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LOW TEMPERATURE CRACKING OF MODIFIED AC MIXES IN ALASKA

Volume 1: Field and Laboratory Studies

by

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FINAL REPORT

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ABSTRACT

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CHAPTER ONE
INTRODUCTION

1.1 PROBLEM STATEMENT

Low temperature cracking is a major distress mode in Alaskan pavements due to the extreme temperature conditions that range, in some instances, from about -50°C in winter to more than 40°C in summer. The mechanism of low temperature cracking is associated with tensile stresses in the asphalt concrete surface, that develop as the temperature drops to a very low value. Since the pavement surface is relatively restrained from contraction, the induced tensile stresses would increase as the surface temperature decreases, until at a given temperature, the stress becomes equal to the tensile strength of asphalt concrete surface, thereby resulting in the initiation of surface cracking. These cracks will propagate through the depth of the surface and increase in frequency with increase in pavement age and additional thermal cycling. Moisture infiltration through these cracks will accelerate the deterioration of the pavement structure and result in increased surface maintenance costs.

The Superpave design system, developed as part of the SHRP (Strategic Highway Research Program), is a comprehensive method of designing paving mixes in order to satisfy given performance requirements that are dictated by the traffic, the environment, and the structural section of the pavement. It includes procedures for selecting an asphalt binder (modified or unmodified) and mineral aggregates and combining them in the right proportions to reduce and control thermal cracking, in addition to other distress modes such as fatigue cracking and permanent deformation. According to the Superpave thermal cracking model, the indirect tensile creep and failure test (SHRP IDT) is used for mix evaluation to control thermal cracking. Predictions of this model for low temperature crack development with time were reported to be accurate on 40 field sections in Canada and the United States, including a number of C-SHRP test sections (Roque 1995).

Another test developed under the SHRP research program is the Thermal Stress Restrained Specimen Test (TSRST). This test is recommended by SHRP as a "proof test" whereby the measured fracture temperature may be compared to the historical low pavement temperatures associated with low temperature cracking. TSRST results, however, cannot be used as part of the Superpave thermal cracking model. Recent application of TSRST provided very good prediction for ranking low temperature cracking response of field sections with conventional hot-mix asphalt (Jung and Vinson 1994) and rubber-modified pavement sections (Raad et al. 1995).

In an attempt to minimize low temperature cracking (in addition to improved resistance to rutting), a number of asphalt pavements have been placed in Alaska over the past 15 years that contained additives such as polymers (both Styrene-Butadiene-Styrene [SBS] and Styrene-Butadiene Rubber [SBR]) and crumb rubber modifier (CRM). The low
temperature cracking performance of these pavements, in comparison with conventional hot-mix asphalt, needs to be evaluated in relation to thermal crack initiation and progression over time. Specifically, the application of the SHRP thermal cracking procedures and models need to be validated for Alaskan conditions. This will provide the engineers of the Alaska Department of Transportation and Public Facilities (AKDOT&PF) with predictive tools to evaluate the thermal cracking resistance of conventional mixes and the degree of improvement, if any, expected for different mix additives.

1.2 BACKGROUND

Over the last 30 years, research has identified a broad category of factors, that affect low temperature cracking and thermal fatigue of pavements, including material parameters, environmental conditions, and pavement structure and geometry. The single most important factor that influences the degree of low temperature cracking in an asphalt concrete mix is the stiffness and temperature susceptibility of the asphalt binder. Lower viscosity asphalts tend to produce a lower rate of increase in stiffness with decreasing temperature thereby reducing the potential for low temperature cracking (Carpenter and VanDam 1985; Anderson et al. 1989; and TRB 1989). The estimation of thermal stresses in asphalt concrete pavements is associated with the load-deformation-time relationship for the asphalt concrete mix obtained over a range of cold temperatures. A number of tests, such as creep (Monismith et al. 1965; Fromm and Phang 1972; Haas 1973), flexure bending (Busby and Rader 1972), direct tension (Haas 1973; Kallas 1982), and indirect diametral tension (Christison et al. 1972; Anderson and Leung 1987), have been used in determining the stress-strain-time relationship for asphalt concrete. Thermal stresses could be estimated from the strains associated with the cooling of the mix (determined from the coefficient of thermal expansion) and the corresponding stress-strain-time relationship. Alternatively, the development of thermal stresses may be determined in the laboratory by measuring the stress required to maintain a specimen at constant length when subjected to a given rate of cooling (Monismith et al. 1965; Fabb 1974; Sugawara and Moriyoshi 1984; Arand 1987). Mechanistic models that address thermal crack initiation and propagation under repeated thermal cycles have been proposed by a number of investigators (e.g., Majidzadeh et al. 1976 & 1977; Lytton and Shanmugham 1982; Little and Mahboub 1985). Accordingly, if the asphalt concrete is brittle, the time associated with crack initiation will constitute the major part of the thermal fatigue life, as crack propagation is very fast. On the other hand, for more ductile asphalt concrete, the rate of thermal crack propagation becomes relatively slow and the time needed to propagate the crack to failure could significantly influence the thermal fatigue life. Field data were used to develop regression equations for predicting low temperature cracking as a function of asphalt binder properties, mix properties, minimum pavement temperature, and pavement geometry (Fromm and Phang 1972; Haas et al. 1987).

The SHRP asphalt program resulted in the development of performance-based tests to control thermal cracking in asphalt pavements. Two laboratory tests have been developed as part of the Superpave system: 1) the bending beam rheometer test (BBR) (Bahia and
Anderson 1992; Petersen et al. 1993); and 2) the indirect tensile creep and failure test (SHRP IDT) (Roque and Buttlar 1992; Lytton et al. 1993; Buttlar and Roque 1994). Binder and mixture properties determined from BBR and SHRP IDT are used by the mixture model within the Superpave to calculate the viscoelastic and fracture properties necessary to predict thermal stresses and crack development. Pavement material types and thicknesses, air temperature data (maximum and minimum daily air temperature for the analysis period) and the latitude of the site are also needed by Superpave to evaluate pavement temperature for analysis. Mix properties, pavement temperature, and asphalt layer thickness are then used as part of the Superpave thermal cracking model to estimate cracking as a function of time. According to Roque et al. (1995), Superpave's thermal cracking model has been proved accurate on nearly 40 field sections in the United States and Canada. Performance predictions using Superpave thermal cracking model for Level III analysis showed excellent agreement with field observations for all seven test sections evaluated as part of the C-SHRP study in Alberta, Canada (Roque and Hiltunen 1994).

The thermal stress restrained specimen test (TSRST) was developed at Oregon State University under the SHRP A-003A contract as an accelerated laboratory test to evaluate the low temperature cracking resistance of asphalt concrete mixes (Jung and Vinson 1994; Kanerva et al. 1994). In this test, rectangular or cylindrical specimens are cooled at a constant rate as they are restrained from contraction. The tensile load and specimen surface temperature are monitored periodically until the specimen fails. TSRST is identified by SHRP as a proof test to provide an independent confirmation of routine Superpave test results and performance predictions related to low temperature cracking (Cominsky 1994). The fracture strength and temperature are evaluated only by comparative analysis, and no prediction using Superpave models can be made. Applications of TSRST to study the thermal cracking properties of a number of mixes were reported by Jung and Vinson (1994). Results of fracture temperature were in excellent agreement with the asphalt binder low temperature index tests, specifically, the limiting stiffness temperature (obtained from BBR) and the ultimate strain at failure (obtained from the direct tension test). Good correlations were also obtained between TSRST fracture temperature and the observed frequency of thermal cracks in a number of field sections (Kanerva et al. 1994).

Recently, Raad et al. (1995) published the results of a study entitled "Thermal Cracking Resistance of Rubber Modified Pavements" funded by AKDOT&PF under project SPR-UAF-93-03-C. The study aimed at determining the low temperature cracking resistance of rubber modified pavements in Alaska in comparison with conventional asphalt concrete pavements. Laboratory tests were conducted on field specimens using TSRST. Tested materials included: 1) conventional HMA with AC-2.5 and AC-5; 2) PlusRide RUMAC with AC-5; 3) asphalt-rubber concrete with AC-2.5 (wet process); 4) rubberized asphalt-rubber concrete with AC 2.5 (wet/dry process); 5) gap-graded asphalt-rubber hot-mix with AR 4000 (wet process); and 6) dense-graded HMA with AR 4000. The improvement of low temperature performance in terms of fracture temperature and strength was demonstrated for the rubberized mixes over conventional mixes. The resistance to thermal cracking was most significant for the asphalt-rubber and rubberized asphalt-rubber
materials. Field observations and performance comparisons confirmed the laboratory based predictions. In this case, TSRST results were essentially used to rank the thermal cracking resistance of the different mixes considered. Additional research should concentrate on determining the thermal cracking development over time for both conventional and modified Alaskan pavements and its relationship to SHRP recommended procedures.

1.3 Objectives

Asphalt modifiers have been used in Alaskan pavements over the past 15 years. These modifiers include SBR polymers, SBS polymers, Ultrapave, and CRM (both the dry process, PlusRide, and the wet process). Field observations and laboratory studies, performed in Alaska and elsewhere, indicate that the use of these modifiers would improve the low temperature cracking resistance of pavements. The degree of improvement and economical benefits these modifiers provide for Alaskan pavements need to be evaluated. Specifically, the objectives of this research were as follows:

1. To classify asphalt and polymer modified asphalts from a number of selected sites using SHRP Superpave PG grading system, and to characterize the corresponding asphalt and polymer modified mixes using TSRST and Superpave IDT laboratory tests on field specimens.

2. To compare the low temperature cracking performance of the selected field pavements through field surveys and assess observed performance in relation to Superpave binder specifications and TSRST results.

3. To recommend guidelines for predicting minimum pavement temperatures that could be used by AKDOT&PF engineers in thermal cracking analysis. These guidelines would be established from air and pavement temperature measurements at the selected sites and would be compared with Superpave recommended procedures and other appropriate models in Canada and the United States. Recommended minimum pavement temperatures for the Alaskan Road system would be mapped in a simple contour form, to allow AKDOT&PF engineers to select the appropriate asphalt mix, with binder modification if necessary, to meet Superpave binder specifications. The asphalt mix could also be selected so that the predicted minimum pavement temperature will not be colder than the fracture temperature determined from TSRST results. This approach, for example, could be used for estimating thermal crack initiation.

4. To use SHRP's Superpave thermal cracking model (TCMODEL) and other available models to evaluate the resistance of conventional and modified Alaskan mixes to thermal cracking, and to develop improved models, if necessary, for predicting thermal crack progression over time for typical Alaskan pavements.
5. To assess the economic benefits of using polymer modified asphalts as a result of improved low temperature cracking resistance in comparison with conventional asphalts.

1.4 SCOPE OF WORK

This study covered field instrumentation and data collection for air and pavement temperature at selected sites; field sampling (core and slab samples); Superpave binder specification tests (Level I), for both the modified and unmodified binders used in the pavement sections considered (conventional asphalt binders were extracted and tested; modified binders were re-constituted according to the original specifications used during construction); Superpave TCMODEL tests including BBR and SHRP IDT; and thermal cracking tests using TSRST. All laboratory tests were conducted, where applicable, according to SHRP Superpave testing procedures and specifications. Field observations and measurements of low temperature cracking were monitored to determine thermal crack progression over time and to verify predictions by Superpave and TSRST.

Air and pavement temperature data at selected sites were used to develop correlations between minimum air and pavement temperature and verify other pavement temperature prediction models and studies recently conducted in Canada and the United States. These correlations were used together with air temperature data from the Superpave Weather Database to develop contour maps for estimating minimum pavement temperature for Alaskan conditions. These maps could be used by AKDOT&PF engineers for selecting the appropriate asphalt or modified binder according to the Superpave binder specifications.

The Superpave TCMODEL was used as part of the Superpave system to predict the development of thermal cracking with time. SHRP BBR tests, SHRP IDT, mixture data, layer type and thickness, and the latitude of the pavement site (used to estimate the amount of solar radiation reaching the pavement) were used in the analysis.

TSRST studies were performed on field specimens from all selected sites. Correlation between TSRST fracture temperature, fracture strength, mix age, minimum site temperature, and the degree of thermal cracking were developed using field surveys for low temperature cracking for the various sites. This could provide a simpler and more direct means, as compared with the Superpave analysis, for predicting thermal cracking resistance.

Results of field temperature measurements, Superpave analysis, and TSRST studies were used to evaluate the improved resistance of Alaskan pavements with modified binders to thermal cracking. The relative beneficial effects of different modifiers were compared. In addition, predictive models for thermal crack development over time were evaluated and improved models were recommended. AKDOT&PF engineers could use these models to select appropriate modifiers and mix properties in order to minimize thermal cracking in the pavement structure.
Results of field thermal cracking surveys and model predictions were used to assess the economic benefits of using polymer modifiers as a result of improved low temperature cracking resistance in comparison with conventional asphalts.

1.5 ORGANIZATION OF REPORT

This report is organized as follows:

1. Chapter One - Introduction

Problem definition, research objectives and scope of work are presented together with some background information on low temperature cracking of asphalt concrete pavements.

2. Chapter Two - Field Studies

Field studies covering both temperature measurement and low temperature cracking surveys are described. Air and pavement temperature data are presented and typical correlations for individual sites are summarized. Comparisons of low temperature cracking performance of asphalt and polymer modified asphalt pavements are discussed.

3. Chapter Three - Laboratory Studies

SHRP Superpave binder tests and results for asphalt and polymer modified asphalt binders used in the field test sections are summarized. Laboratory tests including TSRST and Superpave IDT on field specimens are also described and a summary of all test results is presented. The significance of results from binder tests and TSRST on observed low temperature cracking behavior is illustrated.

4. Chapter Four - Pavement Temperature Modeling and Mapping

Correlations between minimum air and pavement temperatures are presented in this Chapter for Alaska's climatic zones. These correlations together with the Superpave Weather Database are used to develop contour maps for Alaska for minimum air and minimum pavement temperatures. Comparisons of air-pavement temperature correlations developed for Alaskan conditions and other relations proposed by Superpave and the Asphalt Institute are presented.

5. Chapter Five - Thermal Cracking Models for Alaskan Pavements

Field observations of low temperature cracking performance are compared with predictions using Superpave TCMODEL and other available models. Improved models using TSRST test results, pavement age, and minimum temperature are developed using
field and laboratory data of the surveyed field sections. The economic benefit for using polymer modified asphalts is discussed in relation to reduced thermal cracking and crack sealing costs.

6. Chapter Six - *Summary, Conclusions, and Recommendations*

Summary and conclusions of the research findings are presented. A design methodology for low temperature cracking of Alaskan pavements in relation to crack initiation and progression with time is recommended. Results obtained are used to identify future research needs.
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CHAPTER TWO
FIELD STUDIES

2.1 INTRODUCTION

Field studies were conducted as part of this research project to develop a better understanding of the low temperature cracking behavior of asphalt and polymer modified asphalt pavements in Alaska. These studies concentrated on pavement temperature measurements and thermal cracking surveys of selected sections. In this chapter, location and description of the field sites are presented. Site instrumentation and data collection and analysis procedures are described. Results of analyses are also summarized and discussed.

2.2 PAVEMENT TEMPERATURE MEASUREMENT

The evaluation of low temperature cracking of Alaskan pavements necessitates a careful and thorough analysis of pavement temperature particularly in relation to thermal crack initiation and propagation. In this case, the determination of minimum pavement temperature distribution for the various climatic zones in Alaska becomes important in the selection of the appropriate asphalt or polymer modified asphalt in order to minimize the potential of pavement cracking. The objectives of the field pavement temperature study include:

1. Determine the distribution of minimum pavement temperature for different climatic zones in Alaska.

2. Develop appropriate procedures, including maps and/or tables in order to provide engineering estimates for design pavement temperatures associated with low temperature cracking.

3. Develop appropriate correlations between air and pavement temperature that could be used as part of pavement temperature modeling and crack progression analysis.

2.2.1 Field Instrumentation and Data Collection

A total of 12 sites were instrumented to record both air temperature and pavement temperature. The list of sites including location and route as referenced from the Coordinated Data System (CDS) is summarized in Table 2.1. All these sites have been instrumented with 16 thermistors starting at 25 mm to 50 mm below the pavement surface and continuing to a depth of 182 cm. In addition, each site was instrumented with additional sensors for measuring pavement surface temperature and ambient air temperature. The extra pavement surface sensors were placed at approximately 10 mm below the surface. The Highway Data Section (HDS) of the AKDOT&PF has been
Table 2.1: Instrumented Sites for Temperature Data Collection

<table>
<thead>
<tr>
<th>Name</th>
<th>Route No.</th>
<th>MilePoint (CDS Miles)</th>
<th>Approximate Proximity (Miles)</th>
<th>Highway/Road</th>
</tr>
</thead>
<tbody>
<tr>
<td>Homer (HOM)</td>
<td>110000</td>
<td>4.160</td>
<td>In Homer</td>
<td>Sterling</td>
</tr>
<tr>
<td>Soldotna (SOL)</td>
<td>110000</td>
<td>86.71</td>
<td>6,000 E. of Soldotna</td>
<td>Sterling</td>
</tr>
<tr>
<td>Dutch Harbor (UNA)</td>
<td>070800</td>
<td>2.800</td>
<td>2.00 N. of Dutch Harbor</td>
<td>Unalaska</td>
</tr>
<tr>
<td>Kenai Spur (KSP)</td>
<td>117600</td>
<td>7.000</td>
<td>3.000 W. of Kenai</td>
<td>Kenai Spur</td>
</tr>
<tr>
<td>Kenai River Crossing (KPX)</td>
<td>117626</td>
<td>3.000</td>
<td>3.000 S. of Kenai</td>
<td>Kenai River</td>
</tr>
<tr>
<td>Glenn Hwy MP53 (G53)</td>
<td>130000</td>
<td>173.6</td>
<td>6.000 N. of Palmer</td>
<td>Glenn</td>
</tr>
<tr>
<td>Palmer (PAL)</td>
<td>136800</td>
<td>9.200</td>
<td>4.000 W. of Palmer</td>
<td>Palmer Wasilla</td>
</tr>
<tr>
<td>Chulitna (CHU)</td>
<td>170000</td>
<td>83.00</td>
<td>40 N. of Willow</td>
<td>Parks</td>
</tr>
<tr>
<td>Peger Road (PEG)</td>
<td>176100</td>
<td>0.78</td>
<td>In Fairbanks</td>
<td>Airport Way</td>
</tr>
<tr>
<td>Fox (FOX)</td>
<td>150000</td>
<td>9.580</td>
<td>9.580 N. of Fairbanks</td>
<td>Steese</td>
</tr>
<tr>
<td>Tok (TOK)</td>
<td>230000</td>
<td>122.66</td>
<td>In Tok</td>
<td>Tok cut-off</td>
</tr>
<tr>
<td>Cantwell (CANT)</td>
<td>170000</td>
<td>174.58</td>
<td>Parks Hwy. at Cantwell</td>
<td>Parks</td>
</tr>
</tbody>
</table>
downloading all sites (except for Peger Road [PEG], Cantwell [CANT], and Tok [TOK]) periodically. Data storage at PEG, CANT, and TOK sites is battery operated and could not be downloaded automatically. Data were recorded in formatted arrays identified by array number, which is the first field within each record. Two records were used, one for the temperature probe thermistors identified by array number 210, and one for the extra sensors which could have a number of array identifiers such as 103, 104, 106, or 107. A record format specification for each type of array is as follows:

Temperature probe thermistor data array: 210

<table>
<thead>
<tr>
<th>Field</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Array Number</td>
</tr>
<tr>
<td>2</td>
<td>Year</td>
</tr>
<tr>
<td>3</td>
<td>Julian Date</td>
</tr>
<tr>
<td>4</td>
<td>Time (Military)</td>
</tr>
<tr>
<td>5</td>
<td>Thermistor 1 (Closest to the surface)</td>
</tr>
<tr>
<td>...</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>Thermistor 16 (Furthest from the surface)</td>
</tr>
</tbody>
</table>

Extra sensor data array: 103, 104, 106, 107

<table>
<thead>
<tr>
<th>Field</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Array Number</td>
</tr>
<tr>
<td>2</td>
<td>Year</td>
</tr>
<tr>
<td>3</td>
<td>Julian Date</td>
</tr>
<tr>
<td>4</td>
<td>Time (Military)</td>
</tr>
<tr>
<td>5</td>
<td>Cabinet Temperature Sensor</td>
</tr>
<tr>
<td>6</td>
<td>Ambient Air Temperature Sensor</td>
</tr>
<tr>
<td>7</td>
<td>Surface Temperature Sensor</td>
</tr>
<tr>
<td>8</td>
<td>Battery Voltage (If battery is used)</td>
</tr>
</tbody>
</table>

Temperature data for this study covered the months of December 1995 and January 1996. In a number of cases the data were not complete. Some December and January data were also missing for a number of sites (i.e., Kenai river Crossing [KPx], Palmer [PAL], and Chulitna [CHU]). The analysis of the data presented in this report is based on the available December and January data.

2.2.2 Analysis of Temperature Data

Air and pavement temperature data were analyzed for each site for the periods of December 1995 and January 1996 separately, as well as for the period covering both months as follows:
1. The distributions of average daily air temperature, minimum daily air temperature, and maximum daily air temperature were illustrated.

2. Comparison of the variations of average daily air temperature and average daily pavement temperature were developed. Similar comparisons between minimum daily air temperature and minimum daily pavement temperature, and maximum daily air temperature and maximum daily pavement temperature were also performed.

3. Correlation equations between average daily air temperature and average daily pavement temperature, minimum daily air temperature and minimum daily pavement temperature, maximum daily air temperature and maximum daily pavement temperature were established.

Results of air and pavement temperature comparisons and correlations for every site are presented in Appendix A. These results have been used in Chapter 4 to: 1) develop air-pavement temperature relationships; 2) validate existing prediction methods, including SHRP Superpave, for minimum pavement temperature, and 3) develop contour maps for minimum air and pavement temperature in Alaska.

2.3 LOW TEMPERATURE CRACKING SURVEYS

Surveys were conducted in Fall 1995 and Spring 1996 for a number of pavement sections covering a wide range of climatic zones. The purpose of these surveys was to evaluate the low temperature cracking performance of typical Alaskan pavements with and without polymer modifiers, specifically:

1. Compare the low temperature cracking performance for sections with similar age having both conventional and polymer modified binders.

2. Compare the low temperature cracking performance for different types of polymer modified binders.

3. Compare the low temperature cracking performance for conventional binders at different climatic zones.

4. Use field survey data to recommend low temperature crack progression models for Alaskan pavements.

Items 1-3 are addressed in this Chapter, whereas item 4 is discussed in Chapter 5.
2.3.1 Selection and Survey of Field Sections

A total of 16 pavement sections were selected for this study. These pavements cover a number of climatic zones including maritime, transitional, continental, and arctic. A summary of section properties and location is presented in Tables 2.2 and 2.3. Binder description is shown in Table 2.4. Available details on binder and mix data and specification are included in Appendix B. Condition surveys were conducted in Fall 95 and Spring 96 to determine the extent of low temperature cracking and its progression with time. Field data for each of the surveyed pavements were presented according to the following format:

1. Variation of average crack density (cracks/km) with distance measured relative to a reference location at the start of the surveyed section
2. Variation of average crack spacing
3. Variation of average length of cracks per km
4. Distribution of crack spacing
5. Variation of the cumulative number of observed cracks (expressed in % less than) as a function of crack spacing

A typical data representation for low temperature cracking of the pavement section at Denali is shown in Figures 2.1-2.5. Data for all sites surveyed are presented in Appendix C. A summary of average crack spacing and average crack density is shown in Tables 2.5 and 2.6.

2.3.2 Low Temperature Cracking Performance

Comparison of Polymer Modified and Conventional Sections (Similar Age)

a. C-Street-Anchorage (Conventional AC-5 mix, 1986) and A-Street-Anchorage (Rubberized mix, PlusRide 1985):

The PlusRide section exhibits better low temperature cracking performance when compared with the conventional section. The average crack spacing after 10 years of service was 41 m in comparison with 29 m for the conventional section. The corresponding crack density was 29 cracks/km for the PlusRide section and 45 cracks/km for the conventional pavement. The distribution of crack spacing for the two sections is best illustrated in Figures 2.6 and 2.7. For the 1995 data, the crack spacing for the conventional section is finer than the PlusRide section. However, the 1996 survey indicates that the crack spacing of the two sections is similar.
### Table 2.2: Location and Section Properties of Surveyed Sites

<table>
<thead>
<tr>
<th>Project ID</th>
<th>Site</th>
<th>Binder Type</th>
<th>Placed in (1)</th>
<th>Layer Thickness, (cm)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>C street, Anchorage</td>
<td>AC-5</td>
<td>1986</td>
<td>5</td>
<td>8</td>
</tr>
<tr>
<td>n</td>
<td>A street, Anchorage</td>
<td>PlusRide</td>
<td>1985</td>
<td>7.5</td>
<td>5</td>
</tr>
<tr>
<td>b</td>
<td>Elmendorf AFB</td>
<td>AC-5+3%SBS</td>
<td>1995</td>
<td>6.5</td>
<td>-</td>
</tr>
<tr>
<td>c</td>
<td>Elmendorf AFB</td>
<td>AC-5+6%SBS</td>
<td>1991</td>
<td>6.5</td>
<td>-</td>
</tr>
<tr>
<td>e1</td>
<td>Haines Hwy.</td>
<td>PBA3</td>
<td>1993</td>
<td>5</td>
<td>-</td>
</tr>
<tr>
<td>e2</td>
<td>Haines Hwy.</td>
<td>AC-5</td>
<td>1991</td>
<td>5</td>
<td>-</td>
</tr>
<tr>
<td>f</td>
<td>Denali Park Road</td>
<td>AC-20R</td>
<td>1991</td>
<td>4</td>
<td>7.5</td>
</tr>
<tr>
<td>g1</td>
<td>Danby Str., Fairbanks</td>
<td>Asphalt-Rubber</td>
<td>1988</td>
<td>5</td>
<td>-</td>
</tr>
<tr>
<td>g2</td>
<td>Danby Str., Fairbanks</td>
<td>AC-2.5</td>
<td>1988</td>
<td>5</td>
<td>-</td>
</tr>
<tr>
<td>h2</td>
<td>Parks Hwy., Fairbanks</td>
<td>AC-5</td>
<td>1995</td>
<td>7.5</td>
<td>-</td>
</tr>
<tr>
<td>h1</td>
<td>Parks Hwy., Fairbanks</td>
<td>PBA3</td>
<td>1995</td>
<td>7.5</td>
<td>-</td>
</tr>
<tr>
<td>i</td>
<td>Badger Rd.</td>
<td>SHRP Level I</td>
<td>1995</td>
<td>5</td>
<td>-</td>
</tr>
<tr>
<td>j</td>
<td>Eielson AFB</td>
<td>Arctic Grade AC-5</td>
<td>1986</td>
<td>15</td>
<td>4.5</td>
</tr>
<tr>
<td>k</td>
<td>Fort Wainwright</td>
<td>Arctic Grade AC-2.5</td>
<td>1990</td>
<td>4</td>
<td>6.5</td>
</tr>
<tr>
<td>l</td>
<td>Rewak Dr., Fairbanks</td>
<td>PBA6</td>
<td>1993</td>
<td>5</td>
<td>-</td>
</tr>
<tr>
<td>m</td>
<td>Deadhorse</td>
<td>Arctic Grade AC-2.5</td>
<td>1992</td>
<td>7.5</td>
<td>2.5</td>
</tr>
</tbody>
</table>

**Note:**

(1) : Date when the uppermost layer is placed
Table 2.3: Geographic Location and Low Air Temperature of Surveyed Sites

<table>
<thead>
<tr>
<th>Project ID</th>
<th>Project Site 1</th>
<th>Closest Weather Station 1</th>
<th>Longitude</th>
<th>Latitude</th>
<th>Low Air Temperature (C) (1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>C street, Anchorage</td>
<td>Anchorage WB AP</td>
<td>149.83</td>
<td>61.22</td>
<td>-29</td>
</tr>
<tr>
<td>n</td>
<td>A street, Anchorage</td>
<td>Anchorage WB AP</td>
<td>149.83</td>
<td>61.22</td>
<td>-29</td>
</tr>
<tr>
<td>b, c</td>
<td>Elmendorf AFB</td>
<td>Elmendorf AFB</td>
<td>149.80</td>
<td>61.25</td>
<td>-28</td>
</tr>
<tr>
<td>e1, e2</td>
<td>Haines Hwy.</td>
<td>Haines Terminal</td>
<td>135.45</td>
<td>59.27</td>
<td>-19</td>
</tr>
<tr>
<td>f</td>
<td>Denali Park</td>
<td>Mc. Kinley Park</td>
<td>148.97</td>
<td>63.72</td>
<td>-40</td>
</tr>
<tr>
<td>g1, g2</td>
<td>Danby Str., Fairbanks</td>
<td>Fairbanks WSFO AP</td>
<td>147.87</td>
<td>64.82</td>
<td>-46</td>
</tr>
<tr>
<td>h1, h2</td>
<td>Parks Hwy., Fairbanks</td>
<td>Fairbanks WSFO AP</td>
<td>147.87</td>
<td>64.82</td>
<td>-46</td>
</tr>
<tr>
<td>i</td>
<td>Badger Rd.</td>
<td>North Pole</td>
<td>147.33</td>
<td>64.75</td>
<td>-48</td>
</tr>
<tr>
<td>j</td>
<td>Eielson AFB</td>
<td>Eielson AFB</td>
<td>147.10</td>
<td>64.67</td>
<td>-45</td>
</tr>
<tr>
<td>k</td>
<td>Fort Wainwright, Fairbanks</td>
<td>Fairbanks WSFO AP</td>
<td>147.87</td>
<td>64.82</td>
<td>-46</td>
</tr>
<tr>
<td>l</td>
<td>Rewak Dr., Fairbanks</td>
<td>Fairbanks WSFO AP</td>
<td>147.87</td>
<td>64.82</td>
<td>-46</td>
</tr>
<tr>
<td>m</td>
<td>Deadhorse</td>
<td>Barter Island WPO AP</td>
<td>143.63</td>
<td>70.13</td>
<td>-44</td>
</tr>
</tbody>
</table>

Note:
(1): Low Air Temperatures obtained from Superpave Weather Database
Table 2.4: Binder Description for the Different Projects

<table>
<thead>
<tr>
<th>Project ID</th>
<th>Site</th>
<th>Binder Type</th>
<th>Binder Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>C street, Anchorage</td>
<td>AC-5</td>
<td>Chevron AC-5; 275 F viscosity = 220 cSt</td>
</tr>
<tr>
<td>n</td>
<td>A street, Anchorage</td>
<td>PlusRide</td>
<td>Chevron AC-5; 2.5% tire rubber by total mix in the gap-graded aggregate</td>
</tr>
<tr>
<td>b</td>
<td>Elmendorf AFB</td>
<td>AC-5+3%SBS</td>
<td>Kraton 1101; 275 F viscosity = 500-600 cSt</td>
</tr>
<tr>
<td>c</td>
<td>Elmendorf AFB</td>
<td>AC-5+6%SBS</td>
<td>Kraton 1101; 275 F viscosity = 1000-1200 cSt</td>
</tr>
<tr>
<td>e1</td>
<td>Haines Hwy.</td>
<td>PBA3</td>
<td>Husky 150/200 pen + Ultrapave 70&lt;sup&gt;1&lt;/sup&gt;</td>
</tr>
<tr>
<td>e2</td>
<td>Haines Hwy.</td>
<td>AC-5</td>
<td>Chevron AC-5; 275 F viscosity = 208 cSt</td>
</tr>
<tr>
<td>f</td>
<td>Denali Park Road</td>
<td>AC-20 R</td>
<td>AC-5+3% Goodyear SBR UP-70; 275 F viscosity = 1600-1800 cSt</td>
</tr>
<tr>
<td>g1</td>
<td>Danby Str., Fairbanks</td>
<td>Asphalt-Rubber</td>
<td>78% Mapco AC-2.5+18% Atlas1515 ground tire rubber+4% extender oil</td>
</tr>
<tr>
<td>g2</td>
<td>Danby Str., Fairbanks</td>
<td>AC-2.5</td>
<td>Mapco AC-2.5</td>
</tr>
<tr>
<td>h2</td>
<td>Parks Hwy., Fairbanks</td>
<td>AC-5</td>
<td>Mapco AC-5; 275 F viscosity = 220 cSt</td>
</tr>
<tr>
<td>h1</td>
<td>Parks Hwy., Fairbanks</td>
<td>PBA3</td>
<td>AC-5+2%SBS+2%SBR; 275 F viscosity = 1250 cSt</td>
</tr>
<tr>
<td>i</td>
<td>Badger Rd.</td>
<td>SHRP Level I</td>
<td>Mapco AC-5; 275 F viscosity = 220 cSt&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
<tr>
<td>j</td>
<td>Eielson AFB</td>
<td>Arctic Grade AC-5</td>
<td>Mapco AC-5; 275 F viscosity = 220 cSt&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
<tr>
<td>k</td>
<td>Fort Wainwright</td>
<td>Arctic Grade AC-2.5</td>
<td>Brower AC-2.5+2% Elvax 150 W</td>
</tr>
<tr>
<td>l</td>
<td>Rewak Dr., Fairbanks</td>
<td>PBA6</td>
<td>Binder exact spec. are unknown for this project&lt;sup&gt;3&lt;/sup&gt;</td>
</tr>
<tr>
<td>m</td>
<td>Deadhorse</td>
<td>Arctic Grade AC-2.5</td>
<td>AC-5+6% SBS Kraton 1101</td>
</tr>
</tbody>
</table>

Note:

<sup>1</sup>: In-line injection of polymer at hot plant; binder was supposed to meet PBA3 spec.
<sup>2</sup>: This is a conventional mix; only the aggregate followed SHRP spec.
<sup>3</sup>: Binder meets AC-2.5 spec. and a minimum PVN of -0.2
Figure 2.1: Crack Density for Denali Park Road - Fall'95 & Spring'96 Surveys
Figure 2.2: Average Crack Spacing for Denali Park Road - Fall'95 & Spring'96 Surveys
Figure 2.3: Crack Length per km for Denali Park Road - Fall'95 & Spring'96 Surveys
Figure 2.4: Crack Spacing Histograms for Denali Park Road - Fall'95 & Spring'96 Surveys
Figure 2.5: Crack Spacing Distribution for Denali Park Road - Fall'95 & Spring'96 Surveys
Table 2.5: Crack Spacing for 1995 and 1996 Surveys

<table>
<thead>
<tr>
<th>Project ID</th>
<th>Site / Material</th>
<th>Fall'95 survey</th>
<th>Spring'96 survey</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Transverse Crack Spacing, (m)</td>
<td>Average</td>
<td>Stdev.</td>
</tr>
<tr>
<td>a</td>
<td>C Str., Anchorage</td>
<td>29.1</td>
<td>13.7</td>
</tr>
<tr>
<td>n</td>
<td>A Str., Anchorage</td>
<td>41.5</td>
<td>24.1</td>
</tr>
<tr>
<td>b</td>
<td>Elmendorf 3% SBS</td>
<td>(2)</td>
<td>15</td>
</tr>
<tr>
<td>c</td>
<td>Elmendorf 6% SBS</td>
<td>26.7</td>
<td>13</td>
</tr>
<tr>
<td>e1</td>
<td>Haines PBA3</td>
<td>(2)</td>
<td>155</td>
</tr>
<tr>
<td>e2</td>
<td>Haines AC-5</td>
<td>(2)</td>
<td>(3)</td>
</tr>
<tr>
<td>f</td>
<td>Denali (4)</td>
<td>42.3</td>
<td>19.3</td>
</tr>
<tr>
<td>g1</td>
<td>Danby AR</td>
<td>31.9</td>
<td>12.8</td>
</tr>
<tr>
<td>g2</td>
<td>Danby AC2.5</td>
<td>(1)</td>
<td>(1)</td>
</tr>
<tr>
<td>h2</td>
<td>Parks AC5</td>
<td>(2)</td>
<td>(2)</td>
</tr>
<tr>
<td>h1</td>
<td>Parks PBA3</td>
<td>(2)</td>
<td>(2)</td>
</tr>
<tr>
<td>i</td>
<td>Badger Rd.</td>
<td>(2)</td>
<td>(2)</td>
</tr>
<tr>
<td>j</td>
<td>Eielson AFB (4)</td>
<td>7.2</td>
<td>2.2</td>
</tr>
<tr>
<td>k</td>
<td>Ft. Wainwright (4)</td>
<td>27.3</td>
<td>12.2</td>
</tr>
<tr>
<td>l</td>
<td>Rewak Dr.</td>
<td>(2)</td>
<td>(2)</td>
</tr>
<tr>
<td>m</td>
<td>Deadhorse (4)</td>
<td>23.1</td>
<td>8.8</td>
</tr>
</tbody>
</table>

Note: (1): no survey was done  
(2): no cracks were detected  
(3): one crack only for 266 m surveyed  
(4): projects f, j, k, m are overlays
Table 2.6 : Crack Density for 1995 and 1996 Surveys

<table>
<thead>
<tr>
<th>Project ID</th>
<th>Site / Material</th>
<th>Length of section surveyed, (m)</th>
<th>Total number of cracks</th>
<th>Average number of cracks / km</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Fall'95 survey</td>
<td>Spring'96 survey</td>
</tr>
<tr>
<td>a</td>
<td>C Str., Anchorage</td>
<td>1430</td>
<td>44</td>
<td>45</td>
</tr>
<tr>
<td>n</td>
<td>A Str., Anchorage</td>
<td>1213</td>
<td>29</td>
<td>36</td>
</tr>
<tr>
<td>b</td>
<td>Elmendorf 3%SBS</td>
<td>791 (2)</td>
<td>29</td>
<td>34</td>
</tr>
<tr>
<td>c</td>
<td>Elmendorf 6%SBS</td>
<td>775 (2)</td>
<td>29</td>
<td>34</td>
</tr>
<tr>
<td>e1</td>
<td>Haines PBA3</td>
<td>3600 (2)</td>
<td>13</td>
<td>13 (2)</td>
</tr>
<tr>
<td>e2</td>
<td>Haines AC-5</td>
<td>266 (2)</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>f</td>
<td>Denali</td>
<td>22200</td>
<td>541</td>
<td>734 (2)</td>
</tr>
<tr>
<td>g1</td>
<td>Danby AR</td>
<td>96</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>g2</td>
<td>Danby AC2.5</td>
<td>96 (1)</td>
<td>17</td>
<td>(1)</td>
</tr>
<tr>
<td>h2</td>
<td>Parks AC5</td>
<td>163 (2)</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td>h1</td>
<td>Parks PBA3</td>
<td>201 (2)</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>i</td>
<td>Badger Rd.</td>
<td>15450 (2)</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>j</td>
<td>Eielson AFB</td>
<td>200</td>
<td>28</td>
<td>31</td>
</tr>
<tr>
<td>k</td>
<td>Ft. Wainwright</td>
<td>326</td>
<td>11</td>
<td>14</td>
</tr>
<tr>
<td>l</td>
<td>Rewak Dr.</td>
<td>149 (2)</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>m</td>
<td>Deadhorse</td>
<td>675 (2)</td>
<td>30</td>
<td>32</td>
</tr>
</tbody>
</table>

Note: (1) : no survey was done
(2) : no cracks were detected
C street, Anchorage
AC-5

Figure 2.6: Crack Spacing Distribution for C street, Anchorage - Fall'95 & Spring'96 Surveys
Figure 2.7: Crack Spacing Distribution for A street, Anchorage - Fall'95 & Spring'96 Surveys.
b. Haines Highway (Conventional AC-5 mix, 1993) and Haines Highway (AC-5 + 2.5% SBR, 1993):

The Haines Highway sections surveyed exhibited excellent low temperature cracking performance. No significant cracking was observed. In fact both, the polymer modified section and the conventional section showed no evidence of low temperature cracking after 2 years of age (1995 survey). However, during the following year (1996 survey) there were a total of 13 cracks in the modified section (3600 m) but only one crack in the conventional section (266 m). The average density of cracking for both sections was essentially equal to 4 cracks/km.


The sections surveyed had the same age but their low temperature cracking performance was significantly different. Both the conventional and the asphalt-rubber sections seemed to have reached a steady state in relation to the progression of low temperature cracking with time. The crack spacing in the conventional section ranged between 5 m and 12 with an average spacing of about 6 m (Figure 2.8) and average crack density of 177 cracks/km. On the other hand, only 3 cracks were observed in the asphalt-rubber section (Figure 2.9) with average spacing of 32 m and average crack density of 31 cracks/km. The use of asphalt-rubber in this case reduced the crack density by a factor of 6 approximately.


The low temperature cracking performance of the conventional and modified sections is best illustrated by comparing Figures 2.10 and 2.11 for the variation of cracking frequency with crack spacing. As shown, the range of crack spacing is between 16 m and 60 m for the PBA 3 section with and average spacing and density equal to 34 m and 30 cracks/km respectively (Figure 2.10). For the conventional section, the crack spacing varied from 6 m to 34 m with average spacing and density equal to 20 m and 49 cracks/km respectively. The improvement in low temperature cracking associated with using PBA 3 compared with the conventional AC-5 corresponds to an estimated reduction in crack density of 25 percent after one year of service.
Figure 2.8: Crack Spacing Distribution for Danby street, AC-2.5 Mix - Spring’96 Survey
Figure 2.9: Crack Spacing Distribution for Danby street, Asphalt-Rubber Mix - Spring '96 Survey
Figure 2.10: Crack Spacing Distribution for Parks Hwy., PBA3 Mix - Spring’96 Survey
Figure 2.11: Crack Spacing Distribution for Parks Hwy., AC-5 Mix - Spring’96 Survey
Comparison of Different Types of Polymer Modified Surfaces

a. 3% SBS Versus 6% SBS Binder Modification

In order to illustrate the influence of polymer content on the binder-mix performance two sites were chosen in each of the Northern Region (Parks Highway, AC-5 + 3% SBS (PBA 3), 1995 and Rewak Drive, AC-5 + 6% SBS (PBA 6), 1993) and the Central Region (Elmendorf AFB, AC-5 + 3% SBS, 1995 and Elmendorf AFB AC-5 + 6% SBS, 1991). In both the Northern Region and Central Region sites, the use of 6% SBS with AC-5 improved the low temperature cracking resistance in comparison with a 3% SBS modification. For example, the average crack spacing of the 5 year old pavement section at Elmendorf AFB with 6% SBS is essentially the same (23 m) as the section with 3% SBS after one year of service. Similarly, Rewak Drive in Fairbanks has 6% SBS and is 3 years old, but exhibits crack spacing of 51 m in comparison with 34 m spacing for the 3% SBS section of the Parks Highway after one year of service.

b. Arctic Grade Versus SBS Binder Modification

Field surveys indicate that Arctic Grade AC-2.5 seems to perform better than either AC-5 + 3% SBS or AC-5 + 6% SBS. In this case, comparing the Arctic Grade pavement at Ft. Wainwright with the AC-5 + 6% SBS section at Elmendorf show that after 5 years of service, the two sections had essentially an average crack spacing of 27 m. However, climatic conditions are less severe at Elmendorf since the minimum air temperature (50% reliability) is -28°C whereas the minimum air temperature at Ft. Wainwright is -46°C (Table 2.3).

c. Rubberized Pavements Versus Polymer Modified Pavements

There were only two rubberized pavement sections in this study. A-Street in Anchorage which had a PlusRide surface and Danby Street with an asphalt-rubber pavement. The asphalt-rubber exhibited the greatest resistance to low temperature cracking in comparison with other modified sections. At an age of 8 years, the asphalt-rubber section at Danby Street, under more severe climatic conditions, reached a steady state since no additional cracking was observed between Fall 1995 and Spring 1996. Although the PlusRide section in Anchorage had essentially similar crack spacing after 11 years of service, it did not reach steady state since the average crack spacing between 1995 and 1996 decreased from about 41 m to 33 m. It is expected that asphalt-rubber mixtures similar to those at Danby Street would perform better than the PlusRide used at A-Street under similar climatic conditions.

The asphalt-rubber section at Danby Street performed much better than Arctic Grade AC-5 used at Eielson AFB. In this case, the crack spacing at Eielson AFB reached about 6 m in comparison with 32 m at Danby.
Comparison of Conventional Mixes and Influence of Climatic Conditions

A number of conventional mixes (i.e. with no polymer modification) were covered in the survey. Specifically, mixes including AC-2.5, AC-5, and AC-20R were used in some of the pavements surveyed. The influence of climate on low temperature cracking of the conventional mixes is best illustrated by comparing C-Street (AC-5) and Danby Street (AC-2.5). The minimum crack spacing at Danby reached about 6 m whereas at C-Street the spacing was 29 m. The minimum air temperature in Fairbanks according to Table 2.3 is much lower than minimum air temperature in Anchorage. In this case, the 50% reliability temperature in Fairbanks is -46°C whereas in Anchorage it is equal to -29°C.

2.4 SUMMARY

This chapter describes field instrumentation and data collection for temperature measurement, and presents the results of low-temperature cracking surveys as part of the field studies performed.

A total of 12 sites were instrumented to record air and pavement temperatures. These sites were selected to represent the different climatic zones in Alaska. Both air, and pavement surface temperature at each site were measured. For pavement surface temperature, a thermistor 10 mm below the pavement surface was used. Analysis of air and pavement temperatures was conducted for the periods December 1995 and January 1996.

Low temperature cracking surveys were conducted in Fall 1995 and Spring 1996. A total of 16 pavement sections were selected to cover a number of climatic zones including maritime, transitional, continental, and arctic. Results of these surveys are presented to illustrate the variation of crack spacing and distribution of cracks for each section. Results show a general trend of improvement in the low temperature cracking performance when polymer modified mixes are used in comparison with conventional mixes.
CHAPTER THREE
LABORATORY STUDIES

3.1 INTRODUCTION

Low temperature cracking of asphalt concrete mixtures is a combined effect of stresses generated by low temperatures and traffic loads. The effect of low temperatures has been associated with the volumetric change that occurs in the pavement as the temperature drops. If a pavement is unrestrained, the contraction associated with a drop in temperature will release any tensile stresses. However, in reality asphalt concrete pavements are restrained, and as their temperature drops, tensile thermal stresses are generated. In the same time, traffic loads apply tensile stresses within the asphalt concrete layer which are added to the thermal stresses.

It is expected that cracking will occur if either the individual stress components (i.e. low temperature or traffic) or their combined magnitude exceed the tensile strength of the asphalt concrete mixture. In addition, thermal cracking may also occur as a result of freezing and shrinkage of the supporting pavement layers. The tensile strength of the asphalt concrete mixture is influenced by both the asphalt binder used and the overall properties of the mixture. In this part of the research, the resistance of the various mixtures to low temperature cracking was evaluated through the evaluation of binder and mixtures properties. The rheological properties of the binders are used to evaluate their resistance to low temperature as recommended by the Superpave binder grading system. The fracture temperatures of the mixtures were also measured using the thermal stress restrained specimen test (TSRST). The following represents a summary of the testing programs and the analysis of the data.

3.2 SUPERPAVE BINDER TESTING AND RESULTS

The objective of this testing program is to grade the binders using the Superpave performance grade specification. A total of nine original binders were provided and seven binders were extracted from the core samples that were delivered to the University of Nevada's Laboratory. The following represents a summary of the tested binders:

<table>
<thead>
<tr>
<th>Project</th>
<th>Location</th>
<th>AC Binder Grade</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>A/C Couplet</td>
<td>AC-5</td>
<td>Extracted</td>
</tr>
<tr>
<td>n</td>
<td>A/C Couplet</td>
<td>PlusRide mix (AC-5)</td>
<td>not tested</td>
</tr>
<tr>
<td>b</td>
<td>Elmendorf Roads</td>
<td>AC-5+3%SBS</td>
<td>Original</td>
</tr>
<tr>
<td>c</td>
<td>Elmendorf Roads</td>
<td>AC-5+6%SBS</td>
<td>Original</td>
</tr>
<tr>
<td>d</td>
<td>Elmendorf Runway</td>
<td>AC-5+6%SBS</td>
<td>Original</td>
</tr>
<tr>
<td>e1</td>
<td>Haines Highway</td>
<td>PBA3</td>
<td>Extracted</td>
</tr>
<tr>
<td>e2</td>
<td>Haines Highway</td>
<td>AC-5</td>
<td>Extracted</td>
</tr>
<tr>
<td>Project</td>
<td>Location</td>
<td>AC Binder</td>
<td>Source</td>
</tr>
<tr>
<td>---------</td>
<td>----------------------</td>
<td>---------------</td>
<td>---------</td>
</tr>
<tr>
<td>f</td>
<td>Denali Park Road</td>
<td>AC-20R</td>
<td>Original</td>
</tr>
<tr>
<td>g1</td>
<td>Danby Street</td>
<td>Asphalt Rubber</td>
<td>Original</td>
</tr>
<tr>
<td>g2</td>
<td>Danby Street</td>
<td>AC-2.5</td>
<td>Extracted</td>
</tr>
<tr>
<td>h1</td>
<td>Parks Highway</td>
<td>PBA3</td>
<td>Original</td>
</tr>
<tr>
<td>h2</td>
<td>Parks Highway</td>
<td>AC-5</td>
<td>Extracted</td>
</tr>
<tr>
<td>i</td>
<td>Badger Road</td>
<td>SHRP Level I</td>
<td>Original</td>
</tr>
<tr>
<td>j</td>
<td>Eielson AFB</td>
<td>Arctic Grade AC-5</td>
<td>Extracted</td>
</tr>
<tr>
<td>k</td>
<td>Ft. Wainwright</td>
<td>Arctic Grade AC-2.5</td>
<td>Extracted</td>
</tr>
<tr>
<td>l</td>
<td>Rewak Drive</td>
<td>AC-5+6%SBS</td>
<td>Original</td>
</tr>
<tr>
<td>m</td>
<td>Deadhorse Airport</td>
<td>Arctic Grade AC-2.5</td>
<td>Original</td>
</tr>
</tbody>
</table>

Prior to the start of the laboratory testing program, the Superpave binder specification system was thoroughly examined to identify the test parameters and the material properties needed for the grading process. The following represents a summary of the identified properties.

* Tenderness: original material with a minimum $G^*/\sin(d)$ value of 1.0 kPa measured at the maximum pavement temperature.

* Rutting: Rolling Thin Film Oven (RTFO) residue with a minimum $G^*/\sin(d)$ value of 2.2 kPa measured at the maximum pavement temperature.

* Fatigue cracking: Pressure Aging Vessel (PAV) residue with a maximum $G^*(\sin(d))$ value of 5,000 kPa measured at the intermediate temperature.

* Thermal cracking: PAV residue with a maximum stiffness value of 300,000 kPa and a minimum m-value of 0.30 measured at the minimum pavement temperature.

* Thermal cracking: PAV residue with a minimum direct tension failure strain of 1.0% at 1.0 mm/min at minimum pavement temperature.

In the above, $G^*$ is the complex shear modulus, $d$ is the phase angle between stress and strain, and the m-value is the slope of the log creep stiffness vs. log time plot at 60 sec. loading. It should be noted that the direct tension test failure strain criterion is not used if the creep stiffness measured by the BBR is below 300,000 kPa. If the creep stiffness is between 300,000 and 600,000 kPa the direct tension failure strain requirement can be used in lieu of the creep stiffness requirement, but the log slope of the creep curve must remain greater than 0.30.
3.2.1 The Superpave Grading Process

The process used in this research to test the original asphalt binders under the Superpave grading system is summarized in Figure 3.1 and as fully described in the interim AASHTO test method (1). Since the extracted binders have been in the field for sometime, it was unclear as to what level of aging they have been subjected to. The Superpave binder grading system uses the RTFO test to simulate short term aging (i.e. mixing and compaction) and the PAV to simulate long term aging (i.e. field exposure). In order to capture their full characterization, the extracted binders were evaluated under two conditions: Condition 1; the extracted binders were considered equivalent to RTFO-PAV aged binders, and Condition 2; the extracted binders were considered equivalent to RTFO aged binders. Under condition 1, the extracted binders were tested in the Dynamic Shear Rheometer (DSR) and the Bending Beam Rheometer (BBR) without being aged in the PAV, while under condition 2, the extracted binders were PAV aged and then tested in the DSR and BBR. This report presents the rheological properties and final grades obtained from both conditions.

As is the case with every grading system, the Superpave system can be used in two different ways: a) to check if a binder meets the requirements of a given grade, and b) to identify the appropriate grade for a given binder. Since the objective of this research was to identify the grades of the various binders, process b was followed.

Following process b requires more testing and more elaborate data analysis than process a. The initial objective of process b is to identify all the possible environments that a binder can fit. The final grade of the binder is then based on the widest environmental range. For example, if an AC-5 meets the requirements for PG52-10, PG52-16, and PG52-22, the final grade given to this binder would be PG52-22, since it represents the most extreme environmental conditions that this binder can withstand.

3.2.1.1 Flash Point and Viscosity Measurements

The Superpave specification limits call for a minimum flash point of 230°C and a maximum viscosity of 3 Pa-s at 135°C, using the Rotational Viscometer (RV). These tests were conducted on the original binders only. Checking the binders' data (Table 3.1) against these limits indicate that all binders satisfy the requirements of flash point. The binders from projects f and g have violated the viscosity requirement. Both of these binders contain rubber modifiers which contribute to higher viscosities. According to the specifications, this requirement may be waived if the binder can be adequately pumped and mixed at safe temperatures.

3.2.1.2 Evaluation of Binders Resistance to Tenderness and Rutting

The binder's resistance to tenderness and rutting is evaluated through measuring its rheological properties as it is taken from the tank, and aged through the RTFO test. The DSR
Figure 3.1: Flow Chart of the Laboratory Testing Program to Grade the Asphalt Binder
Table 3.1: Summary of Flash Point, Brookfield Viscosity, and RTFO Test Weight Loss.

<table>
<thead>
<tr>
<th>Project</th>
<th>Flash Point (°C)</th>
<th>Brookfield Viscosity (Pa·s)</th>
<th>RTFO Test Weight Loss (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>b</td>
<td>&gt;230</td>
<td>0.50</td>
<td>0.12</td>
</tr>
<tr>
<td>c</td>
<td>&gt;230</td>
<td>1.08</td>
<td>0.12</td>
</tr>
<tr>
<td>d</td>
<td>&gt;230</td>
<td>0.34</td>
<td>0.12</td>
</tr>
<tr>
<td>f</td>
<td>&gt;230</td>
<td>5.28</td>
<td>0.36</td>
</tr>
<tr>
<td>g1</td>
<td>&gt;230</td>
<td>3.91</td>
<td>0.45</td>
</tr>
<tr>
<td>h1</td>
<td>&gt;230</td>
<td>0.93</td>
<td>0.93</td>
</tr>
<tr>
<td>i</td>
<td>&gt;230</td>
<td>0.23</td>
<td>0.09</td>
</tr>
<tr>
<td>l</td>
<td>&gt;230</td>
<td>1.08</td>
<td>0.12</td>
</tr>
<tr>
<td>m</td>
<td>&gt;230</td>
<td>0.23</td>
<td>1.14</td>
</tr>
</tbody>
</table>
was used to evaluate the complex modulus ($G^*$) and phase lag ($\delta$) of all binders at 10 rad/sec frequency. At this stage, the class of the binder is unknown, therefore a temperature sweep must be conducted in order to identify the highest temperature at which the binder would reach the minimum specified value of $G^*/\sin(\delta)$.

Knowing that the $G^*/\sin(\delta)$ of an asphalt is inversely related to temperature, the testing proceeded at the lowest temperature (46°C) in the specification toward the highest temperature (82°C) (Figure 3.2). The relationships between $G^*/\sin(\delta)$ and temperature were determined for all binders. The first part of this evaluation consists of testing the virgin binders. The temperatures at which the binders reach the minimum value of 1.00 kPa for $G^*/\sin(\delta)$ were identified and are shown below. As mentioned earlier, this step was only completed for the original binders.

<table>
<thead>
<tr>
<th>Project</th>
<th>Temperature (°C) at $G^*/\sin(\delta)= 1.0$ kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>NA</td>
</tr>
<tr>
<td>b</td>
<td>64.5</td>
</tr>
<tr>
<td>c</td>
<td>75.5</td>
</tr>
<tr>
<td>d</td>
<td>54.8</td>
</tr>
<tr>
<td>e1</td>
<td>NA</td>
</tr>
<tr>
<td>e2</td>
<td>NA</td>
</tr>
<tr>
<td>f</td>
<td>63.8</td>
</tr>
<tr>
<td>g1</td>
<td>67.4</td>
</tr>
<tr>
<td>g2</td>
<td>NA</td>
</tr>
<tr>
<td>h1</td>
<td>63.8</td>
</tr>
<tr>
<td>h2</td>
<td>NA</td>
</tr>
<tr>
<td>i</td>
<td>56.8</td>
</tr>
<tr>
<td>j</td>
<td>NA</td>
</tr>
<tr>
<td>k</td>
<td>NA</td>
</tr>
<tr>
<td>l</td>
<td>75.5</td>
</tr>
<tr>
<td>m</td>
<td>52.0</td>
</tr>
</tbody>
</table>

An NA entry indicates that the binders were extracted and therefore, their virgin maximum temperature can't be evaluated.

The second part of this evaluation consists of evaluating the binders' properties after aging through the RTFO test. The RTFO test is used to simulate the aging during the mixing and paving operations. Therefore, the extracted binders were not aged through the RTFO. The percentage of weight loss through RTFO aging should not exceed 1 percent. The percentage weight losses for the original binders are summarized in Table 3.1. The data indicate that all binders except the binder from project m (Arctic Grade AC-2.5) would pass.
The rheological testing of the RTFO residues followed the same procedure as the one used for the virgin binders. This step was conducted for both the original and extracted binders. The only difference is that the extracted binders were not aged through the RTFO. The relationships between $G'/\sin(d)$ and temperature for all binders were measured. Using these relationships, the temperatures at which the binders reach the minimum value of 2.2 kPa for $G'/\sin(d)$ were identified. At this point the high temperature grades of the binders can also be identified according to the specification in Figure 3.2. The high temperature grade of the binder is the lowest between the two temperatures; the one evaluated on the virgin binder and the one evaluated on the RTFO aged binder. The following is a summary of the high temperature grades:

<table>
<thead>
<tr>
<th>Project</th>
<th>Temperature ($^\circ$C) at $G'/\sin(d)= 2.2$ kPa</th>
<th>High Temp Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>59.7</td>
<td>58</td>
</tr>
<tr>
<td>b</td>
<td>61.6</td>
<td>58</td>
</tr>
<tr>
<td>c</td>
<td>69.1</td>
<td>64</td>
</tr>
<tr>
<td>d</td>
<td>51.6</td>
<td>46</td>
</tr>
<tr>
<td>e1</td>
<td>66.2</td>
<td>64</td>
</tr>
<tr>
<td>e2</td>
<td>38.5</td>
<td>34</td>
</tr>
<tr>
<td>f</td>
<td>58.7</td>
<td>58</td>
</tr>
<tr>
<td>g1</td>
<td>60.3</td>
<td>58</td>
</tr>
<tr>
<td>g2</td>
<td>57.3</td>
<td>52</td>
</tr>
<tr>
<td>h1</td>
<td>61.8</td>
<td>58</td>
</tr>
<tr>
<td>h2</td>
<td>54.6</td>
<td>52</td>
</tr>
<tr>
<td>i</td>
<td>54.9</td>
<td>52</td>
</tr>
<tr>
<td>j</td>
<td>53.2</td>
<td>52</td>
</tr>
<tr>
<td>k</td>
<td>61.4</td>
<td>58</td>
</tr>
<tr>
<td>l</td>
<td>69.1</td>
<td>64</td>
</tr>
<tr>
<td>m</td>
<td>51.4</td>
<td>46</td>
</tr>
</tbody>
</table>

3.2.1.3 Evaluation of Binders Resistance to Fatigue

The Superpave binder grading system evaluates the binder's resistance to fatigue through its rheological properties after aging in the PAV. The specification requires the temperature at which $G' \sin(d)$ reaches a maximum value of 5,000 kPa. Therefore, for each binder there will be a minimum temperature selected above which the maximum value of $G' \sin(d)$ is not exceeded. The original binders were aged in the PAV while the extracted binders were tested following conditions 1 and 2 as discussed earlier. All binders were then tested at four temperatures. The relationships between temperature and $G' \sin(d)$ were measured. The minimum temperatures for the various binders are listed below:
### Performance Grade

<table>
<thead>
<tr>
<th>Performance Grade</th>
<th>PG 46</th>
<th>PG 52</th>
<th>PG 58</th>
<th>PG 64</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average 7-day Maximum Pavement Design Temperature, °C</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Minimum Pavement Design Temperature, °C</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flash Point Temp, °C</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Viscosity, ASTM D 4402, Maximum, 3 Pa*s (5000 dP), Test Temp, °C</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dynamic Shear, TP5, G’/sin δ, Minimum, 1000 kPa Test Temperature @ 10 rad/sec, °C</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rollin Thin Film Oven (T 260) or Thin Film Oven (T 179) Residue</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mass Loss, Maximum, %</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dynamic Shear, TP5, G’/sin δ, Minimum, 220 kPa Test Temp @ 10 rad/sec, °C</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pressure Aging Vessel Residue (PPV)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PAV Aging Temperature, °C</td>
<td>90</td>
<td>90</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Dynamic Shear, TP5, G’/sin δ, Maximum, 5000 kPa Test Temp @ 10 rad/sec, °C</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Physical Hardening</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Creep Stiffness, TP1, G’/sin δ, Minimum, 500 MPa Test Temp @ 60 sec, °C</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Direct Tension, TP3, Failure Stress, Minimum, 1.0% Test Temp @ 1.0 mm/min, °C</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Notes:

a. Pavement temperatures can be estimated from air temperatures using an algorithm contained in the Superpave™ software program or may be provided by the specifying agency, or by following the procedures as outlined in PPX.

b. This requirement may be waived at the discretion of the specifying agency if the supplier warrants that the asphalt binder can be adequately pumped and mixed at temperatures that meet all applicable safety standards.

c. For quality control of unmodified asphalt cement production, measurement of the viscosity of the original asphalt cement may be substituted for dynamic shear measurements of G’/sin δ at test temperatures where the asphalt is a Newtonian fluid. Any suitable standard means of viscosity measurement may be used, including capillary or rotational viscometer (AASHTO T 201 or T 202).

d. The PAV aging temperature is based on simulated climatic conditions and is one of three temperatures: 90°C, 100°C or 110°C. The PAV aging temperature is 100°C for PG 64- and above, except in desert climates, where it is 110°C.

e. Physical Hardening - TP 1 is performed on a set of asphalt beams according to Section 13.1 of TP 1, except the conditioning time is extended to 24 hrs ± 10 minutes at 10°C above the minimum performance temperature. The 24-hour stiffness and m-value are reported for information purposes only.

f. If the creep stiffness is below 300 MPa, the direct tension test is not required. If the creep stiffness between 300 and 600 MPa the direct tension failure strain requirement can be used in lieu of the creep stiffness requirement. The m-value requirement must be satisfied in both cases.

Figure 3.2: SHRP’s Binder Specification Chart
<table>
<thead>
<tr>
<th>Performance Grade</th>
<th>PG 70</th>
<th>PG 76</th>
<th>PG 82</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Temperature, °C</td>
<td>-10</td>
<td>-16</td>
<td>-22</td>
</tr>
<tr>
<td>Minimum Pavement Temperature, °C</td>
<td>&gt;-10</td>
<td>&gt;-16</td>
<td>&gt;-22</td>
</tr>
</tbody>
</table>

**Notes:**

a. Pavement temperatures can be estimated from air temperatures using an algorithm contained in the Superpave software program or may be provided by the specifying agency, or by following the procedures as outlined in PPX.

b. This requirement may be waived at the discretion of the specifying agency if the supplier warrants that the asphalt binder can be adequately pumped and mixed at temperatures that meet all applicable safety standards.

c. For quality control of unmodified asphalt cement production, measurement of the viscosity of the original asphalt cement may be substituted for dynamic shear measurements of G'/sin δ at test temperatures where the asphalt is a Newtonian fluid. Any suitable standard means of viscosity measurement may be used, including capillary or rotational viscometer (AASHTO T 201 or T 202).

d. The PAV aging temperature is based on simulated climatic conditions and is one of three temperatures 90°C, 100°C or 110°C. The PAV aging temperature is 100°C for PG 64- and above, except in desert climates, where it is 110°C.

e. Physical Hardening - TP 1 is performed on a set of asphalt beams according to Section 13.1 of TP 1, except the conditioning time is extended to 24 hrs ± 10 minutes at 10°C above the minimum performance temperature. The 24-hour stiffness and m-value are reported for information purposes only.

f. If the creep stiffness is below 500 MPa, the direct tension test is not required. If the creep stiffness between 300 and 600 MPa the direct tension failure strain requirement can be used in lieu of the creep stiffness requirement. The m-value requirement must be satisfied in both cases. AASHTO Performance Graded Binder Specification (MP1)

Figure 3.2: SHRP’s Binder Specification Chart (continued)
3.2.1.4 Evaluation of Binders Resistance to Low Temperature Cracking

The Superpave binder grading system uses the Bending Beam Rheometer (BBR) to evaluate the binder's resistance to low temperature cracking by measuring its low temperature stiffness ($S$) and the rate of change of its low temperature stiffness ($m$). The BBR test is conducted on the PAV residues of original binders and on the extracted binders (conditions 1 and 2) under two temperatures, namely -30 and -10°C. The two BBR points are used to draw the relationships between the stiffness ($S(t)$) and the slope ($m$) as a function of temperature.

Using the bending beam results, the low temperature grades of the binders are identified based on the maximum stiffness value of 300,000 kPa and a minimum slope value of 0.30.
<table>
<thead>
<tr>
<th>Project</th>
<th>Location</th>
<th>AC Binder Grade</th>
<th>PG GRADE</th>
</tr>
</thead>
<tbody>
<tr>
<td>a-unaged</td>
<td>A/C Couplet</td>
<td>AC-5</td>
<td>PG58-22</td>
</tr>
<tr>
<td>a-PAV aged</td>
<td>A/C Couplet</td>
<td>AC-5</td>
<td>PG58-10</td>
</tr>
<tr>
<td>b</td>
<td>Elmendorf Roads</td>
<td>AC-5+3%SBS</td>
<td>PG58-22</td>
</tr>
<tr>
<td>c</td>
<td>Elmendorf Roads</td>
<td>AC-5+6%SBS</td>
<td>PG64-22</td>
</tr>
<tr>
<td>d</td>
<td>Elmendorf Runway</td>
<td>AC-5+6%SBS</td>
<td>PG64-28*</td>
</tr>
</tbody>
</table>

It should be noted that the low temperature grade is 10°C colder than the temperature at which $S(t)$ is less than 300,000 kPa and $m$ is higher than 0.30.

### 3.2.1.5 Selection of Binders Grades

The final grades of all sixteen binders based on the Superpave binder grading system are as follows:

<table>
<thead>
<tr>
<th>Project</th>
<th>Location</th>
<th>AC Binder Grade</th>
<th>PG GRADE</th>
</tr>
</thead>
<tbody>
<tr>
<td>a-unaged</td>
<td>A/C Couplet</td>
<td>AC-5</td>
<td>PG58-22</td>
</tr>
<tr>
<td>a-PAV aged</td>
<td>A/C Couplet</td>
<td>AC-5</td>
<td>PG58-10</td>
</tr>
<tr>
<td>b</td>
<td>Elmendorf Roads</td>
<td>AC-5+3%SBS</td>
<td>PG58-22</td>
</tr>
<tr>
<td>c</td>
<td>Elmendorf Roads</td>
<td>AC-5+6%SBS</td>
<td>PG64-22</td>
</tr>
<tr>
<td>d</td>
<td>Elmendorf Runway</td>
<td>AC-5+6%SBS</td>
<td>PG64-28*</td>
</tr>
</tbody>
</table>
The properties of all binders are summarized in Appendix D. As mentioned earlier, the final grade of the binder is the one representing the widest temperature range. The binders that are labeled with "*" next to their PG grades indicate that their grades have been extrapolated beyond the actual Superpave grading chart. This extrapolation was necessary because these binders were found to be too soft at the high temperature and too brittle at the low temperature. For example, the binder from project "d" had a high temperature grade of 46, the only corresponding low temperature grades for the 46 high temperature grade present in the Superpave chart are -34, -40, and -46 (see Figure 3.2). This binder had a low temperature of -20.5, therefore the Superpave chart was extrapolated which gave this binder a low grade of -28.

The rheological properties of the extracted binders indicated that PAV aging of the extracted binders changes their low temperature grade by at least one level and as much as three levels in some cases. For example, the binder in project e1 changed from PG64-34 (unaged) to PG64-28 (PAV aged) while the binder in project h2 changed from PG52-28 (unaged) to PG52-16 (PAV aged). It was also observed that the impact of PAV aging is more significant on the unmodified binders such as projects a, e2, h2 and g2 than on the modified binders such as projects e1, j, and k. In the case of unmodified binders the PAV aging changed the low temperature grade by two or three levels while in the case of the modified binders, the PAV aging changed the low temperature grade by only one level.
3.3 TSRST TESTING AND RESULTS

The objective of this testing program is to evaluate the fracture temperature and low temperature strength of the mixtures used on the selected field sections. As mentioned earlier the TSRST is used to measure these properties. A total of 14 sections were sampled and tested.

<table>
<thead>
<tr>
<th>Project</th>
<th>Location</th>
<th>AC Binder Grade</th>
<th>Year of Construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>A/C Couplet</td>
<td>AC-5</td>
<td>1986</td>
</tr>
<tr>
<td>b</td>
<td>Elmendorf Roads</td>
<td>AC-5+3%SBS</td>
<td>1991</td>
</tr>
<tr>
<td>c</td>
<td>Elmendorf Roads</td>
<td>AC-5+6%SBS</td>
<td>1995</td>
</tr>
<tr>
<td>e1</td>
<td>Haines Highway</td>
<td>PBA3</td>
<td>1993</td>
</tr>
<tr>
<td>e2</td>
<td>Haines Highway</td>
<td>AC-5</td>
<td>1993</td>
</tr>
<tr>
<td>f</td>
<td>Denali Park Road</td>
<td>AC-20R</td>
<td>1991</td>
</tr>
<tr>
<td>g1</td>
<td>Danby Street</td>
<td>Asphalt Rubber</td>
<td>1988</td>
</tr>
<tr>
<td>h1</td>
<td>Parks Highway</td>
<td>PBA3</td>
<td>1995</td>
</tr>
<tr>
<td>h2</td>
<td>Parks Highway</td>
<td>AC-5</td>
<td>1995</td>
</tr>
<tr>
<td>i</td>
<td>Badger Road</td>
<td>SHRP Level I</td>
<td>1995</td>
</tr>
<tr>
<td>j</td>
<td>Eielson AFB</td>
<td>Arctic Grade AC-5</td>
<td>1986</td>
</tr>
<tr>
<td>k</td>
<td>Ft. Wainwright</td>
<td>Arctic Grade AC-2.5</td>
<td>1990</td>
</tr>
<tr>
<td>l</td>
<td>Rewak Drive</td>
<td>AC-5+6%SBS</td>
<td>1993</td>
</tr>
<tr>
<td>n</td>
<td>A/C Couplet</td>
<td>PlusRide</td>
<td>1985</td>
</tr>
</tbody>
</table>

3.3.1 Thermal Stress Restrained Specimen Test

This test is based on a low temperature testing system originally developed by Arand (1987). The basic concept behind this test is that the asphalt concrete specimen is cooled down at a constant rate while being restrained from contracting. An electro-hydraulic system is used to maintain the specimen at a constant height. Figure 3.3 shows the testing set-up for the TSRST.

As the asphalt concrete is cooled down and forced to maintain a constant height, tensile stresses will be generated throughout the length of the sample. It is expected that the thermally induced stresses will increase as the temperature of the specimen decreases until the specimen fractures. At the breaking point, the stress reaches its maximum value, which is referred to as the fracture strength, with a corresponding fracture temperature, as shown in Figure 3.3. The slope of the thermally induced stress curve, dS/dT, increases until it reaches a maximum value. At colder temperatures, dS/dT becomes constant and the stress-temperature curve is linear. The transition temperature divides the curve into two parts, relaxation and non-relaxation. As the temperature approaches the transition zone, the asphalt cement
Figure 3.3: Thermal Stress Restrained Specimen Test (Ref. 3)
becomes stiffer and the thermally induced stresses are not relaxed below this temperature. The slope tends to decrease again when the specimen is close to fracture. This may be due to the development of micro cracks in the specimen.

3.3.2 Sample Preparation

Asphalt concrete slabs measuring 600x600 mm were cut from each of the sites. This was performed during September and October of 1995. The smaller slabs were transported to the laboratory for sample preparation and testing. The TSRST specimens are 50x50x250 mm beams which were cut from the slabs obtained from each site. Once the specimen is cut, it must be glued to the end platens with perfect alignment using epoxy compound. This is accomplished through a permanent laboratory set-up exclusively developed for this test. Once the specimen is glued, it is left in the alignment stand at room temperature until the epoxy is cured, which is normally about 24 hours.

3.3.3 Test Procedure

The SHRP A003A project has developed a complete laboratory procedure for the TSRST, while the following presents a concise summary of the test (Monismith and Hicks 1992). After the epoxy is cured, the specimen with end platens is placed in an environmental room controlled at 5°C for twelve hours as a preconditioning period. At the end of the preconditioning period, the specimen is moved into the testing machine and two LVDTs are connected to both sides (180 degrees) in order to control the change of length in the specimen. It should be noted here that the main objective of this test is to restrain the specimen from contraction as it cools down. Therefore, the LVDTs used on the specimen should have high resolution in order to tightly control the change in specimen length. In addition, the rods used to connect the LVDTs to platens should have a very small coefficient of thermal expansion.

The environmental chamber on the testing machine can be cooled down by liquid nitrogen or a mechanical chilling device with a variable rate controlling unit. A resistance temperature device (RTD) is used to control the temperature inside the environmental chamber. Once the sample is in place, the LVDT's are zeroed, and the environmental chamber is kept at 5°C for one hour, after which the temperature is dropped at a constant rate of 9°C/hour. During this period, the load necessary to maintain the specimen at a constant length is measured. The fracture temperature is measured as the temperature at which the beam cracks.

3.3.4 TSRST Results

Table 3.2 summarizes the fracture strength and fracture temperature for all sections. Three replicates were tested for each mix. Detailed data for every test, including the variation of thermal stress with specimen temperature are shown in Appendix E. A comparison of the
Table 3.2: Summary of the TSRST Fracture Strength and Temperatures

<table>
<thead>
<tr>
<th>Project</th>
<th>Fracture Strength (kPa)</th>
<th>Measured Frac. Temp.(°C)</th>
<th>Rank (1)</th>
<th>Estimated Original Frac. Temp.(°C)</th>
<th>Rank (2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>2811</td>
<td>-25.7</td>
<td>11</td>
<td>-29.9</td>
<td>9</td>
</tr>
<tr>
<td>b</td>
<td>1931</td>
<td>-33.2</td>
<td>3</td>
<td>-33.7</td>
<td>4</td>
</tr>
<tr>
<td>c</td>
<td>3033</td>
<td>-26.4</td>
<td>9</td>
<td>-28.1</td>
<td>11</td>
</tr>
<tr>
<td>e1</td>
<td>3035</td>
<td>-31.6</td>
<td>5</td>
<td>-32.2</td>
<td>6</td>
</tr>
<tr>
<td>e2</td>
<td>3393</td>
<td>-27.5</td>
<td>8</td>
<td>-28.2</td>
<td>10</td>
</tr>
<tr>
<td>f</td>
<td>922</td>
<td>-23.3</td>
<td>14</td>
<td>-25.1</td>
<td>12</td>
</tr>
<tr>
<td>g1</td>
<td>3276</td>
<td>-33.0</td>
<td>4</td>
<td>-36.0</td>
<td>2</td>
</tr>
<tr>
<td>h1</td>
<td>3441</td>
<td>-33.5</td>
<td>2</td>
<td>-33.0</td>
<td>5</td>
</tr>
<tr>
<td>h2</td>
<td>2479</td>
<td>-23.9</td>
<td>13</td>
<td>-24.1</td>
<td>13</td>
</tr>
<tr>
<td>i</td>
<td>1745</td>
<td>-25.3</td>
<td>12</td>
<td>-25.5</td>
<td>12</td>
</tr>
<tr>
<td>j</td>
<td>3382</td>
<td>-31.6</td>
<td>6</td>
<td>-35.6</td>
<td>3</td>
</tr>
<tr>
<td>k</td>
<td>3268</td>
<td>-34.6</td>
<td>1</td>
<td>-36.6</td>
<td>1</td>
</tr>
<tr>
<td>l</td>
<td>1446</td>
<td>-30.9</td>
<td>7</td>
<td>-31.5</td>
<td>7</td>
</tr>
<tr>
<td>n</td>
<td>1979</td>
<td>-26.4</td>
<td>10</td>
<td>-31.1</td>
<td>8</td>
</tr>
</tbody>
</table>

Note:
(1): Ranking according to the measured Fracture Temp. i.e. does not account for specimen age
(2): Ranking after accounting for specimen age, according to Eq. 5.4
average fracture temperature for all mixes used in this study is illustrated in Figures 3.4 and 3.5. Based on the results of the TSRST, the following observations can be made:

- All polymer modified mixtures, except the AC-20R, Denali (f) and AC-5+6%SBS, Elmendorf (l), exhibited lower fracture temperatures (-31°C to -35°C) in comparison with conventional unmodified mixtures (-24°C to -28°C).

- The AC-2.5 Arctic Grade mixture, Ft. Wainwright (k) had the lowest fracture temperature of -35°C. In addition, it maintained a reasonably high fracture strength of 3268 kPa. The AC-5 Arctic Grade mixture, Eielson (j) also shows good low temperature cracking performance. Fracture temperature is a good indicator of thermal crack initiation whereas fracture strength governs the spacing of the cracking pattern. Higher fracture strength is generally expected to result in greater crack spacing and eventually in lower crack density.

- For rubberized pavement sections, the wet process asphalt-rubber mix, Danby St. (g1) fracture temperature of -33°C is superior to the dry process PlusRide mix, A/C Couplet (n) fracture temperature of -26°C.

- Although the conventional AC-5 mix (h2) and the SHRP mix (i), both placed in Fairbanks in 1995, have essentially the same fracture temperatures, their fracture strengths are different. In this case, the fracture strength of the AC-5 mix (h2) is 2479 kPa, in comparison with 1745 kPa for the SHRP mix. This would probably result in better low temperature performance of the conventional AC-5 mix since higher fracture strength results in larger thermal crack spacing and eventually lower crack density.

### 3.4 ASSESSMENT OF PAVEMENT CRACKING FROM TSRST AND BINDER TESTS

The objective of this analysis is to evaluate the most appropriate approach in assessing the potential of low temperature cracking of flexible pavements in Alaska. This project has evaluated binder and mixture properties that provide indication of the mixtures' resistance to low temperature cracking. In addition the field sections were surveyed and actual low temperature cracks have been measured.

The Superpave system recommends low temperature grade for each of the project locations based on 50 and 98% reliabilities. Therefore, there should not be any low temperature cracking on the sections where the Superpave recommendations have been met. Also the TSRST data provides the fracture temperature above which the pavement should not crack. The following represents a summary of the various recommendations and observations.
Figure 3.4: Comparison of Average Fracture Temperatures for Test Specimens
Figure 3.5: Comparison of Average Fracture Temperatures for Test Specimens

- Asphalt Rubber Mix
- AC5+6%SBS
- AC5 Mix
- AC5+2%SBS+2%SBR
- SHRPMix
- Arctic Grade AC5 Mix
- Arctic Grade AC2.5
<table>
<thead>
<tr>
<th>Project Const</th>
<th>Measured Binder Grade</th>
<th>Measured TSRST Temp.</th>
<th>Estimated Original Temp.</th>
<th>Estimated Superpave 50%</th>
<th>Estimated Superpave 98%</th>
<th>Crack Spacing, m Avg STD</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>86</td>
<td>-22</td>
<td>-26</td>
<td>-30</td>
<td>-29</td>
<td>-37</td>
</tr>
<tr>
<td>n</td>
<td>85</td>
<td>NA</td>
<td>-26</td>
<td>-31</td>
<td>-29</td>
<td>-37</td>
</tr>
<tr>
<td>b</td>
<td>91</td>
<td>-22</td>
<td>-33</td>
<td>-33</td>
<td>-28</td>
<td>-37</td>
</tr>
<tr>
<td>c</td>
<td>95</td>
<td>-22</td>
<td>-26</td>
<td>-28</td>
<td>-28</td>
<td>-37</td>
</tr>
<tr>
<td>d</td>
<td>NA</td>
<td>-28</td>
<td>NA</td>
<td>NA</td>
<td>-28</td>
<td>-37</td>
</tr>
<tr>
<td>e1</td>
<td>93</td>
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<td>-54</td>
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<td>-22</td>
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<td>NA</td>
<td>-44</td>
<td>-49</td>
</tr>
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</table>

For the extracted binders, it was decided to use the unaged binder for all projects since there is no clear indication as to what the PAV aging represents in the field at this point and most projects are older than three years. Based on the 1996 field conditions surveys, the best performance was obtained on sections e1, e2, and i. Section e2 showed no transverse cracking while sections e1 and i had transverse cracking spacing of 155 and 119 m, respectively. Sections e1 and e2 are the only sections that had a binder meeting the Superpave 50% and 98% reliability recommendations. The binder used on section i violated the Superpave recommendations very significantly, however, this project is only one year old (built in 1995). The binders used on all other sections have violated the Superpave 50 and 98% reliability recommendations, and therefore they showed extensive damage due to low temperature cracking.

The worst field performance was measured on sections g2 and j where crack spacing was as low as 6 m. Section g2 had an unmodified AC-2.5 binder with a low temperature grade of -28 while section j had an Arctic Grade AC-5 with a low temperature grade of -34. The severe failure of section j is surprising since neither the Superpave binder grade nor the TSRST data indicated any major problems as compared to the other sections that were evaluated. Such behavior may be a result of reflective cracking when the section is an overlay. For example, section I has a binder with only -22 low temperature grade and still performed significantly better than all other sections (i.e. crack spacing of 51 m).
Figures 3.6 and 3.7 show the relationships between the difference between the actual binder grade and the Superpave recommended grade and the crack spacing on the pavement section. For example, project a has a binder grade of -22 while the Superpave recommended grades are -29 and -37 for 50 and 98% reliabilities, respectively. Therefore, the grade differences for project a are -7 and -15 and the crack spacing is 34m. On the other hand, project e1 has a binder grade of -34 while the Superpave recommended grades are -19 and -29 for 50 and 98% reliabilities, respectively. Therefore, the grade differences for project e1 are +15 and +5 and the crack spacing is 155m. In summary, a negative grade difference indicates that the binder used on the project has a warmer grade than the Superpave recommended binder while a positive grade difference indicates that the binder used on the project has a colder grade than the Superpave recommended binder.

The data in Figures 3.6 and 3.7 show that crack spacing increases significantly when the grade difference is positive (i.e. the binder has a colder grade than the Superpave recommended binder) as is the case for projects e1 and e2. In the cases where the grade difference is negative (i.e. the binder has a warmer grade than the Superpave recommended binder), there is no clear trend between the magnitude of the grade difference and crack spacing. This indicates that once the binder used on a project violates the Superpave recommended grade, there is no clear indication as to how much low temperature cracking the pavement would experience. The data indicate that the crack spacing ranged from 6 to 51 m for a grade difference between -5 and -25°C for 50% reliability and between -15 and -32°C for 98% reliability.

3.5 TESTS AND RESULTS FOR SUPERPAVE TCMODEL

Tests using the SHRP Indirect Tensile Device (IDT) and thermal cracking model (TCMODEL) in Superpave allow for the prediction of low temperature cracking performance for a given mixture, pavement structure, and climatic factors. In this case, the IDT is used to perform a number of tests, specifically, IDT creep, and IDT strength at 12.5 mm/min.

3.5.1 Indirect Tensile Device (IDT)

The IDT was used to perform all non-load related performance tests and the test results were then used in models to predict low temperature cracking. The IDT has four components:

1. Axial loading device;
2. Specimen deformation measurement system;
3. Environmental chamber, and
4. Control and data acquisition system.

The axial loading device is capable of applying fixed loads, loads at constant rates, and loads due to a constant rate of ram or cross head displacement. The rate of ram or cross head
Figure 3.6: Relationship between Grade Difference and Crack Spacing for 50% Superpave Reliability
Figure 3.7: Relationship between Grade Difference and Crack Spacing for 98 % Superpave Reliability
displacement is maintained at 12.5 mm/minute in the non-load related tensile strength tests. In the creep tests, a constant load is applied to specimens for 100 seconds, while the creep deformation caused by the load is measured. In both tests the load is applied across the diametral plane of a specimen, as shown in Figure 3.8. By applying a compressive load to the specimen diameter, a near uniform state of tensile stress is developed along most of the diametral axis of the specimen, and thus the term indirect tensile test. High compressive stresses exist at the surface loading points, but these stresses do not detract from the measurement of tensile strength or creep compliance. A load cell is in place between the upper loading platen and the ram, which measures load and is used as a means of feedback for control of a constant load in the creep tests.

Specimen deformation is measured by means of four linear variable differential transducers (LVDTs). A total of four LVDTs are mounted to each test specimen. Two LVDTs are affixed to gauge points on each side of a specimen, one is mounted horizontally and the other is mounted vertically (90° apart). The LVDTs are actually mounted to eight brass gauge points (two per LVDT) which are epoxied to the test specimen as shown in Figure 3.9. To insure that the gauge points are located in the proper position on the sample, an alignment device is used as shown in Figure 3.10. The test specimen is oriented in the testing device as shown in Figure 3.11. Note that the loading rams are perpendicular to the cut faces of the specimen and the horizontal LVDTs. The proper alignment of the specimen in the testing device is critical.

The environmental chamber serves two purposes, it maintains the temperature of the test specimen during testing and it is large enough to store and condition up to seven samples for subsequent testing. The environmental chamber is capable of maintaining temperatures from -30 °C to 30 °C with resolution of ± 0.2 °C.

The control and data acquisition system is used to automatically control and record the required parameters. The system induces and records load, and also records specimen deformation in two directions on both sides of the specimen (4 LVDTs). When a test is completed a data file is generated, which is automatically formatted for the Superpave analysis software.

This improved measurement and analysis system used to accurately determine properties of asphalt mixtures from the indirect tensile mode addresses the following: 1) effects of localized stress concentrations caused by the metal loading strips must be considered when choosing appropriate gage location and length; 2) because the specimen is of finite thickness, 3-D effects must be accounted for in the analysis of measurements obtained from the test; 3) measurements which allow for accurate determination of Poisson's ratio are needed; 4) measurements obtained in zones within the specimen where the stress states are relatively uniform will reduce the effects of nonlinear behavior on the modulus or stiffness predicted; and 5) the measurement and loading system used should reduce the effects of specimen rotation without inducing restraint or unknown stresses in the specimen.
Figure 3.8: Diagram of Tensile Stress along Diametral Plane

Figure 3.9: LVDTs mounted to sides of IDT Specimen
Figure 3.10: Alignment Device for mounting LVDT Gage Points

Figure 3.11: Test Specimen Orientation in IDT Loading Device
Near the center of the face of the specimen, horizontal and vertical stresses are virtually uniform as depicted in Figure 3.12. Measurements taken in this area nearly eliminate the effects of local stress concentrations around the loading strips because the stresses are not affected by end conditions. The LVDTs span the center of the specimen in both horizontal and vertical directions. Figure 3.13 shows that non-uniform bulging occurs along the Z-axis. Additionally, note that non-uniform bulging on the x and y axes affect the measurements obtained from surface mounted sensors because of LVDT rotation. Correcting the measurements obtained with these sensors is necessary to account for 3-D effects. Correction factors were computed by SHRP researchers for a range of specimen thicknesses and Poisson's ratio using 3-D finite element analysis and geometry of the deformed specimen.

3.5.2 Test Specimens

AASHTO TP9-94, "Test Method for Determining the Creep Compliance and Strength of Hot Mix Asphalt (HMA) Using Indirect Tensile Test Device," provides the procedures for determining the tensile creep compliance at different loading times, tensile strength and Poisson's ratio of hot mix asphalt using the indirect loading technique. This provisional test method is specified for use within the Superpave Analysis System. All of the testing performed in this research was in accordance with AASHTO TP9-94.

The procedure applies to test specimens having a maximum aggregate size less than or equal to 25 mm, with a height of 25 mm to 75 mm and a diameter of 100 ± 6 mm. It also applies to specimens with a maximum aggregate size greater than 25 mm but less than 38 mm, with a height of 38 mm to 50 mm and a diameter of 150 ± 9 mm. Superpave, on the other hand, requires specimens with a maximum size aggregate less than or equal to 25 mm to be 50 ± 1 mm in height and have a diameter of 150 ± 9 mm. According to Superpave, all IDT specimens should be compacted to 7 ± 0.5% air voids.

Specimens must be cut with a diamond saw blade on both sides to conform to the parallel sides desired for testing, regardless of whether the specimens are laboratory prepared or field cores. The cuts are made perpendicular to the longitudinal axis of the specimen to provide smooth and parallel faces of the specimen to within ± 1.0 mm of each other. The specimens tested in this project were field cores with heights in some cases slightly less than the desired 50 mm. The diameter of the specimens was 150 mm. The level of air voids was not necessarily 7 ± 0.5%.

3.5.3 Indirect Tensile Creep Test

In this test, stresses are applied as shown in Figure 3.14. A fixed static diametral load is applied to the specimen for 100 seconds, which causes horizontal strain. The load applied is a function of the horizontal strain induced by the load. The horizontal strain must be greater
Figure 3.12: Elastic Stress Distribution in Indirect Tension Specimen (Ref. 5)
Figure 3.13: Surface Deformation of Indirect Tension Specimen
Figure 3.14: Load Controlled Indirect Tension Creep Tests

Figure 3.15: Load and Deformation in Indirect Tensile Strength Test
than 30 microstrains and less than 500 microstrains in order to avoid non-linear behavior while being high enough to minimize noise-level responses.

The LVDTs mounted on the center of each side of the specimen measure the material response to the applied load and provide information necessary to characterize linear viscoelastic properties at low temperatures. The properties determined from the test and used in the Superpave Analysis Software include, Poisson's ratio, the creep compliance curve, and the master relaxation modulus curve. Each test takes approximately two hours to perform, even though the test time is only 100 seconds. The creep tests are performed at -20 °C, -10 °C, and 0 °C.

3.5.4 Indirect Tensile Strength Test

In this test, the stresses are applied as shown in Figure 3.15. The specimen is deformed at a constant rate of 12.5 mm/min until failure occurs. The testing configuration is identical to that of IDT creep tests. The maximum load along with specimen geometry is used to determine the tensile strength. The tensile strength tests are typically performed immediately after the creep test at -10 °C in order to expedite the testing program.

3.5.5 IDT Creep Test Results

Two properties are determined from the creep tests, creep compliance and Poisson's ratio. Creep compliance D(t) is defined as the amount of strain with time in a material loaded at a particular level of stress. It is equal to the ratio of the strain developed at a given time to the applied stress. The creep compliance test results over a range of test temperatures are used to develop master creep compliance curves, which in turn are used to develop master relaxation modulus curves, both of which are used in predicting thermal cracking. Because creep tests are easier to perform than relaxation tests, the creep test data are manipulated to provide relaxation data. The relaxation modulus is used to make low temperature stress predictions, and the slope of the creep modulus curve is used in predicting crack propagation.

Poisson's ratio is defined as the ratio of the lateral strain to axial strain when a stress is applied to the axis in which the axial strain is measured. The ratio is a function of several variables in asphalt concrete, and it increases with temperature or decreasing modulus of the asphalt concrete. A realistic range of Poisson's ratio is between 0.05 and 0.5.

A plot of load and a plot of deformation with time is included in Appendix F for each test. It should be noted that when the magnitude of a pair of LVDTs (horizontal or vertical) was dissimilar, the test was repeated up to three times in an attempt to obtain the most meaningful data possible.
3.5.6 IDT Strength Test Results

The low temperature strength test results are used to determine the relationship between failure strength and temperature. The tensile strength is used in the prediction of crack depth (propagation) and crack intensity. Table 3.3 summarizes the tensile strength testing data.

3.5.7 Superpave Modeling

The application of IDT test results to predict the progression of low temperature cracking with time is beyond the scope of this chapter. Complete analysis and comparisons of TCMODEL predictions with field observation are presented in Chapter 5.

3.6 SUMMARY

A number of tests were conducted on the binder in addition to field specimens obtained from the field sites used on this project. Superpave binder tests were performed on the original binders but in many cases on the extracted binders since samples for the original binders on some projects were not available. Mix tests included both the thermal stress restrained specimen test (TSRST) and the indirect tensile strength and creep tests using the SHRP IDT. Results of the tests were used in attempt to evaluate the most appropriate approach in assessing the potential of low temperature cracking of flexible pavements in Alaska.

The potential use of the grade difference between the actual binder grade and the Superpave recommended grade as an indicator for low temperature crack spacing was illustrated. Results indicate that crack spacing increases significantly when the grade difference is positive (i.e. binder has colder grade than the Superpave recommended grade). However, when the grade difference is negative (i.e. binder has warmer grade than the Superpave recommended grade) there is not clear indication as to how much low temperature cracking the pavement would experience. The data indicate that the crack spacing ranged from 6 to 51 m for a grade difference between -5°C and -25°C for 50% reliability and between -15°C and -32°C for 98% reliability.

The TSRST results illustrate the improved resistance of polymer modified mixtures to low temperature cracking. In general, the fracture temperature varied between -31°C and -35°C for the polymer modified mixtures in comparison with -24°C and -28°C for the conventional mixtures.

There is some evidence that the PG system for binder specification may “mask” the beneficial effect that the polymer modifier may have on the low temperature grade. This is evidenced by comparing the PG for conventional and polymer modified mixes and TSRST results.
### Table 3.3: Indirect Tensile Strengths at -10°C

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<td></td>
<td></td>
<td></td>
<td>kPa</td>
<td>psi</td>
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<td>AC-5+3%SBS</td>
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<td>540</td>
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<td>PBA6</td>
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REFERENCES

1. AASHTO Provisional Standards, Standard Practice for Grading or Verifying the Performance Grade of an Asphalt Binder, AASHTO Designation: PP6-93, September, 1993.


CHAPTER FOUR

PAVEMENT TEMPERATURE MODELING AND MAPPING

4.1 INTRODUCTION

The selection of asphalt binder or modified asphalt binder using performance-based binder specifications (AASHTO MP1) depends primarily on the expected high and low pavement temperatures. For low temperature cracking considerations, the binder should satisfy minimum pavement temperature requirements selected for design. In this case, knowledge of minimum pavement temperature during the design period or interval is essential for proper binder selection.

In this chapter, field temperature data for the sites described in Chapter 2 are used to establish correlations between pavement temperature and air temperature for Alaskan conditions. These correlations are compared with models proposed by Superpave and the Asphalt Institute. In addition, contour maps for air and pavement temperatures are developed using the Superpave weather database and the proposed pavement-air temperature correlations.

4.2 BACKGROUND

As a result of the SHRP and the Superpave performance-based specifications, more research has concentrated recently on establishing simple and accurate correlations between air and pavement temperature. Solaimanian and Kennedy (1993) developed a simple regression equation for air and pavement temperature based on analysis using the energy balance at the pavement surface and the resulting temperature equilibrium:

\[ T_d = T_s (1 - 2.48 \times 10^{-3} d + 1.1 \times 10^{-5} d^2 - 2.44 \times 10^{-8} d^3) \]  \hspace{1cm} (4.1)

where

- \( T_d \) = pavement temperature at a given depth, \( d \) (\( ^\circ \text{C} \))
- \( T_s \) = pavement surface temperature (\( ^\circ \text{C} \))
- \( d \) = depth below surface (mm)

Kennedy et al. (1994) suggested using a simplified version of Equation 4.1 such that

\[ T_d = T_s + 0.051 d - 6.3 \times 10^{-5} d^2 \]  \hspace{1cm} (4.2)

According to Zubeck and Vinson (1995), the minimum pavement temperature \( T_{dmin} \) could be estimated from Equation 4.2 by substituting \( nT_a \) for \( T_s \), where n-factor (n) is 0.9, and \( T_a \) is the minimum air temperature selected for design interval.
Solaimanian and Kennedy (1993) reported their conclusion on minimum pavement temperature as follows:

Analysis of a number of field cases indicates that the minimum pavement temperature during winter time is in most cases one or two °F higher than the minimum air temperature. Therefore, it seems reasonable and on the safe side to assume that the lowest pavement temperature is the same as the lowest air temperature.

The above recommendation was adopted in Superpave calculation of minimum design pavement temperature (Kennedy et al. 1994). The Asphalt Institute, however, used work by Robertson (1987) as basis for selecting minimum design pavement temperature. Robertson's suggested relationship between minimum pavement temperature $T_{\text{min}}$ (°C) and minimum air temperature $T_{\text{air}}$ (°C) for a given depth $D$ (mm) is given by

$$T_{\text{min}} = 0.895 T_{\text{air}} + (0.02 - 0.0007 T_{\text{air}})D + 1.7$$  \hspace{1cm} (4.3)

The Asphalt Institute (1995) provides this equation for determining the low pavement design temperature:

$$T_{\text{min}} = 0.895 T_{\text{air}} + 1.7$$ \hspace{1cm} (4.4)

This equation was based on data provided by Canadian SHRP and represents Canadian experience with performance of asphalt binder.

**4.3 ANALYSIS OF ALASKAN FIELD PAVEMENT TEMPERATURE DATA**

The main objectives of analyzing field pavement temperature data of the selected sites were:

1. Develop correlations between air and pavement temperature for Alaskan conditions that could be used to validate the present Superpave and Asphalt Institute criteria for low pavement design temperature. These correlations could also be used for improved binder selection according to Superpave performance-based specifications.

2. Use air and pavement temperature correlations together with Superpave Weather Database to develop air and pavement temperature contour plots for the Alaskan highway system.

3. Establish probability levels for low temperature cracking associated with using PG binder specification as applicable for different climatic zones in Alaska.
4.3.1 Correlations between Air and Pavement Temperature

Initial examination of pavement and air temperature data strongly indicate that the variation of minimum air and pavement temperature is dependent on site location, and specifically on the climatic zone. In order to account for the effects of climate on pavement temperature, data from various stations were grouped according to various climatic zones as defined in the Environmental Atlas of Alaska (1984). These zones are shown in Figure 4.1 and described as follows:

Maritime: Dominated by maritime influences. Small temperature variations, high humidity, heavy precipitation, and high cloud and fog frequencies. Little or no freezing weather. Cool summers and warm winters. Surface winds strong and persistent. Mean annual temperature 2 °C to 6 °C.

Transition: More pronounced temperature variations throughout the day and year, less cloudiness, lower precipitation and humidity. Surface winds generally light. Mean annual temperature -4 °C and 2 °C.

Continental: Dominated by continental climatic conditions. Great diurnal and annual temperature variations, low precipitation, low cloudiness, and low humidity. Surface winds generally light. Mean annual temperature -9 °C to -4 °C.

Arctic: Temperature variations lower than continental, precipitation is extremely light and strong winds are not uncommon. Mean annual temperature -12 °C to -7 °C. The arctic climate may be affected by marine influence in the summer but not to any great extent in the winter. Surface winds are strong along the coast, but decrease inland.

The groupings of field sections according to the above climatic zones are summarized as follows:

Maritime (M): Homer, Soldotna, Dutch Harbor

Transitional (T): Kenai Spur, Kenai River Crossing, Glenn Hwy MP53

Continental (C): Peger Road, Fox, Tok, Cantwell, Chulitna

Arctic (A): None

Distribution of temperature data and correlations between air and pavement temperature for the period covering December 1995, January 1996, and both December and January 1995/96 for the different climatic zones are presented in Appendix G.

The difference between air and pavement temperature (10 mm below surface) is illustrated by comparing the variation of minimum daily air and pavement temperatures (Figures 4.2-
Source: Arctic Environmental Information & Data Center (1974)

CLIMATIC ZONES OF ALASKA

M  MARITIME
T  TRANSITIONAL
C  CONTINENTAL
A  ARCTIC
4.4), and also by comparing the hourly air and pavement temperature variation for a selected day (January 15) as shown in Figures 4.5-4.7. The results clearly indicate a significant effect of climatic zone on the trend of air-pavement variation. For all climatic zones, the minimum air and pavement temperatures increase during December but decrease during January (Figures 4.2-4.4). In all cases, the air seems to warm and cool faster than the pavement, however, the difference in minimum air and minimum pavement temperature seems to be least significant for the maritime climatic zone as shown in Figures 4.2-4.4. Comparisons of hourly air and pavement temperature are presented in Figures 4.5-4.7. Results indicate that in the continental climatic zone, pavement temperature seems to be the least affected by changes in air temperature fluctuations. The average and standard deviation of minimum daily air and pavement temperatures for the different climatic zones are summarized in Table 4.1.

<table>
<thead>
<tr>
<th>Climatic Zone</th>
<th>Minimum Air Temp. (°C)</th>
<th>Minimum Pavement Temp. (°C)</th>
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<tr>
<td>Transitional (T)</td>
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</tr>
<tr>
<td>Continental (C)</td>
<td>-27 (10)</td>
<td>-26 (8)</td>
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</tbody>
</table>

* Values in parentheses refer to standard deviation

Although average values of minimum temperature for both air and pavement are quite close, the standard deviation of the pavement temperature distribution is lower. Best fit regression analysis between minimum air and pavement temperature would provide a better comparison over the observed temperature range for the period of analysis. Correlations between minimum air temperature $T_{air}$ (°C) and minimum pavement temperature $T_{min}$ (°C) are as follows:

**Maritime Zone**

$$T_{min} = 0.853 T_{air} - 1.735 \quad \text{(R}^2 = 0.96)$$

**Transitional Zone**

$$T_{min} = 0.779 T_{air} - 3.129 \quad \text{(R}^2 = 0.85)$$

**Continental Zone**

$$T_{min} = 0.677 T_{air} - 7.561 \quad \text{(R}^2 = 0.81)$$
DAILY VARIATION OF AIR AND PAVEMENT TEMPERATURE
(M - HOM/DEC. 95 & JAN. 96)

Figure 4.2: Variation of Daily Minimum Air and Pavement Temperature
(Maritime Zone (M) - Homer)
DAILY VARIATION OF AIR AND PAVEMENT TEMPERATURE (T - KSP/DEC. 95 & JAN. 96)

Figure 4.3: Variation of Daily Minimum Air and Pavement Temperature
(Transitional Zone (T) - Kenai Spur)
DAILY VARIATION OF AIR AND PAVEMENT TEMPERATURE
(C - PEGER/DEC. 95 & JAN. 96)

Figure 4.4 : Variation of Daily Minimum Air and Pavement Temperature
(Continental Zone (C) - Peger)
Figure 4.5: Variation of Hourly Air and Pavement Temperature (Maritime Zone (M) - Homer)
HOURLY VARIATION OF AIR AND PAVEMENT TEMPERATURE
(T - KSP/JAN. 15, 1996)

Figure 4.6: Variation of Hourly Air and Pavement Temperature
(Transitional Zone (T) - Kenai Spur)
Figure 4.7: Variation of Hourly Air and Pavement Temperature (Continental Zone (C) - Peger)
For all zones (M, T, C)

\[ T_{\text{min}} = 0.809 T_{\text{air}} - 3.150 \quad (R^2 = 0.91) \] (4.8)

The variation of minimum air and pavement temperature using Equations 4.5-4.8 is compared with the Asphalt Institute and Superpave relationships for predicting minimum pavement temperature (Figures 4.8-4.11). Results indicate that minimum pavement temperature could be higher or lower than minimum air temperature. For lower temperature range, which is generally more applicable for low temperature design considerations, the minimum pavement temperature is always higher than minimum air temperature. The predicted difference, though, depends on the climatic zone under consideration. For Continental Zone, the minimum pavement temperature could be as much as 7 °C higher than the minimum air temperature, whereas for the Maritime and Transitional Zones the difference could be as much as 2 °C and 4 °C, respectively. In this case, the Superpave criterion for selecting minimum pavement design temperature (i.e. minimum pavement temperature equals minimum air temperature) is conservative, whereas the Asphalt Institute criterion is unconservative since it yields pavement temperature values that are higher than those observed for Alaskan pavements.

4.3.2 Contour Maps for Alaskan Pavements

Data from Superpave Weather Database (SWD) were used to establish contour maps for Alaskan pavements corresponding to 50 percent and 98 percent reliability for minimum air temperature. The location of weather stations is shown in Figure 4.12. Correlations between minimum air and pavement temperatures (Equations 4.5-4.7) were used together with the SWD to develop contour maps for 50 percent and 98 percent reliability minimum pavement temperatures as follows:

1. All SWD stations in Alaska were grouped according to geographic location into their respective climatic zones.

2. Minimum pavement temperature at 50 % and 98 % reliability was determined for each SWD site using Equations 4.5-4.7, depending on the climatic zone of the SWD location.

3. The minimum pavement temperature data were then used in the development of the contour plots.

All contour plots are included in Appendix H. Typical representation for minimum pavement temperature corresponding to 98 percent reliability is shown in Figures 4.13-4.16 for different Alaskan regions, specifically, Fairbanks, Anchorage, and Southeast. These maps could be very easily used to estimate the low temperature design value.
Minimum Daily Air Temperature vs. Minimum Daily Pavement Temperature
(Group M Dec. 95 - Jan. 96)

\[
y = 0.8527x - 1.7346 \\
R^2 = 0.9596
\]

- **SHRP**
- **Asphalt Institute**
- **Regression**

Figure 4.8: Correlation between Minimum Air and Pavement Temperature (Maritime Zone (M) - Homer)
Minimum Daily Air Temperature vs. Minimum Daily Pavement Temperature
(Group T Dec. 95 - Jan. 96)

\[ y = 0.7788x - 3.1297 \]
\[ R^2 = 0.8527 \]

Figure 4.9: Correlation between Minimum Air and Pavement Temperature
(Transitional Zone (T) - Kenai Spur)
Minimum Daily Air Temperature vs. Minimum Daily Pavement Temperature
(Group C Dec. 95 - Jan. 96)

\[ y = 0.6774x - 7.5608 \]
\[ R^2 = 0.8102 \]

Figure 4.10: Correlation between Minimum Air and Pavement Temperature (Continental Zone (C) - Peger)
Minimum Daily Air Temperature vs. Minimum Daily Pavement Temperature
(Group C, M, T Dec. 95 - Jan. 96)

\[ y = 0.809x - 3.1498 \]

\[ R^2 = 0.9091 \]

- SHRP
- Asphalt Institute
- Regression

Figure 4.11: Correlation between Minimum Air and Pavement Temperature (All Zones)
Location of Weather Stations

Transportation Research Center
Alaska Data Visualization & Analysis Lab
University of Alaska Fairbanks

Figure 4.12: Location of Weather Stations
Figure 4.13: Minimum Pavement Temperature Contour Map (98% Reliability)
Pavement Temperature
98% Reliability - Fairbanks

Figure 4.14: Fairbanks Minimum Pavement Temperature Contour Map
(98% Reliability)
Pavement Temperature
98% Reliability – Anchorage

Figure 4.15: Anchorage Minimum Pavement Temperature Contour Map
(98% Reliability)
Pavement Temperature
98% Reliability - Southeast

Transportation Research Center
Alaska Data Visualization & Analysis Lab
University of Alaska Fairbanks

Figure 4.16: Southeast Minimum Pavement Temperature Contour Map
(98 % Reliability)
required for binder selection in accordance with Superpave performance-based specifications.

4.3.3 Low Temperature Probabilities for Alaskan Climatic Zones

According to Superpave binder specifications, the susceptibility of the pavement surface to low temperature cracking increases when its temperature becomes lower than the PG grade low temperature specification. Of particular interest in this case, is the use of minimum pavement temperature distribution data to estimate the probability that the pavement temperature be less than a given PG low temperature end. These estimates were performed for all climatic zones and are summarized in Table 4.2.

Table 4.2 Probability Values Corresponding to Pavement Temperature to be Less than PG Low Temperature Specification

<table>
<thead>
<tr>
<th>PG Grade Low Temp. (°C)</th>
<th>Maritime Zone (% Probability)</th>
<th>Transitional Zone (% Probability)</th>
<th>Continental Zone (% Probability)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-10</td>
<td>55</td>
<td>60</td>
<td>98</td>
</tr>
<tr>
<td>-16</td>
<td>26</td>
<td>30</td>
<td>90</td>
</tr>
<tr>
<td>-22</td>
<td>8</td>
<td>11</td>
<td>70</td>
</tr>
<tr>
<td>-28</td>
<td>1.7</td>
<td>2.3</td>
<td>40</td>
</tr>
<tr>
<td>-34</td>
<td>&lt; 1</td>
<td>&lt; 1</td>
<td>16</td>
</tr>
<tr>
<td>-40</td>
<td>&lt; 1</td>
<td>&lt; 1</td>
<td>4</td>
</tr>
<tr>
<td>-46</td>
<td>&lt; 1</td>
<td>&lt; 1</td>
<td>&lt; 1</td>
</tr>
</tbody>
</table>

4.4 DESIGN CONSIDERATIONS

The binder and mixture specifications proposed by Superpave are tied to maximum and minimum pavement temperatures depending on a location. In this study, since the emphasis is on low temperature cracking, efficient and accurate prediction of minimum pavement temperature is necessary. The following method for predicting minimum design pavement temperature is proposed:

1. Determine the location of the pavement under consideration and the corresponding climatic zone.

2. Choose minimum air temperature for 50% or 98% reliability level using data from the closest Superpave weather station to the pavement location under consideration. A 98% reliability level is suggested for a 20-year design life. Lower reliability levels could be used depending on field experience and economic considerations.
3. Use Equations 4.5-4.7 as applicable to estimate the corresponding minimum pavement temperature.

4. Steps 2 and 3 could be skipped and minimum design pavement temperature could be estimated using the developed contour maps.

5. Once the minimum pavement design temperature is determined, a choice of the corresponding PG grade low temperature could be ascertained. The PG grade low temperature for the selected binder should be less or equal to the minimum pavement design temperature.

6. The minimum pavement temperature reliability for a given PG grade low temperature could be estimated from Table 4.2.

4.5 SUMMARY

In this chapter correlations between minimum air and pavement temperature were established for different Alaskan climatic zones. Results indicate that the minimum pavement temperature could be lower or higher than minimum air temperature. However, for low temperature ranges the pavement temperature is generally warmer than air temperature by 2 °C to 7 °C depending on climate zone. Comparisons of minimum pavement temperature for Alaskan conditions with Superpave and the Asphalt Institute criteria for selecting minimum pavement design temperature were made. Results show that the criteria are not appropriate for Alaskan pavements. The Superpave prediction is conservative whereas the Asphalt Institute’s prediction model is unconservative.

In addition contour maps corresponding to 50 percent and 98 percent reliability were developed for minimum air and pavement temperatures. The temperature correlations and contour maps provide a simple tool for design engineers to estimate minimum design pavement temperature for selecting appropriate binder specifications.
REFERENCES


CHAPTER FIVE
THERMAL CRACKING MODELS FOR ALASKAN PAVEMENTS

5.1 INTRODUCTION

Low temperature cracking occurs as a result of the tendency of the pavement surface or underlying layers to contract as a result of pavement cooling. The development of tensile stresses associated with frictional restraint between the pavement layers could cause thermal cracking in the pavement structure. Specifically, the occurrence of low temperature cracking could be attributed to the following mechanisms (Kelly 1966; McHattie et al. 1980; Anderson et al. 1990):

1. A single low-temperature excursion that causes the pavement to shrink such that the corresponding tensile stresses in the pavement surface exceed its tensile strength.

2. Thermal fatigue caused by repeated thermal cycling that will cause the development and growth of microcracks in the pavement surface.

3. In many cases, low temperature cracking occurs as a result of shrinkage of the whole pavement structure rather than the asphalt surface. These cracks could extend vertically several feet below the pavement structure and are controlled essentially by the thermal response of the soils rather than the asphalt concrete layer.

This chapter aims at investigating the cracking behavior of selected pavement sections in Alaska using both laboratory tests and field observations. The main objective of this study was to determine the applicability of the Superpave model, and other models to predict low temperature cracking of Alaskan pavements, and to develop, if necessary, appropriate models to estimate thermal crack progression with time. It should be noted that this study does not specifically address causes or model low temperature cracking mechanisms induced by shrinkage of the whole pavement structure or reflective cracking for overlays. The low temperature cracking surveys, though, probably include these mechanisms, and are therefore implicitly covered in the analysis presented in this chapter.

5.2 BACKGROUND

A summary of thermal cracking models proposed by a number of researchers has been presented by Anderson et al. (1990). These models include both mechanistic and empirical procedures to estimate low temperature cracking in pavements. More recently, improved models have been developed using advanced analysis and testing procedures such as the thermal stress restrained specimen test (TSRST), and the indirect tension creep test using
Superpave™ indirect tension device (IDT) (Kanerva 1993; Lytton et al. 1993; Kennedy et al. 1994). All these models could be categorized as follows:

1. Limiting stiffness models

2. Mechanistic models using limiting stresses, strains, and the concepts of fracture mechanics

3. Regression models

4. TSRST models

5. Superpave model

5.2.1 Limiting Stiffness Models

According to Anderson et al. (1990), this approach is by far the most common for predicting low temperature cracking susceptibility in asphalt concrete pavements. This method involves comparing the stiffness of the asphalt cement or the asphalt concrete mix corresponding to the minimum service temperature with a limiting stiffness value which, if exceeded, could induce pavement cracking. Limiting stiffness range has been suggested from 140 MPa for a loading time of 2.8 hrs to 1 GPa at 30 min. (Fromm and Phang 1971; The Asphalt Institute 1981). For conventional asphalt concrete mixtures a limiting stiffness value of 18 GPa at 30 min. loading is suggested (Gaw et al. 1974).

The limiting stiffness method predicts the minimum pavement temperature below which cracking could occur. It does not, however, predict the extent of low temperature cracking and its development with time.

5.2.2 Mechanistic Models

Hills and Brien (1966) proposed a procedure for determining the critical pavement temperature associated with low temperature cracking. Their method assumes that the pavement behaves as a linear elastic material with a given coefficient of thermal expansion (2x10^{-4}/°C). The analysis is started from a reference temperature of 0°C where the pavement is assumed to be stress free. A rate of cooling of 10°C per hour is assumed. The stiffness of the asphalt concrete for each increment of temperature is determined from the Van der Poel nomograph. The stresses in the pavement at the end of each increment are summed up and the total stress is compared with the tensile strength of the asphalt concrete estimated from the typical tensile strength-stiffness relationship as that suggested by Heukelom (1966). The analysis is continued until the tensile stress becomes equal to the tensile strength of the asphalt concrete. The method estimates the critical fracture temperature of the pavement but does not predict crack development with time. Another shortcoming of this method is that it does not allow for stress relaxation during cooling caused by the viscoelastic behavior of the asphalt concrete material.
Shahin and McCullough (1972) developed a computer model, TC-1, that calculates thermal stresses and strains, and the percentage of the pavement surface cracked as a function of time using probabilistic methods. The model also accounts for thermal fatigue cycling effects on low temperature cracking. The calculation of thermal stresses and strains in this model assumes that the pavement surface is fully restrained and behaves like an infinite beam. The thermal stresses are calculated over the range of temperatures provided as input to the program. The stiffness of the asphalt mix is estimated using the relationships developed by Van der Poel and Heukelom and Klomp (Van der Poel 1954; Heukelom and Klomp 1964). Thermal stresses are compared with the tensile strength determined from the tensile strength - temperature relationship used in the program input, and the probability of cracking is estimated from the probability of the tensile stress to exceed the tensile strength at any point in the asphalt concrete mixture. Initial predictions of the model seemed to agree with observed cracking of the Ontario and Saint Anne test roads in Canada (Shahin and McCullough 1972). Further comparisons with other pavement sections in Michigan yielded poor correlations (Rauhut et al. 1984).

Finn et al. (1977) developed computer model COLD to estimate thermal stress development and low temperature cracking in the pavement. COLD computes pavement temperature using heat flow equations through a solid medium. It accounts for absorptivity, emissivity, and convection of the surface, and thermal conductivity of the asphalt concrete. The computed pavement temperature is then used to determine the thermal stresses. The variation of creep modulus, tensile strength, and stiffness of the asphalt concrete as a function of temperature is included in the analysis. Comparisons of predictions using COLD with other observations are difficult because of the lack of material and climatic properties required for the test sections (Anderson et al. 1990).

Ruth et al. (1981) developed a mechanistic model to predict low temperature stresses and strains in pavements. The parameters used in this model include temperature-viscosity relationship of the asphalt, pavement cooling rate, and minimum pavement temperature. Both elastic and creep strains are calculated using incremental procedures. The low temperature criteria could include pavement temperature, stresses, strains, and strain energy. The model requires viscosity and flow input from measurements using the Schweyer rheometer. The model does not account for thermal fatigue effects and has not been verified because of the difficulty in determining material and site parameters required for the analysis.

A number of researchers applied the principles of fracture mechanics to determine the load associated fracture behavior of asphalt concrete pavements (Movenzadeh 1967; Herrin and Baghat 1968; Majidzadeh et al. 1969 and 1976). Lytton et al. (1983) used fracture mechanics theory to investigate non-load associated thermal fatigue cracking. Lytton's model, known as THERM, uses the basic Paris-Erdogan fatigue law, but extends its application to include viscoelastic fracture mechanics theory proposed by Schapery (1973). In this case, the methodology proposed in Lytton's model allows the calculation of fatigue crack propagation parameters to account for the viscoelastic behavior of asphalt concrete (Germann and Lytton 1979). The THERM computer program allows prediction...
of thermal crack initiation and the amount of cracking with time. It should be pointed out, however, that a total of 576 computer runs for four sites in northern Texas and four sites in Michigan provided data for the necessary regression equations used in the model (Anderson et al. 1990). Comparisons of predicted cracking using THERM with field observations was quite poor (Rauhut et al. 1984).

5.2.3 Regression Models

Statistical models using regression analysis of field data could be used to develop correlations between low temperature cracking and material and pavement parameters. Although these models are simple to use and do correlate well for a given set of data, their predictive ability is limited in general to a particular location, materials, and type of pavements for which the models were originally developed.

Hajek and Haas (1972) developed correlations between the cracking index, defined as the number of cracks per 152 m (500 ft), and the pavement thickness, age, subgrade type, and asphalt stiffness:

\[
I = 30.3974 - 2.1516 (D) + 1.24958 (M) + (6.7966 - 0.8740 (t) + 1.3388 (A)) \log S_b + 0.06026 (S_b) \log D
\]  

(5.1)

where

- \( I \) = cracking index
- \( D \) = indicator variable for subgrade type, unitless: clay, \( D = 2 \); loam, \( D = 3 \); sand, \( D = 5 \)
- \( M \) = winter design temperature, °C, defined as the temperature below which only 1% of the hourly temperatures occur during the coldest January for a 10 year period
- \( A \) = age of the pavement, years
- \( t \) = thickness of the asphalt concrete layer, inches
- \( S_b \) = stiffness of the original bitumen at 20,000 sec loading time and at the winter design temperature, kg/m²

The Hajek-Haas model was used to predict the cracking index for pavements in Texas and Michigan (Rauhut et al. 1984). The predictions showed poor agreement with field observations.
Haas et al. (1987) used low temperature cracking data from 26 airfields in Canada to establish a regression equation for crack spacing as follows:

\[ CS = 218 + 1.28 \, (t) + 2.52 \, (T) + 30 \, (PVN) - 60 \, (CFX) \]  

(5.2)

where

- \( CS \) = average crack spacing, m
- \( t \) = thickness of the asphalt concrete layer, cm
- \( T \) = minimum recorded site temperature, °C
- \( PVN \) = McLeod's Pen Vis Number (for extracted asphalt cement)
- \( CFX \) = coefficient of thermal contraction, mm/1000mm/°C

The main disadvantage of the Haas model is that it does not account explicitly for the influence of pavement aging on crack spacing and therefore the extent of cracking with time. Moreover, about half the field observations were for overlays, thereby probably influencing the results due to reflection cracking. In addition, extracted asphalt cement properties are used which limits the use of the above equation when new pavements are considered.

McHattie et al. (1980) reported the results of a three-year study to review the construction and performance of pavement structures in Alaska. One hundred twenty pavement sections were chosen and surveyed for fatigue, thermal cracking, roughness. In relation to thermal cracking they observed the following:

Although the formation of these crack types has been ascribed to a variety of mechanisms, those occurring in most areas of Alaska are almost certainly due to prolonged and severe thermal contraction of, in many cases, the entire pavement structure. A number of large transverse cracks have been examined near Fairbanks and some examples extend vertically several feet below the pavement surface.

Osterkamp et al. (1986) excavated a number of transverse cracks of pavements in interior Alaska. They observed that these cracks, in many instances, occur in the embankment and extend to a depth of at least 1 m below the pavement surface. In another study to evaluate the effectiveness of geotextiles to improve performance of Alaskan roads, Reckard (1989) concluded that in many cases, embankment cracks seemed to reflect quickly through the new pavement surface, but that this process might be slowed down by the grade raises or subcuts.
Using thermal cracking data of a number of pavements in Alaska, McHattie et al. (1980) found strong correlations between frequency of transverse cracks, pavement age, and other climatic factors such as freezing index, precipitation, and mean temperature. A typical relationship for crack spacing and freezing index is expressed as follows:

\[
CS = 1609 \left( 114.8 - 0.033 \frac{FI}{FI} \right)
\]

where

- \(CS\) = crack spacing, m
- \(FI\) = freezing index, °C days

\[5.3\]

### 5.2.4 TSRST Models

Laboratory measurements of fracture strength and fracture temperature of asphalt concrete mixtures have been suggested by Monismith et al. (1965). The argument was that by maintaining the test specimen in the laboratory at constant length during cooling, thermal stresses will build up to compensate for the tendency of the specimen to shrink as the temperature drops. Failure will occur when the thermal stress becomes equal to the tensile strength. More recently, Arand (1987) and Jung and Vinson (1992) refined the testing procedure in which the specimen could be cooled at a constant rate and the length is adjusted by using a step-motor that maintains the length of the specimen constant through appropriate feedback of specimen contraction. This testing system, which is referred to as the Thermal Stress Restrained Specimen Test (TSRST) is designed to simulate field conditions associated with low temperature stress development in the asphalt concrete layer (Jung and Vinson 1992).

Kanerva (1993) proposed both deterministic and probabilistic models to determine low temperature cracking of asphalt concrete pavements. One approach to predict low temperature cracking is to use the deterministic model proposed by Kanerva (1993). In this model, cracking is assumed to occur when: 1) restraint force is great enough to prevent the slab movement and subsequent release of thermal stresses, and 2) the pavement temperature is cold enough to produce thermal stresses equal to the tensile strength of the surface material. Kanerva (1993) developed correlations between pavement cracking temperature (CT in °C), TSRST fracture temperature of the original mix (FT₀ in °C), and pavement age (AGE in years) as follows:

\[
CT = 1.03 (FT₀) + 0.51 (AGE)
\]

When pavement temperature (PT) becomes lower or equal to the pavement cracking temperature (CT) cracking will occur. It was assumed that as the pavement cracking temperature increases due to aging, the tensile strength decreases. The corresponding crack spacing depends on the tensile strength of the asphalt concrete layer. In this case, it
was also assumed that the tensile strength - temperature relationship is linear and that its slope does not change with age. The pavement strength \( S_t \) is then given by

\[
S_t = \frac{\left( S_1 - F_S \right)}{\left( T_1 - F_{T_o} \right)} \left( P_T - C_T \right) + F_S
\]  

(5.5)

where

\( F_{T_o}, F_S = \) TSRST fracture temperature and fracture strength for the original mix.

\( T_1, S_1 = \) Another point on the tensile strength-temperature curve corresponding to temperature and strength respectively for the same age (1)

\( C_T = \) pavement cracking temperature for a given age (2)

\( P_T, S_t = \) pavement temperature and corresponding strength for age (2)

The number of cracking incidents, \( N \), is given by

\[
N = \text{INTEGER} \left[ \left( \log \left( \frac{f_L}{2} / S_t \right) / \log 2 \right) + 1 \right]
\]  

(5.6)

and the corresponding crack spacing \( L_{N+1} \) will be

\[
L_{N+1} = \frac{L}{2^N}
\]  

(5.7)

where

\( L = \) original slab length

\( \gamma = \) density of the asphalt concrete layer

\( f = \) coefficient of friction between slab and base

The application of the Kanerva's model requires knowledge of friction coefficient, \( f \), between the asphalt concrete layer and the underlying base. Predicted results are influenced by the value of \( f \) and the original slab length used in Equations 5.6 and 5.7. The derivation of this model does not allow for stress relaxation during thermal stress build-up resulting from the viscoelastic behavior of the asphalt layer. The use of TSRST partially accounts for stress relaxation. However, monotonic loading is assumed.

5.2.5 Superpave Model

The Superpave Mix Design and Analysis is a product of SHRP, which took place from 1987 through 1993. Superpave is a collection of new specifications and test methods for
asphalt binders, aggregates, and mixtures. A performance-based asphalt binder specification with physical property tests is used for binder selection. Asphalt mixtures are designed based on volumetric proportioning, as with traditional methods, but a new type of gyratory compactor is used. The compactor is called the Superpave Gyratory Compactor. The combination of Superpave binder analysis and volumetric proportioning of Superpave mixtures is referred to as "Superpave Mix Design". This was formerly referred to as "Level I Mix Design."

According to SHRP, the Superpave Mix Design is all that is required when the expected traffic is less than $10^6$ ESALs. If traffic is expected to exceed $10^6$ ESALs, then both mix design and analysis are necessary. Mix analysis is conducted to evaluate the ability of the mix to perform at a desired level. In this case, mix properties are determined using two pieces of equipment: the Superpave Shear Tester (SST) and the Indirect Tension Device (IDT). The SST is used to determine the mix mechanical properties which can be related to fatigue and rutting. The IDT is used to determine properties that can be related to fatigue and low temperature cracking. The type and amount of testing required using these devices depend on the level of traffic expected. If traffic is greater than $10^6$ ESALs and less than $10^7$ ESALs an "Abbreviated Analysis," formerly called "Level II Mix Design" is required. If the expected traffic is greater than $10^7$ ESALs a "Complete Analysis," formerly referred to as "Level III Mix Design" is required. In both types of analyses, performance of a Superpave mixture is estimated with sophisticated pavement performance prediction models. The models give consideration to material characteristics, pavement structure, and seasonal environmental conditions. A summary of the theoretical development and validation of Superpave performance prediction models is presented elsewhere (Lytton et al. 1993).

Material properties required for low temperature cracking analysis using the Superpave model include IDT creep and strength characteristics. IDT creep testing yields two properties, creep compliance and Poisson's ratio. The creep compliance, defined as the ratio of the amount of strain with time to the applied stress, is determined over a range of temperatures and used to develop master creep compliance curves and master relaxation modulus curves. The relaxation modulus is used to make low temperature stress predictions and the slope of the creep modulus curve is used in predicting crack propagation. The low temperature strength tests are used to determine the relationship between failure strength and temperature. The tensile failure strength is used in models to predict crack propagation and crack intensity. The Superpave low temperature cracking analysis utilizes the IDT material properties as part of the following scheme:

1. Input module
2. Transformation model for the master relaxation modulus curve
3. Environmental effects model
4. Pavement response model; and
5. Pavement distress model

The relationships of the components to each other are illustrated in Figure 5.1.
Figure 5.1: Superpave Modeling Components for Low Temperature Cracking
Attempts to verify the Superpave model for low temperature cracking have been conducted by a number of investigators (Roque and Buttlar 1992, Roque et al. 1995). Results of observed thermal cracking of three C-SHRP test sections in Canada after three winters confirmed Superpave predictions (Roque et al. 1995). Additional data for seven C-SHRP sections in Alberta, Canada, were presented by Roque and Buttlar (1992). These results showed excellent agreement between the Superpave predictions and observed cracking.

5.3 LOW TEMPERATURE CRACKING MODELS FOR ALASKAN PAVEMENTS

In this study, field observations of low temperature cracking for typical pavements in Alaska having both conventional asphalt and polymer modified asphalt surface layers were compared with predictions using a number of available models, specifically,

1. The Superpave thermal cracking model (TCMODEL)
2. AKDOT&PF freezing-index model (McHattie et al. 1980)
3. Haas model (Haas et al. 1987)
4. Kanerva's TSRST model (Kanerva 1993)

A summary of the field pavements surveyed, including material properties, climate data, and geometric properties is presented in Tables 5.1-5.3.

5.3.1 Comparison with Superpave TCMODEL

Attempts were made to use Superpave TCMODEL to analyze all the surveyed pavement sections. Analyses were conducted using the Superpave software version available at the Asphalt Institute. The software is difficult to work with, and in many cases attempts to obtain thermal cracking predictions were not successful. The current software is not ready for practical applications and in many cases no predictions could be made. In our case, for example, predictions for only 4 out of 16 sections could be obtained. Results are summarized in Table 5.4 and illustrated in Figures 5.2 and 5.3. Comparisons with field observations (Figure 5.4) show that the Superpave model predicts less crack spacing than is observed and is therefore very conservative. The amount of cracking predicted for all four field pavement sections was equal to 57 m/150 m. It should be mentioned that TCMODEL predicts cracking per 500 feet (about 150 m) section.

5.3.2 Comparison with Other Models

Results of comparisons with other models proposed by McHattie et al. (1980), Haas et al. (1987), and Kanerva (1993) are illustrated in Figures 5.5 - 5.7. In case of Kanerva's
## Table 5.1: Summary of Surveyed Field Sections

<table>
<thead>
<tr>
<th>Project ID</th>
<th>Site</th>
<th>Binder Type (1)</th>
<th>Placed in (2)</th>
<th>Length of section surveyed (m)</th>
<th>Surveyed Section</th>
<th>Number of lanes</th>
<th>Width of section surveyed (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>C street, Anchorage</td>
<td>AC5</td>
<td>1986</td>
<td>1430</td>
<td>Benson Blvd.</td>
<td>4</td>
<td>16</td>
</tr>
<tr>
<td>n</td>
<td>A street, Anchorage</td>
<td>PlusRide</td>
<td>1985</td>
<td>1213</td>
<td>Fireweed traffic light</td>
<td>4</td>
<td>16</td>
</tr>
<tr>
<td>b</td>
<td>Elmendorf AFB, Anch.</td>
<td>AC5+3%SBS</td>
<td>1995</td>
<td>791</td>
<td>Aero Club sign on Acacia Str.</td>
<td>2</td>
<td>11</td>
</tr>
<tr>
<td>c</td>
<td>Elmendorf AFB, Anch.</td>
<td>AC5+6%SBS</td>
<td>1991</td>
<td>775</td>
<td>Aero Club sign on Acacia Str.</td>
<td>2</td>
<td>11</td>
</tr>
<tr>
<td>e1</td>
<td>Haines Hwy.</td>
<td>PBA3</td>
<td>1993</td>
<td>3600</td>
<td>Little Boulder bridge</td>
<td>2</td>
<td>8</td>
</tr>
<tr>
<td>e2</td>
<td>Haines Hwy.</td>
<td>AC-5</td>
<td>1993</td>
<td>266</td>
<td>2050 m. North of MP30</td>
<td>2</td>
<td>8</td>
</tr>
<tr>
<td>f</td>
<td>Denali Park Road</td>
<td>AC-20R</td>
<td>1991</td>
<td>22200</td>
<td>Visitors' Center</td>
<td>2</td>
<td>8</td>
</tr>
<tr>
<td>g1</td>
<td>Danby Str., Fairbanks</td>
<td>Asphalt-Rubber</td>
<td>1988</td>
<td>96</td>
<td>Wembley Str. intersection</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>g2</td>
<td>Danby Str., Fairbanks</td>
<td>AC2.5</td>
<td>1988</td>
<td>96</td>
<td>Wembley Str. intersection</td>
<td>1</td>
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</tr>
<tr>
<td>h1</td>
<td>Parks Hwy., Fairbanks</td>
<td>AC5</td>
<td>1995</td>
<td>163</td>
<td>Traffic light at University &amp; Mitchell</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>h2</td>
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<td>PBA3</td>
<td>1995</td>
<td>201</td>
<td>Yield sign at University &amp; Mitchell</td>
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<tr>
<td>i</td>
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<td>SHRP Level I</td>
<td>1995</td>
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<td>Runamuck traffic light</td>
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<td>j</td>
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<td>Arctic Grade AC5</td>
<td>1986</td>
<td>200</td>
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<td>24</td>
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</tr>
<tr>
<td>k</td>
<td>Fort Wainwright</td>
<td>Arctic Grade AC2.5</td>
<td>1990</td>
<td>326</td>
<td>South of Taxiway A</td>
<td>2</td>
<td>8</td>
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<td>l</td>
<td>Rewak Dr., Fairbanks</td>
<td>PBA6</td>
<td>1993</td>
<td>149</td>
<td>Traffic light at University Ave.</td>
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<td>8</td>
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<tr>
<td>m</td>
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<td>675</td>
<td>Glide station between Txwy. B &amp; C</td>
<td>2</td>
<td>8</td>
</tr>
</tbody>
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Note:

(1) Binder Description is given in Table 2.4
(2) Sampling and testing were done in Fall 1995
Table 5.2: Material Properties and Climate Data for the Field Pavements

<table>
<thead>
<tr>
<th>Project ID</th>
<th>Site</th>
<th>Binder Type</th>
<th>PG Grade</th>
<th>Average TSRST Fracture Temp. (C)</th>
<th>Average TSRST Fracture Strength (kPa)</th>
<th>Low Air Temp. C 50% Rel.</th>
<th>98% Rel.</th>
<th>Low Pavement Temp. C 50% Rel.</th>
<th>98% Rel.</th>
<th>Climatic Zone (3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>C street, Anchorage</td>
<td>AC5</td>
<td>PG58-22</td>
<td>-25.7</td>
<td>2811</td>
<td>-29</td>
<td>-37</td>
<td>-27</td>
<td>-32</td>
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</tr>
<tr>
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<td>AC5+3%SBS</td>
<td>PG58-22</td>
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<td>-28</td>
<td>-37</td>
<td>-25</td>
<td>-32</td>
<td>T</td>
</tr>
<tr>
<td>c</td>
<td>Elmendorf AFB, Anch.</td>
<td>AC5+6%SBS</td>
<td>PG64-22</td>
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<td>3033</td>
<td>-28</td>
<td>-37</td>
<td>-25</td>
<td>-32</td>
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<td>PG64-34</td>
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<td>-47</td>
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<td>-39</td>
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<td>Asphalt-Rubber</td>
<td>PG58-40</td>
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<td>AC5</td>
<td>PG52-28</td>
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<td>PG58-34</td>
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<tr>
<td>l</td>
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<td>PBA6</td>
<td>PG64-22</td>
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<td>-54</td>
<td>-38</td>
<td>-44</td>
<td>C</td>
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<tr>
<td>m</td>
<td>Deadhorse</td>
<td>Arctic Grade AC2.5</td>
<td>(1)</td>
<td>(2)</td>
<td>(2)</td>
<td>-44</td>
<td>-49</td>
<td>-37</td>
<td>-40</td>
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Note:
(1) : cannot be graded
(2) : not tested
(3) : C = Continental; T = Transitional; M = Maritime; A = Arctic
Table 5.3: Pavement Section Components and Thicknesses

<table>
<thead>
<tr>
<th>Project ID</th>
<th>Site</th>
<th>Layer Thickness, (cm)</th>
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<th></th>
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<td>original AC</td>
<td>overlay AC</td>
<td>Base Course</td>
<td>Subbase</td>
</tr>
<tr>
<td>a</td>
<td>C street, Anchorage</td>
<td>5</td>
<td>-</td>
<td>8</td>
</tr>
<tr>
<td>n</td>
<td>A street, Anchorage</td>
<td>7.5</td>
<td>5</td>
<td>15</td>
</tr>
<tr>
<td>b</td>
<td>Elmendorf AFB</td>
<td>6.5</td>
<td>-</td>
<td>15</td>
</tr>
<tr>
<td>c</td>
<td>Elmendorf AFB</td>
<td>6.5</td>
<td>-</td>
<td>15</td>
</tr>
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<td>-</td>
<td>15</td>
</tr>
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<td>-</td>
<td>15</td>
</tr>
<tr>
<td>f</td>
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<td>15</td>
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<td>-</td>
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Table 5.4 : Superpave Predictions for Crack Progression with Time

<table>
<thead>
<tr>
<th>Year</th>
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<th>Elmendorf 3%SBS Anchorage</th>
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<th>Rewak Drive Fairbanks</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Crack Depth (mm)</td>
<td>Amount of cracking (m / 150 m)</td>
<td>Crack Depth (mm)</td>
<td>Amount of cracking (m / 150 m)</td>
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<tr>
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Figure 5.2: Prediction of Low Temperature Cracking Using Superpave
Figure 5.3: Crack Spacing as Predicted by Superpave Model
Figure 5.4: Comparison of Observed Cracking with Superpave Model Predictions
Figure 5.5: Comparison of Observed Cracking with AKDOT&PF Freezing Index Model
Figure 5.6: Comparison of Observed Cracking with Predictions Using Haas et al. (1987) Model
Figure 5.7: Comparison of Observed Cracking with Predictions Using Kanerva (1993) Model
model, \( L \) was set equal to the length of the section surveyed, \( f \) was assumed equal to 2, \( \gamma \) equal to 23.3 kN/m\(^2\), and \( S_t \) equal to the measured TSRST strength. Equations 5.6 and 5.7 were then used to estimate crack spacing. Poor correlations are generally obtained between observed and predicted low temperature cracking performance by these models. Both the AKDOT&PF model and Haas model are based on statistical regression analyses that are limited to the type, geometry, location, and number of pavements observed. On the other hand, Kanerva's TSRST model uses both TSRST test results and simple assumptions to estimate the thermal stresses in the asphalt concrete layer resulting from pavement cooling and frictional restraint. Predictions using Kanerva's model could have been influenced by lack of fracture temperature and strength data for original specimens. The application of this model to field conditions may require additional validation and calibration studies.

5.3.3 Proposed Models

In this study two approaches were used to estimate crack progression with time:

1. The rate of crack progression model aims at providing an "average" comparison between polymer modified pavements and conventional pavements using low temperature cracking data for the surveyed sections. In this case, the rate of crack progression data for all the conventional sections are grouped and compared with the rate of crack progression data set for the modified sections.

2. The TSRST model is more site specific and provides estimates for crack progression with time depending on TSRST strength, TSRST fracture temperature, age, and minimum air temperature.

5.3.4 Crack Progression Model

In this case, the rate of crack progression is expressed as follows:

\[
\frac{(\log S)}{\Delta \text{age}} = b + a \log \text{age}
\]  

(5.8)

where

\( S = \) crack spacing

\( \text{age} = \) age of a pavement (years)

\( a, b = \) constants

Crack spacing data for all conventional asphalt concrete pavements were grouped and correlated according to Equation 5.8. If \( \Delta \text{age} \) is 1 (i.e., for surveys conducted after 1 year), then \( \Delta (\log S) \) could be expressed as \( \log (S_f/S_o) \), where \( S_f \) is the crack spacing...
surveyed at a given age, and \( S_0 \) is the crack spacing obtained the previous year. Equation 5.8 for conventional asphalt concrete pavements becomes

\[
\log\left(\frac{S}{S_0}\right) = -1.400 + 1.368 \log(\text{age}) \quad (R^2 = 0.72)
\] (5.9)

Similarly, for polymer modified pavements, Equation 5.8 can be written as

\[
\log\left(\frac{S}{S_0}\right) = -1.231 + 1.423 \log(\text{age}) \quad (R^2 = 0.68)
\] (5.10)

It is interesting to note that the minimum crack spacing predicted by Equations 5.9 and 5.10 occurs after 7 years for the conventional pavements and 11 years for the polymer modified pavements. Equations 5.9 and 5.10 are represented graphically in Figure 5.8.

In order to obtain a relationship between crack spacing and age, Equation 5.8 could be integrated. This was done for both conventional and polymer modified field survey data and an average constant of integration for each data set was determined. In all cases an initial length of uncracked pavement section (i.e., \( S_0 \) corresponding to the placement date) was assumed to be 1000 m. For conventional pavements, the model seems to predict crack spacing less than 1 m. A minimum cut-off point of 1 m was used. The resulting predictions were statistically regressed using a polynomial fit. The following correlations were obtained between crack spacing, \( S \) (meters) and age (years):

For conventional pavements,

\[
S = 4.786 - 1.123 (\text{age}) + 0.0386 (\text{age})^2 + 0.00645 (\text{age})^3 - 0.00039(\text{age})^4
\] (5.11)

For polymer modified pavements,

\[
S = 4.980 - 1.451 (\text{age}) + 0.1925 (\text{age})^2 - 0.01066 (\text{age})^3 + 0.00020 (\text{age})^4
\] (5.12)

These equations are presented graphically in Figure 5.9.

It should be emphasized that this model presents "average" crack spacing based on the very limited data collected during the one-year duration of this study. This model, by definition, cannot be used to differentiate between the different material effects and climatic factors within each data set analyzed, but rather could be used to ascertain the difference in average performance of the conventional and polymer modified pavements.

According to this model, low temperature cracking performance is significantly enhanced on the "average" when polymer modifiers are used (including rubberized pavements). The difference in performance is apparent in Figure 5.9 and seems to increase with age until the crack spacing attains a steady state and no change with age is observed.
Figure 5.8: Variation of Average Crack Spacing Ratio with Age for Conventional and Modified Pavement Sections
Figure 5.9: Predictions of Average Crack Spacing for Conventional and Modified Pavement Sections
5.3.5 TSRST Model

An alternative approach to predicting the low temperature cracking of Alaskan pavements using the data collected in this study is more site specific and utilizes statistical regression to establish a relationship between crack spacing ($S$, in meters) as a function of pavement age ($A$, in years), predicted minimum air temperature at the given site ($T$, 50% reliability, in °C), TSRST fracture temperature ($FT$, in °C), and TSRST fracture strength ($FS$, in kPa):

$$S = 990.01 - 125.92 (A)^{0.1} + 43.46 (T) + 0.5896 (T)^2 - 0.4423 (FT) - 0.024 (FS)$$  \hspace{1cm} (5.13)

It should be noted that the fracture temperature is obtained for field samples of the in-service pavements. A more useful relationship could be obtained if the fracture temperature ($FT_0$) of the original mix (unaged) is used instead of fracture temperature of field samples ($FT$). In this case, Equation 5.4 could be used to shift the measured $FT$ values to their original $FT_0$ and develop the corresponding relationship similar to Equation 5.13:

$$S = 994.11 - 127.69 (A)^{0.1} + 43.89 (T) + 0.5954 (T)^2 - 0.665 (FT_0) - 0.0249 (FS)$$  \hspace{1cm} (5.14)

The $R^2$ for Equations 5.13 and 5.14 is 0.853.

It is evident from Equations 5.13 and 5.14 that the minimum air temperature (50% reliability) obtained from the closest weather station to the site has the most significant influence on crack spacing. A reduction in fracture strength would render the pavement less brittle and would therefore be beneficial as it tends to increase the crack spacing. A comparison of predicted crack spacing using the proposed model with field measurements shows good agreement as illustrated in Figure 5.10. A summary of comparisons using all models considered in this study is presented in Table 5.5.

The proposed model was also applied to evaluate low temperature cracking performance with age for the various field sections used in this study. A comparison of crack progression with pavement age for the Anchorage and Fairbanks sections is presented in Figures 5.11 and 5.12 respectively. Predictions indicate that the use of polymer modification will definitely improve the low temperature cracking performance. Model predictions for the Anchorage sections indicate that the pavement at Elmendorf AFB (AC-5 + 3% SBS) and A-Street (rubberized, PlusRide) exhibit the best low temperature cracking performance. In Fairbanks, both Rewak Drive (AC-5 + 6% SBS) and Badger Road (SHRP Level 1 mix) should have the best low temperature cracking performance according to model predictions. The relatively poor performance of Elmendorf (AC-5+6% SBS) compared to Rewak Drive (AC-5+6%SBS) could be associated with warmer fracture temperature and higher fracture strength according to TSRST results.
Figure 5.10: Comparisons of Observed Cracking with Predictions Using Proposed TSRST Model
Table 5.5: Comparison of Predicted and Observed Low Temperature Cracking

<table>
<thead>
<tr>
<th>Project ID</th>
<th>Site</th>
<th>Observed 1996 Crack Spacing (m)</th>
<th>Superpave</th>
<th>AKDOT &amp; Haas 1987</th>
<th>Kanerva 1993</th>
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<tbody>
<tr>
<td>a</td>
<td>C Str., Anchorage</td>
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<td>11.0</td>
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<td>98.8</td>
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<td>-</td>
<td>121.2</td>
<td>160.0</td>
</tr>
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<td>Elmendorf 3% SBS</td>
<td>22.9</td>
<td>11.0</td>
<td>121.2</td>
<td>161.9</td>
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<td>Elmendorf 6% SBS</td>
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<td>-</td>
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<td>161.9</td>
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<td>5.9</td>
<td>-</td>
<td>9.8</td>
<td>55.9</td>
</tr>
<tr>
<td>h2</td>
<td>Parks AC5</td>
<td>20.2</td>
<td>-</td>
<td>9.8</td>
<td>59.1</td>
</tr>
<tr>
<td>h1</td>
<td>Parks PBA3</td>
<td>33.6</td>
<td>-</td>
<td>9.8</td>
<td>120.3</td>
</tr>
<tr>
<td>i</td>
<td>Badger Rd.</td>
<td>119.0</td>
<td>-</td>
<td>9.8</td>
<td>48.4</td>
</tr>
<tr>
<td>j</td>
<td>Eielson AFB</td>
<td>6.3</td>
<td>-</td>
<td>9.8</td>
<td>128.1</td>
</tr>
<tr>
<td>k</td>
<td>Ft. Wainwright</td>
<td>21.5</td>
<td>-</td>
<td>9.8</td>
<td>124.2</td>
</tr>
<tr>
<td>l</td>
<td>Rewak Dr.</td>
<td>50.6</td>
<td>11.0</td>
<td>9.8</td>
<td>117.1</td>
</tr>
<tr>
<td>m</td>
<td>Deadhorse</td>
<td>21.1</td>
<td>-</td>
<td>1.0</td>
<td>136.1</td>
</tr>
</tbody>
</table>
Figure 5.11: Cracking Prediction for the Anchorage Sections
Figure 5.12: Cracking Predictions for the Fairbanks Sections
5. 4 ECONOMIC CONSIDERATIONS

The decision of using polymer modified asphalts versus conventional asphalts depends on the expected improvement in low temperature cracking performance. Specifically, the cost-effectiveness of using polymer modified binders depends on the savings in maintenance costs associated with reduced thermal cracking, in addition to reduced damage on vehicles and reduced time for traffic closures. In this study, preliminary analyses were performed to ascertain potential improvement in low temperature cracking and corresponding savings resulting from the use of polymer modifiers. Three methods were used in this regard:

1. Compare low temperature cracking performance for polymer modified pavement sections and control sections with conventional asphalt layers and estimate corresponding savings in crack sealing maintenance costs.

2. Determine savings in maintenance costs comparing the "average" crack density of all polymer modified pavements with the "average" crack density of conventional pavements.

3. Determine savings in maintenance costs by comparing crack densities as predicted using the proposed TSRST model.

In all these methods, only the maintenance costs for crack sealing were considered. Initial capital costs, life-cycle costs, user costs, and other cost savings, that could result from improved performance of the polymer modified pavements were not used as they were outside the scope of this project. The maintenance cost for crack sealing was assumed to be proportional to the crack density (i.e. number of cracks/km). It should be noted in this case that the number of cracks per one lane (i.e. 4 m wide) was considered when determining relative costs or percent savings. The relative cost ratio (RC) is expressed as

\[
RC = \frac{C_m}{C_o} = \frac{D_m}{D_o}
\]

Where

- \(C_m, D_m\) = crack sealing cost and density for polymer modified section respectively
- \(C_o, D_o\) = crack sealing cost and density for conventional section, respectively

The corresponding savings in percent (SA) could be expressed as

\[
SA = (1 - RC)100
\]
5.4.1 Analysis Using Control Sections

Control sections for the field pavements surveyed include the following:

**Anchorage**
C Street (AC-5) to be compared with A Street (PlusRide)

**Fairbanks**
Danby Street (AC-2.5) to be compared with Danby Street (Asphalt-rubber)
Parks Highway (AC-5) to be compared with Parks Highway (PBA3)

Crack densities for these sections are shown in Figure 5.13. The corresponding savings for using polymer modified pavements vary between 6 percent for the Anchorage sections to 83 percent for the asphalt-rubber section on Danby Street in Fairbanks (Figure 5.14).

5.4.2 Analysis Using Average Crack Density

Comparison of observed crack densities for all polymer modified sections surveyed with conventional sections was also done by comparing the linear regression fits for the observed crack density with age (Figure 5.15).

For the conventional pavements,

\[ D_0 = 10.75 \text{ (A)} \quad (R^2 = 0.288) \]  

(5.17)

For polymer modified pavements,

\[ D_m = 7.52 \text{ (A)} \quad (R^2 = 0.320) \]  

(5.18)

If Equations 5.17 and 5.18 are substituted in Equation 5.16, then the average percent savings will be 30 percent.

In order to include the effect of age on average maintenance costs, an approach similar to that presented in Equations 5.9 - 5.12 was adopted to predict crack density rather than crack spacing. The variation of average density with age for the conventional pavements and polymer modified pavements is shown in Figure 5.16 and the corresponding savings are illustrated in Figure 5.17. Results indicate that savings in crack sealing cost seem to increase with age and could reach up to 80 percent.

5.4.3 Analysis Using TSRST Model

The proposed TSRST model (Equation 5.14) for predicting crack spacing with time was also used to estimate crack density with age for the field projects surveyed in Anchorage
Figure 5.13 : Comparison of Observed Crack Density for Modified Sections with Conventional Control Sections
Savings based on 1996 observed crack densities

Figure 5.14: Savings Estimate Based on Observed Crack Densities
Figure 5.15: Variation of Observed Crack Density with Age
Figure 5.16: Predicted Average Crack Density with Age for Conventional and Modified Pavements.
Figure 5.17: Predicted Crack Sealing Costs and Savings for Conventional and Modified Pavements
and Fairbanks. Predicted density for ages of 2, 5, and 10 years were then used to calculate percent savings using Equation 5.16.

For the Anchorage projects, variation of density for the different age periods considered is illustrated in Figures 5.18 - 5.20. The corresponding savings were determined using the performance of the C-Street conventional section as a reference (Figures 5.21 - 5.23). Results show that the use of polymer modifiers could yield savings in the range of 6 percent to 76 percent depending on the type of modifier used and pavement age. It is interesting to note that the difference in crack density between the conventional and modified sections increases with age thereby resulting in higher savings. The mix used at Elmendorf (AC-5 + 3%SBS) is predicted to have the best low temperature cracking performance and therefore the highest savings which could reach 76 percent for a pavement age of 10 years.

Similarly, the variation of predicted crack density of the field projects in Fairbanks for selected ages of 2, 5, and 10 years is presented in Figures 5.24 - 5.26. The corresponding savings were determined by using the average crack density for the conventional sections (Badger Road, Danby Street, Parks Highway) as a reference. Comparison of savings is shown in Figures 5.27 - 5.29. Savings in maintenance costs associated with using polymer modifiers range between 6 percent and 63 percent depending on type of modifier and age. The PBA 6 (AC-5 + 6% SBS) mix used in Rewak Drive results in maximum savings of 63 percent corresponding to expected cracking after 10 years.

5.5 SUMMARY

In this chapter, an attempt is made to determine the adequacy of available thermal cracking models to predict low temperature cracking of Alaskan pavements. A number of models in the published literature were used in addition to the Superpave TCMODEL. Comparison of low temperature cracking density between conventional sections and polymer modified sections was used to assess the economic benefits of reduced crack sealing maintenance costs. The main conclusions are summarized as follows:

1. Comparisons between predicted and observed low temperature cracking using available crack progression models, including Superpave TCMODEL were poor.

2. An improved regression model was developed using minimum air temperature, TSRST fracture temperature and strength, and pavement age to fit the observed field data for both conventional and polymer modified sections. This model is based on limited field data and should therefore be used to predict general trends of behavior and need to be upgraded as more data become available.

3. Preliminary assessment of reduced maintenance costs for crack sealing as a result of using polymer modifiers indicate that percent savings varies between 6 percent and 60 percent depending on type of modifier, location, and pavement age. When
Figure 5.18: Predicted 2-year Crack Density for Anchorage Sites
Figure 5.19: Predicted 5-year Crack Density for Anchorage Sites
Figure 5.20: Predicted 10-year Crack Density for Anchorage Sites
Figure 5.21: Predicted 2-year Savings for Anchorage Sites
Figure 5.22: Predicted 5-year Savings for Anchorage Sites
Figure 5.23: Predicted 10-year Savings for Anchorage Sites
Figure 5.24: Predicted 2-year Crack Density for Fairbanks Sites
Figure 5.25: Predicted 5-year Crack Density for Fairbanks Sites
Figure 5.26: Predicted 10-year Crack Density for Fairbanks Sites
Figure 5.27: Predicted 2-year Savings for Fairbanks Sites
Figure 5.28: Predicted 5-year Savings for Fairbanks Sites
Figure 5.29: Predicted 10-year Savings for Fairbanks Sites
the "average" performance of all conventional pavement sections was compared with the "average" performance of the polymer modified sections. The savings after about 5 years of service were estimated to be between 30 and 40 percent.

It should be emphasized that the above conclusions are based on limited data. These predictions need to be refined as more field and laboratory data on the low temperature behavior of conventional and polymer modified asphalt concrete pavements become available.
REFERENCES


CHAPTER SIX
SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

Asphalt modifiers have been used in Alaskan pavements over the past 15 years. These modifiers include SBR polymers, SBS polymers, Ultrapave, and CRM (both the dry process, PlusRide, and the wet process). Field observations and laboratory studies, performed in Alaska and elsewhere, indicate that the use of these modifiers would improve the low temperature cracking resistance of pavements. This project aimed at evaluating the degree of improvement these modifiers provide for Alaskan pavements, specifically, in relation to SHRP Superpave binder specifications, mix evaluation using Superpave TCMODEL and TSRST, and low temperature cracking models.

6.1 SUMMARY OF RESEARCH FINDINGS

1. The low temperature PG of conventional asphalts varied from -16 (AC-5 Badger Road) to -34 (AC-5, Haines Highway). For polymer modified binders, the low temperature grade varied from -22 (AC-5 + 3%SBS (Elmendorf), AC-5 +6% SBS (Rewak Drive)) to -40 (asphalt-rubber, Danby Street).

2. There is some indication that the PG system for binder specification tends to "mask" any beneficial effect that the polymer modifier may have on the low temperature grade. This is evidenced by comparing the specifications for conventional and polymer modified binders used on the same project (Parks Highway, Haines Highway, and Elmendorf). This "masking" effect, however, is not evidenced in the case of asphalt-rubber (Danby Street). On the other hand, the TSRST results clearly reflect the improved low temperature cracking resistance exhibited by the polymer modified mixtures. All polymer modified mixtures, except the AC-20R (Denali Park Road) and the AC-5 +6%SBS (Elmendorf), exhibited lower fracture temperatures (-31 °C to -35 °C) in comparison with the conventional unmodified mixtures (-24 °C to -28 °C).

3. Except for the Haines Highway project, the low temperature PG for all conventional unmodified binders and polymer modified binders was, in most cases, "warmer" than Superpave 50% reliability minimum air temperature, but in all cases, it was "warmer" than the 98% reliability low temperature limit. Similarly, except for the Haines Highway project, all TSRST fracture temperature values were warmer than the 98% reliability low temperature recommended by Superpave.

4. Field surveys indicated that polymer modified mixes exhibit in general better low temperature cracking performance than conventional unmodified mixes. All projects, except for the Haines Highway, showed evidence of low temperature cracking. The extent of cracking depended largely on site location, and type of binder and mix used, and seemed to be related to the difference between binder
low temperature grade and Superpave recommended 50% or 98% reliability grade. Observed low temperature cracking performance of the modified sections in comparison with conventional unmodified sections is summarized as follows:

a) The PlusRide section exhibits better low temperature cracking performance when compared with the conventional section. The average crack spacing after 10 years of service was 41 m in comparison with 29 m for the conventional section. The corresponding crack density was 29 cracks/km for the PlusRide section and 45 cracks/km for the conventional pavement.

b) The Haines Highway sections surveyed exhibited excellent low temperature cracking performance. No significant cracking was observed. In fact, both the polymer modified section and the conventional section showed no evidence of low temperature cracking after 2 years of age (1995 survey). However, during the following year (1996 survey) there were a total of 13 cracks in the modified section (3600 m) but only one crack in the conventional section (266 m). The average density of cracking for both sections was essentially equal to 4 cracks/km.

c) Both the conventional and the asphalt-rubber sections on Danby Street seemed to have reached a steady state in relation to the progression of low temperature cracking with time. The crack spacing in the conventional section ranged between 5 and 12 m with an average spacing of about 6 m and average crack density of 177 cracks/km. On the other hand, only 3 cracks were observed in the asphalt-rubber section with average spacing of 32 m and average crack density of 31 cracks/km. The use of asphalt-rubber in this case reduced the crack density by a factor of 6 approximately.

d) The improvement in low temperature cracking associated with using PBA 3 compared with the conventional AC-5 on the Parks Highway Projects corresponds to an estimated reduction in crack density of 25 percent after one year of service.

e) In order to illustrate the influence of polymer content on the binder-mix performance two sites were chosen in each of the Northern Region (Parks Highway, AC-5 + 3% SBS (PBA 3), 1995 and Rewak Drive, AC-5 + 6% SBS (PBA 6), 1993) and the Central Region (Elmendorf AFB, AC-5 + 3% SBS, 1995 and Elmendorf AFB, AC-5 + 6% SBS, 1991). In both the Northern Region and Central Region sites, the use of 6% SBS with AC-5 improved the low temperature cracking resistance in comparison with a 3% SBS modification. For example, the average crack spacing of the 5 year old pavement section at Elmendorf AFB with 6% SBS is essentially the same (23 m) as the section with 3% SBS after one year of service. Similarly, Rewak Drive in Fairbanks has 6% SBS and is 3 years old, but exhibits crack...
spacing of 51 m in comparison with 34 m spacing for the 3% SBS section of the Parks Highway after one year of service.

f) Field surveys indicate that Arctic Grade AC-2.5 seems to perform better than either AC-5 + 3% SBS or AC-5 + 6% SBS. In this case, comparing the Arctic Grade pavement at Ft. Wainwright with the AC-5 + 6% SBS section at Elmendorf shows that after 5 years of service, the two sections had essentially an average crack spacing of 27 m. However, climatic conditions are less severe at Elmendorf since the minimum air temperature (50% reliability) is -28°C whereas the minimum air temperature at Ft. Wainwright is -46°C.

g) There were only two rubberized pavement sections in this study. A-Street in Anchorage, which had a PlusRide surface, and Danby Street with an asphalt-rubber pavement. The asphalt-rubber exhibited the greatest resistance to low temperature cracking in comparison with other modified sections. At an age of 8 years, the asphalt-rubber section at Danby Street, under more severe climatic conditions, reached a steady state since no additional cracking was observed between Fall 1995 and Spring 1996. Although the PlusRide section in Anchorage had essentially similar crack spacing after 11 years of service, it did not reach steady state since the average crack spacing between 1995 and 1996 decreased from about 41 m to 33 m. It is expected that asphalt-rubber mixtures similar to those at Danby Street would perform better than the PlusRide used at A-Street under similar climatic conditions.

h) The asphalt-rubber section at Danby Street performed much better than Arctic Grade AC-5 used at Eielson AFB. In this case, the crack spacing at Eielson AFB reached about 6 m in comparison with 32 m at Danby.

5. Minimum air and pavement temperature correlations were developed using field data for selected sites covering Alaska's climatic zones. Results indicate that for the lower temperature range generally applicable for low temperature design considerations, the minimum pavement temperature is always higher than the minimum air temperature. The difference, though, depends on the climatic zone under consideration. For the Continental Zone, the difference is about 7°C higher than the minimum air temperature, whereas for the Maritime and Transitional Zones the difference is approximately 2°C and 4°C respectively.

6. Minimum air and pavement temperature correlations also indicate that Superpave's recommendation to use minimum air temperature as equal to the minimum pavement temperature is conservative for Alaskan conditions. On the other hand, the use of the Asphalt Institute recommended correlations could be unconservative.
7. Contour maps for Alaskan roads corresponding to 50% and 98% reliability minimum air and minimum pavement temperatures were developed. These maps could be easily used by AKDOT&PF design engineers in the selection of low temperature binder grade for a given location.

8. Comparisons between predicted and observed low temperature cracking models using available crack progression models, including Superpave TCMODEL were poor. Superpave TCMODEL analysis was performed using the software version currently available at the Asphalt Institute. The software is difficult to work with, and in many cases attempts to obtain thermal cracking predictions were not successful. The current software is not ready for practical applications and in many cases no predictions could be made. Comparisons with field observations show that the Superpave model predicts less crack spacing than currently observed and is therefore very conservative. An improved version of Superpave TCMODEL is currently under development at the University of Maryland. Predictions using Kanerva's TSRST model could have been improved if input values for fracture temperature and strength of the original mixtures were available.

9. A regression model was developed for predicting crack spacing with age for the observed field sections in this project. The model is expressed as a function of minimum air temperature, pavement age, and TSRST fracture temperature and strength.

10. Preliminary assessment of reduced crack sealing maintenance costs as a result of using polymer modifiers indicates that percent savings varies approximately between 6 percent and 60 percent depending on type of modifier, location, and pavement age. When the "average" performance of all conventional pavement sections was compared with "average" performance of the polymer modified sections, the savings after about 5 years of service were estimated between 30 and 40 percent. Total life-cycle costs and users' costs were not considered.

6.2 DESIGN RECOMMENDATIONS

Results of this study are used to propose the following design guidelines for low temperature cracking of Alaskan roads. These guidelines address two criteria: 1) Crack initiation and 2) Crack progression.

Crack Initiation

In order to minimize the probability for low temperature crack initiation, it is proposed that in the case of unmodified binders, the low temperature PG be selected based on the 98% reliability minimum pavement temperature and Superpave PG tests. In other words, the binder grade should be equal or lower than the 98% reliability minimum pavement temperature. Minimum pavement temperature could be determined using one of the following methods:
1) Use the minimum air temperature from the weather station (e.g., Superpave Weather Database) nearest to the pavement site and convert the minimum air temperature to minimum pavement temperature using Equations 4.5-4.8 as applicable.

Or

2) Use the developed contour maps (Appendix G) to estimate the minimum pavement temperature. Using the contour maps is simpler as it provides a quick means for estimating pavement temperature.

For polymer modified binders, it is recommended that fracture temperature determined from the TSRST testing be used in addition to Superpave standard PG Grading tests to assess the low temperature resistance of the polymer modified binder/mix. The use of TSRST as a supplementary or "proof" test is strongly recommended since the standard PG tests may "mask" the improved low temperature cracking resistance of polymer modified binders. The determination of minimum pavement temperature remains the same, as described above, for conventional binders.

**Crack Progression**

In many cases, the use of an appropriate binder with PG Grade that would satisfy 98% reliability minimum pavement temperature is not possible because of a number of limitations that could include cost, availability, site location, and other construction considerations. The design engineer may in this case choose a binder with a PG warmer than the 98% reliability minimum pavement temperature. The determination of how much low temperature cracking is expected after a given period of time or alternatively, the binder/mix properties required to limit thermal crack progression to an "acceptable" level for a given design period, would be very useful to the design engineer. These questions could be answered by using Equation 5.14 which utilizes TSRST fracture temperature and fracture strength to predict the progression of thermal cracking with time. It should be emphasized that Equation 5.14 is empirical and is based on limited field and laboratory data. Its use is recommended particularly in the absence of a reliable Superpave crack progression model or other crack progression models that fit Alaskan conditions.

**6.3 LIMITATIONS**

Although this study provides some insight on the low temperature cracking performance of conventional and polymer modified pavements in Alaska, the following limitations need to be considered as part of evaluating and applying the research findings:

1. Field studies including both temperature measurements at selected sites and field surveys of low temperature cracking were limited. Temperature data was collected and analyzed during December 1995 and January 1996. In addition, low
temperature cracking surveys were performed twice over a one year interval. The temperature data sites were different from crack observation sites.

2. Because of the limited number of field sections, their location, and the types and thicknesses of pavements, the derived equations for air-pavement temperature and for crack progression should be used to predict general trends of behavior and need to be upgraded as more data become available.

3. The present version of Superpave TCMODEL is very difficult to use and does not provide reliable predictions for thermal crack progression with time. An improved version is currently under development at the University of Maryland.

4. The economic benefits for using polymer modified asphalts were assessed in terms of reducing low temperature cracking. Other potential benefits associated with improved performance such as reduced rutting, increased fatigue life, and increased resistance to raveling were not included in the economic study due to lack of data for Alaskan conditions.

6.4 RESEARCH NEEDS

The following are recommended research needs that would complement the findings of this study:

1. More refined PG specifications need to be defined for Alaskan conditions. The grading system should be consistent with minimum pavement temperatures for the different climatic zones in Alaska. The results of this study clearly show that the use of a 6 degree interval for the low temperature PG grade could be too "coarse" for Alaskan conditions. The improved specifications should also consider the use of TSRST fracture temperature and strength in binder selection since the PG Grading tests seem to "mask" the beneficial effects of polymer modification.

2. Field temperature and low temperature cracking data should be monitored on a continuous basis to provide improved air-pavement correlations and crack progression models. This is particularly important since these models and correlations are empirical in nature and therefore require substantial data for reliable predictions.

3. More work needs to be done to evaluate the extent of low temperature cracking associated with embankment shrinkage. The effects of climatic conditions, and embankment materials and geometry on the development of these cracks need to be identified as they would influence the decision to use polymer modifiers.
4. Research on characterizing polymer modified mixes for use in Alaskan urban roads is needed to determine improved fatigue and rutting resistance in comparison with conventional mixes and to ascertain the cost-effectiveness of using polymer modifiers.