Lamé's constants as

$$C_{ijkl} = \lambda \delta_{ij} \delta_{kl} + \mu \delta_{il} \delta_{jk} + \mu \delta_{ik} \delta_{jl} \quad (2.9)$$

In a static analysis the inertia term of equation (2.4) can be neglected. The thermoelastic coefficient β can be expressed in terms of the isotropic elastic constants, λ and μ , and the

coefficient of thermal expansion, α , as

$$\beta = (3\lambda + 2\mu) \alpha \quad (2.10)$$

Thermoelastic analysis is applied to investigate the propagation of cracks and the effect of overlays and reinforcement on a two dimensional pavement structure [Selvadurai et al., 1990].

2.6.3 Thermo-viscoelastic Stress Analysis

The analysis, originally developed by Humphreys and Martin in 1963, and later applied by Monismith et al. in 1965 and 1966 provides a more rigorous approach to thermal stresses in pavements. This analysis method is more applicable to a wider range of temperatures and loading conditions than previous simplified analyses. In this case, a semi-infinite medium is considered, the asphalt surface is stress-free and the thermal and mechanical loads are uncoupled. Thus, it is possible to determine the temperature distribution from the heat conduction equation only. It is first necessary to introduce the concept of time-temperature equivalence principle before proceeding to stress analysis.

Concept of Time-Temperature Equivalence

In asphalt concrete, the stress strain relationship is time dependent and temperature dependent. If an initial strain ϵ_o is applied, a corresponding stress σ will develop. The stress will then decay with time even if the strain remains constant. The relaxation modulus $R(t)$, defines the rate of stress dissipation with time;

$$R(t) = \frac{\sigma(t)}{\epsilon_o} \quad (2.11)$$

Similarly, if a constant initial stress is applied, the creep compliance $D(t)$ defines the rate of strain release with time.

$$D(t) = \frac{\epsilon(t)}{\sigma_o} \quad (2.12)$$

The resulting curves for relaxation modulus and creep compliance versus time are applicable only at the particular test temperature. Asphalt concrete can be modelled as a linear viscoelastic material, and can be approximated as a rheologically simple one. In rheologically simple materials, relaxation modulus or creep compliance curves at different temperatures are identical in shape. Relationships at different temperatures are superimposed using a reference temperature and a time shift function

$$\xi = t^{f(t_o)} \quad (2.13)$$

where

t and t_o = the absolute and the reference temperatures,

ξ = reduced time, and

$f()$ = time shift function.

Thermal Stresses in Pavement

The analysis is based on an infinite slab fully bonded to a rigid underlayer. The equations used in the analysis are [Monismith et al., 1965];

time-dependent strain tensor analogous to equation (2.7):

$$\epsilon_{ij}(x_k, t) = \frac{1}{2} \left[\left(\frac{\partial}{\partial x_j} \right) u_i(x_k, t) + \left(\frac{\partial}{\partial x_i} \right) u_j(x_k, t) \right] \quad (2.14)$$

equilibrium condition:

$$\left(\frac{\partial}{\partial x_j} \right) \sigma_{ij}(x_k, t) = 0 \quad (2.15)$$

stress strain relations:

$$s_{ij}(x_k, t) = \int_{-\infty}^t G_1(t - \tau) \left(\frac{\partial}{\partial \tau} \right) e_{ij}(x_k, \tau) d\tau \quad (2.16)$$

$$\sigma(x_k, t) = \int_{-\infty}^t G_2(t - \tau) \left(\frac{\partial}{\partial \tau} \right) [\epsilon(x_k, \tau) - 3\alpha\theta(x_k, \tau)] d\tau \quad (2.17)$$

where,

- $\sigma_{ij}, \epsilon_{ij}$ = stress and strain tensors,
- u_i, u_j = displacements,
- t and τ = absolute time and the time variable ,
- s_{ij} = deviatoric components of the stress tensor,
- G_1, G_2 = general relaxation moduli,
- e_{ij} = deviatoric components of the strain tensor,
- $\sigma = \sigma_{kk}$ = hydrostatic stress,
- $\epsilon = \epsilon_{kk}$ = hydrostatic strain,
- $\theta(x_k, t)$ = weighted temperature $\frac{1}{\alpha_o} \int_{T(x_k, t_o)}^{T(x_k, t)} \alpha(T) dT$, and
- α = coefficient of thermal contraction.

The thermal stresses are computed by applying the boundary and initial conditions

$$\sigma_{xx}(z, t) = -3\alpha_o \int_0^t R[\xi(z, t) - \xi(z, \tau)] \left(\frac{\partial \theta(z, \tau)}{\partial \tau} \right) d\tau \quad (2.18)$$

where,

$$\xi(x_k, t) = \text{reduced time} = \int_0^t e^{f[T(x_k, t)]} d\tau$$

$$R(s) = \text{modulus function} = \mathcal{L}^{-1} \frac{\mathcal{L}G_1(s) \cdot \mathcal{L}G_2(s)}{2\mathcal{L}G_1(s) + \mathcal{L}G_2(s)}; \text{ and}$$

$$\mathcal{L}G_1, \mathcal{L}G_2 = \text{Laplace transforms of } G_1, \text{ and } G_2.$$

It is reported that the stresses computed from this analysis are greater than the field measured stresses, and are unsuitable for use in the design [Monismith et al., 1965]. The excess of stress is caused by the assumptions made in the analysis. For example, the slab is infinitely long in both directions whereas the field conditions imply a finite lateral dimension, and the slab is considered fully bonded to the subgrade. Haas and Topper endorsed the necessity of an analysis for an infinite strip rather than an infinite slab [Haas and Topper, 1969].

In Canada, based on the Ste. Anne test road, researchers compared five analytical methods for computing thermal stresses in asphalt pavements [Christison et al., 1972]. Observed cracking in a number of test sections were compared to those predicted by the analysis. The methods were, pseudo-elastic beam, approximate pseudo-elastic beam; viscoelastic beam; viscoelastic slab; and approximate viscoelastic slab. The analysis of the pseudo-elastic beam, although based on many assumptions, showed reasonable correlation to field data.

Calculation of thermal stresses in restrained elastic plates is reported by Iyenger et al. (1968). They presented an expression for the stress field, equation (3.4), in terms of a Fourier series that satisfies the boundary conditions. The method can be employed in an iterative manner for small temperature decrements to utilize the varying stiffness modulus of the material.

2.7 Thermal Fatigue

Thermal fatigue is a relatively new and unexplored phenomenon. It evolved to explain serious thermal cracks which were observed in pavements that do not normally experience low temperatures but are subjected to large thermal variations between maximum and minimum

daily temperatures [Lytton et al., 1983, Ponniah and Kennepohl, 1994, Vinson et al., 1989].

There are two main approaches to analyze thermal fatigue; the phenomenological approach which is based on the fatigue law, and the mechanistic approach which is based on crack propagation laws obtained from fracture mechanics [AbdulShafi and Majidzadeh, 1985].

2.7.1 Phenomenological Approach

The phenomenological approach is based on the S-N law (fatigue law)

$$N_f = K_1 \epsilon_t^{-K_2} \quad \text{or} \quad N_f = K_1 \sigma_t^{-K_2} \quad (2.19)$$

where N_f is the allowable number of cycles to cause fatigue cracking under tensile strain $\epsilon_t (= \alpha \sum_i \Delta T)$ or tensile stress $\sigma_t (= \alpha \sum_i S(t, T) \Delta T)$, and K_1, K_2 are the fatigue coefficients to be determined experimentally. The strain (displacement or stroke) controlled fatigue is suitable for thin specimens of 50 mm depth or less. The stress controlled fatigue is suitable for thick and stiff specimens of 150 mm depth or more. Between these limits, a mixed mode prevails [AbdulShafi and Majidzadeh, 1985]. The accumulated damage, D , is determined from Miner's Hypothesis,

$$D = \sum_i \sum_j \frac{n_{ij}}{N_{ij}} \quad (2.20)$$

where

n_{ij} = the actual number of fatigue cycles at age i and load case j , and

N_{ij} = the expected failure number of cycles at age i and load case j .

This analysis does not distinguish between the number of cycles consumed in the crack initiation phase and the crack propagation phase. The process of determining the fatigue

constants K_1 and K_2 , which are not material constants, is costly since they have to be evaluated for the effect of each mixture additive, reinforcement, and similar improvements.

2.7.2 Mechanistic Approach

The mechanistic fracture mechanics approach is based on the investigation of crack growth from Paris law [Paris and Erdogan, 1963]. The fracture mechanics approach is more representative of the fatigue phenomenon. The fracture parameters are material related and are determined experimentally. The fracture parameters were found valid for a wide loading range. Several tests were developed to obtain the fracture parameters. The three point bend specimen test, Figure (2.4), is employed to compute the critical energy release rate J_{IC} and the tearing modulus [Little and Mahboub, 1985]. The C^* -line integral test, Figure (2.5), is used in determining the change in the energy release rate with the crack length, [AbdulShafi and Majidzadeh, 1985].

2.8 Failure Criteria

Failure is manifested by the opening of cracks and the creation of new cracks in the pavement surface. The following failure criteria were reported in the literature:

1. Tensile strength of the asphalt concrete. It is a measure of the resistance to tensile stresses caused by temperature changes during a cooling cycle [Haas, 1973]. Figure (2.6) demonstrates the effect of thermal cooling on the tensile strength and the thermal stresses.

2. **Fracture temperature.** It is the temperature at which the mixture fails in a thermal cooling cycle. It is determined in the lab using the thermal stress restrained specimen test [SHRP, 1994b]. Fracture temperature of a certain mixture depends upon the rate of cooling and specimen shape and dimensions, and consequently it is not a characteristic property of that mixture.
3. **Fracture toughness.** According to the established fracture mechanics approach to cracking, cracks will initiate and propagate only if the stress intensity factor at the crack tip exceeds the fracture toughness of the material. If fracture occurs, the crack growth rate and the stability of this growth are determined.
4. **Creep master curve.** The SuperPave mix design bases the failure on the creep behaviour. The material input to the model includes the master creep compliance curve (100–seconds or 1000–seconds), the m-value which is the slope of the linear portion of the log(compliance)–log(time) master curve, and mixture tensile strength. The environmental conditions include the daily maximum and minimum air temperatures, and the latitude of the pavement section. The structural characteristics include the pavement thickness and type of materials in the base and the subgrade. Material properties are obtained from the indirect tensile creep and failure test at low temperatures (ITLT) at 0 to -20°C . Failure is reached when the build up of stresses exceeds the mixture tensile strength.

The fracture temperature is determined from the thermal stress of restrained specimen test (TSRST) in the SHRP performance models [SHRP, 1994b]. Alternative methods for determining fracture temperature were listed as follows [Kasahara and Haas, 1982]:

1. Binder stiffness methods. Such as limiting asphalt stiffness, critical asphalt stress, and nomographic cracking temperature.
2. Mixture stiffness methods. Such as limiting mix stiffness, calculated mix cracking temperature, and direct pavement cracking observations.

There is no established set of parameters for each of these methods. For example, Table 2.1 summarizes the parameters used by various research groups for determining the fracture temperature by limiting the binder stiffness at the specified loading rates.

2.9 Laboratory Testing

Francken (1989) and Roque and Ruth (1990) presented and compared several numerical and experimental methods for thermal cracking mechanisms and modelling. Transverse cracks are categorized into (i) surface cracks propagating downwards by thermal cycling and (ii) reflection cracks initiated at the bottom of the surface layer and growing upwards due to movements at original cracks in older layers. Shalaby et al. (1995, 1996) proposed two additional types of laterally growing cracks: those developing at pavement edge and moving towards the centerline and those initiating in the middle of the lane and growing laterally towards the pavement edge or shoulders. For simplicity, combinations of the four types are not yet considered until rigorous analytical models are formulated for each crack type. A finite element procedure is developed to compute the propagation rate of each type of cracks, depending on the air temperature variations at the site, pavement material properties, and boundary conditions.

In a detailed literature review [Vinson et al., 1989], available test methods were evalu-

ated based on the following selection criteria:

1. simulation of field conditions,
2. application of test results to mechanistic models,
3. suitability for aging and moisture conditioning,
4. potential to accommodate large stone mixes,
5. ease of conduct, and
6. cost of equipment.

The test methods undergoing this evaluation included:

1. indirect diametral tension test,
2. direct tension test,
3. tensile creep test,
4. flexural bending test,
5. thermal stress in restrained specimen test, and
6. coefficient of thermal contraction test.

The review of test methods revealed that there are two basic approaches to perform low temperature tests. The first relies on load-deformation tests such as tests 1 to 4, and is fundamentally limited with the identification of a proper rate of loading that corresponds to field conditions. The second approach is based on developing thermal stresses directly, such as tests 5 and 6, and also is limited by identifying a proper rate of cooling that corresponds to field conditions and sample size. The second approach, specifically the TSRST, has been

widely implemented in the SHRP program for the studies on the validation of pavement performance. It should be noted that all the existing thermal cracking models do not make use of a performance test method. The purpose of these testing methods have been limited to evaluation of mixture properties and comparison of material and environmental parameters.

The direct tensile test was examined by The Asphalt Institute in search for a suitable routine test method [Bolzan and Huber, 1993]. The test setup, sample size and preparation are very similar to those of the TSRST except that a constant rate of tensile strain is applied instead of the thermal cooling. The repeatability of the test was poor due to misalignments, and failure often occurred at the interface of the sample and the epoxy glue. Because of these problems, the test was subsequently eliminated from consideration for performance based specifications for asphalt mixtures.

The thermal stress in a restrained specimen test, TSRST, was the closest to satisfying the selection criteria and is considered the most related laboratory test to thermal cracking according to the study, and is supported by the Strategic Highway Research Program SHRP [Kanerva et al., 1994]. In the TSRST, the prismatic sample of dimensions 50 x 50 x 250 mm is restrained by a steel rig and the temperature is gradually decreased from 0°C at a rate of 1 to 10°C/hr, while any shrinkage in the sample length is restored using a high precision hydraulic system. The test is continued until failure and the fracture temperature is recorded. The asphalt mixture is approved for use in sites where the lowest winter temperature is higher than fracture temperature with a specified margin of safety. The TSRST approach, in its present format, cannot account for thermal cycling, and the thermal gradients inside the sample cannot be predicted or replicated. The sample size and preparation pose serious concerns whether the test method can be used to investigate crack propagation or can be

applied to reinforced pavements.

The SHRP program has adopted the thermal stress in a restrained specimen test, TSRST, to represent the true behavior of an asphalt mixture under thermal stresses. But the simulation of the field conditions of asphalt pavements under low temperature requires the use of a rigid apparatus to restrain or compensate for the thermal deformations in the specimen and the supporting structures. It also requires the use of advanced monitoring equipment to measure the shrinkage and stresses in the sample. For example, the following mechanisms have been employed to restrain the specimen during the TSRST testing [Haas, 1973, Janoo, 1993], Figure (2.7):

1. A frame constructed from two materials having different coefficients of thermal contraction, such that the thermal contraction of the frame does not impose any strains on the specimen.
2. Mounting the specimen in a frame made of a temperature insensitive material, such as Invar, having a coefficient of thermal contraction of $9 \times 10^{-7}/^{\circ}\text{C}$, and a high modulus of elasticity (148 GPa).
3. Installing the specimen in a Plexiglas chamber with a cooling bath and using a suitable cooling fluid (e.g. liquid Nitrogen) and a testing machine kept at room temperature.
4. Attaching the specimen to a step motor and a microprocessor that adjusts the length during the test.

2.10 Thermal Cracking Prediction Models

There are four types of pavement cracking prediction models as described in *Modern Pavement Management* [Haas et al., 1994]

1. Purely mechanistic models which have not yet been developed, at this time, due to the complexity and the uncertainties involved.
2. Mechanistic-empirical models in which the mechanistic behavior or response is related to the predicted parameters by regression functions or a calibration procedure. There are numerous models that fall into this category and they exhibit promising results and possibility of development and extension.
3. Regression models which are only feasible if a large database is available. The results of these models cannot be extended or generalized to apply to conditions other than those used for constructing the database.
4. Subjective models; based on the experience and insight of local engineers and in all cases only apply to local conditions.

Two mechanistic-empirical thermal cracking models examine the mixture cracking potential, COLD [Finn et al., 1986], CRACK3 [Roque and Ruth, 1990]. Predictive models such as THERM [Lytton et al., 1983] and SHRP models [Hiltunen and Roque, 1994] are based on fracture mechanics principles and can predict the amount of cracking as a function of time. As such, all the models determine a cracking probability on the pavement section. The translation of the cracking probability to a specific length of cracks and average crack length or spacing is largely left to the user and the specifics of each location.

2.10.1 Mechanistic-Empirical Models

Mechanistic models have been proposed to relate material properties and environmental conditions to the observed field performance. The following briefly describes four models which investigate pavement thermal cracking. Another two models, the SHRP model and TCRACK , will be discussed in more details later in this chapter:

1. **COLD Model** [Finn et al., 1986], is based on the stress-strength relationship first formulated by Hills and Brien in 1966. The failure condition is associated with a fracture temperature which when exceeded, cracking and pavement damage are expected. The model calculates the temperature distribution within the asphalt layers and its daily variation. The model relies on the relationship between pavement tensile strength and temperature at a slow rate of loading of 0.01 in/min (0.0004 mm/s). The tensile strength is determined from direct and indirect tension tests.
2. **Ruth Model** [Ruth et al., 1982], combines results of indirect tensile strength tests, low temperature viscosity from the Schewyer Rheometer, and indirect creep test to develop an equation for predicting creep deformation. Cracking criterion is that the tensile stresses exceed the ability of the material to relax the creep strains. The model also gives a correction for creep of the mixture under thermal stresses.
3. **THERM Model** [Lytton et al., 1983], presents a fracture mechanics approach to thermal fatigue cracking. The model requires data from daily temperatures, solar radiation, wind speed and air temperature to find temperature distribution from a modified Barber equation [Lytton et al., 1983]. Thermal fatigue is defined as the fatigue caused by temperature cycling below 25°C (75°F). The stiffness of the asphalt mix is determined

from a van der Poel nomograph and mixture characteristic values. The model provides options for aging of the top layer only or the entire pavement structure. The damage caused by each cycle is accumulated using Miner's hypothesis.

4. **TC1** [Shahin and McCullough, 1972], successfully combined effects of low-temperature and thermal fatigue cracking using a probabilistic approach. The pavement temperature is determined from Barber equation using site average conditions. The model accounts for aging of asphalt properties. Binder stiffness is determined from van der Poel nomograph incorporating aged properties. The mixture tensile strength is determined by physical testing and utilizing Heukelom's findings [Heukelom, 1966]. Probabilistic methods are used to predict the amount of cracking when thermal stress exceeds mixture strength. The thermal fatigue component uses phenomenological fatigue law and Miner's hypothesis to accumulate the damage due to thermal cycling.

2.10.2 Regression Models

Regression models of the cracking frequency provide a powerful tool to correlate maintenance expenditures of transportation agencies to the degree of cracking. For example, one mathematical model for predicting the frequency of thermal cracks was developed in the early 1970s [Haas and Hajek, 1972]. Based on field data obtained from the Ste. Anne test road, as well as data collected by the Ontario Ministry of Transportation, the following model was presented;

$$10^I = 2.497 \times 10^{30} S^{6.7966-0.8740t+1.3388a} \times (7.054 \times 10^{-3})^d (3.193 \times 10^{-13})^m d^{0.6026S} \quad (2.21)$$

where

S = stiffness of the original asphalt for a loading time of 20000 seconds at the winter design temperature,

I = cracking index,

t = overall thickness of the asphalt layers,

a = age of asphalt concrete in years,

d = a code that includes the effect of the subgrade, and

m = winter design temperature.

The model shows the positive correlation between cracking and temperature, stiffness and age. Also a negative correlation exists between cracking and pavement depth. The model limitations, as specified by the authors, included the lack of a more accurate classification of the subgrade and the assumption that all the observed field cracks are essentially low temperature cracks.

Fromm and Phang (1972) presented six mathematical models to predict the cracking index in Ontario. They used step-wise regression and linear multiple regression analyses. Models were developed for the province of Ontario as a whole, the northern region where the penetration index is higher than 110 (150/200 penetration grade), and the southern region where the penetration index is lower than 110 (85/100 penetration grade).

These models emphasized the effect of the freezing index, pavement voids, viscosity ratio, asphaltenes and the critical temperature. Contrary to their expectations, pavement age and thickness showed weak correlation to the cracking index in the studied sections. The authors stressed the significance of these parameters and concluded that statistical

procedures may have been the cause of this confusing result.

Haas et al. (1987) proposed a model that predicts the intensity of transverse cracking, based on data obtained from 26 airfields in Canada. The regression equation of the model is

$$\text{Crack Spacing} = 218 + 1.28 \text{ ACTH} + 2.52 \text{ MTEMP} + 30 \text{ PVN} - 60 \text{ COFX} \quad (2.22)$$

where,

MTEMP = minimum temperature recorded on site in °C,

PVN = pen-vis dimensionless number of McLeod,

COFX = coefficient of thermal contraction in mm/1000 mm/°C,

ACTH = thickness of the asphalt layer in centimeters,

Crack Spacing = average spacing between transverse cracks in metres.

The 26 airfields were located below 50° N latitude, fifteen were coastal airports and half the observations were made on overlays. Properties of extracted asphalt cements were used to develop the regression model.

2.10.3 SuperPave Model

The SHRP program was initiated in 1987, and resulted in the collaboration of many Highway agencies in North America, Europe, and Australia. The program mandate was to improve the durability and performance of highways. The subtask involving low temperature cracking has proposed laboratory test measures in the two following categories

1. Binder Selection:

- Bending beam rheometer test (BBR), to measure the flexural stiffness of the binder.
- direct tensile test, to measure the tensile strength of the binder.

2. Mixture Performance:

- Indirect tensile creep and failure test.
- Thermal stress in a restrained specimen test (TSRST).

Table 2.2 lists the selected tests for binders and mixtures and the measured properties in each test. Also, SHRP initiated a long term pavement performance monitoring program, LTPP. The Canadian counterpart C-LTPP is responsible for monitoring and collecting information on 65 test sections across Canada.

The two-part thermal cracking model in SuperPave consists of a mechanics-based part and a probabilistic part. The former employs creep compliance curves with site-specific weather data to calculate the progression of vertical cracks at average site conditions, and the latter performs level I reliability analysis, to predict the intensity of cracking based on the current average crack depth and the assumed distribution of crack depths in the asphalt layer [Hiltunen and Roque, 1994].

2.10.4 Reliability-Based Models

Due to the nature and scale of pavement projects, the uncertainty in material properties and loading conditions must be considered. The selection of sampling parameters and control of these parameters that accurately reflect the pavement conditions and performance can be achieved using a sensitivity analysis.

Analytical models which predict the amount of cracking resulting from exposure to

cold climates have been developed, including [Hajek and Haas, 1972], [Finn et al., 1986], [Ruth et al., 1982], [Lytton et al., 1983], and [Shahin and McCullough, 1972]. The latter two are the most comprehensive models which examine both low-temperature and thermal-fatigue cracking [Huang, 1993]. The model developed by Lytton et al. is more theoretical and relies on viscoelastic fracture mechanics and crack propagation processes. The model by Shahin and McCullough is more phenomenological, easier to understand, and is based on the nominal stresses and strains in the uncracked pavement. Since pavement properties and loading conditions are probabilistic variables in time and space domains, a prediction model must consider the variation of those parameters from their established means. The method known as Mean Value First Order Second Moment (MVFOSM) is popular in practice because of its simplicity [Cornell, 1969], [Ditlevesen, 1981]. This method aims at obtaining the mean, μ_Z , and standard deviation, σ_Z , of the performance function $Z = f(X_1, X_2, \dots, X_n)$. The resulting factor of safety, μ_Z/σ_Z , is a measure of the confidence in the system and is indicative of the uncracked pavement area.

It has been found that level I reliability methods are biased, and that some parameters may have higher inaccurate influence on the results than others [Augusti et al., 1984, Madsen et al., 1986, Chua et al., 1992]. Different performance functions that employ the same parameters may yield substantially variable results. Such irregularities led to the development of invariant second moment methods, or Level II methods [Hasofer and Lind, 1974]. The failure surface is approximated by a tangent hyper-plane at the design point, which corresponds to the point on the failure surface closest to the origin in a normalized variable space. The order of the approximation plane determines the accuracy of the results. First Order Reliability Method (FORM) approximates the failure surface to a tangent plane at the

failure point, while Second Order Reliability Method (SORM) constructs a second degree surface at the design point that has the same curvatures of the failure surface at that point.

The method has been applied to general pavement design system that includes two distress modes, traffic fatigue and rutting [Chua et al., 1992]. The model accounts for the variations in design, materials, construction, and operating conditions, and predicts the probability of failure for each distress mode and the system probability of failure using FORM, SORM and Monte Carlo simulation. The research concluded that the failure surface is not highly nonlinear and the approximation by a plane is acceptable for design applications. As shown in Chapter 7, a reliability-based thermal cracking model is developed in line with the presented traffic fatigue and rutting models.

2.11 Summary

A review of thermal cracking in the literature has been presented. The problem of thermal cracking poses serious damage to the quality of highway pavements with annual economic costs in the multi-billion dollar range. While there are several models that predict thermal cracking potential, there is a need to improve the predictive abilities of these models and extend their scope. There is agreement among researchers that these improvements can be realized using mechanistic-empirical models [Haas et al., 1994]. Also, there is demand for establishing numerical methods for analysis and design of thermally loaded pavements. Finite element solution to thermal cracking problems are feasible, but the material behaviour in tension is complex and more accurate information on the temperature regimes and cooling rates are essential to the success of these solutions. As shown in the literature, the present

laboratory test methods, TSRST and ITLT, do not examine the problem of thermal fatigue or explore repeated tensile loading conditions. The present test methods also fail to consider effects of pavement reinforcement and the bond between layers.

The presented research in this thesis addresses some of these problems and sets practical examples whenever possible. The findings of this research can be implemented in the areas of pavement low temperature regime, numerical analysis of pavement structures, thermal crack initiation and propagation, asphalt fracture, thermal cracking performance prediction models, reliability of performance prediction models, low temperature testing and monitoring, and validation methods using field data.

Table 2.1: Limiting binder stiffness and the corresponding loading time

Reference	Binder Stiffness MPa	Loading Time Sec.
[Haas, 1973]	140	10080
[Robertson, 1987]	0.5	nomograph
[Readshaw, 1972], [May and Witczak, 1992]	200	7200
[Gaw, 1978], [Kandhal, 1978], [Deme and Young, 1987]	1000	1800

Table 2.2: SHRP binder and mixture tests

Test	Specimen Dimensions (mm)	Temperature Range °C	Measured Properties
<u>Binder tests:</u>			
Bending beam rheometer	127×12.7×6.3	−40 to 25	flexural stiffness
Direct tensile	dog bone	−30 to 10	tensile strength
<u>Mixture tests:</u>			
Indirect tensile strength	100–150 (diam)×50–63		creep compliance
TSRST	50×50×200 or 50 (diam)×200	−50 to 10	fracture temperature

Figure 2.1: Possible mechanism A for crack initiation and propagation in asphalt concrete pavements, after Haas and Topper (1969)

Figure 2.2: Possible mechanism B for crack initiation and propagation in asphalt concrete pavements, after Haas and Topper (1969)

Figure 2.3: Three- and four-element mathematical models

$$J = \frac{2 U}{b (W - a)}$$

J = critical J-integral
 U = flexural fracture energy
 b = sample thickness
 W = sample depth
 a = crack depth

Figure 2.4: Three point bend specimen test, after Little and Mahboub (1985)

Figure 2.5: C^* test method, after Abdulshafi and Majidzadeh (1985)

Figure 2.6: Thermal stresses and tensile strength during a cooling cycle

1. Compensating for shrinkage by materials having different coefficient of thermal contraction.

2. Using temperature resistant materials; Invar or metal alloys.

3. Universal testing machine and a Plexiglas chamber.

4. Apply load to restore thermal shrinkage: step motor, cold room. (SHRP)

Figure 2.7: Different equipment setups for performing TSRST

Chapter 3

Formulation of the Analytical Approach

This chapter deals with the analytical study of thermal crack propagation. It includes a classification of crack types, a detailed description of the three stage thermal cracking model, determination of the temperature field in pavements, and an introduction to fracture mechanics application in pavement cracking problems.

3.1 Classification of Crack Types

Canadian transportation authorities maintain a record of surface cracking and other defects according to the pavement surface condition rating system [RTAC, 1987]. The cracking survey consists of records of the crack type, (centreline, wheel path, transverse or edge), and the approximate length and severity. The new technology of monitoring pavement surface using highway-speed cameras and image processing, will allow for detailed classification of crack types and the progression rates of typical individual cracks. This technology is still undergoing development due to the massive amount of image data and the increasing demand for powerful computing. However, analytical studies require more detailed knowledge of the crack position, boundary conditions and direction than what is provided at the present time.

For the purposes of two and three dimensional analyses, it is proposed to consider each of the following cases independently, Table 3.1 and Figures (3.1,3.2):

1. Potential for surface crack propagation in the vertical direction — The crack is visible at the surface and extends partially into the depth of the layer and across the full lane width.
2. Potential for crack propagation in the vertical direction — The crack is originated by traffic loading at the bottom of the surface layer and extends partially towards the surface and across the full lane width.
3. Potential for edge crack propagation in the horizontal direction — The crack has penetrated the full depth of the asphalt concrete layer but has a limited length along the lane width and is connected to either edge of the paved lane.
4. Potential for general crack propagation in the horizontal direction — Similar to the previous case except that the crack is not connected to either of the pavement edges.

In reality, combination of these four idealized cases are also possible mechanisms for crack growth.

For these reasons, a classification of transverse cracks based on the geometry and location into four distinctive types is proposed as shown in Figure (3.1), [Shalaby et al., 1996a]. Types 1 and 2 run across the full width of the lane and extend partially through the layer thickness. Type 1 initiates at the surface (surface crack, typical of a construction-induced crack), while Type 2 initiates at the bottom of the overlay (reflection crack or traffic fatigue crack). Types 3 and 4 run across a partial width of the lane and extend for the whole depth of the layer. Type 3 originates at the edge of the pavement and propagates laterally towards

the centerline of the pavement (edge crack), while type 4 originates within the pavement layer and extends laterally towards the edges (in-lane crack, crack at utility openings).

The finite element method (FEM) can be used to determine the propagation of each type of cracks under idealized material and thermal conditions as shown in Table 3.1. The analysis is initiated with a transient thermal analysis based on the temperature profile at the site and is followed by a quasi-static stress analysis at discrete time intervals. Stress intensity factors are obtained at different crack lengths and can be used to predict the life cycle. While the first and the second types of cracks are suited for two-dimensional analysis in plane strain mode, the third and the fourth can only be examined with a three-dimensional model [Shalaby et al., 1996a]. To simplify the procedures, an automated mesh generation procedure is invoked through an interface, and the users are not required to create the meshes or have prior experience in the FEM.

It is also noted that transverse cracks can have either a straight line shape or a curved shape concaved with the direction of traffic as shown in Figure (3.2). The “straight shape” suggests a top-bottom crack growth such as Types 1 and 2. The “curved shape” suggests a horizontal crack growth driven by thermal cycling among other factors (for example, traffic associated friction and drag forces) and subject to changes in its direction according to the stress field at the crack tip, which is the case of crack Types 3 and 4.

3.2 Three Stage Model of Thermal Cracking

The three stage model is a conceptual model that deals with the initiation and propagation of thermal cracks, [Abd El Halim et al., 1994]. The framework of the three stage model

and the research approaches for each stage and their relations are as shown in Figure (3.3).

3.2.1 Crack Initiation

Hairline cracks of limited depth, are assumed to develop at the pavement surface in a periodic fashion during compaction as a result of the mismatch in rigidities of the soft and flat asphalt layer and the stiff and circular steel roller. The following observations support this hypothesis:

1. The compaction process involves rolling the asphalt layer in the longitudinal direction, therefore imposing a state of compression which is uniform except closer to the edges of the roller, such a case would develop higher strength in that direction.
2. The transverse direction experiences a state of uniaxial instantaneous compaction with a confining stress governed by Poisson's ratio that would develop lower strength in this direction.
3. The formation of transverse periodic cracks, due to geometry and stiffness of both the asphalt mat and the steel roller and the adverse effect of the drag force at the free rolling drum exhaust this gain in strength and finally the tensile strength in the longitudinal direction is equal to or less than that in the transverse direction.

3.2.2 Crack Propagation

Crack propagation is the second stage in which hairline cracks extend in length and depth until full separation of the crack faces occurs. The crack relieves the thermal stresses in the pavement section. Typically, a performance function is required to determine the

crack growth rate. This can be achieved using the empirical power law proposed by [Paris and Erdogan, 1963] as follows

$$\frac{da}{dN} = A(\Delta K(a))^n \quad (3.1)$$

where,

A and n = the fracture parameters of the material,

a = the crack length,

N = the number of cycles, and

$\Delta K(a)$ = the change in the stress intensity factor at the crack tip.

The fatigue life N_f is determined from integrating over the crack length,

$$N_f = \int_0^h \frac{da}{A(\Delta K(a))^n} \quad (3.2)$$

where h is the smaller of the crack depth causing crack instability and the layer depth. The stability of crack growth is examined from the energy balance of the cracked medium. The condition for crack stability is

$$\frac{\partial^2(U + W)}{\partial a^2} > 0 \quad (3.3)$$

where U is the strain energy, W is the applied work, and a is the crack length. The critical crack length, a_c , defines the upper limit of the crack propagation model as shown in Figure (3.4), and the crack is assumed to extend abruptly whenever this length is reached.

3.2.3 Crack Evolution

Crack evolution deals with the spread of thermal cracks on the pavement surface as a function of average crack growth, which is determined from the crack propagation model.

The model deals with cracks that have already extended beyond the stability condition of equation (3.3). The objective of this stage is to predict the annual increase in cracking.

3.3 Temperature Field in a Pavement Structure

3.3.1 Analytical Evaluation

The study of thermal stresses in pavements relies on the knowledge of the transient temperature field through the pavement during typical cooling cycles. The three dimensional governing equation for heat conduction is given by,

$$\nabla^2 \phi = \frac{c\rho}{k} \frac{\partial \phi}{\partial t} \quad (3.4)$$

where,

ϕ = temperature field,

c = specific heat of the material,

ρ = material density,

k = thermal conductivity of the material, and

t = elapsed time.

For the case shown in Figure (3.5), if the temperature is applied at the surface uniformly, independent of the location, then equation (3.4) reduces to one dimension,

$$\frac{\partial^2 \phi}{\partial z^2} = \frac{c\rho}{k} \frac{\partial \phi}{\partial t} \quad \text{at} \quad 0 < z < \infty, \quad 0 < t < \infty \quad (3.5)$$

The pavement is subject to the boundary conditions $\phi(0, t) = g(t)$, and $\lim_{z \rightarrow \infty} \phi(z, t) = \text{constant}$, where $g(t)$ is the surface temperature determined from ambient temperature, solar radiation, and wind speed, having an initial condition $\phi(0, 0) = T_0$. If the daily air

temperature variation at the pavement surface can be approximated to a cosine curve in the form,

$$g(t) = T_0 + T_1 \left[\cos \left(\frac{2\pi}{24} t \right) - 1 \right] \quad (3.6)$$

where, t = time in hours, and the maximum and minimum daily surface temperatures are T_0 , and $T_0 - 2T_1$, Figure (3.6). The temperature distribution is obtained as a Fourier transform [Dettman, 1969],

$$\phi(z, t) = T_0 + \frac{z T_1}{2\sqrt{\frac{\pi k}{c\rho}}} \int_0^t \frac{\left[\cos \left(\frac{2\pi}{24} \tau \right) - 1 \right]}{(t - \tau)^{3/2}} \cdot e^{-\frac{\frac{\pi k}{c\rho} z^2}{(t - \tau)}} d\tau \quad (3.7)$$

The solution has the following properties;

1. The integral is convergent.
2. The solution is valid for all $z > 0$.
3. The temperature distribution at the pavement surface $z = 0$ is equal to the surface temperature of equation (3.6).

Equation (3.7) is integrated numerically for the entire time range, and an approximate solution may be directly obtained [Lunardini, 1985],

$$\phi(z, t) = T_0 + T_1 \left[\cos \left(\frac{2\pi}{24} t - \sqrt{\frac{\pi c\rho}{24 k}} z \right) \cdot e^{-\sqrt{\frac{\pi c\rho}{24 k}} z} - 1 \right] \quad (3.8)$$

For example, at the following typical properties: $T_0 = -5^\circ\text{C}$, $T_1 = 12^\circ\text{C}$, $c\rho = 1.5 \cdot 10^6 \text{ J}/(\text{m}^3\text{C})$, and $k = 4000 \text{ J}/(\text{m}\cdot\text{s}\cdot^\circ\text{C})$, the temperature distribution at different pavement layers is shown in Figure (3.7). The transient temperature profiles along the depth of the pavement are shown in Figure (3.8).

A comparison of three types of prescribed surface temperature functions is presented in [Selvadurai et al., 1990]; a step function which represents a sudden drop or rise, an

exponential function and a sinusoidal function. All three functions have the same amplitude, T_1 . From examining values of the stress intensity factor in a cracked pavement structure, they concluded that all three functions have approximately the same maximum values when the pavement surface reached its lowest possible temperature.

3.3.2 Estimated Pavement Temperatures

To simplify the procedures for determining the pavement temperature profile, the following equation is suggested [Solaimanian and Kennedy, 1993]

$$T_Z = T_S + 0.051Z - 6.3 \times 10^{-5}Z^2 \quad (3.9)$$

where,

T_Z = temperature at depth Z ($^{\circ}\text{C}$),

T_S = pavement surface temperature ($^{\circ}\text{C}$) and,

Z = depth measured from pavement surface (mm).

The equation is valid for $Z < 200$ mm and the pavement surface temperature is obtained from the air temperature

$$T_S = n T_a \quad (3.10)$$

where n is a factor with a recommended value of 0.9 and 10% variation for low temperature cracking studies. T_a being the coldest air temperature in the study period. This equation was found to give lower than measured temperatures. Another equation is proposed by Robertson [Robertson, 1987] as

$$T_{min} = 0.859 T_{air} + [0.02 - 0.0007T_{air}]Z + 1.7 \quad (3.11)$$

where,

T_{min} = minimum temperature at depth Z ($^{\circ}\text{C}$),

T_{air} = minimum air temperature during winter ($^{\circ}\text{C}$), and

Z = depth measured from pavement surface (mm).

From this section, it is concluded that the pavement transient temperature profiles can be determined, in closed form or using an approximate method, if the surface temperature is given as a sinusoidal function and the pavement is thermally modelled as a single layer. If the pavement layers have different thermal properties, a numerical solution is required. Approximate estimated values can be found for the lowest temperatures in the study period.

3.4 Thermal Crack Analysis Using Fracture Mechanics

A crack in a continuous medium is prone to three orthogonal stresses that can contribute to its propagation. The three cracking modes are a result of tension, shear or torsion stresses on the crack faces, Figure (3.9). The analysis of crack growth can be simplified by separating the components of the stress field into the three directions and studying the contribution of each component independently.

The drop in the temperature of pavement layers causes thermal tensile stresses that will always fall into the mode 1 category whether the crack type is top, bottom, inlane or edge. It is quite feasible that surface transverse cracks extend at a faster rate due to traffic loads. In the case of an edge crack, the passing of axle loads from one side of the crack to the other causes torsional stresses on the crack tip that fall into the mode 3 category. However, cases involving load-associated cracking are beyond the scope of this research.

The fracture behaviour of asphalt pavements is highly sensitive to temperature. The material behaves in a fully plastic manner during the paving and compaction operations with the temperature ranging from 90 to 135 °C. After construction, the asphalt temperature during service life will range between -40 °C and +60 °C depending on the site-specific prevailing climatic conditions. On any day, the asphaltic material can be subject to a temperature drop or rise in the order of 20 °C. The crack growth under these conditions is governed by a combination of the five phases shown in Figure (3.10). The approximate temperature ranges shown in the figure are suggested based on typical values of the fracture temperature, the glass transition temperature, and the softening point.

The linear elastic fracture mechanics, LEFM, is applicable at low temperatures below the freezing point since the material becomes brittle and failure occurs at relatively low strains. Elastic plastic fracture mechanics, EPFM, is more applicable at the intermediate temperatures between -10°C and 40°C, since the material can tolerate higher deformations and a plastic zone is formed around the crack tip. A fully plastic collapse is encountered at high temperatures, above the softening point. The pavement conditions at construction fall into this range where the compaction effort results in surface transverse hair line cracks.

It should be noted that the tensile thermal strain which develops the stress σ is not constant but increases with the drop in temperature and is negligible at temperatures at or above 25°C. The shaded areas in Figure (3.10) represent the shapes of the crack influence zones which are obtained from stress analysis. It is noted that there are no distinctive bounding limits for each phase, and that a gradual change between the phases is assumed. The scope of this study on low temperature cracking is covered by LEFM and EPFM according to phases A, B, and C.

Table 3.1: Types of cracks, range of crack sizes, analysis method, and crack shape

Crack type	Description	Crack geometry		Analysis method	Crack shape
		depth ratio x/a	length ratio b/L		
1	Surface crack	$0.1 \rightarrow a_c/a$	1.0	Plane strain	Straight
2	Reflection crack	$0.1 \rightarrow a_c/a$	1.0	Plane strain	Straight
3	Edge crack	1.0	$0.1 \rightarrow b_c/L$	3-D	Curved
4	Inside-lane crack	1.0	$0.1 \rightarrow b_c/L$	3-D	Curved

x = crack depth, b = crack length, a_c = critical crack depth, b_c = critical crack length, a = pavement layer thickness, and L = lane width.

(a) Longitudinal section of the pavement system showing vertical cracks (Type 1 and Type 2)

(b) Plan of the pavement surface showing full depth lateral cracks (Type 3 and Type 4)

Figure 3.1: Type and location of thermally-driven transverse cracks

Figure 3.2: Plan of pavement surface showing two alternatives for crack shape

Figure 3.3: Framework of the three stage model for thermal transverse cracking.

Figure 3.4: Condition for stability of crack growth from energy balance

Figure 3.5: Slab model, coordinate system and imposed surface temperature

Figure 3.6: Assumed daily air and pavement surface temperature model

Figure 3.7: Time and depth variation of pavement temperature field

Figure 3.8: Typical daily transient temperature gradients along the pavement thickness

Figure 3.9: Fracture mechanics representation of cracking modes

Low temperature
in service

Figure 3.10: Applicability of fracture mechanics methods to describe crack growth in asphalt pavements. Redrawn and revised from Ewalds and Wanhill (1985)

Chapter 4

Fracture Mechanics Approach to Crack

Propagation

The development of a finite element procedure suitable for thermal cracking problems is presented in this chapter. Two and three dimensional analyses are performed and practical applications are discussed.

4.1 Introduction

At the present time, many finite element applications on analyzing pavement structures under thermal loads are using a tensile applied stress in the horizontal direction instead of performing a full thermal analysis. In the case of pavements with cracks, the effect of having more than one crack and the three dimensional cracks have not been explored. In this chapter, the transient response of a pavement structure is obtained by a transient thermal analysis followed by a quasi-static stress analysis at discrete time intervals. The method is applied to pavement systems with a single crack or a multitude of cracks. Two- and three-dimensional solutions are obtained and a sensitivity analysis is performed.

The main objectives of the numerical analyses are:

1. To analyze the thermal and mechanical response of multi-layered pavement systems with emphasis on low-temperature cracking.
2. To identify the main factors that govern the propagation of transverse cracks.
3. To perform fracture analysis in cracked layers subjected to a thermal gradient.
4. To develop a methodology for determining crack growth rates in thermally cracked pavements.
5. To evaluate the use of numerical procedures in thermal cracking problems.

4.2 Selected Numerical Procedure

The ANSYS program is used throughout the finite element procedure. The analysis is divided into the following modules:

1. Mesh generation. Preparing the input files of finite element programs is often a lengthy and cumbersome task. Since pavement analysis requires many runs and a great number of cases needs to be analyzed, it was decided to resort to an automated mesh generation system. In such a case, changes in problem dimensions, material properties or loading conditions required minimal changes in the input file. This provision also simplified simulation of the propagation of cracks by advancing the crack tip into the pavement layer and obtaining the solution on a regenerated problem mesh.
2. Thermal analysis. The boundary conditions of this case included a prescribed pavement surface temperature and that there is no heat flow across the model boundaries due to symmetry. The temperature is assumed to be constant at a sufficient depth in the

subgrade. The outputs of this module are the nodal temperatures at the intervals of a thermal cycle, which are typically hourly intervals.

3. Stress analysis. While the generated mesh and nodal temperatures are prereserved from the two previous modules, the displacement boundary conditions are changed to simulate the new crack extension and the bond between the layers. The solution is generated independently for each interval and the stresses and displacements are obtained.
4. Fracture analysis. The displacement formula and the J-integral methods are employed to determine the stress intensity factor at the crack tip.
5. Crack propagation. The crack is propagated accordingly and the stress and fracture modules are repeated.

4.3 Finite Element Formulation

Numerical modeling of transient thermoelastic equations described earlier in Section 2.6.2 combined with fracture analysis can be approached using various methods. The finite element method and the boundary element method represent the most convenient alternatives. Considering the domain D having a boundary surface S , the boundary conditions of this general case consist of the heat conduction and elastic constraints shown in Figure (4.1). The heat conduction boundary conditions are; a prescribed surface temperature $T = \bar{T}$ on the boundary surface L_1 , and a rate of heat flow across the surface L_2 such that $k\partial T/\partial n = \bar{q}$. The elastic boundary conditions are; a prescribed displacement $u = \bar{u}_i$ on the surface L_3

and tractions $\sigma_{ij} n_j = \bar{P}_i$ on the surface L_4 . The expressions for the finite element Galerkin formulation are derived [Hsu, 1986, Selvadurai et al., 1990].

The thermoelastic stress-strain relationship is in the form:

$$\sigma_{ij} = C_{ijkl}\epsilon_{kl} - \beta(T - T_0)\delta_{ij} \quad (4.1)$$

where β is the thermoelastic coefficient. In thermomechanical problems, δT and δu_i are the variation functions in temperature and displacement.

$$\int_D \delta T [(kT_{,i})_{,i} - \rho c \dot{T}] dD = \int_{L_2} \delta T \left(k \frac{\partial T}{\partial n} - \bar{q} \right) dS \quad (4.2)$$

$$\int_D \delta u_i (\sigma_{ij,j} - b_i) dD = \int_{L_4} \delta u_i (P_i - \bar{P}_i) dS \quad (4.3)$$

In these two equations, integrating the domain term by parts results in

$$\int_D [k \delta T_{,i} T_{,i} + \rho c \delta T \dot{T}] dD - \int_{L_2} \delta T \bar{q} dS = 0 \quad (4.4)$$

$$\int_D [\delta \epsilon_{ij} \sigma_{ij} + \delta u_i b_i] dD - \int_{L_4} \delta u_i \bar{P}_i dS = 0 \quad (4.5)$$

The thermal and mechanical strain components can be treated independently, such that

$$\int_D [\delta \epsilon_{ij} C_{ijkl} \epsilon_{kl} + \delta u_i b_i + \delta \epsilon_{ij} \beta (T - T_0) \delta_{ij}] dD - \int_{L_4} \delta u_i \bar{P}_i dS = 0 \quad (4.6)$$

Equations (4.4) and (4.6) are used in the variational representation of the finite element discretization

$$\sum_D \left(\int_e [k \delta T_{,i} T_{,i} + \rho c \delta T \dot{T}] dD - \int_{e \cap L_2} \delta T \bar{q} dS \right) = 0 \quad (4.7)$$

$$\sum_D \left(\int_e [\delta \epsilon_{ij} C_{ijkl} \epsilon_{kl} + \delta u_i b_i + \delta \epsilon_{ij} \beta (T - T_0) \delta_{ij}] dD - \int_{e \cap L_4} \delta u_i \bar{P}_i dS \right) = 0 \quad (4.8)$$

where e is an element of the discretized domain D .

The matrix form is obtained from the variations and summation on the discretized domain

$$[K_{TT}]\{T\} + [M_{TT}]\{\dot{T}\} = \{P_T\} \quad (4.9)$$

$$[K_{uu}]\{U\} + [K_{uT}]\{T\} = \{P_u\} \quad (4.10)$$

where

$$[K_{TT}] = \int_e k[B_T]^t[B_T]dD \quad (4.11)$$

$$[K_{uu}] = \int_e [B_u]^t[C][B_u]dD \quad (4.12)$$

$$[M_{TT}] = \int_e \rho c[N_T]^t[N_T]dD \quad (4.13)$$

$$[K_{uT}] = \int_e \beta[B_u]^t[N_T]dD \quad (4.14)$$

$$[P_T] = \int_{e \cap L2} [N_T]^t \bar{q} dS \quad (4.15)$$

$$[P_u] = \int_{e \cap L4} [N_u]^t \bar{P} dS - \int_e [N_u]^t b dD \quad (4.16)$$

in which the matrices are obtained by summing over the element domain e , $[]^t$ denotes a matrix transpose, $[N_T]$ and $[N_u]$ are the shape function matrices corresponding to thermal or stress fields with spatial derivatives $[B_T]$ and $[B_u]$, and $[C]$ is the matrix of mechanical constitutive relations.

The solution of the global structure matrices in equations (4.7) and (4.8) is performed using an incremental procedure in the ANSYS program. Increments are performed with full Newton Raphson method and using a frontal equation solver.

4.4 Time Integration Schemes for Thermal Analysis

In nonlinear analysis, a time integration scheme is required to solve the thermoelastic problem in the time domain. For the present analysis, the Crank-Nicolson scheme is adopted. A review of the existing schemes is discussed in [Stuzalec, 1992]. The selected method is presented below. Considering the transient heat conduction problem presented earlier in equation 2.6

$$(k T_{,i})_{,i} - \rho c \dot{T} - (T - T_0)\beta \dot{\epsilon}_{II} + Q = 0 \quad (4.17)$$

with the initial condition $T(0) = T_0$, the system of equations can be put in the matrix form as

$$\dot{T}(t) = A(T)T(t) + D(T, t) = G(T, t) \quad (4.18)$$

where $A(T) = c^{-1}k$, and $D(T, t)$ is a forcing vector. If a linear variation of T is assumed over the time interval $[t_n, t_{n+1}]$, then

$$T_{n+1} - T_n = hG_\theta = hG(T_\theta, t_\theta) \quad (4.19)$$

where

h is the time element ($t_{n+1} - t_n$), θ is a weight factor $0 \leq \theta \leq 1$ that grants a fully explicit forward scheme, $\theta = 0$, a fully implicit backward scheme, $\theta = 1.0$, or a trapezoidal or a mid-point scheme, $\theta = \frac{1}{2}$

$$T_\theta = (1 - \theta)T_n + \theta T_{n+1}, \text{ and} \quad (4.20)$$

$$t_\theta = (1 - \theta)t_n + \theta t_{n+1} = t_n + \theta h \quad (4.21)$$

If G is linear and $\theta = \frac{1}{2}$, we arrive at the Crank-Nicolson scheme stating,

$$(\mathbf{I} + \theta h A_\theta) T_{n+1} = [\mathbf{I} - (1 - \theta) h A_\theta] T_n + \theta D_\theta \quad (4.22)$$

where $\mathbf{I}$ is the identity matrix, and A_θ and D_θ are obtained from the material thermal properties and forcing function at each time step h . The stability of the time integration algorithm is unconditional for all $h \geq 0$ and the accuracy depends on the size of the time element and the nature of the forcing function.

4.5 Material Modeling: Thermal and Mechanical

The model configuration and boundary conditions shown in Figure (4.2) are based on the following assumptions:

1. Phase changes are not considered in this analysis; thus volume change as a result of the drop or rise in surface temperature (freezing and thawing) is not considered.
2. The coefficient of thermal expansion is constant and independent of the temperature.
3. All pavement layers are homogeneous, elastic, and isotropic. A crack in the surface layer is modeled by a discontinuity in the element mesh. The singularity of the numerical solution is avoided by using special crack tip elements.
4. Full bond exists between pavement layers.

These assumptions are considered appropriate for this general type of analysis. When the method is applied for a particular site, material response models and local conditions for the site can replace the assumed values.

4.5.1 Thermal Properties of Granular Layers

The thermal conductivity is highly affected by the moisture content in the soil. For unfrozen soils having 50% or more clay or silt content, the thermal conductivity K is given by Phukan as

$$K = 0.00144 \cdot 10^{0.001373\rho_d} + 0.01226 \cdot 10^{0.000499\rho_d\omega} \quad (4.23)$$

and for sandy soils, the thermal conductivity K is given by

$$K = 0.01096 \cdot 10^{0.0008116\rho_d} + 0.00461 \cdot 10^{0.0009115\rho_d\omega} \quad (4.24)$$

where, K is in $W/m^\circ C$, ρ_d is the dry density in kg/m^3 , and ω is the moisture content in percent [Phukan, 1985].

When the moisture content is high, the thermal conductivity of frozen soils is higher than that of dry soils. For soils with low degree of saturation, the frozen conductivity is less than that of unfrozen soils. The coefficients are sensitive to the quartz and micaeous mineral constituents. If the typical values of the specific heat capacity of soil constituents of 0.17, 0.5 and $1.0 \text{ kcal} \cdot \text{kg}^2/\text{gm}^\circ C$ for the dry soil, frozen and unfrozen moisture, respectively, then the volumetric heat capacity for frozen soils is given by,

$$C_f = 4.187\rho_{df}[0.17 + 0.5(\omega - \omega_{uw}) + \omega_{uw}] \quad (4.25)$$

where C_f = the specific (volumetric) heat capacity in $J/kg^\circ C$, ω and ω_{uw} are the total and unfrozen moisture content in percent, respectively, and ρ_{df} is the dry frozen density in kg/m^3 . In unfrozen soils, $\omega_{uw} = \omega$, the formula (4.25) reduces to

$$C_f = 4.187\rho_d[0.17 + \omega] \quad (4.26)$$

where ρ_d is the dry density in kg/m^3 .

4.5.2 Stiffness of Asphalt Concrete

The stiffness of the asphalt concrete is time dependent and temperature dependent. Due to the slow nature of the thermal loading, a loading time of 3600 seconds is maintained throughout the analysis. The stiffness of the asphalt binder is computed from a relationship based on van der Poel nomograph [Shahin and McCullough, 1972]. The stiffness-temperature relationship is discussed further in a later stage of the thesis in Chapter 7. The mixture stiffness is obtained using Heukelom's relationship to binder stiffness and aggregate concentration in the mixture.

$$S_{mix} = S \left(1 + \frac{2.5 C_v}{A (1 - C_v)} \right)^A \quad (4.27)$$

$$A = 0.83 \log \frac{400000}{S} \quad (4.28)$$

$$C_v = \frac{\text{volume of aggregates}}{\text{volume of (aggregates + asphalt)}} \quad (4.29)$$

where

S and S_{mix} = the binder and mixture stiffnesses in MPa, and

C_v = aggregate concentration factor.

The relationship was developed for well compacted mixtures with air voids of about 3% and C_v ranges from 0.7 to 0.9. A correction factor is used when the air voids are higher than 3%

$$C_v^* = C_v \frac{1}{1 + H} \quad (4.30)$$

where C_v^* is the *corrected* aggregate concentration factor, and $H = (\text{air voids} - 0.03)$

The coefficient of thermal contraction and Poisson's ratio are assumed constant in the analysis using average values for the studied range of temperatures. The numerical

examples presented in the following sections use typical values for thermal and mechanical properties, shown in Table 4.1, obtained from the literature [Shahin and McCullough, 1972, Phukan, 1985, Selvadurai et al., 1990]. Alternatively, the stiffness can be determined from laboratory tests such as those described in Chapter 6. The secant or tangent moduli are determined in closely replicated service conditions. The obtained values will accurately represent the examined mixture at the specified temperature range and rate of loading. It is further possible to take into account any reduction in the mix stiffness due to load cycling by using the results of CSCT test method shown in section 6.3.

4.6 Surface Temperature Model

The temperature of the pavement surface can be closely represented by a cosine function. For the purpose of numerical analysis, it is approximated by a trapezoidal function (ramp freezing) as shown in Figure (4.3). The imposed surface temperature model consists of a two-ramp function for the temperature drop and rise along an extended period of five days each, and a one-day constant temperature following each ramp. The slower rate of change of temperature is applied to simulate the most adverse conditions. In addition, a one-day constant temperature allows for further penetration of the cooling and warming temperature fields. For comparison, the step function shown in the figure was implemented in earlier solutions prior to the use of transient thermal analysis [Selvadurai et al., 1990]. In the case of a step function, a sudden drop or rise in temperature is applied and the steady state temperature profile is determined.

Although the temperature fields of a ramp-freezing and a step function will converge if

the duration of the cycle is long, e.g. several days, the thermal gradients in this case will be different. It is believed that the cosine function or ramp freezing function are the best approximations to the actual pavement thermal regime [Lunardini, 1985].

4.7 Features of the Finite Element Analysis

The numerical analysis is performed using a general FEA source code, ANSYS [Swanson Analysis Systems, 1993], operated at the School of Engineering, Lakehead University. The unix based version of the program is remotely accessed from Carleton University unix computing network. The transient response of the pavement structure is modeled by a transient thermal analysis followed by a quasi-static stress analysis. The quasi-static stress analysis considers the thermal regime at different positions in the thermal cycle when stress relaxation takes place in between thermal loadings. The stresses are not retained from one analysis to another and calculations at a certain thermal condition are based on the temperature change from the initial condition to the current one. In designing the finite element mesh, the following factors are considered:

1. The element mesh used in thermal analysis has to be suited for both stress and fracture analyses,
2. For thermal analysis, a finer element size is required closer to the pavement surface where the temperature gradient is highest,
3. A finer element size around the crack and special crack tip elements are required to model the singularity zone of the crack,

4. Due to symmetry, only one-half of the model needs to be considered, and
5. Because of the large number of problems to be analyzed and the different crack configurations to be considered in each problem, an automatic mesh generator is preferred. This feature allows the user to create a detailed finite element mesh with specified geometry and element types without prior experience with FEA or the ANSYS program.

4.7.1 Element Types and Analytical Procedure

The selection of element types from the ANSYS element library is made according to the number of degrees of freedom and the type of analysis to be performed. Of interest are the thermal and structural elements. The program further allows the change from one element type to another provided that both types have the same number of nodes. When possible, this change preserves results from the already performed runs, and those can be combined with other results or imposed as loads to the new model. The presented analysis is constructed around these features.

For two dimensional analysis, the 8-node isoparametric element, PLANE77, is used in the thermal analysis, and replaced by PLANE82 in the stress analysis, Figure (4.4). In isoparametric elements, the same shape functions are used to define the nodal coordinates and nodal degrees of freedom. The 8-node isoparametric element assumes quadratic approximation of the field variable, temperature or displacement. It ensures continuity of the thermal (or displacement) field between elements, in other words values of temperature (or displacement) at a common edge between two elements are equal when viewed or interpolated from any of the two adjacent elements. Isoparametric elements are useful in

modelling the nonlinear stress changes around the crack tip with higher accuracy than lower order elements [Gdoutos, 1990, ?].

When a cracked elastic and isotropic body is loaded perpendicular to the crack face, the stresses in the vicinity of the crack tip are determined from the following relationship:

$$\sigma_y = \frac{K_I}{\sqrt{2\pi r}} \left(\cos \frac{\theta}{2} \right) \left[1 + \sin \frac{\theta}{2} \sin \frac{3\theta}{2} \right] \quad (4.31)$$

where

r and θ = polar coordinates originating at the crack tip,

σ_y = the stress normal to the crack face, and

K_I = stress intensity factor (SIF).

The main characteristic of the stress field is the singularity, caused by division by zero, that occurs at the crack tip when $r \rightarrow 0$. A material characteristic value, known as the fracture toughness K_{IC} , is evaluated for particular cracking modes from laboratory testing. If the stress intensity factor exceeds the fracture toughness of the material, i.e. $K_I \geq K_{IC}$, the crack will extend until the energy is balanced on the new position or complete separation takes place.

A higher value of K_I indicates a higher rate of crack propagation. The amount of crack extension from a single loading cycle can be estimated from crack growth models using the stress intensity factor. It has been found that if the element mid-side node is moved to a quarter point position closest to the crack tip, as shown in Figure (4.4), then the singularity is achieved in elastic solutions and the element displays the stress field of equation (4.31). The resulting element, termed quarter-point element [Barsoum, 1976], will be used to model the crack tip zone and to determine the stress intensity factor more accurately.

A flow chart describing the numerical treatment of the two-dimensional problem is given in Figure (4.5). Since the procedure includes multiple finite element solutions for various problems, an automated mesh generator is developed for use in modeling the user-specified geometry and crack depth. Transient thermal analysis is performed for the full duration of the thermal loading cycle, and is entirely independent of the stress conditions in the pavement. This approach is acceptable in low temperature pavement applications since the amount of heat generated by strains and deformations is too small to affect the overall thermal conditions of the material. While the mesh geometry and nodes are retained, the thermal element mesh is replaced with stress elements. Temperatures obtained from thermal analysis are imposed on the model. The corresponding stress intensity factors are computed, and the crack is propagated accordingly. A new mesh is generated and the numerical procedure is repeated.

4.7.2 Thermal Analysis

It is assumed that the pavement temperature profile depends entirely upon the air temperature. Thus, radiation and convection heat transfer are not expressed explicitly. The assumed daily surface temperature is represented by a cosine function in the analytical solutions, which is approximated by a trapezoidal function in the finite element analysis.

The transient thermal response of a pavement system with the boundary conditions shown in Figure (4.2b) is one-dimensional. It can be simulated by that of a bar subjected to a prescribed temperature at its end in the form of $g(t) = T_0 + T_1 \cos(\frac{2\pi t}{24})$. The temperature distribution in time and space, $\phi(z, t)$, is obtained as a Fourier transform [Dettman, 1969]

and given as follows:

$$\phi(z, t) = T_0 + \frac{z T_1}{2\sqrt{\frac{\pi k}{c\rho}}} \int_0^t \frac{\left[\cos\left(\frac{2\pi}{24}\tau\right) - 1\right]}{(t - \tau)^{3/2}} \cdot e^{-\frac{z^2}{4k(t-\tau)}} d\tau \quad (4.32)$$

where

z = the vertical location measured from the surface,

t = time in hours,

T_0 = daily maximum temperature,

T_1 = the amplitude of the temperature cycle,

ρ = material density,

c = specific heat, and

τ = time variable.

A finite element mesh is constructed to represent the pavement structure with the following material and loading properties: $c\rho = 5 \times 10^6 \text{ J}/(\text{m}^3 \cdot ^\circ\text{C})$, $k = 1.5 \text{ J}/(\text{m} \cdot \text{s} \cdot ^\circ\text{C})$, $T_0 = 25^\circ\text{C}$, and $T_1 = 12^\circ\text{C}$. The imposed temperature field is approximated by a trapezoidal shape with matching maximum and minimum temperatures that are maintained for two-hour duration and linear gradients elsewhere. The analysis is initiated with the full depth at constant initial temperature, T_0 , and is carried out for a period of 48 hours (two full thermal cycles) prior to the examined cycle in order to maintain the thermal equilibrium in pavement layers.

Comparison of the finite element analysis to the Fourier transform solution is shown in Figure (4.6) for the temperature field at the pavement surface and at depths of 100 and 200 mm from the surface. The temperature gradient along the depth is shown in Figure (4.7). It is shown that the finite element results are in agreement with the analytical model, and

the model can be extended to accommodate varying thermal properties of different layers for which no analytical solution exists.

4.8 Fracture Analysis

Fracture mechanics methods pertaining to thermal stresses at low temperatures include the linear elastic and the elastic plastic phases. It is expected that elastic plastic conditions prevail at moderate temperatures associated with ductile failures where the material can experience plastic flow, typically below 25°C. The linear elastic conditions prevail at lower temperatures associated with brittle elastic behaviour, typically below -5 °C. The zones of applicability were discussed earlier in section 3.4.

4.8.1 Linear Elastic Fracture Mechanics: Stress Intensity Factor Method

The brittle growth of transverse cracks in an elastic asphalt concrete layer can be evaluated using the stress intensity factor at the crack tip. The stress intensity factor is computed in linear elastic fracture using the displacements of the crack face. In the two-dimensional plane strain conditions, the crack opening under temperature variations is of the first mode (tension on the crack faces). The cracked zone is modeled with quarter point crack tip elements and the stress intensity factor, K_I , is obtained from the two-point formula,

$$K_I = \frac{2G}{\gamma + 1} \sqrt{\frac{2\pi}{l}} (4u_2 - u_3) \quad (4.33)$$

where

$\gamma = 3 - 4\nu$ in the plane strain case, and $= (3 - \nu)/(1 + \nu)$ in the plane stress case,

l = length of the element edge,

u_2 and u_3 = displacements normal to the crack face at distances $l/4$ and l from the crack tip, respectively,

E = modulus of elasticity,

ν = Poisson's ratio, and

G = shear modulus equals to $E/[2(1 + \nu)]$.

4.8.2 Elastic-Plastic Fracture Mechanics: The J Integral Method

The path-independent J integral method is used to examine the crack growth potential when the material nonlinearities are considered. The integral is evaluated on an arbitrary path Γ ending at opposite faces of the crack as shown in Figure (4.10), and the singularity of the crack tip is within the integrated zone, [Paris and Erdogan, 1963]. The objective is to determine the rate of energy release due to the growth of the crack from the line integral,

$$J = \int_{\Gamma} U \, dy - \int_{\Gamma} t_i \frac{\partial u_x}{\partial x} \, ds \quad (4.34)$$

in which

U = strain energy density = $\int_0^{\epsilon} \sigma_{ij} d\epsilon_{ij}$,

σ, ϵ = stress and strain tensors,

t_i = traction vector = $\sigma_{ij} n_j$,

n_j = the outward normal to Γ , ($j = 1, 2, 3$)

u = displacement vector, and

s = length of the path Γ .

For linear elastic conditions, the stress intensity factor can be related to J using

$$K = \sqrt{\eta \cdot J} \quad (4.35)$$

where $\eta = E/(1 - \nu^2)$ in plane strain and $= E$ in plane stress conditions.

The numerical procedure to compute the J -integral is integrated into the finite element input data as shown in Appendix A. The first integral of equation (4.34) is evaluated by a summation of strain energy on all elements within the selected path. The second integral of equation (4.34) is determined by shifting the path an amount equal to 1% of its length in the direction of the crack. The tractions t_i are found on the original position while the displacement u_x is computed at both the original and new positions. The derivative $\frac{\partial u_x}{\partial x}$ is obtained from the difference between u_x at the original and at the displaced positions, and the integral can be evaluated with eliminating the need for a second program run at a new crack position.

The two main advantages of the J integral are, 1) only one run is required, and 2) it is path-independent and hence, can be evaluated at a sufficient distance from the crack tip to avoid the singularity zone. The J -integral method will be used in conjunction with the three-dimensional analysis.

4.9 Practical Applications

The current methods for investigating cracking in asphalt pavements are applicable in the presence of one crack only. Obviously, in some cases the effect of a second crack can not be neglected. Moreover, certain crack propagation problems cannot be modelled in 2-D and require 3-D solutions. The proposed FEA method is used to investigate typical crack propagation problems that have been observed in this field. The first application deals with the problem of a single crack that extends across the whole width of the overlay and

propagates vertically through its thickness. The second application is on the interaction between multiple cracks, crack spacing, and bond between layers on crack propagation. The third application is a sensitivity analysis conducted on the variables of the multiple cracking problem. The three selected problems are amenable to two-dimensional analysis.

The problem of a surface crack that extends fully through the depth of the overlay and propagates horizontally across the pavement width is not suited to two-dimensional finite elements, and therefore is modeled using three-dimensional elements and presented in a later section.

4.9.1 Single Crack

The finite element method is used to compute the linear elastic stress intensity factor, K_I , as a measure of the crack opening mode during a temperature cycle and for different crack depths. The crack extends to the full pavement width in the transverse direction while it extends partially through the overlay thickness. Two-dimensional (2-D) analysis is sufficient to model the given problem. An example of the FEA mesh used in this study is presented in Figure (4.8), and the detailed mesh configuration at the vicinity of the crack tip is shown in Figure (4.9). The automated mesh generation routine places quarter point crack tip elements at the crack and element sizes are proportioned according to their proximity to the crack. The same mesh is suitable for thermal, stress and fracture analyses. The surface temperature cycle is given by,

$$\begin{aligned} T(0, t) &= -4 \text{ } ^\circ\text{C}, & t \leq 5 \text{ (days)} \\ T(0, t) &= -20 \text{ } ^\circ\text{C}, & t > 5 \text{ (days)} \end{aligned} \tag{4.36}$$

The material properties for the presented example are shown in Table 4.1.

The normalized stress intensity factors are shown in Figure (4.11). The effect of loading time and crack depth are shown in Figure (4.12). The stress intensity factors are given in the nondimensional form (K_I/K_0) where $K_0 = \alpha G(T_f - T_0)a^{1/2}$, α and G are material properties given in Table 4.1, $(T_f - T_0)$ is the temperature drop, and a is the layer thickness. It is expected that stress relaxation and the plasticity zone at the crack tip form a buffer region which slows the propagation of the crack and prevents the development of the full stress concentration computed using the linear elastic fracture. Effects of stress relaxation and plasticity zone are at a minimum at severe low temperatures due to the high stiffness of the asphalt concrete pavement. The life estimates obtained from this analysis are considered conservative.

4.9.2 Multiple Cracking

The influence of secondary cracks on the propagation of a main crack is modeled. Two limiting cases for the interface between the original asphalt layer and the overlay are considered: the first is the case of a smooth interface which can represent an overlay on a cracked old pavement layer and the second case is of a rough interface implying stiffer support from the original pavement. The geometry of the problem is shown in Figure (4.13). Symmetry is assumed about the vertical plane at the tip of the main crack. All cracks are assumed to initiate from the surface, the main crack extends to one half of the overlay thickness, and each of the secondary cracks extend to one quarter of the overlay thickness. All cracks may have been initiated by the compaction process, but vary in depth

following a statistical distribution and propagate under thermal stresses during the service life.

The relationship between the normalized stress intensity factor and the crack spacing is shown in Figure (4.14) for the specific geometry and conditions of this example. The influence of adjacent cracks affect the crack sealing process. Crack sealants often exhibit higher stiffnesses at low temperatures. The single crack condition is reached if all surrounding cracks are sealed with stiff sealants or asphalt concrete. The reduced flexibility of the system results in higher stresses on the remaining crack, and its propagation is then accelerated. Comparison of the results of a single crack and multiple cracks in the smooth interface case shows that the stress intensity factor increases in proportion to crack spacing until it reaches a stable condition at $S/(a/2) = 20$. It is logical to assume that secondary cracks would effectively reduce the growth rate of a larger crack in a manner similar to the behaviour of contraction joints. Also, the effect of base restraint is evident from comparing the cases of smooth and rough interfaces. It is concluded that increasing the base restraint would result in higher stresses on the crack tip. These stresses are not affected by the spacing of cracks, as shown in the case of rough interface.

4.9.3 Sensitivity Analysis

A sensitivity analysis is carried out on the multiple cracking problem. The pavement structure consists of three layers with the properties shown in Table 4.1. The section length is 12 m and because of symmetry, only one half of the section is modeled. The pavement section has an original surface crack at the middle of the studied zone, and is analyzed under

a system of new equal-spaced transverse cracks. The crack depth is varied to explore all different possibilities of crack extension. The terminology of “original” and “new” cracks is introduced to distinguish between two families of cracks and independently change the depth of one versus the depth of a group of cracks. The temperature is prescribed on the pavement surface and at a depth of 2.0 m and a steady state thermal analysis is performed and the thermal gradient in the pavement system is determined. The thermal strains are computed from nodal temperatures and applied to a restrained pavement structure and used to carry out the stress analysis. The pavement is modelled using 2-D plane isoparametric elements in plane strain with the following characteristics:

thickness of overlay = 100 mm,

thickness of base = 150 mm,

depth of original crack = 60 mm,

depth of new cracks = 60 mm, and

spacing of cracks = 3000 mm.

The sensitivity analysis is performed to obtain the stress intensity factors at the main and secondary cracks while independently changing the following variables:

1. shear modulus of the subgrade = (0.0, 0.09, 0.45, 1.12, 2.25) MPa,
2. shear modulus of the base = (0.0, 0.09, 0.45, 1.12, 2.25) MPa,
3. thermal gradient along the pavement depth = (-5, -15, -25) °C/m,
4. depth of original crack = (20, 40, 60, 80, 100) mm,
5. spacing of cracks = (1000, 3000, 6000, and ∞), while the depth of all cracks = (20, 40, 60, 80, 100) mm.

The underlined values represent values used for the reference case and in this case the depth of all cracks is equal to 60 mm and there is no difference between original and new cracks.

The stress intensity factor is obtained from the two point displacement formula, equation (4.33), and the results of the sensitivity analysis are displayed in Tables 4.2 to 4.6. It is noted that the stress intensity factor and hence the growth rate of cracks are most sensitive to the temperature changes.

The stress intensity factor appeared to be very sensitive to decreased base layer shear modulus, but less sensitive to increased shear modulus as shown in Table 4.2. Although the stress intensity factor is not very sensitive to changes in subgrade shear modulus, as shown in Table 4.3 and Figure (4.16), subgrade material type appears to have greater influence on crack growth than what has been shown in the present analysis, for the following reasons:

1. Drainage and moisture conditions are not considered. According to recent findings from the LTPP distress surveys, transverse cracks grow faster in wet-freeze climatic zones than dry-freeze ones.
2. The thermal properties, specific heat and thermal conductivity, are different according to the type and composition of the subgrade, which can alter the temperature profile in the pavement layer.
3. The bond between pavement layers may not be fully developed or mobilized as assumed in the present analysis. The load transfer between layers affects the deformation of the surface layer and its resistance to cracking.

The effect of thermal gradient is evident from Table 4.4. The linear relationship, depicted in this case, results from the simplified input material thermal properties, and the use of

a constant coefficient of thermal contraction. The finite element solutions accommodate for nonlinear properties and temperature dependent coefficient of thermal cracking, if field data is available.

The effect of crack spacing and depth on crack growth is presented in Table 4.6 and Figure (4.17). The full bond between the layers reduces the interaction effects between cracks, however, the crack growth rate increases with the increase of crack depth and crack spacing. This effect is very noticeable near sealed cracks. The impact of sealing some but not all active cracks is visible from this figure. It is shown that at a crack spacing less than 3 meters, the stress intensity factor is affected by the spacing between cracks. One important case to consider occurs following the use of stiff sealants to seal the visible cracks in a pavement layer. The crack spacing is increased and the remaining unsealed cracks are subjected to higher loads, which will lead to faster growth rates and opening of new cracks.

To maximize the benefit of this treatment, it is necessary to study the extent of cracking in the pavement and the average crack spacing prior to selecting crack sealing as a rehabilitation option. If the pavement surface is more likely to crack under the new conditions, the use of stress relieving joints or soft sealants is recommended. The timing of crack sealing requires careful planning. The sealants can be applied when the crack width is approximately in the middle of its range, therefore reducing the potential for tensile or compressive failure of the pavement when the crack width reaches the maximum or minimum of its range. The tensile and compressive properties of the crack sealant and the pavement layer should be used to optimize the scheduling of this treatment.

4.10 Three-Dimensional Analysis

The three-dimensional (3-D) analysis is invoked to examine the full-depth partial-width cracks. To reduce the size of the numerical problem, the surface layer is modeled in lower-order brick eight-node elements, SOLID45. The zone around the crack tip is meshed in isoparametric 20-node elements, SOLID90. Similar to the 2-D elements shown in Figure (4.4), the solid 20-node element is collapsed to a wedge crack tip quarter-point isoparametric element, suited for modeling the $\sqrt{r}$ singularity in the immediate vicinity of the crack, where r is the distance away from the crack, Figure (4.18). A plan view of the element mesh is shown in Figure (4.19).

4.10.1 Numerical Verification

The accuracy of the proposed 3D mesh is investigated by comparing the handbook solution to calculations of the two independent methods, the two point displacement formula and the energy balance criterion. The case of a central crack in a rectangular sheet is analyzed. The sheet has a height and width of $2b$ each, and contains a central crack of length $2a$ parallel to the edge. It is subject to a uniform far-field tensile stress σ perpendicular to the crack face. The interpolation formula for infinitely long sheets with 1% accuracy is reported in SIF handbook, [Rooke and Cartwright, 1976], as

$$K_I = K_0 \frac{1 - \frac{a}{2b} + 0.326(\frac{a}{b})^2}{\sqrt{1 - \frac{a}{b}}} \quad (4.37)$$

where $K_0 = \sigma(\pi a)^{1/2}$.

Because of the geometric and loading symmetries, only one-quarter of the sheet is modeled using two layers of the 3-D elements. The generated 3-D mesh is shown in

Figure (4.19). The stress intensity factor is obtained at the center of the sheet thickness using the two point displacement formula of equation (4.33), and the J -integral method of equation (4.34). Analysis is performed for crack length increments of 0.1 a/b , and a comparison of the results is shown in Figure (4.20) for the plane stress case and in Figure (4.21) for the plane strain case. The numerical results showed an excellent agreement to the handbook solution. The model can therefore be used in analyzing the thermal cracking problem. The numerical solution could not be extended to areas of thermal gradients, varying stiffness and material properties which are typical applications in pavement related problems. Therefore, only the finite element solutions will be used in analyzing such problems.

4.10.2 Practical Applications of 3-D Model

In this section, a pavement layer with a crack type 4 is examined. The crack extends for the full depth of the overlay and is not connected to the pavement edges. The temperature field is computed using one- or two-dimensional thermal analysis and is applied as a body force to the nodes of the meshed model. Two cases are presented, the first, shown in Figure (4.22), simulates a cracked overlay over a non-cracked original pavement layer. The second, shown in Figure (4.23), simulates a cracked overlay constructed on a cracked original pavement layer. The stress intensity factors for the conditions of plane strain and plane stress are determined using the two-point displacement formula in linear elastic fracture and the J integral for the elastic-plastic fracture.

The difference between displacement and energy methods in the current analysis is

due to the difference in the elevation of the sampling points. The displacement formula employs the nodal (element surface) displacements of the crack tip elements, while the J integral computes the energy balance at the centroid of each element within the path. Due to the limitations of the program and computer capabilities, and the fact that 3-D problems require significantly larger storage space and computing time than 2-D problems, the overlay is meshed into two layers of elements. Hence the vertical location of the centroid is considerably shifted from the surface nodes, causing the difference in the obtained results. Convergence of the two methods may be obtained by reducing the element size used for modeling the surface layer.

These results also illustrate the effect of a cracked underlayer on crack propagation. They show that crack growth at increasing crack lengths is accelerated with the existence of an underlying crack, but is not affected when the underlayer is uncracked. The reasons for this difference is that in the case of uncracked old pavement layer, the tensile stresses are carried by that layer when the surface crack length exceeds 75% of the lane width. If the original pavement layer is also cracked, the tensile stresses in the surface layer lead to a faster crack growth.

4.10.3 Discussion

The preceding applications demonstrate the possibilities of analyzing the pavement structures under conditions of low-temperature and thermal cycling in an efficient and consistent manner. The presented methodology is particularly useful when a pavement reinforcement is considered. Invariably, the properties of the reinforcing material are available, and the

finite element mesh can be modified to include the stiffness of the reinforcement. The effect of the reinforcement on crack growth can be determined by comparing different configurations of the pavement system with or without reinforcement. Clearly, before incorporating the results of the presented model in design procedures, it is necessary to validate the model using laboratory testing or field observations.

From numerical modeling, it is shown that sealing one crack will result in opening more cracks in its vicinity. A different rehabilitation alternative should be used if this condition is encountered. For example, cutting and replacing the cracked area can be more economical in some cases. Also, the timing of crack sealing is very important. Many highway agencies prefer to perform crack sealing during the fall or spring seasons to reduce the range of tensile and compressive stresses at the joints.

The proposed approach presents a finite element method to determine the stress intensity factors, due to temperature regimes in cracked asphalt pavements. The method simulates the thermoelastic response using decoupled thermal and stress analyses. The method provides an accurate representation of thermal loads by performing thermal analysis prior to the stress analysis. The common practice in analytical methods similar to this work, is to apply a unit tensile load or a given thermal gradient. The stress intensity factors are obtained using a displacement formula for special crack tip elements or an energy balance principle. These factors are indicative of the crack propagation during thermal cooling cycles, and can be used to evaluate pavement surface conditions and hence predict the service life. Typical cases of single and multiple cracks are presented. The interaction between adjacent cracks and the effect of base restraint were found to be significant.

The 3-D analysis can be used to identify the severity and progression rate of transverse

cracks that do not fully extend across the pavement lane (for example edge cracks). In this case, the presence of cracks in the underlayers and the lack of sufficient bond between layers effectively reduce the service life of the overlay. It is possible to determine the effect of various parameters on low-temperature cracking using the proposed method.

4.11 Future Use of Finite Element Analysis

The use of finite element analysis in pavement design and management is currently examined on the research scale and not in actual professional practice. The finite element software programs are becoming easier to develop and more user-friendly than before. Finite element procedures developed in the sixties and seventies are now well established in many areas in the field of civil engineering.

Pavement problems, particularly environment-associated ones, carry certain features that prevented the direct application of these programs. First, the material properties in relation to temperature and thermal loading rates are not well established yet, and selecting typical values for these parameters narrows the range of applications. Studying effects of mix additives, air voids or aggregate size on pavement life is not feasible in the current practice. Second, the evaluation of bond between layers and its effect on damage propagation is of extreme importance, since the stress field in pavements is significantly affected by the possible slip between layers. Presently, values of bond ranges from 0% to 100%, and test methods for determining the bond are not established yet. This important factor has been overlooked by the pavement engineers and requires more attention from researchers. Third, the finite element solutions have been an instrumental tool of fracture

analysis and possible applications to thermal crack propagation problems are demonstrated in the thesis. However, establishing the fracture toughness parameters such as K_{IC} and J_{IC} is necessary, preferably from extracted samples of the studied road section. Fourth, the loading rate associated with thermal build up of stresses has to be determined. A correlation between thermally and mechanically applied stresses can facilitate the estimation of finite element material parameters from lab or field tests such as falling weight deflectometer (FWD) or benkelman beam. Finally, there is a large number of parameters in finite element solutions. The solutions presented in this chapter were nondimensional as far as practically possible. However, selection of such parameters as the time interval, minimum and maximum element size and type of element may affect the results. The research presented in this chapter has adhered to the most suitable analysis methods available to the author. The presented results, however, should serve as an example until the model is validated by experimental or field data.

In brief, there seems to be high prospects for using finite element analysis in pavement thermal cracking problems. The proper application requires establishing several relationships to determine input material properties and loading and failure criteria. Determining material properties and establishing laboratory tests, comparative indicators, and parameters will be examined in more detail as part of this research.

Table 4.1: Typical material parameters used in the numerical analysis

Property	Units	Asphalt Overlay	Asphalt Layer	Granular Base	Subgrade
Density ρ	(kg/m^3)	2500	2500	1500	1500
Specific heat c	$(J/kg \cdot ^\circ C)$	600	600	2670	3330
Coefficient of thermal conductivity K	$(J/m \cdot s \cdot ^\circ C)$	1.1	1.1	1.2	1.5
Shear modulus G	$(N/m^2) \times 10^7$	2.25	2.25	1.12	0.45
Poisson's ratio ν	–	0.3	0.3	0.3	0.3
Coefficient of thermal contraction α	$(^\circ C)^{-1} \times 10^{-5}$	1.4	1.4	1.4	1.4

Table 4.2: Effect of base shear modulus on crack growth. $K_o = \alpha G \Delta T a^{\frac{1}{2}} = 2314 \text{ Nm}^{3/2}$

Subgrade shear modulus MPa	Base shear modulus MPa	AC shear modulus MPa	Stress intensity factor K/K_o	Remarks
4.50	0.00	22.50	6.700	very weak base
4.50	0.90	22.50	6.130	weak base
4.50	4.50	22.50	4.430	weak base
4.50	11.20	22.50	3.678	reference case
4.50	22.50	22.50	3.507	stiff base

Table 4.3: Effect of subgrade shear modulus on crack growth. $K_o = \alpha G \Delta T a^{\frac{1}{2}} = 2314 \text{ Nm}^{3/2}$

Subgrade shear modulus MPa	Base shear modulus MPa	AC shear modulus MPa	Stress intensity factor K/K_o	Remarks
0.00	11.20	22.50	4.231	very weak subgrade
0.90	11.20	22.50	3.896	weak subgrade
4.50	11.20	22.50	3.678	reference case
11.20	11.20	22.50	3.580	stiff subgrade
22.5	11.20	22.50	3.502	stiff subgrade

Table 4.4: Effect of temperature gradient on stress intensity factor at the crack tip. The thermal gradient is the difference between prescribed temperatures at the pavement surface and that at a depth of 2 m. $K_o = \alpha G \Delta T a^{\frac{1}{2}} = 2314 \text{ Nm}^{3/2}$

Surface temperature °C	Temperature at -2.00 m °C	Thermal gradient °C/m	Stress intensity factor K/K_o	Remarks
-10	0.0	-5	1.226	mild temperature drop
-30	0.0	-15	3.678	reference case
-50	0.0	-25	6.131	severe temperature drop

Table 4.5: Effect of the depth of the main crack on crack growth. Secondary crack depth = 60 mm. Secondary crack spacing = 3000 mm. $K_o = \alpha G \Delta T a^{\frac{1}{2}} = 2314 \text{ Nm}^{3/2}$

Depth of main crack	Original crack K/K_o	New crack K/K_o	Remarks
0 mm	–	3.694	
20 mm	2.068	3.692	
40 mm	2.891	3.687	
60 mm	3.678	3.678	reference case
80 mm	4.495	3.664	
100 mm	5.232	3.643	full depth

Table 4.6: Effect of crack spacing S on stress intensity factor K . $K_o = \alpha G \Delta T a^{\frac{1}{2}} = 2314 \text{ Nm}^{3/2}$

Depth of cracks	K/K_o				Remarks
	$S = 1000 \text{ mm}$	$S = 3000 \text{ mm}$	$S = 6000 \text{ mm}$	$S = \infty$	
20 mm	2.079	2.098	2.090	2.092	
40 mm	2.853	2.949	2.919	2.923	
60 mm	3.559	3.678¹	3.711	3.719	
80 mm	4.059	4.590	4.525	4.547	
100 mm	4.269	5.101	5.249	5.291	full depth

¹ reference case

Figure 4.1: Boundary conditions of the thermoelastic problem

Figure 4.2: Pavement structural model: (a) Pavement relative dimensions (b) Boundary conditions and imposed surface temperature loading

Figure 4.3: Pavement-surface-temperature model used in the FEA

(a) Typical 8-node parabolic isoparametric element, 8 DOF thermal or 16 DOF Structural

(b) Quarter point element (Crack tip) formed by collapsing the side of 1, 4 and 8, and relocating midside nodes 5 and 7.

Figure 4.4: Selected element type and crack tip element of the two-dimensional analysis

Figure 4.5: Procedures of thermal and mechanical FEA using ANSYS

Figure 4.6: Comparison of temperature fields obtained by analytical method (Dettman 1969) and FEA method in a typical 24 hour period

Figure 4.7: Comparison of daily transient temperature gradients obtained by analytical method (Dettman 1969) and FEA method

Figure 4.8: Selected two-dimensional mesh for the single and multiple cracking problems

Figure 4.9: Detailed two dimensional mesh arrangement at the crack tip showing crack tip elements and crack face

Figure 4.10: Path independent J integral around the crack tip

Figure 4.11: Relationship of crack depth, x/a , and the stress intensity factor during a cooling cycle. x = crack depth, a = layer thickness, and $K_o = \alpha G(T_f - T_o)a^{1/2}$

Figure 4.12: Effect of crack depth, x/a , and loading time on the stress intensity factor. x = crack depth, a = layer thickness, and $K_o = \alpha G(T_f - T_o)a^{1/2}$

Figure 4.13: Configuration of the multiple cracking problem

Figure 4.14: Stress intensity factor at the main crack. $K_o = \alpha G(T_f - T_0)a^{\frac{1}{2}}$

Figure 4.15: Reference case for the sensitivity analysis

Figure 4.16: Effect of base and subgrade shear moduli on the stress intensity factor at the original crack. $K_o = \alpha G \Delta T a^{\frac{1}{2}} = 2314 \text{ Nm}^{3/2}$

Figure 4.17: Effect of crack spacing on the stress intensity factor at the original crack. $K_0 = \alpha G \Delta T a^{\frac{1}{2}} = 2314 \text{ Nm}^{3/2}$

(a) 8-node brick element, having
8 DOF thermal or 24 DOF structural

(b) 20-node isoparametric element, having
20 DOF thermal or 60 DOF structural

(c) Quarter point wedge element (Crack tip)
formed by collapsing one face of the 20-node
element and relocating midsides 10, 12, 18,
and 20.

Figure 4.18: Element type and crack tip element in the three dimensional analysis

Figure 4.19: Three-dimensional finite element mesh used in modeling crack type 4

Figure 4.20: K_I , in plane stress, obtained from displacement formula, energy balance and handbook solution (Rooke and Cartwright 1976), $b/h=1.0$, $K_0 = \sigma(\pi a)^{1/2}$

Figure 4.21: K_I , in plane strain, obtained from displacement formula and energy balance, $b/h=1.0$, $K_0 = \sigma(\pi a)^{1/2}$

(a) Plane strain.

(b) Plane stress.

Figure 4.22: Stress intensity factor, K_I for crack type 4 and the original pavement layer is *uncracked*

(a) Plane strain.

(b) Plane stress.

Figure 4.23: Stress intensity factor, K_I for crack type 4 and original pavement layer is *cracked*

Chapter 5

Laboratory Testing Program

This chapter deals with the preparations for the testing program, the asphalt mix properties, the three types of tests that were performed, and test variables.

5.1 Introduction

Transverse cracks may develop in the top or bottom surfaces of the asphalt layer and then penetrate the layer thickness, or alternatively they may develop at the pavement edge or center of a lane and extend laterally across the lane. A laboratory investigation for assessing the severity and propagation rates of the four distinctive types of transverse cracks is presented in this chapter. An accelerated testing approach for resistance to thermal fatigue cracking is developed. The method specifically focuses on measuring the performance of asphalt mixes under repeated loading through the mechanical properties that directly affect thermal cracking. The laboratory testing program is developed to:

1. Simulate actual thermal conditions and correlate results to mechanical loading conditions.
2. Determine material parameters, such as the stiffness and tensile strength required for the analytical study.

3. Evaluate existing testing procedures from the standpoint of thermal cracking and develop a uniform testing procedure.
4. Study some particular aspects of asphalt concrete pavement conditions such as direction of rolling, crack location, and crack growth rate.

Thermal cracking occurs under two different conditions: (1) a severe temperature drop that causes thermal strains to exceed the material limiting tensile strain, and (2) cyclic temperature changes within the plastic zone that causes irreversible damage which causes the loosening of aggregate particles and eventually cracking after a relatively low number of cycles. It is hypothesized that damage indicators obtained from cyclic loading data can be used to predict the structural condition of the pavement and its remaining life. Since there is no clear consensus on which damage indicator is more appropriate to the case of thermal cracking, a comparison of different indicators will be presented.

5.2 Test Objectives

In designing the testing program, it is important to satisfy the following set of objectives:

1. It is required to simulate actual field conditions as closely as possible. Test samples will be saw-cut from field extracted slabs and loaded in direct tension.
2. Failure can be induced by monotonic loading to simulate extreme low-temperature or by cyclic loading to simulate thermal fatigue. The test method must be capable of modelling the two cases.

3. Temperature cycling is costly and time consuming, since each thermal cycle requires more than two hours to establish isothermal conditions across the test sample. In accelerated laboratory tests, thermal cycling can be substituted by mechanical (displacement controlled) loading. The benefits are a faster and less expensive test method with precise temperature control within the samples. The shortcoming is that the change in material strength due to changes in temperature is not considered. However, if the tests are performed at sufficiently small temperature intervals, the effect of thermal changes can be obtained.
4. Potential to accommodate field samples from large stone mixes and reinforced pavements.
5. Complex test procedures should be avoided in order to minimize the variability in the results and increase the chances of successful repetitions.
6. Economic cost of the testing equipment.

It is noted that SHRP test methods such as bending beam rheometer, TSRST, and indirect tensile strength test are not suitable for performing the tests since it will not be possible to simulate crack propagation or load cycling.

5.3 Test Selection

A comprehensive study on testing fatigue properties of asphalt pavements introduced a comparison of eleven test methods [Matthews et al., 1993]. These test methods included; repeated flexure test, direct tension test, diametral repeated load, and dissipated energy

method. The comparison of the eleven methods have concluded that repeated flexure test and direct tension test are ranked first based on application of test results, advantages and disadvantages, simulation of field conditions and simplicity. For low temperature cracking, SHRP examined several testing methods including indirect diametral tensile test at low temperature (ITLT), direct tensile test, tensile creep test, flexural bending test, thermal stress in restrained specimen test (TSRST), and coefficient of thermal contraction test.

While the TSRST is selected by SHRP as the test method for examining mixture thermal cracking potential, this test does not suit the objectives of the present study for the following reasons. First, sufficient accuracy cannot be achieved during this test because of its extremely low level of strains. Furthermore, the test cannot be used to model crack propagation or on large field specimens. Also, the objective of this research is to replicate field loading and boundary conditions, as far as practically possible. For these reasons, the direct tensile test is selected instead of the TSRST. However, the test setup is different from that examined by the Asphalt Institute and SHRP [Bolzan and Huber, 1993]. In the proposed direct tensile test set-up, a larger size specimen can be used, up to 300 x 600 x 100 mm, providing higher uniformity in the specimens and the ability to use field extracted specimens. Asphalt with geogrid reinforcement, bond between layers, and crack configurations can also be tested.

While low temperature cracking is best modelled by a single loading cycle up to failure, the propagation of cracks in a pavement layer is affected by thermal cycling which is represented with cyclic tensile loading. The test method should reflect the nature of the slow rate of thermal loading and that failure occurs under relatively low number of cycles.

5.4 Test Facility

Based on the requirements of the previous section, the test facility shown in Figures (5.1,5.2,5.3) was designed and assembled at the Civil and Environmental Engineering Laboratory of Carleton University. The facility consisted of:

1. An environmental chamber with a temperature range of +60°to – 40°C.
2. A hydraulic actuator with 100 kN capacity and 250 mm stroke.
3. A direct tensile strength testing table equipped with fixed and moving mounting plates.
4. The moving plate is connected to a load cell and a positioning system consisting of a linear variable displacement transducer (LVDT) that feeds back to a data acquisition computer.
5. The moving plate is placed on a Teflon sheet to reduce friction forces and the load cell is adjusted accordingly to eliminate the loads required to move the plate and resist friction.
6. Computer, data acquisition system, and programs for monitoring and control of the test equipment.

The failure is restricted to the middle third of the sample and end effects are minimal, which cannot be guaranteed in the TSRST method and the test is also applicable to reinforced pavements and large stone mixes. Figure (5.4) illustrates the crack development during a fatigue test, and Figure (5.5) shows the failed sample after a direct tension test. The test facility is capable of performing monotonic and cyclic tests at very slow rates of loading up to 0.001 mm/s as well as at higher frequencies up to 60 Hz.

5.5 Testing Program

Table 5.1 shows the three testing procedures that are employed in this testing program. The first is the direct tensile test performed at warm and cold temperatures, the second is the fatigue test on notched samples, and the third is the constant speed cyclic test.

5.5.1 Direct Tension Test

Stated simply, a uniform displacement rate is applied to the base of a moving plate at one end of the sample while the other end is restrained. The displacement is controlled with an LVDT while a load cell is used to measure the tensile load on the sample. The test is performed in the temperature-controlled chamber, and can be applied to field extracted slabs or lab prepared prisms. It is useful for determining the tensile strength as well as the stiffness and limiting strain of any given mix.

5.5.2 Tensile Fatigue Test

The tensile fatigue test is similar to the direct tensile test except that a haversine displacement is applied. Notched samples can be used to determine the rate of crack propagation and to compare the crack patterns, Figure (5.6). Due to the inelastic nature of the asphalt, the load cycling affects the displacement rate of the crack tip in the loading and return cycles. This test produces the S-N curve; stress level versus number of loading cycles at a constant temperature, Figure (5.9).

5.5.3 Constant Speed Cyclic Test (CSCT)

Prediction of thermal fatigue life is often based on the more established isothermal material behavior. It is generally recognized that accuracy of thermal fatigue predictions is considerably poorer than the corresponding isothermal predictions, [Halford, 1987]. It should be also noted that the preceding statements are based on high temperature conditions in metals where the temperature range is between 300 and 1000 °C. The problem is further magnified by low temperature conditions and relatively thick and less conductive asphalt sections. The drawbacks of isothermal testing are that it cannot capture the differential thermal expansion between the various components and the change in strength and build up or decay of strains during thermal cycling.

A testing method for the isothermal cyclic loading is implemented. The purpose of the test is to apply cyclic loads on asphalt samples within the elastic and inelastic ranges. The test focuses on the dissipated energy per cycle and the shape of individual loading cycles [Shalaby et al., 1996d, and 1997]. The formation of thermal cracks requires a relatively low number of thermal cycles (< 10000 cycle). Therefore, it can be hypothesized that the failure will conform to the low-cycle fatigue mode, where large inelastic strains (in the order of elastic strains) lead to a form of premature failure. It is interesting to note that, contrary to typical fatigue tests, it is more rational to perform the test at a constant speed rather than a given frequency. The material is very sensitive to the rate of loading specially at the low strain rate of thermal cycling. The test variables are: the sample temperature which is maintained constant during the test, the displacement amplitude, and the strain rate. The displacement amplitude and strain rate are selected to cause some plastic deformation

in the samples but remains appreciably below the failure strain. In the present study, the displacement amplitude is about 50% of the failure strain computed from direct tension tests at the given test temperature.

5.6 Performance Indices

The study of damage propensity in asphalt pavements can be conducted through the analysis of material strength and resistance to cracking. Obviously, a material which has little or no resistance to loading can be assumed separated or “cracked” under typical thermal deformation conditions. Secondly, since the aim is to examine the damage due to cyclic loads, the strength parameters obtained from the first loading cycle are the reference for any material degradation that occurs thereafter. The damage to the test sample can be monitored through the change in one of the performance indices with each loading cycle. The selection of an appropriate performance index is made according to the test objectives and type of application.

Under strain controlled uniaxial cyclic loading, the following performance indices are found relevant to material resistance to transverse cracking:

1. Maximum tensile and compressive stresses: the decrease in the maximum tensile stress during load cycling is a measure of the decay of the material. In cyclic tensile tests, the compressive stresses are indicative of plastic deformation and matrix rearrangement during loading. The increase of compressive stresses is a measure of the resilience and hardening of the material.
2. Stress drop: in strain controlled tests, the difference between maximum and minimum

stresses decreases with load cycling. Since the displacement is fixed, the drop in stresses reflects an equal drop in the secant stiffness modulus.

3. Strain energy: strain energy is a very favoured parameter since it can capture both the elastic and plastic deformations. Results are compiled from several readings in each cycle which is more meaningful and precise than selecting the highest or lowest reading. In the CSCT test of the present study, 100 readings of load and displacement are recorded on the data acquisition computer for each cycle. Strain energy calculations are produced for loading and unloading zones.
4. Stiffness modulus: several stiffness moduli can be computed. The initial stiffness and the unloading stiffness on each cycle are the most significant. Due to the large applied strains, it is not recommended to use a single stiffness indicator since it will not reflect the inelastic behaviour.
5. Permanent deformation: the intercept of the unloading curve with the horizontal axis defines the permanent deformation on each cycle. This factor correlates to the amount of irrecoverable damage incurred to the test sample.
6. Number of cycles: the number of cycles to failure is not a good indicator since it can be affected by the various material and test conditions. In the absence of a clear failure criterion, the variability and repeatability of test results is questionable. In low cycle fatigue, it is not advisable to use this number as a performance measure since all tests will produce fairly consistent results within the same order of magnitude.
7. Hysteresis loop: the information gathered from the test includes the range of elastic deformation and the onset of plastic deformation, the tangential stiffness moduli, the

comparison of loading and unloading paths, permanent deformation, and the shape of the loop.

8. Other indicators: nondestructive test methods such as X-ray and ultra sound waves can be used to evaluate the material properties.

A detailed description and ranking for suitability of these indicators is available in the literature [Lemaitre, 1992].

5.7 Materials

Two asphalt mixes are used in this study. Both mixes are designed to meet HL3 aggregate gradation as specified by the Ontario Ministry of Transportation. The aggregate gradations are shown in Figure (5.7) with the control points and restricted zone recommended by Superpave. Large aggregates of both mixes were local HL3 stones. It is noted that the gradation should avoid the restricted zone to improve rutting resistance by enforcing the S-shape gradation. This restriction has not been found relevant to thermal cracking. Aggregates and binders for both mixes are obtained from independent and unrelated sources. Mixing and compaction were conducted in the field by two local paving contractors.

A summary of mix properties is presented in Table 5.2, while the mix design reports are included in Appendix B. From the shown properties, it is reasonable to assume that both mixes have relatively stiff binders, and have similar aggregate type, and air voids ratio. The binder of Mix A is specified according to the older penetration based method and that of Mix B is specified according to SHRP performance grades. The two mixes differ mainly in aggregate source, density, and gradations. Stability and flow of the two mixes produced

comparable results. It is also reasonable to assume that the two mixes should produce very consistent performances with respect to thermal cracking resistance, if analyzed based only on the Marshall test results.

5.8 Experimental Procedures

5.8.1 Mix Preparation and Compaction

Test samples are retrieved from asphalt concrete mixed and compacted in the field. The asphalt concrete mixes are laid into 1.2m x 2.4m forms at the paving site and compacted with eight passes of a half-ton steel roller to achieve a pavement thickness of 75 mm and left for seven days to cure on site. After 7 days, each slab is saw-cut to manageable dimensions, properly marked, and transported to the laboratory. Slabs are then saw-cut to actual test sizes of (150-250) x 450 x 70 mm. The marking scheme kept track of sample location during compaction, and the direction of rolling is marked on each sample.

5.8.2 Preparation of Test Slabs

Each test sample is glued to two 10 mm steel plates at room temperature. The plates are aligned and positioned using a set of two removable side handles. A 24-hour two-component epoxy glue is used to bond the asphalt samples and the steel plates. The sample surface area in contact with the glue exceeded twice the area of the cross section. Since the shear strength and tensile strength are approximately equal and the shear stresses never exceeded one half of the tensile stresses, the failure is dominated by tensile stresses.

A dummy specimen is placed in the environmental chamber with the test sample. A thermocouple is connected to the center of the dummy specimen to monitor the temperature and establish isothermal conditions before the starting of the test. After completing the test, the steel plates are heated to 175°C for one hour to facilitate the removal of the epoxy glue and the asphalt.

5.8.3 Test Variables

Samples are conditioned at the desired test temperature for 24 hours prior to the actual testing. The test temperature is set to -20°C, 0°C, and +20°C. Direct tensile tests (section 6.1) are performed at two loading rates 0.3 mm/min and 0.6 mm/min. The test is continued until visible failure or separation has occurred or the total displacement is greater than 10 mm.

In cyclic tests designed for comparing crack types (section 6.2), the loading cycle is triangular, strain-controlled with a base opening displacement equal to 0.5 mm, 1.0 Hz frequency, and no rest periods. It should be noted that the rapid load cycling (one cycle/second) may not accurately represent thermal loading cycles which vary in duration between a few hours to more than ten days. However, the high frequency of displacement is required for accelerated test conditions and should be correlated to the actual field conditions. In CSCT test (section 6.3), the base displacement rate r is equal to 6.0 mm/min. The load cycle is strain-controlled and the base displacement amplitude (a) ranges from 0.5 to 2.0 mm. The load frequency f , in Hertz, is computed by

$$f = \frac{r}{60 \times 2 \times a} \quad (5.1)$$

5.9 Summary

Laboratory test methods that simulate field conditions of pavements subjected to thermal stresses are developed. The tests also examine the effects of “roller checking” on tensile strength and strain energy of asphalt paving mixtures at different temperatures, and establishes an accelerated testing methodology. Asphalt samples are tested in the temperature range of -20°C to $+20^{\circ}\text{C}$. The tests are performed on slab specimens having approximate dimensions of 150 x 450 x 75 mm. Each sample is glued to a set of moving and fixed plates using high strength epoxy. A forced monotonic or cyclic displacement is applied in the horizontal direction only. Three groups of tests are performed until failure; direct tensile test at a rate of 0.60 mm/min, cyclic tensile test at 1.0 Hz on the four crack types, and constant speed cyclic test at 6.0 mm/min. The test results are discussed in the next chapter. The research revealed that the proposed crack classification is indicative of the crack severity and can be further extended to assess pavement condition and deterioration rates with higher accuracy. Also, the utilized laboratory tests are capable of comparing and cataloging the low-temperature behaviour of different paving mixtures and rehabilitation alternatives.

Table 5.1: Comparison of performed tests and expected results

TESTING PROGRAM			
Test	Direct tensile test (DTT)	Tensile fatigue test	Constant speed cycle test (CSCT)
Description	Direct uniaxial tensile test at very slow strain rate	Cyclic tensile loading of pre-notched slabs	Low cycle tensile fatigue at isothermal conditions
Asphalt mixtures	A, B, C, D	A	A, B
Test temperature	+20, 0, -20 °C	+25 °C	0, -20 °C
Strain rate	0.005 or 0.010 mm/s	1.0 mm/s, no rest period	0.10 mm/s - 0.025 Hz, no rest period
Maximum Strain	Up to failure	0.0025	0.010
Wave form	Uniform	Sinusoidal	Uniform
Test outputs	Load-displacement relationships	Load vs. No. of cycles (S-N curves)	Hysteresis loop, Damage evolution, Energy dissipation, S-N curves
Test variables	Direction of rolling	Four crack positions	Strain amplitude
No. of tests	20	10	15
Expected test duration	One hour	1 to 8 hours	1 to 8 hours

Table 5.2: Design material properties of Mix A and Mix B

Property	Mix A	Mix B
Asphalt cement content	5%	5%
Binder grade	85/100 (unmodified)	PG 58-34 (unmodified)
Air voids	4.3 %	4.1 %
Voids in mineral aggregates (VMA)	16.1 %	15.2%
Bulk relative density (BRD)	2.436 gm/cm ³	2.393 gm/cm ³
Stability	13100 N	12262 N
Flow	8.9	9.2
Date of construction	December 2nd, 1995	May 23, 1996
Air temperature at construction site	-10 °C	16 °C

Figure 5.1: Setup for direct tensile and cyclic tests

Figure 5.2: Photograph of the laboratory test facility

Figure 5.3: Photograph showing the test sample setup

Figure 5.4: Crack development during fatigue testing

Figure 5.5: Failed sample after direct tensile test at 20°C

(a) Elevation of Direct Tensile Testing Machine
Showing Slab Arrangement for Crack Type 1

(b) Plan of Direct Tensile Testing Machine
Showing Slab arrangement for Crack Type 3

Figure 5.6: Fatigue testing of precracked specimens

Figure 5.7: Gradation of tested mixes with respect to Superpave design

Figure 5.8: Stress strain curves from direct tensile test results

Figure 5.9: Typical S-N curve plotted from fatigue test results

Figure 5.10: Loading and unloading energies, hysteresis loops and permanent deformation from repeated tensile strain test

Chapter 6

Laboratory Test Results

This chapter presents and discusses the results of the three laboratory tests performed in this study. These are; the direct tensile test, the tensile fatigue test, and the constant speed cyclic tensile test. Eleven groups of tests are conducted in this investigation according to the definition given in Table 6.1.

6.1 Direct Tensile Test

6.1.1 Results and Analysis

The direct tensile test is performed on saw-cut slabs from the two mixes. The samples are conditioned at the test temperature for at least 24 hours prior to testing. Typical test results from four groups of tests are shown in Tables 6.2, 6.3, 6.4, and 6.5. As shown previously in Figure (5.8), failure strains are those associated with the maximum stress. The stress is calculated by dividing the load by the gross section area. The tangential and secant stiffness moduli are computed. The strain energy is calculated as the area under the stress strain curve divided by the sample volume to obtain the strain energy density per unit volume in MPa. Stress strain curves are plotted in Figures (6.1, 6.2, and 6.3) for the mean of each two replicate samples. The test results show the distinct material response to the change

of temperature from +20°C to 0°C and -20°C. The optimum performance was obtained at 0°C where the ductility of the asphalt concrete is sufficient to prevent failure at very high tensile strains up to 0.02. In cases where the tensile strength is higher at -20°C than that at 0°C, failure occurred at a lower strain indicating the increased brittleness of the samples.

Two base displacements are used on Mix A as noted in Table 6.1. Figure (6.4) illustrates the comparison between the two strain rates. It is noted that at high temperatures, for example 20°C, the effect of the strain rate is minimum. This conclusion stresses the sensitivity of the results to the strain rate even at lower temperatures where the material is presumably more elastic. The same conclusion applies to TSRST results where the temperature drop rate is generally 10°C/hour in the present SHRP specification, SuperPave 1. A rate of 1 °C/hour is recommended by SHRP in more detailed performance studies, such as SuperPave 2 and 3.

At +20°C, failure occurs gradually and only through the binder as a mode of material softening while failure occurred abruptly and through the aggregates in the frozen samples at -20°C. The samples tolerate very high strains at 0°C, and the failure is more gradual indicating possibilities of aggregate rearrangement and high bond strength (adhesion).

Based on visual observations of the tests, the following factors are significant to low-temperature performance: 1) using high quality aggregates to resist thermal stresses at extreme low temperatures, 2) the strain rate appears to be a sensitive parameter and should be correlated to actual field conditions, and 3) the orientation of the aggregates in laboratory prepared samples should be representative of field conditions. For example, Skabova et al. (1996) note that Gyratory compaction influences the aggregate orientation and hence the tensile strength at low temperature based on indirect tensile test results. A comparison of

summary test results is presented in Table 6.6. The results in Tables 6.2 to 6.6 supported the following:

1. With the exception of strain energy, each of the measured properties is based on a single recorded measurement. Therefore, the reliability of these readings is lower than that of the strain energy which incorporates the full stress strain curve.
2. Effect of test temperature: temperature is the single most influential factor on the material response. At lower temperatures, significant increase in the tensile strength and tangent and secant moduli is observed. The strain energy in Mix A is optimum at 0 °C and decreases afterwards due to loss of ductility. In comparison, strain energy in mix B is not noticeably affected by the lower temperatures.
3. Effect of loading rate: from groups I and II, the increase of loading rate from 0.005 mm/s to 0.010 mm/s results in higher tensile stresses and strain energy below 0°C. In effect, the viscoelastic response remains a critical factor at low temperatures and elastic response alone will not be sufficient to characterize the material behaviour below the freezing point.
4. Mix Performance: from groups II and III, Mix B has higher tensile strength and strain energy at +20 °C where Mix A is superior at 0 and -20 °C. More significantly, Mix A has higher ductility and therefore resistance to thermal cracking at all temperatures.
5. Effect of the direction of rolling: the comparison is presented in section 6.1.3.

According to direct tensile test results, mix A is preferred for having higher ductility and resistance to cracking. Although Mix B exhibited lower resistance at cold conditions, its tensile stiffness remained high which led to numerous brittle failures. The low resistance

may not be attributed to the use of a soft binder, but also to a mix with low quality aggregates or low adhesion.

6.1.2 Comparison of Mixture Performances

To illustrate the use of this testing procedure in assessing the mixture performance, the behaviour of the two asphalt mixes “A” and “B” is compared at the three test temperatures. The following conclusions are drawn from stress strain curves shown in Figures (6.5, 6.6, and 6.7): (1) Mix A has higher strength at 0 and -20°C, (2) Mix B has higher strength at +20°C, (3) Mix A has higher ductility at low temperature, and (4) Mix A is preferred in designing against thermal cracking.

6.1.3 Effect of Direction of Rolling

Surface cracks are often initiated during the compaction process due to the mismatch between the rigidities of the compactor and the soft asphalt mix, a phenomenon which is often termed as roller checking, as shown by Abd El Halim (1985) and (1986). The rigidity is a term defined by the ratio of the geometries and moduli of the system. The asphalt layer is soft and flat during construction while the compactor has a high stiffness and a circular shape. Kanerva et al. (1993) found that the presence of these cracks did not affect the fracture temperature based on the results of a limited testing program. In the present program, it was possible to test samples of larger size and having the full layer depth to examine the reported phenomenon. Although the samples are field compacted, they were not subjected to traffic or freeze and thaw cycling and therefore would only reflect the effect

of construction cracks on the tested samples. It is evident that service conditions would accelerate the damage associated with the presence of these surface cracks.

In the present study, monotonic tensile loading tests are performed on mix B samples loaded parallel or transverse to the direction of rolling. Tests are performed at +20°C, 0°C and -20°C at a loading rate of 0.01 mm/sec (0.60 mm/min) until failure. Two replicates of each sample were tested with a total of 12 tests, Figure (6.8). The peak stress (tensile strength) is recorded and the area under the stress-strain curve is calculated as a strain energy density, as shown in Table 6.7 and Figures 6.9 and 6.10.

The results show a significant difference between identical pairs of samples based on the direction of loading only. The pavement exhibited higher strength in the transverse direction, the increase in tensile strength was between 10.5 % and 21.1%. The decrease in strength can be explained by the presence of construction induced cracks. These cracks are formed because the pressure and curvature of the drums of the roller develop a deflection basin that can create construction cracks in the transverse direction, thus reducing the load carrying capacity in the longitudinal direction, Figure (6.11).

The results of strength and strain energy presented in this section are not conclusive, and the following three factors should also be considered in a more detailed analysis:

1. The present samples are taken from new constructed pavements and are not subjected to traffic or thermal cycling.
2. The adopted test method is monotonic and uniaxial, while field observations indicate the presence of repeated loads and multiaxial or general state of stresses, and
3. In-depth analysis is needed to study the shielding effect between the shallow surface

cracks that may alleviate the stresses on deeper cracks and delay failure as discussed earlier in the analytical study in Section 4.9.2.

6.2 Comparison of Crack Types

Identical samples are loaded in tensile fatigue. Samples representing the four crack types and uncracked samples are used in this test. The comparison is based on the number of cycles required to cause failure in the specimens. Each set of tests consisted of two identical slabs. The tests are performed at a temperature of 25°C, and test results are shown in Figures (6.12) and (6.13). The rate of loading equals 1 Hz with an average rate of extension of 1 mm/s. The crack depth is varied so that the net uncracked area is equal to 60% of the original cross section area in all precracked samples. Due to the inelastic nature of the asphalt at this warm temperature and the rapid rate of loading, maximum compressive stresses in the return strokes are as high as the maximum tensile stresses although the sample is not compressed beyond its original unloaded state.

Failure is defined by the drop in tensile stresses in the sample below 0.1 MPa which is about 30% of the initial strength at this temperature and rate of loading. From the results, Type 2 bottom cracks are more prone to propagate than Type 1 surface cracks when the load is applied at the bottom of the layer. Also, edge cracks are more likely to propagate faster than in-lane cracks. The remaining life ratios estimated from the normalized number of cycles in the investigated cases are given in Table 6.8. The same problem was analyzed earlier in Chapter 4 using the finite element method. It should be noted that the analytical model represented the entire pavement structure under thermal loading cycles while the

laboratory test samples are subjected to tensile strains at isothermal conditions.

6.3 Constant Speed Cyclic Test (CSCT)

Results of CSCT samples are presented. The testing program includes 12 samples as shown in Table 6.1. Samples from Mix A are tested in group X while samples from Mix B are tested in group XI. For a successful completion of a test, the results must show propagation of damage during load cycling. Each test started with a new damage-free sample. Several tests at low temperatures failed during the first few load cycles in a brittle fashion. The maximum strains are reduced and the test is repeated on new samples. The numerical results record the cycle number and 100 equally-spaced pairs of load and displacement readings from each cycle. The analysis of results is performed using the mathematical software "Matlab". Special functions are developed to analyze the data, obtain the maximum and minimum recorded stresses, and compute the permanent deformation and the strain energy of each cycle. In addition, Matlab is used to produce all the plots of the experimental analysis.

Lemaitre (1992) notes that the peak stress drop is a good measure of damage evolution in low cycle fatigue. The drop in the peak tensile stress is caused by crack growth through the sample. In this application, the dissipated energy per cycle and the drop of stress amplitude are selected as a measure of damage evolution and hence crack growth.

6.3.1 Test Results

The stress strain relationship at selected cycles is illustrated in Figure (6.14). The decay of material stiffness is evident from the figure and can be characterized by the strain energy, the permanent deformation, or the drop in maximum stresses. The strain energy per cycle $E(i)$ is the area enclosed in the hysteresis loop i , Figure (6.14). The strain energy is separated into loading and unloading energies, which are the areas under the loading and the unloading stress strain curves as previously illustrated in Figure (5.10). The accumulated damage $D(i)$ at cycle i , as shown in Figure (6.15), is the decay of the total strain energy with the maximum value of one corresponding to failure, and is computed by

$$D(i) = 1.0 - \frac{S.E.(i)}{S.E.(1)} \quad (6.1)$$

where

$S.E.(i)$ = the strain energy of cycle (i) , and

$S.E.(1)$ = the strain energy of the first loading cycle, assuming it is not yet affected by load cycling.

From examining the failure surface, it is noted that black surface conditions prevail indicating failure through the binder only with few exceptions where failure occurs through the aggregates. If the thermal contraction coefficient of the test sample is approximately $2.5 \times 10^{-5}/^{\circ}\text{C}$, then a maximum imposed strain of 0.1% will be equivalent to a temperature change of -40°C in fully restrained conditions. The strain amplitude is relatively high considering that stresses from seasonal temperature variations will be relieved with time, and the pavement will only experience thermal fatigue in the order of daily temperature changes of $\pm 20^{\circ}\text{C}$. From Figures (6.15) and (6.16), it is shown that failure occurred

abruptly after about 50 cycles as indicated by the sudden change in strain energy and that of maximum stresses.

It is feasible to use the CSCT and similar forced displacement tests to characterize and catalogue low temperature response of different asphalt mixtures. Franchen and Vanelstraete (1993) performed a similar test using a two-hour cycle on reinforced and plain samples with 1 mm displacement.

Figures (6.17) to (6.26) summarize the test results. Each figure represents one successful test and consists of the following five plots:

1. Stress versus number of cycles:

The plot displays the maximum recorded stresses in each cycle. Since the test is displacement-controlled in the plastic strain range, there is a noticeable amount of compressive stresses that occurs in the return stroke.

2. Strain energy versus number of cycles:

The plot illustrates the absolute loading and unloading strain energies computed in each cycle. The energies represent the areas under the respective loading and unloading stress strain curves. Since the tests impose tensile strains only, it is expected that the loading energy is consistently higher than the unloading energy. The amount and rate of change of strain energy are useful in detecting propagation of damage.

3. Permanent deformation versus number of cycles:

The plot displays the increase in the absolute permanent deformation in (mm). The permanent deformation indicates the amount of plastic strains imposed in the test. It is noted that the unsupported sample length equals 200 mm in all the tested samples.

4. Damage versus number of cycles:

The plot shows two damage propagation indices. The damage ratio ranges from 0 to 100% and is measured by the drop in maximum stresses or by the change in dissipated energy. The damage is computed as a ratio of stress drop (or dissipated energy) of each cycle to the maximum recorded values, typically in the first loading cycle. The stress drop index is the change in the range between maximum compressive and tensile stresses with each new load cycle, and the dissipated energy index is the rate of change of the sum of loading and unloading energies.

5. Hysteresis loops:

The last plot in each figure displays the stress strain relationship. Due to the large number of cycles, selected cycles at a reasonable interval are extracted from the raw data and plotted. The shape of the hysteresis loop is considered a fingerprint of the material response.

6.3.2 Test Group X

The tested samples in group (X) are from mix "A", and the CSCT tests are performed at 0.0 and -20°C. The results of CSCT at +0°C and at a low strain (0.5%) are shown in Figure (6.17). The effect of applying higher strains (1.0%) at the same temperature is shown in Figures (6.18) and (6.19). At the low strain, the sample showed 30% damage at 600 cycles where at the higher strain, both samples failed within 100 cycles of loading. The results of Figures (6.18) and (6.19) represent two tested replicates and show the consistency and repeatability of the test since the figures correspond to two identical samples.

At -20°C, it was not possible to reach the 1.0% strain under load cycling, although

previously achieved in direct tensile tests. The maximum strain is reduced to 0.25% and 0.5% as shown in Figures (6.20) and (6.21). The elastic behaviour is obvious in the shape of the hysteresis loop as well as the strain energy. In both cases no appreciable damage is observed, however, at a strain of 0.6% failure occurs abruptly in the first 5 cycles. There appears to be a limiting strain at -20°C. The mix can tolerate a large number of cycles above the limiting strain, while a brittle collapse occurs below the limit.

6.3.3 Test Group XI

All tested samples in group (XI) are from mix "B" and the tests are carried out at 0 and -20°C. The results of CSCT at +0°C and low strain (0.5%) are shown in Figure (6.22). The effect of increasing the maximum strains to (1.0%) at the same temperature is shown in Figures (6.23) and (6.24). At the low strain, the sample showed 25% damage at 350 cycles where at the higher strain, the samples failed at 140 and 180 cycles. Figures (6.23) and (6.24) represent two replicates having the same test variables to illustrate the repeatability of the test.

At -20°C, it was not possible to reach the 1.0% strain under load cycling, although previously achieved in direct tensile tests. The maximum strain is reduced to 0.2% and to 0.5% as shown in Figures (6.25) and (6.26). In both cases no appreciable damage is observed, however, at a strain of 0.6% failure occurs abruptly in the first 5 cycles.

6.3.4 Comparison of the Two Tested Groups

The strain energy damage indicator between the two test groups is compared as shown in Table 6.9. The tested samples are compared at two levels of damage, 25% and 50%. The comparison indicates that the second test group consisting of Mix B specimens has a lower extent of damage propagation due to cycling than the first test group, Mix A specimens. Therefore, it is reasonable to assume that Mix B will perform better in the conditions of thermal fatigue. It should be noted that although each sample is referenced to its response in the first loading cycle, the comparison remains valid since the test specifically targets the relative damage propagation and not the absolute resistance to load or cracking. In designing against thermal fatigue, it is necessary to test the material under repeated loads regardless of the magnitude of stresses that are developed. Comparison across different mix types should be based on damage propagation rates, rather than maximum strength or limiting strains. In addition, a form of strain stiffening is observed on samples tested at 0°C, for example Figure (6.18) and (6.22). The causes for this behaviour require further investigation.

6.4 Practical Application of Damage Indices

6.4.1 Estimating Fatigue Life

Using the results of CSCT, it is possible to estimate the fatigue life under simulated field conditions, using one of several existing methods described by Ponniah and Kennepohl (1994). Fatigue curves can be developed by combining results of high cycle and low cycle

fatigue.

The complex temperature cycling that occur in real life situations can be broken into small interval basic components using probability density functions. The accumulated damage in the pavement structure can be estimated using Miner's hypothesis. Certainly, the selection of loading amplitudes and strain rates that characterize the range of random variations encountered in field conditions is very important. A similar process will be applied in section 7.5.3 to estimate the pavement life using a phenomenological fatigue model.

6.4.2 Obtaining Material Properties

Mechanical properties of pavement materials are easily obtained at, or shortly after, the time of construction. Performance prediction models require information on material properties as a function of pavement life or number of loading cycles. The information is not readily available for a wide spectrum of mixes. In many cases, it is obtained indirectly from accelerated aging tests such as the Pavement Aging Vessel (PAV).

In thermal cracking, the required properties are the tensile stiffness modulus and the tensile strength, for comparing the tensile strength and thermal stresses. Aging models for penetration and softening point are used in Section 7.4.4 to estimate tensile stiffness and tensile strength in the reliability-based model TCRACK . The use of accelerated damage indices can provide direct estimates of stiffness and tensile strength for any number of cycles, and eliminate the need for aging models. Furthermore, this method can be applied to aged specimens to investigate the long-term effects of load cycling and aging.

6.4.3 Testing of Reinforced Pavement

The testing approach proves to be advantageous in testing of reinforced pavement sections for the following reasons:

1. Field extracted reinforced samples of relatively large size can be used. Therefore, the size limitation of the TSRST equipment can be overcome.
2. Reinforcement materials carry loads after considerable deformations when aggregate interlock is fully mobilized.
3. Reinforcement materials often fail in bond with the pavement causing delamination or cracking. Therefore, repeated loading tests such as CSCT are suitable for testing the bond strength.
4. The load redistribution can cause stress concentrations around the highly stressed reinforcement strands. Repeated loading tests can show the damage propagation rates due to stress concentrations.

6.5 Closing Remarks

The results of experimental cyclic tensile loading of uncracked and cracked asphalt slabs are presented in this chapter. The influence of construction cracks on the static tensile strength of the pavement is evaluated at three different temperatures. It is expected that these effects will be magnified if the load cycling and aging are considered. It is shown that the remaining life of the pavement denoted by the number of cycles to failure is sensitive to the crack type and location. Damage evolution based on the stress amplitude and strain

energy criteria is evaluated and the incurred inelastic strains were sufficient to cause failure within the first 50 loading cycles in the presented case.

In comparison with the TSRST method, the proposed CSCT method has the following advantages: (1) CSCT is more fundamental to thermal loading of pavements including the two failure modes low-temperature cracking and thermal fatigue, (2) CSCT is capable of modelling multi-layer pavement and reinforced pavement systems to study delamination and bond between layers, (3) the test outputs can be interpreted to evaluate damage, crack propagation and relevant mechanical properties, (4) the CSCT method is designated to testing actual field extracted and relatively large samples, (5) the CSCT test can provide critical data on the aging of the mix as an input to mechanistic life prediction models, and (6) in retrospect, the TSRST can only be used to obtain the fracture temperature within a single cooling cycle and at a specified thermal gradient, usually 10°C/hour.

In terms of future research, the proposed CSCT approach can be used to construct a larger database and develop prediction models based on thermal and material conditions in addition to crack type, length and location. Further testing is required to ascertain the results and cover the low temperature range and the slow loading rates.

Table 6.1: Laboratory testing program

Direct Tensile Test					
Group	Mix	Rate mm/s	Temperature °C	Number of Samples	Direction of Loading
I	A	0.005	-20, 0, 20	6	longitudinal
II	A	0.010	-20, 0, 20	6	longitudinal
III	B	0.010	-20, 0, 20	6	longitudinal
IV	B	0.010	-20, 0, 20	6	transverse
Tensile Fatigue Test at +25°C and 0.2% strain					
Group	Mix	Rate mm/s	Temperature °C	Number of Samples	Cracking Condition
V	A	1.0	25	2	uncracked
VI	A	1.0	25	2	Type 1
VII	A	1.0	25	2	Type 2
VIII	A	1.0	25	2	Type 3
IX	A	1.0	25	2	Type 4
Constant Speed Cyclic Test					
Group	Mix	Rate mm/s	Temperature °C	Number of Samples	Applied Strain
X	A	0.10	0, -20	6	0.25,0.5,1.0 %
XI	B	0.10	0, -20	6	0.25,0.5,1.0 %
Total number of samples =				46	

Table 6.2: Direct tensile test results of Group I

Test Temperature °C	Specimen		Max.	Failure	Energy	Tangent	Secant
	Width mm	Height mm	Stress MPa	Strain %	Density MPa	Modulus MPa	Modulus MPa
+20	150.00	60.00	0.07	0.50	27.47	54.00	14.29
	142.00	77.00	0.04	1.75	15.67	17.08	2.31
0.0	149.20	67.80	0.63	1.13	147.19	231.58	56.84
	152.70	78.00	0.73	1.75	205.11	147.31	42.12
-20	149.70	75.63	0.80	0.63	54.68	203.86	155.47
	162.00	76.75	0.67	0.38	15.83	185.14	133.82

Table 6.3: Direct tensile test results of Group II

Test Temperature °C	Specimen		Max.	Failure	Energy	Tangent	Secant
	Width mm	Height mm	Stress MPa	Strain %	Density MPa	Modulus MPa	Modulus MPa
+20	152.34	71.10	0.07	0.75	19.27	43.23	10.28
	149.83	70.05	0.05	0.37	16.31	16.22	15.54
0.0	136.30	70.00	0.63	0.75	78.42	196.42	46.32
	154.80	68.00	0.83	1.50	276.42	219.81	74.97
-20	146.40	72.30	1.27	1.50	132.24	162.09	85.07
	146.80	70.50	1.20	1.50	93.18	170.91	80.60

Table 6.4: Direct tensile test results of Group III

Test Temperature °C	Specimen		Max.	Failure	Energy	Tangent	Secant
	Width mm	Height mm	Stress MPa	Strain %	Density MPa	Modulus MPa	Modulus MPa
+20	150.25	70.20	0.09	0.12	26.35	79.80	79.80
	156.00	75.75	0.12	0.87	24.26	76.44	14.25
0.0	165.50	74.90	0.52	0.62	40.90	158.60	84.64
	165.00	77.00	0.44	0.75	46.16	190.60	59.96
-20	161.00	76.83	0.87	0.75	52.56	253.06	143.71
	150.00	73.40	0.81	0.50	50.59	237.59	123.95

Table 6.5: Direct tensile test results of Group IV, loaded perpendicular to the direction of rolling

Test Temperature °C	Specimen		Max.	Failure	Energy	Tangent	Secant
	Width mm	Height mm	Stress MPa	Strain %	Density MPa	Modulus MPa	Modulus MPa
20	154.00	74.80	0.14	1.12	35.12	50.52	12.75
	158.40	78.50	0.10	1.25	31.40	33.86	8.05
0.0	148.00	75.20	0.49	0.50	50.70	169.51	86.69
	156.20	77.30	0.60	0.50	48.38	135.90	79.60
-20	164.20	78.25	1.11	1.00	75.34	215.40	111.45
	158.00	76.65	1.01	0.87	54.34	210.85	114.76

Table 6.6: Summary of direct tensile test results, groups I-IV, mean of two samples

Test Temperature °C	Disp. rate mm/s	Mix type	Max. Stress MPa	Failure Strain %	Strain Energy MPa	Tangent Modulus MPa	Secant Modulus MPa
+20	0.005	A	0.06	1.13	21.57	35.54	8.80
0.0			0.68	1.44	176.15	189.45	49.48
-20			0.74	0.51	35.25	194.50	144.65
+20	0.010	A	0.06	0.66	17.79	29.73	12.91
0.0			0.73	1.13	177.42	190.95	60.65
-20			1.24	1.50	112.71	166.50	82.84
+20	0.010	B	0.10	0.50	25.31	78.12	47.03
0.0			0.48	0.69	43.52	174.60	72.30
-20			0.84	0.63	51.58	245.33	133.83
+20	0.010	B ^a	0.13	1.19	33.25	42.19	10.40
0.0			0.54	0.50	49.54	152.71	83.15
-20			1.06	0.94	64.84	213.13	113.10

^a tested in the transverse to rolling direction

Table 6.7: Effect of direction of rolling: loading rate 0.010 mm/s, groups III and IV, mean of 2 samples

Test Parameter	Temperature °C	Parallel to Rolling	Normal to Rolling	Ratio parallel/ normal
Tensile Stress MPa	+20	0.101	0.128	78.9 %
	0.0	0.486	0.543	89.5 %
	-20	0.849	1.059	80.1 %
Strain Energy MPa	+20	25.314	33.254	76.1 %
	0.0	43.520	49.536	87.9 %
	-20	51.578	64.843	79.5 %

Table 6.8: Number of cycles and remaining life of test samples at 25°C

Specimen	Group	Crack type	No. of cycles to failure	Remaining life (percent)
Uncracked	V	–	34000	100%
Top crack	VI	1	7500	22%
Bottom crack	VII	2	4000	12%
Edge crack	VIII	3	23000	67%
In-lane crack	IX	4	28500	83%

Table 6.9: Comparison of number of cycles required for damage propagation using strain energy data from CSCT test

Test Temperature	Number of loading cycles					
	0.25 % strain		0.50 % strain		1.00 % strain	
	Damage level *		Damage level *		Damage level *	
	25 %	50 %	25 %	50 %	25 %	50 %
<u>Mix A</u>						
0.0 °C	N/A	N/A	120	> 500	40	100
-20°C	220	550	> 200	> 200	< 5	< 5
<u>Mix B</u>						
0.0 °C	N/A	N/A	> 500	> 500	60	110
-20 °C	> 2000	> 2000	> 200	> 200	< 5	< 5

* damage is computed from drop of strain energy as compared to the strain energy measured from the first loading cycle

Figure 6.1: Stress strain relationship of Mix A at 0.005 mm/s. (Group I mean of two replicates)

Figure 6.2: Stress strain relationship of Mix A at 0.010 mm/s. (Group II mean of two replicates)

Figure 6.3: Stress strain relationship of Mix B tested in direct tension at 0.010 mm/s. (Group III mean of two replicates)

Figure 6.4: Comparison of maximum tensile stress in Mix A at two strain rates

Figure 6.5: Stress strain relationship of Mixes A and B at 0.01 mm/s and 20.0°C

Figure 6.6: Stress strain relationship of Mixes A and B at 0.01 mm/s and 0.0°C

Figure 6.7: Stress strain relationship of Mixes A and B at 0.01 mm/s and -20.0°C

Figure 6.8: Effect of the direction of rolling on the stress strain relationship of Mix B

Figure 6.9: Tensile strength of samples tested parallel and normal to the direction of rolling

Figure 6.10: Strain energy of samples tested parallel and normal to the direction of rolling

Figure 6.11: Construction-induced cracks caused by compactor curved geometry and high stiffness

Figure 6.12: Relationship between tensile stress and number of cycles to cause failure for part-through cracks (Types 1 and 2)

Figure 6.13: Relationship between tensile stress and number of cycles to cause failure for full-depth part-width cracks (Types 3 and 4)

Figure 6.14: Strain-controlled hysteretic loop. Temperature = 0°C and displacement amplitude = 2 mm

Figure 6.15: Evolution of damage in low cycle fatigue measured by energy dissipation. Temperature = 0°C and displacement amplitude = 2 mm

Figure 6.16: Envelopes of peak tensile and compressive stresses in a cyclically loaded sample. Temperature = 0°C and displacement amplitude = 2 mm

Figure 6.17: CSCT results on Mix A: at 0.0°C, 0.1 mm/s and 0–0.5% strain

Figure 6.18: CSCT results on Mix A: at 0.0°C, rate of loading: 0.1 mm/s and 0–1.0% strain (test sample no. 1)

Figure 6.19: CSCT results on Mix A: at 0.0°C, rate of loading: 0.1 mm/s and 0–1.0% strain (test sample no. 2)

Figure 6.20: CSCT results on Mix A: at -20.0 °C, 0.1 mm/s and 0–0.25 % strain

Figure 6.21: CSCT results on Mix A: at -20.0 °C, 0.1 mm/s and 0–0.5 % strain

Figure 6.22: CSCT results on Mix B: at 0.0°C, 0.1mm/s and 0–0.5% strain

Figure 6.23: CSCT results on Mix B: at 0.0°C, 0.1 mm/s and 0–1.0% strain (test sample no. 1)

Figure 6.24: CSCT results on Mix B: at 0.0°C, 0.1 mm/s and 0–1.0% strain (test sample no. 2)

Figure 6.25: CSCT results on Mix B: at -20.0 °C, 0.1 mm/s and 0–0.2 % strain

Figure 6.26: CSCT results on Mix B: at -20.0 °C, 0.1 mm/s and 0–0.5 % strain

Chapter 7

Reliability-Based Thermal Cracking Prediction

Model

The development of the thermal cracking prediction model TCRACK is presented in this chapter. The model is verified using Monte Carlo simulation and a sensitivity analysis is carried out. TCRACK is used to predict thermal cracking on the C-SHRP test sections. The probability of failure obtained by the model is converted to a length of cracks and compared to results of distress surveys conducted on the test sites after five years of operation.

7.1 Introduction

The successful management of pavement maintenance and construction expenditures is critical to highway agencies, and is controlled by the present and future performance of pavements. Typically, a performance prediction model consists of several modules which are used to evaluate the pavement response under expected loading and environmental conditions, including temperature. If the basic parameters of a performance function are stochastic variables, a reliability analysis is required to determine the probability of achieving a satisfactory performance. Pavement parameters and environmental conditions are in

essence stochastic variables due to the variations in climate, materials, and workmanship that occur during highway construction and rehabilitation. Some variations can be reduced by ensuring that intensive testing and quality control procedures are followed. A reliability-based model can be used to determine the significance of each variable at the desired level of quality control, and consequently the expected pavement distress conditions at any given time.

Classical probabilistic approaches have been applied to pavement performance prediction, including thermal cracking, during the past three decades. Advances in reliability analysis, however, have proved that classical reliability is inconsistent, and mandated that design procedures should be revised accordingly. The advent of invariant reliability methods eliminated the inconsistencies experienced in the classical approach.

There are two main types of pavement thermal cracking. The first is the low temperature cracking which is manifested by direct progression of existing transverse cracks during severe cooling cycles. The second is the thermal fatigue cracking which is triggered by the degradation of material properties due to aging and accumulated residual stresses caused by cyclic thermal loading. It is believed that the first type will occur during the first few years in the pavement life and propagate at a slower rate afterwards. The second type starts at a later stage and propagates at an increasing rate until failure. The classification of thermal cracks into low-temperature and thermal-fatigue cannot be applied to individual cracks. In other words, it is still not possible to identify if a particular crack is caused solely by one of the two mechanisms. A prediction model can determine the percentage of each type of cracking from material tests and the temperature patterns.

In this chapter, a new reliability-based prediction model TCRACK is developed and used

to analyze the pavement thermal cracking susceptibility as a function of the variation in design and environmental parameters.

7.2 Selection of a Reliability Method

Reliability methods are categorized into four levels according to the required extent of information on the problem variables [Madsen et al., 1986, Augusti et al., 1984]. The four levels are shown in Table 7.1 including inputs, outputs, and examples. Level I methods employ only one value for each uncertain variable. In simple structural applications, the safety index obtained from factorized load and resistance values constitutes a level I method. Level II methods require two values for each uncertain variable and also the correlation between the variables usually in the form of a covariance matrix. Level III methods will further employ the knowledge of the joint distribution of the uncertain variables and Level IV methods compare structural prospects with reference prospects using economic analysis under uncertainty such as the economic consequences of a major structural failure such as an earthquake or an accident in a nuclear power plant. Level IV methods are very complex and are still in the development stages. It should be noted that the classification is not exhaustive and some solution methods may fall in between any two levels.

In the thermal cracking area, the probability of failure can be translated to a predicted length of cracks. Level II methods are best suited for this application since information about the mean and variance of basic variables can be obtained from standard quality control tests. The First Order Reliability Method, FORM, and the Second Order Reliability Method, SORM, are well recognized level II methods that have been used in predicting pavement

performance. In typical structural safety problems, the risk of failure must be extremely low, between 10^{-5} and 10^{-7} , in order to minimize the risks of loss of human lives and the damage to properties. In checking the structural safety of such problems, the reliability index falls in the tail and is very sensitive to the accuracy of the reliability method. In this context, pavement failures are not catastrophic from a structural point of view. The probability of failure may exceed 0.5 which allows for using a lower level reliability method. However, the inconsistency of Level I methods cannot be overlooked. In this chapter, FORM the simplest of level II methods, will be applied to thermal cracking. It is decided not to use SORM because neither further accuracy in the results can be verified nor is required for practical purposes. The reliability-based model is integrated into a computer software, termed TCRACK [Shalaby et al., 1994, Shalaby and Easa, 1995a, Easa et al., 1996].

7.3 Level-II Reliability Formulation

The level-II reliability method involves computing a reliability index and a probability of failure, given a performance function (sometimes called failure or limit state function). The performance function Z is given by

$$Z = f(\mathbf{X}) = f(X_1, X_2, \dots, X_n) \quad (7.1)$$

where $\mathbf{X} = (X_1, X_2, \dots, X_n)$ are the basic random variables such as the tensile strength, pavement temperature, and penetration. Let us first assume that the basic variables are uncorrelated and normally distributed. The n -dimensional basic variable space is divided by the failure surface $f(X) = 0$, into a failure region $f(X) < 0$ and a safe region $f(X) > 0$. The set of basic variables is transformed to give a set of standardized normal

variables $\mathbf{Y} = (Y_1, Y_2, \dots, Y_n)$, where

$$Y_i = \frac{X_i - \mu_{X_i}}{\sigma_{X_i}}, \quad i = 1, 2, \dots, n \quad (7.2)$$

μ_{X_i} and σ_{X_i} are the mean and standard deviation of the random variable X_i , respectively.

The mean of Y_i equals *zero* and the standard deviation equals *one*. The performance function in terms of the transformed variables is then

$$Z = h(\mathbf{Y}) = h(Y_1, Y_2, \dots, Y_n) \quad (7.3)$$

For a two-variable problem, the fail and safe regions and the failure surface are as shown in Figure (7.1). The shortest distance from the origin to the failure surface in the Y -coordinate system is the reliability index, β . The corresponding location on the failure surface is the design point. The important feature of this reliability index is that all equivalent performance functions result in the same failure surface and in turn the same reliability index. For normal random variables, the probability of failure, P_F , is estimated using the following n -fold integral

$$P_F = \Phi(-\beta) = \int_{-\infty}^{-\beta} pdf dZ \quad (7.4)$$

where $\Phi(-\beta)$ = area under the *pdf* of the standard normal variate from $-\infty$ to $-\beta$, and *pdf* is the joint probability density function. Equation (7.4) is exact for linear performance functions, but is found to be quite accurate for various nonlinear functions in practical engineering problems [Ang and Tang, 1984].

7.4 TCRACK Variables

The reliability formulation of low-temperature cracking and thermal-fatigue cracking, presented below, involves several basic pavement properties. These include coefficient of thermal contraction, pavement temperature, penetration and softening point, penetration index, binder stiffness, mixture stiffness, and pavement stresses.

7.4.1 Coefficient of Thermal Contraction

The coefficient of thermal contraction of the asphalt concrete, α , generally depends upon the type and temperature of asphalt cement, the type of aggregate used in the mixture, and the percentage of binder and aggregate in the asphalt concrete. However, this coefficient is considered as a random variable with given mean and variance.

7.4.2 Pavement Temperatures

The temperature at the middle of each pavement layer is assumed to be the dominating layer temperature. Pavement temperatures are assumed to alternate about an annual mean in a sinusoidal manner. The daily temperature cycle consists of two ramp phases, each of a 12-hour duration and a temperature range, T_r . Only the cooling (loading) phase is considered in the analysis, Figure (7.2). In practical design conditions, the pavement temperature profiles can be derived from the weather data.

7.4.3 Penetration Index

The penetration index (PI) of asphalt, as defined by Pfeiffer and Van Doormaal (1936) is a function of the penetration (100 g, 5 sec, 0.1 mm) and the softening point (Ring and Ball temperature, $T_{R\&B}$). The lower the PI, the higher the temperature susceptibility. The penetration index is considered as a random variable; whose variation is determined from the penetration and softening point test results.

The penetration index is determined from equations (7.5) and (7.6) as a function of the slope of the penetration temperature curve, PTS , and the temperature of the penetration test, TPT . If the penetration at the softening point is assumed 800, then

$$PTS = \frac{\log 800 - \log Pen(t)}{T_{R\&B}(t) - TPT} \quad (7.5)$$

$$PI = \frac{20 - 500 PTS}{1 + 50 PTS} \quad (7.6)$$

7.4.4 Pavement Aging

The following aging models for penetration, PEN , and softening point, $T_{R\&B}$, are employed in TCRACK [Shahin and McCullough, 1972]

$$PEN(t) = -48.258 - 2.561 \sqrt{t} + 0.1438 PEN(0) - 8.466 \frac{V_A}{1 + \sqrt{t}} + 1.363 TFOT + 0.9225 \frac{PEN(0)}{1 + \sqrt{t}} \quad (7.7)$$

$$T_{R\&B}(t) = -4.632 + 3.162 \sqrt{t} + 1.585 T_{R\&B}(0) - 0.93 TFOT \quad (7.8)$$

where $PEN(0)$, $T_{R\&B}(0)$ are the initial penetration and softening point, $TFOT$ is the percent penetration after thin film oven test, V_A is the percent of air voids in the mixture, and t is the pavement age in months. For the material properties shown in Table 7.2, the two

aging models are illustrated in Figure (7.3). The aging models indicate increased hardening expressed by a loss of penetration and a rise of the softening point as the binder ages.

The aging models are used in TCRACK to determine the binder stiffness and hence the mixture stiffness and tensile strength. An alternative to the use of aging models is to determine the aged material properties directly from an accelerated laboratory mechanical test such as the CSCT presented in section 6.4.2.

7.4.5 Binder Stiffness

The penetration and softening point determined from aging models are used to calculate the penetration index. The binder stiffness, S , is then determined from regression equations developed by [Shahin and McCullough, 1972], which are based on van der Poel nomograph. The flowchart of determining the binder and mixture stiffnesses is shown in Figure (7.4). The stiffness is a function of the loading time, pavement temperature, and the penetration index.

For binder stiffness ranging from 10^{-8} to 1.0 MPa:

$$\begin{aligned} \log S = & -2.35927 - 0.06743 T - 0.90251 \log t + 3.8 \cdot 10^{-4} T^2 \\ & - 1.38 \cdot 10^{-3} T \log t + 6.61 \cdot 10^{-3} P I T \end{aligned} \quad (7.9)$$

with $R^2 = 0.99$, standard error of the estimated equation is equal to 0.1616, and $n = 126$ data points. The regression model for binder stiffness ranging from 1 to 2000 MPa is;

$$\begin{aligned} \log S = & -2.90072 - 0.11485 T - 0.38423 P I - 0.94259 \log t \\ & - 0.000879 T \log t - 0.05643 P I \log t - 0.02915 \log^2 t \\ & - 0.00051837 T^2 + 0.00113 P I^3 T - 0.01403 \cdot 10^{-5} P I T^3 \end{aligned} \quad (7.10)$$

$R^2 = 0.98$, standard error of the estimated equation is 0.1638, and $n = 79$ data points. In these models,

T = mean pavement temperature of the day measured from the softening point,

t = loading time in seconds, and

PI = penetration index.

The model limitations are;

Parameter	Symbol	Range
Penetration Index	PI	-1.5 to 2.0
Temperature drop	T	+50 to -100°C
Loading time	t	0.01 to 10^5 seconds

7.4.6 Mixture Stiffness

Mixture stiffness is a fundamental property for determining thermal stresses. Mixture stiffness, S_{mix} , can be obtained from the binder stiffness, S , and the aggregate concentration factor, C_v , [Heukelom, 1966]

$$S_{mix} = S \left(1 + \frac{2.5 C_v}{A (1 - C_v)} \right)^A \quad (7.11)$$

$$A = 0.83 \log \frac{4 \cdot 10^5 \text{MPa}}{S} \quad (7.12)$$

The effect of loading time and temperature on the mixture stiffness is shown in Figure (7.5).

The variability in mixture stiffness is determined from that of the binder as,

$$\frac{\partial S_{mix}}{\partial S} = \frac{S_{mix}}{S} \left[1 + \frac{0.83 B}{A + B} - \frac{0.83}{A} \log \frac{S_{mix}}{S} \right] \quad (7.13)$$

where $B = \frac{2.5 C_v}{1 - C_v}$.

7.4.7 Pavement Thermal Stresses

Stresses are calculated using the approximate elastic method of Hills and Brien (1966), which assumes that: (1) the loading is uniaxial and the material response is purely elastic, (2) thermal stresses are uniformly distributed along the depth of the pavement layer, (3) coefficient of thermal contraction is constant with time and temperature, and (4) pavement layers are homogeneous and uncracked. The limiting stresses are found by summing the stresses over the daily temperature drop, T_r , during a loading cycle

$$\text{Accumulated stresses} = \int_{T_r} \alpha S_{mix}(T, t) dT \quad (7.14)$$

7.4.8 Mixture Tensile Strength

The maximum tensile strength of a mixture is considered a basic property with a given mean and variance. The relationship between tensile strength of the mixture and the binder stiffness can be normalized with respect to the maximum tensile strength (Heukelom, 1966). The resulting relationship can be used to find the relative tensile strength, R_H , of a mixture if its binder stiffness is determined. The method is applied to a reported set of data, [Heukelom, 1966], and the relative strength R_H is fitted to a fourth degree logarithmic polynomial function of the binder stiffness as follows

$$R_H = r_0 + r_1 \log S + r_2 (\log S)^2 + r_3 (\log S)^3 + r_4 (\log S)^4 \quad (7.15)$$

where $r_0, \dots, r_4$ are regression coefficients obtained by the method of Least Squares. The test data of [Heukelom, 1966] and the interpolated polynomial are shown in Figure (7.6).

7.5 Component and System Reliability

The reliability indices of individual failure modes, low-temperature cracking and thermal-fatigue cracking, are needed for estimating the system probability of failure and, in turn, the percent of the pavement surface subject to cracking.

7.5.1 FORM Iterative Procedure

The reliability index is obtained using the iterative Rackwitz-Fiessler Procedure, which consists of the following steps [Smith, 1986]:

1. Determine an expression for $Z = f(X)$
2. Find a corresponding expression in standard normal space $Z = h(Y)$
3. Find the first partial derivative with respect to each variable Y_i , $h'_i(Y)$
4. Set both $Y = 0$ and $\beta = 0$
5. Compute $h(Y)$
6. Compute $h'_i(Y)$, for $i = 1, 2, \dots, n$
7. Compute the standard deviation of Z

$$\sigma_Z = \sqrt{\sum_{i=1}^n [h'_i(Y)]^2} \quad (7.16)$$

8. Compute new values of Y_i ,

$$Y_i = -\frac{h'_i(Y)}{\sigma_Z} \left[\beta + \frac{h(Y)}{\sigma_Z} \right] \quad (7.17)$$

9. Compute β

$$\beta = \sqrt{\sum_{i=1}^n Y_i^2} \quad (7.18)$$

10. Repeat steps 5–9 until the values of β converge.

A flowchart describing the general procedure for the two failure modes and their correlation is shown in Figure (7.7), and the application of this procedure to thermal cracking is derived in the following sections.

7.5.2 Failure Mode 1: Low-Temperature Cracking

In order to compute the accumulated stresses during a cooling cycle, the temperature drop, T_r , is divided into n equal intervals each of length ΔT . The mixture stiffness at the midpoints of each interval i , is then computed and the accumulated stresses are found by summing on all intervals. The performance function for low-temperature cracking, Z_L , is given by the difference between the tensile strength of the asphalt concrete and the accumulated stresses during a cooling cycle from equation (7.14) as follows

$$Z_L = R_H H_{max} - \sum_i \alpha S_i(T, t) \Delta T \quad (7.19)$$

where, R_H = percent of tensile strength to the maximum tensile strength, which is a function of the binder stiffness S , and is given by

$$R_H = r_0 + r_1 \log S + r_2 \log^2 S + r_3 \log^3 S + r_4 \log^4 S \quad (7.20)$$

where

H_{max} = maximum tensile strength of the mix,

$S_{mix}(T, t)$ = mixture stiffness at the midpoint temperature T of interval i and loading time t , and

ΔT = temperature interval.

The standard deviation of Z_L , is obtained using a first-order approximation of the Taylor series expansion as

$$\sigma_{Z_L} = \sqrt{\sum_{i=1}^n \left(\frac{\partial Z_L}{\partial x_i} \right)^2 \sigma_{x_i}^2} \quad (7.21)$$

where

$\partial Z_L / \partial x_i$ = partial derivative of Z_L with respect to the random variable x_i , and

σ_{x_i} = standard deviation of x_i .

The variables x_i ($i = 1, 2, \dots$) represent the independent variables involved in (7.19) such as α , T_r , $T_{R\&B}$, and H_{max} .

7.5.3 Failure Mode 2: Thermal-Fatigue Cracking

The accumulated damage due to temperature cycling is the cause of thermal fatigue. It is assumed that only one thermal loading cycle will occur each day. The following fatigue law is used:

$$N_f = K_1 \epsilon_t^{-K_2} \quad (7.22)$$

where N_f is the allowable number of cycles to cause fatigue cracking under tensile strain ϵ_t ($= \alpha \sum_i \Delta T$) and K_1, K_2 are fatigue coefficients which are logarithmic functions of S_{mix} such that

$$K_1 = c_1 \log S_{mix} + c_2 \quad (7.23)$$

$$K_2 = c_3 \log S_{mix} + c_4 \quad (7.24)$$

Fatigue coefficients $c_1, \dots, c_4$ and consequently K_1 and K_2 are determined from fatigue experiments and the mixture stiffness at each temperature step during the cooling cycle

as shown in Figure (7.9). The accumulated damage, D , is determined from Miner's Hypothesis,

$$D = \sum_i \sum_j \frac{n_{ij}}{N_{ij}^*} \quad (7.25)$$

where n_{ij} is the actual number of repetitions at strain level i and stiffness level j , and N_{ij}^* is the expected number of cycles to cause failure at the same conditions. The performance function of thermal-fatigue cracking, Z_F , is defined as

$$Z_F = 1 - \sum_{i=1}^n \frac{30}{N_f^i} \quad (7.26)$$

where n is the number of months, and N_f^i is the number of cycles to cause fatigue cracking for month i . The second term on the right hand side of equation (7.26) is the accumulated fatigue damage according to Miner's hypothesis. Failure occurs when the accumulated damage function $Z_F \leq 0$. The number of cycles N_f is assumed to have a lognormal distribution. The variability in the number of cycles N is determined, similar to equation (7.21), from

$$\sigma_{\log N} = \sqrt{\sum_{i=1}^n \left(\frac{\partial \log N}{\partial x_i} \right)^2 \sigma_{x_i}^2} \quad (7.27)$$

where x_i = original independent random variables, such as the penetration index, PI , α , and so on. A reversed iterative procedure is required for the application of FORM. The targeted number of cycles, N_i , corresponding to a specific failure probability, β , is determined by iteration. The accumulated damage, D , is computed and the pavement age at which the probability of failure will cause cracking is then determined when D reaches *one*. The procedure is repeated for the entire range of probabilities of failure P_{FF} between 0 and 1.0.

7.5.4 System Probability of Failure

The reliability indices β_L and β_F are used to compute the component probability of failure.

The bimodal bounds of the system probability of failure that account for m failure modes are given by [Ditlevesen, 1981]

$$P_F \geq P_{Fi} + \sum_{i=2}^m \max \left(P_{Fi} - \sum_{j=1}^{i-1} P_{ij}, 0 \right) \quad (7.28)$$

$$P_F \leq \sum_{i=1}^m P_{Fi} - \sum_{i=2}^m \max (P_{ij}) \quad (j < i) \quad (7.29)$$

The system failure probability for the two present distress modes is given by,

$$P_F = P_{FL} + P_{FF} - P_{FLF} \quad (7.30)$$

where P_{FL} and P_{FF} = component probabilities of failure for low-temperature and thermal-fatigue cracking, respectively, and P_{FLF} = joint failure probability of the two distress modes, approximated by the following single integration [Madsen et al., 1986]

$$P_{FLF} = \Phi(-\beta_{LF}) \cdot \Phi(-\beta_{FF}) + \int_0^\rho \phi_2(-\beta_{LF}, -\beta_{FF}; z) dz \quad (7.31)$$

where ρ = coefficient of correlation between the two distress modes, and $\phi_2(\cdot, \cdot; \rho)$ = bivariate normal probability density function which has *zero* means, *unit* variances, and a correlation coefficient ρ . The probability density function ϕ_2 is given by

$$\phi_2(x, y, \rho) = \frac{1}{2\pi\sqrt{1-\rho^2}} \exp \left[\frac{-\frac{1}{2}(x^2 + y^2 - 2\rho xy)}{(1-\rho^2)} \right] \quad (7.32)$$

The coefficient of correlation is determined from the area mapped by the two modes, linearized to the dot product of the out-normals to each surface at the design point.

$$\rho_{LF} = \frac{\sum_K h'_{LK} \cdot h'_{FK}}{\sigma_L \cdot \sigma_F} \Big|_{\text{design point}} \quad (7.33)$$

where K denotes those variables used in both performance functions and h' is the surface derivatives for the selected variables.

7.6 Converting Probability of Failure to Length of Cracks

The probability of failure obtained from the model is the percent of pavement area subjected to cracking. The length of transverse cracks is considered an important measure of surface distress, in particular if crack routing and sealing is involved. In practice, the conversion from area to linear cracks ranged from 0.3 m (1 ft) of linear cracking for every 0.46 m² (5 ft²) of area cracking in the Shahin and McCullough model, and up to 0.3 m (1 ft) for every 0.28 m² (3 ft²) in the viscoelastic system computer program VESYS [Federal Highway Administration, 1978]. Furthermore, in the SHRP model, [Hiltunen and Roque, 1994], a cracking frequency of *one* crack per 4.572 m (15 ft) is selected as the maximum amount of thermal cracking and equivalent to a failure probability of 100%. The SHRP model predicts up to 50% probability only, which is *one* crack per 9.144 m (30 ft) of pavement.

7.7 Program Modules

The computer program TCRACK comprises three modules. Module 1 is used to obtain the failure probabilities in low-temperature and thermal-fatigue cracking using FORM and MVFOSM. Module 2 performs Monte Carlo simulation to compute the component and system failure probabilities at specified time intervals, the default is 6 months. Module 3 computes the correlation of the two failure modes and determines the joint failure probability P_{FLF} , the system failure probability P_F , and the coefficient of correlation ρ . Module 3 uses the output of Module 1 to calculate the joint and system reliabilities.

7.8 Application Example

A numerical analysis is performed using TCRACK to illustrate the use of the model in predicting both types of thermal cracking. Table 7.2 illustrates the probabilistic variables used in the analysis. The results of component and system reliabilities of the reference case are shown in Figure (7.10). The correlation between the two failure modes, ρ , is significant and ranged from 0.52 in the first 12 months to 0.62 after 72 months. As shown in Figure (7.11), controlling the variability of the coefficient of thermal contraction, α , in the range of 5% to 30 %, have a considerable effect in delaying the propagation of thermal cracks. The influence of the variability in the expected temperature range, T_r , from 1% to 10% is shown in Figure (7.12).

7.8.1 Monte Carlo Simulation

A simulation program based on Monte Carlo technique was developed to verify the results. A sample size of 2000 with 20 replications is chosen for each failure mode. The number of simulations required to achieve a particular level of reliability is

$$N = \frac{\beta^2(\alpha/2)}{4\alpha^2} \quad (7.34)$$

where β is the normal distribution variate with a confidence level = $1 - \alpha$ [Harr, 1987]. The selected number of sampling points, 40000 points, exceeds the requirement for 99% confidence and provides an acceptable representation of the distribution tails. A random number generator in Module 2 of TCRACK is used to produce the samples according to the specified variability and estimated means of the input data to Module 1. FORM results for both types of cracking are in close agreement with the simulation as shown in

Figures (7.13) and (7.14). In low-temperature cracking, FORM maps the failure surface to minimize the strength-stress equilibrium function resulting in more conservative predictions than those given by MVFOSM, as shown in Figure (7.13). For thermal-fatigue cracking, the performance function determines the required number of cycles to failure. Application of the FORM method as compared to MVFOSM led to a higher and more accurate estimate of the cycles to failure at each step of time, and therefore a lower accumulated damage compared to MVFOSM and lower prediction of the pavement susceptibility to cracking, Figure (7.14). In both cases, FORM results are closer to the Monte Carlo simulation and provides an invariant estimate of the failure probability in comparison to MVFOSM irrespective to problem formulation.

7.8.2 Sensitivity Analysis

The changes in the predicted cracking intensity due to the increase in the variance of individual model parameters are presented in Tables 7.3, and 7.4. For the reference case considered in the example, it is clear that thermal cracking is more sensitive to the variability in the coefficient of thermal contraction, particularly for the case of thermal-fatigue cracking. For example, an increase of 20% in the variability of α , while all other variabilities and all the expected values are maintained constant, led to a significant 10% (2.89/27.26) increase in the failure probability of thermal-fatigue, as shown in Table 7.4. The effect of variability of each basic stochastic variable on the the component reliability indices β_L and β_F can be evaluated from the sensitivity factor, u_i , given by

$$u_i = \frac{\partial \beta(\mathbf{Y})}{\partial Y_i} = \frac{\partial}{\partial Y_i} \left(\sum_{j=1}^n Y_j^2 \right)^{1/2} = \frac{h'_i(\mathbf{Y})}{\sigma_Z} \quad (7.35)$$

and n is the number of independent basic variables. The index β , representing confidence in the results increases by $1/(1 - u_i^2)$ for each perfect information $\sigma_{x_i} = 0$ [Madsen et al., 1986]. Parameters with low sensitivity factors $|u_i| \rightarrow 0$ can be considered deterministic without a significant error in the computed failure probability.

The sensitivity factors of equation (7.35) are plotted in Figure (7.15) for the first six years of pavement life. The performance function in low-temperature describes the equilibrium between tensile strength and thermal stresses, whereas in thermal-fatigue it is the accumulation of damage in the pavement. The sensitivity factors of resistance parameters are positive and those of load and damage parameters are negative. This argument is verified from the results shown in Figure (7.15). The uncertainty in the coefficient of thermal contraction, α , and maximum tensile strength, H_{max} , have the highest impact on the cracking probability. The change in sensitivity factors with time is not significant in the present example.

7.9 TCRACK Advantages and Limitations

Advances in the field of reliability and structural safety during the past two decades have proven the inconsistency of conventional reliability analysis methods that are based on safety indices evaluated at the mean values rather than at the design point on the failure surface. A new performance prediction method TCRACK is utilizing level-II reliability to obtain more accurate and invariant predictions. The new model provides rational predictions to assist in selecting paving materials and scheduling maintenance operations. Also, the model can be incorporated in performance-based specifications to control pavement thermal cracking. The presented model accommodates for the sensitivity of the input material

and environmental parameters and can be extended to reflect the sensitivity of additional parameters whenever such data become available.

The program is divided into three separate modules to allow for easier use and future enhancements. In comparison to existing candidate models, TCRACK is noted for considering two values for each parameter, the estimated mean and variance and in accounting for the correlation between two failure modes.

The developed thermal cracking model is based on a phenomenological fatigue approach. Similar to many pavement cracking models, TCRACK cannot deal with the growth of individual cracks. The model requires information on the pavement conditions, and temperature patterns with a specified variability. It produces the probability of cracking which can be calibrated to reflect crack spacing or the total length of cracks. The program is limited by the precision of the employed aging and material strength models. At its present state, it is recommended not to exceed eight years of predictions since the aging models are not valid beyond this limit. Reliability index methods are regarded as the best measure of structural reliability. However, Augusti et al. (1984) denoted some of their undesirable features which should be considered within each specific application framework.

7.10 Application Example Using Field Data

7.10.1 C-LTPP Database

The Canadian Long Term Pavement Performance project (C-LTPP) is concerned with the design and construction of pavement experiments and monitoring of pavement performance for a projected twenty years of service. Ten provinces participated in the C-LTPP program

with a total of 24 experimental sites. Each experimental site comprises of two to four adjacent sections of 150 metres length.

The purpose of C-LTPP experiments is to compare different rehabilitation strategies [C-SHRP, 1996]. The primary variables on adjacent test sections are:

1. the thickness of the overlay.
2. use of hot mix asphalt concrete (HMAC) versus recycled asphalt pavements (RAP).
3. Effect of surface treatments prior to overlaying the pavement layer.
4. Use of performance enhancing additives such as polymer-modified binders.

The climatic and traffic conditions differ significantly from one experimental site to another. All test sections were constructed during 1989 and 1990. Annual or biannual monitoring measurements are being collected from each section throughout the life cycle. The database containing first five years of service, was first released in February 1996. In 1994, the first comparison between C-SHRP transverse cracking and a prediction model was performed [Hiltunen and Roque, 1994b]. Distress data after three years of service were available at that time. The authors compared predicted cracking according to the SHRP model to field observed cracking from distress surveys. They concluded that SHRP indirect tensile test at low temperatures should be conducted at lower temperatures for the Canadian sites and that the model predictions were acceptable considering the difficulties of predicting the amount of cracking and the time of cracking.

7.10.2 Extent of Transverse Cracking on C-SHRP Sections

After examining of the pavement distress data for all test sections, it was decided to select sections from diverse climatic conditions and at different stages of cracking intensity. The selected sites are shown in Table 7.5, and represent all geographic and climatic zones and are at varying states of observed cracking.

The cracking data is summarized into four types of cracks; transverse, centreline, edge and wheel path. Distress data after five years of service confirmed that transverse cracking is the most common type and the most influential on pavement life. Figure (7.16) illustrates the relationship between the age of the overlay in years and the extent of transverse cracks in metres for the selected C-LTPP sections. All sections were scheduled for rehabilitation in 1989/90. A distress survey was conducted prior to the construction of the overlay, and the amount of cracking on the original layer is shown in the figures. It should also be noted that the C-LTPP database records the severity of each crack on a scale from 1 to 5 according to crack width in millimetres. In this study, the total extent of all cracks is used without any weighing factors for the crack severity.

7.10.3 TCRACK Inputs

Performance prediction models, by nature, require massive information about the pavement materials and loading conditions in order to improve their predictive abilities. The C-LTPP experiment has the largest national pavement database that was ever collected. However, the database does not include laboratory test results on pavement aging, tensile strength or stiffness, which are a primary factor in the study of pavement response to thermal stresses.

It should be noted that some parameters can be indirectly estimated from field tests such as the falling weight deflectometer and the benkelman beam, both of which are in the database. In this study, it was decided as a first approximation to neutralize the effect of the unknown parameters by selecting a reasonable value for all tested sections.

It is noted that the asphalt cement ratio in the selected sections ranged between 4.77% and 6.30%, *C-LTPP Table: AC-TST2*. Since there is no sufficient information on the coefficient of thermal contraction and the variation in asphalt cement ratio is very limited, a reasonable value is selected and applied for all sites.

7.10.4 Climatic Conditions

The climatic conditions data for the C-LTPP database are obtained from weather data collected by Environment Canada at the climatic stations closest to the test sites. The climatic stations produce hourly records of temperature, and wind speed, in addition to daily amounts of precipitation and sunshine. The C-LTPP database retains only monthly means of these climatic variables. The available temperature data contains 3 data elements per month which are the averages of the mean daily temperature, the maximum daily temperature, and the minimum daily temperatures. The inputs to the program rely on these three elements and follow the climatic model presented earlier in section 7.4.2, and include:

1. Daily air temperature range in the coldest month:

The range expresses the difference between the highest and the lowest recorded temperatures in one day. This parameter is directly related to the amount of thermal stresses induced in the pavement and hence the susceptibility to low temperature cracking and

thermal fatigue. The database contains averaged ranges grouped by the month and the year for each site. In this study, the daily temperature range is computed from the difference between the mean daily maximum temperature and the mean daily minimum temperature, where both are taken for the coldest month of the year, which is typically January. The average of this temperature range over the five years of service is computed and used in the temperature model.

2. Annual mean temperature:

This parameter represents the reference temperature for the pavement climatic model. The annual mean air temperature is equal to the average of all daily recorded mean temperatures over the five recorded years. The standard deviation of this variable is computed from the monthly records. The annual mean pavement temperature is estimated at the centre of the surface layer according to the relationship previously discussed in Chapter 3:

$$T_Z = T_S + 0.051Z - 6.3 \times 10^{-5}Z^2 \quad (7.36)$$

where,

T_Z = temperature at depth Z (°C),

T_S = pavement surface temperature (°C) = $0.9T_{air}$ and,

Z = depth measured from pavement surface (mm).

3. Annual temperature range:

It is the average of all monthly temperature ranges at the site over the five year period.

It is used in the climatic model to reflect the seasonal changes in temperature.

7.10.5 Material Parameters

The following C-LTPP material parameters, obtained from tables “*Asphalt Concrete Test Results: AC-TST 1 and AC-TST 2*”, are used in the analysis: penetration at 25°C, softening point and percent of air voids. The database does not include information on the variations of these parameters, typical default values were applied to all sections. The van der Poel nomograph is used to obtain the binder stiffness at an assumed loading time of 3600 seconds. The coefficient of thermal contraction is not specified in the C-LTPP, therefore an average value of $1.4 \times 10^{-5}/^{\circ}\text{C}$ is used for all test sections.

7.10.6 TCRACK Output

The outputs of the reliability-based model are the probability of cracking in two failure modes and the joint and system probabilities of cracking. In the present analysis, performed after five years of service, most of the sections showed very low probability of thermal fatigue at the given temperature ranges. Furthermore, the fatigue coefficients of the overlays are not collected in the database, therefore it is decided to consider the low temperature cracking results only.

To convert the probability of cracking to length of cracks, the method discussed in section 7.6 is employed. Each crack is assumed to have a width, or a zone of influence, of 1.52 m (5 ft). Since the test sections are 150 m long, then 1% probability of cracking is equivalent to *one* metre of cracks.

7.10.7 Comparison of Observed and Predicted Results

Results of the prediction models are grouped by crack intensity as shown in Table 7.6 and Figure (7.17). The model results are in good agreement with the observed total crack extents. In the selected 29 cases, plotted in Figure (7.18) there are:

- Diagonal cells: 18 cases of good predictions (predicted - observed) < 20 m.
- Lower estimates: 4 case of underestimated cracking (predicted - observed) < - 20 m.
- Higher estimates: 7 cases of overestimated cracking (predicted - observed) > 20 m.

Among the eleven off-diagonal data points, only four showed high discrepancies between estimated and observed cracking, which translated into placing the data points two or more cells away from the diagonal in the zone marked as poor predictions, Figure (7.18). The four sections are; 8701021 (ON), 8701022 (ON), and 8705041 (ON) and 8705052 (ON).

It is noted that Ontario sections 8701021 and 8701022 showed the highest amount of cracking among all 65 C-LTPP sections. These two sections are constructed over thick and well bonded layers of old pavement. The two new overlays have a thickness of 46mm and 95mm, respectively, and appear to crack independently from the older layers. On Figure (7.18), most predictions appear to coincide with the diagonal of the matrix, therefore the proposed conversion factor from probability of failure to length of crack (1 ft of crack length $\approx$ 5 ft² of crack area) is considered acceptable in this analysis.

The prediction program is successful in projecting little or no cracks on fourteen sites in all climatic conditions as shown in Figure (7.18). Predicted and observed transverse cracking on four selected sections are also shown in Figures (7.19) to (7.22). It should

be noted that New Brunswick and Ontario sections showed extensive amounts of cracking at very early ages which could not be explained by this predictive model. The predicted cracking in all sections is lower than the observed cracking during the first three years. This finding can be explained by the presence of construction-induced cracks. TCRACK predictions are based on the climatic conditions and do not account for the presence of construction cracks. Prediction models will have to acknowledge the appearance of these cracks at the time of construction in order to improve the predictions for the first few years. Moreover, it is stated earlier that the input climatic data to the program is obtained as averages of monthly extremes and monthly means only. Cases involving severe temperature cycles are only formed by the program according to a statistical distribution of the temperature range. The predictions will be improved if the input data can be modified to show actual worst case scenarios instead of the monthly means. This modification can be made either in the C-LTPP database or by utilizing the climatic data obtained from Environment Canada weather stations.

7.11 Concluding Remarks

The C-LTPP database is an important source of pavement information and response. Records for construction and the first five years of service are used to predict the amount of thermal cracking. The employed data elements were useful in making low temperature cracking predictions with a varying degree of success. The distress summaries and climate tables are very useful in characterizing site conditions, however it is recommended that C-SHRP expands on material testing and study analytical tools for proper estimation of

material related factors such as stiffness, tensile strength and aging.

The results presented in this chapter suggest that TCRACK can be used with more confidence to predict thermal cracking in the field. Site specific temperature data and variability, mix penetration, softening point and air voids provided accurate predictions in 62% of the sections. Laboratory tests of mix properties and improvement of aging, tensile strength and fatigue relationships in TCRACK are essential for higher predictive abilities. The results are also affected by the presence of other distress types such as edge, meander and block cracks.

TCRACK is more suitable for network level thermal cracking predictions, where sufficient information can be obtained. The present state of the model is not very suited to detect minor changes in mix properties, unless if supported with sufficient laboratory test results.

Table 7.1: Levels of reliability methods

Level	Input	Output	Example
I	one value for each parameter (estimated mean)	factor of safety	MVFOSM $F = \frac{R}{L}$ $F = R - L$
II	two values for each parameter (estimated mean, standard deviation) and covariance matrix	reliability index at the design point	FORM and SORM
III	two values and joint distribution	reliability index	Only in special projects
IV	complete information required	planning of a major project	techniques are not available yet

MVFOSM = mean value first order second moment reliability method,

FORM = first order reliability method,

SORM = second order reliability method,

F = factor of safety,

R = resistance, and

L = load.

Table 7.2: Main input parameters to TCRACK application example

Variable	Distribution	Mean	C.O.V. [†]
Loading time	normal	3600 s	25 %
Maximum tensile strength, H_{max}	normal	2.4 MPa	15 %
Coefficient of thermal contraction, α	normal	$1.4 \times 10^{-5}/^{\circ}\text{C}$	5, 10, 20 [‡] , 30 %
Daily temperature range, T_r	normal	24.0 $^{\circ}\text{C}$	1, 5 [‡] , 10 %
Annual temperature range	normal	8 $^{\circ}\text{C}$	0.0 %
Softening point, $T_{R\&B}$	normal	43.0 $^{\circ}\text{C}$	3 %
Aggregate vol. conc.	deterministic	0.871	N/A
Air voids	deterministic	7.5 %	N/A
Retained penetration after TFOT	deterministic	47 %	N/A

[†] Coefficient of variation. In addition, a standard deviation of 0.05 is assumed for the penetration index, PI , and 1 $^{\circ}\text{C}$ in the annual mean temperature

[‡] Reference case

Table 7.3: Effect of the variance of basic parameters on low-temperature cracking

Pavement Age (Years)	Probability of Low-Temperature Cracking P_{FL} (Reference Case) (%)	Increase in P_{FL} due to 20% increase in coefficient of variation of each parameter				
		$T_{R\&B}$	T_r	t	α	H_{max}
(1)	(2)	(3)	(4)	(5)	(6)	(7)
2	14.92	0.94	0.23	0.94	1.56	0.23
3	17.75	1.05	0.26	0.92	1.60	0.26
4	21.77	1.03	0.30	0.89	1.50	0.30
5	27.09	1.00	0.17	0.84	1.34	0.17
6	33.72	0.78	0.19	0.55	0.55	0.19

Table 7.4: Effect of the variance of basic parameters on thermal-fatigue cracking

Pavement Age (Years)	Probability of Thermal-Fatigue P_{FF} (Reference Case) (%)	Increase in P_{FF} due to 20% increase in coefficient of variation of each parameter			
		$T_{R\&B}$	T_r	T_m	α
(1)	(2)	(3)	(4)	(5)	(6)
2	0.63	0.07	0.03	0.07	0.93
3	3.84	0.17	0.17	0.17	2.40
4	9.68	0.35	0.35	0.35	4.22
5	19.49	0.20	0.60	0.20	2.28
6	27.26	0.17	0.84	0.17	2.89

Table 7.5: Selected C-LTPP Sites and Climatic Zones

Province	C-SHRP ID	Number of sections	Location of test site	Climatic zone
Alberta	810404	4	Hwy 16, Wolf Creek	dry-freeze
British Columbia	820205	2	Hwy 19, Port McNeill	wet-no freeze
	820502	2	Hwy 99, White Rock North	
	820605	2	Hwy 99, White Rock South	
New Brunswick	840101	3	Hwy 15, East of Moncton	wet-freeze
	840204	2	Hwy 101, near Oromocto River	
	840604	4	Hwy 2, (TCH) Mactaquac	
Ontario	870102	2	Hwy 80, near Sarnia	wet-freeze
	870504	2	Hwy 11, North of Bracebridge	
	870505	4	Hwy 57, near Newcastle	
	870701	2	Hwy 31, Greely south of Ottawa	
Total	11	29		

Table 7.6: Comparison of the extent of transverse cracks from field surveys and the predicted extent of cracks using TCRACK , five years after construction.

Province	Section ID	Observed Cracks m	Predicted Cracks m
Alberta	8104041	0	26
	8104042	0	17
	8104043	0	16
	8104044	0	21
British Columbia	8202051	3	5
	8202052	0	3
	8205021	0	9
	8205022	0	6
	8206051	0	8
	8206052	0	11
New Brunswick	8401011	0	0
	8401012	0	0
	8401013	2	0
	8402041	8	17
	8402042	30	31
	8406041	62	95
	8406042	65	99
	8406043	70	89
	8406044	78	94
Ontario	8701021	100	0
	8701022	170	0
	8705041	50	99
	8705042	80	99
	8705051	0	0
	8705052	59	15
	8705053	9	0
	8705054	28	2
	8707011	92	85
	8707012	82	85

Figure 7.1: Reliability index β of a two parameter system

Figure 7.2: Pavement daily and annual assumed temperature models

Figure 7.3: Penetration and Ring & Ball temperature aging models

Figure 7.4: Computing binder and mixture stiffnesses based on regression models of the van der Poel Nomograph

Figure 7.5: Effect of temperature and loading time on mixture stiffness according to van der Poel Nomograph

Figure 7.6: Relationship between relative tensile strength and binder stiffness

Figure 7.7: Predicting the intensity of thermal cracks

Figure 7.8: Predicting failure probability of thermal-fatigue cracking

Figure 7.9: Determining fatigue coefficients K_1 and K_2 from interpolation of fatigue test results.

Figure 7.10: Component and system probability of cracking of the numerical example

Figure 7.11: Effect of the variability of the coefficient of thermal contraction on the probability of cracking

Figure 7.12: Effect of the variability of the daily temperature range on the probability of cracking

Figure 7.13: Comparison of FORM, MVFOSM and Monte Carlo Simulation in low-temperature cracking

Figure 7.14: Comparison of FORM, MVFOSM and Monte Carlo Simulation in thermal fatigue cracking

Figure 7.15: Normalized sensitivity factors of the model parameters computed at the design point

Figure 7.16: Extent of transverse cracks in metres from C-LTPP database

Figure 7.16: (Continued) Extent of transverse cracks in metres from C-LTPP database

Figure 7.16: (Continued) Extent of transverse cracks in metres from C-LTPP database

Figure 7.16: (Continued) Extent of transverse cracks in metres from C-LTPP database

Figure 7.17: Observed and predicted extents of transverse cracks on C-LTPP sites five years after construction

Figure 7.18: Comparison of crack extents from C-LTPP condition surveys and TCRACK predictions, five years after construction

Figure 7.19: Predicted and observed cracking on CSHRP section 8402042

Figure 7.20: Predicted and observed cracking on CSHRP section 8406043

Figure 7.21: Predicted and observed cracking on CSHRP section 8705042

Figure 7.22: Predicted and observed cracking on CSHRP section 8707012

Chapter 8

Summary, Conclusions, and Recommendations

8.1 Summary

Highway networks are one of the nation's most valuable resources. The ability to perform timely and efficient maintenance of the pavement surfaces is a very important issue to highway agencies as well as local municipalities. Thermal cracking of asphalt pavement is a severe and costly problem. There are many approaches to analyze pavement structures under the effect of changing temperatures. The main purpose of this thesis is to study the thermal cracking problem of asphalt pavements. Understanding, predicting and analyzing pavements subject to thermal cracking will lead to improving the quality of the roads and reducing the costs of maintaining the highway network.

The thesis starts with an introduction and a literature review to present the state-of-the-art on thermal cracking from a research and practice perspective. The literature review documents research findings, available models and testing methods. The research completed in this thesis is divided into three main phases.

The first is the analytical modeling of a crack in a pavement structure. Four distinctive crack types are proposed and a decoupled thermal and stress analysis is performed. The ANSYS finite element program is used to solve numerous practical examples. Finite

element results are based on the actual transient thermal regimes which are difficult to replicate in laboratory controlled environment. Linear elastic and elastic plastic fracture mechanics methods are applied. The selection between the two methods relies on material response which depends on the temperature range and stress conditions. The results are of great value for the comparison between crack types since all cracks are subject to the same environmental conditions. The analysis is performed at the defect (crack) level, and therefore is useful to study crack sealing techniques, overlay design, and surface treatments prior to overlaying.

The second phase is the laboratory testing program, in which three different tests were performed. The tests were direct tensile test, tensile fatigue test and a constant speed cyclic test. The latter is a form of repeated constant rate of extension test. All tests are conducted on field compacted specimens with a tensile displacement applied in the horizontal direction. The tests are designed to induce failure conditions under low temperature and low cycle thermal fatigue conditions. The tests are conducted under isothermal conditions at three temperatures +20, 0, and -20°C. These test methods are useful for determining necessary input parameters to analytical methods, or for developing aging and fatigue models for life cycle analysis. Using the laboratory tests, better selection of material, reinforcement, or additives can be made.

The third phase is the development of a reliability-based model, TCRACK, for predicting thermal cracking to compliment similar recent models in the areas of traffic fatigue and rutting. TCRACK main features are the ability to predict two independent modes of failure, and the ability to determine the interaction between them and the system probability of failure. The model predictions are affected by the level of quality control and available

information on material properties and environmental conditions at the site. The model utilizes information on the statistical distribution and the variability of each input parameter. TCRACK is evaluated by comparing the predictions of thermal cracking in twenty-nine test sections monitored by the Canadian Highway Research Program, C-SHRP. The model is successful in predicting occurrence of thermal cracking and the approximate extent of cracks.

The research offers three levels of detail into pavement thermal cracking. The finite element formulation suits defect level analysis when detailed information on the materials and boundary conditions is available. The method is particularly useful to study the effects of crack interaction and crack growth rates. The laboratory test methods can be used for a project level decision on comparing overlay alternatives. Finally, a prediction model is used on a network level to assist in projecting maintenance requirements and deterioration rates on different links in the network.

8.2 Conclusions and Findings

The analysis and results presented in this thesis support the following conclusions:

- A simple classification of thermal cracks based on location and geometry is developed and presented. The classification is useful for comparing finite element results and determining the rate of crack growth and can form a basis for detailed cracking surveys.
- The finite element method is used to determine the ground thermal regime for a ramp freezing thermal cycle. The nodal temperatures are applied as a load to find the deformations and stress intensity factors in a cracked pavement structure. Furthermore,

the rate of crack propagation can be determined for different crack types to be used in predicting the remaining life of the overlay.

- The finite element method is used in a systematic fashion to determine the stress intensity factors due to transient temperature regimes. Both 2-D and 3-D problems are investigated and a sensitivity analysis is performed to demonstrate the effects of different variables. This method makes it possible, for the first time, to evaluate the interaction between surface cracks.
- The finite element analysis shows that the effects of base restraint and crack spacing should be considered in the selection of crack sealants, and the timing of the crack sealing operations.
- Determining material properties and expected performance under thermal stresses is examined through a testing program. The direct tensile test on asphalt concrete samples simulated the conditions of low temperature cracking, while the constant speed cyclic test simulated the conditions of thermal fatigue. Tensile loads, applied through mechanical means, replaced temperature loading to simplify the procedures and reduce the test duration. Both tests are conducted at low and constant temperatures to improve the repeatability of the tests.
- The direct tensile test is used to determine stiffness, failure strain and strain energy. The CSCT test is used to calculate a number of damage indices, however, the choice of the most appropriate damage index requires further research. A higher speed fatigue test is also conducted to compare crack propagation rates from the four types of cracks.

- To closely simulate field conditions of thermal loading, the laboratory test methods should utilize field extracted samples loaded in the horizontal direction.
- Asphalt concrete tensile properties are affected by the direction of rolling. Laboratory results showed that the tensile strength of the asphalt concrete is 10 to 20% lower in the longitudinal direction than that in the transverse direction. The reduction of strength is caused by construction-induced cracks.
- The tensile fatigue test indicated that crack propagation is faster in the vertical direction than in the horizontal direction. Top and bottom cracks propagate at a higher rate than edge and in-lane cracks. The main reason for the difference is the geometry of each crack. While top-bottom cracks have a wide crack front (equal to the lane width) and a short crack face, the lateral cracks have a front equal to the thickness of the layer and a longer crack face and hence longer propagation time.
- Thermal cycling of asphalt concrete mixes produces plastic strains that may cause failure in the range of low cycle fatigue. The material response to cyclic plastic strains is a fingerprint of the asphalt concrete mixture. The CSCT is proposed as an accelerated test method designed to capture this response. The propagation of damage due to load cycling is a true indicator of pavement life under repeated thermal strains. Stress drop and dissipation of strain energy are recommended as damage indices.
- A new reliability-based thermal cracking prediction model is formulated. The model, TCRACK is unique in its ability to use information on material, climatic and construction variabilities to predict the extent of cracking. TCRACK also determines the interaction between two failure modes and the sensitivity of each input variable. The program is

verified using Monte Carlo Simulation.

- TCRACK replaces older thermal cracking prediction models and employs a non-biased level-II reliability approach. The mean and statistical distribution of probabilistic input variables are used to improve the predictive ability of the program.
- The transverse cracking conditions of 29 sites are evaluated using TCRACK . The sites are monitored annually through the C-LTPP project and the provincial highway agencies. The C-LTPP database retains detailed information on materials, climate, distress and traffic on the test sites. Some input parameters to TCRACK are not available from the database and are assigned typical values. The model predictions are in agreement with field observations in more than 60 % of the cases.

8.3 Recommendations

The presented research explored many possibilities of advancing our knowledge of asphalt pavement performance in order to control or reduce thermal cracking. While low temperature cracking is a problem in regions of cold climates, thermal fatigue cracking remains a world-wide problem that received very little attention. There is a new potential in studying this problem using simple simulated laboratory tests such as the CSCT.

Crack routing and sealing require detailed analysis when the section appears to have a multitude of cracks. The sealing of only severe (wide) cracks will most probably result in a rapid opening of other cracks. Therefore, the use of a sealant with a high stiffness is not recommended. Determining the influence zone of a crack relies on the bond between layers and will require further investigation.

Isothermal fatigue tests are a viable and simple alternative to the more complex thermal fatigue tests. The relationship between mechanical tests at constant temperature and imposing a temperature change on a restrained specimen requires further study.

TCRACK is a successful tool for predicting thermal cracking. At present, the program relies on classical relationships for determining binder and mix stiffness and aged mix properties. These relationships were developed using unmodified mixes in the 1960s and 1970s. It is necessary to update these relationships in view of polymer modified mixes, geogrid reinforcement and stone mastic asphalts. It is important to note that the results obtained by TCRACK can be improved significantly with the update of these models. A calibration study is also required to determine the conversion factors from probability of cracking to length of cracks. In addition to the comparison to C-LTPP database, TCRACK results can be compared to the three test roads in Almonte, Alberta, Hearst, Ontario and Sherbrooke, Quebec. The three sites were constructed to complement C-SHRP test sites. The sites are monitored annually by the provincial agencies.

The high level of detail in the C-LTPP and LTPP databases presents a unique opportunity for detailed comparison of rehabilitation strategies and examining performance models for asphalt pavements.

References

- Abd El Halim, A.O., "Influence of relative rigidity on the problem of reflection cracking" *Transportation Research Record 1007*, pages 53-58, 1985.
- Abd El Halim, A.O., "Experimental and field investigation of the influence of relative rigidity on the problem of reflection cracking" *Transportation Research Record 1060*, pages 88-98, 1986.
- Abd El Halim, A.O. Aljassar, A., Easa S.M., Hassan, Y., and Haas, R., "Transverse cracking: analytical, experimental, and field investigations." Proceedings, *Canadian Technical Asphalt Association*, Vol. 39, pages 173-192, 1994.
- AbdulShafi, A.A., and Majidzadeh, Karman, "J-integral and cyclic plasticity approach to fatigue and fracture of asphaltic mixtures." *Transportation Research Record 1034*, pages 112-123.
- Ang, A.H., and Tang, W.H. "Reliability concepts in engineering planning and design. Vol. II: decision, risk, and reliability." John Wiley & Sons, New York, N.Y., 1984.
- Augusti, G., Baratta, A., and Casciati, F. "Probabilistic methods in structural engineering." Chapman and Hall, New York, N.Y., 1984.
- Bai, B.Q., and Anderson, K.O. "An improved theoretical method for prediction of cracking temperatures in asphalt pavements." Proceedings, *Canadian Technical Asphalt Association*, vol. 39, pages 124-155.
- Barsoum, R.S. "On the use of isoparametric elements in linear fracture mechanics." *International Journal of Numerical Methods in Engineering*, Vol. 10, pages 25-37, 1976.
- Bolzan, P., and Huber, G. "Direct tension test experiments" Report *SHRP-A-641*, National Research Council, Washington D.C., 1993.
- Canadian Strategic Highway Research Program "Canadian Long Term Pavement Performance (C-LTPP): database user's guide", Transportation Association of Canada, (<http://www.tac-atc.ca/programs/cshrp.html>), Ottawa, 1996.

- Carpenter, S.H., and Lytton, R.L. "Procedure for predicting occurrence and spacing of thermal-susceptibility cracking in flexible pavements.", *Transportation Research Record*, 1978.
- Christensen, R.M., "*Theory of viscoelasticity*." Academic Press, 1982.
- Christison, J.T., Murray, D.W., and Anderson, K.O. "Stress prediction and low temperature fracture susceptibility of asphaltic concrete pavements." *Association of Asphalt Paving Technologists*, Vol. 41, pages 494–523, 1972.
- Chua, K.H., Der Keureghian, A., and Monismith, C.L. "Stochastic model for pavement design." *Journal of Transportation Engineering*, American Society of Civil Engineers, 118 (6), 769-786, 1992.
- Cornell, C.A. "A probabilistic-based structural code." *Journal of the American Concrete Institute*, 66 (12), 974-985, 1969.
- Darter, M.I., and Hudson, W.R. "Probabilistic design concepts applied to flexible pavement system design." Research Report No. CFHR 1-8-69-123-18, *University of Texas at Austin*, Texas, 1973.
- Deme, I.J., and Young, F.D. "Ste. Anne test road revisited twenty years later" Proceedings, *Canadian Technical Asphalt Association*, pages 254–283, 1987.
- Dettman, J.W. "*Mathematical methods in physics and engineering*." McGraw Hill, pages 390–392, 1969.
- Ditlevesen, O. "*Uncertainty modeling with applications to multidimensional civil engineering systems*." McGraw-Hill, New York, N.Y., 1981.
- Easa, S.M., Shalaby, A., and Abd El Halim, A.O. "Reliability-based model for predicting pavement thermal cracking." *Journal of Transportation Engineering*, *American Society of Civil Engineers*, 122 (5), pages 374–380, 1996.
- Ewalds, H.L., and Wanhill, R.J.H. "Fracture Mechanics." Edward Arnold Publishers Ltd, U.K., 1985.
- Fabb, T.R.J., "The influence of mix composition, binder properties, and cooling rates on asphalt cracking at low temperatures." Proceedings, *Association of Asphalt Paving Technologists*, Vol. 43, pages 285–331, 1973.

- Federal Highway Administration, "Predictive design procedures, VESYS user's manual." *Report No. FHWA-RD-77-154*, Washington, D.C., 1978.
- Finn, F., Saraf, C.L., Kulkarni, R., Nair, K. Smith, W. and Abdullah, A. "Development of pavement structural subsystems." *NCHRP Report 291*, Transportation Research Board, National Research Council, Washington, D.C., 1986.
- Francken, L., and Vanelstraete, A. "On the thermological properties of interface systems," in *Reflective Cracking in Pavements*, (ed. J.M. Rigo, R. Degeimbre and L. Francken), E & FN Spon, London, pages 206–219, 1993.
- Francken, L. "Laboratory simulation and modelling of overlay systems," in *Reflective cracking in pavements-Assessment and control*, (ed. J.M. Rigo and R. Degeimbre), E & FN Spon, London, pages 75-99, 1989.
- Fromm, H.J., and Phang, W.A. "A study of transverse cracking of bituminous pavements." *Proceedings, Association of Asphalt Paving Technologists*, Vol. 41, pages 383–423, 1972.
- Fromm, H.J., and Phang, W.A. "Temperature susceptibility control in asphalt cement specifications." *Highway Research Record*, Highway Research Board, No.350, page 30, 1971.
- Gavin, J., Langlois, P., and Kennepohl, G. "Low temperature performance of three Canadian test roads", *Proceedings of Canadian Technical Asphalt Association*, vol. XLI, pages 416–444, 1996.
- Gaw, W.J. "The measurement and prediction of asphalt stiffnesses at low and intermediate pavement service temperatures." *Proceedings, Association of Asphalt Paving Technologists*, Vol. 47, 1978.
- Gdoutos, E.E., "*Fracture mechanics criteria and applications*." Kluwer Academic Publishers, The Netherlands, 1990.
- George, K.P., and Husain, S. "Thickness design for flexible pavement—A probabilistic approach." 65th Annual Meeting of the *Transportation Research Board*, Washington, D.C., 1986.
- Haas, R.C.G., and Anderson, K.O. "A design subsystem for the response of flexible pavements at low temperatures." *Proceedings, Association of Asphalt Paving Technologists*, Vol. 38, pages 179–223, 1969.

- Haas, R.C.G., and Topper, T.H. "Thermal fracture phenomena in bituminous surfaces." *Highway Research Board*, special report 101, pages 136–153, 1969.
- Haas, R.C.G., and Hajek, J.J. "Predicting low-temperature cracking frequency of asphalt concrete pavements." *Highway Research Board*, HR record 407, pages 39–54, 1972.
- Haas, R.C.G. "A method for designing asphalt pavements to minimize low temperature shrinkage cracking." *Asphalt Institute*, Research report 73–1, 1973.
- Haas, R., Meyer, F., Assaf, G., and Lee, R. "A comprehensive study of cold climate airport pavement cracking." Proceedings, *Association of Asphalt Paving Technologists*, Vol. 56, pages 198–245, 1987.
- Haas, R.C.G., and Phang, W.A. "Relationships between mix characteristics and low-temperature pavement cracking." Proceedings, *Association of Asphalt Paving Technologists*, Vol. 57, pages 290–319, 1988.
- Haas, Ralph, Hudson, Ronald, and Zaniewski, John, "*Modern pavement management*." Krieger Publishing Company, Malabar, Florida, 1994.
- Hajek, J.J., and Haas, R.C.G. "Predicting low temperature cracking frequency of asphalt concrete pavements." *Transportation Research Record* 407, 39–54, 1972.
- Halford, G.R. "Low-cycle thermal fatigue" in *Thermal Stresses II*, (ed. Hetnarski, R.B.), Elsevier Science Publishers B.V., North Holland, The Netherlands, 329–428, 1987.
- Harr, M. "*Reliability-based design in civil engineering*", McGraw-Hill, New York, N.Y., 1987.
- Hasofer, A.M., and Lind, N.C. "An exact and invariant second-moment code format." *Journal of the engineering mechanics division*, American Society of Civil Engineers, 100 (1), 111–121, 1974.
- Heukelom, W. "Observations on the rheology and fracture of bitumens and asphalt mixes." *Association of Asphalt Paving Technologists*, Vol. 35, 358–399, 1966.
- Hills, J.F., and Brien, D. "The fracture of bitumens and asphalt mixes by temperature induced stresses." Proceedings, *Association of Asphalt Paving Technologists*, Vol. 35, pages 292–309, 1966.

- Hiltunen, D., and Roque R. "A mechanics-based prediction model for thermal cracking of asphalt concrete pavements." Proceedings, *Association of Asphalt Paving Technologists*, Vol. 53, 1994.
- Hiltunen, D., and Roque R. "Use of Canadian SHRP test sections to evaluate the SHRP indirect tensile creep and failure test for control of thermal cracking", Polyscience Publications Inc., Morin Heights, Quebec, Vol. 39, 441–464, 1994.
- Hsu, Tai-Ran "*The finite element method in thermomechanics*." Allen & Unwin, Boston, 1986.
- Huang, Y.H. "*Pavement analysis and design*." Prentice Hall, Englewood Cliffs, N.J., 1993.
- Hugo, F., McCullough, B.F., and Van der Walt, B. "Development of a strategy for the implementation of full-scale accelerated pavement testing for the Texas SDHPT", Annual meeting of the Transportation Research Board, Washington, D.C., 1991.
- Humphreys, John and Martin, Charles J. "Determination of transient thermal stresses in a slab with temperature-dependent viscoelastic properties." Transactions, Society of Rheology, Vol. VII, 1963.
- Iyengar, K.T., "Thermal stresses in restrained rectangular plates." Proceedings, *American Society of Civil Engineers*, Journal of Engineering Mechanics, EM1, 1968.
- Irick, P., Hudson, W.R., and McCullough, B.F. "Application of reliability concepts to pavement design." 6th International Conference on Structural Design of Asphalt pavements, Ann Arbor, Mich., 1987.
- Janoo, Vincent "Thermal stress measurement in asphalt concrete." Cold regions research and engineering laboratory, U.S. Army Corps of Engineers, report 93-10, 1993.
- Joseph, P., Haas, R., and Phang, W. "Thermally associated fatigue crack growth through asphalt overlays: an experimental investigation" Proceedings, Paving in Cold Areas/Workshop 3, Ottawa, Ontario, 455-491, 1987.
- Kandhal, P.S., "Low temperature shrinkage cracking of pavements in Pennsylvania." Proceedings, *Association of Asphalt Paving Technologists*, Vol. 47, 1978.

- Kanerva, H., Vinson, T., and Zeng, H. "Low-temperature cracking: field validation of the thermal stress restrained specimen test," Report *SHRP-A-401*, National Research Council, Washington, D.C., 1994.
- Kasahara, Atsushi and Haas, Ralph "Low temperature properties of some selected bituminous mixes and application to design," Proceedings, *Paving in Cold Areas Mini-Workshop*, pages 155–181, 1982.
- Kovalenko, A.D., "*Thermoelasticity*." Wolters-Noordhoff, 1969.
- Lamb, W. "Roadway failure study no. 1." Highway engineering research publication H-14, *Natural resources research institute*, University of Wyoming, 1966.
- Lemaitre, J. "*A course on damage mechanics*" Springer-Verlag, Berlin, 1992.
- Little, D.N., and Mahboub, K., "Engineered properties of first generation plasticized sulfur binders and low temperature fracture evaluation of plasticized sulfur paving mixtures" Transportation Research Record 1034, 103–111, 1985.
- Lunardini, V.J. "Analytical methods for ground thermal regime calculations" part of Thermal design considerations in frozen ground engineering, American Society of Civil Engineers state of the practice report, pages 204–257, 1985.
- Lytton, R.L., Shanmugham, U., and Garrett, B.D. "Design of asphalt pavements for thermal-fatigue cracking." *RR 284-4*, Texas Transportation Institute, Texas A & M University, 1983.
- Madsen, H.O., Krenk, S., and Lind, N.C. "*Methods of structural safety*." Prentice Hall, Englewood Cliffs, N.J., 1986.
- Matthews, J., Monismith, C., and Craus, J. "Investigation of laboratory fatigue testing procedures for asphalt aggregate mixtures" *Journal of Transportation Engineering*, Vol. 119, No. 4, 634–654, 1993
- May, R.W., and Witczak, M.W. "An automated asphalt concrete mix analysis system" Proceedings, *Association of Asphalt Paving Technologists*, Vol. 61, pages 154–187, 1992.
- Moavenzadeh, F. "Asphalt fracture." Proceedings, *Association of Asphalt Paving Technologists*, Vol. 36, pages 51–79, 1967.

- Moavenzadeh, F., and Brademeyer, B. "A stochastic model for pavement performance and management." Proc., *4th International Conference on Structural Design of Asphalt Pavements*, Ann Arbor, Mich, 1977.
- Monismith, C.L., Secor, G.A., and Secor, K.R. "Temperature-induced stresses and deformation in asphalt concrete." Proceedings, *Association of Asphalt Paving Technologists*, Vol. 34, pages 248–285, 1965.
- Monismith, C.L., Alexander, R.L., and Secor, K.R. "Rheologic behavior of asphalt concrete." Proceedings, *Association of Asphalt Paving Technologists*, Vol. 35, pages 400–450, 1966.
- Paris, P.C., and Erdogan, F. "A critical analysis of crack propagation laws." *Journal of Basic Engineering, American Society of Mechanical Engineers*, Series D, Vol. 85, pages 528–553, 1963.
- Pfeiffer, V.Ph., and Van Doormaal, P.M. "The rheological properties of asphaltic bitumen." *Proc. Assoc. of Asphalt Paving Technologists*, Vol. 22, 1936.
- Phukan, Arvind "Frozen ground engineering," Prentice-Hall international series in civil engineering and engineering mechanics, Prentice-Hall Inc., Englewood Cliffs, N.J., 1985.
- Ponniah, Joseph, and Kennepohl, Gerhard "The economic benefits of sealing cracks in flexible pavements." Proceedings, *Canadian Technical Asphalt Association*, Vol. 39, pages 402–421, 1994.
- Rackwitz, R., and Fiessler, B. "Structural reliability under combined random load sequences." *Computers and Structures*, 9, 489–494, 1978.
- Ramsamooj, D.V. "Fracture of highway and airport pavements." *Engineering fracture mechanics*, Vol. 44, No. 4, pages 609–626, 1993.
- Readshaw, E.E. "Asphalt specifications in British Colombia for low temperature performance." Proceedings, *Association of Asphalt Paving Technologists*, Vol. 41, pages 502, 1972.
- Riggins, M., Lytton, R., and Garcia-Diaz, A. "Developing stochastic flexible pavement distress and serviceability equations." 64th Annual meeting, *Transportation Research Board*, Washington, D.C., 1985.

- *Roads and Transportation Association of Canada (RTAC) "Pavement surface condition rating system"*, Ottawa, 1987.
- *Roads and Transportation Association of Canada (RTAC) "Canada's roadway infrastructure: selected facts and figures"*, Ottawa, 1990.
- Robertson, W.D. "Selection of paving asphalt cement for low temperature service" *Proceedings, Paving in Cold Areas Mini-Workshop*, vol. 4, pages 43–81, 1987.
- Rooke, D.P., and Cartwright, D.J. "Compendium of stress intensity factors." Her Majesty's Stationery Office, London, 1976.
- Roque, R., and Ruth, B.E. "Mechanisms and modeling of surface cracking in asphalt pavements." *Association of Asphalt Paving Technologists*, Vol. 59, pages 396–421, 1990.
- Ruth, B.E., Bloy, L.A., and Avital, A.A. "Prediction of pavement cracking at low temperatures." *Association of Asphalt Paving Technologists*, Vol. 51, 53–90, 1982.
- Salam, Y.M. and Monismith, C.L. "Fracture characteristics of asphalt concrete." *Proceedings, Association of Asphalt Paving Technologists*, Vol. 41, pages 215–256, 1972.
- Selvadurai, A.P.S., Au, M.C., and Phang, W.A. "Modelling of low-temperature behavior of cracks in asphalt pavement structures." *Canadian Journal of Civil Engineering*, 17, pages 844–858, 1990.
- Shahin, M. Y., and McCullough, B.F. "Prediction of low-temperature and thermal fatigue cracking in flexible pavements." *Texas Transportation Institute*, research report 123–14, 1972.
- Shalaby, A., Abd El Halim, A.O., and Easa, S.M. "Damage propagation in asphalt pavements at low temperatures" *Annual Conference, Canadian Society for Civil Engineering*, Sherbrooke, Quebec, 1997.
- Shalaby, A., Easa, S.M., and Abd El Halim, A.O. "Effect of crack position on transverse cracking of asphalt pavements" *Proceedings, Paving in cold areas (PICA-6) workshop*, Niigata, Japan, 1996d.
- Shalaby, A., Abd El Halim, A.O., and Easa, S.M. "Thermally induced cracks in asphalt pavements: analytical and experimental studies" *1st Transportation Specialty Conference, Canadian Society for Civil Engineering*, Edmonton, Alberta, Vol IIIb, pages 101–110, 1996c.

- Shalaby, A., Abd El Halim, A.O., and Easa, S.M. "Influence of thermal stresses on construction-induced cracks" *Rilem Third Conference on Reflection Cracking*, Maastricht, The Netherlands, (ed. E. Beuving and L. Franchen), E & FN Spon, pages 30–39, 1996b.
- Shalaby, A., Abd El Halim, A.O., and Easa, S.M. "Low temperature stresses and fracture analysis of asphalt overlays" *Transportation Research Record 1539*, pages 131–138, Washington D.C., 1996a.
- Shalaby, A., Easa, S.M., and Abd El Halim, A.O. "Thermal stress analysis of asphalt overlays" *Al-Azhar Engineering Fourth International Conference*, Al-Azhar University, Cairo, Vol. 4, pages 385–394, December, 1995b.
- Shalaby, A. and Easa, S.M., "Modeling pavement thermal cracking: sensitivity analysis" Proceedings, *Canadian Society for Civil Engineering*, Ottawa, Ontario, Vol. IV, pp. 285–294, June, 1995a.
- Shalaby, A., Easa, S.M., and Abd El Halim, A. O. "User's guide to TCRACK: A program for predicting pavement thermal cracking using advanced reliability method." Research Report *LU-TRC-RR-94-2*, Transportation Research Centre, Lakehead University, Thunder Bay, Ontario, 1994.
- Shields, B.P. "Current studies in transverse cracking of asphalt pavements." Proceedings, *Conference on recent developments in the design and construction of asphalt pavements*, University of Alberta, 1964.
- Skabova, I., Svec., O., Shalaby, A., and Abd El Halim, A.O. "Laboratory results of comparison between Marshall and Gyratory compaction methods", Proceedings, *Canadian Technical Asphalt Association*, Edmonton, Alberta, 1996.
- Smith, G.N. "Probability and statistics for civil engineers." Nichols Publishing Company, N.Y., 1986.
- Solaimanian, M., and Kennedy, T. "Predicting maximum pavement temperature using maximum air temperature and hourly solar radiation" presented at 72nd annual meeting of the Transportation Research Board, Washington, D.C., 1993.
- Soussou, J.E., and Moavenzadeh, F. "Statistical characteristics of fatigue damage accumulation in flexible pavements." *Fatigue and dynamic testing of bituminous mixtures, ASTM STP 561*, pages 3–11, 1979.

- *Strategic Highway Research Program*. “Low-temperature cracking: test selection” Report SHRP-A-400, National Research Council, Washington D.C., 1994
- Stuzalec, Andrzej “Introduction to nonlinear thermomechanics.” Springer-Verlag, London, 1992.
- Sugawara, T., Kubo, H., and Moriyoshi, A. “Low temperature cracking of asphalt pavements.” Proceedings, *Paving in Cold Areas Mini-Workshop*, pages 1–42, 1982.
- Swanson Analysis Systems “*ANSYS Reference Manual*” Houston, Pennsylvania, 1993.
- Thoft-Christenson, P., and Baker, M.J. “*Structural reliability theory and its applications.*” Springer-Verlag, Berlin, 1982.
- Ullidtz, Per “*Pavement analysis.*” Developments in civil engineering, Elsevier Science Publishers B.V., 1987.
- Ullidtz, Per “*Modelling of pavement performance.*” Proceedings, *7th International Conference on asphalt pavements*, Vol. 2, pages 338–355, 1992.
- Vinson, T.S., Zomerman, I., Berg, R., and Tomita, H. “ Survey of airport pavement distress in cold regions” Proceedings, *4th International Conference on Cold Regions Engineering*, Anchorage, Alaska, 1986.
- Vinson, T.S., Janoo, V., and Haas, R. “Summary report on low temperature and thermal fatigue cracking” Report SHRP-A-306, National Research Council, Washington D.C., 1989.
- Yoder, E.J.. and Witczak, M.W. “*Principles of pavement design.*” Second edition, John Wiley & Sons, 1975.

Appendix A

ANSYS Input Files

A.1 Transient Two Dimensional Analysis

```
/FILENAME,KTEMP5
/UNIT,SI
/TITLE, TRANSIENT THERMO-ELASTIC ANALYSIS-CRACKED
Z = 1.97          ! ELEVATION OF THE CRACK TIP in Metres
/PREP7           ! ENTER PREPROCESSOR
/NOPR
ET,1,PLANE77
MP,KXX,1,1.5      ! SUBGRADE CONDUCTIVITY
MP,KXX,2,1.2      ! BASE CONDUCTIVITY
MP,KXX,3,1.1      ! SURFACE CONDUCTIVITY
MP,DENS,1,1.5E3    ! SUBGRADE density for thermal analysis only
MP,DENS,2,1.5E3    ! BASE density for thermal analysis only
MP,DENS,3,2.5E3    ! SURFACE density for thermal analysis only
MP,C,1,3.33E3      ! SUBGRADE Specific heat
MP,C,2,2.667E3     ! BASE Specific heat
MP,C,3,.6E3        ! SURFACE Specific heat
K,1,
K,2,.05,
K,3,2,
K,4,2,1.75
K,5,0.05,1.75
K,6,0,1.75
K,7,2,1.9
K,8,0.05,1.9
K,9,0,1.9
K,10,2,2
K,11,0.05,2
K,12,0,2
K,13,0,Z
A,1,2,5,6
A,2,3,4,5
A,6,5,8,9
```

```

A,5,4,7,8
A,8,7,10,11
A,9,8,11,12,13
AGLUE,ALL
ESHAPE,2
DESIZE,,,,,,,,.2
LESIZ,1,,,3
LESIZE,5,,,10,5
LESIZE,2,,,12,1/5
LESIZE,6,,,12,1/5
LESIZE,7,,,10,1/5
LESIZE,3,,,3,
LESIZE,8,,,3,
LESIZE,4,,,12,5
LESIZE,9,,,3,
LESIZE,10,,,3,
LESIZE,11,,,3,
LESIZE,12,,,10,1/5
LESIZE,13,,,4
LESIZE,15,,,4
LESIZE,16,,,3
LESIZE,17,,,2
LESIZE,18,,,2
LESIZE,14,,,10,1/5
KSCON,13,.012,1,6
APLO
MAT,1
AMESH,1,2
MAT,2
AMESH,4
AMESH,3,
MAT,3
AMESH,5
EPLO
ESHAPE,1
AMESH,6,
EPLO
FINI
!  SOLUTION MODULE
/SOLU
ANTYPE,TRAN,NEW
OUTRES,NSOL,2
OUTPR,BASIC,NONE
BFUNIF,TEMP,0
NSEL,S,LOC,Y,1.995,2.005
CM, TOP, NODE
D, ALL, TEMP, -20
ALLSEL

```

```

TIME, 1
NSUBST, 2
SOLVE
CMSEL, S, TOP
D, ALL, TEMP, -20
ALLSEL
!AUTOTS, ON
NSUBST, 10
TIME, 60*60*24*5
SOLVE
CMSEL, S, TOP
D, ALL, TEMP, -20
ALLSEL
!AUTOTS, ON
NSUBST, 14
TIME, 60*60*24*12
SOLVE
SAVE
FINI
/GOPR
/PREP7
ETCHG                ! CHANGE FROM THERMAL TO STRESS ANALYSIS
KEYOPT, 1, 3, 2       ! PLANE STRAIN OPTION
KEYOPT, 1, 5, 1       ! INTEGRATION POINT RESULTS
MP, EX, 3, 2.25E7*2.6 ! Stiffness
MP, EX, 2, 1.12E7*2.6
MP, EX, 1, .45E7*2.6
MP, ALPX, 1, 1.4E-5    ! thermal contraction coefficient
MP, ALPX, 2, 1.4E-5
MP, ALPX, 3, 1.4E-5
MP, NUXY, 1, .3        ! Poisson's ratio
MP, NUXY, 2, .3
MP, NUXY, 3, .3
FINISH
/SOLU
ANTYPE, TRANS
OUTRES, NSOL, 1
NSEL, S, LOC, Y, -0.001, .001
D, ALL, UY, 0
ALLSEL
NSEL, S, LOC, X, -0.001, .000001
NSEL, R, LOC, Y, -.1, Z+.000001
D, ALL, UX, 0
ALLSEL
NSEL, S, LOC, X, 1.999, 2.0001
D, ALL, UX, 0
ALLSEL
TIME, 0

```

```
LDREAD,TEMP,1,2,,,,RTH
NSUBST,2
SOLVE
TIME, 1
LDREAD,TEMP,2,2,,,,RTH
NSUBST,2
SOLVE
TIME, 2
LDREAD,TEMP,2,4,,,,RTH
SOLVE
TIME, 3
LDREAD,TEMP,2,6,,,,RTH
SOLVE
TIME, 4
LDREAD,TEMP,2,8,,,,RTH
SOLVE
TIME, 5
LDREAD,TEMP,2,10,,,,RTH
SOLVE
TIME, 6
LDREAD,TEMP,3,2,,,,RTH
SOLVE
TIME, 7
LDREAD,TEMP,3,4,,,,RTH
SOLVE
TIME, 8
LDREAD,TEMP,3,6,,,,RTH
SOLVE
TIME, 9
LDREAD,TEMP,3,8,,,,RTH
SOLVE
TIME, 10
LDREAD,TEMP,3,10,,,,RTH
SOLVE
TIME, 11
LDREAD,TEMP,3,12,,,,RTH
SOLVE
TIME, 12
LDREAD,TEMP,3,14,,,,RTH
SOLVE
SAVE
FINISH
/EOF
/POST1
LPATH,464,460
SET,3
PDEF,3_DAYS ,S,1
SET,5
```

```
PDEF,5_DAYS,S,1  
SET,,,,,7*86400  
PDEF,7_DAYS,S,1  
SET,,,,,9*86400  
PDEF,9_DAYS,S,1  
SET,,,,,10*86400  
PDEF,10_DAYS,S,1  
PLPATH,
```

A.2 Three Dimensional Fracture Analysis

```

C**** ANSYS INPUT FILE FOR PARAMETRIC 3-D FRACTURE ANALYSIS
C**** TEMPERATURE PROFILE IS COMPUTED
C**** FROM LAGRANGIAN INTERPOLATION
C**** COMPUTES STRESS INTENSITY FACTOR FROM
C****      1. TWO POINT FORMULA AND CRACK TIP ELEMENT
C****      2. ENERGY BALANCE AND J INTEGRAL METHOD
C****
FINI
/CLE
/FILNAME,K9
/TITLE, THERMO-ELASTIC THREE DIMENSIONAL ANALYSIS
!      CRACKED UNDERLAYER, CRACK TYPE 4
C***** INPUT PARAMETERS
XT = 6.0      ! LANE WIDTH
YT = 3.0      ! HALF LENGTH
ZT = 0.1      ! THICKNESS OF ASPHALT LAYER
ZTT = -0.25   ! THICKNESS OF SUBLAYER
X = 5.5       ! UNCRACKED WIDTH OF LANE
A = .25       ! AREA MESHED IN HIGHER ORDER ELEMENTS
T1 = -15      ! INPUT TEMP1
T2 = -5       ! INPUT TEMP2
T3 = 0        ! INPUT TEMP3
Z1 = ZT       ! INPUT Z LOC OF T1
Z2 = 0        ! INPUT Z LOC OF T2
Z3 = ZTT      ! INPUT Z LOC OF T3
/PREP7
/NOPR
ET,1,PLANE82      ! PLANE77
ET,3,SOLID95      ! SOLID90
ET,4,SOLID45      ! SOLID70
ET,2,PLANE42      ! PLANE55
K,1
K,2,A
K,3,A,A
K,4,-A,A
K,5,-A
K,6,X
K,7,X,A
K,8,X,YT
K,9,A,YT
K,10,-A,YT
K,11,X-XT,YT
K,12,X-XT,A
K,13,X-XT,
KSCON,1,.05,1,8
A,1,2,3,4,5

```

```

A,2,6,7,3
A,3,7,8,9
A,3,9,10,4
A,4,10,11,12
A,5,4,12,13
AGLUE,ALL
LESIZE,2,,,4
LESIZE,3,,,5
LESIZE,4,,,4
LESIZE,9,,,5,6      ! 5 - 10 SEGMENTS
LESIZE,11,,,5,1/6
LESIZE,13,,,5,1/6
LESIZE,15,,,5,1/6
MAT,1
DESIZE,,4
ESHAPE,2
TYPE,2
AMESH,2,6
ESHAPE,0
TYPE,1
AMESH,1
/VIEW,,7,4,-4
KPLO
K,20,,,ZT
L,1,20
*GET,LMAXI,LINE,,NUM,MAX      ! GET LAST ELEMENT NUMBER
LESIZE,LMAXI,,,2             ! TWO IN DEPTH ELEMENT
TYPE,3
VDRAG,1,,,,,LMAXI
TYPE,4
VDRAG,2,3,4,5,6,,LMAXI
K,60,,,ZTT
L,1,60
*GET,LMAXII,LINE,,NUM,MAX
LESIZE,LMAXII,,,1
MAT,2
TYPE,4
VDRAG,1,2,3,4,5,6,LMAXII
ACLEAR,1,6
NUMMRG,NODE
WSORT,
EPLO
MP,EX,1,2.25E7*2.6
MP,ALPX,1,1.4E-5
MP,NUXY,1,.3
MP,EX,2,2.25E7*2.6
MP,ALPX,2,1.4E-5
MP,NUXY,2,.3

```

```

FINI
/SOLU
ANTYPE,STAT,NEW
AUTOTS,ON
NSEL,S,LOC,Y,YT
D,ALL,UY
D,ALL,UX
NSEL,ALL
NSEL,S,LOC,Z,ZTT
D,ALL,UZ
NSEL,S,LOC,Y,0
NSEL,R,LOC,X,0,X
NSEL,R,LOC,Z,0,ZT
D,ALL,UY
NSEL,ALL
TIME,1
OUTPR,ALL,NONE
C*****  MACRO FOR TEMPERATURE LAGRANGE INTERPOLATION
                                         SURFACE
C*****      Z1      | _____ / T1
C*****      Z2      | _____ / T2  INTERFACE (ASPHALT/ASPHALT)
C*****      | _____ /
C*****      Z3      | _/ T3  INTERFACE (ASPHALT/BASE)

      NDEV = 20          ! INPUT NO OF DEVISIONS
      DD = (ZT -ZTT)/NDEV ! TEMPERATURE LAGRANGE INTERPOLATION
      NSEL,ALL
      *DO, IZ, 1, NDEV+1
      DDD = (IZ - 0.5)*DD
      DZ = ZT - DDD
      TT = (DZ - Z2)*(DZ-Z3)/(Z1-Z2)/(Z1-Z3)*T1
      TT= TT + (DZ - Z1)*(DZ - Z3)/(Z2-Z1)/(Z2-Z3)*T2
      TT = TT + (DZ - Z1)*(DZ - Z2)/(Z3-Z1)/(Z3-Z2)*T3
      NSEL,S,LOC,Z,ZT-(IZ-1)*DD,ZT-IZ*DD
      BF,ALL,TEMP,TT
      NSEL,ALL
      *ENDDO

NSEL,ALL
SOLVE
SAVE
FINI
/POST1          ! POSTPROCESSOR
/PBC,PATH,1
C*****        COMPUTE J INTEGRAL AT MIDDLE OF LAYER
ETABL,SENE,SENE
ETABL,VOLU,VOLU
PDEF,CLEAR
A = .25

```



```

SEXP,W,SENE,VOLU,1,-1 ! CALCULATE STRAIN ENERGY DENSITY
                        ! DEFINE THE PATH FROM FOUR NODAL ARGUMENTS
LPATH,NODE(A,0,ZT/2),NODE(A,A,ZT/2),NODE(-A,A,ZT/2),NODE(-A,0,ZT/2)
PDEF,W,ETAB,W          ! PUT STRAIN ENERGY DENSITY ON THE PATH
PCALC,INTG,J,W,YG      ! INTEGRATE ENERGY W.R.T. GLOBAL Y
*GET,JA,PATH,,LAST,J   ! FINAL VALUE OF INTEGRAL JA
PDEF,SAVE              ! CLEAR OLD PATH VARIABLES
PVECT,NORM,NX,NY,NZ    ! DEFINE THE PATH UNIT NORMAL VECTOR
PDEF,INTR,SX,SX        ! PUT STRESS SX ON THE PATH
PDEF,INTR,SY,SY        ! PUT STRESS SY ON THE PATH
PDEF,INTR,SKY,SKY      ! PUT STRESS SKY ON THE PATH
PCALC,MULT,TX,SX,NX     ! CALCULATE TRACTION TX
PCALC,MULT,C1,SKY,NY    ! TX = SX*NX + SKY*NY
PCALC,ADD,TX,TX,C1
PCALC,MULT,TY,SKY,NX    ! CALCULATE TRACTION TY
PCALC,MULT,C1,SY,NY     ! TY = SKY*NX + SY*NY
PCALC,ADD,TY,TY,C1
*GET,DX,PATH,,LAST,S   ! DEFINE PATH SHIFT AS 1% OF PATH LENGTH
DX=DX/100
PCALC,ADD,XG,XG,,,,-DX/2 ! SHIFT PATH FROM X TO X-DX/2 (GLOBAL X DIR)
PDEF,INTR,UX1,UX        ! DEFINE UX AT X-DX
PDEF,INTR,UY1,UY        ! DEFINE UY AT X-DX
PCALC,ADD,XG,XG,,,,DX   ! SHIFT PATH FROM X-DX/2 TO X+DX/2
PDEF,INTR,UX2,UX        ! DEFINE UX AT X+DX
PDEF,INTR,UY2,UY        ! DEFINE UY AT X+DX
PCALC,ADD,XG,XG,,,,-DX/2 ! SHIFT PATH BACK TO ORIGINAL POSITION
C=(1/DX)
PCALC,ADD,C1,UX2,UX1,C,-C ! CALCULATE DERIVATIVE DUX/DX
PCALC,ADD,C2,UY2,UY1,C,-C ! CALCULATE DERIVATIVE DUY/DX
PCALC,MULT,C1,TX,C1      ! DEFINE INTEGRAND
PCALC,MULT,C2,TY,C2      ! = TX*DUX/DX + TY*DUY/DX
PCALC,ADD,C1,C1,C2
PCALC,INTG,J2,C1,S       ! FORM SECOND INTEGRAL(W.R.T. PATH LENGTH S)
*GET,JB,PATH,,LAST,J2   ! FINAL VALUE OF INTEGRAL FOR 2ND TERM OF J
JINT=(2*(JA-JB))        ! ADD BOTH TERMS AND DOUBLE FOR HALF MODELS
KCALCO = ((JINT*(2.25E7)*2.6/.91)**.5)
KCALCO2= ((JINT*(2.25E7)*2.6)**.5)
C*****                COMPUTE KI FROM TWO POINT FORMULA
LPATH,NODE(0,0,ZT/2),NODE(-.05/4,0,ZT/2),NODE(-.05,0,ZT/2)
/OUTPUT,SUN99,,,APPEND
*STAT,X
*STAT,KCALCO
*STAT,KCALCO2
KCALC,0,1,0,0
KCALC,1,1,0,0
/OUTP
/PBC,PATH,1
C*****                REPEAT POSTPROCESSOR ANALYSIS AT THE SURFACE

```

```

PDEF,CLEAR
SEXP,W,SENE,VOLU,1,-1      ! CALCULATE STRAIN ENERGY DENSITY
! DEFINE THE PATH FROM FOUR NODAL ARGUMENTS
LPATH,NODE(A,0,ZT), NODE(A,A,ZT), NODE(-A,A,ZT), NODE(-A,0,ZT)
PDEF,W,ETAB,W              ! PUT STRAIN ENERGY DENSITY ON THE PATH
PCALC,INTG,J,W,YG          ! INTEGRATE ENERGY W.R.T. GLOBAL Y
*GET,JA,PATH,,LAST,J       ! GET FINAL VALUE OF INTEGRAL JA
PDEF,SAVE                  ! CLEAR OLD PATH VARIABLES
PVECT,NORM,NX,NY,NZ        ! DEFINE THE PATH UNIT NORMAL VECTOR
PDEF,INTR,SX,SX            ! PUT STRESS SX ON THE PATH
PDEF,INTR,SY,SY            ! PUT STRESS SY ON THE PATH
PDEF,INTR,SKY,SKY          ! PUT STRESS SKY ON THE PATH
PCALC,MULT,TX,SX,NX        ! CALCULATE TRACTION TX
PCALC,MULT,C1,SKY,NY        ! TX = SX*NX + SKY*NY
PCALC,ADD,TX,TX,C1
PCALC,MULT,TY,SKY,NX        ! CALCULATE TRACTION TY
PCALC,MULT,C1,SY,NY        ! TY = SKY*NX + SY*NY
PCALC,ADD,TY,TY,C1
*GET,DX,PATH,,LAST,S      ! DEFINE PATH SHIFT AS 1% OF PATH LENGTH
DX=DX/100
PCALC,ADD,XG,XG,,,,-DX/2 ! SHIFT PATH FROM X TO X-DX/2 (GLOBAL X DIR)
PDEF,INTR,UX1,UX           ! DEFINE UX AT X-DX
PDEF,INTR,UY1,UY           ! DEFINE UY AT X-DX
PCALC,ADD,XG,XG,,,,DX     ! SHIFT PATH FROM X-DX/2 TO X+DX/2
PDEF,INTR,UX2,UX           ! DEFINE UX AT X+DX
PDEF,INTR,UY2,UY           ! DEFINE UY AT X+DX
PCALC,ADD,XG,XG,,,,-DX/2  ! SHIFT PATH BACK TO ORIGINAL POSITION
C=(1/DX)
PCALC,ADD,C1,UX2,UX1,C,-C  ! CALCULATE DERIVATIVE DUX/DX
PCALC,ADD,C2,UY2,UY1,C,-C  ! CALCULATE DERIVATIVE DUY/DX
PCALC,MULT,C1,TX,C1        ! DEFINE INTEGRAND
PCALC,MULT,C2,TY,C2        ! = TX*DUX/DX + TY*DUY/DX
PCALC,ADD,C1,C1,C2
PCALC,INTG,J2,C1,S         ! FORM SECOND INTEGRAL(W.R.T. PATH LENGTH S)
*GET,JB,PATH,,LAST,J2     ! FINAL VALUE OF INTEGRAL FOR 2ND TERM OF J
JINT=(2*(JA-JB))          ! ADD BOTH TERMS AND DOUBLE FOR HALF MODELS
KCALCO = ((JINT*(2.25E7)*2.6/.91)**.5)
KCALCO2= ((JINT*(2.25E7)*2.6)**.5)
!PDEF,CLEAR                ! CLEAR PATH VARIABLES
LPATH,NODE(0,0,ZT),NODE(-.05/4,0,ZT),NODE(-.05,0,ZT)
/OUTPUT,SUN99,,,APPEND
*STAT,KCALCO
*STAT,KCALCO2
KCALC,0,1,0,0
KCALC,1,1,0,0
/OUTP
FINI

```

Appendix B

Mix Design Reports

The experimental work reported in the thesis was performed on two asphalt mixes denoted as Mix A and Mix B.

The following mix design reports are prepared by contractor for each job. Samples from Mix A were extracted after construction of a parking lot in December 1995. The ambient temperature at the time of construction ranged between -10 to -20 °C. Mix B samples were constructed at the contractor batch plant in May 1996. The ambient temperature at the time of construction was +20 °C. In both cases, an asphalt concrete layer was constructed in wooden boxes of 1200 × 2400 × 75 mm and compacted with 8 passes of a half-ton steel compactor. After one week, the slabs were saw cut into smaller sizes and shipped to the laboratory for further tests.

B.1 Mix A report

Aggregate Designation	Proportion By Weight (%)	Description	Date Received
C-1	61.0	HL 3 Stone	May 5, 1994
F-1	9.6	Stone Dust (Screenings)	May 5, 1994
F-2	23.4	Asphalt Sand	May 5, 1994
F-3	1.0	Saghouse Dust	May 5, 1994

Mix properties at the design asphalt cement content of 5.0 % by weight of the total mix

			OPSS 1149 Specification
Marshall Stability	(N @ 60°C)	11,300	2900 (min.)
Flow Value	(@ 3.5% air voids)	9.0	9 (min.)
SSD Air Voids in Mixture	(%)	3.9	4.0 ± 0.5
Void in Mineral Aggregate	(%)	15.0	14.6 (min.)
Density	(Mg/m ³)	2.435	

Aggregate Properties

Aggregate Designation	C-1	F-1	F-2	F-3
Specific Gravity	2.731	2.742	2.645	2.750
Absorption (%)	0.72	0.85	0.81	-

Grading Analysis

% Passing				Job Mix Formula	OPSS 1149 Specification
Sieve Size					
26.5 mm					
19.0 mm					
16.0 mm	100			100	100
13.2 mm	99.7			99.3	93 - 100
9.5 mm	64.9	100	100	82.1	75 - 90
4.75 mm	4.8	93.6	93.8	50.9	50 - 60
2.36 mm	1.7	61.3	96.4	44.8	36 - 60
1.18 mm	1.5	39.0	90.3	40.4	25 - 53
600 µm	1.3	27.3	79.7	34.9	16 - 45
300 µm	1.3	19.5	32.4	16.0	7 - 26
150 µm	1.2	14.6	6.9	5.7	3 - 10
75 µm	1.1	10.6	2.4	3.5	0 - 5

Date Issued : May 26, 1994

Issued by :

Alexander Brown
A.W. Brown, P.Eng.

Marshall Properties at an asphalt cement content of 5.0 %

One Point Marshal Mix Design OPSS 1150 HL 3 Project No. 20903	Stability (N)	13100	OPSS 1150 2900 (Min)
	Flow @ 3.5 % Air Voids	8.9	8 (Min)
	Air Void Content	4.3	3 TO 5%
	Voids in Mineral Aggregate	16.1	
	Bulk Relative Density	2.436	

Aggregate Properties and Sources

Aggregate Designation	C1	F1	F2	Baghouse Fines
Description	HL 3 Stone			
Source	Beaver	Beaver	Lafarge	Beaver
Source ID				
Specific Gravity	2.776	2.778	2.642	2.75
Absorption (%)	0.84	0.77	0.79	—

sieve Opening (mm)	Gradation Percent Passing			
16	100	100	100	100
13.2	98.33	100	100	100
9.5	65.95	100	100	100
4.75	4.85	92.7	99.2	100
2.36	1.4	62.9	96.1	100
1.18	1.1	42.8	92.1	100
0.6	1	30.2	71.4	100
0.3	0.9	21.5	30.9	100
0.15	0.8	16.7	7.9	100
0.075	0.8	11.9	3.8	100

Proportion by weight	51.00%	9.60%	38.40%	1.00%
----------------------	--------	-------	--------	-------

OPSS 1150 Gradation Bounds

sieve Opening	Job Mix Formula	HL3		Swingback	Remarks
		Lower	Upper		
16	100.0	100	100	0	ok
13.2	99.1	98	100	0	ok
9.5	82.6	75	90	0	ok
4.75	50.5	45	65	0	ok
2.36	45.4	36	64	0	ok
1.18	41.0	25	58	0	ok
0.6	31.8	16	45	0	ok
0.3	15.4	7	26	0	ok
0.15	6.0	3	10	0	ok
0.075	4.0	0	5	0	ok

Issue Date
Certified

Kevin A. MacDonald, M.A.Sc., P.Eng.

B.2 Mix B report

FAVEMIX - MARSHALL MIX DESIGN REPORT
Prepared for: MOODIE (PLANT 14)

JOB NO: 96-F

CONTRACT NO: _____ HOT MIX TYPE: HL 3 PG 58-34 ITEM NO: _____
HWY: _____ LOCATION: _____
DATE SAMPLES RECEIVED: Mar 7, 1996 DATE COMPLETED: April 1, 1996
TEST RESULTS SUPPLIED BY: *Steph D. Smith, P.Eng* WIMPEY MINERALS

JOB MIX FORMULA - GRADATION PERCENT PASSING *

% AC	26.5	19.0	16.0	13.2	9.5	4.75	2.36	1.18	600	300	150	75
5.0			100.0	98.9	82.0	55.0	46.7	39.8	31.4	17.7	6.7	4.2

MARSHALL PROPERT.	REQUIRED	SELECTED	% CA#1	43.8	% RAP	—
% VOIDS, SSD (min)	3.0		% FA#1	33.7	% AC RAP	—
(max)	5.0	4.1	% FA#2	22.5	RAP FEN	
FLOW (min)	8.0	9.2			ERIQ. BRD	2.393
STABILITY, N(min)	8900	12262			MRD	2.495
% VMA (min)	15.0	15.2	Gb	2.682	MRD (SSD)	**

ASPHALT CEMENT		ADDITIVE		
SUPPLIER	PENETRATION	SUPPLIER	TYPE	% of AC
SHELL	PG 58-34	-	-	-

AGG. TYPE	DESCRIPTION - SOURCE	INVENTORY NO	BRD	ABS %
CA#1	HL-3 STONE- MOODIE QUARRY	005-108-0	2.697	0.707
FA#1	BLEND SAND- BURNSIDE	K03-076-0	2.674	0.710
FA#2	SCREENINGS- MOODIE QUARRY	005-108-0	2.665	1.072

AGGREGATE GRADATION - PERCENT PASSING												
AGG. TYPE	26.5	19.0	16.0	13.2	9.5	4.75	2.36	1.18	600	300	150	75
CA#1			100.0	97.6	58.8	3.5	1.4	1.3	1.3	1.2	1.1	1.0
FA#1						100.0	97.1	90.3	73.2	37.2	7.5	1.6
FA#2					100.0	88.0	58.5	37.7	25.2	17.8	12.8	9.7

* Fines Returned to the Mix (1.0%). Dry Collection System.
Minus 4.75 Aggregate Component of JMF Adjusted.
** No Air Voids 'Correction' Necessary, No Visible Aggregate Absorption for
MRD Mix at 5.0 - 5.5% AC.
REMARKS: Gradations from Process Control
Briquettes Compacted at 138°C, 75 Blows Manual Hammer.
AI MS-2 Procedures Followed. This Mix Design is Subject to Marshall
Compliance Checks that May Require JMF Adjustment.

HL 3 Marshall Parameters

COMMENTS: See Marshall Mix Design Worksheets. Designed at 5.0% Asphalt Cement Content and 4.1% Air Voids to Meet Requirements of Special Provisions. AI MS-2 Procedure Followed.

CONTRACT NO: PG 58-34

Sieve Designation - Aggregates

Sieve Designation - Blend

MARSHALL MIX DESIGN WORKSHEET

JOB NO: 96-F

CONTRACT NO: PG 53-34

DATE: April 1, 1996

MIX TYPE: HL 3

BLEND: CA#1- 43.8 %

% AC: 4.0

FA#1- 33.7 %

Gb: 2.682

% PASS 4.75mm: 55.0

FA#2- 22.5 %

% Fines Added: 1.0

75 BLOWS AT 138°C

		BRIQUETTE NUMBER			AVERAGE
		1	2	3	
1	MASS OF COMP. SPECIMEN IN AIR	1228.7	1232.3	1232.0	
2	SURF DRY MASS OF SPECIMEN IN AIR	1229.4	1232.9	1232.7	
3	MASS OF COMP. SPECIMEN IN WATER	709.0	710.2	712.2	
4	VOLUME OF BRIQUETTE	520.4	522.7	520.5	
5	BULK RELATIVE DENSITY	2.361	2.358	2.367	2.362
6	MASS OF FLASK & MIXTURE IN AIR				
7	MASS OF FLASK IN AIR				
8	MASS OF MIXTURE IN AIR				
9	SURFACE DRY MASS OF MIXTURE IN AIR				
10	MASS OF FLASK & MIXTURE IN WATER				
11	MASS OF FLASK IN WATER				
12	MASS OF MIXTURE IN WATER				
13	VOLUME OF SPECIMEN				
14	SURFACE DRY VOLUME OF SPECIMEN				
15	MAXIMUM RELATIVE DENSITY				2.534 *
16	MAXIMUM RELATIVE DENSITY, SURFACE DRY				
17	% VOIDS IN MIXTURE				6.8
18	SURFACE DRY VOIDS IN MIXTURE				**

MARSHALL TEST VALUES

	BRIQUETTE NUMBER			AVG	VISUAL OBSERVATIONS	
	1	2	3			
STABILITY (Dial)	10890	10550	11450		MIX APPEARANCE	DRY-MEDIUM
FLOW NO 1 (s)					BRIQ. APPEARANCE	DRY-MEDIUM
FLOW NO 2 (s)					COATING C. AGG	FAIR
					COATING F. AGG	FAIR
					STRIPPING	NIL-SLIGHT
AVERAGE FLOW	8.2	8.3	8.6	8.4	C. AGG FRACTURE	SLIGHT

STABILITY CORRECTION

BRIQ. NO	VOLUME CORRECT.	NEWTONS	CORRECT. NEWTONS	AVG
1	1.00	10890	10890	
2	0.96	10550	10128	10823
3	1.00	11450	11450	

SUMMARY OF TEST RESULTS

% VOIDS	6.8
% VOIDS, SSD	
FLOW, 0.25mm	8.4
CORRECTED STABILITY, N	10823
% VMA	15.5

* Calculated from Average Gse (2.704) at 5.0 - 5.5 % AC.

** No Visible Coarse Aggregate Absorption for MRD Mix at 5.0 - 5.5 % AC.
(AI MS-2, ASTM D2041).

MARSHALL MIX DESIGN WORKSHEET

JOB NO: 95-F
 MIX TYPE: HL 3
 % AC: 4.5
 % PASS 4.75mm: 55.0
 75 BLOWS AT 138°C

CONTRACT NO: PG 59-34
 BLEND: CA#1- 43.8 %
 FA#1- 33.7 %
 FA#2- 22.5 %

DATE: April 1, 1996
 Gb: 2.682
 % Fines Added: 1.0

		BRIQUETTE NUMBER			AVERAGE
		4	5	6	
1	MASS OF COMP. SPECIMEN IN AIR	1230.1	1231.6	1228.6	
2	SURF DRY MASS OF SPECIMEN IN AIR	1230.7	1232.2	1229.4	
3	MASS OF COMP. SPECIMEN IN WATER	711.7	714.4	712.3	
4	VOLUME OF BRIQUETTE	519.0	517.8	517.1	
5	BULK RELATIVE DENSITY	2.370	2.379	2.376	2.375
6	MASS OF FLASK & MIXTURE IN AIR				
7	MASS OF FLASK IN AIR				
8	MASS OF MIXTURE IN AIR				
9	SURFACE DRY MASS OF MIXTURE IN AIR				
10	MASS OF FLASK & MIXTURE IN WATER				
11	MASS OF FLASK IN WATER				
12	MASS OF MIXTURE IN WATER				
13	VOLUME OF SPECIMEN				
14	SURFACE DRY VOLUME OF SPECIMEN				
15	MAXIMUM RELATIVE DENSITY				2.515 *
16	MAXIMUM RELATIVE DENSITY, SURFACE DRY				
17	% VOIDS IN MIXTURE				5.6
18	SURFACE DRY VOIDS IN MIXTURE				**

MARSHALL TEST VALUES

	BRIQUETTE NUMBER			AVG	VISUAL OBSERVATIONS	
	4	5	6			
STABILITY (Dial)	10408	11988	11880		MIX APPEARANCE	MEDIUM
FLOW NO 1 (s)					BRIQ. APPEARANCE	MEDIUM
FLOW NO 2 (s)					COATING C. AGG	GOOD
					COATING F. AGG	GOOD
					STRIPPING	NIL-SLIGHT
AVERAGE FLOW	8.5	9.0	9.0	8.8	C. AGG FRACTURE	SLIGHT

STABILITY CORRECTION

BRIQ. NO	VOLUME CORRECT.	NEWTONS	CORRECT. NEWTONS	AVG
4	1.00	10408	10408	
5	1.00	11988	11988	11425
6	1.00	11880	11880	

SUMMARY OF TEST RESULTS

% VOIDS	5.6
% VOIDS, SSD	
FLOW, 0.25mm	8.8
CORRECTED STABILITY, N	11425
% VMA	15.4

* Calculated from Average Gse (2.704) at 5.0 - 5.5 % AC.

** No Visible Coarse Aggregate Absorption for MRD Mix at 5.0 - 5.5 % AC.
 (AI MS-2, ASTM D2041).

MARSHALL MIX DESIGN WORKSHEET

JOB NO: 96-F
 MIX TYPE: HL 3
 % AC: 5.0
 % PASS 4.75mm: 55.0
 75 BLOWS AT 138°C

CONTRACT NO: PG 53-34
 BLEND: CA#1- 43.8 %
 FA#1- 33.7 %
 FA#2- 22.5 %
 DATE: April 1, 1996
 Gb: 2.682
 % Fines Added: 1.0

		BRIQUETTE NUMBER			AVERAGE
		7	8	9	
1	MASS OF COMP. SPECIMEN IN AIR	1228.5	1247.4	1247.3	
2	SURF DRY MASS OF SPECIMEN IN AIR	1228.8	1247.8	1248.0	
3	MASS OF COMP. SPECIMEN IN WATER	716.1	726.0	726.6	
4	VOLUME OF BRIQUETTE	512.7	521.8	521.4	
5	BULK RELATIVE DENSITY	2.396	2.391	2.392	2.393
6	MASS OF FLASK & MIXTURE IN AIR	2127.0		2111.9	
7	MASS OF FLASK IN AIR	616.1		609.5	
8	MASS OF MIXTURE IN AIR	1510.9		1502.4	
9	SURFACE DRY MASS OF MIXTURE IN AIR				
10	MASS OF FLASK & MIXTURE IN WATER	1444.3		1432.9	
11	MASS OF FLASK IN WATER	538.3		532.4	
12	MASS OF MIXTURE IN WATER	906.0		900.5	
13	VOLUME OF SPECIMEN	604.9		601.9	
14	SURFACE DRY VOLUME OF SPECIMEN				
15	MAXIMUM RELATIVE DENSITY	2.498		2.496	2.495 *
16	MAXIMUM RELATIVE DENSITY, SURFACE DRY				
17	% VOIDS IN MIXTURE				4.1
18	SURFACE DRY VOIDS IN MIXTURE				**

MARSHALL TEST VALUES

	BRIQUETTE NUMBER			AVG	VISUAL OBSERVATIONS	
	7	8	9			
STABILITY (Dial)	12655	11698	12232		MIX APPEARANCE	MEDIUM-RICH
FLOW NO 1 (s)					BRIQ. APPEARANCE	MEDIUM-RICH
FLOW NO 2 (s)					COATING C. AGG	GOOD
					COATING F. AGG	GOOD
					STRIPPING	NIL
AVERAGE FLOW	9.3	9.2	9.2	9.2	C. AGG FRACTURE	NIL-SLIGHT

STABILITY CORRECTION

BRIQ. NO	VOLUME CORRECT.	NEWTONS	CORRECT. NEWTONS	AVG
7	1.00	12855	12855	
8	1.00	11698	11698	12262
9	1.00	12232	12232	

SUMMARY OF TEST RESULTS

% VOIDS	4.1
% VOIDS, SSD	
FLOW, 0.25mm	9.2
CORRECTED STABILITY, N	12262
% VMA	15.2

* Calculated from Average Gse (2.704) at 5.0 - 5.5 % AC. (Gse=2.707).

** No Visible Coarse Aggregate Absorption for MRD Mix at 5.0 - 5.5 % AC. (AI MS-2, ASTM D2041).

MARSHALL MIX DESIGN WORKSHEET

JOB NO: 96-F CONTRACT NO: PG 58-34 DATE: April 1, 1996
 MIX TYPE: HL 3 BLEND: CA#1- 43.8 %
 % AC: 5.5 FA#1- 33.7 % Gb: 2.682
 % PASS 4.75mm: 55.0 FA#2- 22.5 % % Fines Added: 1.0
 75 BLOWS AT 138°C

		BRIQUETTE NUMBER			AVERAGE
		10	11	12	
1	MASS OF COMP. SPECIMEN IN AIR	1227.0	1230.0	1231.0	
2	SURF DRY MASS OF SPECIMEN IN AIR	1227.4	1230.4	1231.5	
3	MASS OF COMP. SPECIMEN IN WATER	717.1	717.1	717.5	
4	VOLUME OF BRIQUETTE	510.3	513.3	514.0	
5	BULK RELATIVE DENSITY	2.404	2.396	2.395	2.399
6	MASS OF FLASK & MIXTURE IN AIR	2117.9		2112.7	
7	MASS OF FLASK IN AIR	616.1		609.5	
8	MASS OF MIXTURE IN AIR	1501.8		1503.2	
9	SURFACE DRY MASS OF MIXTURE IN AIR				
10	MASS OF FLASK & MIXTURE IN WATER	1432.7		1428.4	
11	MASS OF FLASK IN WATER	538.3		532.4	
12	MASS OF MIXTURE IN WATER	894.4		896.0	
13	VOLUME OF SPECIMEN	607.4		607.2	
14	SURFACE DRY VOLUME OF SPECIMEN				
15	MAXIMUM RELATIVE DENSITY	2.473		2.476	2.476 *
16	MAXIMUM RELATIVE DENSITY, SURFACE DRY				
17	% VOIDS IN MIXTURE				3.1
18	SURFACE DRY VOIDS IN MIXTURE				**

MARSHALL TEST VALUES

	BRIQUETTE NUMBER			AVG	VISUAL OBSERVATIONS	
	10	11	12			
STABILITY (Dial)	12130	11720	11330		MIX APPEARANCE	MEDIUM-RICH
FLOW NO 1 (s)					ERIQ. APPEARANCE	MEDIUM-RICH
FLOW NO 2 (s)					COATING C. AGG	GOOD
					COATING F. AGG	GOOD
					STRIPPING	NIL
AVERAGE FLOW	9.9	9.5	9.9	9.8	C. AGG FRACTURE	SLIGHT

STABILITY CORRECTION

BRIQ. NO	VOLUME CORRECT.	NEWTONS	CORRECT. NEWTONS	AVG
10	1.00	12130	12130	
11	1.00	11720	11720	11727
12	1.00	11330	11330	

SUMMARY OF TEST RESULTS

% VOIDS	3.1
% VOIDS, SSD	
FLOW, 0.25mm	9.8
CORRECTED STABILITY, N	11727
% VMA	15.5

* Calculated from Average Gse (2.704) at 5.0 - 5.5 % AC. (Gse=2.702).

** No Visible Coarse Aggregate Absorption for MRD Mix at 5.0 - 5.5 % AC. (AI MS-2, ASTM D2041).

MARSHALL MIX DESIGN WORKSHEET

JOB NO: 96-F
 MIX TYPE: HL 3
 % AC: 6.0
 % PASS 4.75mm: 55.0
 75 BLOWS AT 138°C

CONTRACT NO: PG 58-34
 BLEND: CA#1- 43.8 %
 FA#1- 33.7 %
 FA#2- 22.5 %

DATE: April 1, 1996
 Gb: 2.682
 % Fines Added: 1.0

		BRIQUETTE NUMBER			AVERAGE
		13	14	15	
1	MASS OF COMP. SPECIMEN IN AIR	1234.5	1231.5	1231.2	
2	SURF DRY MASS OF SPECIMEN IN AIR	1234.8	1231.7	1231.5	
3	MASS OF COMP. SPECIMEN IN WATER	719.9	717.1	716.8	
4	VOLUME OF BRIQUETTE	514.9	514.6	514.7	
5	BULK RELATIVE DENSITY	2.398	2.393	2.392	2.394
6	MASS OF FLASK & MIXTURE IN AIR				
7	MASS OF FLASK IN AIR				
8	MASS OF MIXTURE IN AIR				
9	SURFACE DRY MASS OF MIXTURE IN AIR				
10	MASS OF FLASK & MIXTURE IN WATER				
11	MASS OF FLASK IN WATER				
12	MASS OF MIXTURE IN WATER				
13	VOLUME OF SPECIMEN				
14	SURFACE DRY VOLUME OF SPECIMEN				
15	MAXIMUM RELATIVE DENSITY				2.457 *
16	MAXIMUM RELATIVE DENSITY, SURFACE DRY				
17	% VOIDS IN MIXTURE				2.6
18	SURFACE DRY VOIDS IN MIXTURE				**

MARSHALL TEST VALUES

	BRIQUETTE NUMBER			AVG	VISUAL OBSERVATIONS	
	13	14	15			
STABILITY (Dial)	11450	10950	11120		MIX APPEARANCE	RICH
FLOW NO 1 (s)					BRQ. APPEARANCE	RICH
FLOW NO 2 (s)					COATING C. AGG	GOOD
					COATING F. AGG	GOOD
					STRIPPING	NIL
AVERAGE FLOW	10.2	10.0	10.0	10.1	C. AGG FRACTURE	SLIGHT

STABILITY CORRECTION

BRIQ. NO	VOLUME CORRECT.	NEWTONS	CORRECT. NEWTONS	AVG
13	1.00	11450	11450	
14	1.00	10950	10950	11173
15	1.00	11120	11120	

SUMMARY OF TEST RESULTS

% VOIDS	2.6
% VOIDS, SSD	
FLOW, 0.25mm	10.1
CORRECTED STABILITY, N	11173
% VMA	16.1

* Calculated from Average Gse (2.704) at 5.0 - 5.5 % AC.

** No Visible Coarse Aggregate Absorption for MRD Mix at 5.0 - 5.5 % AC.
 (AI MS-2, ASTM D2041).

CHARTRE DE LA VISCOSITE CINEMATIQUE (MTQ-001)

VARIATION DE LA VISCOSITE EN FONCTION DE LA TEMPERATURE