AN INTEGRATED MODEL TO ASSESS ASPHALT CEMENT QUALITY ON LOW TEMPERATURE PERFORMANCE AND LIFE CYCLE COST

By:

Susan Louise Tighe

A thesis
presented to the University of Waterloo
in fulfilment of the
thesis requirement for the degree of
Doctor of Philosophy
in
Civil Engineering

Waterloo, Ontario, Canada, 1999

National Library
of Canada

Acquisitions and
Bibliographic Services

395 Wellington Street
Ottawa ON K1A 0N4
Canada

Bibliothèque nationale
du Canada

Acquisitions et
services bibliographiques

395, rue Wellington
Ottawa ON K1A 0N4
Canada

Your file Votre référence

Our file Notre référence

0-612-51232-0

Canada

BORROWER'S PAGE

The University of Waterloo requires the signatures of all persons using or photocopying this thesis. Please sign below, and give address and date.

ABSTRACT

The performance of an asphalt pavement is strongly affected by the response of the asphalt cement in the mix formulation to the in-service condition. In Canada, one of the major obstacles to achieving long term pavement performance is low temperature cracking. The Strategic Highway Research Program (SHRP) Superpave methodology incorporates design requirement based on the low and high in-service temperatures. This research examines how existing practice can be incorporated into the requirements of the SHRP Superpave Performance Graded (PG) asphalts. As well, it examines how modified asphalts relate to the low temperature susceptibility criteria. The research involves the development of a framework for an integrated model which examines low temperature cracking and how it relates to pavement performance and life-cycle cost. The model was divided into four modules and focuses on asphalt pavements.

Module One focuses on the material characterization of the asphalt cement. This analysis examines the McLeod Penetration Viscosity Number (PVN) as a low temperature susceptibility indicator. The results show that PVN is a fingerprint as it remains constant over time. PVN also relates to the SHRP Superpave Performance Graded (PG) asphalts. A decrease in the minimum temperature corresponding to the PG asphalt is consistent with an increase in the PVN. As well, the PVN shows variation within a crude source as would be expected and it shows PVN is constant regardless of the calculation method. For modified asphalts, PVN is related to the minimum Superpave PG temperature. Based on the analysis for a small number of samples, indications are that PVN does not remain constant with time for the modified asphalts.

Module Two uses the material properties to predict low temperature cracking. The Canadian Airport Model and Hajek model is used to predict cracking on the C-LTPP and C-SHRP test sites. The predicted cracking is compared to the observed cracking. The analysis indicates that the Hajek model and Canadian Airport model show good correlation to the observed cracking. The thermal contraction coefficient is examined and found to be a very good indicator of low temperature cracking based on observed cracking.

Module Three involves predicting performance using the cracking prediction provided in the previous Module. Using roughness as the measure of pavement performance, the performance of the pavement is predicted. Low temperature cracking is related to roughness in terms of Riding Comfort Index (RCI). Various relationships which relate RCI to the International Roughness Index (IRI) is examined. The predicted IRI values is compared to the observed values using the C-LTPP test sites. The Canadian Airport Model is considered to be used as a starting point with recognition that it is conservative.

The life-cycle cost analysis framework is described in Module Four. In general, a formal life-cycle costing procedure is not carried out for most pavement designs in Canada. If a life-cycle cost is carried out, it is a deterministic analysis. This research recommends a probabilistic life-cycle cost procedure should be carried out. A lognormal distribution is determined to be the most appropriate distribution for pavement lift thickness. An extensive analysis of material costs is presented. The analysis indicates that a lognormal distribution in general is also the best fit distribution for material costs. Costs should be grouped according to the quantity as economies of scale have a significant impact on the magnitude and the standard deviation associated with pavement material costs.

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

ACKNOWLEDGEMENTS

I wish to express a sincere thank you to my supervisor Professor Ralph Haas. Thank you for guiding my career and helping me to see my potential. Your advice, dedication, humour and encouragement are greatly appreciated. I look forward to future endeavors.

I would also like to thank my co-supervisor, Professor Leo Rothenburg for his advice and constructive comments. A special thank you is also extended to Professor Park Reilly for his invaluable statistical advice and guidance. Thank you to the remainder of my examining committee, Professors Li Fu, Wei-Chau Xie, Richard Cockfield and Don MacLeod for participating in this research initiative.

I thank my colleague, Ms. Barbara Shannon, Imperial Oil Limited for sharing her knowledge and insight of asphalt behaviour.

I thank my friend, Mrs. Betty Brown, in the Transportation group for being so helpful. As well I would like to thank Ruba AlAssar and Nancy Button for their support and encouragement.

I thank my many colleagues at the Ontario Ministry of Transportation for their assistance and technical support.

I would also like to gratefully acknowledge with gratitude the funding that I received in support of my research:

- the Natural Sciences and Engineering Research Council of Canada, through a Postgraduate Scholarship
- the Transportation Association of Canada, through the generosity of the Emery family for the Emery-Lindsay scholarship
- Imperial Oil Limited, through the University Research Grant (URG)
- Canadian Strategic Highway Research Program C-SHRP, through a Graduate Research Grant
- the University of Waterloo, through the Provost/Faculty of Engineering Women's Incentive Fund Scholarship, and the Faculty of Engineering Graduate Scholarship

I am deeply indebted to my parents, Charles and Theresa Tighe, who gave me the love, self-confidence and moral support to undertake so many years of school. I will always be grateful.

I give my deepest thanks to my husband and best friend, Chris, without whom this work would not have been possible. His support, laughter, love and encouragement have allowed me to pursue my goal. Thank you, with all my love.

TABLE OF CONTENTS

AUTHOR'S DECLARATION II

BORROWER'S PAGE III

ABSTRACT IV

ACKNOWLEDGEMENTS VI

TABLE OF CONTENTS 1

LIST OF FIGURES 5

LIST OF TABLES 8

CHAPTER ONE INTRODUCTION 10

1.1 BACKGROUND 10

1.2 SUMMARY OF RESEARCH APPROACH 13

1.3 RESEARCH SCOPE 14

1.4 RESEARCH PROBLEM 15
1.4.1 Low Temperature Cracking 15
1.4.2 Uncertainty 16

1.5 RESEARCH OBJECTIVE AND ASSOCIATED TASKS 16

1.6 ORGANIZATION OF THESIS 17

CHAPTER TWO LITERATURE REVIEW 19

2.1 INTRODUCTION TO BASIC CONCEPTS 19

2.2 ASPHALT CEMENT BASICS 19

2.3 ASPHALT CEMENT CHARACTERIZATION 21
2.3.1 Consistency Testing 21
2.3.2 Static Load Test 22
2.3.3 Cyclic (Dynamic) Testing 23

2.4 ASPHALT TEMPERATURE SUSCEPTIBILITY 24
2.4.1 Characterization of Low Temperature Susceptibility of Asphalt Mixes 27

2.4.2 Low Temperature Cracking Models 29
2.4.3 PVN and Cracking 29
2.4.4 Hajek Model 30
2.4.5 Superpave Low Temperature Models 31

2.5 PAVEMENT PERFORMANCE 33

2.6 ASPHALT MODIFICATION 34
2.6.1 Modified Asphalts 35
2.6.2 Supply of Modified or Enhanced Asphalts 36

2.7 STRATEGIC HIGHWAY RESEARCH PROGRAM 37
2.7.1 SHRP Testing 37
2.7.2 SHRP Performance Variables 38
2.7.3 Long Term Performance 40

2.8 SUMMARY OF CHAPTER TWO 42

CHAPTER THREE RESEARCH METHODOLOGY 43

3.1 THE INTEGRATED MODEL 43

3.2 DATA SOURCES 45

3.3 ANALYSIS OF VARIANCE 47

3.4 PROBABILITY DISTRIBUTIONS 50

CHAPTER FOUR MODULE ONE : MATERIAL CHARACTERIZATION 52

4.1 ASPHALT BINDER SELECTION 52

4.2 PVN CALCULATIONS 55

4.3 IS PVN A LONG TERM FINGERPRINT? 64

4.4 SUMMARY OF ANOVA 66

4.5 DETERMINING POWER OF THE ANOVA 67
4.5.1 Approach to Power Analysis 69

4.6 PVN AND SUPERPAVE PG COMPARISON 73
4.6.1 PVN and Superpave PG Minimum Temperature Comparison Results 74

4.7 PVN AND CRUDE SOURCE 77

4.8 PVN AND MODIFIED ASPHALT 84

4.9 SUMMARY OF CHAPTER FOUR 88

CHAPTER FIVE MODULE TWO : LOW TEMPERATURE CRACKING 89

5.1 MAGNITUDE OF PVN AND HOW IT INFLUENCES CRACKING 89

5.2 MODULE TWO: PREDICTION OF CRACKING 91

5.3 MODEL ONE: HAJEK MODEL PREDICITON 92

5.4 MODEL TWO: CANADIAN AIRPORT MODEL PREDICITON 95

5.5 HAJEK MODEL PREDICTIONS FOR THE C-LTPP SITES 96

5.6 AIRPORT MODEL PREDICTIONS AND OBSERVED DATA 99
- 5.6.1 Thermal Contraction Coefficient Analysis Using C-LTPP Data 101
- 5.6.2 Thermal Contraction Coefficient Analysis Using C-SHRP Test Sites 108

5.7 THREE WAY COMPARISON 108

5.8 SUMMARY OF MODULE TWO 115

CHAPTER SIX MODULE THREE : PERFORMANCE PREDICTION 117

6.1 PAVEMENT DETERIORATION 117

6.2 RELATING LOW TEMPERATURE CRACKING TO ROUGHNESS 120

6.3 RELATING RCI/RCR TO IRI 122

6.4 DATA USED FOR MODULE THREE 122

6.5 ANALYSIS OF C-LTPP SECTIONS 123

6.6 SUMMARY OF FINDINGS 128

CHAPTER SEVEN MODULE FOUR : LIFE-CYCLE COSTING 131

7.1 LIFE-CYCLE COST EVALUATION 131

7.2 COST ESTIMATES 133

7.3 INFLUENCE OF LOW TEMPERATURE CRACKING ON LIFE-CYCLE COSTING 134

7.4 MODULE FOUR METHODOLOGY 136

7.5 THICKNESS VARIATION 136

7.6 COST VARIATION 140

7.7 LIFE-CYCLE COST AND MODIFIED ASPHALTS 143

7.8 TOWARD A LIFE-CYCLE COST ANALYSIS PACKAGE 148

CHAPTER EIGHT CONCLUSIONS AND RECOMMENDATIONS 151

8.1 OBJECTIVES 151

8.2 MODULE ONE: MATERIAL CHARACTERIZATION 152
  8.2.1 Conventional Asphalt Analysis 152
  8.2.2 Modified Asphalt Analysis 153

8.3 MODULE TWO: LOW TEMPERATURE CRACKING PREDICTION 153

8.4 MODULE THREE: PAVEMENT PERFORMANCE 154

8.5 LIFE-CYCLE COST ANALYSIS 155

8.6 RECOMMENDATIONS 155

REFERENCES **158**

APPENDIX A **166**

APPENDIX B **193**

APPENDIX C **207**

APPENDIX D **214**

LIST OF FIGURES

Figure 2.1	Time Delays Between Applied Stress/Strain	24
Figure 3.1	The Integrated Model Framework	44
Figure 3.2	Typical Lognormal Distribution	50
Figure 4.1	Summary of Data Needs For ANOVA	56
Figure 4.2	PVN Absolute Viscosity Calculation	57
Figure 4.3	Comparison of Three PVN Values Calculated Using Original and Aged Asphalt Cements	59
Figure 4.4	PVN Comparison for Typical Canadian Asphalt Cements	59
Figure 4.5	Verification of PVN Calculations on Original Asphalt Samples	63
Figure 4.6	Verification of PVN Calculations Using Absolute Viscosity	63
Figure 4.7	Verification of PVN Calculations Using Original and Aged Asphalt Samples	65
Figure 4.8	PVN Over Time	65
Figure 4.9	Operating Curve for ANOVA B	70
Figure 4.10	Operating Curve for ANOVA C	70
Figure 4.11	Operating Curve for ANOVA D	71
Figure 4.12	Operating Curve for ANOVA E	71
Figure 4.13	Operating Curve for ANOVA F	72
Figure 4.14	Average PVN Values For Superpave Performance Grades	76
Figure 4.15	Operating Curve for ANOVA 6 Crude Sources	79
Figure 4.16	Average PVN Values for Crude Source A	82
Figure 4.17	Average PVN Values for Crude Source B	82
Figure 4.18	Average PVN Values for Crude Source C	83
Figure 4.19	PVN For Selected Modified Asphalts Over Time	88
Figure 5.1	Overall Comparison of PVN Calculations	90

Figure 5.2 Modification of Heukelom's Version of Pfeiffer's and VanDoormal's Nomograph For Relationship between Penetration, Penetration Index and Base Temperature 93

Figure 5.3 Modification of Heukelom's and Klomp's Version of VanderPoel's Nomograph For Determining Modulus of Stiffness of Asphalt Cements 94

Figure 5.4 Operating Curve For Hajek Prediction and Observed Cracking 100

Figure 5.5 Influence of Thermal Coefficients on Transverse Crack Spacing 102

Figure 5.6 Transverse Crack Prediction Using Various Coefficients Compared to Observed Cracking 102

Figure 5.7 Operating Curve For Airport Model Prediction and Observed Cracking 107

Figure 5.8 Cracking Model Comparison For C-LTPP Sections 114

Figure 5.9 Cracking Model Comparison With Less Than 100 Cracks For C-LTPP Sections 114

Figure 5.10 Comparison of Cracking Between Airport and Hajek Model For C-LTPP Sections 116

Figure 5.11 Comparison of Cracking (<100 Cracks) Between Airport and Hajek Model 116

Figure 6.1 Schematic Illustration of Pavement Deterioration Prediction 119

Figure 6.2 IRI Roughness Scale 119

Figure 6.3 Factors and Interaction Which Can Affect Pavement Performance 121

Figure 6.4 Influence of Cracking on Pavement Life 121

Figure 6.5 Analysis Procedure Module Three 124

Figure 6.6 Relationship Between IRI and Observed Cracking 126

Figure 6.7 Relationship Between RCI and Observed Cracking 126

Figure 7.1 Example of Extended Pavement Life Due to Crack Treatment (rout and Seal) 135

Figure 7.2 Thickness Distribution Curves Based on C-LTPP Core Thicknesses 139

Figure 7.3 Cost Distributions Curves For Granular 144

Figure 7.4 Cost Distributions Curves For HL1, HL3 and HL4 145

Figure 7.5 Cost Distributions Curves For HL8, HDBC and DFC 146

Figure 7.6 Sample Input Sheet For Life-Cycle Cost Analysis 149
Figure 7.7 Sample Output Sheet For Life-Cycle Cost Analysis 150

LIST OF TABLES

Table 2.1 SHRP Superpave Tests 38
Table 2.2 Superpave Performance Parameters 39
Table 4.1 PVN Calculation Method Comparison Using All Asphalts (ANOVA A) 58
Table 4.2 PVN Calculation Method Comparison Using Typical Canadian Asphalt (ANOVA B) 60
Table 4.3 PVN Calculation Method Comparison Using Original Asphalt Only (ANOVA C) 61
Table 4.4 PVN Calculation Method Comparison Using Absolute Viscosity (ANOVA D) 61
Table 4.5 PVN Comparison Using Original (@ 135°C) and Aged (@60°C) (ANOVA E) 62
Table 4.6 PVN Over Time (ANOVA F) 66
Table 4.7 Summary of the Purpose and Result of ANOVA 66
Table 4.8 Power Calculation Method Comparison 68
Table 4.9 PVN Power Summary for ANOVA's 73
Table 4.10 ANOVA Summary For Superpave Comparison 74
Table 4.11 Summary of PG and PVN ANOVA 75
Table 4.12 Summary of Crude Source ANOVA 77
Table 4.13 ANOVA Summary for Superpave PG and PVN Crude Source Comparison 80
Table 4.14 PVN Crude Source Variability Associated with PG Asphalts 81
Table 4.15 PVN of Selected Modified Asphalt Cements 85
Table 4.16 Summary of PVN Values For Multiple Points as Simulated In The Laboratory 85
Table 4.17 Two Way ANOVA Comparison for Modified Asphalt 86
Table 4.18 Three and Four Way ANOVA Comparison for Modified Asphalt 87
Table 5.1 Low Temperature Cracking Using Hajek Prediction 97

Table 5.2 ANOVA Summary for Hajek Prediction and Observed Cracking 99

Table 5.3 Airport Transverse Cracking Prediction 103

Table 5.4 ANOVA Summary for Airport Prediction and Observed Cracking 105

Table 5.5 ANOVA for Airport Prediction and Observed Cracking (< 100 Cracks) 106

Table 5.6 ANOVA for Airport Prediction and Observed Cracking (< 100 Cracks) 108

Table 5.7 Comparison of Transverse Crack Prediction 109

Table 5.8 Comparison of Transverse Cracking Models with Observed Cracking 112

Table 6.1 Riding Comfort Rating 118

Table 6.2 IRI Comparison 125

Table 6.3 ANOVA for IRI Prediction and Observed IRI 127

Table 6.4 ANOVA for IRI Prediction and Observed IRI Less Than 10 Cracks 128

Table 6.5 Summary of IRI Comparison C-LTPP 129

Table 7.1 Best Fit Distributions For Thickness 138

Table 7.2 Summary of Best Fit Distributions Using All Values 141

Table 7.3 Summary of Most Appropriate Distributions 142

Table 7.4 Summary of Prices Based on Quantity Breakdowns 147

CHAPTER ONE INTRODUCTION

1.1 BACKGROUND

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

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

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

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

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

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

Modifiers or enhancers are combined with softer base asphalt to provide reduced stiffness at low temperatures. Based on laboratory studies it is known that when combined with modifiers, the chemical and physical properties dramatically change [King 88]. In particular, asphalt modifiers have been shown to reduce temperature susceptibility. Various laboratory tests have been designed to help understand these enhancers and they attempt to correlate these changes to performance. However, asphalt cement is a unique viscoelastic material. The addition of a modifier further complicates the structure.

Currently, modified products are still poorly understood scientifically and research efforts must be undertaken to better understand how the structure and properties of modified asphalt relate to performance. So, although modified asphalt technology has been in existence for over twenty years, it appears that there are very few modifications, which have been evaluated in a comprehensive way. In particular, the modified asphalt technology has primarily assessed laboratory material performance and very little information is available on performance and life cycle costs related to these enhanced asphalt cements.

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

This thesis presents a mathematical model, which relates the laboratory material characterization of asphalts to pavement performance in terms of low temperature susceptibility and life cycle cost. The model developed is probabilistically based and quantifies the associated pavement performance with life-cycle cost prediction.

1.2 SUMMARY OF RESEARCH APPROACH

The research described herein predicts the performance and life-cycle cost of various asphalt pavements. Within this research methodology, various laboratory test results have been examined and assessed in terms of how they relate to field performance. Low temperature susceptibility indicators are analyzed extensively in terms of relevance and uncertainty with regard to field performance. Once the low temperature performance is determined, the roughness progression is estimated. To determine the life-cycle, the pavement is assessed in terms of maintenance and rehabilitation treatments and costs.

The research produces a framework for a probabilistic model of pavement performance and cost using six databases from both the private and public sectors in Canada. Information is combined to produce a mathematical model for use in Canada. The research extracts material characterization data (laboratory and field), cracking data, pavement factors such as subgrade and traffic levels, performance data such as roughness and cracking and cost data including both supplier and agency costs from the six databases. Some of the data has been used for model development while other data was used for verification. Probabilistic simulation is used to determine uncertainty associated with performance and cost of various asphalt pavements.

1.3 RESEARCH SCOPE

Pavement engineers must design, construct and maintain safe, long lasting and cost effective pavements. This thesis specifically addresses how to predict low temperature cracking of asphalt pavements. Based on the 100 billion dollar current in place asset value of Canadian pavements, it seems imperative that pavements be managed efficiently. In order to do this it is necessary to quantify variables such as the in-service environment and the life-cycle activities (i.e. initial construction, maintenance, rehabilitation, etc.), determine the variability associated with the prediction variables and predict performance accurately. Performance prediction is a challenge. However, it is imperative to good asset management. Moreover, even marginal improvements in managing pavements can result in large absolute savings [Haas 97].

Asphalt materials are very difficult to model and relate to performance. The new SHRP Superpave technology is intended to link asphalt characterization to field performance but there are still many deficiencies. Most public agencies have addressed surface distresses (i.e., raveling, flushing, rutting) through materials selection and mix design procedures. In Canada, there is particular interest in low temperature cracking of asphalt pavements as low temperature cracking markedly accelerates the loss of serviceability. This results in bumps, dips and potholes that result in increased pavement roughness.

Although modified asphalt technology has been in existence for over twenty years, as previously noted, it appears that there are few modifications that have been evaluated in a comprehensive way. To date, low temperature materials characterization of modified or enhanced asphalts has not been effectively related to low temperature cracking in the field. An integrated model is necessary to effect this linkage.

1.4 RESEARCH PROBLEM

This research problem focuses on low temperature cracking. Low temperature cracking is predicted based on asphalt cement qualities. Once the low temperature cracking is predicted, it is related to performance and life-cycle cost. The uncertainty associated with the asphalt cement testing, cracking prediction and costs are determined. Low temperature cracking issues and uncertainty are discussed herein.

1.4.1 Low Temperature Cracking

Fracture markedly accelerates the loss of serviceability in a pavement, thus resulting in bumps, dips and potholes. In 1965, behavior of flexible pavements and their components at low temperatures was judged by the Canadian Good Roads Association to be the highest priority road research need in Canada [Haas 69]. It is still today one of the first priority road research needs, as evidenced by the Canadian Strategic Highway Research Program (C-SHRP).

To properly manage a pavement network in Canada it is important that low temperature cracking can be predicted. Laboratory testing must indicate low temperature susceptibility so that proper materials and designs are selected for long term performance. The laboratory tests should accurately predict in-service cracking and in a timely manner (i.e. testing should be relatively accessible and not overly time consuming).

To estimate cracking, life-cycle performance and cost, various pavement variables need to be determined. This means examining material tests to see if they are accurate and reliable in predicting field performance. Maintenance plays an important role in long term performance. If cracks are sealed in a timely manner, environmental damage will be minimized and optimum performance can be achieved (i.e. the pavement reaches design life).

1.4.2 Uncertainty

Quantitative methods of modelling, analysis and evaluation are the tools of modern engineering. In practice, it is important to quantify the effect of uncertainty and evaluate its effect on performance and design [Ang 75]. Thus probability should be included in an engineering system. However, a "point estimate" or single value is often used as an input variable despite the importance of uncertainty. Using a point value, often a mean value, fails to account for the randomness of the phenomena or process and may result in a different conclusion than having used a probability distribution [Button 99].

Uncertainty in engineering prediction can be expressed in a number of ways including: uncertainty associated with randomness, namely variation in observed or measure values, and the frequency of those values, and uncertainty with respect to the inference space (i.e. regional construction variation), uncertainty associated with imperfect modelling and estimation and the possible omission of a variable based on limited data [Ang 75, Button 99].

It is apparent that when predicting pavement performance of a road, uncertainty must be considered so that the results are relevant to the "real world". By addressing uncertainty, the asphalt cement effect on low temperature cracking, and thus performance and life-cycle cost can be predicted more realistically.

1.5 RESEARCH OBJECTIVE AND ASSOCIATED TASKS

Based on the need for a comprehensive model, the overall intent of this research is to develop an integrated model, which can accommodate both conventional and modified or enhanced asphalts and relate them to design, performance and life-cycle cost. The material characterization would incorporate mix, aggregates, low temperature susceptibility, strength, deformation, and stiffness. The material characterization would then be linked to field performance.

The main objective of the research is to investigate the relationship between asphalt cement, low temperature performance and life-cycle cost. The methodology and model quantify the asphalt cement selection in terms of both technical and economic benefits. To achieve the main objective, the basic tasks required are summarized, in terms of modules, in the following discussion.

First, the low-temperature material characterization methods are assessed, and those properties, which are most influential in terms of low temperature performance are identified. The inputs for pavement structural design are also identified to ensure that the most appropriate design for cracking resistance is structurally adequate to transmit the designated traffic loads. The results from this task, comprising Module One are then used as inputs to the second module.

Module Two predicts low temperature cracking with respect to age based on the inputs from Module One. In this module, a cracking model was selected and is used to predict cracking. This prediction is verified using field performance data.

Module Three predicts performance in terms of roughness based on the predicted cracking from Module Two. Structural design for various "best practice" pavements are estimated and verified using field data. As part of this module, probabilistic maintenance and rehabilitation schedules are determined.

Module Four determines the probability distributions for the life-cycle costs of the respective asphalt pavements. The structural design costs have been assessed in a probabilistic manner and distributions for each material have been constructed using an extensive cost database. The distributions for pavement lift thickness are also determined. A framework for pavement life-cycle costing is presented.

1.6 ORGANIZATION OF THESIS

The thesis contains eight chapters as follows:

Chapter One: Introduction provides a brief background for the problem and approach to how the problem is solved.

Chapter Two: Literature Review describes the important concepts and approaches related to asphalt cement, low temperature susceptibility, performance and life-cycle cost.

Chapter Three: Research Methodology outlines the integrated model presented and the data sources used in the analysis. The fundamental probability concepts and framework are discussed.

Chapter Four: Module One: Material Characterization is developed in this chapter. Penetration Viscosity Number, PVN and how it relates to Superpave, how it is influenced over time and the variability associated with PVN is examined in detail. PVN and how it relates to modified asphalt is also presented.

Chapter Five: Module Two: Low Temperature Cracking describes the methods used to predict cracking. These methods are then statistically compared to the observed cracking on the C-LTPP test sites located throughout Canada. The thermal contraction coefficients are also examined.

Chapter Six: Module Three: Performance Prediction relates the thermal cracking obtained in Module Two to the roughness. The predictions are then compared to observed roughness on the C-LTPP test sections so that pavement performance can be predicted in a probabilistic manner.

Chapter Seven: Module Four: Life-Cycle Costing this chapter documents the probability distribution for lift thickness and material costs for carrying out a life cycle cost analysis. The framework for life-cycle costing is provided.

Chapter Eight: Conclusions and Recommendations summarizes the final conclusions and recommendations of the thesis.

CHAPTER TWO LITERATURE REVIEW

2.1 INTRODUCTION TO BASIC CONCEPTS

The purpose of this chapter is to review basic concepts and approaches to research related to asphalt cement, low temperature susceptibility, performance and life-cycle cost. A synopsis of information related to these areas is presented.

Although research has been carried out in these various areas, there are no models in existence which combine the variables. Based on the literature review herein it was also established that uncertainty has not typically been considered with respect to the asphalt cement qualities, low temperature susceptibility and life-cycle cost. This omission further emphasizes the important contribution that a probabilistic, integrated model makes to the research.

2.2 ASPHALT CEMENT BASICS

Scientists and engineers have determined that carbon and hydrogen are the principal elements in asphalt cements[1]. Although the chemical composition and physical properties vary from source to source, asphalt cement is typically composed of straight or branched chains (aliphatic or paraffinic), simple or complex saturated rings (naphthenic) and one or more stable six carbon condensed unsaturated rings. These various heteratoms molecules form functional groups or polar groups which in turn result in molecular interactions. These interactions strongly influence the physical and chemical properties of the asphalt cement. The three major functional groups most often identified within asphalt cement are asphaltenes, resins and oils [Roberts 91].

The asphalt cement is regarded as a colloidal system where the rheological properties are dictated by the composition of the constituents [Rozeveld 97]. Three asphalt cement characteristics that are important to performance include temperature susceptibility, viscoelasticity and aging.

---

[1] It is common to use the term binder or bitumen for asphalt cement particularly in the U.K., Australia, etc.. In other words, the terms are used interchangeably. As well, the term asphalt is commonly used in place of asphalt cement in North America. However, in some countries asphalt refers to the asphalt concrete mix or bituminous layer.

Asphalt is temperature susceptible as it is stiffer at low temperatures. Thus, when asphalt cement is tested in the laboratory it is essential that the temperature is specified to make the results meaningful. Similarly, asphalt cement behaviour is dependent on time loading, meaning that it is stiffer under a shorter loading time. This dependence of behaviour on temperature and loading time means the two factors can be used interchangeably. Slow loading rate can be interchanged as high temperature while fast loading rate by low temperatures. Asphalt cement is viscoelastic as it displays both viscous and elastic characteristics. At high temperatures (e.g. > 100°C), it acts as a viscous fluid, while at low temperature (e.g. < 0°C) it behaves like an elastic solid, rebounding to it's original shape when loaded and unloaded. At intermediate temperatures, it displays both elastic and viscous properties, making the behaviour more complex [Soleymani 97].

Aging or oxidation has a major effect on asphalt cement. Oxidation results in a more brittle structure. Heating also results in rapid oxidation and for this reason, it is important that temperatures be monitored during the construction process [SHRP 93]. Jongepier showed that in addition to temperature and loading time, a third factor can influence the behavior of asphalt cement. This third factor can show up as a stress, strain or rate of strain which is indicative of change or even failure of material during a test [Jongepier 69]. These nonlinear properties are most likely related to failure characteristics of asphalt cement [Soleymani 97].

Various researchers have analyzed the molecular formations and constituents of asphalt cements to understand the temperature susceptibility, visocoelasticity and oxidation process. Some examples of chemical analytical techniques on asphalt cements include: morphology and microstructure using various microscopes [Shin 96], differential scanning calorimetry to study asphalt crystallinity [Daly 96] and chromatography to determine rheology [Roberts 91], to name a few. Overall these techniques have the potential of characterizing the primary molecular components that influence rheological properties and aging characteristics related to pavement performance. However, in general, these techniques are expensive and have been used for research purposes only. They have not been related to field performance and it may be a long time until the results can be fully understood.

2.3 ASPHALT CEMENT CHARACTERIZATION

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

2.3.1 Consistency Testing

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

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

Viscosity is the ratio of shear stress to shear rate at any given temperature. Asphalt viscosity is a fundamental measure of the asphalt flowability that is not affected by changes in testing conditions (i.e., configuration of test instruments, geometry of sample). Viscosity is typically performed at 60°C, which is denoted as the absolute viscosity, and 135°C, which is denoted as the kinematic viscosity.

The absolute viscosity is defined as the resistance to flow of a fluid at 60°C and it approximates the maximum in-service summer pavement temperature. To determine absolute viscosity, a viscometer is mounted in a thermostatically controlled temperature and is charged with asphalt cement. A partial vacuum is applied and the flow is timed in seconds and then multiplied by a calibration factor (unit in poise) [Roberts 96].

The kinematic viscosity describes high temperature behavior and essentially characterizes the flow behavior under gravity. The asphalt cement flows through a capillary tube under gravitational force and approximates the mixing and lay down temperatures during pavement construction. The kinematic viscosity is measured in centistokes because the gravitational forces induce flow and density of the material affects the rate of flow [Roberts 96].

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

2.3.2 Static Load Test

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

The moduli from the loading modes are calculated to characterize the material behavior as a function of loading time [Soleymani 97]. The creep mode is the simplest and most convenient. Schweyer [Schweyer 74] introduced various types of viscometers to examine creep behavior.

These include: rotational type (utilizes coaxial cylinders or cone and plate) and a specialized capillary type in which a piston is used to drive the asphalt cement through a capillary tube [Soleymani 97]. The bending beam rheometer developed by SHRP is a creep loading test [Bahia 92].

A constant stress creep test result exhibits three stages [Tia 87]. In the first stage, the initial elastic response can be used to calculate the elastic modulus E_0. The second stage is denoted as the creep portion which becomes linear, resulting in a constant slope (ε_{cr}) or velocity. This slope is the strain or shear rate (ε or γ) corresponding to a given stress and test temperature. Once the applied stress is released, an immediate recovery of elastic strain occurs followed by the gradual, time dependent recovery of elastic strain ($\varepsilon_{recovered}$, delayed elastic strain). The residual strain that exists after complete elastic recovery is the nonrecoverable strain or permanent deformation (creep strain) [Tia 87, Ruth 96].

To analyze the creep test data, the stress and strain development at a specified temperature are examined. Note that at the lowest temperature T1, the viscosity is very high resulting in low creep rates (ε_{cr}) and high stress (σ). The stiffness modulus is then defined by:

$$S(t) = \frac{\sigma_o}{\varepsilon(t)} \tag{2.1}$$

where $S(t)$ = time dependent modulus of stiffness
t = loading time
σ_o = applied uniaxial stress
$\varepsilon(t)$ = resulting uniaxial strain at time t

Overall, the creep test can be used to effectively calculate the stresses and strains developed during cooling of a bituminous pavement [Ruth 96].

2.3.3 Cyclic (Dynamic) Testing

In cyclic (dynamic) testing, a sinusoidal stress is applied to a specimen and the resulting strain is monitored as a function of frequency. Strain controlled testing (sinusoidal varying stress is applied and strain is measured) is more common than stress controlled testing. The primary responses in dynamic testing are the complex modulus (G*) and the phase angle (δ). G* is a measure of the asphalt cement resistance to deformation while δ is the time delay or phase angle between the applied stress/strain, and the response stress/strain as depicted in Figure 2.1 [SHRP 95].

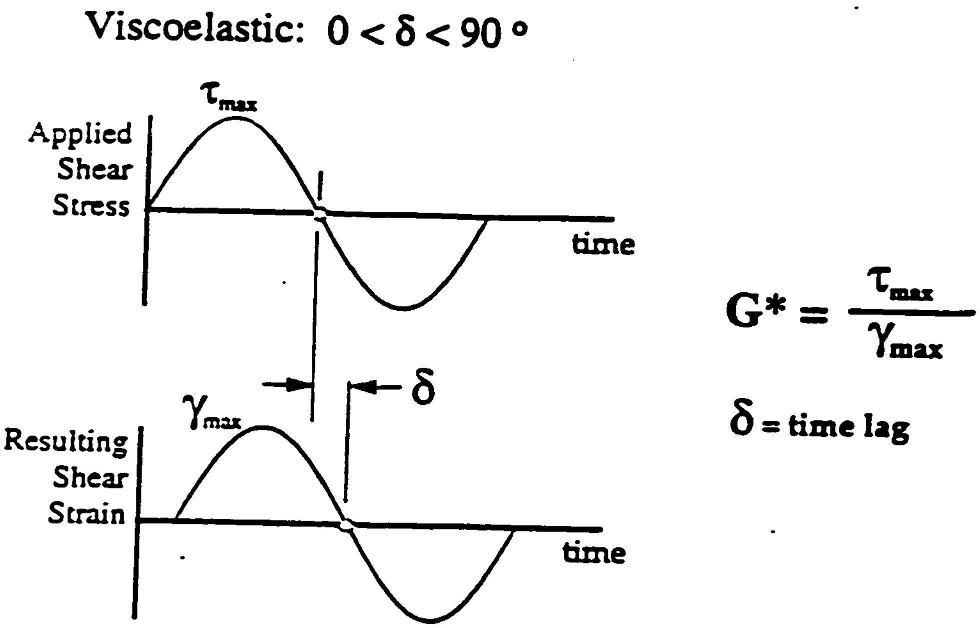


Figure 2.1/ Time Delays Between Applied Stress/ Strain [SHRP 95]

2.4 ASPHALT TEMPERATURE SUSCEPTIBILITY

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

1. Either a microcrack initiates at the surface when the temperature induced stresses exceed the tensile strength (due to decreasing temperature) and the crack propagates only to some limited depth because of varying stiffness gradient.

2. Or the crack propagates to full depth of the surface upon one or more cycles of sudden warming due to the creation of stress imbalances. Continued warming (in spring) leads to firstly open, visually apparent cracks which then close with further warming (late spring) [Haas 87].

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

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

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

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

$$PVN = -1.5 \frac{[L - \log X]}{[L - M]} \qquad (2.2)$$

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

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

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

$$PI = [30/(1+90B)] - 10 \qquad (2.3)$$

where:

$$B = [2.9031 - \log(Pen25)]/(T_{R-B} - 77) \qquad (2.4)$$

Pen 25 = the penetration at 25°C

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

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

$$PI = (20 - 500A)/(1 + 50A) \qquad (2.5)$$

where

$$A = (\log \text{Pen at } T_1 - \log \text{Pen at } T_2)/(T_1 - T_2) \qquad (2.6)$$

T_1, T_2 = temperatures in °C.

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

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

$$VTS = \frac{\log\log(\text{viscosity at } T_2) - \log\log(\text{viscosity at } T_1)}{\log T_1 - \log T_2} \qquad (2.7)$$

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

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

2.4.1 Characterization of Low Temperature Susceptibility of Asphalt Mixes

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

Stiffness is a fundamental property in the evaluation of low temperature response. Indirect methods are termed so because they estimate stiffness without direct laboratory measurements (using routine index test data and transforming it into stiffness values) while direct methods for measuring stiffness include uni-axial tension testing [Haas 73]. The two methods commonly used to estimate thermal fracture in an asphalt layer are described herein. The first method calculates stress in a long completely restrained strip [Hills 66] or an extension which incorporates both temperature and stiffness gradients through the depth of the bituminous layer [Haas 69]. Both of these methods assume the bituminous layer is completely restrained and free of cracks. The data acquired from the constant rate of extension tests is than used to estimate the fracture temperature, T_{FR}, as follows [Seddick 95]:

1) Values of stiffness modulus are determined at each temperature at a certain strain rate.

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

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

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

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

2.4.2 Low Temperature Cracking Models

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

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

2.4.3 PVN and Cracking

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

Transverse cracking frequency = f(asphalt concrete layer thickness, minimum design temperature, stiffness modulus increase between 0°C and −17°C) and PVN) (2.8)

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

2.4.4 Hajek Model

Hajek [Hajek 71] identified five independent variables which influence low temperature cracking as follows: stiffness of original asphalt cement, in kg/cm², according to McLeod's method, winter design temperature, °C; age of pavement, in years; thickness of bituminous layer, in inches and subgrade soil type. This is summarized in functional form as follows:

$$I = f(S, t, a, m, d) \tag{2.9}$$

where
I = cracking index (transverse cracks per 500/ft)
S = Stiffness of original asphalt cement, kg/cm² determined for loading time of 20, 000 seconds, winter design temperature (°C)
t = combined thickness of bituminous layers, inches
a = age of pavement, years
m = winter design temperature, °C
d = subgrade type (dimensionless code 1-sand 2-loam 3-clay)

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

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

2.4.5 Superpave Low Temperature Models

The non load related material characterization SUPERPAVE model utilizes laboratory test data on asphalt mix samples to determine tensile strength and master relaxation curves [Witczak 97]. All other properties are either assumed (default) or estimated using empirical relations. The tensile strength obtained from the low temperature indirect tension test is defined as the stress at which the difference between the vertical and horizontal diametric deviation reach a maximum. The relaxation master curve is obtained from the low temperature indirect tensile creep test assuming viscoelastic material behavior. Bending beam rheometer (BBR) data for the binder is used to supplement the mixture requirements. Prony series expansion are fitted to all compliance curves and then shifted to form a master curve at a reference temperature of –20°C. The master compliance curve is converted to a master relaxation curve via a Laplace transformation [Witczak 97]. The crack extension is modeled using the Paris law as expressed in equation 2.10.

$$\Delta C = A(\Delta K_I)^n \tag{2.10}$$

where ΔC = the increment of crack extension

ΔK_I = the increment in the stress intensity factor

A = fracture parameter of the mix which is estimated from an empirical expression developed by Molenaar [Molenaar 83]

n = fracture parameter of the mix which is obtained from slope m of the log creep compliance curve $(n = 0.8 * (1 + 1/m))$ (2.11)

$\text{Log } A = 4.389 - 2.52 * \log(E * \sigma_m * n)$ (2.12)

E = relaxation modulus of the mixture (kPa)

σ_m = mixture tensile strength (kPa)

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

where $A = 4.389 - 2.52 * \log(k * \sigma_m * n)$ (2.13)

k = field calibration set to 10,000

σ_m = mixture tensile strength (kPa)

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

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

1. The coefficients of thermal expansion α calculated in the Superpave program were found to be too high by approximately one order of magnitude.

2. Variation of α with temperature is ignored, which contrasts with known test results that show it is not constant at low temperatures.

3. Although tensile strength is measured at -20°C, -10°C and 0°C the thermal cracking model to estimate the A and n fracture parameters, uses –10°C.

4. The empirically derived A and n parameters in the Paris crack extension law are assumed constant with respect to temperature. In reality, they are expected to vary and these empirical relationships may not be applicable for modified mixes.

Additionally, a separate crack initiation phase is not included in the low temperature cracking model; and field calibration was based on material properties measured from field aged cores.

Based on this field aged core measurement, the calibration constants could change if material properties were measured from laboratory specimens and or production cores [Witczak 97].

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

2.5 PAVEMENT PERFORMANCE

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

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

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

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

2.6 ASPHALT MODIFICATION

Additives and modifiers in hot mix asphalt have been in use for over 50 years [Roberts 96]. The purpose of an additive and or modifier is to improve the binder and mix properties and the overall performance of the asphalt [Haas 83]. Although additives and modifiers are not required for all binders, there are some specific technical benefits. Some of these benefits include:

1. Ability to obtain stiffer mixtures at high in service temperatures to minimize rutting.
2. Ability to obtain softer mixtures at low in service temperatures to minimize nonload associated thermal cracking.
3. Improved fatigue resistance.
4. Improved asphalt aggregate bonding to reduce stripping or moisture susceptibility.
5. Minimize tender mix problems during construction.
6. Improved abrasion resistance of mixture to reduce raveling.
7. Rejuvenate aged binders.
8. Reduce flushing or bleeding.
9. Improve resistance to aging or oxidation.
10. Improve overall performance.
11. Reduce life cycle costs

In terms of modification of asphalt binders, many laboratory results indicate these modifications can lower stiffness at high construction temperatures to enable mixing and compacting but also reduce rutting in-service. Lower stiffness values can also be achieved to reduce thermal cracking and provide adhesion between the asphalt binder and aggregate in the presence of moisture to reduce stripping [Roberts 96]. These benefits are economically and technically desirable especially in light of the recently developed SHRP program where the asphalt binder is required to meet stiffness requirements at both high and low in-service temperatures. Other contributing factors that have lead to emphasis on modification include high traffic volumes, which can result in premature rutting and fatigue cracking, and low temperature thermal cracking.

In addition, public agencies are increasingly carrying out life-cycle cost analysis of pavements. As demonstrated in a recent study [Tighe 97], modifications can provide significant costs savings for agencies particularly on high volume facilities. Clearly, costs and performance of asphalt modifications must be carefully evaluated. However, environmental effects, energy requirements, availability of modifiers and recyclability are also important factors in evaluating effectiveness [Haas 83].

2.6.1 Modified Asphalts

A softer asphalt cement grade is typically selected for the modification process. Softer asphalt cements are selected as they are able to relieve stresses at lower temperature, have higher failure strains and a greater ability to promote healing of microcracks at high temperatures [Lee 95]. The softer binders are also typically selected so that the stiffness temperature is below that of the coldest in-service temperature and sufficient polymer is added to give the high temperature characteristics needed to resist deformation [King 88]. The following are examples of polymer modified asphalts currently supplied in Canada. There are many types of modifications available, however.

Unstable polyethylene systems have been industrially produced since the late 1970's under the tradename Novophalt. The system is unstable as the polyethylene emulsion tends to phase separate after mixing. This phase separation occurs in the form of a thick viscous layer on the surface within minutes after mixing. The problem can be rectified through continuous agitation.

Stabilized polyethylene systems graft a polybutadiene polymer on the surface of an individual polyethylene particle. This results in a stabilization of the polyethylene emulsion. The process is sold under the tradename Polyphalt.

Another method involves chlorinated polyethylene systems. In this case a known amount of the chlorinated polyethylene is added to the system under high shear so that no discernible phase separation occurs. Dissolved polyethylene systems add a small amount of asphaltene to the asphalt cement at high temperature which result in a stabilized system [Lee 95].

These are a few examples of the polymer modified mixes available commercially. Many other mixes are proprietary and it is thus difficult to assess them even on a general basis.

2.6.2 Supply of Modified or Enhanced Asphalts

The Canadian Strategic Highway Research Program (C-SHRP) initiated a survey in 1993 to assess the availability of various enhanced asphalts [C-SHRP 95, C-SHRP 95a]. Based on this survey the following observations were made. Only enhanced binders would be suitable for southern Ontario based on the low and high temperatures. Although these products were available for Toronto and Montreal, they would not be suited for the northern Ontario sections where very low temperatures are experienced during the winter months. There was also a serious lack of suitable products available for western Canada. These results were very disturbing as agencies were examining the feasibility of implementing the SHRP specifications which would require a range of polymer products in many paving projects.

Fortunately, since 1993, a number of asphalt suppliers have started supplying new and different modified asphalts as a result of the anticipated SHRP implementation. Although many suppliers will be unable to supply a range of modified asphalts based on limited storage facilities [C-SHRP

95b], public agencies and suppliers are working together in an attempt to ensure that the changeover to SHRP and use of enhanced asphalt is as smooth as possible.

2.7 STRATEGIC HIGHWAY RESEARCH PROGRAM

2.7.1 SHRP Testing

SHRP has enabled the pavement community to redefine the evaluation criteria for modified binders in terms of low temperature and high temperatures using newly defined measures of thermal susceptibility [Brule 95].

High temperature susceptibility is tested using the Dynamic Shear Rheometer (DSR) and the Rotational Viscometer (RV). Low temperature properties are determined using the Bending Beam Rheometer (BBR) and the Direct Tension Tester (DTT). Aging of the asphalt binders is measured by using these tests on the Rolling Thin Film Oven (RTFO) residue and the Pressurized Aging Vessel (PAV) residue [SHRP 95]. RTFO is intended to simulate aging of the binder as it goes through the hot mix plant and is placed, while PAV simulates aging after several years in-service. Table 2.1 provides a summary of the SHRP Superpave Tests.

In terms of enhanced asphalt characterization, lower slopes of G* (DSR) and lower phase angles (δ) have been obtained (see Figure 2.1). This means that the $G^*/\sin\delta$ should indicate increased resistance to rutting with the polymer modified asphalts [Maccarrone 94]. Some other observations based on the SHRP testing include reduced fatigue cracking as demonstrated by $G^*\sin\delta$ [Ponniah 95, Bahia 96].

Overall, these tests have enabled modified products to be examined in the laboratory and provided a basis for the SHRP Performance Grade (PG) specification. However, researchers continue to improve the test methods and make appropriate revisions so that modified asphalts can be evaluated accurately with good repeatability.

Table 2.1 SHRP Superpave Tests [SHRP 95a]

EQUIPMENT	PURPOSE
Rolling Thin Film Oven (RTFO) Pressurized Aging Vessel (PAV)	Simulate binder aging (hardening)characteristics
Dynamic Shear Rheometer (DSR)	Measures binder properties at high and intermediate temperatures
Rotational Viscometer (RV)	Measures binder properties at high temperatures
Bending Beam Rheometer (BBR) Direct Tension Tester (DTT)	Measures binder properties at low temperatures

2.7.2 SHRP Performance Variables

SHRP PG has attempted to address performance by delivering two main products: (a) performance based asphalt binder specifications and performance based asphalt and (b) asphalt-aggregate mixture specification including mix design and analysis. The specifications have been developed to address pavement deformation, fatigue cracking, low temperature cracking, moisture sensitivity, aging and adhesion. The underlying rationale is that performance is influenced by asphalt binder and aggregate [Kandahl 96]. Essentially the concept is that pavement performance is based on calculating stresses and strains to predict cracking and rutting. The overall stresses and strains are then related to the natural performance characteristics [Uzan 96]. Table 2.2 summarizes the Superpave performance parameters for the binders.

The specifications relate to three stages of testing to evaluate performance. Stage one covers the transportation, storage and handling. Stage two simulates mix production and construction for specification by aging the binder (i.e. RTFO) while stage three simulates aging over a period of time (i.e., PAV) [SHRP 95].

Table 2.2/ Superpave Performance Parameters for Binders [SHRP 95a]

EQUIPMENT	PERFORMANCE PARAMETER
Rolling Thin Film Oven (RTFO)	Resistance to aging (durability) during construction
Pressurized Aging Vessel (PAV)	Resistance to aging (durability) during construction
Dynamic Shear Rheometer (DSR)	Resistance to permanent deformation (rutting) and fatigue cracking
Rotational Viscometer (RV)	Handling and pumping
Bending Beam Rheometer (BBR)	Resistance to thermal cracking
Direct Tension Tester (DTT)	Resistance to thermal cracking

Deformation is characterized by the complex shear modulus, G* and the phase angle, δ (see Figure 2.1) which are obtained by using the DSR. This test characterizes the elastic and viscous properties according to the loading time and temperature effects.

The Rotational Viscometer (RV) is used to determine the storing and handling properties. The Bending Beam Rheometer (BBR) evaluates low temperature performance through the creep stiffness, S and the logarithmic creep rate, m. While the Direct Tension Tester (DTT) also evaluates low temperature susceptibility by determining the tensile strain at failure, the m value in the BBR test has been determined to be the controlling factor. If the creep stiffness is too high, the asphalt will become brittle and crack. Thus a low value of S is desired while a high value of m is desirable. As stiffness (ratio of stress to strain) decreases, this leads to smaller tensile stresses and less chance of cracking.

In terms of selecting a PG asphalt, the design air temperatures are determined. The air temperatures are then converted to pavement temperatures and the expected high and low temperatures are determined. These temperatures are then used by the designer to specify the overall grade [SHRP 95a]. It has been noted that at fracture there are differences between air temperature and pavement temperature. It appears that the pavement temperature at fracture is related to both the subgrade temperature and air temperature and not only to air temperature [C-SHRP 95a]. However, in general good correlation between the C-SHRP low temperature performance criteria and the pavement fracture temperatures was observed in the field.

Other observations noted by C-SHRP include that limiting stiffness criteria is not mechanically correct based on the fact that most of the stresses that develop in the pavement accumulate above the fracture temperature where the binder stiffness is high. C-SHRP also suggests that the low temperature design based on the minimum air temperature is too conservative, especially when very low temperatures are experienced, as the pavement temperatures are warmer than those indicated by the SHRP assessment. Instead, the minimum expected pavement surface temperature would be more reliable for predicting cracking. The low temperature using the BBR correlated well with the fracture and provides a good correlation to thermal cracking [C-SHRP 95a].

An approximate general rule of 90 can be used as a guideline to determine if a modifier will be required based on the PG specification. If the low temperature (absolute value) and the high temperature for the design situation are added together and the total is greater than 90 then a modified asphalt will probably be required; however if the total is less than 90 then a conventional asphalt will probably be specified.

For example PG 64-22 (64 + 22 = 86) will probably not require a modifier
PG 64-28 (64 + 28 = 92) will likely require a modifier

where 64 represents the high temperature requirement of the specification, in °C and –22 and –28 represent the corresponding low temperature requirement, in °C.

2.7.3 Long Term Performance

There are very few data available that relate low temperature cracking to performance. As well, there is limited information on the long term performance of modified asphalt pavements. The only long term performance model described in the literature seems to be the Markov based ones developed by Tighe [Tighe 97]. This involved using expert opinion to predict the future state of the model element by examining the present state. The Markov models were developed for both rutting and PCI. The rutting model was verified by using actual field data while the PCI models were verified using OPAC 2000 (a new pavement design system) and actual field data.

Based on this study, the modified sections were shown to have the least amount of rutting. In terms of cracking, the PCI values based on the Markov models predicted that extended service lives could be achieved by using modified asphalts. It should be noted that the results were in most cases reasonable and consistent with the various laboratory results, and they provide good initial working models. However, clearly more data are required to further validate the models and it must be recognized that this modeling works best with a strong record of experience. Thus, while the results may reflect some degree of expectation regarding long term polymer modified asphalt performance, they form a good basis for further analysis [Tighe 97]. However, models relating modified asphalts at low temperatures to long term performance remain to be developed.

Another potential long term performance model is the Exxon MOEBIUS model which performs structural design based on modified mixes. It is based on a heavily instrumented and well documented road in the Netherlands [Eckman 87]. Specifically, the model uses traffic histograms to represent the distribution of traffic stress over time. It also uses mean monthly air temperature to estimate hourly temperature at various depths within the pavement. MOEBIUS estimates stresses and temperatures combined with the creep damage as measured in the laboratory to compare rut depth with time [Nahos 90]. Although it could possibly provide some insight into long-term performance, it is also a proprietary product, which may make it difficult to use in assessing performance.

In essence, there is little information to date on relating modified asphalt properties to long term performance. MOEBIUS may provide some insight into performance although assessing it may be difficult. The Markov modeling approach also provides some insight into long term performance; however, further validation is required.

2.8 SUMMARY OF CHAPTER TWO

Various important concepts related to the research problem are reviewed. Asphalt cement is a complex material used in the construction of roads. Material characterization of asphalt cement, low temperature cracking, SHRP PG asphalt methodology, modified asphalts and pavement performance are reviewed.

The intent of laboratory testing is to simulate in-service conditions. Consistency, static and cyclic tests are used to assess in-service asphalt performance. Various low temperature susceptibility values have been developed to examine how the material properties determined in the laboratory will predict performance. This research problem focusses on low temperature cracking. PVN, PI and VTS are examined in addition to the new test methods introduced by SHRP. The SHRP tests have been reviewed as well as the low temperature cracking models. Pavement performance and modified asphalts have also been presented. This information provides the basis for the integrated model subsequently described.

CHAPTER THREE RESEARCH METHODOLOGY

The purpose of this chapter is to outline the integrated model, document the data sources used to develop and verify the model and describe the major research tasks within each module. In addition, analysis of variance, probability distributions and Monte Carlo simulation are explained, as these concepts are fundamental to the framework of the model.

3.1 THE INTEGRATED MODEL

The model is composed of four modules. Module One discusses material characterization properties of asphalt cement. Module Two relates the asphalt properties to low temperature cracking while Module Three relates the low temperature cracking to pavement performance. The final Module, Module Four determines the life-cycle cost. The integrated model is outlined in Figure 3.1, while the major research tasks within each can be summarized as follows:

Module One: Material Characterization

1. Determine if PVN is a fingerprint for asphalt cements.
2. Determine the uncertainty associated with PVN and crude source.
3. Identify how PVN relates to Superpave Performance Grades.
4. Examine PVN and how it relates to enhanced asphalt.

Module Two: Low Temperature Cracking Prediction

1. Estimate low temperature cracking using PVN.
2. Verify Hajek cracking model.
3. Explore feasibility of updating the Hajek model.
4. Verify Canadian Airport cracking model.
5. Compare Hajek and Canadian Airport cracking models.

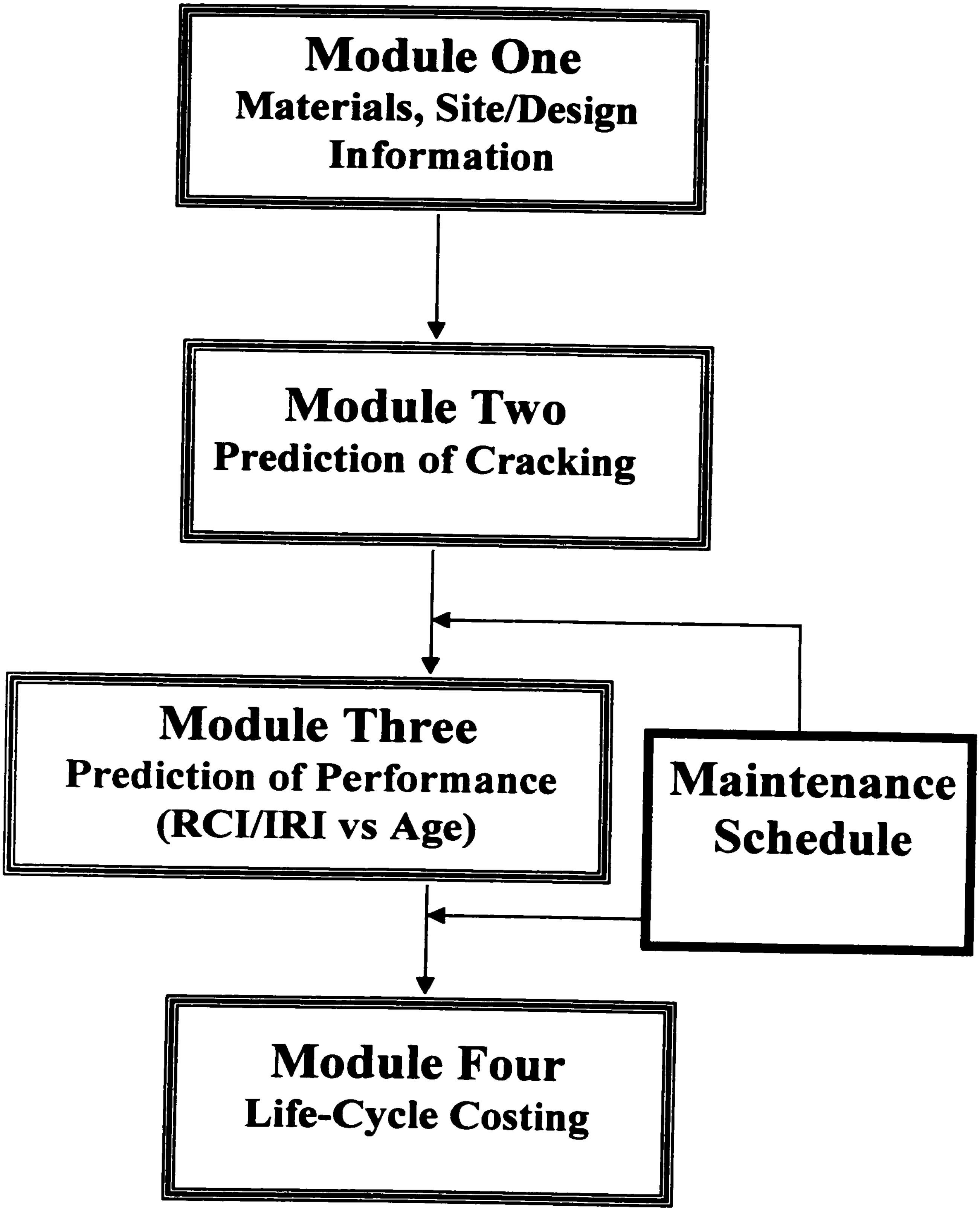


Figure 3.1/ The Integrated Model Framework

Module Three: Pavement Performance Prediction

1. Predict roughness associated with cracking.
2. Verify Canadian airport model.
3. Explore feasibility of a model update.

Module Four: Framework for Life-Cycle Cost

1. Determine thickness probability distributions.
2. Determine material cost probability distributions
3. Establish framework for life-cycle economic analysis.

3.2 DATA SOURCES

The databases used in this research to develop the integrated model have come from a number of sources. The three primary sources include: data obtained through research grants, data obtained through professional contacts and data obtained from the literature review. Some of the data was used to develop the model while some was used to verify the model. If the data was used for development, it was not used for verification and vice versa.

Two of the primary databases used to develop and verify the model came from research grants specifically related to this research project. Imperial Oil Limited (IOL) has supported much of the research in the form of a University Research Grant (URG). Through this grant an extensive database on material characterization was obtained. The database includes asphalt properties from around the world. It is extensive in terms of both the number of asphalt cement samples and the testing data made available. Both chemical and physical test results are available for various asphalt cements. This database was used primarily for the development of Module One. Due to the proprietary nature of the data, actual crude sources have not been identified. Additional cost data and cracking performance data from test sites was obtained from IOL

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

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

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

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

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

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

3.3 ANALYSIS OF VARIANCE

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

ANOVA procedures separate a portion of the variation observable in a response variable into two basic components: variations due to assignable causes and variation due to uncontrolled or random causes [Mason 89]. Assignable causes are generally known sources of variation that can be controlled or measured during an experiment. Random sources are all other sources of variation not controlled or measured. The ANOVA variability is measured as sum of squares. The total variability is referred to as the total sum of squares (TSS). For a single factor experiment, the variability is partitioned as an assignable cause (A) or as an error cause (E) as depicted in equation 3.1.

$$TSS = SS_A + SS_E \qquad (3.1)$$

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

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

In most cases, the experimenter wants to determine if the variation between the "treatment" means is significantly larger than the variation that occurs within treatments [Duever 98]. The Null Hypothesis (H_o) generally tests if the parameters corresponding to the main effects and interactions are zero. The complement, the Alternative Hypothesis (H_a) would suggest that there were differences in the parameters (they are not equal) for at least one pair of treatments. The H_o and H_a can be summarized as follows:

$$H_o: \quad \mu_1 = \mu_2 = \ldots = \mu_n \qquad (3.2)$$

$$H_a: \quad \mu_i \neq \mu_j \qquad (3.3)$$

where: H_o = Null Hypothesis
H_a = Alternative Hypothesis
n = Number of samples
$\mu_1, \mu_2\ \mu_n, \mu_i, \mu_j$ = Mean values for given experiment

The $F_{calculated}$ in the ANOVA is compared to the tabulated value of F, or $F_{critical}$ given by the degrees of freedom between and within the experiment at the pre-selected significance level (α). The significance level of the test represents the Type I error. Type I error is the chance of rejecting the null hypothesis when it is true. A 95% confidence level or $\alpha = 0.05$ is commonly specified.

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

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

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

3.4 PROBABILITY DISTRIBUTIONS

Results from an experiment can be described as discrete or continuous. Discrete variables are those that are finite while continuous variables can take on an infinite number of possible values. For a discrete variable, the probability of outcome is described as a summation over all possible values, while the relative frequency behaviour of continuous variables is called a probability density function [Scheaffer 82]. For continuous variables, once the random variable are obtained, probability distributions can be assigned to serve as models for various observed phenomena. If a distribution is continuous, it is then important to check symmetry and skewness of the distribution [Button 95].

For continuous variables the bounds of the distribution are extremely important. Most of the variables in this research are bounded by zero but have no upper limit. In addition they generally have one mode and are positively skewed with most values closest to the lower limit. When there is no sharp upper bound, a skew and a single mode, then a lognormal or gamma distribution are good candidate distributions [Button 99]. Lognormal distributions as shown in Figure 3.2, are used in most of this research based on best-fit principles.

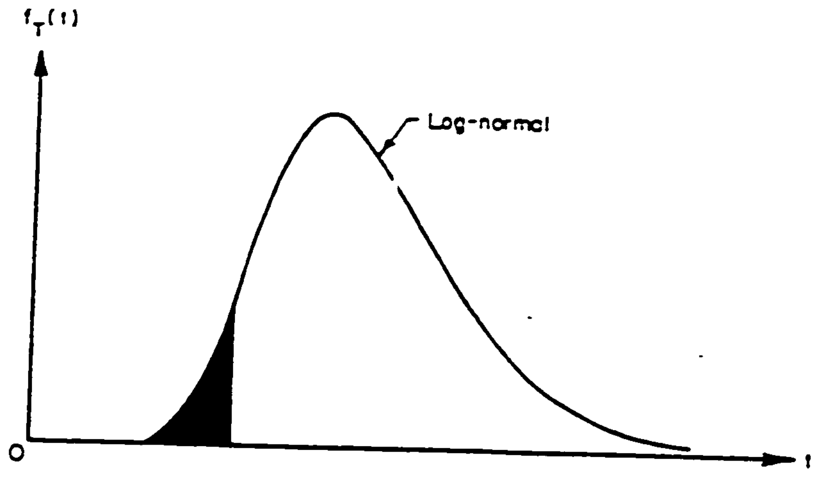


Figure 3.2 Typical Lognormal Distribution [Ang 75]

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

CHAPTER FOUR MODULE ONE : MATERIAL CHARACTERIZATION

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

4.1 ASPHALT BINDER SELECTION

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

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

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

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

Module One focuses on temperature susceptibility. The penetration at 25°C and the viscosity either at 60°C or at 135°C have been traditionally used to assess material properties. However, with SHRP Superpave design methodology, penetration would no longer be used and would be replaced with alternative measures. In any case, viscosity and penetration continue to be used as in-line tests in asphalt plants as a quality control indicator[2]. From these two material tests, the PVN can be calculated. If an asphalt cement has a PVN = 0.0, then it has a low temperature susceptibility while a PVN = -1.5 would mean the asphalt cement is highly temperature susceptible (i.e. high probability of low temperature cracking). Based on studies performed by McLeod [McLeod 75, 76], and an Iowa Department of Transportation (DOT) study [Marks 85], it appears that PVN is a good indicator of susceptibility. PVN has also shown to be a statistically significant variable in the progression of low temperature cracking on 22 Canadian Airports [Haas 87].

Based on this information, and the fact that many databases including the C-SHRP C-LTPP database do not include the SHRP test parameters, PVN was used to examine low temperature

---

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

susceptibility. PVN, which utilizes penetration, is considered an empirical measure. However, given that long-term performance data are not available for many in-service roads, and PVN appears to be a good measure, it was used to assess the material properties of the asphalt cements. One of the important assumptions is that PVN was developed using asphalts manufactured by steam and vacuum reduction processes and thus its applicability to all asphalt cements needed to be examined.

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

The purpose of this analysis is to determine:

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

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

4.2 PVN CALCULATIONS

This analysis examines whether the PVN values calculated on original or aged asphalt using either the kinematic viscosity (@ 135°C) and/or absolute viscosity (@ 60°C) are statistically different. For most of the asphalt test results available, both the kinematic and absolute viscosities on original asphalt cements were available. If an aged sample was available, then in most cases only the absolute viscosity was available. Six ANOVA's have been calculated for the initial PVN examination as shown in Figure 4.1. This figure details the data that were used to calculate the PVN's using a 95% confidence level. When the absolute viscosity was available, Figure 4.2 [McLeod 76] was used to determine the PVN.

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

$$PVN = -1.5 \frac{[L - \log X]}{[L - M]} \qquad (4.1)$$

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

Initially 45 asphalt cement samples[3] from the C-SHRP test roads and IOL database were used to test if the PVN calculated for original samples using kinematic viscosity and absolute viscosity and the PVN calculated using aged material (aged penetration and absolute viscosity of the sample) were statistically the same. Although, there are more than 45 samples in the combined database, there were only 45 samples where these three PVN values could be calculated. ANOVA A, Figure 4.1 was used to determine if the PVN values calculated using the asphalt data from the various laboratory tests at various times would be significantly different statistically. The null hypothesis for between asphalt cements was established as follows:

$$H_o : \quad PVN_{Asphalt\ 1} = PVN_{Asphalt\ 2} = \ldots = PVN_{Asphalt\ N} \qquad (4.2)$$

$$H_a : \quad PVN_{Asphalt\ i} \neq PVN_{Asphalt\ j} \qquad (4.3)$$

---

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

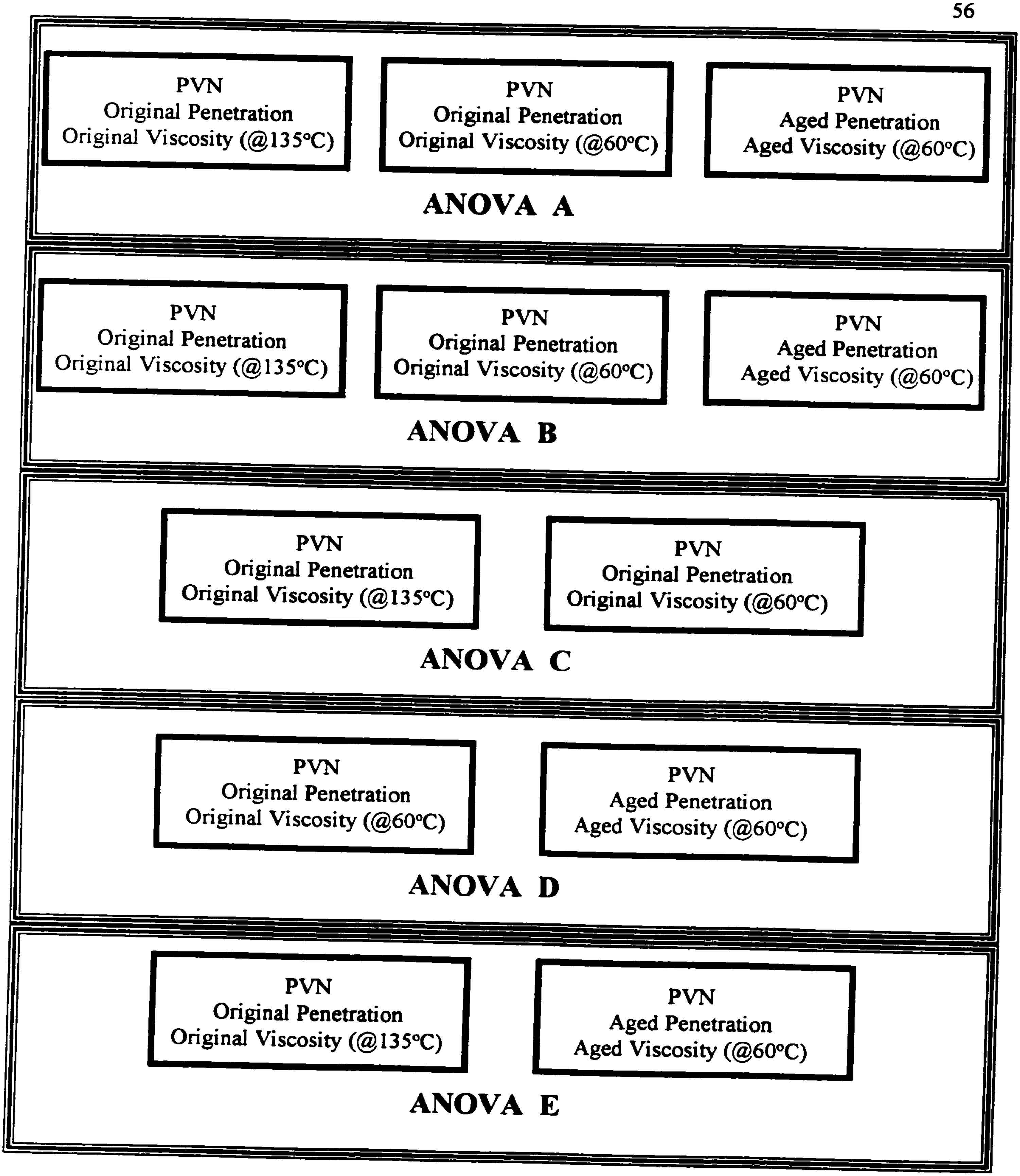


Figure 4.1 Summary of Data Needs For ANOVA

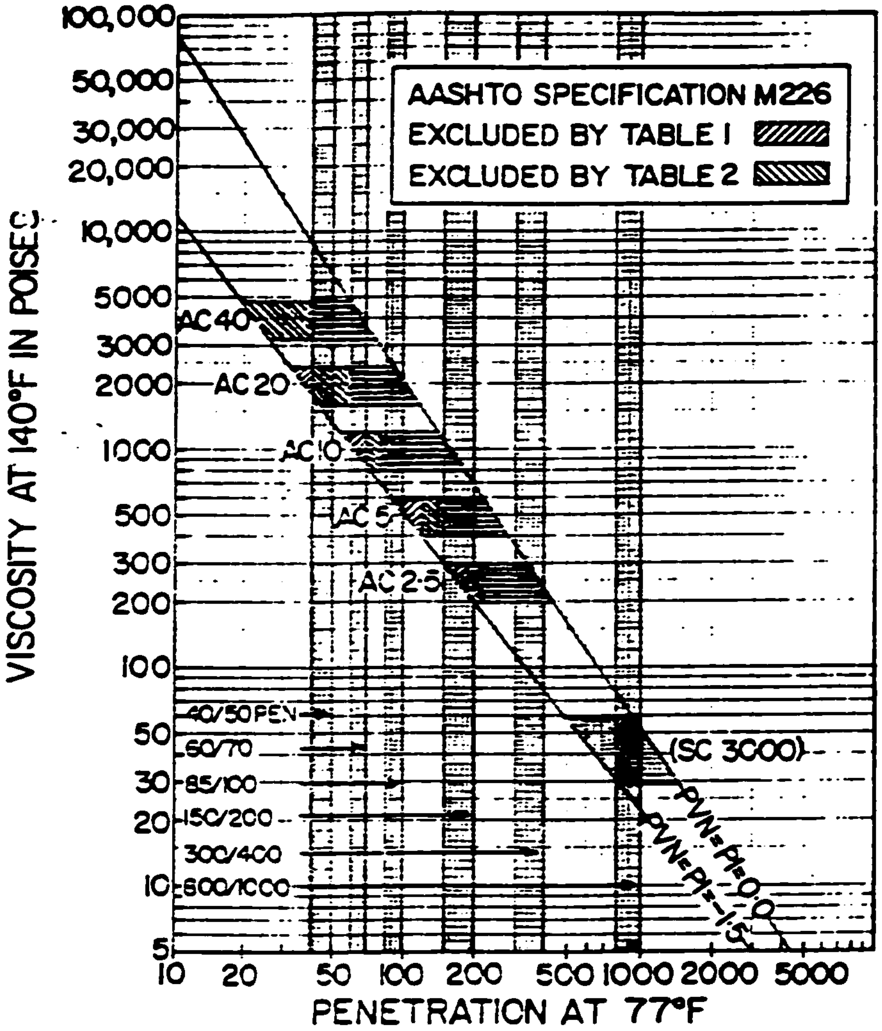


Figure 4.2 PVN Absolute Viscosity Calculation Chart [McLeod 76]

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

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

$$H_o: \quad PVN_{Asphalt\ 1} = PVN_{Asphalt\ 2} = PVN_{Asphalt\ n} \qquad (4.4)$$

$$H_a: \quad PVN_{Asphalt\ i} \neq PVN_{Asphalt j} \qquad (4.5)$$

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

$PVN_{Asphalt\ 2}$ = PVN(original penetration, original viscosity @60°C)
$PVN_{Asphalt\ 3}$ = PVN(aged penetration, aged viscosity @60°C)

Complete ANOVA results are presented in Appendix A.1 and are summarized in Table 4.1

Table 4.1 PVN Calculation Method Comparison Using All Asphalts (ANOVA A)

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

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

The results indicate that the PVN variation between the true value of the asphalt cement, or at least one of values are different ($F_{CALCULATED} > F_{CRITICAL}$). The low p-value indicates there is a small probability under the null hypothesis of having an F value more extreme than 3.9650 for comparison one and 4.2366 for comparison two. This result is consistent with the engineering experience that different asphalts would have different low temperature susceptibilities. Figure 4.3 shows a comparison of the PVN values calculated using the penetration and viscosity levels for the aged and original samples in cases where all three PVN's could be calculated. For some samples there are larger differences in the calculated PVN value, while in other cases they are similar.

The within asphalt result indicates that there is a significant difference between the PVN calculation methods. Upon further examination of the PVN values for the 45 asphalt cements, using the three laboratory samples, it would seem reasonable that the differences can be explained. The laboratory tests performed on the various asphalt cements were not necessarily tested or performed by the same person. Thus, some differences or errors due to random causes may not be properly assigned. However, the repeatability of penetration and viscosity testing is

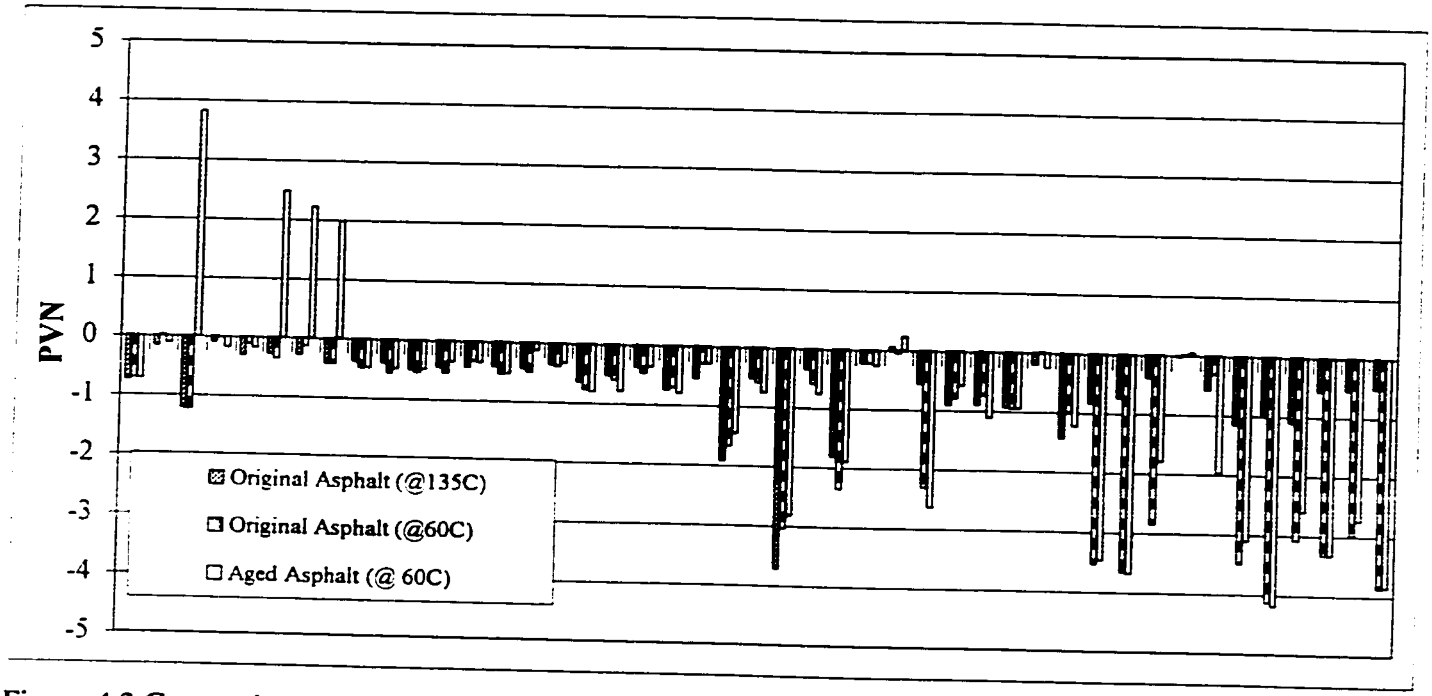


Figure 4.3 Comparison of Three PVN Values Calculated Using Original and Aged Asphalt Cements

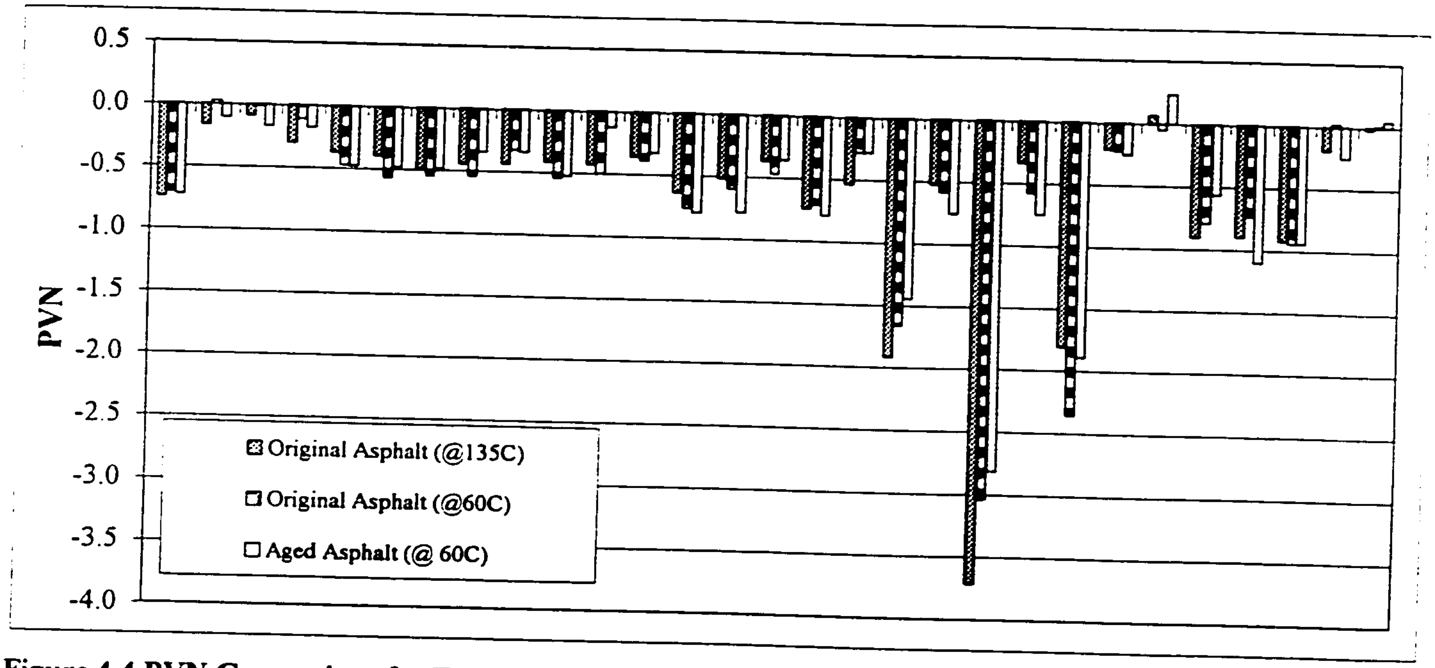


Figure 4.4 PVN Comparison for Typical Canadian Asphalt Cements

very good. In addition, some of the asphalts analyzed were not typical asphalts used in North America.

Based on these findings, the analysis was repeated using "typical" asphalt cements used in Canada (ANOVA B). The data was examined to check for unusual results that may be attributed to testing errors and/or data input errors. The results are presented in Table 4.2 and Appendix A.2.

Table 4.2 PVN Calculation Method Comparison Using Typical Canadian Asphalt (ANOVA B)

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

Notes: 1) Represents the difference between asphalt samples where the null hypothesis states all asphalts are equal.

2) Represents the differences in PVN values calculated for the same asphalt where the null hypothesis states regardless of how PVN is calculated the values will be the same.

In total, 29 asphalt samples were included in the ANOVA B analysis where the three calculated PVN values were compared. The first null hypothesis was rejected as the $F_{CALCULATED} > F_{CRITICAL}$ (55.4561 > 1.6775). This result was consistent with ANOVA A and is consistent with the theory of differences in low temperature susceptibility. Again, the p-value is very small indicating consistency.

The ANOVA for within variation associated with the three values of PVN calculation results in the acceptance of the null hypothesis as the $F_{CALCULATED} < F_{CRITICAL}$ (0.3643 < 3.1619). This indicates that it is not possible to find a difference which is significant at the 95% confidence level. Consequently, the PVN calculated using PVN (original penetration, original viscosity @135°C), PVN (original penetration, original viscosity @60°C) and PVN (aged penetration, aged viscosity @60°C) are statistically equal, for these 29 asphalts.

Figure 4.4 summarizes the PVN calculations using the three PVN values for the typical Canadian asphalts. Further to the ANOVA, PVN values calculated using the three sources of information give more consistent results as compared to those PVN values shown in Figure 4.3.

Based on the high p-value obtained in ANOVA A for within group variation, three additional ANOVA's were carried out, ANOVA C, ANOVA D and ANOVA E. Data from the Canadian asphalts analysis, ANOVA B plus additional points for asphalt samples where only two PVN values were available were included in the subsequent analysis.

ANOVA C compares the PVN calculated on original binder using only original asphalt data. Table 4.3 summarizes the ANOVA C for the original asphalts while the more complete analysis is located in Table A.3 in Appendix A. Tables 4.4 and 4.5 summarize the difference between PVN calculations using only absolute viscosities (ANOVA D) and PVN calculations using kinematic viscosities on original and absolute viscosities on aged asphalts (ANOVA E). Detailed results for these two evaluations are in Tables A.4 and A.5 in Appendix A.

Table 4.3 PVN Calculation Method Comparison Using Original Asphalt Only (ANOVA C)

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

Table 4.4 PVN Calculation Method Comparison Using Absolute Viscosity (ANOVA D)

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

Notes: 1) Represents the difference between asphalt samples where the null hypothesis states all asphalts are equal.

2) Represents the differences in PVN values calculated for the same asphalt where the null hypothesis states the PVN true values are the same.

Table 4.5 PVN Comparison Using Original (@135°C) and Aged (@60°C) (ANOVA E)

Comparison	Test	$F_{CALCULATED}$	$F_{CRITICAL}$	P-value	Degrees of Freedom
$PVN_{Asphalt\ 1}$ To $PVN_{Asphalt\ 57}$	Between[1] Asphalt	1.6124	1.5579	0.0383	56
$PVN_{Original}$ PVN_{Aged}	Within[2] Asphalt	1.0365	4.013	0.3130	1

Notes: 1) Represents the difference between asphalt samples where the null hypothesis states all asphalts are equal.

2) Represents the differences in PVN values calculated for the same asphalt where the null hypothesis states the true values of the PVN are the same.

In all three cases where two PVN values are compared, the null hypothesis is rejected for the between asphalt analysis. This is consistent with ANOVA A and B. With regard to the within asphalt test, in these three cases the null hypothesis is accepted, as the $F_{CALCULATED} < F_{CRITICAL}$. This indicates that the differences between the PVN calculations are not statistically significant or that regardless of what is used for the PVN calculation, the overall result is that the PVN would be statistically the same. Given that the analysis is carried out for original binder and binder which has been aged, there are two pertinent implications to this finding.

First, it indicates that PVN can be calculated with either absolute viscosity or kinematic viscosity and the results will be the same. The second implication is that the PVN remains constant over short term aging as simulated in the laboratory, as the differences between the original asphalt and aged sample are not statistically significant. Consequently, in terms of short term aging the PVN remains constant.

Figure 4.5 shows the relationship between the PVN calculated using original penetration values and original viscosity values at 135°C and 60°C. The line represents equality and ideally the points should be as close to the line as possible. Good correlation is evident for PVN values calculated on asphalts with PVN between about 0.0 and −1.0. There is more discrepancy associated with PVN values between −1.0 and −4.0. However, the discrepancy appears to be evenly distributed in this range.

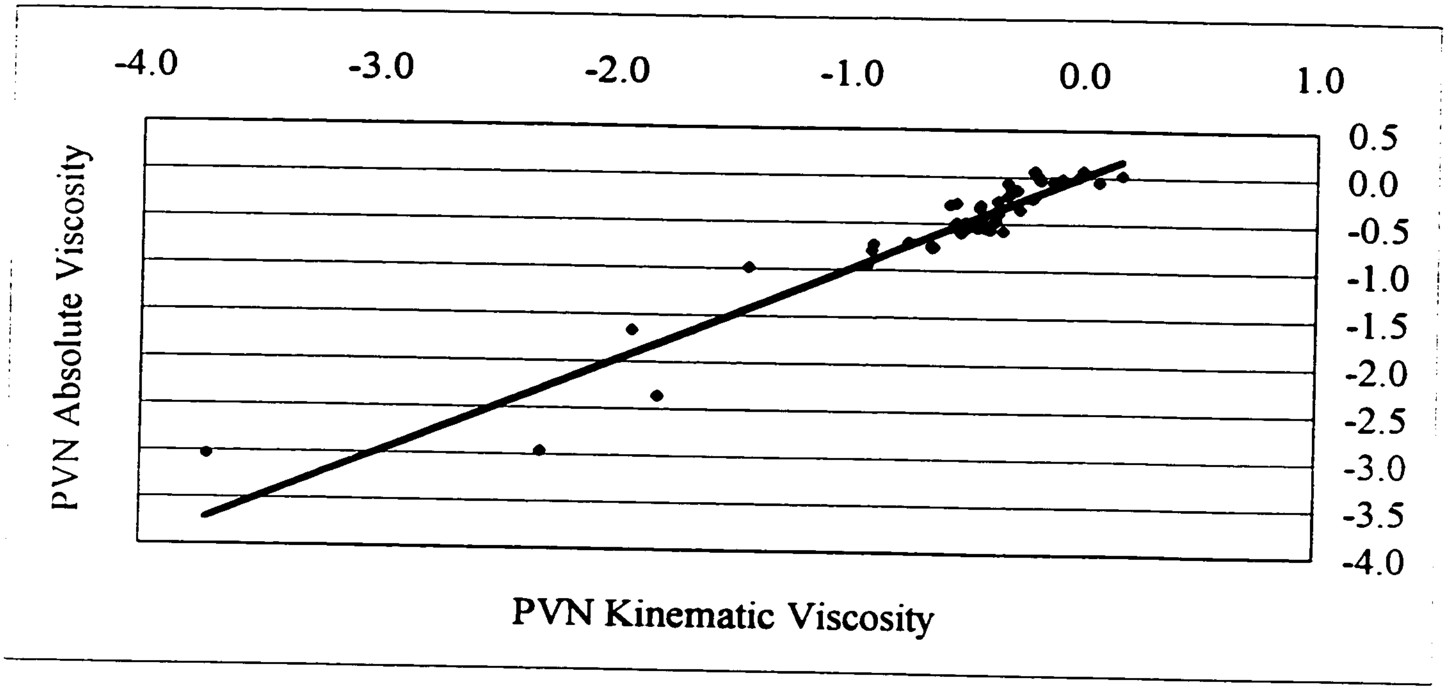


Figure 4.5 Verification of PVN Calculations on Original Asphalt Samples

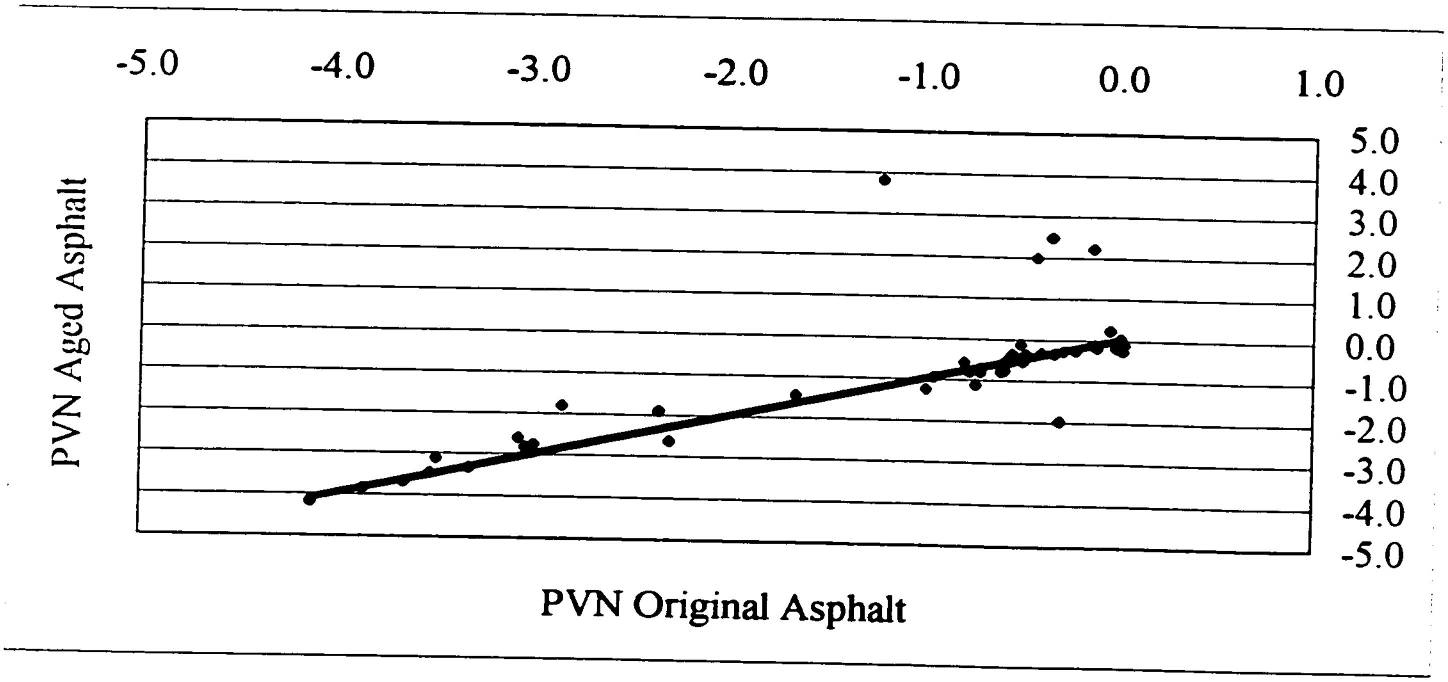


Figure 4.6 Verification of PVN Calculations Using Absolute Viscosity

Figure 4.6 depicts the PVN calculations using the absolute viscosities for original and aged samples. Many of the calculated PVN values fall along the equality line indicating good correlation. However, there are a few points above the line. In these cases, the PVN calculated on the aged sample gives a higher value when compared to the PVN on the original sample. This is most likely related to the variation in the test methods.

Figure 4.7 shows that many asphalts fall directly on the line of equality, meaning that there are very few differences between the calculation methods. However, there are some points scattered around the line. The points above the line represent the case where the PVN calculated on the original asphalt is giving higher values (showing it is less temperature susceptible), while the PVN values below the line show the opposite, indicating that the aged PVN has a lower temperature susceptibility.

4.3 IS PVN A LONG TERM FINGERPRINT?

In 1996, a study was published which presented penetration, viscosity, BBR data and cracking data on six samples at four different stages [Kandahl 96]. The test data were compiled on the original asphalt cement, short term aging, after construction and seven years in-service. Analysis of the data as part of this thesis research, as shown in Figure 4.8, indicates that the PVN value remains constant over time for these six asphalt cements. Although the PVN values have a limited range, it does indicate temporal stability, which supports the McLeod theory that PVN is a "fingerprint" [McLeod 76]. The concept of a fingerprint is very powerful as PVN can be used to determine low temperature susceptibility in a quick and simple manner. Table 4.6 summarizes ANOVA F while, complete results are detailed in Table A.6 in Appendix A.

The ANOVA indicates that between asphalts, the calculated PVN values are statistically significant. With regard to the within asphalt analysis, the $F_{CALCULATED} < F_{CRITICAL}$, which means the null hypothesis is accepted and the PVN remains constant with time. This result is very

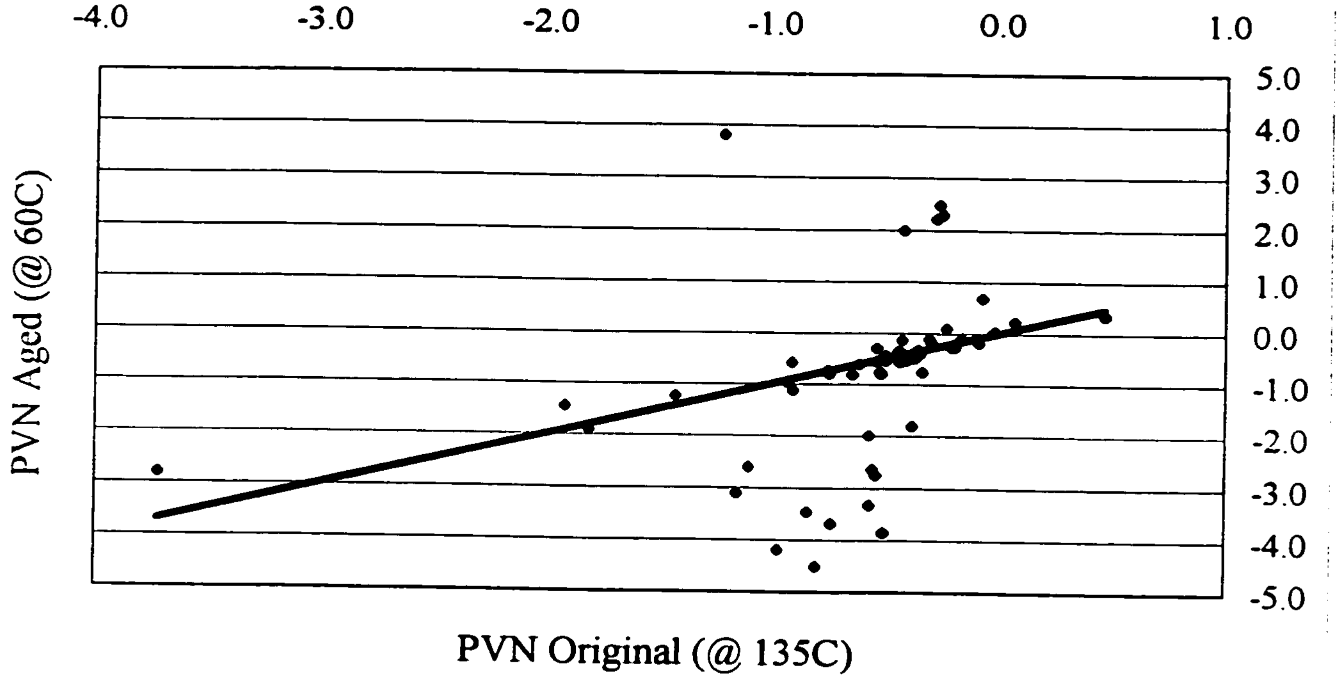


Figure 4.7 Verification of PVN Calculations Using Original and Aged Asphalt Samples

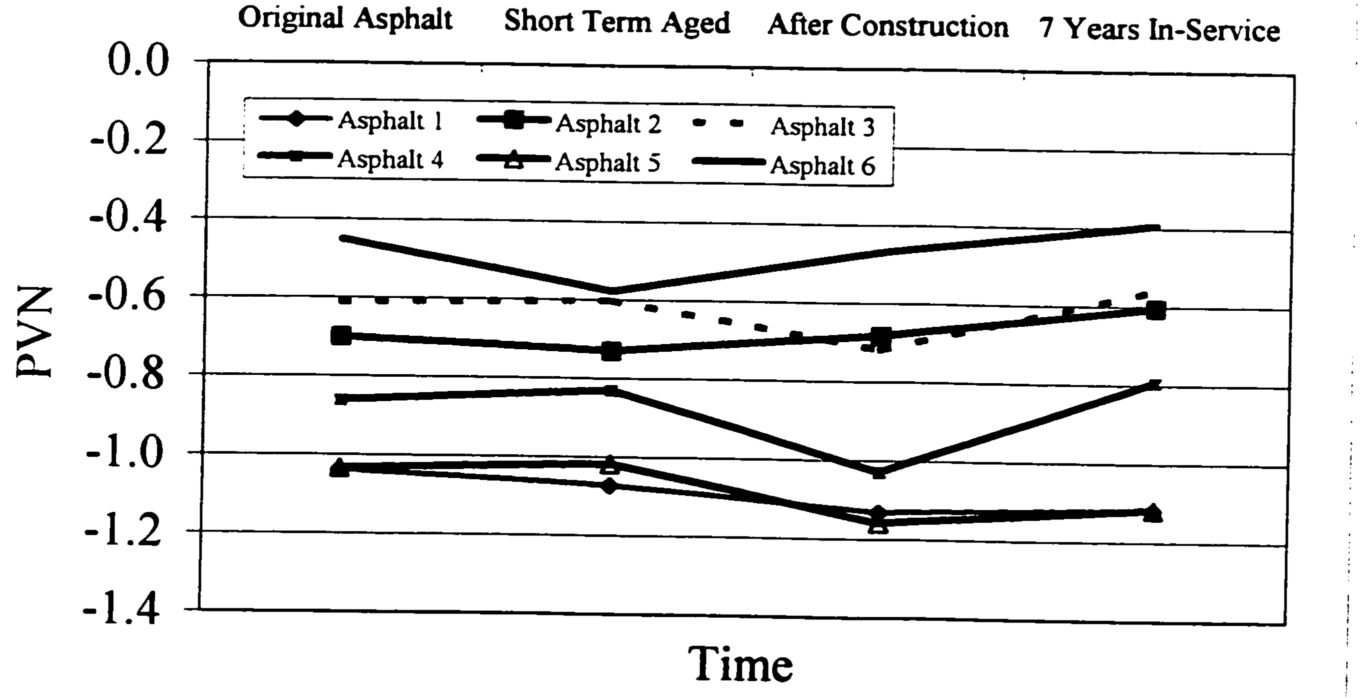


Figure 4.8 PVN Over Time, Based on Analysis of data in [Kandahl 96]

important as it confirms that PVN does remain constant with time.

Table 4.6 PVN Over Time (ANOVA F)

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

Notes: 1) PVN calculated using data from [Kandahl 96].

2) Represents the difference between asphalt samples where the null hypothesis states the true PVN values for the asphalt are equal.

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

4.4 SUMMARY OF ANOVA

Overall, six ANOVA's were performed. The results summarized in Table 4.7 indicate that PVN does effectively remain constant over time and that the same PVN is obtained regardless of which calculation method (based on absolute or kinematic viscosity) is used.

Table 4.7 Summary of the Purpose and Result of ANOVA

Comparison	PVN Varies Between Asphalt[1]	PVN Fingerprint with Time[2]	PVN Calculation Independent[3]
ANOVA A	Yes	No[4]	No[4]
ANOVA B	Yes	Yes	Yes
ANOVA C	Yes	NA[5]	Yes
ANOVA D	Yes	Yes	Yes
ANOVA E	Yes	Yes	Yes
ANOVA F	Yes	Yes	NA[5]

Notes 1) Null Hypothesis rejected meaning statistical significance between different asphalts.

2) ANOVA tests indicate statistical significance of PVN over time.

3) ANOVA tests indicate all PVN calculation methods are the same.

4) PVN concept not developed for all asphalts and reliability of data questionable.

5) Not applicable as ANOVA does not indicate this.

4.5 DETERMINING POWER OF THE ANOVA

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

This risk is almost never known to the experimentor, although, it can be calculated for specific values of interest. The calculation is known as the power calculation or the probability of correctly rejecting the H_o when the H_o is false [Mason 89]. Power depends on the sample size and the degree of difference between the hypothesized and true populations [Scheffé 59]. If the H_o is true, the maximum power of a test equals α. Based on data available, this power test would be a two-way layout with equal numbers of observations in the cells and would be described in equation 4. 6 [Scheffé 59].

$$Y_{ij} = \mu_{ij} + \alpha_i + \beta_j + \varepsilon_{ij} \qquad (4.6)$$

Where: μ_{ij} = mean row value
α_i = a x [row averages matrix] i.e. .01 x [1 0 −1]
β_j = [column averages matrix] i.e. [1 2 −1 −2]
ε_{ij} = iid N (μ, σ^2)
a = percent difference in the true populations

The power calculation can be simulated using this equation with a Monte Carlo analysis. The probability of rejecting the null hypothesis is determined by dividing the number of simulations where the H_o is rejected by the total number of simulations.

Alternatively, the power can be calculated by calculating φ for the Pearson and Hartley tables provided in Scheffé and described in equation 4.7 [Scheffé 59].

$$\varphi = [(1 / (\upsilon_1 + 1).5) + 1 / \sigma_R \text{ x } ((\upsilon_2) \Sigma (\alpha_i - \alpha_m)^2)^{.5}] / (\sigma^2) \qquad (4.7)$$

Where: φ = value required for Pearson and Hartley Table [Scheffé 59]
υ_1 = rows degree of freedom
υ_2 = column degree of freedom
σ_R = standard deviation of rows
σ^2 = variance entire sample
α_i = mean for the i^th row under the alternative hypothesis Ha
α_m = a [row average matrix]
a = percent difference in the true populations

The degrees of freedom (υ_1 and υ_2) are used to select the appropriate Pearson and Hartley table [Scheffé 59]. The probability of rejecting the null hypothesis (power) can then be determined from the charts. Table 4.8 summarizes a comparison by Reilly [Reilly 99] where a power calculation using a Monte Carlo simulation is compared to the value obtained using the Pearson and Hartely tables and equation 4.7.

Table 4.8 Power Calculation Method Comparison [Reilly 99]

Degree of Difference in True Values	Number of Iterations (Monte Carlo)	Probability of Rejecting H_o	φ	Pearson and Hartley Table [Scheffé 59]
0.00	278848	.05048	0.00	0.05
0.01	249490	.22789	0.89	0.23
0.02	628541	.74499	1.77	0.745
0.03	205025	0.98257	2.66	0.98

When the degree of difference in true values is zero, the probability of rejecting the null hypothesis is 5%. This would be expected as the F test is being conducted at the 95 % confidence level and the power would be 5%. In other words, when two values are equal, there is a 5% probability that the null hypothesis would be rejected. As the percent difference between values increases, the power increases indicating a better probability of detecting differences.

4.5.1 Approach to Power Analysis

To determine the power of ANOVA's B, C, D, E and F, the value was calculated according to equation 4.7. The probability of rejecting H_o was then determined using the Pearson and Hartley tables in Scheffé [Scheffé 59]. The limitations of these calculations were that the tables provided values for degrees of freedom of rows (υ_1) between one and six and degrees of freedom for columns (υ_2) between six and sixty.

Operating curves were constructed as shown in Figure 4.9 through 4.13 for ANOVA B, C, D, E and F respectively. The complete results are in Table A.7 in Appendix A. The x axis on the operating curve is the percent difference between the true values, in this case the PVN, while the y axis is the probability of rejecting the null hypothesis. If the percent difference between the true values is low corresponding to a high rejection of the null hypothesis, then the test would have a high power and it would indicate the ANOVA result is good.

To assess the power, the 95% probability of rejecting the null hypothesis was examined on the operating curves. The 95% was selected for consistency with the ANOVA. Table 4.9 summarizes the percent difference between the two values when the power is 95 %. The percent difference in the true values at a 95% probability of rejection ranges from 3.3% to 45%. Based on this finding, it would indicate that ANOVA F, which tests the temporal nature of PVN is an extremely good test. At a 95% rejection of the null hypothesis, the difference in values would be very small for the null hypothesis to be rejected. In fact for ANOVA B, C, and D, the results are also very good as they are all under 10 %.

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

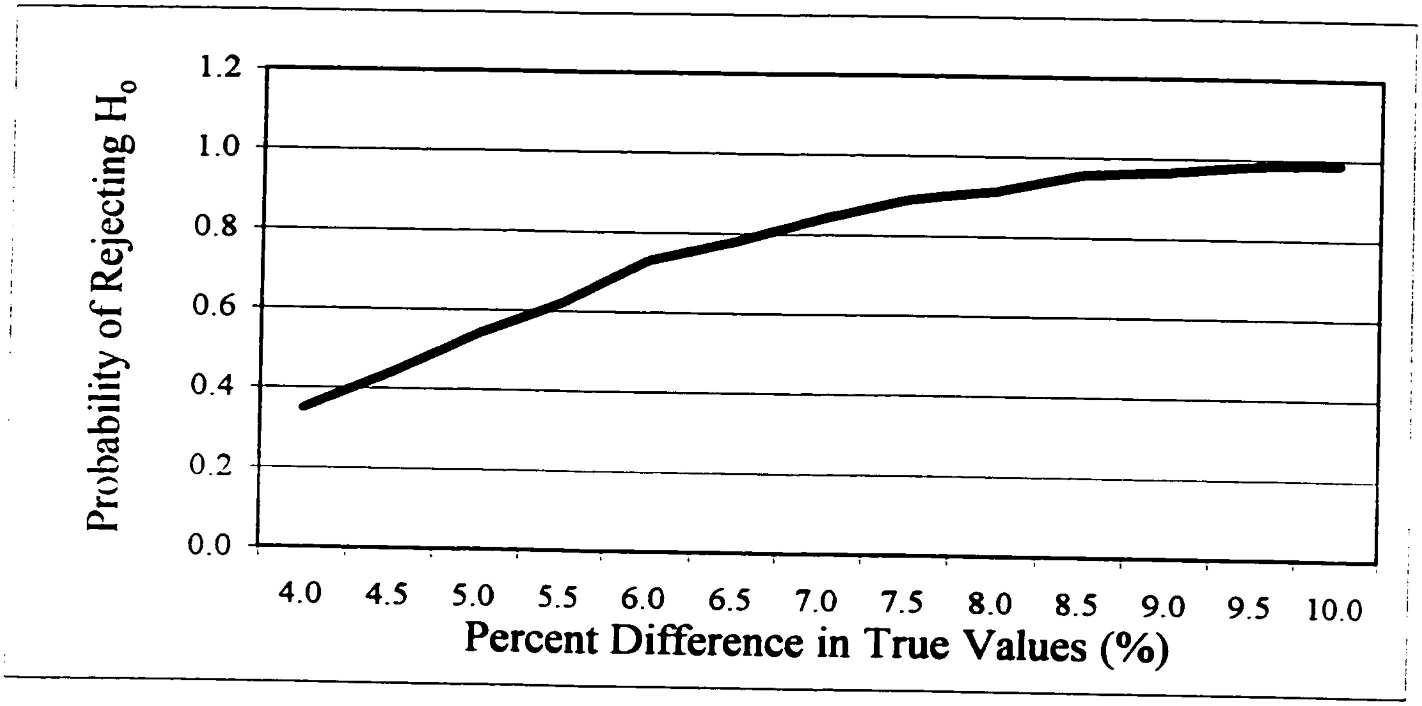


Figure 4.9 Operating Curve for ANOVA B

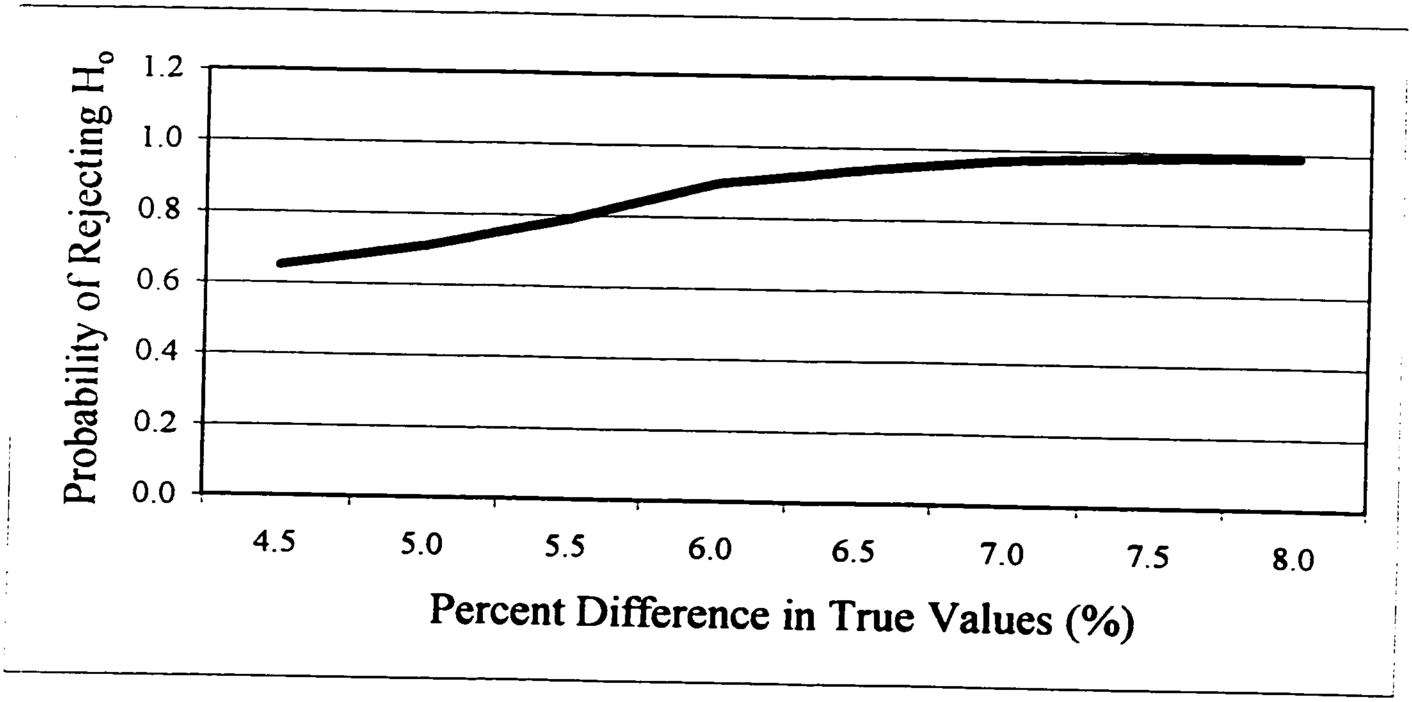


Figure 4.10 Operating Curve for ANOVA C

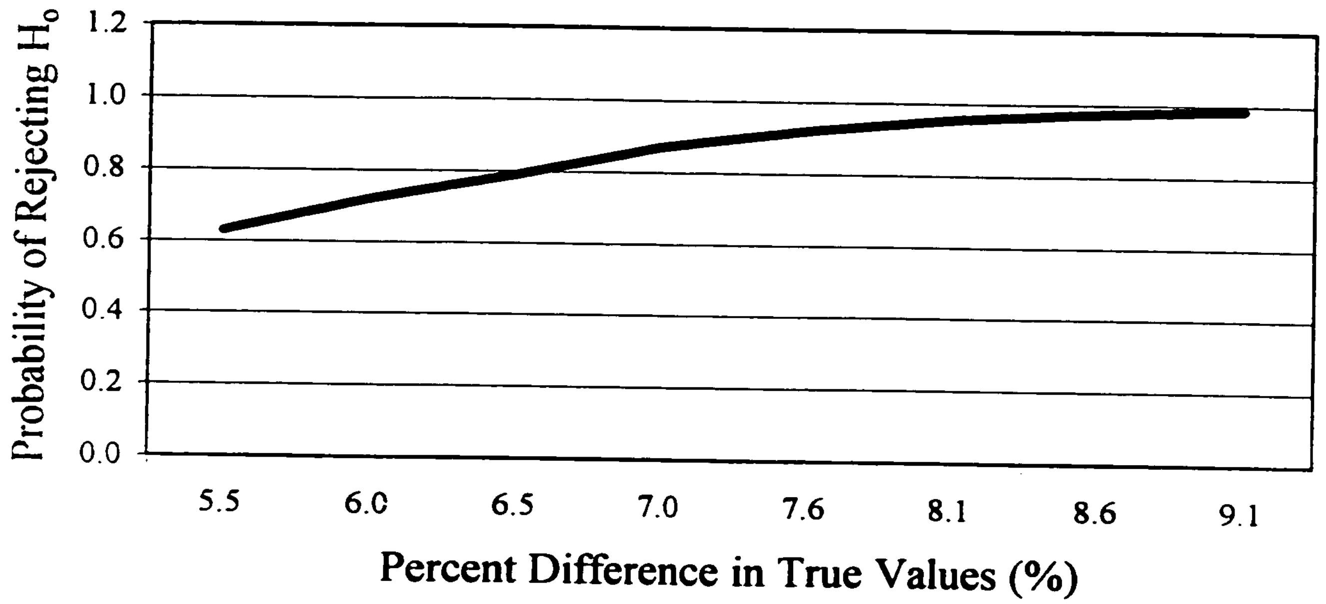


Figure 4.11 Operating Curve for ANOVA D

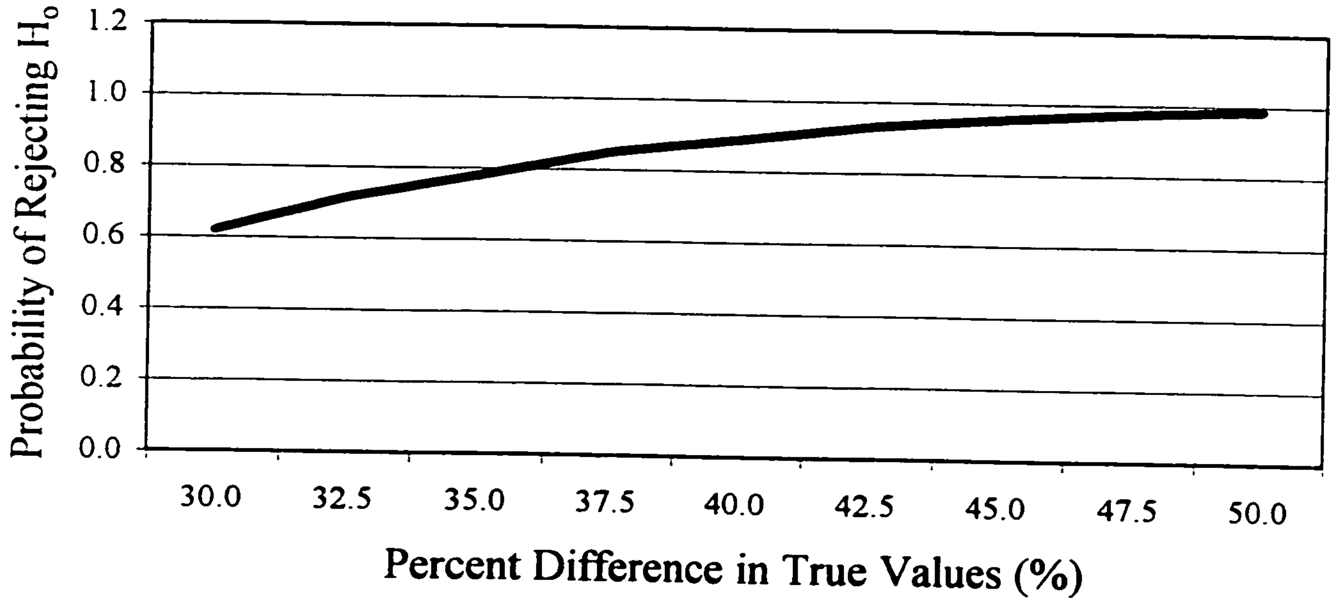


Figure 4.12 Operating Curve for ANOVA E

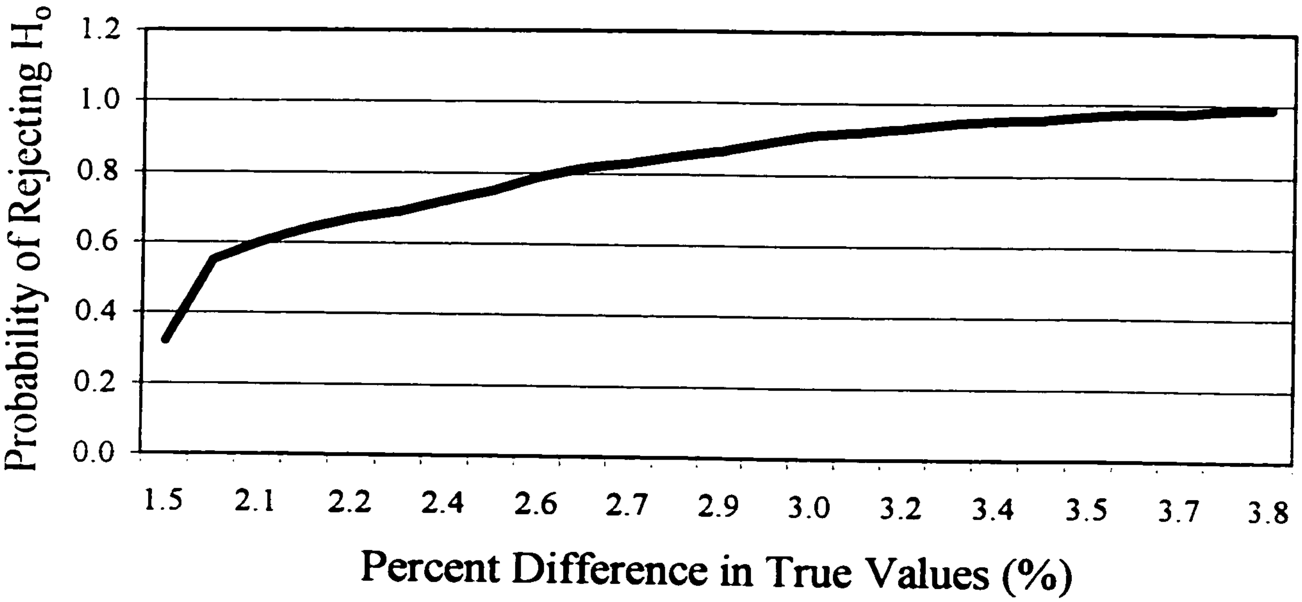


Figure 4.13 Operating Curve for ANOVA F

Table 4.9 PVN Power Summary for ANOVA's

ANOVA	% Difference @ 95%	υ_1	υ_2	Purpose of ANOVA Test
B	7.9 %	2	28	Repeatability of PVN and PVN over time
C	6.7%	1	48	Repeatability of PVN only
D	8.0%	1	47	Repeatability of PVN and PVN over time
E	45%	1	56	PVN over time
F	3.3%	3	5	PVN over time

4.6 PVN AND SUPERPAVE PG COMPARISON

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

CGSB specification bands have been shown to be much narrower at high and low in-service temperatures [Jhanwar 98]. In general the band is usually 1°C to 2°C as compared to the 6°C for the Superpave increments. Thus, to apply a transformation between systems is technically not advisable. For example a CGSB 150 – 200 A, which meets a PG 58-28, does not mean that a PG 58-28 meets a CGSB 150 – 200 A [Jhanwar 98]. As well, variability associated with intra-laboratory, "within site" reproducibility and inter-laboratory, "different sites" could result in misclassification of a PG asphalt [Puzic 96]. To avoid such misclassifications, the actual grades should be used as opposed to the official grades with the 6°C band.

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

4.6.1 PVN and Superpave PG Minimum Temperature Comparison Results

The asphalt cements included in this analysis are those samples for which both the PG grade was available and PVN could be calculated. The PVN values for the PG asphalts were compared both within groups and between groups at the 95% confidence level. Four PVN comparisons were made for each of the following : PG 58 – 22 and PG 58 – 28, PG 52 – 28 and PG 52 – 34, PG 64 – 22 and PG 64 – 28 and PG 46 – 28 and PG 46 –34, as summarized in Table 4.10.

Table 4.10 ANOVA Summary for Superpave Comparison

Comparison1)	**Test**	**$F_{CALCULATED}$**	**$F_{CRITICAL}$**	**P-value**	**Degrees of Freedom**
$PVN_{PG\ 58-22}$ AND $PVN_{PG\ 58-28}$	Between2) Grade	12.184	4.279	0.002	1
$PVN_{PG\ 52-28}$ AND $PVN_{PG\ 52-34}$	Between2) Grade	20.928	6.608	0.006	1
$PVN_{PG\ 64-22}$ AND $PVN_{PG\ 64-28}$	Between2) Grade	51.287	6.608	0.0008	1
$PVN_{PG\ 46-28}$ AND $PVN_{PG\ 46-34}$	Between2) Grade	5.636	10.128	0.098	1

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

Detailed ANOVA are in Appendix A, Tables A.8. The null hypothesis is rejected for the first three comparisons (i.e. $F_{CRITICAL} < F_{CALCULATED}$) as shown in Table 4.10. This means that for these comparisons, there are statistically significant differences for the PVN's in these groups. However, in the last case, the PG 46 – 28 and PG 46 – 34, the difference is not statistically significant.

Figure 4.14 shows the magnitude of the PVN for the respective PG Superpave asphalts. In all cases, as one moves from a less conservative low temperature grade to a more conservative grades, for example from PG 46 – 28 to PG 46 – 34, or from PG 52 – 28 to PG 52 – 34, the PVN value increases. The practical significance of these test results is summarized in Table 4.11.

Table 4.11 Summary of PG and PVN ANOVA

Comparison	**PVN Varies Between PG Asphalt Grades**[1]	**PVN Remains Constant Within PG Asphalt Grades**[2]
PG 58 – 22 AND PG 58 – 28	Yes	Yes
PG 52 – 28 AND PG 52 – 34	Yes	Yes
PG 64 – 22 AND PG 64 – 28	Yes	Yes
PG 46 – 28 AND PG 46 – 34	No[3]	Yes

Notes:
1) Null hypothesis rejected meaning the true PVN values between PG asphalt grades are different.
2) Null hypothesis is accepted meaning there is no differences of the true value of the PVN within a PG asphalt.
3) Null hypothesis accepted meaning there are no differences between the PG asphalt.

This finding has various implications, as it indicates that PVN does relate to low temperature susceptibility because the comparison has essentially isolated the PG low temperature influence. A power test was not possible based on limited data.

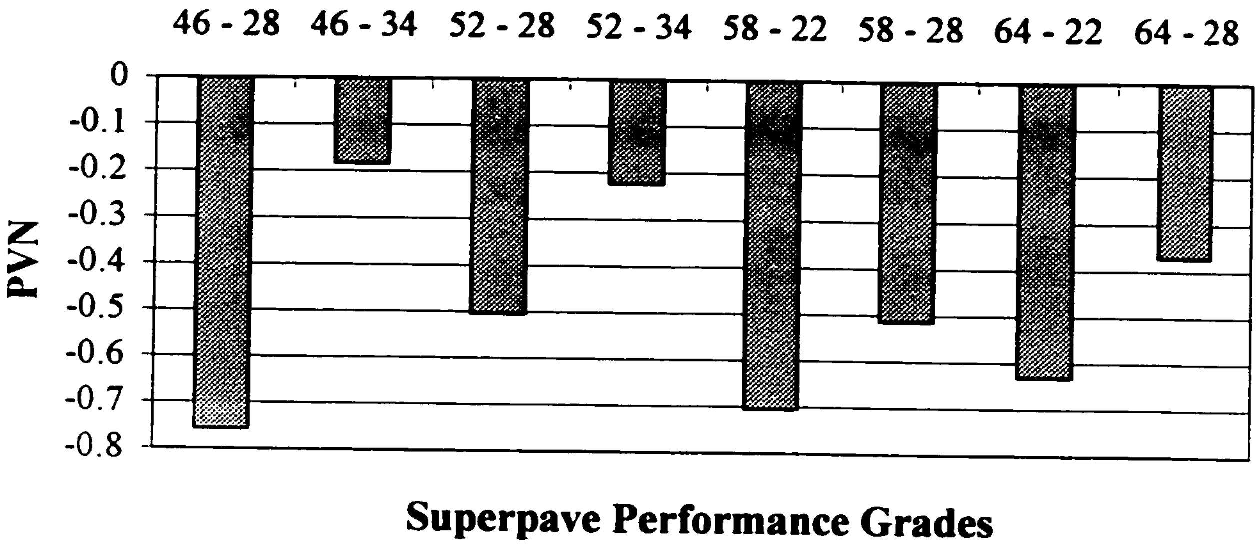


Figure 4.14 Average PVN Values for Superpave Performance Grades

4.7 PVN AND CRUDE SOURCE

The purpose of this analysis is to determine what type of variability is associated with a PVN for a given crude source. The asphalt samples were separated according to crude source and the number of samples available for each source. If the crude source had only one or two samples, it was not included in the analysis. The crudes were analyzed based on the number of samples that were available. Table 4.12 summarizes the ANOVA results when three, four, six, fourteen, twenty-three and thirty-two PVN values were available per crude source. Detailed calculations are located in Appendix A, Tables A.9.

Table 4.12 Summary of Crude Source ANOVA

Comparison[1]	Test	$F_{CALCULATED}$	$F_{CRITICAL}$	P-value	Degrees of Freedom
3 PVN/ Crude Sources	Between[2] Source	11.0680	1.4950	1.3E-22	47
	Within[3] Source	19.9922	3.0932	5.83E-8	2
4 PVN / Crude Sources	Between[2] Source	15.4545	10.1280	0.0293	1
	Within[3] Source	0.7136	9.2799	0.6059	3
6 PVN / Crude Sources	Between[2] Source	2.8045	2.6030	0.0382	5
	Within[3] Source	0.6850	2.6030	0.6391	5
14 PVN / Crude Sources	Between[2] Source	16.1103	3.4028	3.66E-5	2
	Within[3] Source	1.0474	2.1834	0.4413	12
23 PVN / Crude Sources	Between[2] Source	1.9244	3.2093	0.1580	2
	Within[3] Source	1.5904	1.7889	0.0942	22
32 PVN / Crude Sources	Between[2] Source	0.1317	4.1596	0.7192	1
	Within[3] Source	1.8835	1.8221	0.0414	31

Notes: 1) Comparison categorizes according to the number of PVN's available per crude source.
2) Represents the difference between asphalt samples where the null hypothesis states all asphalts are equal.
3) Represents the differences in PVN values calculated for the same asphalt where the

null hypothesis states regardless of how PVN is calculated the values will be the same.

The values indicate that in some cases the null hypothesis is rejected while in other cases it is accepted for the between crude source variation. One possible explanation is that in these cases, the asphalt is not within the "typical within normal PVN range" (i.e. PVN = -9.75 and –11.56, which is much beyond the McLeod range). The null hypothesis for within grades is also inconsistent.

In an engineering sense, it would be expected that the PVN within a crude source would vary depending on the fraction of the distillate in the refining process; as well, as the crude may change over time or because "contaminants" (i.e. mixed in the pipeline). In addition, different crude sources could exhibit similar low temperature properties. Thus, it seems more reasonable to carry out the crude source analysis incorporating the Superpave PG asphalt grades and to determine the power for one of the crude sources.

The power test was performed based on the ANOVA of six crude source samples. As shown in Table 4.12, the within source analysis indicates that the true PVN values are equal as the null hypothesis is accepted ($F_{CALCULATED} < F_{CRITICAL}$). The power, shown in Figure 4.15 (Appendix Table A.10) is the operating curve for this test. The magnitude of the x value is much higher in this example as compared to the previous power tests for PVN. At the 95% probability of rejecting H_o, the percent difference in true values would be 55%. The power of the test is not very good as the differences in PVN will not be detected very easily; for a rejection to occur the true values must differ by 55%. This finding would further emphasize the point that the PVN within a crude source could vary for a number of reasons.

Consequently, the following is a crude source analysis incorporating the Superpave PG asphalt grades. Three "typical" Canadian crude sources have been selected for this analysis. Each

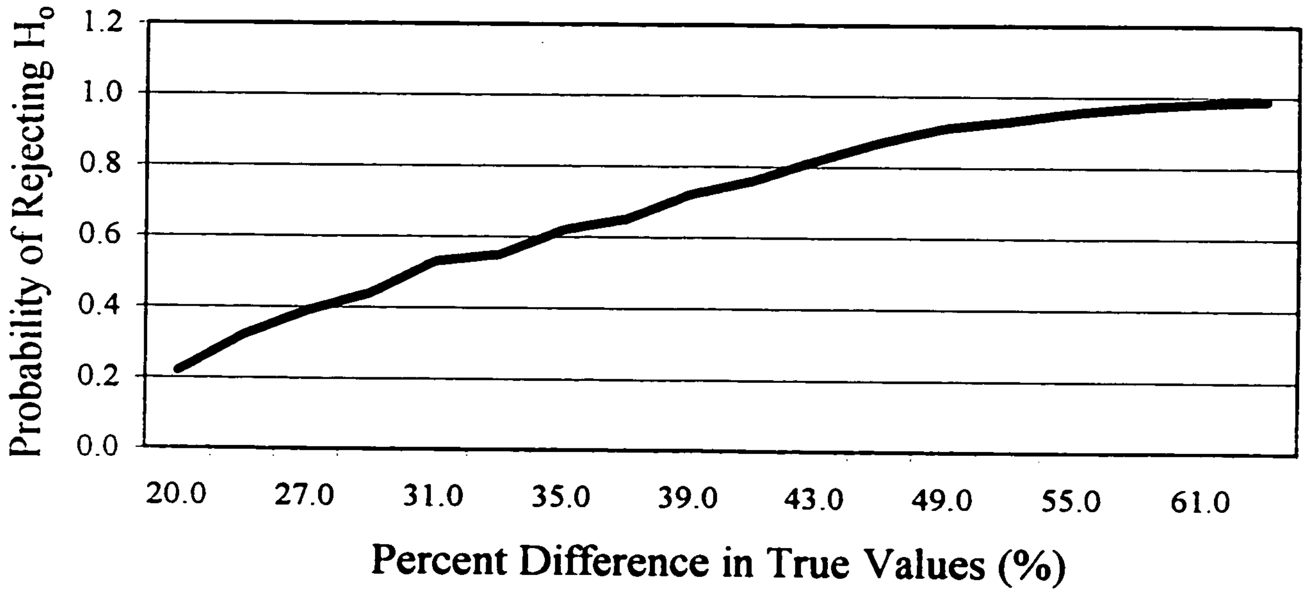


Figure 4.15 Operating Curve for ANOVA 6 Crude Sources

sample has the PG asphalt grade and the PVN on the original asphalt. The ANOVA results are presented in Tables 4.13 and A.11.

Table 4.13 ANOVA Summary for Superpave PG and PVN Crude Source Comparison

Comparison1)	**Test**	**FCALCULATED**	**FCRITICAL**	**P-value**	**Degrees of Freedom**
Crude Source A	**Between2) Grade**	49.6921	4.7571	1.0E-4	3
	Within3) Grade	2.0489	5.1432	0.2098	2
Crude Source B	**Between2) Grade**	26.8346	4.7571	7.0E-4	3
	Within3) Grade	1.4103	5.1432	0.3147	2
Crude Source C	**Between2) Grade**	19.8759	4.7571	1.6E-3	3
	Within3) Grade	1.0759	5.1432	0.3987	2

Notes: 1) PVN calculated and categorized according to crude source and PG grade.
2) Represents the difference between asphalt samples where the null hypothesis states all asphalts are equal.
3) Represents the differences in PVN values calculated for the same asphalt where the null hypothesis states regardless of how PVN is calculated the values will be the same.

The ANOVA for crude source A involved comparing PVN values which would be classified as PG 46 – 34, PG 52 – 28, PG 58 – 22, PG 58 – 28, PG 64 – 22 and PG 64 – 28. There were statistically significant differences between the PVN's between the PG asphalt grades. This means that the PVN values calculated for a specific grade were different from those corresponding to a different low temperature grade when the high PG temperature is kept constant. However, within grades, there were no differences and the null hypothesis was accepted.

Similar findings were obtained for crude source B (PG 46 – 34, PG 52 – 28, PG 58 – 22, PG 64 – 22, PG 64 – 28 and PG 70 – 22) and crude source C (PG 46 – 34, PG 52 – 34, PG 58 – 28, PG 64 – 22 and PG 64 – 34). Overall, the findings indicate that within a crude source and PG grade, the differences between the PVN values are statistically insignificant. This means that when the PVN values within the PG grade are compared, they are consistent.

Table 4.14 provides the average PVN value, standard deviation and number of samples for each crude source and corresponding Superpave PG asphalt grade. The results again indicate that the PVN corresponds to the Superpave grading. As the asphalt meets the lower temperature requirements, the PVN increases, indicating better tolerance to low temperature cracking. These results will be incorporated into the probabilistic low temperature cracking analysis in Module Two.

Table 4.14 PVN Crude Source Variability Associated with PG Asphalts

Crude Source	Superpave PG Asphalt Grade	PVN		Number of Samples
		Average	**Standard Deviation**	
A	PG 46 –34	-0.2285	0.0340	3
	PG 52 – 28	-0.6837	0.0655	3
	PG 58 – 22	-0.7122	0.0361	4
	PG 58 – 28	-0.6856	0.0734	8
	PG 64 – 22	-0.7068	0.0423	2
	PG 64 – 28	1.0533	0.0493	2
B	PG 46 – 34	-0.4096	0.0963	3
	PG 52 – 28	-0.3480	0.1381	3
	PG 58 – 22	-0.6952	0.2381	8
	PG 64 – 22	0.1454	0.9286	2
	PG 64 – 28	0.9997	0.3654	2
	PG 70 – 22	0.8330	0.4064	3
C	PG 46 – 34	-0.1887	0.0132	3
	PG 52 – 34	-0.2064	0.0935	3
	PG 58 – 28	-0.3932	0.0694	6
	PG 64 – 22	-0.4693	0.0348	3
	PG 64 – 34	1.0576	0.0289	2

Figures 4.15, 4.16 and 4.17 further emphasize that the PVN value increases when the high PG temperature remains constant and the low temperature decreases, thus, further indicating that PVN does characterize low temperature susceptibility.

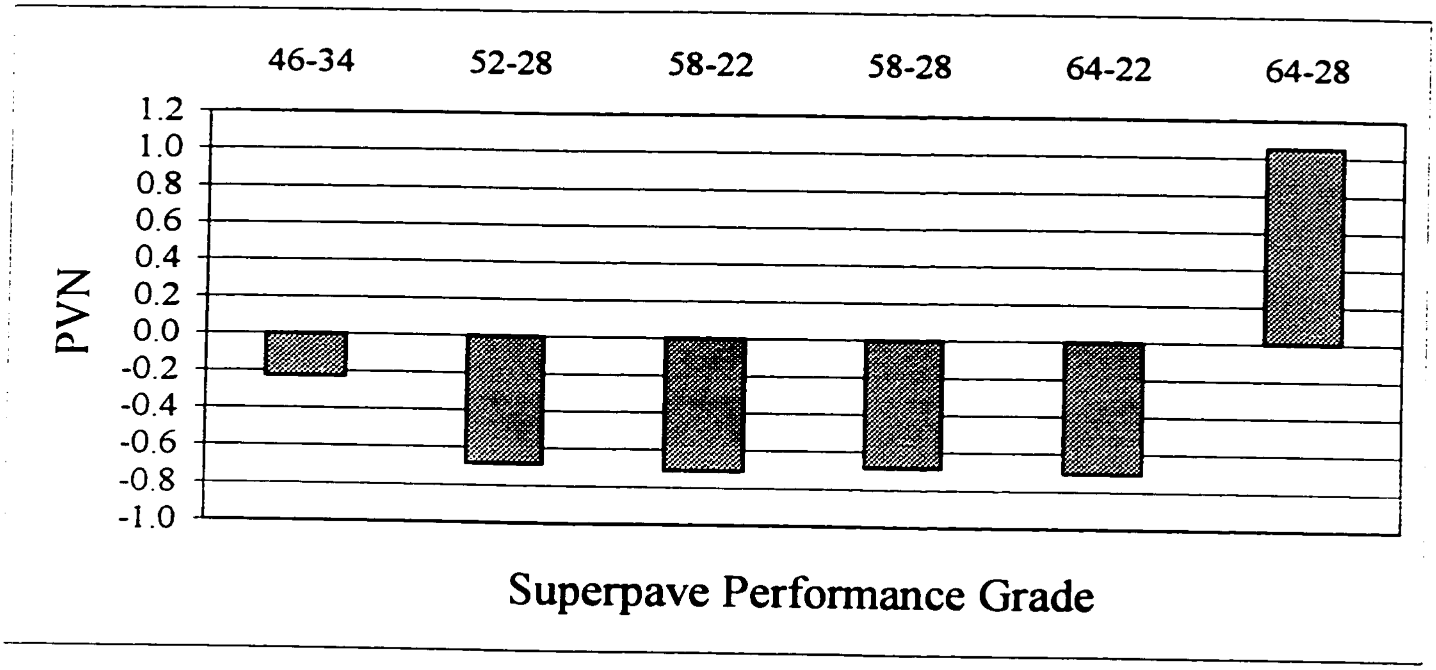


Figure 4.16 Average PVN Values for Crude Source A

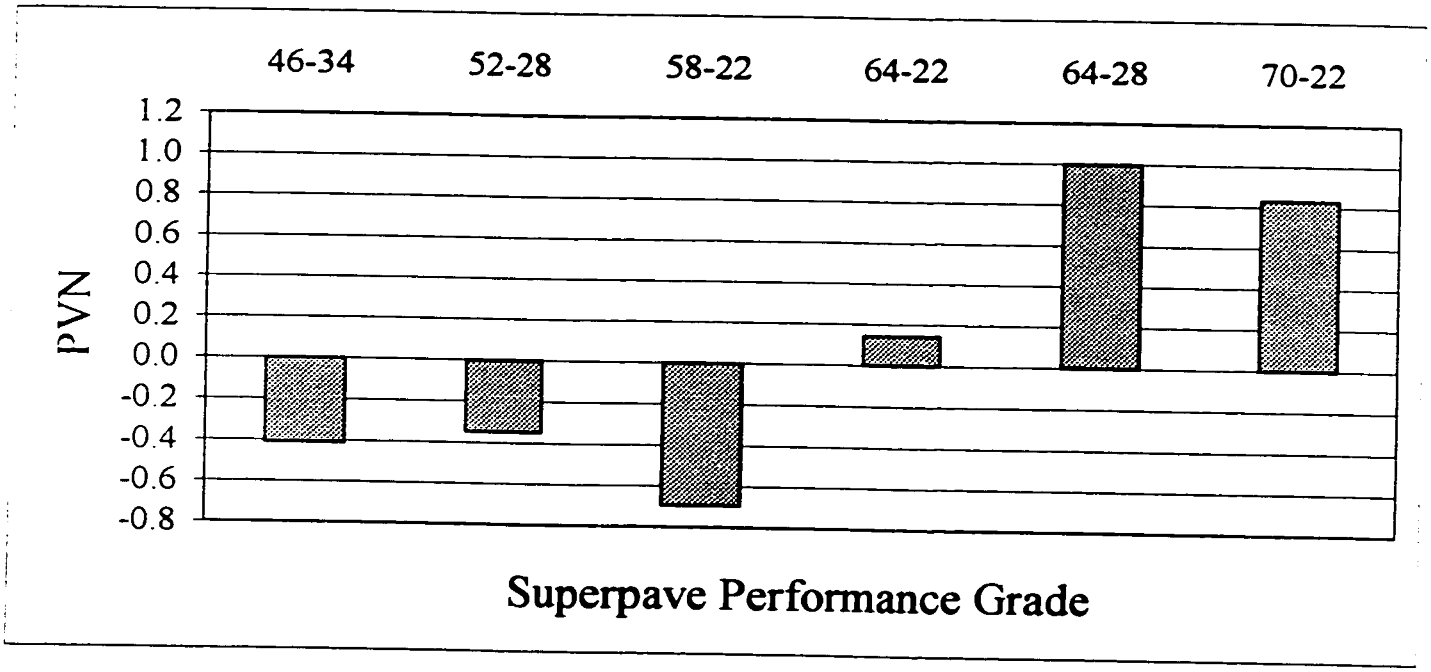


Figure 4.17 Average PVN Values for Crude Source B

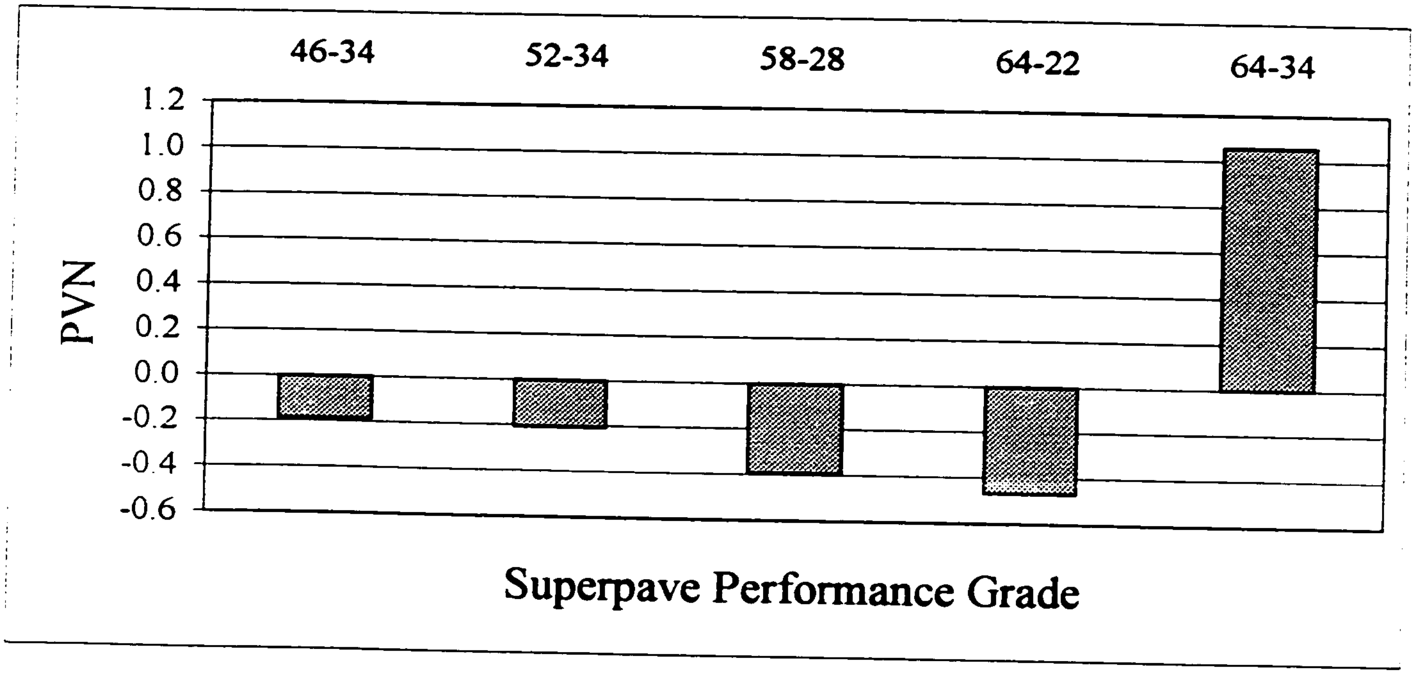


Figure 4.18 Average PVN Values for Crude Source C

4.8 PVN AND MODIFIED ASPHALT

The intent of this section is to examine how PVN is influenced by a modification to asphalt and if there is some correlation between the PVN and the Superpave performance grade. Asphalt modifications were not considered in the development of the PVN concept or the Superpave system. In both cases only conventional asphalt binders were considered. Thus, the assumptions related to PVN and the Superpave testing protocol may not be valid for some modified binder systems. Furthermore, because the type of modifier and quantity will dramatically influence the physical and chemical properties of the asphalt, there appears to be a need for elaborate evaluation of these characteristics [Bahia 96]. This analysis examines the relationship between the temporal nature of PVN and the magnitude of PVN for modified systems.

The values used in the modified asphalt analysis are provided in Table 4.15. The first analysis focuses on those modified asphalts where multiple PVN values can be calculated, using laboratory results on both aged and original modified asphalt samples. Table 4.16 shows the PVN values for seven modified asphalts where more than one PVN value could be calculated.

As shown in two cases three values could be calculated. In three cases, four PVN values were calculated. Three series of ANOVA's were carried out. Table A.12 summarizes the two-way comparison (i.e. original @ 135°C and original @ 60°C, original PVN @ 135°C and aged @ 135°C, etc.). Based on the data available, this two-way analysis was carried out five times as shown in Table 4.17.

In three cases, the two way ANOVA shows that the differences between the PVN calculation methods are not significant. In two other cases, the methods are shown to be statistically different. Based on this result it is difficult to determine if PVN is a good indicator for modified asphalts. When the three and four way analysis is performed, the differences in the calculated PVN's are shown to be statistically significant as shown in Appendix Table A.13 and summarized in Table 4.18. Overall the results indicate that for modified asphalts, the PVN does

Table 4.15 PVN for Selected Modified Asphalt Cements

ID	PVN	ID	PVN
STM 1	-0.7864	STM 17	0.8020
STM 2	0.0537	STM 18	1.088
STM 3	0.0369	STM 19	0.7414
STM 4	0.3439	STM 20	1.2581
STM 5	.5174	STM 21	1.0185
STM 6	0.3139	STM 22	1.0371
STM 7	-.2205	STM 23	1.0781
STM 8	-0.7532	STM 24	.5462
STM 9	-.2483	STM 25	.3646
STM 10	-.2483	STM 26	.6547
STM 11	-.1752	STM 27	1.2980
STM 12	-.5528	STM 28	-.5567
STM 13	-.7070	STM 29	.8203
STM 14	-.3694	STM 30	2.1883
STM 15	-.7196	STM 31	-.4348
STM 16	-.5018	STM 32	.0159

Table 4.16 Summary of PVN Values For Multiple Points as Simulated In The Laboratory

ID	PVN Original @ 135°C	PVN Original @ 60 °C	PVN Aged @ 135°C	PVN Aged @ 60°C
STM 1	-0.7864	-0.8452	NA	-0.4432
STM 2	0.0537	0.2449	NA	0.7089
STM 3	0.0369	-0.2351	NA	NA
STM 4	0.3439	0.0930	NA	NA
STM 5	0.5174	0.5661	1.2639	1.0139
STM 6	0.3139	0.3507	0.8296	0.6171
STM 7	-0.2205	-0.1701	0.0825	-0.0761

Note: NA refers to not available.

Table 4.17 Two Way ANOVA Comparison for Modified Asphalt

Comparison	Test	$F_{CALCULATED}$	$F_{CRITICAL}$	P-value	Degrees of Freedom
Original $PVN_{135^\circ C}$ AND $PVN_{60^\circ C}$	Between1) Methods	0.3191	5.9874	0.5927	1
	Within2) Methods	27.2475	4.2839	0.0004	6
Original $PVN_{135^\circ C}$ AND Aged $PVN_{135^\circ C}$	Between1) Methods	16.5931	18.5128	0.0553	1
	Within2) Methods	19.4138	19.0000	0.0490	2
Original $PVN_{135^\circ C}$ AND Aged $PVN_{60^\circ C}$	Between1) Methods	19.8971	7.7086	0.0112	1
	Within2) Methods	31.7417	6.6882	0.0027	4
Original $PVN_{60^\circ C}$ AND Aged $PVN_{135^\circ C}$	Between1) Methods	13.7365	18.5128	0.0657	1
	Within2) Methods	19.1861	19.000	0.0495	2
Original $PVN_{60^\circ C}$ AND Aged $PVN_{60^\circ C}$	Between1) Methods	23.2040	7.7086	0.0085	1
	Within2) Methods	54.7376	6.3882	0.0010	4

Notes: 1) Represents the difference between calculation methods where the null hypothesis states that PVNs are equal.

2) Represents the difference between asphalts where the null hypothesis states they are equal.

not appear to be consistent. However, based on the limited sample size it is difficult to make a final decision. More data on modified asphalts, with multiple points, would be desirable.

The second analysis performed using modified asphalts was the PVN and PG comparison. The intent was to examine if the magnitude of the PVN values changed when the PG changed. Figure 4.19 shows the average PVN values for the various modified asphalts. When the high end PG is kept constant and the minimum temperature decreases by a PG grade –6°C, the average PVN value is shown to increase. This would be consistent with what would be expected, as when the PG grade decreases at the low end, the asphalt should be less temperature susceptible and have a higher PVN values as shown in this Figure.

Table 4.18 Three and Four Way ANOVA Comparison for Modified Asphalts

Comparison1)	Test	F$_{CALCULATED}$	F$_{CRITICAL}$	P-value	Degrees of Freedom
Four Way Comparison	Between1) Methods	13.1452	4.7571	0.0048	3
	Within2) Methods	66.7843	5.1432	7.94E-05	2
Three Way Comparison Original PVN$_{135oC}$, Original PVN$_{60oC}$ AND Aged PVN$_{60oC}$	Between1) Methods	15.2136	6.9443	0.0135	2
	Within2) Methods	37.1884	6.9443	0.0026	2
Three Way Comparison Original PVN$_{60oC}$, Aged PVN$_{60oC}$ AND Aged PVN$_{135oC}$	Between1) Methods	12.3790	6.9443	0.0193	2
	Within2) Methods	56.1772	6.9443	0.0012	2

Notes: 1) Represents the difference between calculation methods where the null hypothesis states that PVNs are equal.

2) Represents the difference between asphalts where the null hypothesis states they are equal.

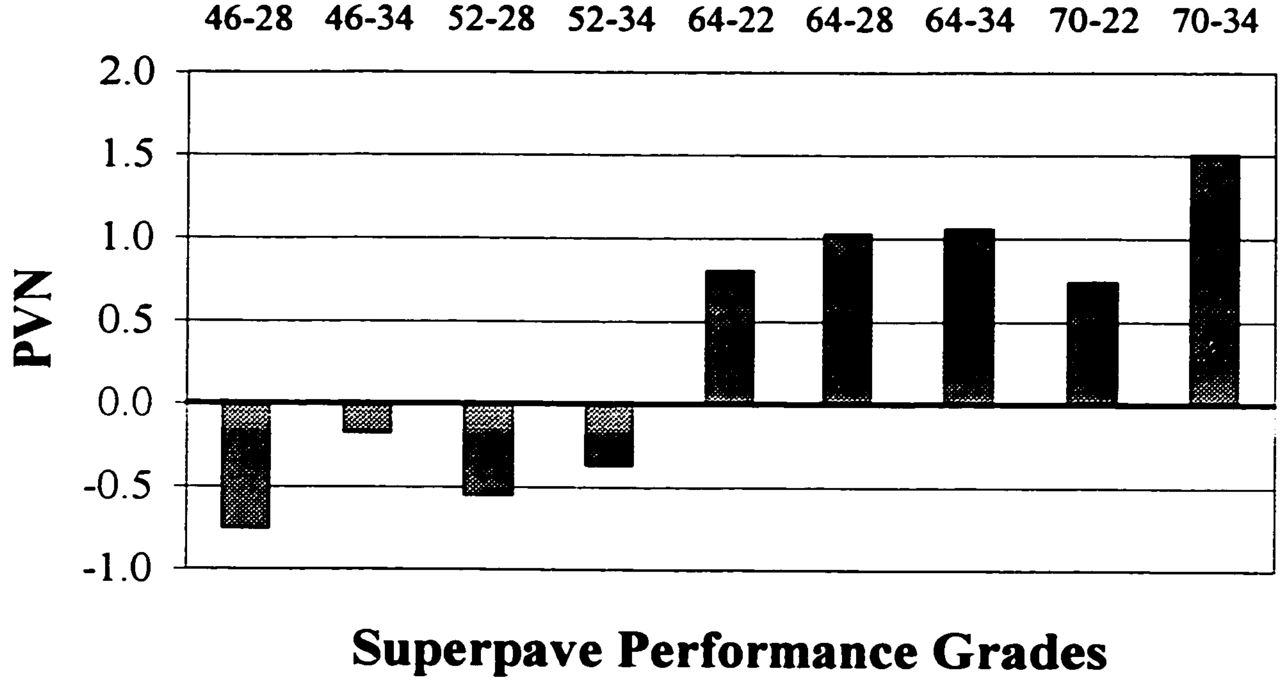


Figure 4.19 PVN For Selected Modified Asphalts

4.9 SUMMARY OF CHAPTER FOUR

Various ANOVA's are carried out in Module One to examine McLeod's Penetration Viscosity Number and how it relates to the Superpave low temperature parameters, how PVN varies over time, what type of variation is related to PVN and how modified asphalts relate to PVN.

The analysis showed that PVN is a fingerprint as it remains constant over time. It is shown to be an indicator of low temperature susceptibility and it is related to the minimum temperature specified in the Superpave design methodology. PVN within a crude source varies as would be expected. It is also shown that regardless of how PVN is calculated, it remains constant.

For modified asphalt, the PVN did not remain constant with time. Although, the sample size was limited. PVN for modified asphalts did relate to the Superpave PG minimum temperature.

CHAPTER FIVE MODULE TWO : LOW TEMPERATURE CRACKING

The purpose of this chapter is to describe the low temperature cracking module. Two methods used to predict cracking are described and examined. Cracking predictions are compared to observed cracking on pavement test sections located throughout Canada. Thermal contraction coefficients are also examined. The results in this module are used as input to the subsequent module.

5.1 MAGNITUDE OF PVN AND HOW IT INFLUENCES CRACKING

Differences in PVN and how they influence cracking are important. Figure 5.1 shows the absolute difference in the magnitude of the PVN values for all of the conventional asphalts (not including modified) based on the calculation method. The PVN values are calculated using the following combinations of the three laboratory test results:

1) PVN (original penetration, original kinematic viscosity) &
   PVN (original penetration, original absolute viscosity)
2) PVN (original penetration, original absolute viscosity) &
   PVN (aged penetration, aged absolute viscosity)
3) PVN (original penetration, original kinematic viscosity) &
   PVN (aged penetration, aged absolute viscosity)

Some notable differences are observed. Ideally the differences should be close to zero, or all columns should be close to the x-axis, indicating there are no differences in the method of calculation. In other words, the PVN calculation method should be irrelevant and remain constant over time. Based on the fact that there are three bars for each sample in cases where there are two columns higher than zero, this would indicate that two results are very similar to each other and one is different, while if there is one column by itself, this indicates one inconsistent result.

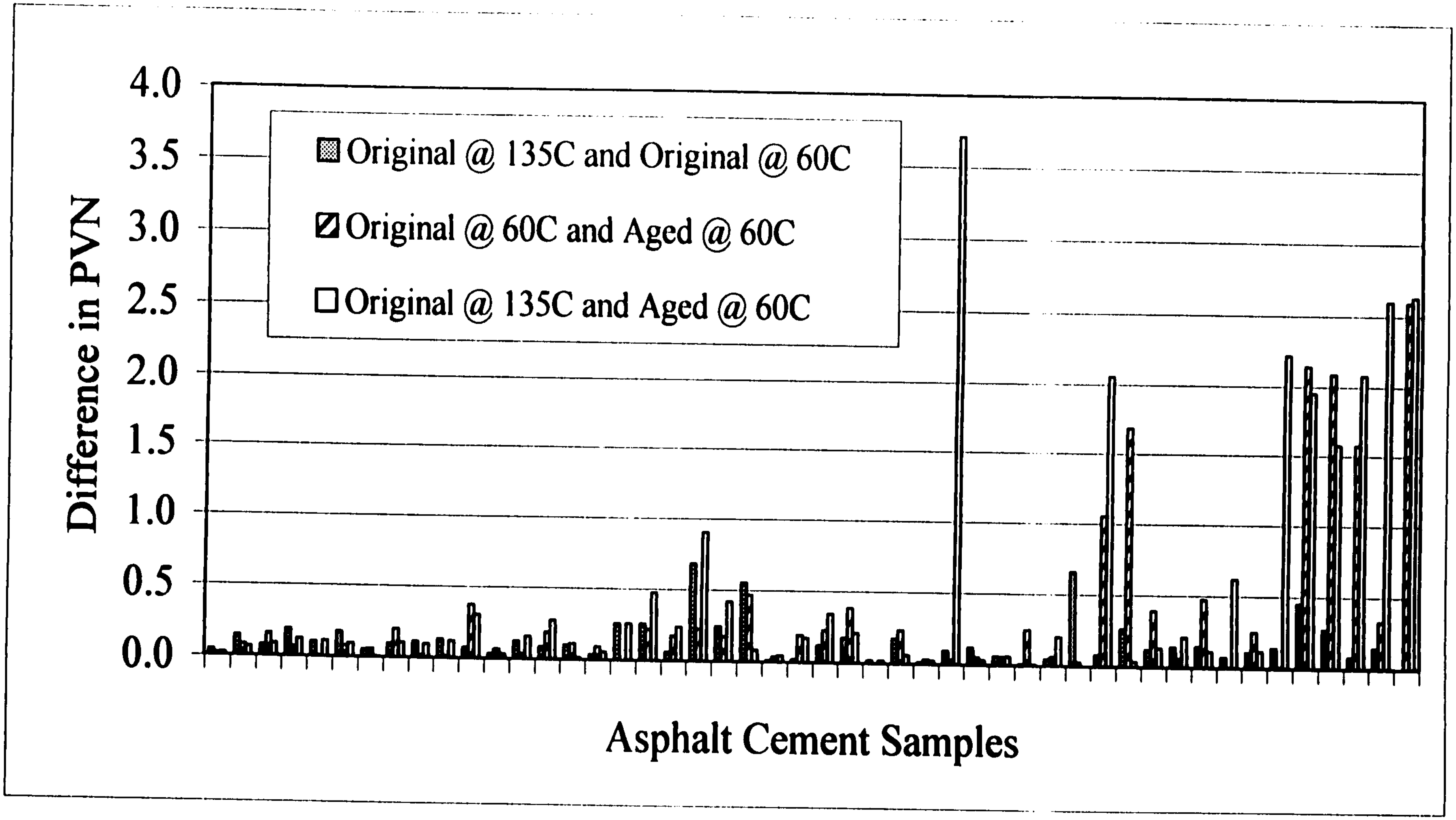


Figure 5.1 Overall Comparison of PVN Calculations

This analysis is presented as it is important to establish the variability associated with the PVN calculation so that the cracking analysis can be carried out in this Module. As well, this analysis will assist in determining fundamentally what the difference between the magnitude of the PVN values means in terms of low temperature cracking.

5.2 MODULE TWO: PREDICTION OF CRACKING

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

If the premise holds that cracking is related to stiffness, and that stiffness is related to PVN, as indicated in Module One, then a cracking frequency model using stiffness directly as an independent variable would be very useful for predicting performance. It is desirable for a pavement designer to estimate cracking frequency as in Canada low temperature cracking can substantially affect the long term performance and life-cycle cost of a pavement. It may even be possible to design a pavement without temperature related cracks. However, due to structural requirements for fatigue and rutting this may not be technically or economically feasible. Hence a trade-off may be necessary. However, regardless of the pavement design factors, if the designer can predict low temperature performance, particularly in Canada, and optimize the design by relating the cracking to performance and life cycle cost, it could result in tremendous cost savings.

The Hajek model [Hajek 71] and the Canadian Airport Model [Haas 87] are used to predict low temperature cracking frequency. Test sections from the C-LTPP graduate grant, as summarized in Table B.1 in Appendix B and the C-SHRP test sections [Anderson 99] are used in this analysis.

5.3 MODEL ONE: HAJEK MODEL PREDICITON

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

$$\log CI = 30.3974 + 0.6026 * S * \log(d) - 12.4958 * m + 1.3388 \log(S) - 2.1316 * d - 0.874 * t * \log(S) + 6.7977 * \log(S) \quad (5.1)$$

where: CI = Cracking Index
S = Stiffness of original asphalt cement in $kg/cm^2 * 10^{-1}$ as determined by McLeod's method for loading time 20,000 seconds and for winter design temperature
a = Age of the pavement, for the time of prediction
m = Winter design temperature in ($°C \times -10^{-1}$)
d = Subgrade type (dimensionless code 1-sand 2-loam 3-clay)
t = Combined thickness of the bituminous layers in inches

The stiffness variable of the asphalt cement is used to represent the asphalt cement and the mix properties. The original asphalt cement variables are used to determine stiffness using Figures 5.2 and 5.3. More specifically, the PVN and original penetration are used to determine the temperature difference between the base temperature and temperature employed for the penetration value. This value obtained from Figure 5.2 is added to 25°C to give a base temperature of a given asphalt cement. The specified service temperature is calculated by adding the specified minimum temperature to the base temperature. The stiffness can then be determined by using Figure 5.3 and a standard loading time of 20,000 seconds.

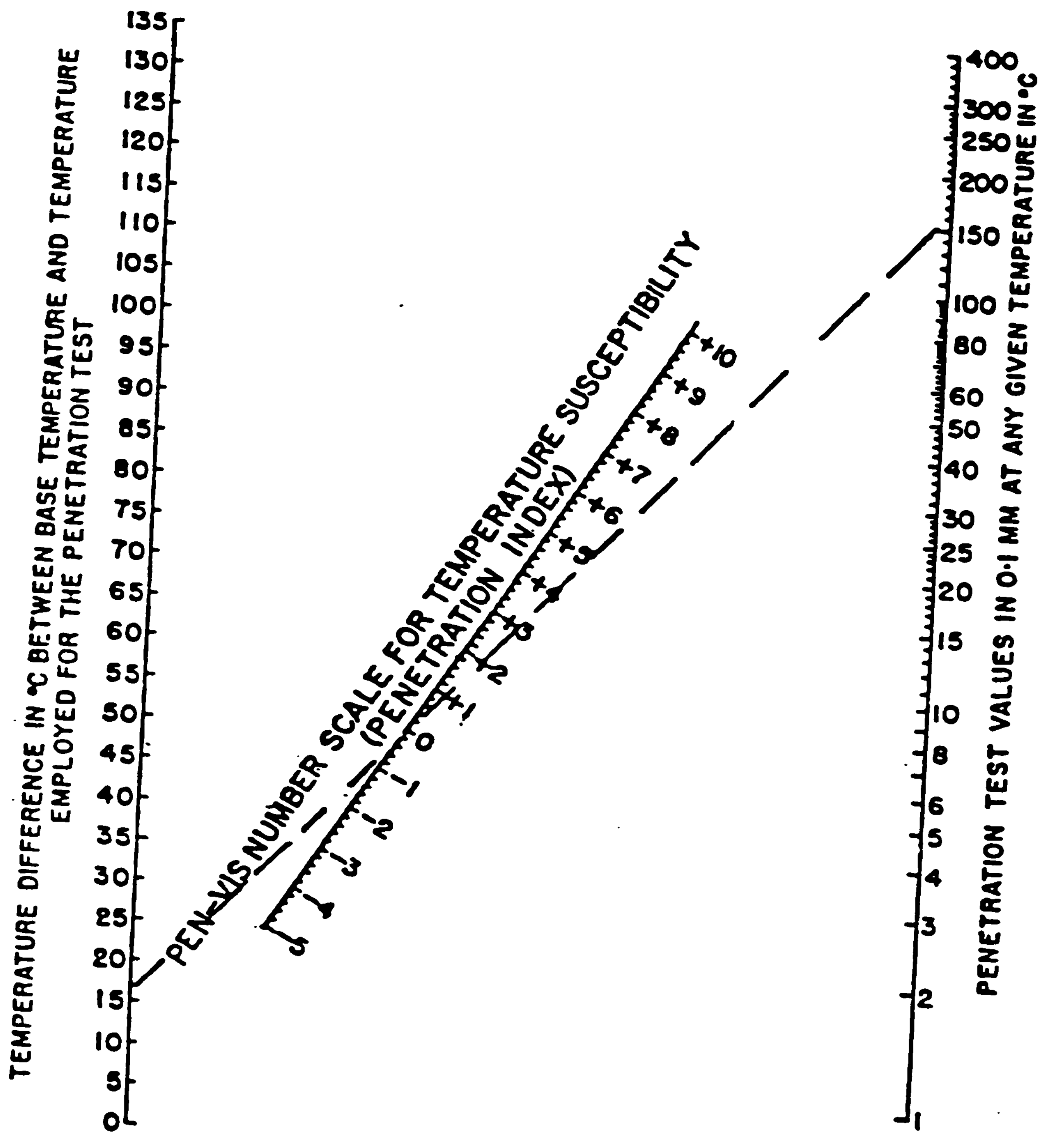
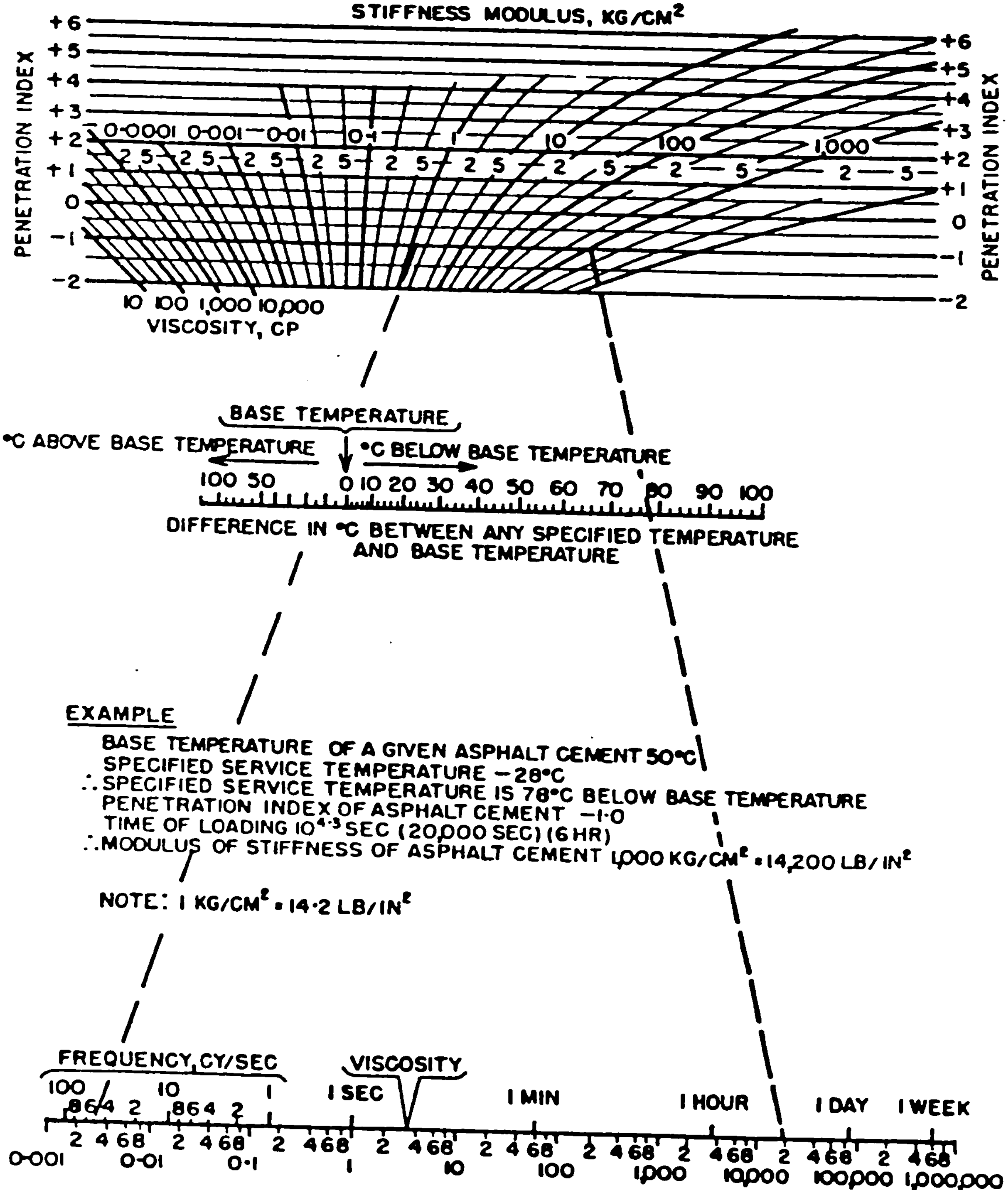


Figure 5.2 Modification of Heukelom's Version of Pfeiffer's and VanDoormal's Nomograph For Relationship between Penetration, Penetration Index and Base Temperature (McLeod 76)



EXAMPLE

BASE TEMPERATURE OF A GIVEN ASPHALT CEMENT 50°C
SPECIFIED SERVICE TEMPERATURE −28°C
∴ SPECIFIED SERVICE TEMPERATURE IS 78°C BELOW BASE TEMPERATURE
PENETRATION INDEX OF ASPHALT CEMENT −1·0
TIME OF LOADING $10^{4 \cdot 3}$ SEC (20,000 SEC) (6 HR)
∴ MODULUS OF STIFFNESS OF ASPHALT CEMENT 1,000 KG/CM² = 14,200 LB/IN²

NOTE: 1 KG/CM² = 14·2 LB/IN²

Figure 5.3 Modification of Heukelom's and Klomp's Version of VanderPoel's Nomograph for Determining Modulus of Stiffness of Asphalt Cements (McLeod 76)

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

5.4 MODEL TWO: CANADIAN AIRPORT MODEL PREDICITON

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

$$\text{TRANCRAK} = 218 + 1.28\ \text{ACTHICK} + 2.52\ \text{MINTEMP} + 30\ \text{PVN} - 60\ \text{COEFFX} \qquad (5.2)$$

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

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

5.5 HAJEK MODEL PREDICTIONS FOR THE C-LTPP SITES

Cracking frequency was estimated for those sections where the five independent variables as outlined in equation 5.1 were available. The stiffness value was determined based on the penetration and PVN for the respective test sections while the thickness value was obtained from the available core data. Both the total thickness and overlay thickness were examined. The overlay thickness or new thickness for the C-LTPP sites was used as it was found to give more reliable cracking results. The total thicknesses of the asphalt for the three C-SHRP test roads (new construction) were used. The subgrade and minimum temperature were categorized according to the data provided. Complete results are summarized in Table B.2 in Appendix B. Table 5.1 summarizes the results for the sections where cracks could be estimated using the Hajek model and where the observed transverse crack data was available.

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

An ANOVA was performed only on those sections where both Hajek predictions and observed were available. The results are summarized in Table 5.2 while the detailed results can be found in Table B.3.

The results indicate that the differences between the predicted cracking and the observed are statistically insignificant ($F_{CALCULATED}$ 0.4407 < $F_{CRITICAL}$ 3.9798). Thus it would indicate that the Hajek model is a good predictor of cracking frequency.

Table 5.1 Low Temperature Cracking Using Hajek Prediction (Page 1 of 2)

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

Table 5.1 Low Temperature Cracking Using Hajek Prediction (Page 2 of 2)

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

Table 5.2 ANOVA Summary for Hajek Prediction and Observed Cracking

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

Notes: 1) Uses C-LTPP and C-SHRP test data [Anderson 99].

2) Represents the difference between test sections where the null hypothesis states all sections are equal.

3) Represents the differences in Hajek prediction and observed cracking where the null hypothesis states regardless of how cracking is determined (observed or predicted), the values will be the same.

A power test was performed on the ANOVA results for the between model and observed cracking presented in Table 5.2. The results identify the percent difference between the true values based on the Hajek model and observed model comparison. The operating curve for this analysis is presented in Figure 5.4. When the probability of rejecting the null hypothesis is 95%, the percent difference in true values would be 3.4%. The values used in the analysis are provided in Appendix B, Table B.4. This small difference indicates the Hajek cracking prediction is a very good model and the buyers risk is low.

5.6 AIRPORT MODEL PREDICTIONS AND OBSERVED DATA

Similar to the Hajek model evaluation, a comparison of the Canadian Airport Model [Haas 87] and observed data was performed. The complete results are contained in Table B.5. Thickness, PVN and minimum temperatures were the same values used as in the Hajek predictions for the various sections. The fourth independent variable in this model is the thermal contraction coefficient (α). This value refers to the asphalt mix volume that decreases when the temperature decreases. Ideally a direct measurement is preferable. However, when a direct measurement cannot be taken, the contraction coefficient can be estimated using prediction methods [Anderson 99]. The predictive equations for thermal contraction used by the Superpave system at the time of this study were not validated and overall there are various methods of estimating thermal

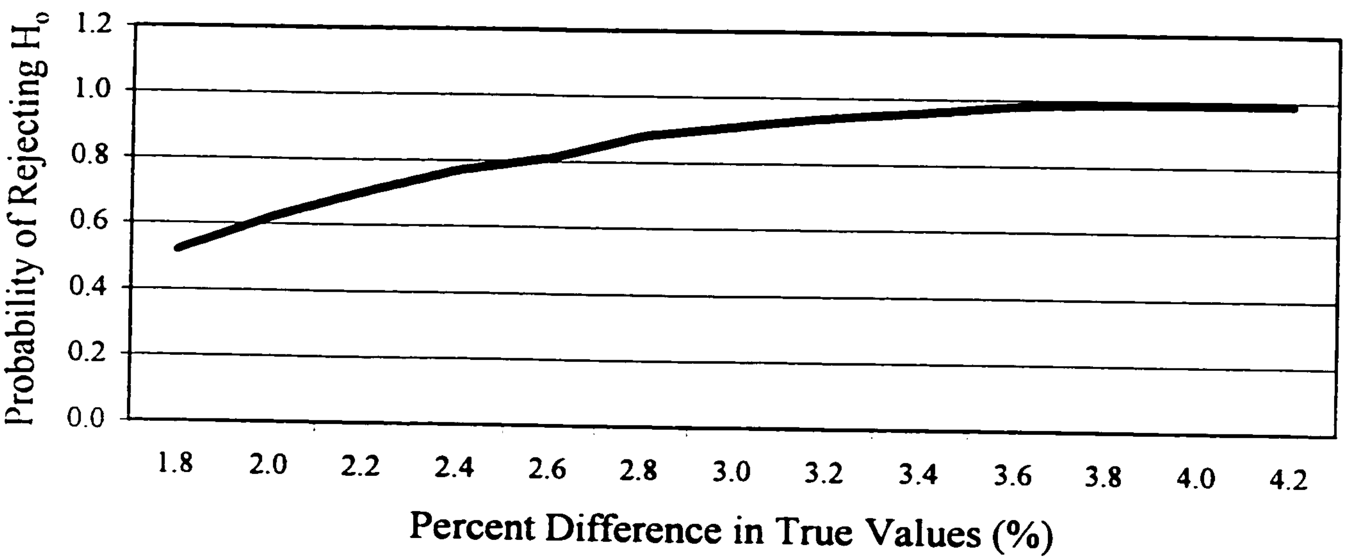


Figure 5.4 Operating Curve for Hajek Prediction and Observed Cracking

contraction coefficients as outlined in [Anderson 99]. However, the use of typical coefficients is often a reasonable procedure when direct measurements are not available [Anderson 99].

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

5.6.1 Thermal Contraction Coefficient Analysis Using C-LTPP Data

Figure 5.5 shows the influence of the thermal contraction coefficient on the transverse crack spacing for the C-LTPP sections, when all other variables remain constant. The figure indicates that as the thermal coefficient increases, the spacing between cracks decreases. Obviously, the thermal coefficient has a major influence on the crack spacing and accurate measurement would be desirable. The complete results are in Appendix B Table B.5. The cracking index is calculated as per equation 5.2 and the cracks per 150m has been calculated so that it corresponds with the observed cracking and Hajek model prediction.

The cracking predicted using the Canadian Airport Model [Haas 87] was compared to the observed cracking as summarized in Table 5.3. The values highlighted indicate the cracking that is closest to the observed for the various thermal contraction coefficients.

The second comparison uses the thermal coefficients provided for the C-SHRP test sections located in Lamont Alberta, Hearst Ontario and Sherbrooke Quebec. For the majority of cases, the Airport model appears to be a conservative cracking prediction method.

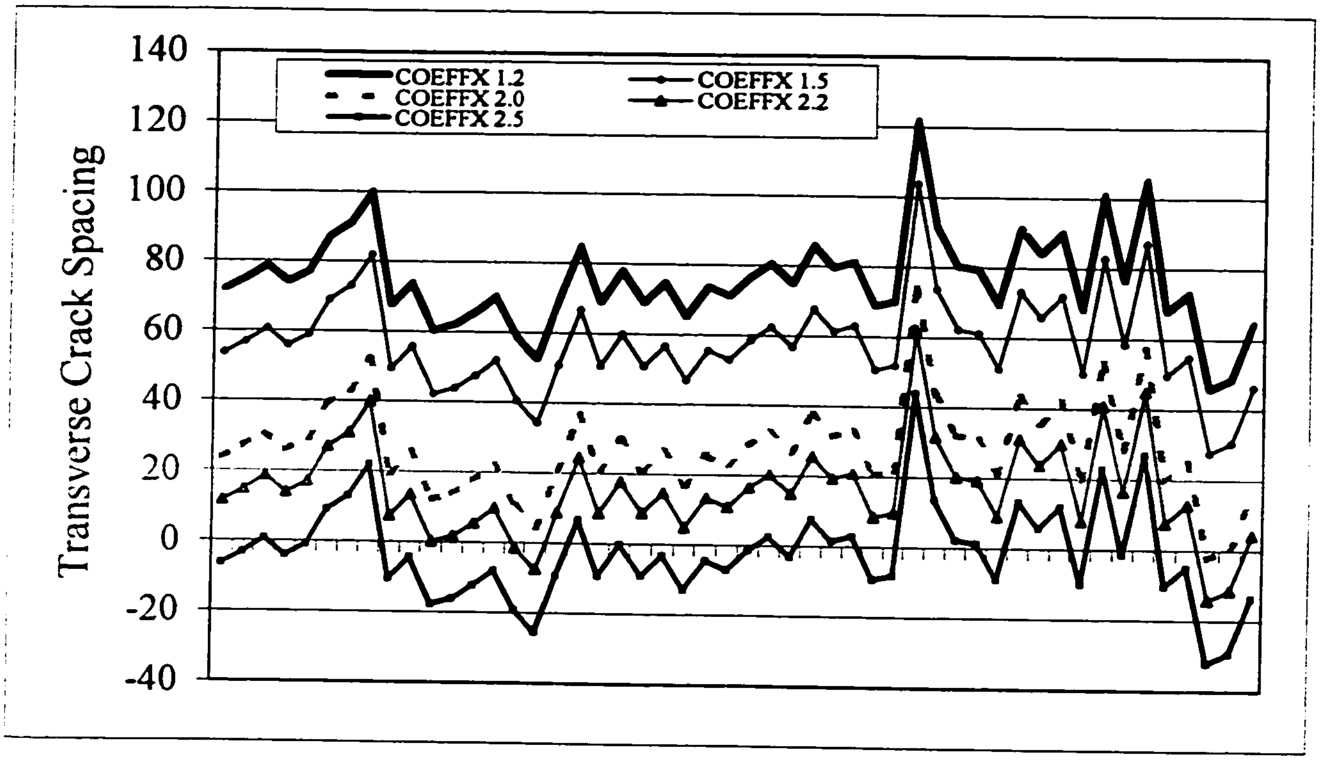


Figure 5.5 Influence of Thermal Coefficients on Transverse Crack Spacing for the C-LTPP Sections

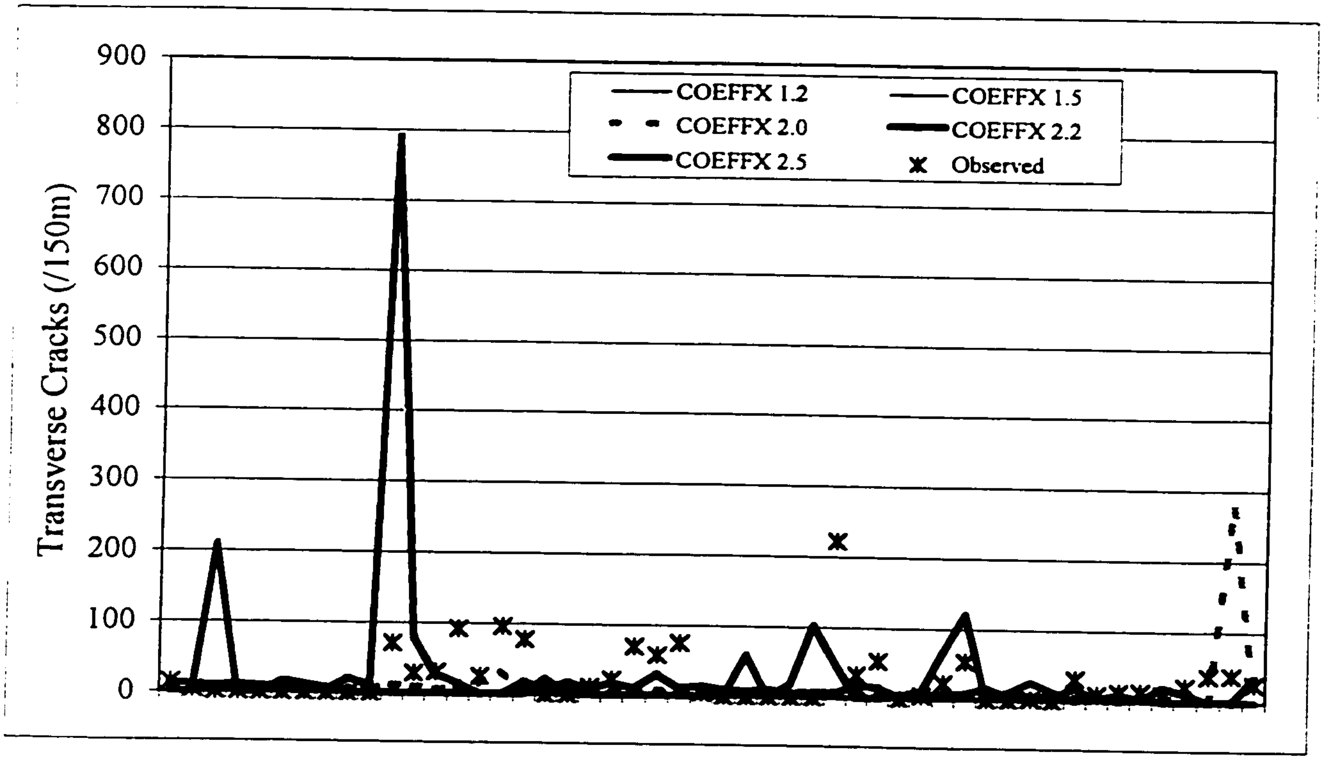


Figure 5.6 Transverse Crack Prediction Using Various Coefficients Compared to Observed Cracking

Table 5.3 Transverse Cracking Predictions (Page 1 of 2)

Test Site	Section	PVN	Thickness (mm)	Minimum Temperature (°C)	Cracks (/150m)* Using Various Coefficient of Thermal Contraction**					Observed Cracks (/150m)
					1.2	1.5	2.0	2.2	2.5	
810404	1	-0.22	61	-30	2.1	2.8	6.3	**12.9**	•	14.3
	2	-0.29	103	-30	2.0	2.6	**5.6**	10.2	•	4.4
	3	-0.12	94	-30	**1.9**	2.5	4.9	8.0	210.1	1.7
	4	-0.12	55	-30	**2.0**	2.7	5.8	10.7	•	1.9
820205	1	-1.22	42	-15	**1.9**	2.5	5.2	8.8	•	0.5
	2	-1.06	83	-15	**1.7**	2.2	3.8	5.5	16.5	0.0
820502	1	-1.02	104	-15	**1.6**	2.1	3.5	4.8	11.5	0.0
	2	-0.78	118	-15	**1.5**	1.8	2.9	3.7	6.8	0.0
820605	1	-1.15	50	-20	**2.2**	3.0	7.7	19.9	•	0.0
	2	-1.04	73	-20	**2.0**	2.7	5.8	10.9	•	0.0
830403	1	-0.35	100	-35	2.5	3.6	**12.3**	789.5	•	71.5
	2	-0.35	113	-35	2.4	3.4	**10.8**	78.9	•	29.0
	3	-0.37	148	-35	2.3	3.2	8.5	**26.9**	•	30.9
830801	1	-0.38	185	-35	2.1	2.9	6.8	**14.8**	•	92.6
	2	-0.41	103	-35	2.6	3.7	**14.1**	•	•	25.4
	3	-0.71	126	-35	2.9	4.3	**33.1**	•	•	96.6
	4	-0.36	170	-35	2.2	3.0	7.2	**17.1**	•	78.2
840101	1	-0.85	174	-23	**1.8**	2.2	4.1	6.1	22.4	0.0
	2	NA	179	-23	NA	NA	NA	NA	NA	0.0
	3	-1.01	87	-23	**2.2**	3.0	7.2	17.0	•	0.7
840204	1	-0.74	114	-24	1.9	2.5	5.0	**8.4**	•	11.8
	2	-0.93	88	-24	2.2	2.9	7.2	**16.9**	•	22.8
840604	1	-0.73	107	-25	2.0	2.6	5.6	**10.2**	•	69.9
	2	-0.98	90	-25	2.3	3.2	8.8	**29.8**	•	58.6
	3	NA	30	-25	NA	NA	NA	NA	NA	64.3
	4	-0.47	35	-25	2.0	2.7	5.9	**11.1**	•	75.2
850201	1	NA	117	-30	NA	NA	NA	NA	NA	0.0
	2	NA	73	-30	NA	NA	NA	NA	NA	0.0
850206	1	-0.43	106	-30	2.1	2.8	**6.5**	13.7	•	5.2
	2	NA	63	-30	NA	NA	NA	NA	NA	19.8
850601	1	NA	122	-30	NA	NA	NA	NA	NA	0.0
	2	NA	74	-30	NA	NA	NA	NA	NA	0.0
860501	1	-0.70	55	-22	**2.0**	2.6	5.2	9.0	•	0.0
	2	-0.70	85	-22	**1.9**	2.4	4.6	7.3	59.9	0.0
	3	-0.70	41	-22	**2.0**	2.6	5.6	10.1	•	0.0
860603	1	-0.70	86	-20	**1.8**	2.2	4.0	5.8	19.5	0.0
	2	-0.68	80	-20	NA	NA	NA	NA	NA	0.0
	3	-0.68	34	-20	**1.9**	2.4	4.8	7.7	103.0	0.0
870102	1	-0.41	95	-20	NA	NA	NA	NA	NA	40.4
	2	-0.39	46	-20	1.9	2.4	4.5	7.1	**50.1**	66.8
870504	1	-0.61	31	-25	2.2	3.0	7.3	**17.3**	•	33.8
	2	-0.70	61	-25	2.1	2.9	6.8	**15.1**	•	54.0
870505	1	0.64	75	-21	**1.2**	1.4	2.0	2.4	3.4	0.0
	2	NA	73	-21	NA	NA	NA	NA	NA	49.0
	3	-0.49	105	-21	1.6	2.0	3.4	**4.7**	11.0	5.5
	4	-0.88	106	-21	1.9	2.4	4.7	**7.4**	67.8	23.8
870701	1	NA	43	-25	NA	NA	NA	NA	NA	23.8
	2	-0.58	106	-25	1.9	2.4	4.8	7.8	**120.7**	54.6
880203	1	-1.10	51	-20	**2.2**	2.9	7.1	16.4	•	0.8
	2	-0.61	109	-20	**1.6**	2.0	3.5	4.8	11.3	0.0
	3	-0.61	51	-20	1.8	**2.3**	4.2	6.3	25.6	2.5
880203	4	-0.61	99	-20	**1.7**	2.1	3.6	5.0	12.5	0.0
890503	1	-0.50	40	-27	2.2	3.0	7.5	**18.6**	•	32.2
	2	0.55	51	-27	1.5	1.8	**2.8**	3.7	**6.5**	9.7

airport.xls
CrackFrequency

Table 5.3 Transverse Cracking Predictions (Page 2 of 2)

Test Site	Section	PVN	Thickness (mm)	Minimum Temperature (°C)	Cracks (/150m)* Using Various Coefficient of Thermal Contraction**					Observed Cracks (/150m)
					1.2	1.5	2.0	2.2	2.5	
890503	3	-0.50	106	-27	2.0	2.6	5.3	**9.1**	•	14.0
	4	0.55	83	-27	1.4	1.7	2.6	3.3	**5.5**	15.0
890702	1	-0.45	44	-28	2.2	3.0	**7.6**	19.6	•	10.9
	2	-0.45	85	-28	2.1	2.7	6.0	**11.6**	•	22.7
900402	1	-0.77	86	-35	3.3	**5.4**	•	•	•	37.1
	2	NA	126	-35	NA	NA	NA	NA	NA	12.9
900802	1	-0.54	55	-35	3.1	**4.9**	275.7	•	•	38.3
	2	NA	67	-35	NA	NA	NA	NA	NA	32.1
	3	NA	104	-35	NA	NA	NA	NA	NA	26.2
900803	1	-0.45	158	-35	2.3	3.2	9.2	**34.2**	•	26.4
	2	NA	108	-35	NA	NA	NA	NA	NA	55.2

Notes: NA means not available, * denotes a negative prediction, ** bolded values are closed to observed cracking

Test Site	Section	PVN	Thickness (mm)	Minimum Temperature (°C)	Coefficient	Cracks (/150m)	Actual Cracks (/150m)
Lamont	1	-0.74	120	-40	1.6	10.4	12.0
	2	-0.74	97	-40	1.6	13.1	17.9
	3	-0.17	121	-40	1.6	4.7	0.0
	4	-1.22	105	-40	1.2	6.8	18.1
	5	0.05	104	-40	1.6	4.2	7.1
	6	-0.09	115	-40	2.7	-4.6	2.9
	7	-0.30	110	-40	1.6	3.2	1.0
Hearst	AA	-0.27	54	-35	1.8	7.3	0.6
	A	-0.27	50	-35	1.8	7.5	0.3
	B	-0.42	59	-35	1.8	9.0	0.6
	BB	-0.42	100	-35	1.8	6.8	1.6
Sherbrooke	A	0.04	120	-35	1.5	2.7	1.2
	B	-0.70	120	-35	1.5	4.4	6.8
	C	0.12	120	-35	1.5	2.6	4.9
	D	-0.72	120	-35	1.5	4.5	1.6

Figure 5.6 shows the comparison between the transverse cracks predicted for the C-LTPP test sections and the observed cracks. When the observed cracking on the various test sections is greater than 70 cracks per 150m it indicates that the cracking prediction for the airport model for the various thermal coefficients is not very close to the observed. One possible explanation for this occurrence is that when more than 70 cracks occur, this is outside the range of the data used to develop the model.

Based on this discrepancy, two ANOVA's were run to compare the airport cracking prediction to the observed cracking as summarized in Tables 5.4, 5.5 and B.6.

Table 5.4 ANOVA Summary for Airport Prediction and Observed Cracking

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

Notes: 1) Uses C-LTPP test data.

2) Represents the difference between test sections where the null hypothesis states all section are equal.

3) Represents the differences in Airport prediction and observed cracking where the null hypothesis states regardless of how cracking is determined (observed or predicted), the true values will be equal.

Table 5.4 summarizes the ANOVA when all sections are included where both the airport prediction can be made and observed values are available. The differences between the airport model prediction and the observed are shown to be statistically significant. Thus the model does not appear to predict results close to those observed using typical thermal contraction coefficients.

When the ANOVA is rerun (Table 5.5) excluding the sections where more than 70 cracks have been observed, the differences between the model and observed are shown to be not statistically significant, with a p value of 0.0687 or 6.87% chance that the result will be greater than the F calculated value. Consequently, it reinforces the fact that the Airport model appears to be conservative in predicting cracking. The Type II error was assessed on this result.

Table 5.5 ANOVA for Airport Prediction and Observed Cracking (< 70 Cracks)

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

Notes: 1) Uses C-LTPP test data.

2) Represents the difference between test sections where the null hypothesis states all section are equal.

3) Represents the differences in Airport prediction and observed cracking where the null hypothesis states regardless of how cracking is determined (observed or predicted), the true values will be equal.

The operating curve for the power test based on the ANOVA in Table 5.5 is presented in Figure 5.7. The complete results are in Appendix B, Table B.4. At the 95% rejection rate of the null hypothesis, the difference between the true values would only be 7.5%. This difference is small and it reinforces that the Airport Model [Haas 87] is a good model for predicting cracking when less than 70 cracks occur. However, as pointed out in Table 5.4, when more than 70 cracks occur it is not a good predictor.

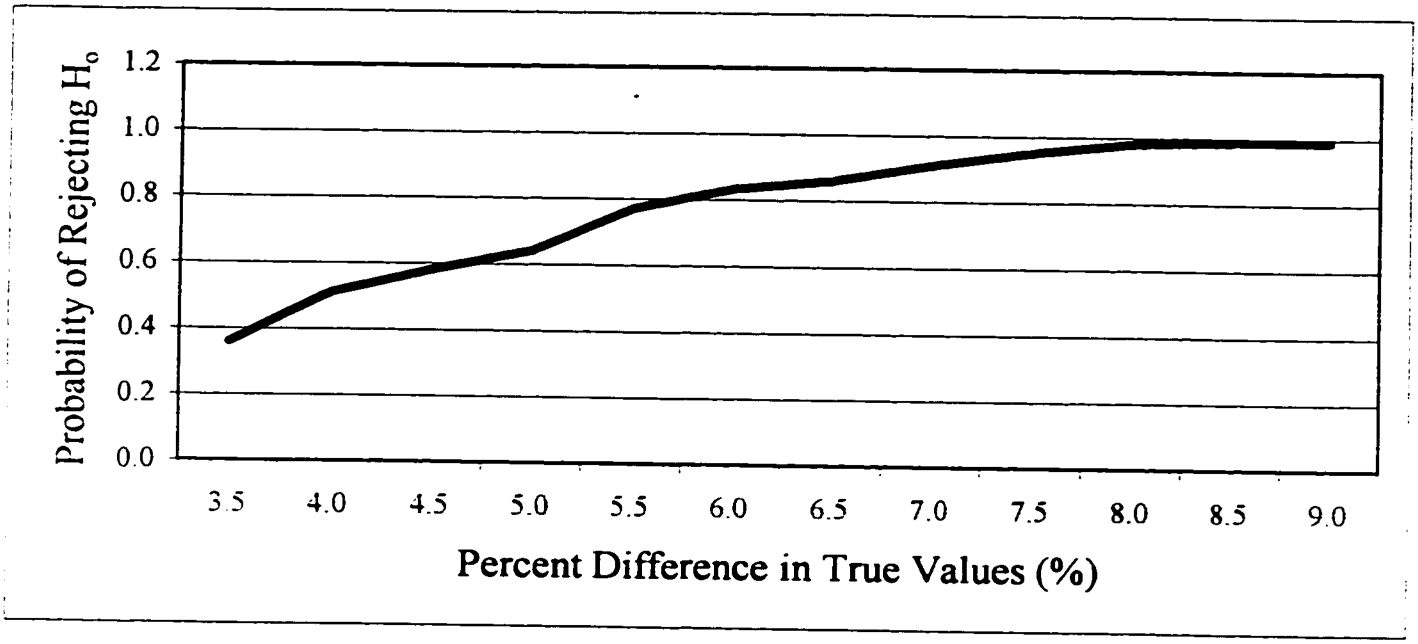


Figure 5.7 Operating Curve for Airport Model Prediction and Observed Cracking

5.6.2 Thermal Contraction Coefficient Analysis Using C-SHRP Test Sites

The measured thermal coefficients, as provided by Anderson [Anderson 99] for the three C-SHRP test roads located in Lamont Alberta, Hearst Ontario and Sherbrooke Quebec, were used to predict cracking at the minimum temperature as shown in Table 5.3. The thermal coefficient based on the observed cracking was also calculated and an ANOVA, Table B.7 and Table 5.6 compare the two coefficients for C-SHRP test roads. Only those test roads where greater than 1 crack per 150 m was included. The results indicate that the differences between the measured thermal contraction coefficient and the calculated thermal contraction coefficient based on the observed cracking are not statistically significant. Additionally, the high p value (.8838) indicates that there is a strong indication that the measured value is consistent with the value based on the observed cracking on the various test sections.

Table 5.6 ANOVA for Airport Prediction and Observed Cracking (< 100 Cracks)

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

Notes: 1) Uses C-SHRP test sections.

2) Represents the difference between test sections where the null hypothesis states all section are statistically equal.

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

5.7 THREE WAY COMPARISON

Table 5.7 summarizes the cracking predictions for the Hajek model, Canadian Airport model and the observed cracking on the various test sections. The thermal coefficient calculated based on

Table 5.7 Comparison of Transverse Crack Prediction (Page 1 of 2)

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

Table 5.7 Comparison of Transverse Crack Prediction (Page 2 of 2)

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

Notes: NA means not available

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

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

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

the observed cracking is summarized in the last column. For the C-LTPP numbered sites (test site 810404 through 900803), the thermal coefficients range between 0.1 to 2.78. The 0.1 to .6 range is based on a very small amount of observed cracking. The majority of the values appear reasonable based on the literature [Anderson 99].

Table 5.8 summarizes the differences between the models in terms of the number of transverse cracks. Three comparisons were made. The Airport model versus the observed cracking, the Hajek model versus the observed cracking and the Airport and Hajek model comparison. The bolded values represent the smallest difference, or in other words the closest prediction. For example, for the test site 810404, Section 1, the Airport model and Hajek model showed the closest comparison with a difference of only 0.4 transverse cracks. The difference between the Airport model and observed is 1.4 cracks and Hajek and observed showed 1.0 cracks.

The third comparison, the Hajek model and Airport model comparison was included as it indicates those cases where both these models predicted similar performance while the observed cracking was different. The average and standard deviations of the differences are also presented at the bottom of Table 5.8. When all test sections are included in the analysis, the Hajek model and the observed cracking show the closest average prediction. This is closely followed by the Airport prediction comparison to the observed cracking. On average, the Hajek model slightly over predicted the observed cracking by 4.5 transverse cracks, while the Airport model under predicted cracking by 8.4 transverse cracks.

Based on the large standard deviation values, the average and standard deviation values were also calculated for cases where less than 70 cracks occurred. This analysis indicates that the Airport model and observed values were very close (-0.8 difference) on average with a standard deviation of 17.8. It also shows that the differences between the Hajek and observed cracking are small, only 10.1 transverse cracks and for the Airport and Hajek models only 10.9 transverse cracks.

Figure 5.8 summarizes the crack comparison between the two cracking prediction models and the observed cracking for all sections. This is a summary of the number of times the comparison (i.e.

Table 5.8 Comparison of Transverse Cracking Models with Observed Cracking (Page 1 of 2)

Test Site	Section	Airport Model Crack Prediction (/150m)	Hajek Prediction (/150m)	Actual Cracks (/150m)	Transverse Cracking Comparison[1]		
					Airport & Observed[2]	Hajek & Observed[2]	Airport & Hajek[3]
810404	1	12.9	13.3	14.3	-1.4	-1.0	**-0.4**
	2	5.6	6.8	4.4	**1.2**	2.4	**-1.2**
	3	2.5	12.7	1.7	**0.8**	11.0	-10.2
	4	2.7	12.8	1.9	**0.8**	10.9	-10.1
820205	1	1.9	12.8	0.5	**1.4**	12.3	-10.9
	2	1.7	9.7	0.0	**1.7**	9.7	-8.0
820502	1	1.6	10.0	0.0	**1.6**	10.0	-8.4
	2	1.5	8.8	0.0	**1.5**	8.8	-7.3
820605	1	2.2	9.2	0.0	**2.2**	9.2	-7.0
	2	2.0	4.7	0.0	**2.0**	4.7	-2.7
830403	1	12.3	53.9	71.5	-59.2	**-17.6**	-41.6
	2	78.9	32.7	29.0	49.9	**3.7**	46.2
	3	26.9	59.9	30.9	**-4.0**	29.0	-33.0
830801	1	14.8	60.2	92.6	-77.8	**-32.4**	-45.4
	2	14.1	11.9	25.4	-11.3	-13.5	**2.2**
	3	33.1	95.8	96.6	-63.5	**-0.8**	-62.7
	4	17.1	45.8	78.2	-61.1	-32.4	**-28.7**
840101	1	6.1	9.8	0.0	6.1	9.8	**-3.7**
	3	2.2	12.7	0.7	**1.5**	12.0	-10.5
840204	1	8.4	33.9	11.8	**-3.4**	22.1	-25.5
	2	16.9	45.2	22.8	**-5.9**	22.4	-28.3
840604	1	10.2	49.3	69.9	-59.7	**-20.6**	-39.1
	2	29.8	26.2	58.6	-28.8	-32.4	**3.6**
	4	11.1	54.6	75.2	-64.1	**-20.6**	-43.5
850206	1	6.5	5.5	5.2	1.3	**0.3**	1.0
860501	1	2.0	4.1	0.0	**2.0**	4.1	-2.1
	2	1.9	7.2	0.0	**1.9**	7.2	-5.3
	3	2.0	8.2	0.0	**2.0**	8.2	-6.2
860603	1	1.8	11.2	0.0	**1.8**	11.2	-9.4
	3	1.9	12.5	0.0	**1.9**	12.5	-10.6
870102	2	50.1	21.9	222.5	-172.4	-200.6	**28.2**
870504	1	17.3	41.9	33.8	-16.5	**8.1**	-24.6
	2	15.1	70.0	54.0	-38.9	**16.0**	-54.9
870505	1	1.2	9.0	0.0	**1.2**	9.0	-7.8
	3	4.7	11.8	5.5	**-0.8**	6.3	-7.1
	4	67.8	27.1	23.8	44.0	**3.3**	40.7
870701	2	120.7	17.8	54.6	66.1	**-36.8**	102.9
880203	1	2.2	13.0	0.8	**1.4**	12.2	-10.8
	2	1.6	11.7	0.0	**1.6**	11.7	-10.1
	3	4.2	13.0	2.5	**1.7**	10.5	-8.8
	4	1.7	11.9	0.0	**1.7**	11.9	-10.2
890503	1	18.6	41.4	32.2	-13.6	**9.2**	-22.8
	2	6.5	14.7	9.7	**-3.2**	5.0	-8.2
	3	9.1	17.3	14.0	-4.9	**3.3**	-8.2
	4	5.5	17.0	15.0	-9.5	**2.0**	-11.5

Table 5.8 Comparison of Transverse Cracking Models with Observed Cracking (Page 2 of 2)

Test Site	Section	Airport Model Crack Prediction (/150m)	Hajek Prediction (/150m)	Actual Cracks (/150m)	Transverse Cracking Comparison[1]		
					Airport & Observed[2]	Hajek & Observed[2]	Airport & Hajek[3]
890702	1	19.6	33.5	10.9	**8.7**	22.6	-13.9
	2	11.6	40.4	22.7	**-11.1**	17.7	-28.8
900402	1	5.4	48.3	37.1	-31.7	**11.2**	-42.9
900802	1	4.9	60.6	38.3	-33.4	**22.3**	-55.7
900803	1	34.2	39.8	26.4	7.8	13.4	**-5.6**
Lamont	1	10.4	67.2	12.0	**-1.6**	55.2	-56.8
	2	13.1	45.5	17.9	**-4.8**	27.6	-32.4
	3	4.7	1.1	0.0	4.7	**1.1**	3.6
	4	6.8	92.7	18.1	**-11.3**	74.6	-85.9
	5	4.2	16.5	7.1	**-2.9**	9.4	-12.3
	6	-4.6	1.7	2.9	1.7	**-1.2**	2.9
	7	3.2	8.3	1.0	**2.2**	7.3	-5.1
Hearst	AA	7.3	0.0	0.6	6.7	**-0.6**	7.3
	A	7.5	0.9	0.3	7.2	**0.6**	6.6
	B	9.0	2.1	0.6	8.4	**1.5**	6.9
	BB	6.8	2.6	1.6	5.2	**1.0**	4.2
Sherbrooke	A	2.7	5.8	1.2	**1.5**	4.6	-3.1
	B	4.4	57.8	6.8	**-2.4**	51.0	-53.4
	C	2.6	25.6	4.9	**-2.3**	20.7	-23.0
	D	4.5	32.2	1.6	**2.9**	30.6	-27.7
				Average	-8.4	**4.5**	-12.9
				Standard Deviation	31.9	31.7	27.1
				Average[4]	**-0.8**	10.1	-10.9
				Standard Deviation[4]	17.8	16.9	26.1

Notes: 1) Bolded Value Indicates Smallest Difference

2) Negative Values indicate the observed cracking is higher than Hajek model predicted

3) Negative value indicate the Hajek model predicted more cracking than the Airport model predicted

4) Includes Cases With Less Than 70 Cracks

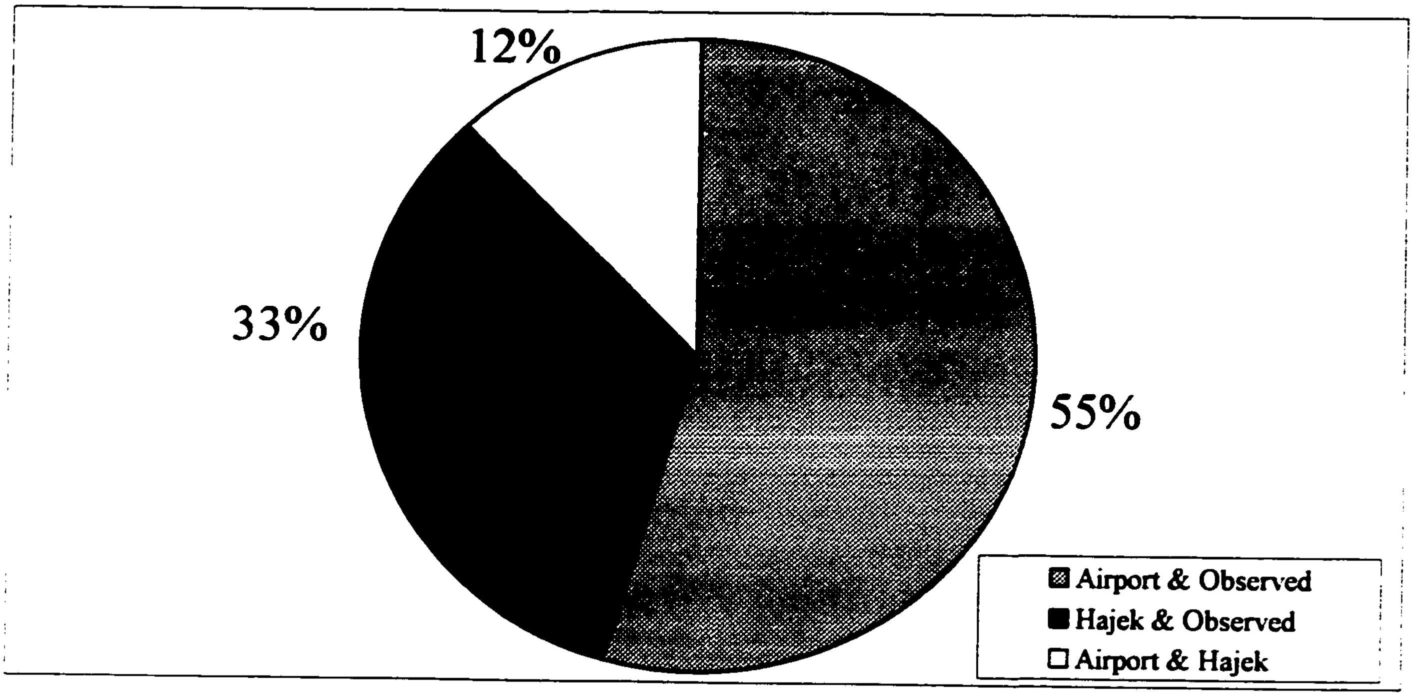


Figure 5.8 Cracking Model Comparison For C-LTPP Sections

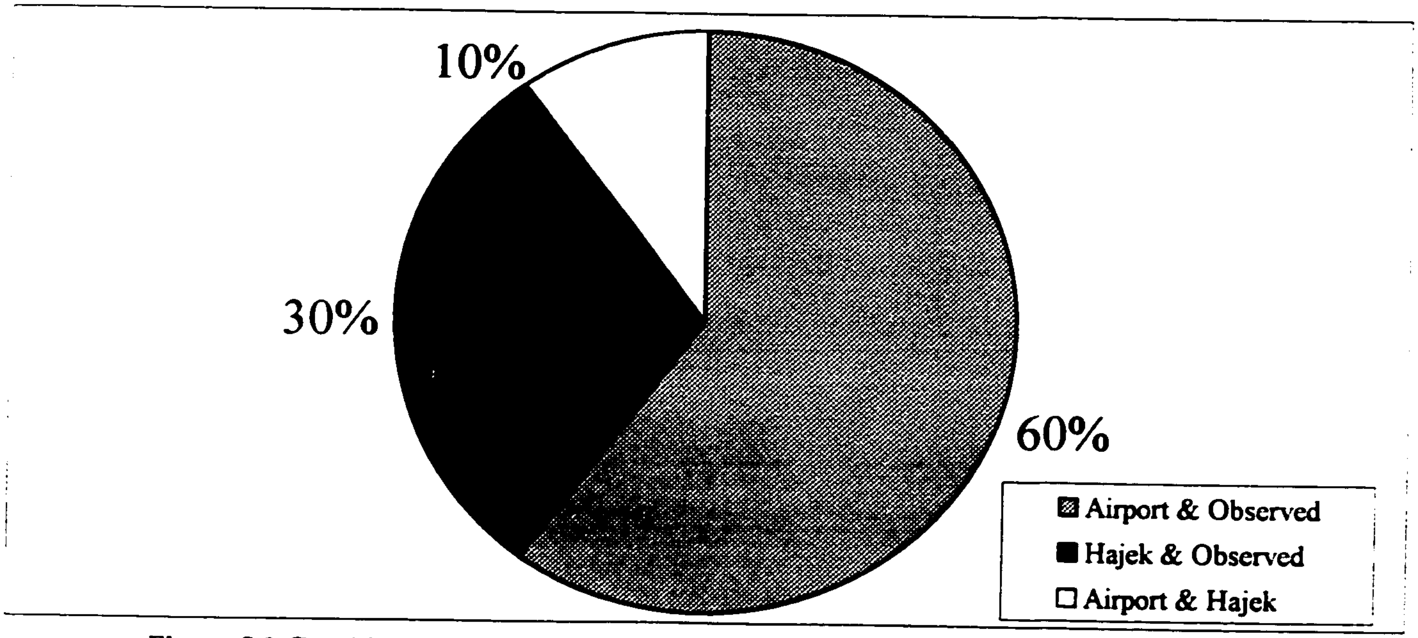


Figure 5.9 Cracking Model ComparisonWith Less Than 70 Cracks For C-LTPP Sections

Airport model and observed, Hajek model and observed and Airport and Hajek models) is the best (or the case where the comparison provides the smallest difference). As shown, for 55% of the test sections, the Canadian Airport and observed showed the smallest difference in cracking, while for 33% of the sections, the Hajek and observed showed the smallest difference. Based on this observation combined with the average and standard deviation values in Table 5.8, it appears that the Hajek model was close to the observed it was very close, particularly in cases where the amount of observed cracking was very small. However, when the observed cracking increased, the airport model was better at predicting observed cracking.

Figure 5.9 shows the crack comparisons for sections where less than 70 cracks were observed. Although it is similar to Figure 5.8, it indicates that in cases where less than 70 cracks were observed, that the models were predicting well (as observed by the decrease in the percent of times where the Airport and Hajek model has the closest predictions). Again, despite the small average difference between the Hajek model and observed cracking, the Airport and observed model show that for 60% of the sections, it is the best comparison.

Figures 5.10 and 5.11 express the magnitude of the cracking difference for all sections and those sections where less than 70 cracks were observed. This further emphasizes the fact that when a small amount of cracking is observed, the airport model is closest. However, as the amount of cracking observed increases, the Hajek model gives a closer prediction.

5.8 SUMMARY OF MODULE TWO

The analysis indicates that the Hajek model and Canadian Airport model show good correlation to the observed cracking. However, an update to the models based on the long term performance information would be desirable. In any case, the results provide sufficient confidence for their use in Module Three to predict performance.

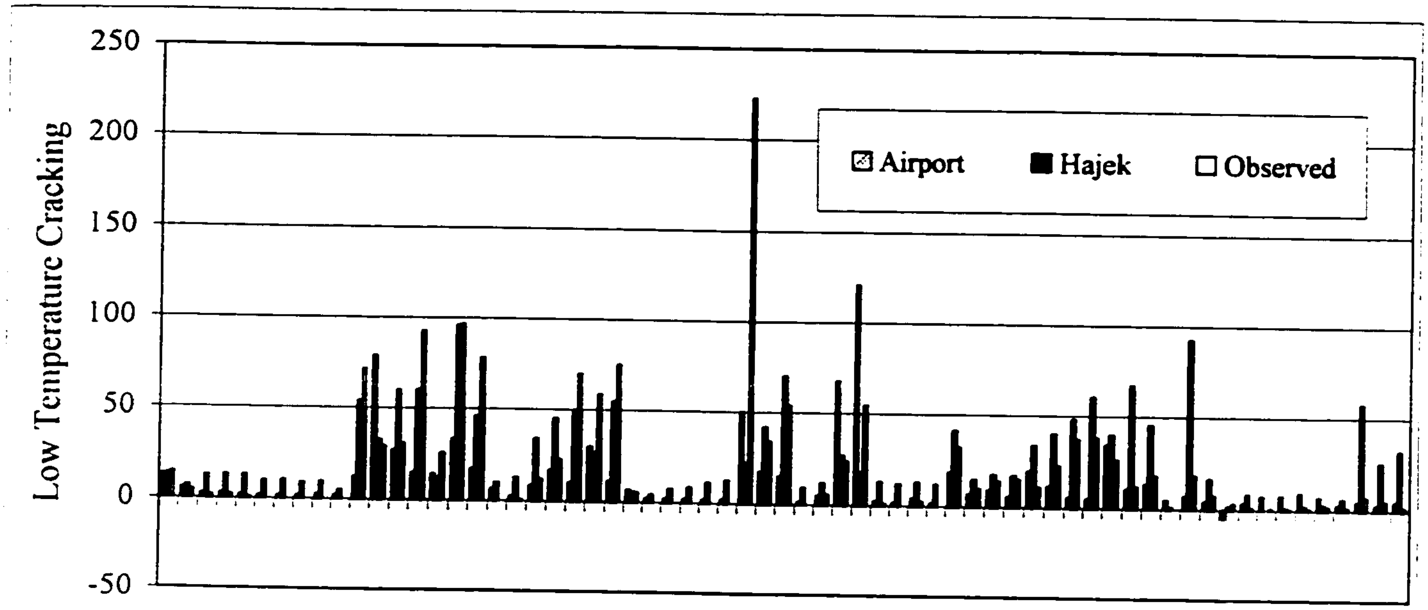


Figure 5.10 Comparison of Cracking Between Airport and Hajek Model For C-LTPP Sections

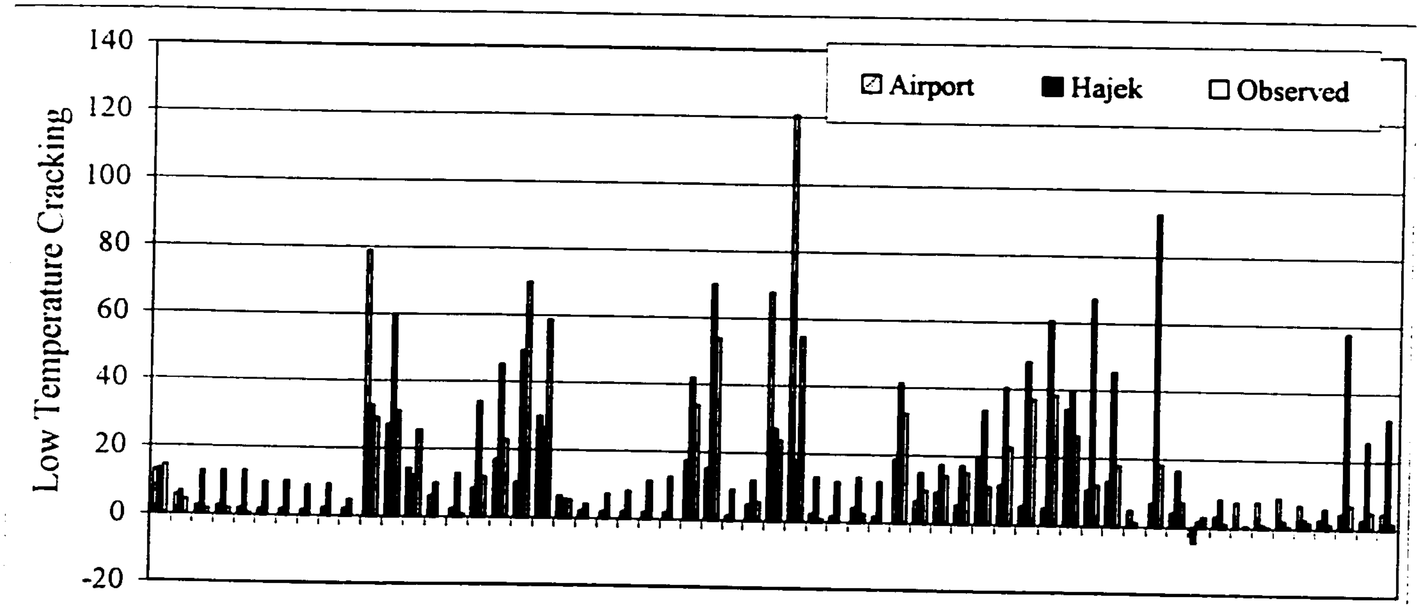


Figure 5.11 Comparison of Cracking (< 70 Cracks) Between Airport and Hajek Model

CHAPTER SIX MODULE THREE : PERFORMANCE PREDICTION

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

6.1 PAVEMENT DETERIORATION

The purpose of a performance model is to estimate the future deterioration of the various pavement sections and ultimately determine when the section will need to be rehabilitated [Haas 97]. Traditionally, the Riding Comfort Index (RCI) has been the selected method for characterizing roughness in Canada. This is based on a subjective measure which describes the pavement-vehicle-human interaction. Table 6.1 outlines the typical range of RCI values and their practical significance [MTO 89].

Using the RCI value, the remaining life can be calculated based on the functional classification (i.e., freeway, principal arterial, etc.) and the pavement factors (i.e. traffic level, subgrade, environment, etc.). Typical pavement deterioration is depicted in Figure 6.1. The minimum acceptable level for a pavement section will be determined based on the functional classification. For example a freeway would generally have a higher minimum acceptable level than a local road. If the current RCI level is known, then based on the pavement factors, the rate of deterioration and time of rehabilitation can be determined [Haas 97].

Table 6.1 Riding Condition Rating [MTO 89]

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

Many agencies use their own roughness measures. In an attempt to standardize roughness, the International Roughness Index (IRI) has been developed [Sayers 86]. It is the measure used for the C-LTPP sections.

The IRI is a measurement scale which simulates a standardized response type road roughness measuring system in terms of a quarter car. IRI uses the longitudinal road profile by accumulating the output from a quarter car model and dividing by the profile length to yield a summary roughness index with units of slope such as metres per kilometre (m/km). It has become recognized as a general purpose roughness index that is strongly correlated to most kinds of vehicle response type measuring devices. The IRI is linearly proportional to roughness and it describes profile roughness that causes vehicle vibrations. An IRI of 0.0 m/km means the profile is perfectly flat or smooth. An IRI of 8m/km would be nearly impassible to drive except at reduced speeds. Figure 6.2 shows the typical IRI ranges based on a World Bank Study [Sayers 86].

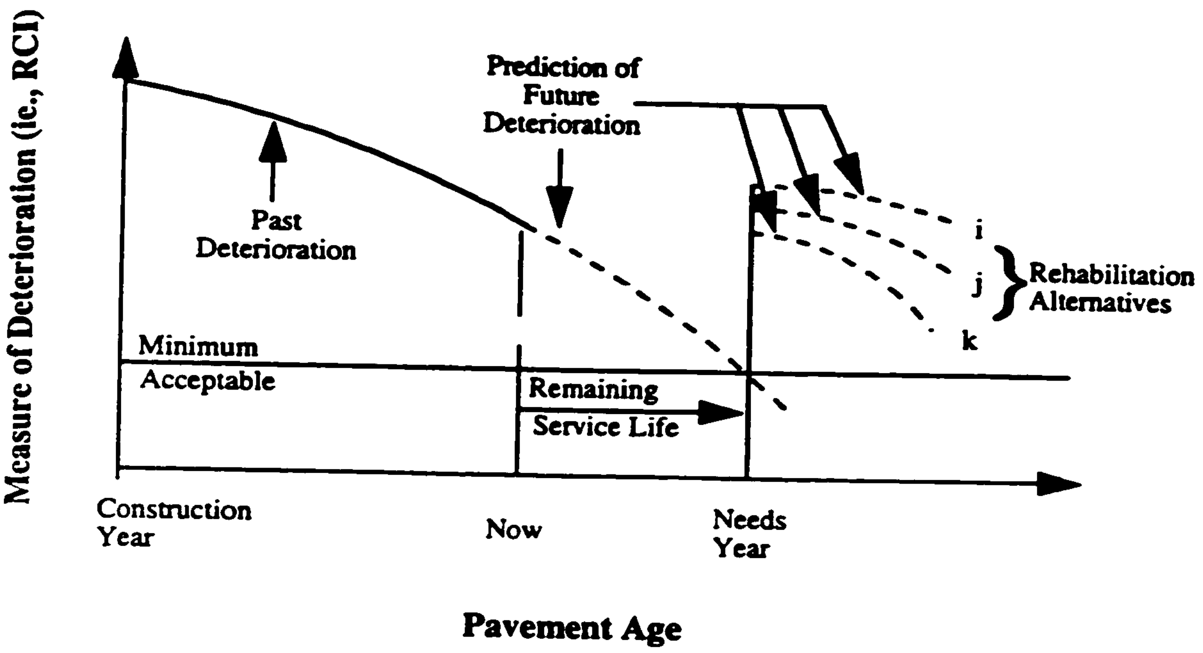


Figure 6.1 Schematic Illustration of Pavement Deterioration Prediction [Hill 90]

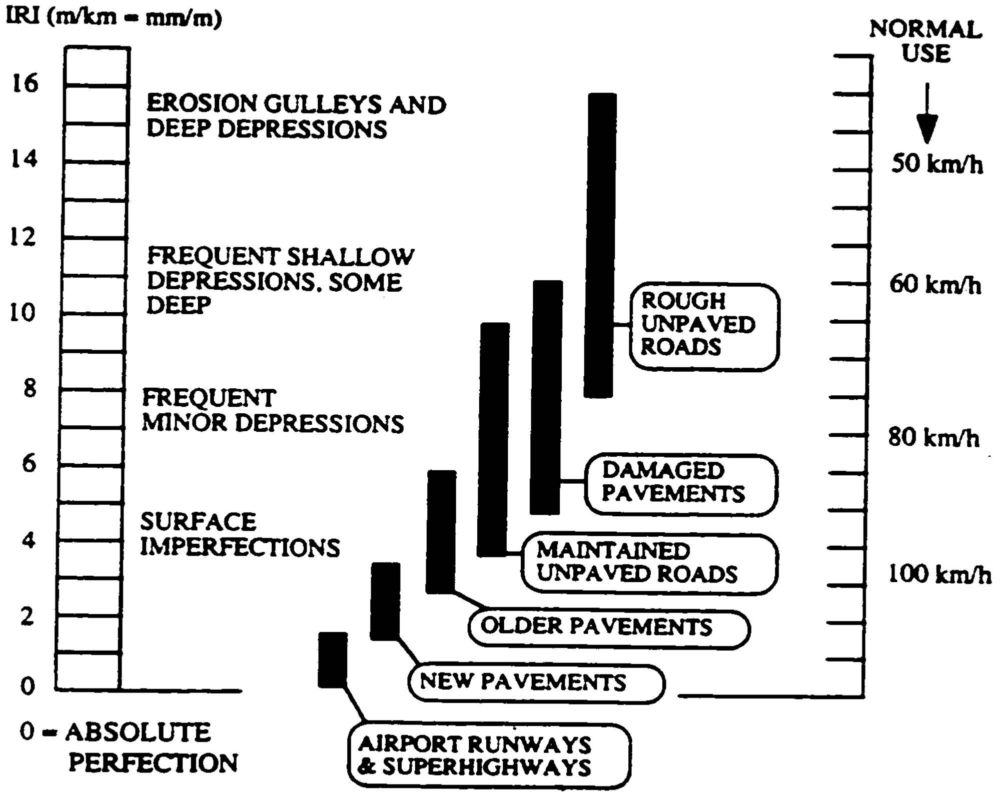


Figure 6.2 IRI Roughness Scale [Sayers 86]

6.2 RELATING LOW TEMPERATURE CRACKING TO ROUGHNESS

The results in Module Two focus on predicting low temperature cracking in a pavement structure. The intent of Module Three is to determine roughness based on cracking. This is a very difficult and challenging engineering task, as discussed in Chapter Two, because of all the confounding factors. In other words, the problem is to isolate the contribution of cracking to roughness progression or loss of performance. It appears that the only project able to do this involved the Canadian Airport Study [Haas 87], which developed the model of equation 6.1. The equation relates the transverse cracking to the RCI.

$$\text{RCI} = 5.4 + 0.02\ \text{TRANCRAK} - 11.6 / (\text{TRANCRAK})^2 \qquad (6.1)$$

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

TRANCRAK = Transverse crack spacing, m

Cracking represents damage to the pavement. This damage is often associated with reduced in-service life. However, very little data is available which isolates the effect of cracking on performance. One of the major problems in isolating this effect, is that there are so many other factors which can influence pavement performance. An example of such factors and their interaction are given in Figure 6.3 [Haas 99a].

Equation 6.1 is shown in graphical terms in Figure 6.4 [Haas 91]. It clearly indicates that pavement life is increased with increased crack spacing (i.e. lower frequency of cracking). For example, five years longer life is obtained if the crack spacing increases from 5m to 20m, where the 5m spacing corresponds to the normal initial design life of 15 years for these pavements. This increased life has significant implications with regards to the life cycle cost of the pavement.

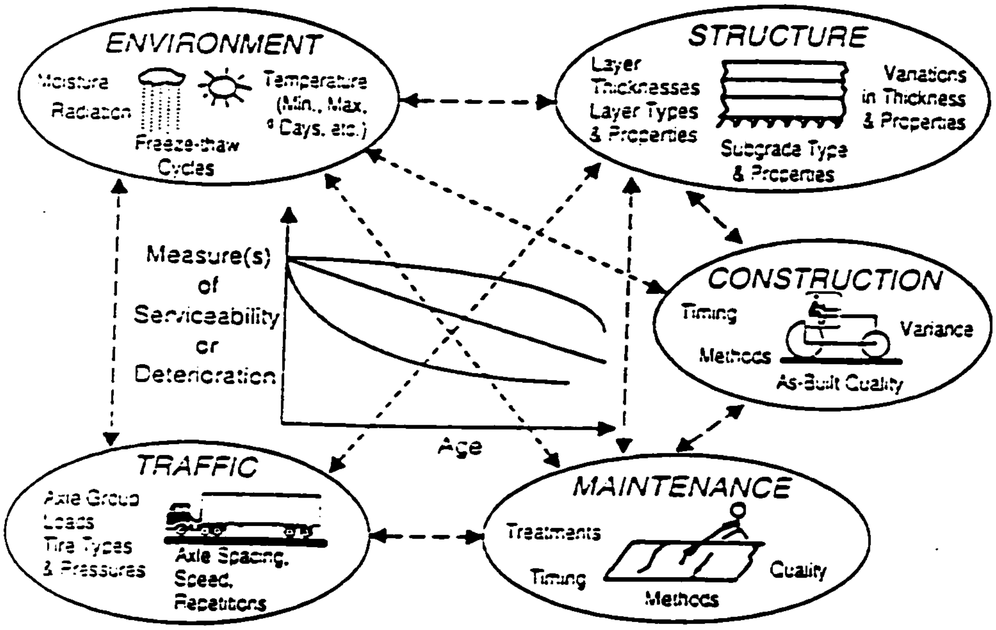


Figure 6.3 Factors and Interactions Which Can Affect Pavement Performance [Haas 99a]

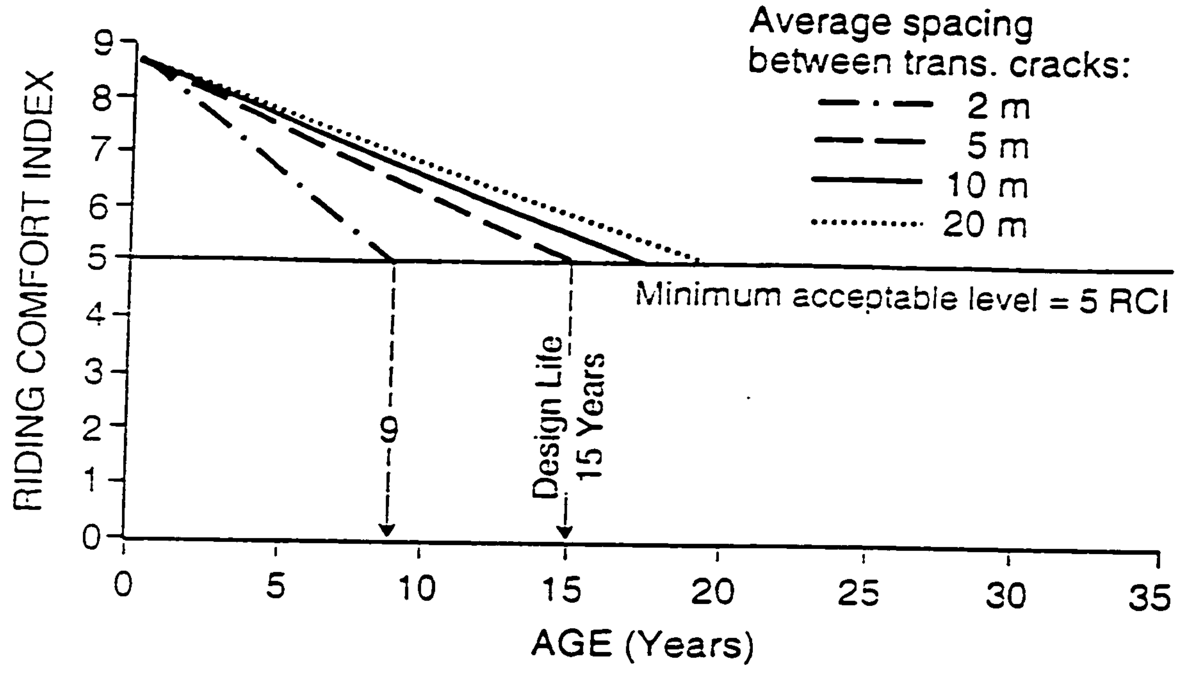


Figure 6.4 Influence of Cracking on Pavement Life [Haas 91]

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

6.3 RELATING RCI/RCR TO IRI

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

$IRI = 5.588 - 0.578 * RCI$ [Hein 89] (6.2)

$RCI = 10 * e^{-0.18IRI}$ [Paterson 86] (6.3)

$RCI = 10 * e^{-0.26IRI}$ [Al-Omari 84] (6.4)

$RCR_{carusers} = 9.11 - 1.39 * IRI$ [Hajek 95] (6.5)

$RCR_{truckusers} = 9.37 - 1.71 * IRI$ [Hajek 95] (6.6)

Where: IRI = International Roughness Index
RCI = Riding Comfort Index
RCR = Riding Comfort Rating

Only equation 6.2 uses RCI as an independent variable to estimate IRI. Thus, it is the only "valid" one in a statistical sense. However, the others, equations 6.3 to 6.6 were also used, with full recognition of this caveat.

6.4 DATA USED FOR MODULE THREE

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

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

6.5 ANALYSIS OF C-LTPP SECTIONS

The analysis procedure for Module Three is outlined in Figure 6.5. The RCI values for the C-LTPP sections were calculated using observed thermal cracking. IRI's were then calculated using equations 6.2 through 6.6 for the various sections. The observed IRI's for the C-LTPP sections are then compared to the predicted IRI values using the equations.

Table 6.2 summarizes the RCI and IRI calculations using the aforementioned equations. The observed IRI values are also included. The sections included in this summary are only those with less than 100 observed cracks, with RCI values between 0 and 10 and sections where all the IRI values calculated using the equations were positive. It is apparent that the observed IRI values on the C-LTPP sections are consistently less than predicted.

Figure 6.6 indicates that the observed cracking is not well correlated to the observed IRI. It would be expected that as the amount of observed cracking increases, the observed IRI would increase. However, based on this figure there does not seem to be any relationship between the observed cracking and observed IRI. The reason is not apparent, other then it is early in the life

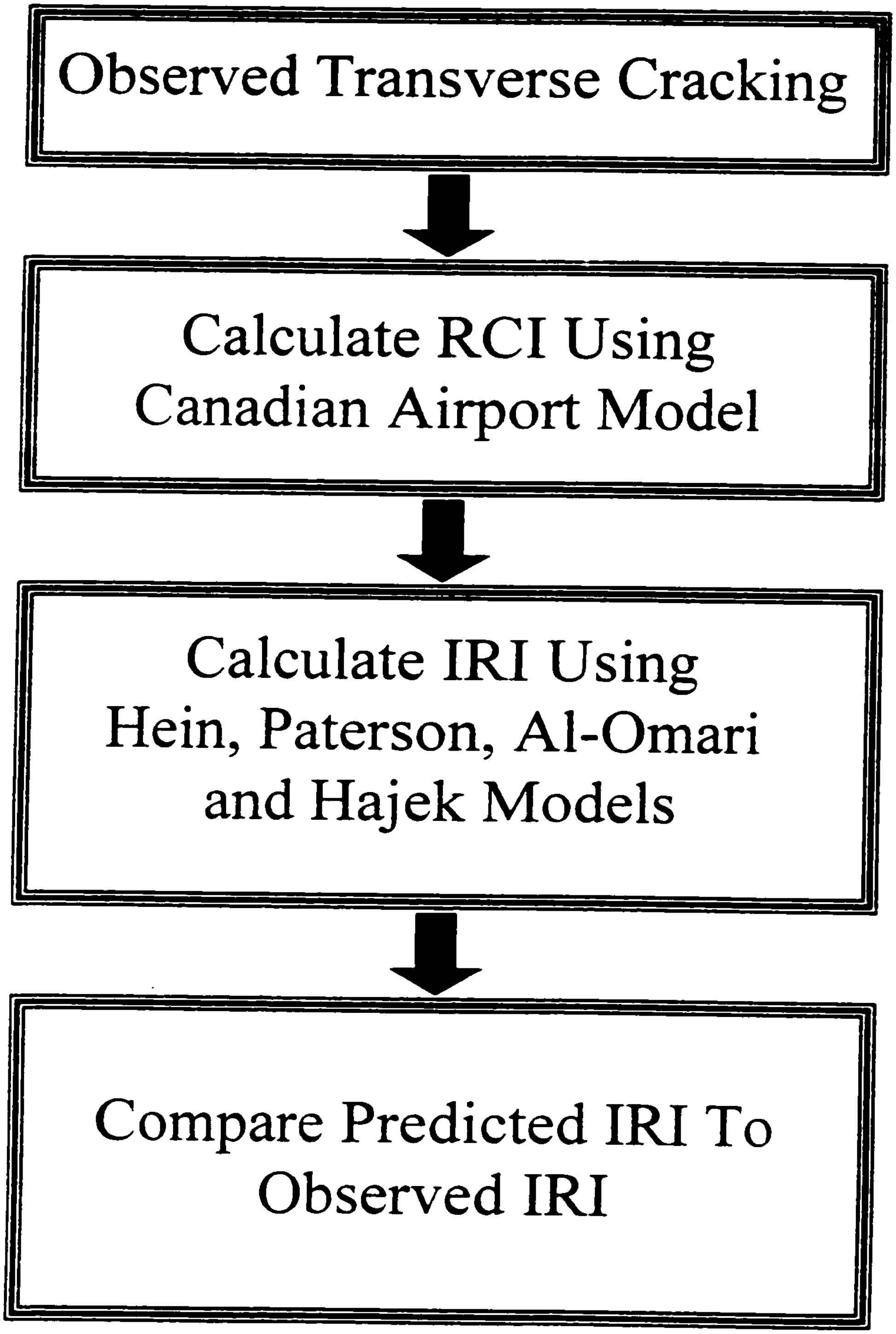


Figure 6.5 Analysis Procedure Module Three

Table 6.2 IRI Comparison

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

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

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

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

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

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

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

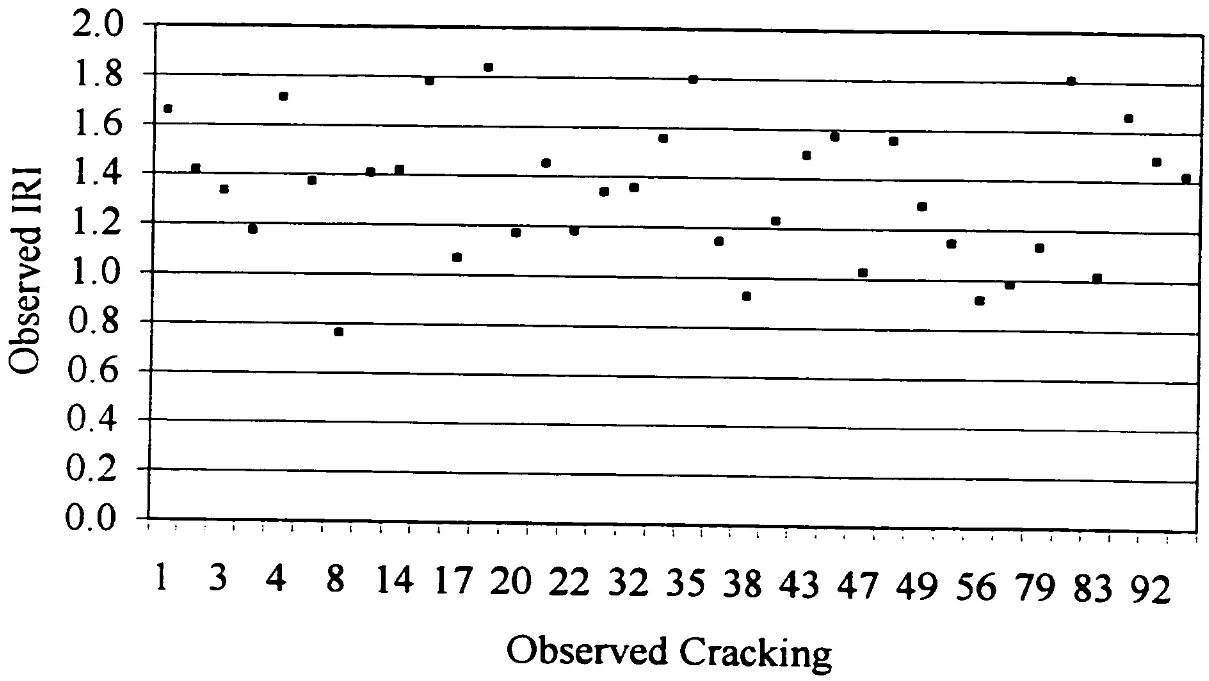


Figure 6.6 Relationship Between IRI and Observed Cracking

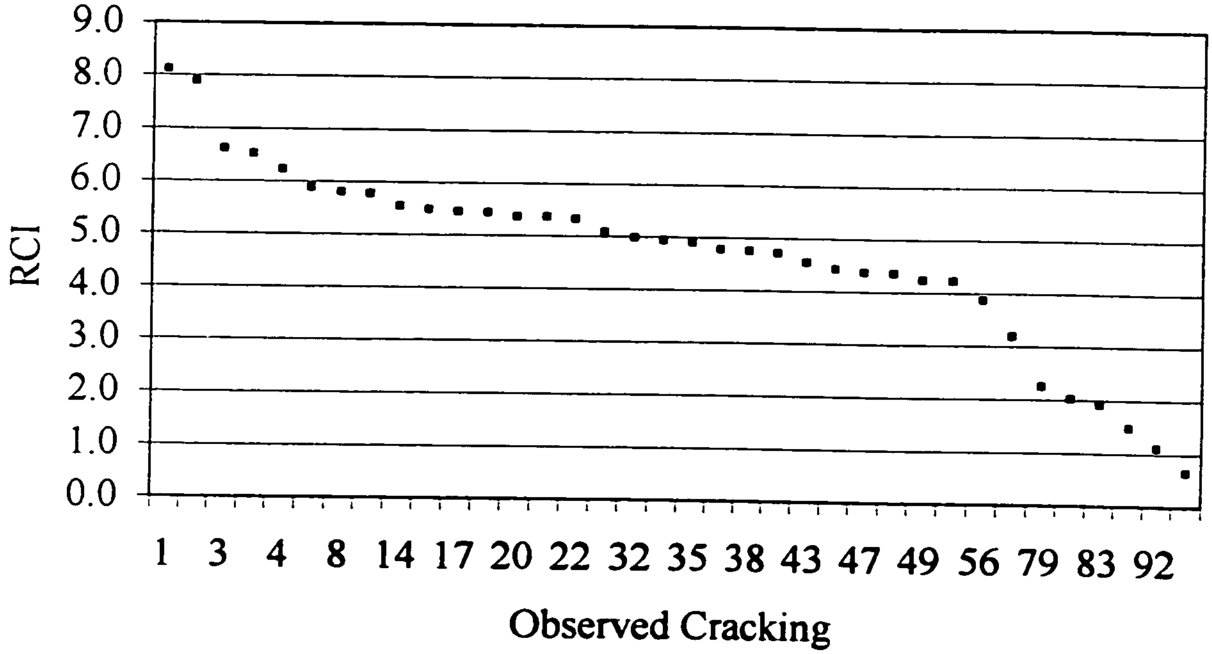


Figure 6.7 Relationship Between RCI and Observed Cracking

of the sections and the IRI's all are "clustered" in a fairly narrow range. Figure 6.7 shows that the calculated RCI using the Canadian Airport Model decreases with increased cracking (as would be expected) in a form which appears to be sinusoidal. Based on these initial findings, two subsequent analyses were performed. Five ANOVA's were performed and a cracking grouping based on the observed cracks were performed.

An ANOVA for the calculated IRI, using equations 6.2 through 6.6, versus the observed IRI values was performed. The complete summary is found in Appendix C Table C.1 to C.5 while the summarized results are in Table 6.3. All five of the IRI predictions are shown to be statistically different as compared to the observed IRI values ($F_{CALCULATED} > F_{CRITICAL}$).

Table 6.3 ANOVA for IRI Prediction and Observed IRI

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

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

Based on the findings, another ANOVA was performed. This ANOVA focused on the situation where less than 10 cracks were observed. The observed IRI was then compared to the predicted. Based on the results presented in Table 6.4 and Table C.6, when there were less than 10 cracks the IRI predicted using the RCI based on the Canadian Airport Model [Haas 87] was statistically not different from the observed IRI with the exception of the Paterson equation.

Table 6.4 ANOVA for IRI Prediction and Observed IRI Less Than 10 Cracks

Comparison1)	**Test**	$F_{CALCULATED}$	$F_{CRITICAL}$	**P-value**	**Degrees of Freedom**
Hein IRI AND Observed IRI	Between2) Observed & Predicted	2.6991	5.5915	0.1444	1
Paterson IRI AND Observed IRI	Between2) Observed & Predicted	9.3023	5.5915	0.0186	1
Al-Omari IRI AND Observed IRI	Between2) Observed & Predicted	1.1980	5.5915	.3099	1
$Hajek_{carusers}$ IRI AND Observed IRI	Between2) Observed & Predicted	2.2790	5.5915	0.1749	1
$Hajek_{truckusers}$ IRI AND Observed IRI	Between2) Observed & Predicted	1.0518	5.5915	0.3392	1

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

The average IRI predicted for the various equations was compared to a grouped observed cracking. The cracks were categorized as 0 –10 cracks, 10 – 20 cracks, 20 – 30 cracks, 30 – 40 cracks 40 – 50 cracks and 50 – 100 cracks. Based on Table 6.5, it indicates that the Hajek [Hajek 95] model for truck users shows the closest prediction to the observed.

6.6 SUMMARY OF FINDINGS

The findings indicate that the RCI model of equation 6.1 combined with the various IRI predictions are not a good means of estimating IRI based on the C-LTPP sections. This lack of consistency may be related to the very smooth observed C-LTPP test sections. In short, the IRI predictions using equations 6.2 through 6.6 appear to consistently predict higher values than

Table 6.5 Summary of IRI Comparison

Observed Cracking (Cracks/150m)		**Measured IRI[1]**	**IRI Equation**				
			Hein[2]	**Paterson[3]**	**Al-Omari[4]**	**Hajek Car[5]**	**Hajek Truck[6]**
0 - 10	Average	**1.36**	1.77	2.33	1.63	1.80	**1.62**
	Standard Deviation	0.29	0.53	0.74	0.51	0.66	0.54
10 - 20	Average	**1.53**	2.42	3.32	2.31	2.61	**2.27**
	Standard Deviation	0.36	0.03	0.05	0.03	0.03	0.03
20 - 30	Average	**1.29**	2.53	3.52	2.45	2.75	**2.39**
	Standard Deviation	0.13	0.08	0.14	0.10	0.10	0.08
30 - 40	Average	**1.34**	2.78	4.00	2.78	3.06	**2.64**
	Standard Deviation	0.31	0.06	0.12	0.09	0.08	0.06
40 - 50	Average	**1.35**	3.07	4.58	3.19	3.41	**2.93**
	Standard Deviation	0.23	0.07	0.16	0.11	0.09	0.07
50 - 100	Average	**1.30**	4.39	9.43	6.56	5.07	**4.27**
	Standard Deviation	0.33	0.61	3.15	2.19	0.76	0.62

Notes 1) Bolded Values are the average IRI and closest calculated IRI

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

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

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

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

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

observed. In addition, the predicted values are very similar. However, as the number of cracks increases, the predicted IRI values diverge. The Hein model [Hein 89] and Hajek models [Hajek 95] predict similar results while the Al-Omari [Al-Omari 94] and Paterson [Paterson 86] equations, result in much higher IRI predictions as the number of cracks increases, or an increased RCI based on the Canadian Airport Prediction [Haas 87].

Based on these findings, it would be recommended to use the Canadian Airport Model (equation 6.1) as an initial or starting point, with recognition that it is quite conservative. An update of this model and/or the development of a new model is highly desirable and should be a priority of future C-LTPP data analysis, particularly as the sections age and deteriorate. As well, there are undoubtedly factor interactions that should be captured in such an updated model (e.g., low temperature cracking frequency versus number of freeze-thaw cycles, or versus subgrade soil type, or versus amount of rainfall, etc.).

CHAPTER SEVEN MODULE FOUR : LIFE-CYCLE COSTING

The purpose of this chapter is to relate the pavement performance to life-cycle cost. Life-cycle costing procedures are reviewed and a detailed evaluation of current practices is presented. Pavement cost data from MTO is examined and the probability distributions associated with pavement thickness and costs associated with various pavement materials are summarized. A proposed framework to carry out a life-cycle cost analysis (LCCA) is also presented.

7.1 LIFE-CYCLE COST EVALUATION

Life-cycle cost analysis (LCCA) is a technique that uses economic principles to compare competing alternative investment strategies [FHWA 98]. It incorporates initial and discounted future costs over the life cycle of the alternative investments and attempts to identify the best value or the lowest cost over time. This task should be carried out during the initial design stages of a project and should only compare the differential cost among the alternatives.

There are various methods that can be used for LCCA. Some of the more common economic analysis strategies include Net Present Value (NPV), Equivalent Uniform Annual Costs (EUAC), Rate of Return (ROR), Benefit – Cost (B/C) ratios and Break-Even Analysis. According to the United States Federal Highway Administration (FHWA) [FHWA 98], the Net Present Value is the method of choice. The EUAC indicator is also recommended as long as the cost is derived from the NPV. The rate of return, benefit cost ratio and break even point are generally not recommended based on the difficulty of quantifying costs and benefits for use in these types of evaluations.

If the NPV method is selected, the designer will have a choice on whether to use constant dollars or nominal dollars. Constant dollars are also called real dollars as they reflect a "constant" purchasing power over time. For example, it is presumed that to purchase hot mix today at $20/tonne will remain at $20/tonne in the future. Alternatively, nominal dollars reflect fluctuations in purchasing power as a function of time. Thus, hot mix at $20/tonne today, may be

$22/tonne in the future. Regardless of the choice, the two types of dollars should not be mixed and should be consistent throughout an analysis.

The discount rate is one of the variables necessary to calculate NPV. It is used to reduce future expected expenditures to present day terms and is one of the most controversial variables in the NPV equation [Tighe 97]. The discount rate (true interest rate) should be selected based on inflation rate (annual compound rate of increase in the cost of pavement construction, etc.) and the interest rate associated with the agency borrowing money (market interest rate). The discount rate should also reflect historical trends over long periods of time. Historically these nominal discount rates (interest rate minus inflation rate), over an extended period of time has been three to four percent [Kerr 87]. In Canada, values of up to ten percent have been used but the range of four to eight percent is more common [TAC 97].

It is recommended that constant dollars and real discount rates should be used. This eliminates the need to estimate and include premium for both cost and discount rates. Real discount rates are recommended over nominal discount rates (inflation) as they reflect the true value of money over time with no inflation premiums and should be used in conjunction with non-inflated dollar cost estimates [FHWA 98].

The analysis period is the length of time selected for the life cycle cost and it should not extend beyond the period of reliable forecasts. In general 20 – 30 year analysis periods are selected for pavements [TAC 97]. The net present value (NPV) is calculated according to equation 7.1.

$$\text{NPV} = \Sigma \ \text{Costs} / (1 + i)^n \qquad (7.1)$$

where: Costs = Initial Construction, Maintenance and Rehabilitation Costs
i = Discount Rate
n = Year activity occurs

7.2 COST ESTIMATES

The quantification of costs can be determined based on the availability of previous construction and maintenance projects [Tighe 97]. The initial construction, major maintenance, rehabilitation, and salvage value are most frequently included in the life-cycle economic analysis. User delay costs are very important. However, in the majority of cases, it is not included in the analysis as public agencies do not consider them as direct costs. The initial construction cost includes the material costs associated with the pavement design. Only those costs that represent significant differences between the various alternatives should be included [Paterson 85]. Key considerations should only be given to those components of the design which are unique to that design alternative [ERES 98].

Maintenance costs can be categorized as routine maintenance and major maintenance. Routine maintenance is relatively inexpensive activities such as filling potholes and performing drainage improvements. These treatments have a service life of one to four years [Haas 97]. Major maintenance is more substantial and is usually associated with a structure or surface improvement such as patching or microsurfacing. These treatments have an expected service life of five to ten years [Haas 97]. It is recommended that only major maintenance be included in the LCCA as routine activities tend to be consistent across pavement design types.

Rehabilitation cost can be determined based on pavement performance prediction. The initial pavement design and the maintenance activities will have a large influence on what will be required in the future and when it will be required.

The salvage value should also be included as it indicates the remaining service life at the end of the analysis period. It involves a value of reusable materials at the end of the design period. The portion of the remaining life is often used to calculate the salvage values [Tighe 97].

The user delay cost is an important cost as it indicates the delays associated with the work zone activities and the implications of these activities. Based on the proposed alternatives, there may be considerable differences between traffic delays, accident costs, vehicle operating costs, discomfort costs and various others [ERES 98].

7.3 INFLUENCE OF LOW TEMPERATURE CRACKING ON LIFE-CYCLE COSTING

There are tremendous economic benefits to reducing the amount of cracking by either using a high performance asphalt [Deme 96] and/or timely maintenance to seal cracks [Haas 99a]. The Ontario Ministry of Transportation has carried out various studies [Chong 89, Joseph 90, Ponniah 95] which assess the influence of crack sealing on pavement performance. Their observations show if cracks are left untreated, the following occurrs:

1. Severe erosion at the bottom of the asphalt layers occurs due to the pumping action of water caused by traffic loads.
2. Upheaval of the pavement surface in the winter due to frost action on saturated base and subgrade layer, followed by depressions at the crack in the summer.
3. Accelerated loss in riding quality, with associated reduction of pavement life.
4. Frequent necessity of cutting a 1 m width out of the pavement and carrying out repairs, prior to rehabilitation, due to multiple cracking, depression, etc. At the original crack location.

The Ontario field studies involved 37 test sites. Crack mapping was carried out each year and performance curves which use Pavement Condition Index (PCI) were constructed for both treated and untreated (control) portions of each test site. Figure 7.1 is a typical section. It shows an estimated increase in pavement life of 2 years for the treated section. The range of increase life was up to 5 years.

A comprehensive life-cycle cost analysis of the crack sealing was also carried out [Ponniah 95]. In an example of a two-lane 21 km road in Southwestern Ontario, with an initial Average Annual Daily Traffic (AADT) of 750, annual growth rate of 4% and 18% trucks, and a design 115mm asphalt concrete on a 350mm of granular layers, the following two alternative strategies over a 30 year life-cycle period were considered:

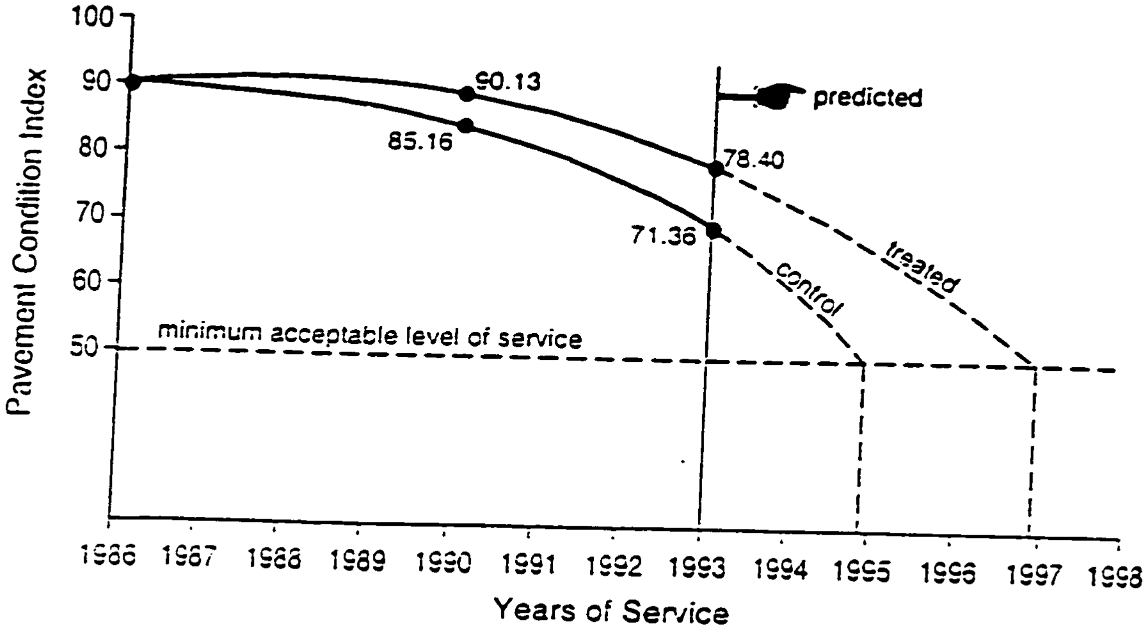


Figure 7.1 Example of Extended Pavement Life Due to Crack Treatment (rout and seal) [Ponniah 95]

1. 50 mm overlay at years 11 and 21, with no crack sealing.
2. Crack rout and sealing at years 4 and 8, 50mm overlay at year 13, crack routing and sealing at years 17 and 21, 50mm overlay at year 25 and crack routing and sealing at year 29.

Using a discount rate of 5%, the total present worth costs of each alternative was $24,000 per lane km for alternative one and $17,000 per lane km for alternative two. In essence, the increased service life due to the rout and seal was more cost effective than not routing and sealing.

Overall, these Ontario studies [Chong 89, Joseph 90, Ponniah 95] indicate that by reducing the amount of cracking, the service life of a pavement can be extended. In general, this will provide significant cost savings [Haas 99a]. In short, rout and seal is a major component to a pavement management system and is an important consideration in the life-cycle cost of a pavement.

7.4 MODULE FOUR METHODOLOGY

The intent of this module is to examine how life-cycle costing was currently carried out in Canada and to provide recommendations on how it should be carried out. Despite the fact that LCCA is recognized as an important tool in properly managing pavements, it is not routinely carried out [ERES 98].

In Canada, the Ministry of Transportation of Ontario and the Ministry of Transport Quebec are the only two provincial agencies that seem to have a formal LCCA procedure. The other provinces do not generally use it as a tool for design as they tend to use standard (deterministically based) designs.

Two analyses were performed in this module. The first focussed on determining the variation associated with thickness and the second involved pavement cost variation and how to use this variation in an analysis.

7.5 THICKNESS VARIATION

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

The C-LTPP test sections are used in this analysis. The design overlay thickness for each test section was categorized as thin, medium and thick. The sections were categorized based on the average thickness according to the asphalt cores taken after construction. The thin overlay thicknesses were those which fell between 10 mm and 38 mm, medium overlay thicknesses ranged from 25 mm to 80 mm and the thick overlay thicknesses were those with more than 80 mm thickness. Table D.1 summarizes the core thicknesses provided for the C-LTPP test sections. Table 7.1 summarizes the mean and standard deviation for the three pavement overlay thicknesses. Although most of the averages appear to be close to the designs, the standard deviation associated with the designs is very high. Figure 7.2 shows the best fit distributions for the three pavement thicknesses. The lognormal distribution is found to have the best χ^2 and be the closest distribution. The chi-square (χ^2) is a goodness of fit test for the distribution models, which is used to indicate the relative degree of validity of the different distributions [Ang 75]. The lower the χ^2, the better the fit.

For thin overlay pavement thicknesses, the best fit distributions are not commonly used (i.e. extreme value, weibull, triangular, beta). However, the lognormal is the most common distribution out of the five. Based on the χ^2 analysis, the lowest χ^2 are shown with the thick overlay thicknesses. The lognormal best fit would be the most reasonable choice as it is more common and appears to be the most reasonable choice as it is more common and seems to be appropriate. It should be noted that although the thick pavement overlay is showing 100mm, it is reasonable to presume that this was placed as two medium lifts (i.e. two at 50mm). However, this breakdown was not available. For the thin overlay thicknesses, the magnitude of the average value and standard deviation appear to be reasonable. The best fit distributions are not commonly used distributions with the exception of the lognormal. For the purpose of the analysis, the lognormal has been selected and depicted on Figure 7.2.

For medium overlay thickness, which is the most common thickness in Ontario and in fact throughout Canada, the lognormal distribution is also selected as the best fit. The higher χ^2 for this analysis can be explained by the split between the typcial lift thickness of 40 and/or 50mm.

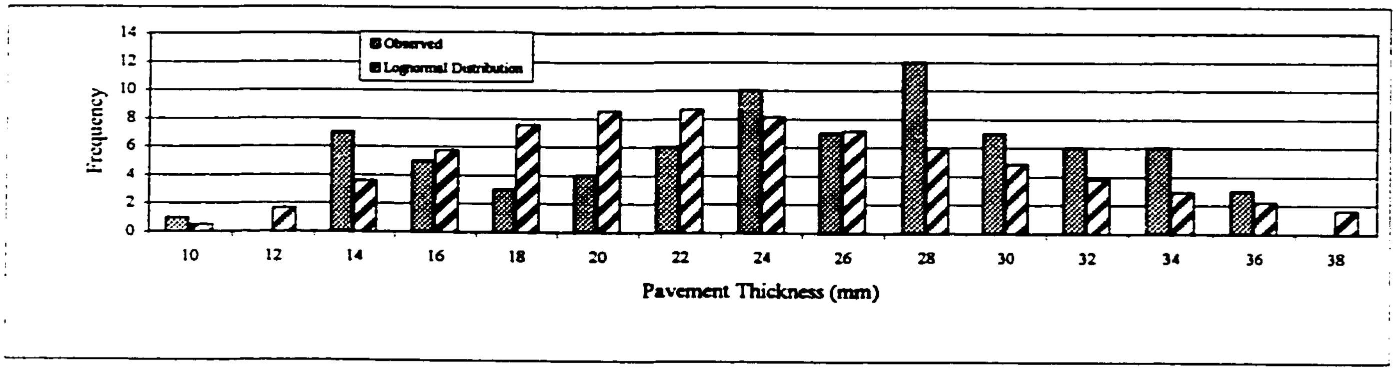
Table 7.1 Best Fit Distributions For Thickness

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

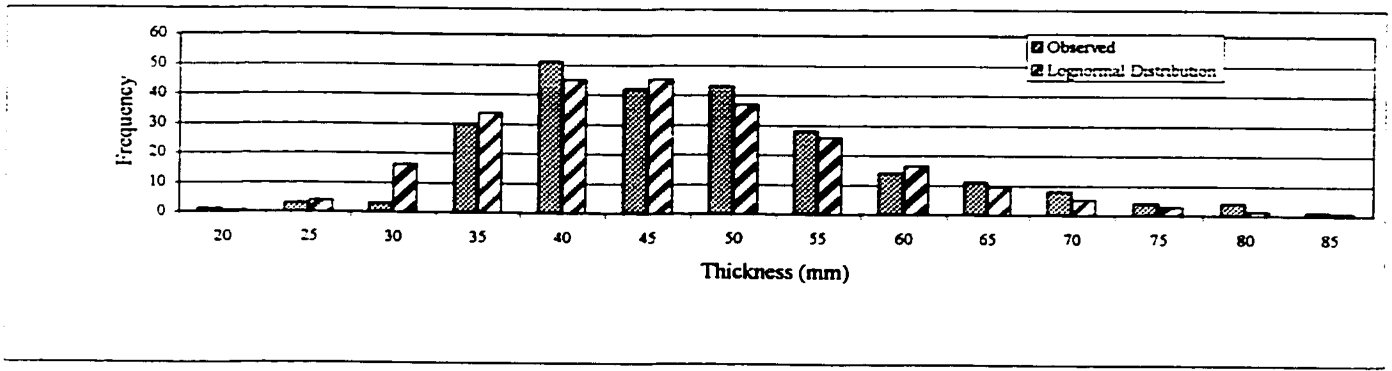
The lognormal distribution was selected as an overall guide for overlay thickness contrary to the standard belief which assumes it is best described by a normal distribution. The lognormal distribution is represented by equation 7.2 [Ang 75]. In addition, the use of a lognormal distribution would appear to be most appropriate as the values of the random variable with the lognormal variates are always positive.

$$F(x) = 1 / ((2\pi)^{1/2} \xi X) \exp[-1/2 ((\ln X - \lambda) / \xi)^2] \qquad (7.2)$$

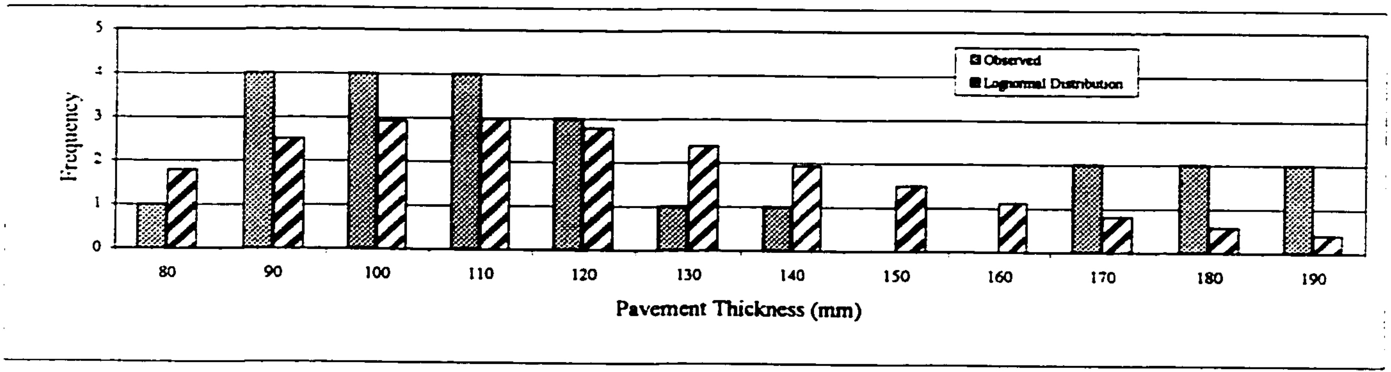
Where: ξ = $(\mathrm{Var}(\ln X))^{.5}$
λ = $E(\ln X)$
X = random variable



Thickness Distribution for Thin Pavement Overlay Thickness



Thickness Distribution for Medium Pavement Overlay Thickness



Thickness Distribution for Thick Pavement Overlay Thickness

Figure 7.2 Overlay Thickness Distribution Curves Based on C-LTPP Core Thicknesses

7.6 COST VARIATION

The MTO Project Value System (PVS) provided the unit item prices. The PVS calculates the item cost by averaging the three lowest contractor tender bids for the particular material item and contract. All districts in the province were included in the analysis. The prices were the most current available and covered the period from 1993 to 1997. Only those materials pertinent to the pavement structure were included in the assessment. Tables D.2 through D.11 summarize the unit costs provided by PVS for the following items: Granular A, Granular B, HL1 (where HL refers to Hot Laid Asphalt and the number, 1 refers to the mix design), HL3, HL4, HL8, Heavy Duty Binder Course (HDBC), Dense Friction Course (DFC), and Asphalt Removal Partial Depth, Cold –In-Place Recycled Mix, and Rout and Seal Cracks.

Table 7.2 summarizes the best fit distributions when all the values for all quantity contracts were included. The best fit distributions, based on the χ^2 vary between Gamma, Extreme Value, Pareto, Lognormal and Weibull. Although gamma and lognormal are more common, the others tend not to be used all that often. The lack of fit (high χ^2) and the types of distributions can be explained by the influence of economies of scale on pricing. Because all of the prices are included and they are not separated by quantity, there is a large amount of variation. For example, as the quantity of an item increases, the prices tend to decrease and stabilize. Consequently, the distributions presented in the quantity breakdown analysis are more common (i.e., approach lognormal or normal). Additionally, the χ^2 decreases, indicating a better fit, and the standard deviation decreases.

Based on this influence of economies of scale, the analysis was redone with a breakdown in terms of quantity. The quantity ranges are based on typical capital construction contract quantities. Table 7.3 summarizes the best fit distributions for the various materials and the associated parameters. Table D.12 in Appendix D shows the top five best fit distributions according to the values. The values are much lower when the costs are categorized by quantity. The predominant best fit distribution is the lognormal distributions. In two cases, the normal

Table 7.2 **Summary of Best Fit Distributions Using All Values**

Material	**Distribution**	**χ Squared**
Granular A	Gamma	33.49
	Extreme Value	49.40
	Lognormal	65.00
	Logistic	138.96
	Weibull	105.36
Granular B	Gamma	17.72
	Weibull	19.95
	Lognormal	22.74
	Extreme Value	23.44
	Beta	32.79
HL1	Extreme Value	8.83
	Pareto	13.75
	Weibull	13.75
	Gamma	14.33
	Lognormal	17.52
HL3	Gamma	28.75
	Pareto	37.55
	Extreme Value	41.18
	Lognormal	49.46
	Weibull	48.94
HL4	Gamma	36.40
	Pareto	48.61
	Weibull	66.70
	Extreme Value	83.55
	Lognormal	103.96
HL8	Extreme Value	9.42
	Gamma	16.65
	Weibull	23.89
	Logistic	29.32
	Lognormal	33.84
HDBC	Extreme Value	19.01
	Lognormal	26.33
	Logistic	28.40
	Gamma	29.57
	Beta	39.55
DFC	Extreme Value	17.89
	Gamma	20.42
	Logistic	27.61
	Beta	27.61
	Lognormal	32.89
REMOVAL OF ASPHALT PAVEMENT PARTIAL DEPTH	Weibull	43.33
	Gamma	49.87
	Lognormal	52.11
	Pareto	80.67
	Exponential	91.51

Table 7.3 Summary of Material Best Fit Distributions

Material	Quantity (tonnes)	Distribution	χ Squared	Parameter	
				One	Two
Granular A	100 - 1000	Lognormal	2.56	25.33	10.85
	1000 - 10000	Lognormal	9.36	14.38	3.46
	10000 - 100000	Lognormal	11.08	10.78	2.39
	100000+	Lognormal	4.3	9.21	1.83
Granular B	1000 - 10000	Normal	12.95	8.98	2.49
	10000 - 100000	Lognormal	7.14	7.16	1.86
	100000+	Lognormal	6.8	5.57	1.17
HL1	100 - 1000	Lognormal	1.111	86.91	25.79
	1000 - 10000	Lognormal	2.13	58.93	9.86
	10000+	Lognormal	0.18	46.48	4.25
HL3	100 - 1000	Lognormal	15.54	70.12	30.05
	1000+	Lognormal	4.95	46.81	8.3
HL4	100 - 1000	Lognormal	7.6	109.42	54.56
	1000 - 10000	Lognormal	9.83	56.27	15.17
	10000+	Lognormal	14.58	43.6	6.95
HL8	100 - 1000	Exponential	15.6	0.01	
	1000 - 10000	Lognormal	2.43	42.32	6.28
	10000+	Lognormal	5.55	38.52	4.25
HDBC	100 - 1000	Exponential	8.9	0.01	
	1000 - 10000	Lognormal	3.36	49.44	9.14
	10000+	Lognormal	17.38	42.33	3.29
DFC	1000 - 10000	Normal	10.88	64.97	9.18
	10000+	Lognormal	13	58.73	8.83
Removal of Asphalt Pavement Partial Depth	100 - 1000	Lognormal	5.81	10.86	5.38
	1000 - 10000	Lognormal	2.69	4.11	2.36
	10000 - 100000	Lognormal	7.15	1.98	0.98
	100000+	Lognormal	7.56	1.18	0.38

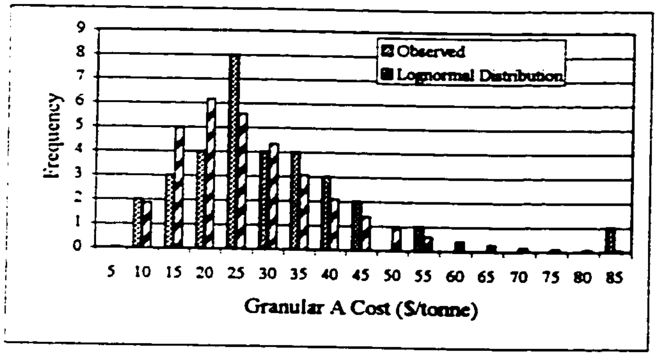
distribution is most suitable while in two other cases, the exponential is most suitable. In these four cases, the "non lognormal" distributions occur when there are smaller quantities and a smaller sample size.

As the material quantity increases (i.e. as the size of the construction or maintenance job increases), the best fit distribution tends to follow the lognormal distribution. Figures 7.3, 7.4 and 7.5 show the observed cost data and the best fit distributions. Some of the maintenance items are not presented based on limited data for the distribution analysis. The histograms show the material price and the frequency of the observed price. The lognormal distribution is the best fit distribution in the majority of cases.

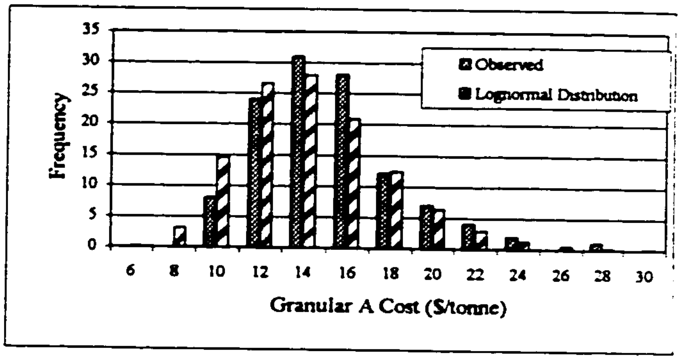
Table 7.4 summarizes the average cost values and standard deviations for the various materials for the respective quantities. As the quantity increases, the unit cost decreases and generally the standard deviation decreases. This can be explained by the ability to take advantage of the economies of scale.

7.7 LIFE-CYCLE COST AND MODIFIED ASPHALTS

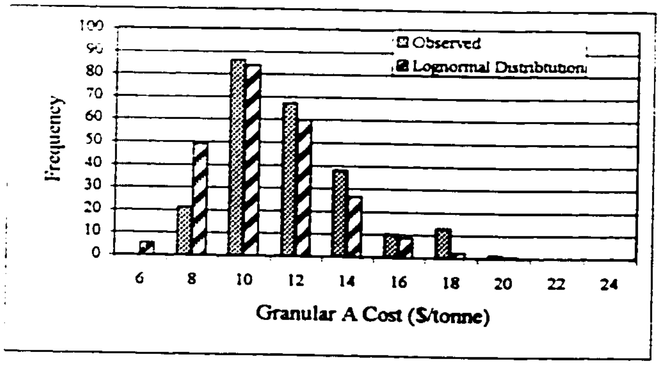
The LCCA for pavements constructed with modified asphalt would be carried out in a similar manner. It is apparent from previous work [Tighe 97], that the initial cost for modified asphalt is higher as compared to conventional asphalt. The increased initial cost, depending on the design varies between ten to twenty percent higher with the modified asphalt. However, this higher initial cost is offset by reduced maintenance and longer pavement service lives in certain circumstances. This further emphasizes the need for LCCA especially when these products are proposed. Despite the higher initial cost, over the long term, the modified asphalts are shown in some cases to have lower net present value when examined over a 30 year period.



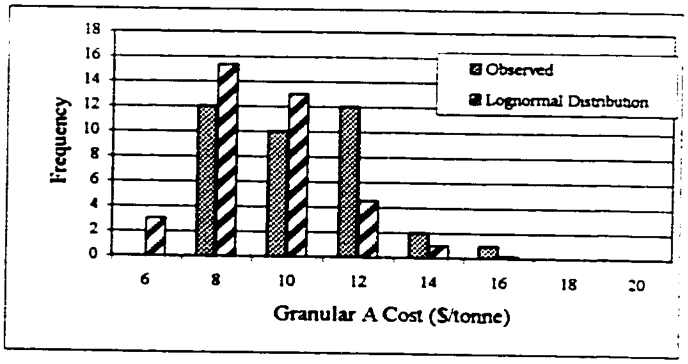
Cost Distribution of Granular A for 100 - 1000 tonnes



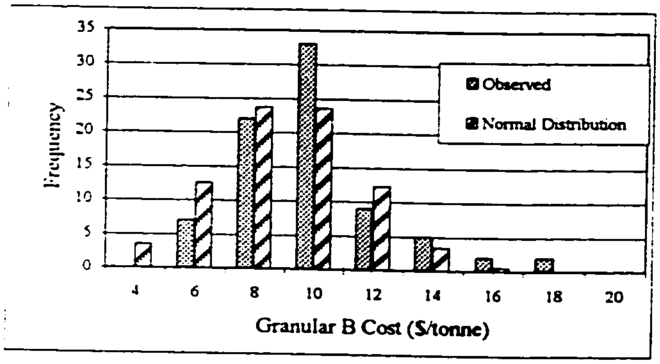
Cost Distribution of Granular A for 1000 - 10000 tonnes



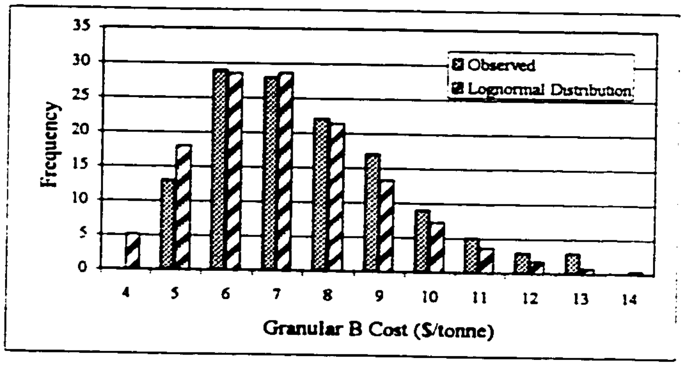
Cost Distribution of Granular A for 10000 - 100000 tonnes



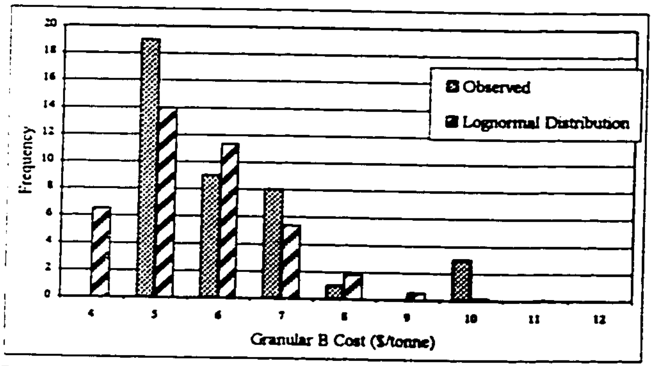
Cost Distribution of Granular A for 100000+ tonnes



Cost Distribution of Granular B for 1000 - 10000 tonnes

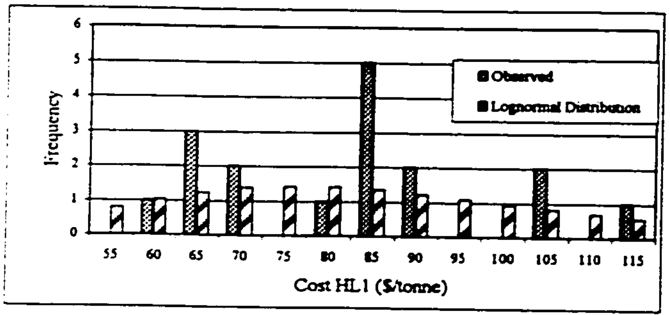


Cost Distribution of Granular B for 10000 - 100000 tonnes

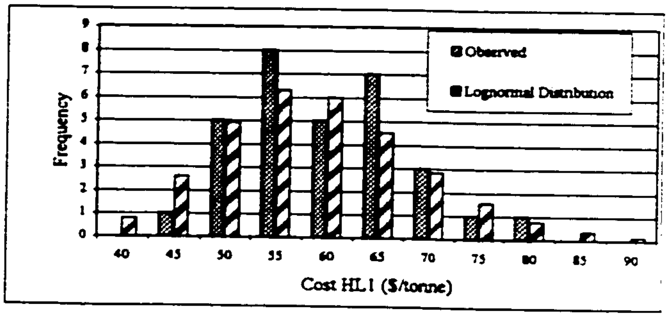


Cost Distribution of Granular B for 100000+ tonnes

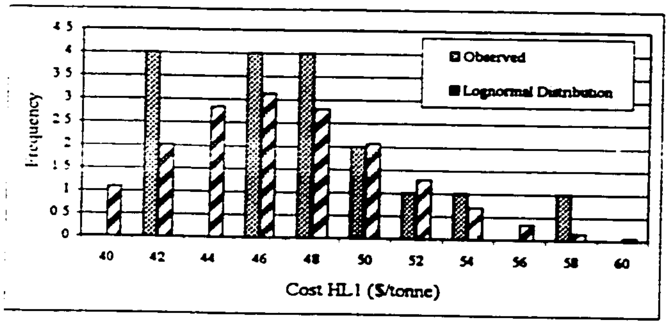
Figure 7.3 Cost Distribution Curves for Granular



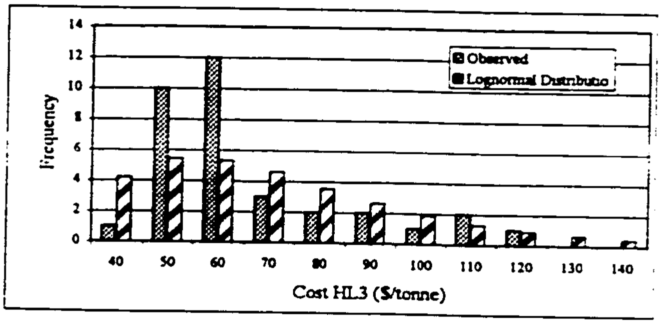
Cost Distribution of HL1 for 100 - 1000 tonnes



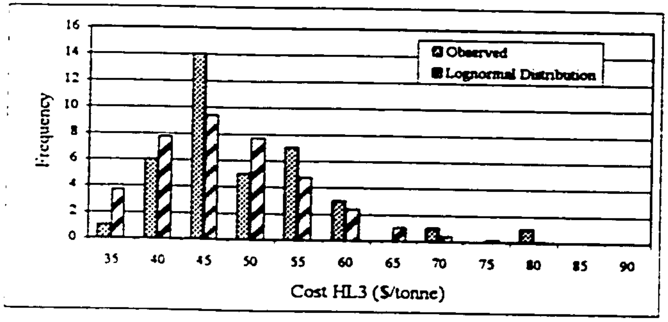
Cost Distribution of HL1 for 1000 - 10000 tonnes



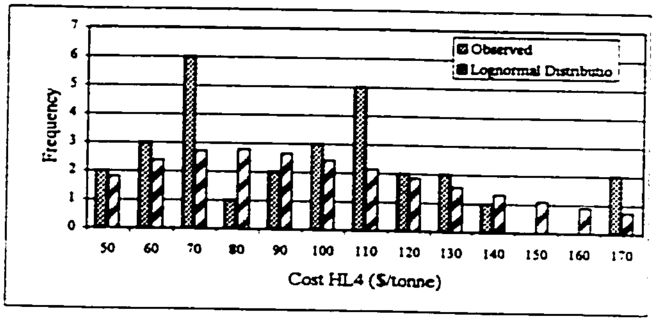
Cost Distribution of HL1 for 10000+ tonnes



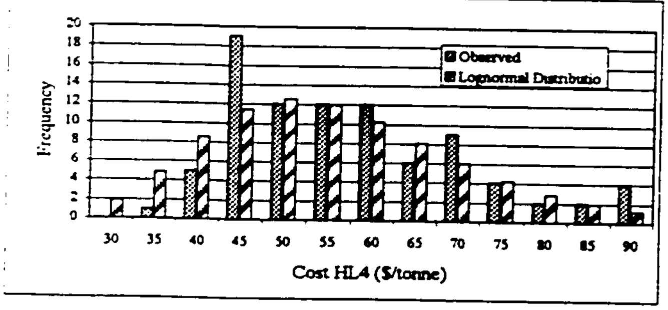
Cost Distribution of HL3 for 100 - 1000 tonnes



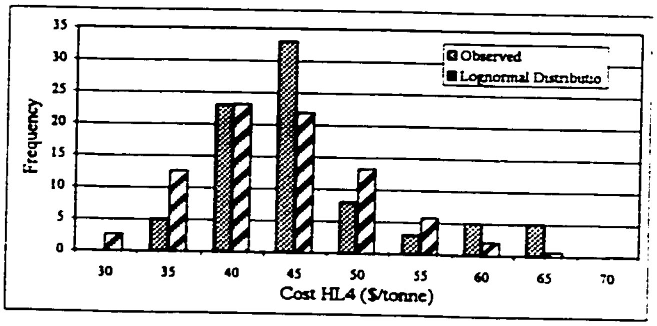
Cost Distribution of HL3 for 1000+ tonnes



Cost Distribution of HL4 for 100 - 1000 tonnes

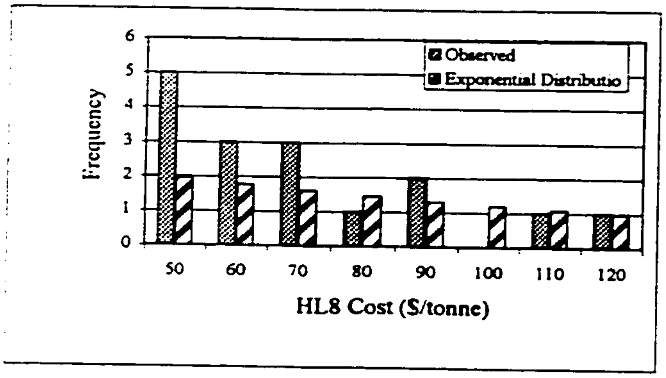


Cost Distribution of HL4 for 1000 - 10000 tonnes

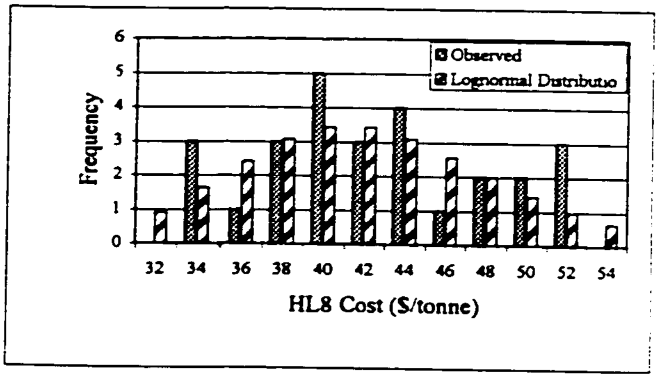


Cost Distribution of HL4 for 10000+ tonnes

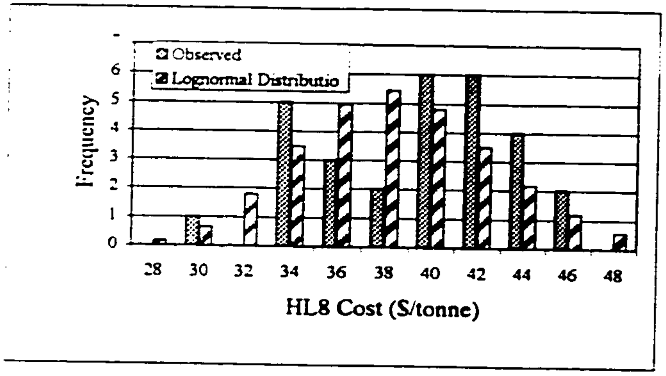
Figure 7.4 Cost Distribution Curves for HL1, HL3 and HL4



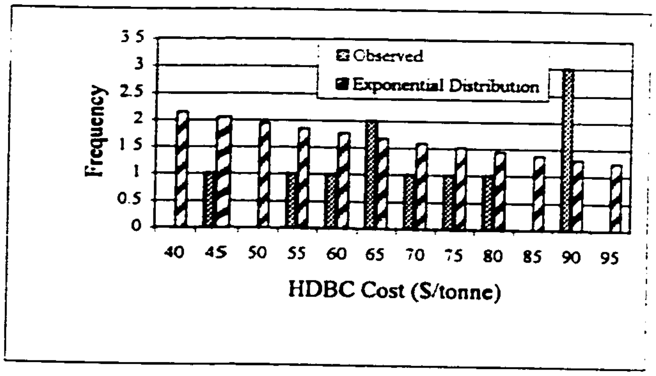
Cost Distribution of HL8 for 100 - 1000 tonnes



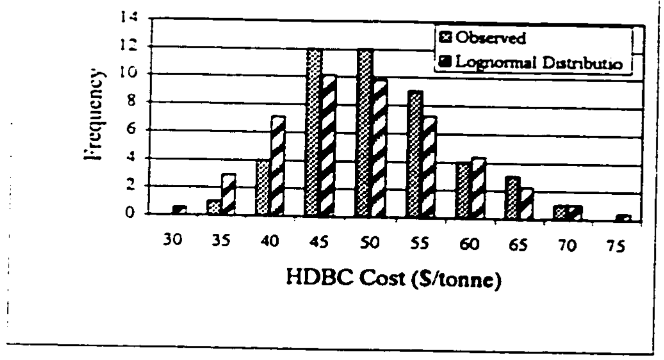
Cost Distribution of HL8 for 1000 - 10000 tonnes



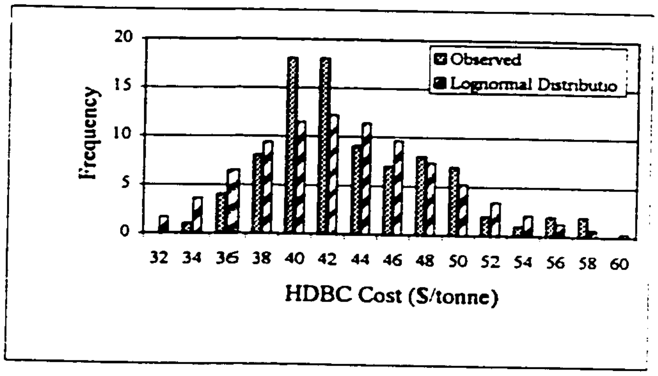
Cost Distribution of HL8 for 10000+ tonnes



Cost Distribution of HDBC for 100 - 1000 tonnes



Cost Distribution of HDBC for 1000 - 10000 tonnes



Cost Distribution of HDBC for 10000+ tonnes

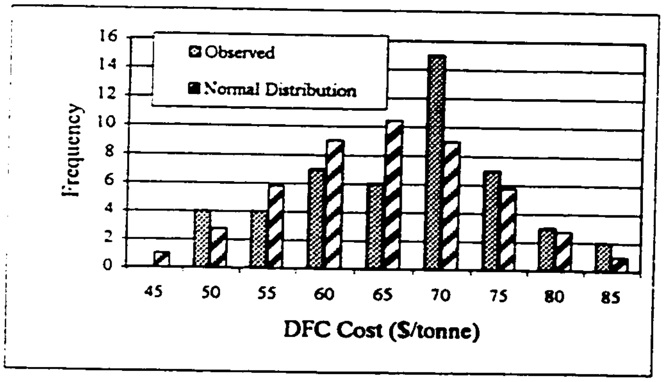


Figure 7.5 Cost Distribution Curves for HL8, HDBC and DFC

Table 7.4 Summary of Prices Based On Quantity Breakdowns

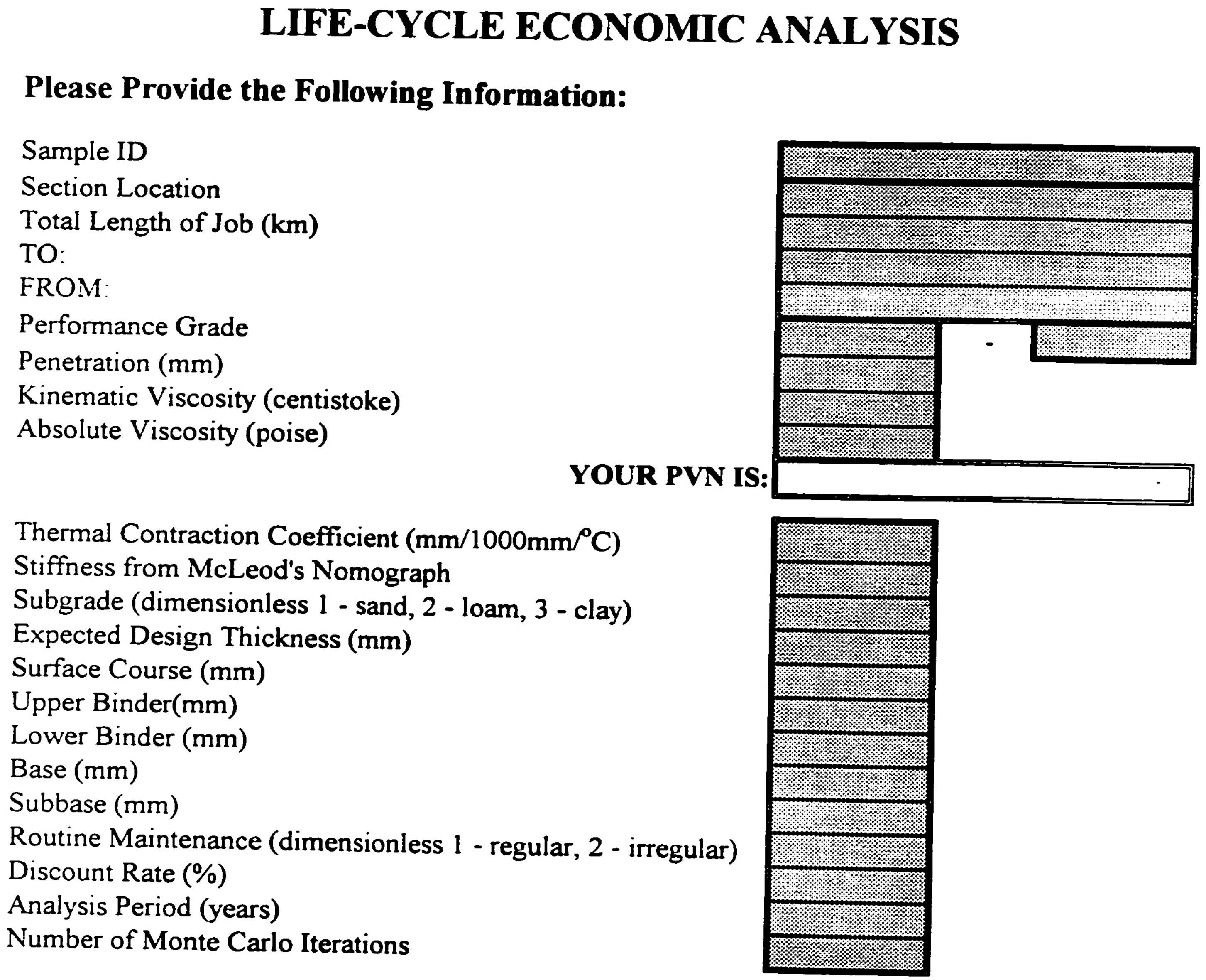
Material	**Quantity**	**Average**	**Standard Deviation**
Granular A	0 - 1000	$27.89	14.23
	1000 - 10000	$14.40	3.77
	10000 - 100000	$10.79	2.50
	100000+	$9.22	1.90
Granular B	0 - 1000	$10.29	2.85
	1000 - 10000	$8.98	2.49
	10000 - 100000	$7.16	1.92
	100000+	$5.58	1.33
HL-1	0 - 1000	$97.12	45.16
	1000 - 10000	$58.96	10.64
	10000+	$46.49	4.49
HL-3	0 - 100	$140.15	120.31
	100 - 500	$74.89	48.04
	0 - 1000	$87.70	73.07
	1000+	$46.83	8.85
HL4	100 - 500	$118.31	56.27
	100 - 1000	$109.35	53.02
	1000 - 10000	$56.50	18.75
	10000+	$43.62	7.47
HL8	100 - 1000	$62.90	17.97
	1000 - 5000	$45.17	6.01
	5000+	$38.18	4.03
HDBC	0 - 1000	$69.08	15.42
	1000 - 5000	$51.14	11.74
	5000+	$43.73	6.61
DFC	0 - 1000	$85.94	27.25
	1000 - 5000	$65.57	9.26
	5000+	$59.32	9.35
COLD IN-PLACE RECYCLED MIX	10000+	$4.00	1.07
ROUT AND SEAL	100 - 1000	$7.24	2.82
	1000 - 15000	$4.27	4.55
	100000+	$1.10	0.18
REMOVAL ASPHALT PARTIAL DEPTH	100 - 1000	$10.82	5.09
	1000 - 10000	$4.10	2.25
	10000 - 100000	$2.00	1.16
	100000+	$1.19	0.43

7.8 TOWARD A LIFE-CYCLE COST ANALYSIS PACKAGE

The best fit distributions for thickness and material costs have been established. In addition, the standard deviation of the variables outlined in the model (i.e. PVN, thermal contraction coefficient, material costs, etc.) have been determined in this research.

It is recommended that a comprehensive probabilistic life-cycle cost package be the next step in this model development. This would be a major task it itself and beyond the scope of this research. However, the following discussion is a proposed format for the comprehensive life-cycle package.

It is suggested that an input sheet, three calculation sheets and a final output be developed which in essence, would be a computerized version of the integrated model. Figure 7.6 would be a suggested input sheet for a Monte Carlo simulation where all the required variables for prediction would be entered in the shaded boxes. Once the variables were entered the program could be run. A series of calculation sheets would be used to predict cracking and performance. The timing of maintenance and rehabilitation is then related to the performance of the pavement as a function of time. If a treatment is required, the associated cost is then automatically calculated. This procedure is carried out in an iterative manner to establish the probabilistic cost. Once the number of iterations is complete, the values obtained using a Monte Carlo simulation would be organized and displayed as shown in Figure 7.7, whereby both the observed and best fit distributions are displayed.



LIFE-CYCLE ECONOMIC ANALYSIS

Please Provide the Following Information:

Sample ID
Section Location
Total Length of Job (km)
TO:
FROM:
Performance Grade -
Penetration (mm)
Kinematic Viscosity (centistoke)
Absolute Viscosity (poise)

YOUR PVN IS:

Thermal Contraction Coefficient (mm/1000mm/°C)
Stiffness from McLeod's Nomograph
Subgrade (dimensionless 1 - sand, 2 - loam, 3 - clay)
Expected Design Thickness (mm)
Surface Course (mm)
Upper Binder(mm)
Lower Binder (mm)
Base (mm)
Subbase (mm)
Routine Maintenance (dimensionless 1 - regular, 2 - irregular)
Discount Rate (%)
Analysis Period (years)
Number of Monte Carlo Iterations

Figure 7.6 Sample Input Sheet To Probabilistic Life-Cycle Cost Analysis

LIFE-CYCLE ECONOMIC ANALYSIS
SUMMARY SHEET

INTERVAL $1,000		FREQUENCY 1000
0	25	1205
25	50	3250
50	75	3247
75	100	30000
100	125	50369
125	150	42143
150	175	43791
175	200	27945
200	225	13884
225	250	10467
250	275	74561
275	300	3999
300	325	3894
325	350	991
350	375	0
375	400	0
400	425	0
425	450	0
450	475	0
475	500	0
500	525	-
525	550	-
550	575	-
575	600	-
600	625	-
625	650	-
650	675	-
675	700	-
700	725	-
725	750	-
750	775	-
775	800	-
800	825	-
825	850	-
850	875	-
875	900	-
900	925	-
925	950	-
950	975	-
975	1000	-

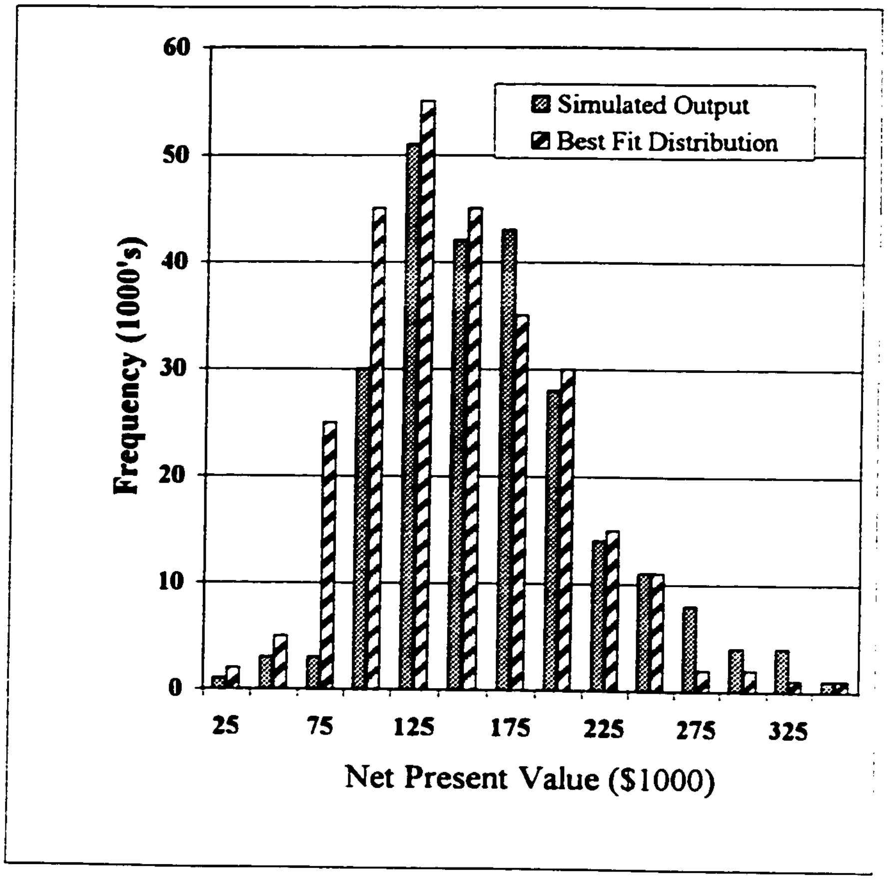


Figure 7.7 Sample Output Sheet To Probabilistic Life-Cycle Cost Analysis

CHAPTER EIGHT CONCLUSIONS AND RECOMMENDATIONS

The purpose of this chapter is to summarize the major finding of the research and provide recommendations on future direction. The thesis establishes a framework for assessing low temperature cracking based on material properties.

The results presented provide a methodology for predicting performance. Pavement designers can use PVN as a tool for predicting field performance. The models proposed predict cracking which is similar to that observed in the field. However an update is recommended to reflect current as-built and in-service conditions. Roughness trends are predicted and compared to those observed. Recommendations are proposed to updating roughness prediction based on in-service conditions. The probability distributions associated with pavement thickness and life-cycle cost are presented. A life-cycle cost analysis framework is established which proposes the use of the lognormal distribution for both lift thickness and material cost. Modified asphalts are examined and PVN appears to have some relevance to Superpave PG methodology.

8.1 OBJECTIVES

The objectives of research which are the basis of the integrated model were as follows:

- Determine if PVN was a fingerprint over time for both conventional and modified asphalts.

- Determine if PVN was repeatable for calculations using absolute viscosity (@ 60°C) and kinematic viscosity (@ 135°C).

- Relate PVN to Superpave methodology.

- Determine if PVN was an indicator of low temperature susceptibility.
- Predict thermal cracking using the Hajek and Canadian Airport Model.
- Compare the predictions to the observed cracking on the C-LTPP test sections.
- Predict roughness based on observed thermal cracking on the C-LTPP test sections.
- Compare the predicted roughness with the observed roughness.
- Determine the probability distributions associated with lift thickness.
- Determine probability distributions associated with material costs.
- Establish a framework for predicting life-cycle costing and pavement performance for conventional and modified asphalts.

8.2 MODULE ONE: MATERIAL CHARACTERIZATION

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

8.2.1 Conventional Asphalt Analysis

- PVN is a "fingerprint" as it remains constant over time.
- PVN is an indicator of low temperature susceptibility.
- PVN is related to the minimum temperature specified in the Superpave design methodology.

- PVN within a crude source varies.

- PVN can be calculated with either absolute viscosity (@ 60°C) or kinematic viscosity (@ 135°C) and they are equal.

8.2.2 Modified Asphalt Analysis

- PVN did not remain constant with time, although the sample size was limited.

- PVN did relate to the Superpave minimum temperature.

8.3 MODULE TWO: LOW TEMPERATURE CRACKING PREDICTION

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

- The Hajek model is good for predicting low temperature cracking based on the C-LTPP and C-SHRP test sections. It is also easy for designers to determine the variables necessary for pavement life-cycle performance prediction.

- The Canadian Airport model is a good model for predicting thermal cracking. However, it does not perform well in cases where a large amount of thermal cracking (more than 100 cracks per 150m) was observed.

- In cases where the thermal coefficient was measured and used to predict observed cracking, the Canadian Airport Model provided a good prediction.

- The thermal contraction coefficient and the minimum observed temperature is an important variable for predicting thermal cracking.

8.4 MODULE THREE: PAVEMENT PERFORMANCE

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

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

- The observed IRI on the C-LTPP sites was in a very smooth range. This could be related to new and improved construction methods which improve the initial roughness and ultimately increase the service life of the pavement.

- When the predicted IRI values based on the equations were compared to the observed roughness on the C-LTPP sites, the values were not very close. This in part could be due to the fact the IRI values on the C-LTPP sites are based on "Dipstick" measurements and the other relationships were based on different measurements.

- When there were less than 10 thermal cracks per 150m, the IRI predictions were close to observed.

8.5 LIFE-CYCLE COST ANALYSIS

Life-cycle cost procedures and probability distributions were examined. The following conclusions can be drawn from this work.

- Thickness of pavement layers is best described by a lognormal distribution based on the C-LTPP test sections for thin, medium and thick lift thickness.

- Pavement life-cycle costing which uses material costs should break costs down according to quantity to account for economies of scale. A lognormal distribution should be selected for the probabilistic analysis.

- The probabilistic analysis should include thickness and cost distributions to simulate in-service pavement variation.

- A probabilistic analysis provides better results compared to the current life-cycle costing analysis procedures used in Canada.

8.6 RECOMMENDATIONS

The following recommendations are presented based on the research carried out in this thesis.

- As pavement technology moves into the next century, there is a concerted movement away from conventional testing methods and movement towards the SHRP Superpave methodology. The results presented here indicate that PVN is a good indicator of low temperature susceptibility. PVN also relates to the Superpave PG minimum service temperature and can be easily calculated in a timely manner (i.e. the penetration test and viscosity tests are relatively simple with good repeatability and require a finite sample size). PVN can be used by the designer to predict low temperature cracking, pavement performance and life-cycle cost.

- Modified asphalt PVN characterization should be examined more thoroughly. A larger sample size would be extremely useful. The Superpave interval (the maximum PG temperature plus the minimum PG temperature) and how it relates to modified asphalts should be evaluated as it may provide more insight into how the modifications influence performance.
- Use the Hajek model for predicting low temperature cracking as a design tool.
- Carry out a Bayesian update on the Canadian Airport Model on both the cracking prediction and RCI model to reflect current values. It appears that changes to construction practices have changed the RCI range and how it relates to low temperature cracking.
- Examine thermal contraction coefficients and how they relate to PVN. Carry out a sensitivity analysis to examine how variability influences low temperature cracking.
- Examine as-built roughness and examine the possibility of updating the values provided by Sayers [Sayers 86]. It appears the Sayers values are not reflective of the IRI values being measured on the C-LTPP test sections.
- Examine how as-built roughness influences crack progression and long term performance.
- Life-cycle costing is paramount to determine the most appropriate design. The Net Present Value should be employed in a probabilistic manner which incorporates lift thickness and cost distributions. The lognormal should be used as the best fit distribution for the probabilistic analysis.

- Develop a comprehensive life-cycle cost package which incorporates PVN, thermal cracking prediction and life-cycle cost. This package could be established such that the user has a single input page. Various calculation spreadsheets would be used to determine a probability distribution associated with the cost of a given pavement design.

REFERENCES

[Al-Omari 94] Al-Omari, Basher and Michael Darter, "Relationships Between IRI and PSR", Transportation Research Board 73rd Annual Meeting, Washington, January 1994

[Altgen 75] Altgen, K., and O. Harle, "The Effect of Asphaltenes on Asphalt Viscosity", Industrial and Engineering Chemistry Product Research and Development, 1975

[Anderson 99] Anderson, Kenneth O., J.T. Christison, and Colin D. Johnston, "Low Temperature Performance Based on C-SHRP Test Roads", Prepared for Transportation Association of Canada Research and Development Council, Ottawa, 1999

[Ang 75] Ang, A., and W. Tang, Probability Concepts in Engineering Planning and Design Volume I – Basic Principles, John Wiley and Sons, Toronto, 1975

[AI 98] Asphalt Institute, "A Tribute to the Canadian Asphalt Industry", The Magazine of the Asphalt Institute Fall 1998, Volume 12, Number 2

[Bahia 96] Bahia, H., "Critical Evaluation of Asphalt Modification Using Strategic Highway Research Program Concepts, Proceedings Transportation Research Board, Washington D.C., 1996

[Bahia 92] Bahia, H., D. Anderson and D. Christensen, "The Bending Beam Rehometer; a Simple Device for Measuring Low-Temperature Rheology of Asphalt Binders", Proceeding of Association of Aspahlt Paving Technologies, Volume 61, pp. 117 - 153, 1992

[Bhurke 97] Bhurke, A., E. Shin and L. Drzal, "Fracture Morphology and Fracture Toughness Measurement of Polymer Modified Asphalt Concrete", Proceedings Transportation Research Board, Washington D.C., 1997

[Brodnyan 60] Brodnyan J., et al, "The Rheology of Asphalts III Dynamic and Mechanical Properties of Asphalts", Transportation Society, Rheology 4, 1960

[Brule 96] Brule, B., "Polymer Modified Asphalt Cements Used in the Road Construction Industry:Basic Principles", Proceedings Transportation Research Board, Washington D.C., 1996

[Brule 95] Brule, B., and M. Maze, "Application of SHRP Binder Tests to the Characterization of Polymer Modified Bitumens", Proceedings Association of Asphalt Pavement Technologists, Volume 64, 1995

[Brule 88] Brule, B., Y. Brian, and A. Tanguay, "Paving Asphalt Polymer Blends; Relationship Between Comparison, Structure and Properties", Proceedings Association of Asphalt Pavement Technologists, Volume 57, 1988

[Button 99] Button, N.P., "Release and Fire Incident Rates For Trucks Carrying Dangerous Goods", Doctoral Thesis, University of Waterloo, Waterloo, 1999

[Christensen 96] Christensen, R. Analysis of Variance: A Handbook of Techniques, Spring-Verlog, New York, Chapter 5: pg. 108 – 146, Chapter 6 pg. 146 – 165, 1982

[Chong 89] Chong, G.J., "Rout and Seal Cracks in Flexible Pavement: A Cost-Effective Maintenance Procedure", Report PAV-89-04, Ontario Ministry of Transportation, August 1989

[C-SHRP 95] Canadian Strategic Highway Research Program, "Superpave Binder Specifications and Test Methods", Technical Briefing #9, Ottawa, 1995

[C-SHRP 95a] Canadian Strategic Highway Research Program, "CSHRP Test Roads:Early Performance Results", Technical Briefing #10, Ottawa, 1995

[C-SHRP 95b] Canadian Strategic Highway Research Program, "Canadian Asphalt Binders and Superpave Performance Grades", Technical Briefing #11, Ottawa, 1995

[C-SHRP 96] Canadian Strategic Highway Research Program, Canadian Long Term Pavement Performance C-LTPP Database Users Guide, Transportation Association of Canada, 1996

[Daly 96] Daly, W., Z. Qui and I. Negulesai, "Differential Scanning Calorimetry Study of Aspahlt Crystallinity", Transportation Research Board Record 1535, pp. 54 – 60, 1996

[Decisionengineering 96] Decisionengineering, Crystal Ball Forecasting and Risk Analysis for Spreadsheet Users:Users Manual Microsoft Corporation, California, p. 1 – 275, 1996

[Deme 1996] Deme, Imants, "Prevention of Pavement Cracking and Rutting With Multigrade Type Bitumens", Paper presented to 31st Annual Congress of the Association Québécoise due Transport et des Routes, Québec, Québec, March 24 – 27, 1996

[Duever 98] Duever, T.A., "Statistics in Engineering", Course Notes Chemical Engineering 622, Chapter on the Design and Analysis of Single Factor Experiments, pg. 1 – 35, University of Waterloo, Waterloo, 1998

[Eckmann 87] Eckmann, B., "Rut Depth Prediction: A Practical Verification", Proceedings, 6th International Conference on the Structural Design of Asphalt Pavements, Volume 1., Ann Arbor, 1987

[EBA 93] EBA Engineering Consultant Ltd. Retained by C-SHRP, "Performance Correlation of Quality Paving Asphalt", Interim Report prepared for C-SHRP Project 0101-10537, October 1993

[ERES 98] ERES Consultants, "Review of Life-Cycle Costing Analysis Procedures", Prepared for Ministry of Transportation Ontario, Final Report, ERES Consultants, Champaign Illinos, 1998

[FHWA 98] United States Federal Highway Administration, " Life cycle Cost Analysis In Pavement Design – In Search of Better Investment Decisions", United States Federal Highway Administration, Washington D.C., 1998

[Haas 73] Haas R., "A Method for Designing Asphalt Pavements to Minimize Low-Temperature Shrinkage Cracking", Research Report 73-1 Prepared for the Asphalt Institute, Maryland, 1973

[Haas 83] Haas R., et al, "Study of Asphalt Cement Additives and Extenders", Roads and Transportation Association of Canada, Ottawa, 1983

[Haas 87] Haas, R., H. Lec, F. Meyer and G. Argue, "Cold Climate Performance of Canadian Airport Pavements", Proceedings Paving in Cold Areas, Canada/Japan Science and Technology consultations, Ottawa, Volume 1, Pg. 303 - 332, 1987

[Haas 88] Haas R., and Phang W., "Relationships Between Mix Characteristics and Low Temperature Pavement Cracking", Proceedings Association of Asphalt Pavement Technologists, 1988

[Haas 91] Haas, Ralph, "Low Temperature Cracking of Airport Pavement ", Proceeding 5th Annual International Airport Pavement Maintenance and Management Symposium, Bloomington, Minnesota, April 1991

[Haas 94] Haas, R., W. Hudson, and W. Zaniewski, Modern Pavement Management, Krieger Publishing Company, Florida, 1994

[Haas 97] Haas, R. et al., Pavement Design and Management Guide, Transportation Association of Canada, Ottawa, 1997

[Haas 99] Haas, Ralph, Ningyuan Li and Susan Tighe, "Roughness Trends At C-SHRP LTPP Sites", final Project Report Prepared for Transportation Association of Canada, University of Waterloo, 1999

[Haas 99a] Haas, Ralph and Susan Tighe, "Economic Benefits of Reducing and Treating Reflection Cracking", paper submitted to RILEM Conference, Ottawa, September 1999

[Hajek 95] Hajek, Jerry, Thomas Kazmierowski and Graham Musgrove, "International Roughness Index as a measure of Customer Satisfaction", Paper Presented to Annual Conference, Transportation Association of Canada, Victoria, October 1995

[Hajek 71] Hajek, Jaroslav, "A Comprehensive System for Estimation of Low-Temperature Cracking Frequency of Flexible Pavements", Doctoral Thesis, University of Waterloo, Waterloo, 1971

[Hein 89] Hein, D.K. and M.H. MacKay, "Analysis of Runway Roughness", Transportation Development Centre Publication TP 9718E, Transport Canada, Ottawa, 1989

[Hein 96] Hein, D., J. Emery, J. Carrick, and B. Fraser, "Pavement Performance and Life Cycle Cost Evaluation of a Polymer Modified Asphalt Cement", Proceedings of the 6th Workshop in Cold Areas, Volume 2, Japan, 1996

[Heukelom 73] Heukelom, W., "An Improved Method of Characterizing Asphaltic Bituminous With the Aid of Their Mechanical Properties", Proceedings of Association of Asphalt Paving Technologies, Volume 42, pp. 67-98, 1973

[Hill 90] Hill, Loren and Ralph Haas, "Mulit-year Priorization", Module E of Federal Highway Administration Advanced Course on Pavement Management Systems, November 1990

[Jhanwar 98] Jhanwar, D., L. Zanzotto and J. Berti, "Specification Requirements and Asphalt Properties", Proceedings of Canadian Technical Asphalt Association, pp. 73 - 92, Vancouver, 1998

[Jongepier 69] Jongepier, R., and B. Kuilman, "Characteristics of the Rheology of Bitumens", Proceeding of Association of Asphalt Paving Technologists, Vol 38, pp. 98 - 121, 1969

[Joseph 90] Joseph, P., "Field Evaluation of Rout and Seal Treatment in Flexible Pavement", Report PAV-90-03, Ontario Ministry of Transportation, 1990

[Kandhal 96] Kandhal, K., R. Dongre, and M. Malone, "Prediction of Low-Temperature Cracking of Pennsylvania Project Using Superpave Binder Specification", Proceeding of Association of Asphalt Paving Technologists, Vol 65, pp. 491 - 531, 1996

[Kerr 87] Kerr, W., and Ryan, B., "Avoiding the Pitfalls of Life Cycle Cost Analysis", Report, Washington Economic Research Consultants, 1987

[King 88] King G., H. King, T. Haute, O. Harders, and P. Chaverot, "Benefits of Polymer Modified Asphalts" Proceedings Association of Asphalt Pavement Technologists, Volume 57, 1988

[Lee 95] Lee, N., G. Morrison, and S. Hesp, "Low Temperature Fracture of Polyethylene Modified Asphalt Binders and Asphalt Concrete Mixes", Proceedings Association of Asphalt Pavement Technologists, Volume 64, 1995

[LeFebvre 70] LeFebvre, J.A., "A Modified Penetration Index For Canadian Asphalts", Association of Asphalt Paving Technologists, Volume 39, Pg. 443 – 480, Kansas City Missouri, 1970

[Lipatov 77] Lipatov, A., "Physical Chemistry of Filled Polymers", International Polymer Science Technical Monograph, No. 2, Moscow, 1977

[Maccarrone 94] Maccarrone, S., G. Hollerman, G. Knanseelan, "Polymer Modified Binders and Relationships to Performance", Proceedings 17th Australian Research Board Ltd. Conference, Queensland, 1994

[Marks 85] Marks, V.J. and C. Huisman, "Reducing The Adverse Effects of Transverse Cracking", Association of Asphalt Paving Technologists, Volume 54, Pg. 651 – 667, 1985

[Mason 89] Mason, R.L., R.F. Gunst, and J.L. Hess, Statistical Design and Analysis of Experiments with Applications to Engineering and Science, John Wiley and Sons, Toronto, 1989

[McLeod 87] McLeod, N.W., "The Design of dense Graded Asphalt Concrete Pavements", Proceedings of Canadian Technical Asphalt Association, pp. 352, 1987

[McLeod 78] McLeod, N.W., "Test Data From Three Ontario Test Roads After 15 Years of Service", Proceeding of Canadian Technical Asphalt Association, pp. 600, 1978

[McLeod 76] McLeod, N.W., "Asphalt Cements: Pen-Vis Number and it's Application to Moduli Stiffness", ASTM Journal of Testing and Evaluation", Volume 4, Number 4, 1976

[McLeod 75] McLeod, N.W., "The Case for Grading Asphalt Cements by Penetration at 77°F (25°C)", Canadian Technical Asphalt Association, Volume 20, 1975

[MTO 89] Ontario Ministry of Transportation, Manual For Condition Ratings of Flexible Pavements, Research and Development Branch, Toronto, 1989

[MTO 90] Ontario Ministry of Transportation, Pavement Design and Rehabilitation Manual, Surveys and Design Office, Toronto, 1990

[Nahos 90] Nahos, N., J. Bardet, B. Eckmann and D. Siano, "Binders for High Performance Hot Mix Pavement Binders", Proceedings Association of Asphalt Pavement Technologists, Volume 59, 1990

[Newman 97] Newman, K., "Rheological Properties of Polymer Modified Asphalt Binders", Proceedings Transportation Research Board, Washington D.C., 1997

[NSB 90] National Standards of Canada, "Asphalt Cements for Road Purposes", Canadian General Standard Board, CAN/CGSB-16.3-M90, Ottawa, 1990

[Paterson 86] Paterson, W., "International Roughness Index: Relationship to Other Measures of Roughness and Riding Quality", Transportation Research Record 1084, 1986

[Pfeiffer 50] Pfeiffer, J., "Properties of Asphalt Bitumen", Elsevier publishing Company Inc., New York, 1950

[Ponniah 95] Ponniah, J., and G. Kennepohl, "Polymer Modified Asphalt Pavements in Ontario:Performance and Cost Effectiveness", Ontario Ministry of Transportation, Research and Development Branch, PAV 95-01, 1995

[Ponniah 95] Ponniah, Joseph and Gerhard Kennepohl, "Crack Sealing in Flexible Pavements: A Life-Cycle Cost Analysis", Ontario Ministry of Transportation, Research and Development Branch Paper presented to Transportation Research Board, Washington D.C., 1995

[Puzic 96] Puzic, O., B.U. Achia and L.E. Moran, "Challenges and Opportunities in the Implementation of Superpave Binder Specification", Canadian Technical Asphalt Association, Volume 41, Pg. 178 – 199, Edmonton, 1996

[Reilly 99] Reilly, P., Personal Communications, University of Waterloo, 1999

[Richardson 96] Richardson, S., "Valuation of the Canadian Road and Highway System", Transport Canada Publication TP12794E, Ottawa Ontario, 1996

[Roberts 96] Roberts, F., P. Kandhal, E. Brown, D. Lee and T. Kennedy, "Hot Mix Asphalt Materials, Mixture Design, and Construction", Second Edition NAPA Education Foundation, Maryland, 1996

[Roberts 91] Roberts, F., et al, "Hot Mix Asphalt Materials, Mixture Design, and Construction", NAPA Education Foundation, Maryland, 1991

[Rozeveld 97] Rozeveld, S., E. Shin, A. Bhurke, L. France, and L. Drzal, "Morphology of Straight and Polymer Modified Asphalt Cements", Proceedings Transportation Research Board, Washington D.C., 1997

[RTAC 88] Roads and Transportation Association of Canada, " Transportation in Canada 1992 Report", Transportation Association of Canada, Ottawa, 1992

[Ruth 96] Ruth, B, "Graduate Course Notes: Bituminous Materials", University of Florida, 1996

[Sayers 86] Sayers, M.W. Et al., "Guidelines for Conducting and Calibrating Road Roughness Measurements", World Bandk Technical Paper Number 46, The World Bank, Washington, 1986

[Scheaffer 82] Scheaffer, R., and J. McClave, Statistics for Engineers, Duxbury Press Boston, Boston, p. 2 – 145, 1982

[Scheffé 59] Scheffé, H. The Analysis of Variance, pg. 3 – 5 and pg. 38 – 42, John Wiley and Sons, New York, 1959

[Schweyer 74] Schweyer, H.E., "A Pictorial Review of Asphalt Rheology", Proceeding of Association of Asphalt Paving Technologists", Special Publication, Volume 43A, pp. 121, 1974

[Seddick 95] Seddick, H., and R. Haas, "Comparison of Superpave and Other Models For Predicting Low Temperature Asphalt Pavement Cracking", Proceedings Canadian Technical Asphalt Association, 1995, pp. 197 - 227

[Seddick 95a] Seddick, H., "Establishing fracture Temperatures of Asphaltic Concrete Mixes for Low Temperature Cracking", Masters Thesis, University of Waterloo, 1995

[Shin 96] Shin, E., A. Bhurke, e. Scott, S. Rozeveld, L. Drzal, "Microstructure, Morphology, and Failure Modes of Polymer Modified Pavements", Transportation Research Record 1535, pp. 61 – 73, 1996

[SHRP 95] Asphalt Institute, "Performance Graded Asphalt Binder Specifications", Superpave Series No. 1, SP-1, 1996

[SHRP 95a] Asphalt Institute, "Superpave Mix Design", Superpave Series No. 2, SP-2, 1996

[SHRP 93] SHRP, "Binder Characterization and Evaluation, Volume 2: Chemistry", Strategic Highway Research Program, Special Report, A-368, National Research Council, Washington D.C., 1993

[Simpson 61] Simpson, W., R. Griffen, and T. Miles, "Relationship of Asphalt Properties to Chemical Constitution", Journal Chemical Engineers Data, 6, 1961

[Soleymani 97] Soleymani, H., "Viscoelastic Characterization of Blending Binders For Asphalt Pavement Recycling", Doctorate Thesis, University of Saskatchewan, 1997

[TAC 92] Transportation Association of Canada, " Transportation in Canada 1992 Report", Transportation Association of Canada, Ottawa, 1992

[Tia 87] Tia, M, and B. Ruth, "Basic Rheology and Rheological Concepts Established by H.E. Schweyer", ASTM, STP 941, American Society of Testing and Materials, pp. 118-145, 1987

[Tighe 97] Tighe, S., "The Technical/Performance and Economic Benefits of Modified Asphalts", Masters Thesis, University of Waterloo, 1997

[Tighe 98] Tighe, S., Rehabilitation and Reconstruction Candidates in Ontario, Report for the Canadian Portland Cement Association, Toronto, 1998

[Uzan 96] Uzan J., "Asphalt Concrete Characterization for Pavement Performance Prediction" Proceedings Association of Asphalt Pavement Technologists, Volume 65, 1996

[Van Der Poel 54] Van Der Poel, C., "A General System Describing the Visco-Elastic Properties of Bitumens and It's Relation to Routine Test Data",Journal of Applied Chemistry, Volume 4, pp. 221, 1954

[Witczak 97] Witczak, M., H. VonQuintus, and C. Schwartz, "Superpave Support and Performance Models Management: Evaluation of SHRP Performance Models System", Proceedings International Conference on Asphalt Pavements, Seattle Washington, August 1997

[Xie 98] Xie, W.C., "Civ.E. 601 Class Notes: Engineering Risk and Reliability", Chapter 5: Analysis of Complex Probability Problems Using Monte Carlo Simulation, University of Waterloo, Waterloo, 1997

APPENDIX A

MODULE ONE: MATERIAL CHARACTERIZAION

Table A.1 ANOVA A Asphalt PVN Calculations (Page 1 of 2)

ID	Section	Original PVN		RTFO PVN
		135°C	60°C	60°C
ST	1	-0.7439	-0.7059	-0.7235
ST	2	-0.1658	0.0256	-0.1043
ST	3	-1.2199	-1.2268	3.8150
ST	4	-0.0863	-0.0137	-0.1725
ST	5	-0.2983	-0.1117	-0.1750
ST	6	-0.2647	-0.3429	2.4862
ST	7	-0.2744	-0.1325	2.2224
ST	8	-0.4190	-0.4191	1.9927
ST	9	-0.3694	-0.4728	-0.4779
ST	10	-0.3897	-0.5629	-0.4833
ST	11	-0.4907	-0.5424	-0.4870
ST	12	-0.4436	-0.5387	-0.3422
ST	13	-0.4348	-0.3261	-0.3399
ST	14	-0.4106	-0.5386	-0.5264
ST	15	-0.4249	-0.4959	-0.1230
ST	16	-0.3551	-0.3882	-0.3252
ST	17	-0.6386	-0.7584	-0.7956
ST	18	-0.5148	-0.5953	-0.7848
ST	19	-0.3668	-0.4667	-0.3553
ST	20	-0.7375	-0.7026	-0.7919
ST	21	-0.5346	-0.2792	-0.2778
ST	22	-1.9184	-1.6616	-1.4466
ST	23	-0.5185	-0.5784	-0.7526
ST	24	-3.7129	-3.0352	-2.8124
ST	25	-0.3351	-0.5773	-0.7498
ST	26	-1.8090	-2.3564	-1.8921
ST	27	-0.2099	-0.2199	-0.2539
ST	28	0.0664	-0.0458	0.2350
ST	29	-0.5482	-2.3036	-2.6297
ST	30	-0.9045	-0.7882	-0.5666
ST	31	-0.8978	-0.7262	-1.1048
ST	32	-0.9255	-0.9452	-0.9440
ST	33	-0.1926	0.0192	-0.2526
ST	34	-1.4305	-0.9844	-1.2297
ST	35	-0.8323	-3.5232	-3.4593
ST	36	-0.7262	-3.6531	-3.6820
ST	37	-0.3779	-2.8517	-1.7986
ST	38	-0.0196	0.0094	0.0419
ST	39	-0.5651	-0.2966	-1.9808
ST	40	-1.1532	-3.4885	-3.1014
ST	41	-0.9656	-4.1236	-4.1842
ST	42	-1.0978	-3.0714	-2.6016
ST	43	-0.5600	-3.3234	-3.3279
ST	44	-0.5344	-2.9932	-2.7470
ST	45	-0.5014	-3.8574	-3.8491

Table A.1 ANOVA A Asphalt PVN Calculations (Page 2 of 2)

Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	3	-2.1733	-0.7244	0.0004
Row 2	3	-0.2445	-0.0815	0.0095
Row 3	3	1.3683	0.4561	8.4616
Row 4	3	-0.2725	-0.0908	0.0063
Row 5	3	-0.5850	-0.1950	0.0090
Row 6	3	1.8785	0.6262	2.5961
Row 7	3	1.8156	0.6052	1.9665
Row 8	3	1.1546	0.3849	1.9389
Row 9	3	-1.3201	-0.4400	0.0038
Row 10	3	-1.4360	-0.4787	0.0075
Row 11	3	-1.5201	-0.5067	0.0010
Row 12	3	-1.3246	-0.4415	0.0097
Row 13	3	-1.1009	-0.3670	0.0035
Row 14	3	-1.4756	-0.4919	0.0050
Row 15	3	-1.0438	-0.3479	0.0392
Row 16	3	-1.0685	-0.3562	0.0010
Row 17	3	-2.1926	-0.7309	0.0067
Row 18	3	-1.8949	-0.6316	0.0192
Row 19	3	-1.1889	-0.3963	0.0038
Row 20	3	-2.2321	-0.7440	0.0020
Row 21	3	-1.0915	-0.3638	0.0219
Row 22	3	-5.0267	-1.6756	0.0558
Row 23	3	-1.8496	-0.6165	0.0148
Row 24	3	-9.5605	-3.1868	0.2199
Row 25	3	-1.6623	-0.5541	0.0434
Row 26	3	-6.0576	-2.0192	0.0870
Row 27	3	-0.6837	-0.2279	0.0005
Row 28	3	0.2557	0.0852	0.0200
Row 29	3	-5.4815	-1.8272	1.2533
Row 30	3	-2.2594	-0.7531	0.0295
Row 31	3	-2.7287	-0.9096	0.0360
Row 32	3	-2.8147	-0.9382	0.0001
Row 33	3	-0.4259	-0.1420	0.0204
Row 34	3	-3.6446	-1.2149	0.0499
Row 35	3	-7.8148	-2.6049	2.3578
Row 36	3	-8.0613	-2.6871	2.8840
Row 37	3	-5.0282	-1.6761	1.5413
Row 38	3	0.0317	0.0106	0.0009
Row 39	3	-2.8425	-0.9475	0.8188
Row 40	3	-7.7431	-2.5810	1.5664
Row 41	3	-9.2733	-3.0911	3.3893
Row 42	3	-6.7707	-2.2569	1.0629
Row 43	3	-7.2113	-2.4038	2.5496
Row 44	3	-6.2746	-2.0915	1.8337
Row 45	3	-8.2079	-2.7360	3.7449
Column 1	45	-30.2533	-0.6723	0.3911
Column 2	45	-54.9708	-1.2216	1.6556
Column 3	45	-41.8592	-0.9302	2.7382

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	139.9448	44	3.1806	3.9651	1.9539E-08	1.5147
Columns	6.7968	2	3.3984	4.2366	0.0175	3.1001
Error	70.5890	88	0.8021			
Total	217.3305	134				

Table A.2 ANOVA B Typical Canadian Asphalt PVN Calculations (Page 1 of 2)

ID	**Section**	**Original PVN**		**RTFO PVN**
		135°C	**60°C**	**60°C**
ST	1	-0.7439	-0.7059	-0.7235
ST	2	-0.1658	0.0256	-0.1043
ST	4	-0.0863	-0.0137	-0.1725
ST	5	-0.2983	-0.1117	-0.1750
ST	9	-0.3694	-0.4728	-0.4779
ST	10	-0.3897	-0.5629	-0.4833
ST	11	-0.4907	-0.5424	-0.4870
ST	12	-0.4436	-0.5387	-0.3422
ST	13	-0.4348	-0.3261	-0.3399
ST	14	-0.4106	-0.5386	-0.5264
ST	15	-0.4249	-0.4959	-0.1230
ST	16	-0.3551	-0.3882	-0.3252
ST	17	-0.6386	-0.7584	-0.7956
ST	18	-0.5148	-0.5953	-0.7848
ST	19	-0.3668	-0.4667	-0.3553
ST	20	-0.7375	-0.7026	-0.7919
ST	21	-0.5346	-0.2792	-0.2778
ST	22	-1.9184	-1.6616	-1.4466
ST	23	-0.5185	-0.5784	-0.7526
ST	24	-3.7129	-3.0352	-2.8124
ST	25	-0.3351	-0.5773	-0.7498
ST	26	-1.8090	-2.3564	-1.8921
ST	27	-0.2099	-0.2199	-0.2539
ST	28	0.0664	-0.0458	0.2350
ST	30	-0.9045	-0.7882	-0.5666
ST	31	-0.8978	-0.7262	-1.1048
ST	32	-0.9255	-0.9452	-0.9440
ST	33	-0.1926	0.0192	-0.2526
ST	38	-0.0196	0.0094	0.0419

Table A.2 ANOVA B Typical Canadian Asphalt PVN Calculations (Page 2 of 2)

Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	3	-2.1733	-0.7244	0.0004
Row 2	3	-0.2445	-0.0815	0.0095
Row 3	3	-0.2725	-0.0908	0.0063
Row 4	3	-0.5850	-0.1950	0.0090
Row 5	3	-1.3201	-0.4400	0.0038
Row 6	3	-1.4360	-0.4787	0.0075
Row 7	3	-1.5201	-0.5067	0.0010
Row 8	3	-1.3246	-0.4415	0.0097
Row 9	3	-1.1009	-0.3670	0.0035
Row 10	3	-1.4756	-0.4919	0.0050
Row 11	3	-1.0438	-0.3479	0.0392
Row 12	3	-1.0685	-0.3562	0.0010
Row 13	3	-2.1926	-0.7309	0.0067
Row 14	3	-1.8949	-0.6316	0.0192
Row 15	3	-1.1889	-0.3963	0.0038
Row 16	3	-2.2321	-0.7440	0.0020
Row 17	3	-1.0915	-0.3638	0.0219
Row 18	3	-5.0267	-1.6756	0.0558
Row 19	3	-1.8496	-0.6165	0.0148
Row 20	3	-9.5605	-3.1868	0.2199
Row 21	3	-1.6623	-0.5541	0.0434
Row 22	3	-6.0576	-2.0192	0.0870
Row 23	3	-0.6837	-0.2279	0.0005
Row 24	3	0.2557	0.0852	0.0200
Row 25	3	-2.2594	-0.7531	0.0295
Row 26	3	-2.7287	-0.9096	0.0360
Row 27	3	-2.8147	-0.9382	0.0001
Row 28	3	-0.4259	-0.1420	0.0204
Row 29	3	0.0317	0.0106	0.0009
Column 1	29	-18.7828	-0.6477	0.5438
Column 2	29	-18.3794	-0.6338	0.4567
Column 3	29	-17.7841	-0.6132	0.3724

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	37.1017	28	1.3251	55.4561	1.6614E-31	1.6775
Columns	0.0174	2	0.0087	0.3643	0.6963	3.1619
Error	1.3381	56	0.0239			
Total	38.45721	86				

Table A.3 ANOVA C PVN Calculations on Original Asphalt (Page 1 of 2)

ID	Section	Original PVN	
		135°C	60°C
ST	1	-0.7439	-0.7059
ST	2	0.1658	0.0256
ST	4	-0.0863	-0.0137
ST	5	-0.2983	-0.1117
ST	9	-0.3694	-0.4728
ST	10	-0.3897	-0.5629
ST	11	-0.4907	-0.5424
ST	12	-0.4436	-0.5387
ST	13	-0.4348	-0.3261
ST	14	-0.4106	-0.5386
ST	15	-0.4249	-0.4959
ST	16	-0.3551	-0.3882
ST	17	-0.6386	-0.7584
ST	18	-0.5148	-0.5953
ST	19	-0.3668	-0.4667
ST	20	-0.7375	-0.7026
ST	21	-0.5346	-0.2792
ST	22	-1.9184	-1.6616
ST	23	-0.5185	-0.5784
ST	24	-3.7129	-3.0352
ST	25	-0.3351	-0.5773
ST	26	-1.8090	-2.3564
ST	27	-0.2099	-0.2199
ST	28	0.0664	-0.0458
ST	30	-0.9045	-0.7882
ST	31	-0.8978	-0.7262
ST	32	-0.9255	-0.9452
ST	33	-0.1926	0.0192
ST	38	-0.0196	0.0094
ST	46	-0.6397	-0.7333
ST	47	-0.6335	-0.7526
ST	48	-0.4981	-0.5582
ST	49	-0.4975	-0.4891
ST	50	-0.5335	-0.4845
ST	51	-2.3014	-2.9576
ST	52	0.0007	0.0775
ST	52	-0.3163	-0.0592
ST	53	-0.3129	-0.1936
ST	54	-0.4313	-0.2956
ST	55	-0.1772	-0.0367
ST	56	-0.1208	-0.0490
ST	57	-0.3582	-0.2508
ST	58	-0.2031	0.0673
ST	59	-1.4305	-0.9844
ST	60	-0.5651	-0.2966
ST	80	-0.2647	-0.3429
ST	81	-0.2744	-0.1325
ST	82	-0.4190	-0.4191
ST	91	-0.3545	-0.3477

Table A.3 ANOVA C PVN Calculations on Original Asphalt (Page 2 of 2)

Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	2	-1.4498	-0.7249	0.0007
Row 2	2	-2.4467	-1.2234	0.0000
Row 3	2	-0.1000	-0.0500	0.0026
Row 4	2	-0.4101	-0.2050	0.0174
Row 5	2	-0.6076	-0.3038	0.0031
Row 6	2	-0.4068	-0.2034	0.0101
Row 7	2	-0.8381	-0.4190	0.0000
Row 8	2	-1.3730	-0.6865	0.0044
Row 9	2	-1.3861	-0.6930	0.0071
Row 10	2	-1.0563	-0.5281	0.0018
Row 11	2	-0.9866	-0.4933	0.0000
Row 12	2	-1.0180	-0.5090	0.0012
Row 13	2	-0.8421	-0.4211	0.0053
Row 14	2	-0.9527	-0.4763	0.0150
Row 15	2	-1.0331	-0.5165	0.0013
Row 16	2	-0.9823	-0.4912	0.0045
Row 17	2	-0.7610	-0.3805	0.0059
Row 18	2	-5.2590	-2.6295	0.2153
Row 19	2	-0.9492	-0.4746	0.0082
Row 20	2	-0.9208	-0.4604	0.0025
Row 21	2	-0.7434	-0.3717	0.0005
Row 22	2	-1.3970	-0.6985	0.0072
Row 23	2	-1.1101	-0.5551	0.0032
Row 24	2	-0.8336	-0.4168	0.0050
Row 25	2	-1.4401	-0.7201	0.0006
Row 26	2	-0.8137	-0.4069	0.0326
Row 27	2	-3.5800	-1.7900	0.0330
Row 28	2	0.0782	0.0391	0.0030
Row 29	2	-1.0970	-0.5485	0.0018
Row 30	2	-6.7481	-3.3741	0.2296
Row 31	2	-0.9124	-0.4562	0.0293
Row 32	2	-4.1654	-2.0827	0.1498
Row 33	2	-0.4298	-0.2149	0.0001
Row 34	2	0.0207	0.0103	0.0063
Row 35	2	-1.6928	-0.8464	0.0068
Row 36	2	-1.6239	-0.8120	0.0147
Row 37	2	-1.8707	-0.9353	0.0002
Row 38	2	-0.1733	-0.0867	0.0224
Row 39	2	-0.3754	-0.1877	0.0330
Row 40	2	-0.5065	-0.2532	0.0071
Row 41	2	-0.7021	-0.3511	0.0000
Row 42	2	-0.7269	-0.3635	0.0092
Row 43	2	-0.2139	-0.1070	0.0099
Row 44	2	-0.1698	-0.0849	0.0026
Row 45	2	-0.6091	-0.3045	0.0058
Row 46	2	-0.1358	-0.0679	0.0366
Row 47	2	-2.4149	-1.2075	0.0995
Row 48	2	-0.0102	-0.0051	0.0004
Row 49	2	-0.8617	-0.4309	0.0360
Column 1	49	-30.16793207	-0.615672083	0.428684
Column 2	49	-28.8703351	-0.589190512	0.442947

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	40.7627	48	0.8492	37.8962	6.31E-26	1.6154
Columns	0.0172	1	0.0172	0.7667	0.3856	4.0426
Error	1.0756	48	0.0224			
Total	41.85547	97				

Table A.4 ANOVA D PVN Calculations on Original and Aged Asphalt @ 60°C (Page 1 of 2)

ID	Section	Original	RTFO
		60°C	60°C
ST	1	-0.7059	-0.7235
ST	2	0.0256	-0.1043
ST	4	-0.0137	-0.1725
ST	5	-0.1117	-0.1750
ST	9	-0.4728	-0.4779
ST	10	-0.5629	-0.4833
ST	11	-0.5424	-0.4870
ST	12	-0.5387	-0.3422
ST	13	-0.3261	-0.3399
ST	14	-0.5386	-0.5264
ST	15	-0.4959	-0.1230
ST	16	-0.3882	-0.3252
ST	17	-0.7584	-0.7956
ST	18	-0.5953	-0.7848
ST	19	-0.4667	-0.3553
ST	20	-0.7026	-0.7919
ST	21	-0.2792	-0.2778
ST	22	-1.6616	-1.4466
ST	23	-0.5784	-0.7526
ST	24	-3.0352	-2.8124
ST	25	-0.5773	-0.7498
ST	26	-2.3564	-1.8921
ST	27	-0.2199	-0.2539
ST	28	-0.0458	0.2350
ST	30	-0.7882	-0.5666
ST	31	-0.7262	-1.1048
ST	32	-0.9452	-0.9440
ST	33	0.0192	-0.2526
ST	38	0.0094	0.0419
ST	48	-0.5582	-0.5284
ST	49	-0.4891	-0.4371
ST	50	-0.4845	-0.5406
ST	59	-0.9844	-1.2297
ST	61	-3.5232	-3.4593
ST	62	-3.6531	-3.6820
ST	63	-2.8517	-1.7986
ST	64	-0.2966	-1.9808
ST	65	-3.4885	-3.1014
ST	66	-4.1236	-4.1842
ST	67	-3.0714	-2.6016
ST	68	-3.3234	-3.3279
ST	69	-2.9932	-2.7470
ST	70	-3.8574	-3.8491
ST	80	-0.3429	2.4862
ST	81	-0.1325	2.2224
ST	82	-0.4191	1.9927
ST	83	-2.3036	-2.6297
ST	84	-1.2268	3.8150

Table A.4 ANOVA D PVN Calculations on Original and Aged Asphalt @ 60°C (Page 2 of 2)
Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	2	-1.4294	-0.7147	0.0002
Row 2	2	-0.0787	-0.0394	0.0084
Row 3	2	2.5881	1.2941	12.7099
Row 4	2	-0.1861	-0.0931	0.0126
Row 5	2	-0.2867	-0.1433	0.0020
Row 6	2	2.1432	1.0716	4.0019
Row 7	2	2.0899	1.0450	2.7726
Row 8	2	1.5737	0.7868	2.9084
Row 9	2	-0.9507	-0.4754	0.0000
Row 10	2	-1.0462	-0.5231	0.0032
Row 11	2	-1.0294	-0.5147	0.0015
Row 12	2	-0.8810	-0.4405	0.0193
Row 13	2	-0.6660	-0.3330	0.0001
Row 14	2	-1.0650	-0.5325	0.0001
Row 15	2	-0.6189	-0.3095	0.0696
Row 16	2	-0.7134	-0.3567	0.0020
Row 17	2	-1.5540	-0.7770	0.0007
Row 18	2	-1.3801	-0.6901	0.0179
Row 19	2	-0.8221	-0.4110	0.0062
Row 20	2	-1.4946	-0.7473	0.0040
Row 21	2	-0.5569	-0.2785	0.0000
Row 22	2	-3.1082	-1.5541	0.0231
Row 23	2	-1.3311	-0.6655	0.0152
Row 24	2	-5.8476	-2.9238	0.0248
Row 25	2	-1.3271	-0.6636	0.0149
Row 26	2	-4.2485	-2.1243	0.1078
Row 27	2	-0.4739	-0.2369	0.0006
Row 28	2	0.1892	0.0946	0.0394
Row 29	2	-4.9332	-2.4666	0.0532
Row 30	2	-1.3549	-0.6774	0.0246
Row 31	2	-1.8310	-0.9155	0.0717
Row 32	2	-1.8892	-0.9446	0.0000
Row 33	2	-0.2333	-0.1167	0.0369
Row 34	2	0.0513	0.0257	0.0005
Row 35	2	-1.0866	-0.5433	0.0004
Row 36	2	-0.9262	-0.4631	0.0014
Row 37	2	-1.0251	-0.5126	0.0016
Row 38	2	-2.2141	-1.1071	0.0301
Row 39	2	-6.9825	-3.4913	0.0020
Row 40	2	-7.3351	-3.6676	0.0004
Row 41	2	-4.6504	-2.3252	0.5545
Row 42	2	-2.2774	-1.1387	1.4183
Row 43	2	-6.5898	-3.2949	0.0749
Row 44	2	-8.3077	-4.1539	0.0018
Row 45	2	-5.6730	-2.8365	0.1104
Row 46	2	-6.6513	-3.3257	0.0000
Row 47	2	-5.7402	-2.8701	0.0303
Row 48	2	-7.7064	-3.8532	0.0000
Column 1	48	-56.5026	-1.1771	1.5802
Column 2	48	-43.3653	-0.9034	2.5745

ANOVA

ource of Variatio	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	171.8915	47	3.6573	7.3516	1.15E-10	1.6238
Columns	1.7978	1	1.7978	3.6138	0.0634	4.0471
Error	23.3817	47	0.4975			
Total	197.0710	95				

Table A.5 ANOVA E PVN Calculations on Original @135°C and Aged Asphalt @ 60°C (Page 1 of 2)

ID	Section	Original	RTFO
		135°C	60°C
ST	1	-0.7439	-0.7235
ST	2	-0.1658	-0.1043
ST	4	-0.0863	-0.1725
ST	5	-0.2983	-0.1750
ST	9	-0.3694	-0.4779
ST	10	-0.3897	-0.4833
ST	11	-0.4907	-0.4870
ST	12	-0.4436	-0.3422
ST	13	-0.4348	-0.3399
ST	14	-0.4106	-0.5264
ST	15	-0.4249	-0.1230
ST	16	-0.3551	-0.3252
ST	17	-0.6386	-0.7956
ST	18	-0.5148	-0.7848
ST	19	-0.3668	-0.3553
ST	20	-0.7375	-0.7919
ST	21	-0.5346	-0.2778
ST	22	-1.9184	-1.4466
ST	23	-0.5185	-0.7526
ST	24	-3.7129	-2.8124
ST	25	-0.3351	-0.7498
ST	26	-1.8090	-1.8921
ST	27	-0.2099	-0.2539
ST	28	0.0664	0.2350
ST	30	-0.9045	-0.5666
ST	31	-0.8978	-1.1048
ST	32	-0.9255	-0.9440
ST	33	-0.1926	-0.2526
ST	38	-0.0196	0.0419
ST	46	-0.7914	-4.5016
ST	48	-0.4981	-0.5284
ST	49	-0.4975	-0.4371
ST	50	-0.5335	-0.5406
ST	59	-1.4305	-1.2297
ST	71	-0.6105	-0.6093
ST	72	-0.2496	2.3070
ST	73	-0.3470	-0.3829
ST	74	-0.2291	0.1016
ST	75	-0.3060	-0.0967
ST	76	0.4623	0.3543
ST	77	-0.0722	0.6871
ST	78	-0.4362	-0.5511
ST	80	-0.2647	2.4862
ST	81	-0.2744	2.2224
ST	82	-0.4190	1.9927
ST	83	-0.5482	-2.6297
ST	84	-1.2199	3.8150
ST	85	-0.8323	-3.4593
ST	86	-0.7262	-3.6820
ST	87	-0.3779	-1.7986
ST	88	-0.5651	-1.9808
ST	89	-1.1532	-3.1014
ST	90	-0.9656	-4.1842
ST	92	-1.0978	-2.6016
ST	93	-0.5600	-3.3279
ST	94	-0.5344	-2.7470
ST	95	-0.5014	-3.8491

Table A.5 ANOVA E PVN Calculations on Original @135°C and Aged Asphalt @ 60°C (Page 2 of 2)
Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	2	-1.4674	-0.7337	0.0002
Row 2	2	-0.2701	-0.1350	0.0019
Row 3	2	2.5951	1.2975	12.6749
Row 4	2	-0.2588	-0.1294	0.0037
Row 5	2	-0.4733	-0.2367	0.0076
Row 6	2	2.2215	1.1107	3.7835
Row 7	2	1.9480	0.9740	3.1169
Row 8	2	1.5737	0.7869	2.9083
Row 9	2	-0.8473	-0.4236	0.0059
Row 10	2	-0.8731	-0.4365	0.0044
Row 11	2	-0.9776	-0.4888	0.0000
Row 12	2	-0.7858	-0.3929	0.0051
Row 13	2	-0.7748	-0.3874	0.0045
Row 14	2	-0.9370	-0.4685	0.0067
Row 15	2	-0.5478	-0.2739	0.0456
Row 16	2	-0.6803	-0.3402	0.0004
Row 17	2	-1.4342	-0.7171	0.0123
Row 18	2	-1.2996	-0.6498	0.0364
Row 19	2	-0.7222	-0.3611	0.0001
Row 20	2	-1.5294	-0.7647	0.0015
Row 21	2	-0.8123	-0.4062	0.0330
Row 22	2	-3.3651	-1.6825	0.1113
Row 23	2	-1.2712	-0.6356	0.0274
Row 24	2	-6.5253	-3.2626	0.4054
Row 25	2	-1.0849	-0.5425	0.0860
Row 26	2	-3.7012	-1.8506	0.0035
Row 27	2	-0.4638	-0.2319	0.0010
Row 28	2	0.3015	0.1507	0.0142
Row 29	2	-3.1779	-1.5890	2.1662
Row 30	2	-1.4712	-0.7356	0.0571
Row 31	2	-2.0026	-1.0013	0.0214
Row 32	2	-1.8695	-0.9347	0.0002
Row 33	2	-0.4451	-0.2226	0.0018
Row 34	2	-4.2916	-2.1458	3.4508
Row 35	2	-4.4082	-2.2041	4.3684
Row 36	2	-2.1765	-1.0882	1.0093
Row 37	2	0.0223	0.0112	0.0019
Row 38	2	-2.5459	-1.2730	1.0022
Row 39	2	-4.2546	-2.1273	1.8977
Row 40	2	-5.1497	-2.5749	5.1796
Row 41	2	-3.6993	-1.8497	1.1307
Row 42	2	-3.8879	-1.9440	3.8306
Row 43	2	-3.2814	-1.6407	2.4479
Row 44	2	-4.3505	-2.1753	5.6034
Row 45	2	-5.2930	-2.6465	6.8828
Row 46	2	-1.0266	-0.5133	0.0005
Row 47	2	-0.9346	-0.4673	0.0018
Row 48	2	-1.0741	-0.5371	0.0000
Row 49	2	-2.6602	-1.3301	0.0202
Row 50	2	-1.2198	-0.6099	0.0000
Row 51	2	2.0574	1.0287	3.2679
Row 52	2	-0.7299	-0.3649	0.0006
Row 53	2	-0.1275	-0.0637	0.0547
Row 54	2	-0.4026	-0.2013	0.0219
Row 55	2	0.8166	0.4083	0.0058
Row 56	2	0.6149	0.3074	0.2882
Row 57	2	-0.9872	-0.4936	0.0066
Column 1	57	-34.3620	-0.6028	0.3455
Column 2	57	-46.0569	-0.8080	2.6784

ANOVA

ource of Variatio	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	104.5189	56	1.8664	1.6124	0.0383	1.5579
Columns	1.1997	1	1.1997	1.0365	0.3130	4.0130
Error	64.8220	56	1.1575			
Total	170.5407	113				

Table A.6 ANOVA F PVN Calculations Over Time (Page 1 of 1)

Asphalt Sample Number	Original Asphalt	Short Term Aged	After Construction	7 Years In-Service
1	-1.04	-1.07	-1.13	-1.12
2	-0.70	-0.73	-0.68	-0.60
3	-0.61	-0.60	-0.72	-0.56
4	-0.86	-0.83	-1.03	-0.79
5	-1.03	-1.02	-1.16	-1.12
6	-0.45	-0.58	-0.47	-0.39

Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	4	-4.3576	-1.0894	0.0018
Row 2	4	-2.7187	-0.6797	0.0029
Row 3	4	-2.4911	-0.6228	0.0047
Row 4	4	-3.5139	-0.8785	0.0109
Row 5	4	-4.3287	-1.0822	0.0045
Row 6	4	-1.8944	-0.4736	0.0060
Column 1	6	-4.7001	-0.7833	0.0558
Column 2	6	-4.8305	-0.8051	0.0431
Column 3	6	-5.1917	-0.8653	0.0789
Column 4	6	-4.5822	-0.7637	0.0912

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	1.2874	5	0.2575	67.2410	9.83765E-10	2.9013
Columns	0.0348	3	0.0116	3.0336	0.0620	3.2874
Error	0.0574	15	0.0038			
Total	1.3797	23				

Table A.7 Power Calculations For ANOVA (Page 1 of 4)

Power Calculations for ANOVA B, Appendix A.2

a	Sum of Squares	K	Degrees of Freedom	Variance	Number of Columns	Phi	Tables
0.000	2	0.0000	2	0.0239	29	0.0000	
0.005	2	0.0001	2	0.0239	29	0.1422	
0.010	2	0.0002	2	0.0239	29	0.2844	
0.015	2	0.0005	2	0.0239	29	0.4266	
0.020	2	0.0008	2	0.0239	29	0.5688	
0.025	2	0.0013	2	0.0239	29	0.7110	
0.030	2	0.0018	2	0.0239	29	0.8532	
0.035	2	0.0025	2	0.0239	29	0.9955	
0.040	2	0.0032	2	0.0239	29	1.1377	0.35
0.045	2	0.0041	2	0.0239	29	1.2799	0.44
0.050	2	0.0050	2	0.0239	29	1.4221	0.54
0.055	2	0.0061	2	0.0239	29	1.5643	0.62
0.060	2	0.0072	2	0.0239	29	1.7065	0.73
0.065	2	0.0085	2	0.0239	29	1.8487	0.78
0.070	2	0.0098	2	0.0239	29	1.9909	0.84
0.075	2	0.0113	2	0.0239	29	2.1331	0.89
0.080	2	0.0128	2	0.0239	29	2.2753	0.915
0.085	2	0.0145	2	0.0239	29	2.4175	0.958
0.090	2	0.0162	2	0.0239	29	2.5597	0.967
0.095	2	0.0181	2	0.0239	29	2.7020	0.985
0.100	2	0.0200	2	0.0239	29	2.8442	0.989

Power Calculations for ANOVA C, Appendix A.3

A = [-1 1]

a	Sum of Squares	K	Degrees of Freedom	Variance	Number of Columns	Phi	Tables
0.000	2	0.0000	1	0.0224	49	0.0000	
0.010	2	0.0002	1	0.0224	49	0.4677	
0.020	2	0.0008	1	0.0224	49	0.9354	
0.030	2	0.0018	1	0.0224	49	1.4031	
0.040	2	0.0032	1	0.0224	49	1.8708	
0.045	2	0.0041	1	0.0224	49	2.1047	0.65
0.050	2	0.0050	1	0.0224	49	2.3385	0.71
0.055	2	0.0061	1	0.0224	49	2.5724	0.79
0.060	2	0.0072	1	0.0224	49	2.8062	0.895
0.065	2	0.0085	1	0.0224	49	3.0401	0.937
0.070	2	0.0098	1	0.0224	49	3.2740	0.971
0.075	2	0.0113	1	0.0224	49	3.5078	0.985

a	Sum of Squares	K	Degrees of Freedom	Variance	Number of Columns	Phi	Tables
0.080	2	0.0128	1	0.0224	49	3.7417	0.99
0.085	2	0.0145	1	0.0224	49	3.9755	
0.090	2	0.0162	1	0.0224	49	4.2094	
0.095	2	0.0181	1	0.0224	49	4.4432	
0.100	2	0.0200	1	0.0224	49	4.6771	
0.105	2	0.0221	1	0.0224	49	4.9109	
0.110	2	0.0242	1	0.0224	49	5.1448	
0.115	2	0.0265	1	0.0224	49	5.3786	
0.120	2	0.0288	1	0.0224	49	5.6125	

Power Calculations for ANOVA D, Appendix A.4

A = [-4 4]

a	Sum of Squares	K	Degrees of Freedom	Variance	Number of Columns	Phi	Tables
0.000	32	0.0000	1	0.4975	48	0.0000	
0.010	32	0.0032	1	0.4975	48	0.3929	
0.020	32	0.0128	1	0.4975	48	0.7858	
0.030	32	0.0288	1	0.4975	48	1.1787	
0.040	32	0.0512	1	0.4975	48	1.5716	
0.050	32	0.0800	1	0.4975	48	1.9645	
0.055	32	0.0972	1	0.4975	48	2.1649	0.63
0.060	32	0.1160	1	0.4975	48	2.3653	0.72
0.065	32	0.1365	1	0.4975	48	2.5656	0.79
0.070	32	0.1586	1	0.4975	48	2.7660	0.87
0.076	32	0.1824	1	0.4975	48	2.9664	0.92
0.081	32	0.2079	1	0.4975	48	3.1668	0.955
0.086	32	0.2350	1	0.4975	48	3.3672	0.975
0.091	32	0.2638	1	0.4975	48	3.5675	0.987
0.096	32	0.2943	1	0.4975	48	3.7679	
0.101	32	0.3264	1	0.4975	48	3.9683	
0.106	32	0.3602	1	0.4975	48	4.1687	
0.111	32	0.3957	1	0.4975	48	4.3691	
0.116	32	0.4328	1	0.4975	48	4.5694	
0.121	32	0.4716	1	0.4975	48	4.7698	
0.127	32	0.5121	1	0.4975	48	4.9702	

Power Calculations for ANOVA E, Appendix A.5

A = [-1 1]

a	Sum of	K	Degrees of	Variance	Number of	Phi	Tables

a	Sum of Squares	K	Degrees of Freedom	Variance	Number of Columns	Phi	Tables
	Squares		Freedom		Columns		
0.000	2	0.0000	1	1.1575	57	0.0000	
0.025	2	0.0013	1	1.1575	57	0.1754	
0.050	2	0.0050	1	1.1575	57	0.3509	
0.075	2	0.0113	1	1.1575	57	0.5263	
0.100	2	0.0200	1	1.1575	57	0.7017	
0.125	2	0.0313	1	1.1575	57	0.8772	
0.150	2	0.0450	1	1.1575	57	1.0526	
0.175	2	0.0613	1	1.1575	57	1.2280	
0.200	2	0.0800	1	1.1575	57	1.4035	
0.225	2	0.1013	1	1.1575	57	1.5789	
0.250	2	0.1250	1	1.1575	57	1.7544	
0.275	2	0.1513	1	1.1575	57	1.9298	
0.300	2	0.1800	1	1.1575	57	2.1052	0.62
0.325	2	0.2113	1	1.1575	57	2.2807	0.715
0.350	2	0.2450	1	1.1575	57	2.4561	0.78
0.375	2	0.2813	1	1.1575	57	2.6315	0.85
0.400	2	0.3200	1	1.1575	57	2.8070	0.89
0.425	2	0.3613	1	1.1575	57	2.9824	0.93
0.450	2	0.4050	1	1.1575	57	3.1578	0.955
0.475	2	0.4513	1	1.1575	57	3.3333	0.975
0.500	2	0.5000	1	1.1575	57	3.5087	0.987

Power Calculations for ANOVA F, Appendix A.6

A = [-3 -1 0 1 3]

a	Sum of Squares	K	Degrees of Freedom	Variance	Number of Columns	Phi	Tables
0.000	20	0.0000	3	0.003829	6	0.0000	
0.010	20	0.0020	3	0.003829	6	0.8852	
0.015	20	0.0045	3	0.003829	6	1.3277	0.32
0.020	20	0.0080	3	0.003829	6	1.7703	0.55
0.021	20	0.0087	3	0.003829	6	1.8411	0.6
0.022	20	0.0093	3	0.003829	6	1.9119	0.64
0.022	20	0.0100	3	0.003829	6	1.9827	0.67
0.023	20	0.0108	3	0.003829	6	2.0536	0.69
0.024	20	0.0115	3	0.003829	6	2.1244	0.723
0.025	20	0.0123	3	0.003829	6	2.1952	0.75
0.026	20	0.0131	3	0.003829	6	2.2660	0.79
0.026	20	0.0139	3	0.003829	6	2.3368	0.815

Table A.7 Power Calculations For ANOVA (Page 4 of 4)

a	Sum of Squares	K	Degrees of Freedom	Variance	Number of Columns	Phi	Tables
0.027	20	0.0148	3	0.003829	6	2.4076	0.83
0.028	20	0.0157	3	0.003829	6	2.4784	0.85
0.029	20	0.0166	3	0.003829	6	2.5492	0.865
0.030	20	0.0175	3	0.003829	6	2.6201	0.89
0.030	20	0.0185	3	0.003829	6	2.6909	0.91
0.031	20	0.0195	3	0.003829	6	2.7617	0.92
0.032	20	0.0205	3	0.003829	6	2.8325	0.93
0.033	20	0.0215	3	0.003829	6	2.9033	0.945
0.034	20	0.0226	3	0.003829	6	2.9741	0.954
0.034	20	0.0237	3	0.003829	6	3.0449	0.957
0.035	20	0.0248	3	0.003829	6	3.1157	0.968
0.036	20	0.0259	3	0.003829	6	3.1865	0.975
0.037	20	0.0271	3	0.003829	6	3.2574	0.975
0.038	21	0.0297	3	0.003829	6	3.4104	0.987
0.038	22	0.0324	3	0.003829	6	3.5649	0.99
0.039	23	0.0353	3	0.003829	6	3.7209	

Table A.8 ANOVA PG/PVN Comparison (Page 1 of 2)
ANOVA PVN Comparison of PG 58 - 22 and PG 58 - 28

	PG 58 - 22	PG 58 - 28
	-0.5284	-0.3422
	-0.9045	-0.4656
	-1.1814	-0.4037
	-0.6253	-0.3602
	-0.5407	-0.0963
	-0.6680	-0.1287
	-0.4610	-0.1373
	-0.6719	-0.9258
	-0.6818	-0.6191
	-0.6859	-0.4381
	-0.7101	-0.5790
	-0.6402	-0.6554
	-0.6618	-0.4966
	-0.6395	-0.5374
	-0.7074	-0.7657
	-0.8313	-0.7069
	-0.7773	-0.3129
	-0.5849	-0.7196
	-0.6680	-0.7277
	-0.6663	-0.3692
	-0.6919	-0.7498
	-0.6456	-0.5020
	-0.5059	-0.7216
	-1.2527	-0.6445
	-	-0.6311
	-	-0.4118
	-	-0.5790
	-	-0.3656
Average	**-0.7055**	**-0.5140**
Standard Deviation	**0.1849**	**0.2073**

PG 58 - 22 and PG 58 - 28
Anova: Single Factor

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
Column 1	24	-16.9318	-0.7055	0.0342
Column 2	24	-12.4051	-0.5169	0.0482

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	0.4269	1	0.4269	10.3573	0.0024	4.0517
Within Groups	1.8959	46	0.0412			
Total	2.3228	47				

ANOVA PVN Comparison of PG 52 - 28 and PG 52 - 34

	PG 52 - 28	PG 52 - 34
	-0.5766	-0.1923
	-0.4362	-0.0563
	-0.6860	-0.1208
	-0.6171	-0.3062
	-0.7481	-0.2260
	-0.4578	-0.4406
	-0.3579	-
	-0.2572	-
	-0.4037	-
Average	-0.5045	-0.2237
Standard Deviation	0.1619	0.1367

PG 52 - 28 and PG 52 - 34
Anova: Single Factor

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
Column 1	6	-3.5218	-0.5870	0.0152
Column 2	6	-1.3422	-0.2237	0.0187

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	0.3959	1	0.3959	23.3339	0.0007	4.9646
Within Groups	0.1697	10	0.0170			
Total	0.5655	11				

Table A.8 ANOVA PG/PVN Comparison (Page 2 of 2)
ANOVA PVN Comparison of PG 64 - 22 and PG 64 - 28

	PG 64 - 22	PG 64 - 28
	-0.6494	-0.2000
	-0.8793	-0.5637
	-0.6702	-0.5368
	-0.6770	-0.3190
	-0.7546	-0.3024
	-0.6417	-0.3024
	-0.3245	-
	-0.6494	-
	-0.4850	-
	-0.6770	-
	-0.5427	-
Average	-0.6319	-0.3707
Standard Deviation	0.1437	0.1456

PG 64 - 22 and PG 64 - 28
Anova: Single Factor

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
Column 1	6	-4.2721	-0.7120	0.0083
Column 2	6	-2.2242	-0.3707	0.0212

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	0.3495	1	0.3495	23.6774	0.0007	4.9646
Within Groups	0.1476	10	0.0148			
Total	0.4971	11				

ANOVA PVN Comparison of PG 46 - 28 and PG 46 - 34

	PG 46 - 28	PG 46 - 34
	-1.6873	-0.1814
	-0.3064	0.1868
	-0.5260	-0.0144
	-0.5087	-0.3099
	-	-0.1702
	-	-0.5021
	-	-0.1777
	-	-0.2031
	-	-0.1857
	-	-0.3208
	-	-0.3362
	-	0.5938
	-	-0.1890
	-	-0.4167
	-	-0.1329
	-	-0.3889
	-	-0.5063
	-	-0.0451
	-	-0.1986
Average	-0.7571	-0.1841
Standard Deviation	0.6281	0.2531

PG 46 - 28 and PG 46 - 34
Anova: Single Factor

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
Column 1	4	-3.0283	-0.7571	0.3945
Column 2	4	-0.3190	-0.0797	0.0462

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	0.9176	1	0.9176	4.1640	0.0874	5.9874
Within Groups	1.3222	6	0.2204			
Total	2.2398	7				

Table A.9 ANOVA CRUDE SOURCE/PVN Comparison (Page 1 of 4)
ANOVA PVN Comparison with 3 PVN's/Crude Source

CRUDE SOURCE	PVN	CRUDE SOURCE	PVN
A	-0.8313	X	-0.1986
A	-0.8603	X	-0.3656
A	-0.7980	X	-0.5427
B	-0.3707	Y	-1.0616
B	-0.0643	Y	-1.0067
B	-0.9121	Y	-0.8119
C	-1.0703	Z	-0.1086
C	-2.1234	Z	-0.5844
C	-3.0937	Z	-0.7112
D	-0.3501	AA	-0.9110
D	-0.2810	AA	-0.5831
D	-1.7522	AA	-1.3718
E	-0.7545	BB	-0.0451
E	-0.6818	BB	-0.4118
E	-0.8316	BB	-0.3617
F	-0.3033	CC	-0.8734
F	-0.3555	CC	-0.8733
F	-0.5325	CC	-0.9588
G	-0.7463	DD	-0.8172
G	-0.8497	DD	-0.5543
G	-0.9600	DD	-1.6529
H	-1.0481	EE	-0.6779
H	-2.1553	EE	-1.0103
H	-3.1775	EE	-1.0237
I	-0.5816	FF	-0.6263
I	-0.3413	FF	-0.5863
I	-0.6192	FF	-0.5164
J	-1.0109	GG	-1.7557
J	-0.6445	GG	-2.7631
J	-0.4575	GG	-3.6578
K	-3.0486	HH	-0.8900
K	-3.8042	HH	-0.8222
K	-4.7073	HH	-0.7481
L	-2.2398	II	-2.2154
L	-2.1858	II	-2.4189
L	-2.0240	II	-2.4841
M	-1.3650	JJ	0.3205
M	-2.3829	JJ	0.2012
M	-3.3520	JJ	-0.4368
N	-0.2745	KK	-0.7001
N	-0.8875	KK	-2.5693
N	-1.9401	KK	-0.8002
O	-1.3962	LL	-0.9726
O	-2.3957	LL	-0.8432
O	-3.2682	LL	-0.5898
P	-1.0443	MM	-0.9570
P	-0.8629	MM	-0.7781
P	-1.4308	MM	-1.0073
Q	-0.2022	NN	-0.8650
Q	-0.2606	NN	-0.8340
Q	-0.1246	NN	-0.8335
R	-2.3792	OO	-1.5490
R	-2.3051	OO	-1.6873
R	-2.3722	OO	-1.5428
S	-0.2925	PP	-2.3791
S	-1.0061	PP	-3.2629
S	-2.1519	PP	-4.1105
T	-1.4771	QQ	-0.8015
T	-2.5685	QQ	-1.0069
T	-3.3814	QQ	-1.0205
U	-0.5688	RR	-0.3064
U	-1.7965	RR	-0.5154
U	-2.7945	RR	-0.6417
V	-0.9414	SS	-0.2256
V	-1.0496	SS	-0.8775
V	-1.2709	SS	-1.1070
W	-0.3782	-	-
W	-0.1347	-	-
W	0.0663	-	-

3 PVN's/Crude Samples
Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	48	-42.5396	-0.8862	0.4930
Row 2	48	-55.9769	-1.1662	0.8716
Row 3	48	-70.6171	-1.4712	1.3209
Column 1	3	-2.4896	-0.8299	0.0010
Column 2	3	-1.3471	-0.4490	0.1843
Column 3	3	-6.2874	-2.0958	1.0241
Column 4	3	-2.3833	-0.7944	0.6892
Column 5	3	-2.2679	-0.7560	0.0056
Column 6	3	-1.1913	-0.3971	0.0144
Column 7	3	-2.5560	-0.8520	0.0114
Column 8	3	-6.3809	-2.1270	1.1342
Column 9	3	-1.5421	-0.5140	0.0227
Column 10	3	-2.1129	-0.7043	0.0793
Column 11	3	-11.5601	-3.8534	0.6897
Column 12	3	-6.4496	-2.1499	0.0126
Column 13	3	-7.0999	-2.3666	0.9872
Column 14	3	-3.1021	-1.0340	0.7096
Column 15	3	-7.0601	-2.3534	0.8774
Column 16	3	-0.5874	-0.1958	0.0047
Column 17	3	-7.0566	-2.3522	0.0017
Column 18	3	-3.4505	-1.1502	0.8798
Column 19	3	-7.4270	-2.4757	0.9131
Column 20	3	-5.1599	-1.7200	1.2429
Column 21	3	-3.2619	-1.0873	0.0282
Column 22	3	-0.4466	-0.1489	0.0495
Column 23	3	-1.1069	-0.3690	0.0296
Column 24	3	-2.8801	-0.9600	0.0172
Column 25	3	-2.8801	-0.9600	0.0172
Column 26	3	-1.4042	-0.4681	0.1010
Column 27	3	-2.8659	-0.9553	0.1570
Column 28	3	-0.8186	-0.2729	0.0395
Column 29	3	-2.7055	-0.9018	0.0024
Column 30	3	-2.7055	-0.9018	0.0024
Column 31	3	-3.0244	-1.0081	0.3291
Column 32	3	-2.7119	-0.9040	0.0384
Column 33	3	-2.7119	-0.9040	0.0384
Column 34	3	-1.7290	-0.5763	0.0031
Column 35	3	-8.1765	-2.7255	0.9055
Column 36	3	-2.4603	-0.8201	0.0050
Column 37	3	-7.1184	-2.3728	0.0196
Column 38	3	0.0849	0.0283	0.1658
Column 39	3	0.0849	0.0283	0.1658
Column 40	3	-4.0696	-1.3565	1.1056
Column 41	3	-2.4056	-0.8019	0.0379
Column 42	3	-2.7424	-0.9141	0.0145
Column 43	3	-2.5325	-0.8442	0.0003
Column 44	3	-4.7791	-1.5930	0.0067
Column 45	3	-9.7525	-3.2508	0.7496
Column 46	3	-2.8258	-0.9429	0.0151
Column 47	3	-1.4635	-0.4878	0.0287
Column 48	3	-2.2100	-0.7367	0.2091

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	8.2170	2	4.1085	19.9922	5.83E-08	3.0933
Columns	106.9026	47	2.2745	11.0680	1.3E-22	1.4950
Error	19.3175	94	0.2055			
Total	134.4371	143				

Table A.9 ANOVA CRUDE SOURCE/PVN Comparison (Page 2 of 4)

ANOVA PVN Comparison with 4 PVN's/Crude Source

CRUDE SOURCE	PVN
A4	-2.3502
A4	-1.9175
A4	-2.0189
A4	-2.0876
B4	-0.6663
B4	-0.6602
B4	-1.7019
B4	-0.6629

ANOVA PVN Comparison with 6 PVN's/Crude Source

CRUDE SOURCE	PVN	CRUDE SOURCE	PVN
A6	-0.3386	D6	-0.5094
A6	-0.4603	D6	-3.1502
A6	-0.5969	D6	0.1868
A6	-0.5988	D6	-0.0052
A6	-0.4254	D6	-0.0963
A6	-0.2063	D6	-0.5790
B6	0.6916	E6	0.0039
B6	0.6079	E6	-0.2929
B6	0.1647	E6	-0.3326
B6	-0.2693	E6	0.0897
B6	-0.3692	E6	-0.4348
B6	0.5938	E6	-0.4436
C6	-0.5651	F6	-0.8390
C6	-0.5261	F6	-0.8401
C6	-0.2880	F6	-0.6983
C6	-0.4254	F6	-0.6599
C6	-0.8548	F6	-0.9297
C6	-0.5637	F6	-1.7236

ANOVA PVN Comparison with 13 PVN's/Crude Source

CRUDE SOURCE	PVN	CRUDE SOURCE	PVN
A13	-0.4692	B13	-0.2572
A13	-0.3811	B13	-0.3024
A13	-0.3811	B13	-0.3024
A13	-0.2948	B13	-0.3190
A13	-0.1764	B13	2.1883
A13	-0.4706	B13	-0.0144
A13	-0.4327	C13	-1.0097
A13	-0.5260	C13	-1.0257
A13	-0.5448	C13	-0.9226
A13	-0.1702	C13	-0.9074
A13	-0.4966	C13	-0.9250
A13	-0.4362	C13	-0.8935
A13	-0.5284	C13	-0.8907
B13	-0.3510	C13	-0.9086
B13	-0.3193	C13	-0.8640
B13	-0.2686	C13	-0.8992
B13	-0.1532	C13	-0.9258
B13	0.0209	C13	-0.8793
B13	-0.1329	C13	-0.9356
B13	-0.3245	-	-

4 PVN's/Crude Samples

Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	4	-8.374201	-2.09355	0.034145
Row 2	4	-3.69139	-0.922847	0.269783
Column 1	2	-3.016489	-1.508245	1.417616
Column 2	2	-2.577744	-1.288872	0.790406
Column 3	2	-3.720857	-1.860429	0.050232
Column 4	2	-2.7505	-1.37525	1.014931

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	2.74109	1	2.74109	15.4545	0.029306	10.12796
Columns	0.379691	3	0.126564	0.713576	0.605913	9.276619
Error	0.532095	3	0.177365			
Total	3.652877	7				

6 PVN's/Crude Samples

Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	6	-2.626391	-0.437732	0.023068
Row 2	6	1.419612	0.236602	0.220113
Row 3	6	-3.22312	-0.537187	0.035411
Row 4	6	-4.153272	-0.692212	1.537522
Row 5	6	-1.41039	-0.235065	0.051766
Row 6	6	-5.690687	-0.948448	0.154168
Column 1	6	-1.556736	-0.259456	0.294393
Column 2	6	-4.661659	-0.776943	1.59032
Column 3	6	-1.564288	-0.260715	0.138331
Column 4	6	-1.868827	-0.311471	0.094699
Column 5	6	-3.110306	-0.518384	0.099689
Column 6	6	-2.922432	-0.487072	0.558479

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	4.987542	5	0.997508	2.804506	0.038228	2.602988
Columns	1.218224	5	0.243645	0.68501	0.639154	2.602988
Error	8.892014	25	0.355681			
Total	15.09778	35				

13 PVN's/Crude Samples

Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	13	-5.3081	-0.4083	0.0157
Row 2	13	-0.5358	-0.0412	0.4635
Row 3	13	-11.9870	-0.9221	0.0022
Column 1	3	-1.8299	-0.6100	0.1233
Column 2	3	-1.7262	-0.5754	0.1530
Column 3	3	-1.5723	-0.5241	0.1222
Column 4	3	-1.3554	-0.4518	0.1607
Column 5	3	-1.0805	-0.3602	0.2490
Column 6	3	-1.4969	-0.4990	0.1452
Column 7	3	-1.6480	-0.5493	0.0904
Column 8	3	-1.6917	-0.5639	0.1071
Column 9	3	-1.7112	-0.5704	0.0793
Column 10	3	-1.3717	-0.4572	0.1508
Column 11	3	-1.7414	-0.5805	0.0973
Column 12	3	0.8729	0.2910	2.7491
Column 13	3	-1.4785	-0.4928	0.2131

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	5.0901	2	2.5450	16.1102	3.66E-05	3.4028
Columns	1.9856	12	0.1655	1.0474	0.4413	2.1834
Error	3.7914	24	0.1580			
Total	10.8671	38				

Table A.9 ANOVA CRUDE SOURCE/PVN Comparison (Page 3 of 4)
ANOVA PVN Comparison with 23 PVN's/Crude Source

CRUDE SOURCE	PVN	CRUDE SOURCE	PVN
A23	-0.5790	B23	-0.6680
A23	-0.6770	B23	-0.4167
A23	-0.7367	B23	-0.2054
A23	-0.2483	B23	-0.6618
A23	-0.2483	B23	0.6547
A23	-0.7216	B23	-0.6402
A23	-0.1890	B23	-0.5373
A23	-0.7481	B23	-0.5021
A23	-0.7074	B23	-0.4610
A23	-0.7498	B23	0.8203
A23	-0.7277	B23	-0.3099
A23	-0.6171	C23	-0.3164
A23	-0.5567	C23	-0.4313
A23	-0.6860	C23	-0.3582
A23	-0.7176	C23	-0.3624
A23	-0.7069	C23	-0.3248
A23	-0.7657	C23	-0.3129
A23	-0.6554	C23	-0.2823
A23	-0.6680	C23	-0.1208
A23	1.0185	C23	-0.1926
A23	-0.5790	C23	-0.2086
A23	1.0882	C23	-0.1001
A23	-0.7558	C23	-0.1772
B23	-0.5628	C23	-0.0969
B23	-0.5885	C23	-0.5018
B23	-0.5073	C23	-0.2260
B23	-0.5812	C23	-0.5020
B23	-0.4481	C23	-0.3362
B23	-0.4240	C23	-0.3208
B23	-0.4840	C23	-0.1857
B23	-0.3381	C23	-0.3062
B23	-0.6329	C23	-0.3163
B23	-1.2527	C23	-0.3129
B23	-0.6456	C23	-0.2031
B23	-0.6919	-	-

23 PVN's/Crude Samples
Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	23	-10.9342	-0.4754	0.2607
Row 2	23	-10.0846	-0.4385	0.1769
Row 3	23	-6.4956	-0.2824	0.0127
Column 1	3	-1.4581	-0.4860	0.0217
Column 2	3	-1.6968	-0.5656	0.0155
Column 3	3	-1.6022	-0.5341	0.0364
Column 4	3	-1.1919	-0.3973	0.0286
Column 5	3	-1.0212	-0.3404	0.0102
Column 6	3	-1.4585	-0.4862	0.0447
Column 7	3	-0.9553	-0.3184	0.0227
Column 8	3	-1.2071	-0.4024	0.1015
Column 9	3	-1.5328	-0.5109	0.0774
Column 10	3	-2.2111	-0.7370	0.2726
Column 11	3	-1.4734	-0.4911	0.1163
Column 12	3	-1.4863	-0.4954	0.0773
Column 13	3	-1.3216	-0.4405	0.0917
Column 14	3	-1.6044	-0.5348	0.0189
Column 15	3	-1.1490	-0.3830	0.0841
Column 16	3	-1.8707	-0.6236	0.0116
Column 17	3	-0.4473	-0.1491	0.5306
Column 18	3	-1.6164	-0.5388	0.0357
Column 19	3	-1.3911	-0.4637	0.0622
Column 20	3	0.2102	0.0701	0.6842
Column 21	3	-1.3563	-0.4521	0.0173
Column 22	3	1.5955	0.5318	0.5532
Column 23	3	-1.2688	-0.4229	0.0859

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	0.4827	2	0.2413	1.9244	0.1580	3.2093
Columns	4.3878	22	0.1994	1.5904	0.0942	1.7889
Error	5.5180	44	0.1254			
Total	10.3885	68				

Table A.9 ANOVA CRUDE SOURCE/PVN Comparison (Page 4 of 4)
ANOVA PVN Comparison with 32 PVN's/Crude Source

CRUDE SOURCE	PVN	CRUDE SOURCE	PVN
A32	-0.5628	B32	-0.3164
A32	-0.5885	B32	-0.4313
A32	-0.5073	B32	-0.3582
A32	-0.5812	B32	-0.3624
A32	-0.4481	B32	-0.3248
A32	-0.4240	B32	-0.3129
A32	-0.4840	B32	-0.2823
A32	-0.3381	B32	-0.1208
A32	-0.6329	B32	-0.1926
A32	-1.2527	B32	-0.2086
A32	-0.6456	B32	-0.1001
A32	-0.6919	B32	-0.1772
A32	-0.6680	B32	-0.0969
A32	-0.4167	B32	-0.5018
A32	-0.2054	B32	-0.2260
A32	-0.6618	B32	-0.5020
A32	0.6547	B32	-0.3362
A32	-0.6402	B32	-0.3208
A32	-0.5373	B32	-0.1857
A32	-0.5021	B32	-0.3062
A32	-0.4610	B32	-0.3163
A32	0.8203	B32	-0.3129
A32	-0.3099	B32	-0.2031
A32	-0.4812	B32	-0.4313
A32	-0.3576	B32	-0.1772
A32	-0.5407	B32	-0.1208
A32	-0.5113	B32	0.2940
A32	1.2581	B32	-0.4734
A32	0.7414	B32	1.0781
A32	1.2980	B32	1.0371
A32	0.5462	B32	-0.4381
A32	0.8020	B32	-0.3602

32 PVN's/Crude Samples
Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	32	-7.3296	-0.2290	0.3970
Row 2	32	-6.0875	-0.1902	0.1309
Column 1	2	-0.8792	-0.4396	0.0304
Column 2	2	-1.0198	-0.5099	0.0124
Column 3	2	-0.8655	-0.4328	0.0111
Column 4	2	-0.9436	-0.4718	0.0239
Column 5	2	-0.7729	-0.3864	0.0076
Column 6	2	-0.7369	-0.3684	0.0062
Column 7	2	-0.7663	-0.3832	0.0203
Column 8	2	-0.4589	-0.2295	0.0236
Column 9	2	-0.8254	-0.4127	0.0969
Column 10	2	-1.4612	-0.7306	0.5450
Column 11	2	-0.7457	-0.3729	0.1488
Column 12	2	-0.8691	-0.4346	0.1324
Column 13	2	-0.7649	-0.3825	0.1631
Column 14	2	-0.9185	-0.4592	0.0036
Column 15	2	-0.4314	-0.2157	0.0002
Column 16	2	-1.1638	-0.5819	0.0128
Column 17	2	0.3185	0.1592	0.4909
Column 18	2	-0.9610	-0.4805	0.0510
Column 19	2	-0.7230	-0.3615	0.0618
Column 20	2	-0.8083	-0.4042	0.0192
Column 21	2	-0.7773	-0.3886	0.0105
Column 22	2	0.5074	0.2537	0.6421
Column 23	2	-0.5130	-0.2565	0.0057
Column 24	2	-0.9125	-0.4562	0.0012
Column 25	2	-0.5348	-0.2674	0.0163
Column 26	2	-0.6615	-0.3307	0.0881
Column 27	2	-0.2173	-0.1086	0.3242
Column 28	2	0.7846	0.3923	1.4991
Column 29	2	1.8194	0.9097	0.0567
Column 30	2	2.3352	1.1676	0.0340
Column 31	2	0.1081	0.0540	0.4844
Column 32	2	0.4418	0.2209	0.6753

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	0.0241	1	0.0241	0.1317	0.7192	4.1596
Columns	10.6889	31	0.3448	1.8835	0.0414	1.8221
Error	5.6749	31	0.1831			
Total	16.3879	63				

Table A.10 Power Calculations for ANOVA, Appendix A.9

A = [-1 1 -2 2 0 0]

6 Crude Sources

a	Sum of Squares	K	egrees o Freedom	Variance	umber o Columns	Phi	Tables
0.000	10	0.0000	5	0.35568	6	0.0000	
0.100	10	0.1000	5	0.35568	6	0.5302	
0.200	10	0.4000	5	0.35568	6	1.0605	0.22
0.250	10	0.6250	5	0.35568	6	1.3256	0.32
0.270	10	0.7290	5	0.35568	6	1.4316	0.39
0.290	10	0.8410	5	0.35568	6	1.5377	0.44
0.310	10	0.9610	5	0.35568	6	1.6437	0.53
0.330	10	1.0890	5	0.35568	6	1.7498	0.55
0.350	10	1.2250	5	0.35568	6	1.8558	0.62
0.370	10	1.3690	5	0.35568	6	1.9619	0.65
0.390	10	1.5210	5	0.35568	6	2.0679	0.72
0.410	10	1.6810	5	0.35568	6	2.1740	0.76
0.430	10	1.8490	5	0.35568	6	2.2800	0.817
0.460	10	2.1160	5	0.35568	6	2.4391	0.87
0.490	10	2.4010	5	0.35568	6	2.5982	0.91
0.520	10	2.7040	5	0.35568	6	2.7572	0.93
0.550	10	3.0250	5	0.35568	6	2.9163	0.955
0.580	10	3.3640	5	0.35568	6	3.0754	0.972
0.610	10	3.7210	5	0.35568	6	3.2344	0.982
0.640	10	4.0960	5	0.35568	6	3.3935	0.989
0.670	10	4.4890	5	0.35568	6	3.5526	

Table A.11 ANOVA CRUDE SOURCE/PG/PVN Comparison (Page 1 of 1)
ANOVA PVN Comparison with 3 PVN's/Crude Source

CRUDE SOURCE	PG	PVN
A	46-34	-0.1890
A	46-34	-0.2483
A	46-34	-0.2483
A	52-28	-0.6860
A	52-28	-0.6171
A	52-28	-0.7481
A	58-22	-0.7558
A	58-22	-0.6680
A	58-22	-0.7176
A	58-22	-0.7074
A	58-28	-0.5790
A	58-28	-0.6554
A	58-28	-0.7657
A	58-28	-0.7069
A	58-28	-0.7277
A	58-28	-0.7498
A	58-28	-0.7216
A	58-28	-0.5790
A	64-22	-0.7367
A	64-22	-0.6770
A	64-28	1.0882
A	64-28	1.0185
A	70-28	-0.5567
B	46-34	-0.3099
B	46-34	-0.5021
B	46-34	-0.4167
B	52-28	-0.3576
B	52-28	-0.4812
B	52-28	-0.2054
B	58-18	-0.6329
B	58-22	-0.5407
B	58-22	-0.4610
B	58-22	-0.6402
B	58-22	-0.6618
B	58-22	-0.6680
B	58-22	-0.6919
B	58-22	-0.6456
B	58-22	-1.2527
B	58-28	-0.5373
B	64-22	0.8020
B	64-22	-0.5113
B	64-28	0.7414
B	64-28	1.2581
B	70-22	1.2980
B	70-22	0.5462
B	70-22	0.6547
B	70-34	0.8203
C	46-34	-0.3099
C	46-34	-0.5021
C	46-34	-0.4167
C	52-28	-0.3576
C	52-28	-0.4812
C	52-28	-0.2054
C	58-22	-0.5407
C	58-22	-0.4610
C	58-22	-0.6402
C	70-22	1.2980
C	70-22	0.5462
C	70-22	0.6547
C	64-22	0.8020
C	64-22	-0.5113
C	64-28	0.7414
C	64-28	1.2581

Summary of ANOVA for Crude Source A
Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	3	-0.6855	-0.2285	0.0012
Row 2	3	-2.0512	-0.6837	0.0043
Row 3	3	-2.1414	-0.7138	0.0019
Row 4	3	-2.0002	-0.6667	0.0088
Column 1	4	-2.2098	-0.5525	0.0640
Column 2	4	-2.1888	-0.5472	0.0402
Column 3	4	-2.4797	-0.6199	0.0618

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	0.4786	3	0.1595	49.6921	0.0001	4.7571
Columns	0.0132	2	0.0066	2.0489	0.2098	5.1432
Error	0.0193	6	0.0032			
Total	0.5110	11				

Summary of ANOVA for Crude Source B
Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	3	-1.2287	-0.4096	0.0093
Row 2	3	-1.0441	-0.3480	0.0191
Row 3	3	-1.6419	-0.5473	0.0081
Row 4	3	2.4989	0.8330	0.1652
Column 1	4	0.0899	0.0225	0.7330
Column 2	4	-0.8981	-0.2245	0.2643
Column 3	4	-0.6076	-0.1519	0.3207

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	3.6797	3	1.2266	26.8346	0.0007	4.7571
Columns	0.1289	2	0.0645	1.4103	0.3147	5.1432
Error	0.2742	6	0.0457			
Total	4.0829	11				

Summary of ANOVA for Crude Source C
Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	3	-0.5660	-0.1887	0.0002
Row 2	3	-0.6193	-0.2064	0.0087
Row 3	3	-1.1061	-0.3687	0.0010
Row 4	3	-1.4078	-0.4693	0.0012
Column 1	4	-1.1443	-0.2861	0.0151
Column 2	4	-1.2011	-0.3003	0.0274
Column 3	4	-1.3538	-0.3385	0.0172

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	0.1627	3	0.0542	19.8759	0.0016	4.7571
Columns	0.0059	2	0.0029	1.0759	0.3987	5.1432
Error	0.0164	6	0.0027			
Total	0.1850	11				

Table A.12 ANOVA For Two Modified PVN Values (Page 1 of 2)

Original Comparison

Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	2	-1.6317	-0.8158	0.0017
Row 2	2	0.2986	0.1493	0.0183
Row 3	2	-0.1982	-0.0991	0.0370
Row 4	2	0.4369	0.2185	0.0315
Row 5	2	1.0834	0.5417	0.0012
Row 6	2	0.6646	0.3323	0.0007
Row 7	2	-0.3906	-0.1953	0.0013
Column 1	7	0.2588	0.0370	0.1910
Column 2	7	0.0044	0.0006	0.2184

ID	Original	
	PVN 135	PVN 60
STM 1	-0.7864	-0.8452
STM 2	0.0537	0.2449
STM 3	0.0369	-0.2351
STM 4	0.3439	0.0930
STM 5	0.5174	0.5661
STM 6	0.3139	0.3507
STM 7	-0.2205	-0.1701

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	2.3700	6	0.3950	27.2475	0.0004	4.2839
Columns	0.0046	1	0.0046	0.3191	0.5927	5.9874
Error	0.0870	6	0.0145			
Total	2.4616	13				

PVN Using 135 PVN

Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	2	1.7813	0.8907	0.2787
Row 2	2	1.1435	0.5718	0.1330
Row 3	2	-0.1380	-0.0690	0.0459
Column 1	3	0.6108	0.2036	0.1452
Column 2	3	2.1760	0.7253	0.3571

ID	Original	Aged
	PVN 135	PVN 135
STM 5	0.5174	1.2639
STM 6	0.3139	0.8296
STM 7	-0.2205	0.0825

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	0.9555	2	0.4778	19.4138	0.0490	19.0000
Columns	0.4083	1	0.4083	16.5931	0.0553	18.5128
Error	0.0492	2	0.0246			
Total	1.4131	3				

Original @ 135, Aged @ 60

Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	2	-1.2296	-0.6148	0.0589
Row 2	2	0.7626	0.3813	0.2147
Row 3	2	1.5312	0.7656	0.1232
Row 4	2	0.9310	0.4655	0.0460
Row 5	2	-0.2966	-0.1483	0.0104

ID	Original	Aged
	PVN 135	PVN 60
STM 1	-0.7864	-0.4432
STM 2	0.0537	0.7089
STM 5	0.5174	1.0139
STM 6	0.3139	0.6171
STM 7	-0.2205	-0.0761

Table A.12 ANOVA For Two Modified PVN Values (Page 2 of 2)

Column 1	5	-0.1220	-0.0244	0.2583
Column 2	5	1.8206	0.3641	0.3627

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	2.4080	4	0.6020	31.7417	0.0027	6.3882
Columns	0.3774	1	0.3774	19.8971	0.0112	7.7086
Error	0.0759	4	0.0190			
Total	2.8612	9				

Original @ 60 and Aged @ 135

Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	2	1.8300	0.9150	0.2435
Row 2	2	1.1804	0.5902	0.1147
Row 3	2	-0.0877	-0.0438	0.0319
Column 1	3	0.7467	0.2489	0.1433
Column 2	3	2.1760	0.7253	0.3571

ID	Original	Aged
	PVN	PVN
	60	135
STM 5	0.5661	1.2639
STM 6	0.3507	0.8296
STM 7	-0.1701	0.0825

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	0.9512	2	0.4756	19.1861	0.0495	19.0000
Columns	0.3405	1	0.3405	13.7365	0.0657	18.5128
Error	0.0496	2	0.0248			
Total	1.3413	5				

Original @ 60 and Aged @ 60

Anova: Two-Factor Without Replication

ID	Original	Aged
	PVN	PVN
	60	60
STM 1	-0.8452	-0.4432
STM 2	0.2449	0.7089
STM 5	0.5661	1.0139
STM 6	0.3507	0.6171
STM 7	-0.1701	-0.0761

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	2	-1.2884	-0.6442	0.0808
Row 2	2	0.9539	0.4769	0.1076
Row 3	2	1.5799	0.7900	0.1003
Row 4	2	0.9678	0.4839	0.0355
Row 5	2	-0.2462	-0.1231	0.0044
Column 1	5	0.1464	0.0293	0.3106
Column 2	5	1.8206	0.3641	0.3627

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	2.6448	4	0.6612	54.7376	0.0010	6.3882
Columns	0.2803	1	0.2803	23.2040	0.0085	7.7086
Error	0.0483	4	0.0121			
Total	2.9734	9				

Table A.13 Four and Three Way ANOVA for Modified PVN (Page 1 of 1)

Four Way ANOVA
Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	4	3.3612	0.8403	0.1297
Row 2	4	2.1114	0.5278	0.0587
Row 3	4	-0.3842	-0.0961	0.0177
Column 1	3	0.6108	0.2036	0.1452
Column 2	3	0.7467	0.2489	0.1433
Column 3	3	2.1760	0.7253	0.3571
Column 4	3	1.5549	0.5183	0.3043

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	1.8182	2	0.9091	66.7843	7.94E-05	5.1432
Columns	0.5368	3	0.1789	13.1452	0.0048	4.7571
Error	0.0817	6	0.0136			
Total	2.4367	11				

Three Way Analysis (Original @ 135 and 60 and Aged @ 135)
Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	3	2.3474	0.7825	0.1745
Row 2	3	1.4943	0.4981	0.0828
Row 3	3	-0.3082	-0.1027	0.0264
Column 1	3	0.6108	0.2036	0.1452
Column 2	3	0.7467	0.2489	0.1433
Column 3	3	2.1760	0.7253	0.3571

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	1.2254	2	0.6127	37.1884	0.0026	6.9443
Columns	0.5013	2	0.2506	15.2136	0.0135	6.9443
Error	0.0659	4	0.0165			
Total	1.7925	8				

Three Way Analysis (Original @ 60 and Aged @ 135 and 60)
Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	3	2.8438	0.9479	0.1250
Row 2	3	1.7975	0.5992	0.0576
Row 3	3	-0.1637	-0.0546	0.0163
Column 1	3	0.7467	0.2489	0.1433
Column 2	3	2.1760	0.7253	0.3571
Column 3	3	1.5549	0.5183	0.3043

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	1.5541	2	0.7770	56.1772	0.0012	6.9443
Columns	0.3425	2	0.1712	12.3790	0.0193	6.9443
Error	0.0553	4	0.0138			
Total	1.9519	8				

APPENDIX B

MODULE TWO: LOW TEMPERATURE CRACKING

Table B.1 Material Data C-LTPP Database (Page1 of 5)

C-SHRP ID	Section	Pavement original	Layer Location	SpecificGrav Asphalt Cement	Penetration @25C(mm X 1/10)	KinViscosity @135C(mm2/sec)	PVN
810404	1	overlay	Surface	1.0660	95	417	-0.2179
810404	2	overlay	Surface	1.0660	94	400	-0.2936
810404	2	overlay	1st Intermediate	1.0660	111	369	-0.2218
810404	3	overlay	Surface	1.0720	61	627	-0.1239
810404	3	overlay	1st Intermediate	1.0700	81	318	-0.8002
810404	4	overlay	Surface	1.0760	71	560	-0.1158
810404	4	original	Surface	1.0470	78	436	-0.3754
810404	4	original	1st Intermediate	1.0470	78	422	-0.4233
810404	4	original	2nd Intermediate	1.0710	51	780	-0.0170
820205	1	original	Surface	1.0090	51	268	-1.4734
820205	1	overlay	Surface	1.0250	72	259	-1.2162
820205	2	original	Surface	1.0210	50	283	-1.4159
820205	2	overlay	Bottom	1.0230	70	267	-1.1986
820205	2	overlay	Surface	1.0250	72	289	-1.0576
820502	1	N/A	1st Intermediate	1.0340	25	877	-0.5637
820502	1	N/A	1st Intermediate	1.0240	28	623	-0.8851
820502	1	original	Surface	1.0220	33	336	-1.5258
820502	1	overlay	Bottom	1.0260	120	153	-1.5291
820502	1	overlay	Surface	1.0180	80	277	-1.0167
820502	2	original	Surface	1.0260	32	383	-1.3840
820502	2	overlay	Bottom	1.0300	38	345	-1.3801
820502	2	overlay	Surface	1.0160	88	305	-0.7761
820605	1	N/A	1st Intermediate	1.0280	28	437	-1.3241
820605	1	N/A	1st Intermediate	1.0340	28	496	-1.1673

Table B.1 Material Data C-LTPP Database (Page2 of 5)

C-SHRP ID	Section	Pavement original	Layer Location	SpecificGrav Asphalt Cement	Penetration @25C(mm X 1/10)	KinViscosity @135C(mm2/sec)	PVN
820605	1	original	Surface	1.0270	46	249	-1.6569
820605	1	overlay	Surface	1.0190	64	293	-1.1493
820605	2	original	Surface	1.0280	39	313	-1.4859
820605	2	overlay	Surface	1.0190	70	298	-1.0404
830403	3	original	Surface	1.1010	21	1170	-0.3722
830403	1	overlay	Surface	1.0610	61	535	-0.3470
830403	2	overlay	Bottom	1.0640	74	463	-0.3454
830801	1	overlay	Surface	1.0420	117	321	-0.3786
830801	2	overlay	Surface	1.0400	108	334	-0.4103
830801	3	overlay	Surface	1.0430	88	318	-0.7134
830801	4	overlay	Surface	1.0390	124	311	-0.3594
840101	3	overlay	Bottom	1.0340	76	264	-1.1372
840101	3	overlay	Bottom	1.0360	92	285	-0.8316
840101	3	overlay	Bottom	1.0330	70	304	-1.0117
840101	1	overlay	Bottom	1.0450	80	283	-0.9851
840101	1	overlay	Bottom	1.0400	98	304	-0.6656
840101	1	overlay	Bottom	1.0390	86	276	-0.9495
840101	1	overlay	2nd Intermediate	1.0450	85	370	-0.5241
840101	1	overlay	2nd Intermediate	1.0400	97	270	-0.8581
840101	1	overlay	2nd Intermediate	1.0380	83	307	-0.8272
840101	1	overlay	1st Intermediate	1.0380	71	314	-0.9512
840101	1	overlay	1st Intermediate	1.0350	65	292	-1.1397
840101	1	overlay	1st Intermediate	1.0370	68	289	-1.1121
840101	1	overlay	Surface	1.0340	75	252	-1.2177
840101	1	overlay	Surface	1.0330	91	283	-0.8537

Table B.1 Material Data C-LTPP Database (Page3 of 5)

C-SHRP ID	Section	Pavement original	Layer Location	SpecificGrav Asphalt Cement	Penetration @25C(mm X 1/10)	KinViscosity @135C(mm2/sec)	PVN
840101	1	overlay	Surface	1.0330	91	283	-0.8537
840101	1	original	Surface	1.0430	44	581	-0.5648
840204	2	overlay	Bottom	1.0290	76	326	-0.8289
840204	2	overlay	Bottom	1.0280	82	280	-0.9762
840204	2	overlay	Bottom	1.0250	64	301	-1.1111
840204	2	overlay	1st Intermediate	1.0370	45	391	-1.0718
840204	2	overlay	1st Intermediate	1.0420	68	446	-0.4904
840204	2	overlay	1st Intermediate	1.0370	38	488	-0.9298
840204	1	overlay	Bottom	1.0290	72	289	-1.0576
840204	1	overlay	Bottom	1.0290	95	283	-0.8084
840204	1	overlay	Bottom	1.0310	81	283	-0.9727
840204	1	overlay	Surface	1.0450	40	528	-0.7808
840204	1	overlay	Surface	1.0440	38	568	-0.7326
840204	1	overlay	Surface	1.0460	35	599	-0.7389
840204	1	overlay	Surface	1.0460	35	599	-0.7389
840204	1	overlay	Surface	1.0460	35	599	-0.7389
840204	1	original	Surface	1.0470	26	904	-0.4917
840604	1	overlay	Bottom	1.0430	86	360	-0.5523
840604	1	overlay	Bottom	1.0430	92	314	-0.6849
840604	1	overlay	Bottom	1.0430	72	351	-0.7762
840604	1	overlay	Surface	1.0450	34	643	-0.6745
840604	1	overlay	Surface	1.0490	30	671	-0.7329
840604	1	overlay	Surface	1.0490	30	671	-0.7329
840604	2	overlay	Surface	1.0320	74	320	-0.8828
840604	2	overlay	Surface	1.0330	74	332	-0.8293

Table B.1 Material Data C-LTPP Database (Page4 of 5)

C-SHRP ID	Section	Pavement original	Layer Location	SpecificGrav Asphalt Cement	Penetration @25C(mm X 1/10)	KinViscosity @135C(mm2/sec)	PVN
840604	2	overlay	Surface	1.0330	62	317	-1.0676
840604	4	overlay	Bottom	1.0450	43	503	-0.7780
840604	4	overlay	Bottom	1.0480	38	497	-0.9060
840604	4	overlay	Bottom	1.0500	33	686	-0.6194
840604	4	original	Surface	1.0460	44	626	-0.4656
850206	1	overlay			175	230	-0.4327
860501	1	overlay	Surface	1.0128	165	206	-0.6978
860501	2	overlay	Surface	1.0128	165	206	-0.6978
860501	3	overlay	Surface	1.0128	165	206	-0.6978
860603	1	overlay	Surface	1.0128	165	206	-0.6978
860603	1	overlay	Bottom	1.0128	167	206	-0.6832
860603	2	overlay	Surface	1.0128	167	206	-0.6832
860603	2	overlay	Bottom	1.0128	167	206	-0.6832
860603	3	overlay	Surface	1.0128	167	206	-0.6832
870102	1	overlay	Surface	1.0276	81	415	-0.4065
870102	2	overlay	1st Intermediate	1.0250	79	427	-0.3920
870504	1	overlay	Surface	1.0350	33	691	-0.6100
870504	2	overlay	Surface	1.0390	28	726	-0.6960
870504	2	overlay	Bottom	1.0370	33	707	-0.5807
870504	1	original	Bottom	1.0350	33	691	-0.6100
870504	2	original	Bottom	1.0390	28	726	-0.6960
870504	2	original	2nd intermeditat	1.0370	33	707	-0.5807
870505	1	overlay	Surface	1.0343	54	1200	0.6385
870505	3	overlay	Surface	1.0307	64	465	-0.4945
870505	3	overlay	1st Intermediate	1.0284	84	513	-0.0505

Table B.1 Material Data C-LTPP Database (Page5 of 5)

C-SHRP ID	Section	Pavement original	Layer Location	SpecificGrav Asphalt Cement	Penetration @25C(mm X 1/10)	KinViscosity @135C(mm2/sec)	PVN
870505	4	overlay	Bottom	1.0314	45	451	-0.8812
870505	4	overlay	Bottom	1.0249	71	406	-0.5802
870701	2	overlay	Bottom	1.0238	65	475	-0.4481
870701	2	overlay	Surface	1.0297	57	477	-0.5775
880203	1	original	Surface	1.0340	88	341	-0.6090
880203	1	original	1st Intermediate	1.0450	137	277	-0.4244
880203	1	original	Bottom	1.0420	127	276	-0.5223
880203	1	overlay	Surface	1.0230	83	256	-1.1000
880203	2	original	Surface	1.0420	88	341	-0.6090
880203	2	original	1st Intermediate	1.0450	137	277	-0.4244
880203	2	original	Bottom	1.0420	127	276	-0.5223
880203	3	original	Surface	1.0340	88	341	-0.6090
880203	3	original	1st Intermediate	1.0450	137	277	-0.4244
880203	3	original	Bottom	1.0420	127	276	-0.5223
880203	4	original	Surface	1.0340	88	341	-0.6090
880203	4	original	1st Intermediate	1.0450	137	277	-0.4244
880203	4	original	Bottom	1.0420	127	276	-0.5223
890503	1	overlay	Surface		99	369	-0.3569
890503	1	overlay	Surface	1.0400	25	924	-0.4998
890503	2	overlay	Surface	1.0400	27	2046	0.5482
890503	3	overlay	Surface	1.0400	25	924	-0.4998
890503	4	overlay	Surface	1.0400	27	2046	0.5482
890702	1	overlay	Surface	1.0400	30	843	-0.4469
890702	2		Surface	1.0400	30	843	-0.4469

Table B.2 Transverse Cracking Prediction Using Hajek Model (Page 1 of 2)

Test Site	Section	Thickness (mm)	Stiffness (kg/cm^2)	Minimum Temperature (°C)	Subgrade	Cracking Index With Time (Years)[1]																			
						1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
810404	1	61	40	-30	1	0.4	2.6	4.7	6.9	9.0	11.2	13.3	15.4	17.6	19.7	21.9	24.0	26.2	28.3	30.5	32.6	34.7	36.9	39.0	41.2
810404	2	103	18.5	-30	1	-3.4	-1.7	0.0	1.7	3.4	5.1	6.8	8.5	10.1	11.8	13.5	15.2	16.9	18.6	20.3	22.0	23.7	25.4	27.1	28.8
810404	3	94	50	-30	1	-0.9	1.4	3.6	5.9	8.2	10.5	12.7	15.0	17.3	19.6	21.8	24.1	26.4	28.7	30.9	33.2	35.5	37.8	40.0	42.3
810404	4	55	35	-30	1	0.4	2.5	4.5	6.6	8.7	10.7	12.8	14.9	16.9	19.0	21.1	23.1	25.2	27.3	29.3	31.4	33.5	35.5	37.6	39.7
820205	1	42	45	-15	3	10.2	11.1	11.9	12.8	13.7	14.6	15.4	16.3	17.2	18.1	18.9	19.8	20.7	21.6	22.4	23.3	24.2	25.1	25.9	26.8
820205	2	83	25	-15	3	8.2	8.7	9.2	9.7	10.3	10.8	11.4	11.9	12.4	13.0	13.5	14.0	14.6	15.1	15.6	16.2	16.7	17.2	17.8	18.3
820502	1	104	17	-15	3	7.1	7.5	8.0	8.4	8.8	9.2	9.6	10.0	10.4	10.8	11.2	11.6	12.0	12.4	12.8	13.2	13.6	14.0	14.4	14.8
820502	2	118	17	-15	3	6.6	6.9	7.2	7.6	7.9	8.2	8.5	8.8	9.1	9.4	9.7	10.0	10.3	10.6	10.9	11.3	11.6	11.9	12.2	12.5
820605	1	50	38	-20	3	3.8	4.5	5.3	6.1	6.9	7.6	8.4	9.2	10.0	10.7	11.5	12.3	13.1	13.9	14.6	15.4	16.2	17.0	17.7	18.5
820605	2	73	22	-20	3	1.5	2.0	2.4	2.9	3.3	3.8	4.3	4.7	5.2	5.6	6.1	6.6	7.0	7.5	7.9	8.4	8.8	9.3	9.8	10.2
830403	1	100	1500	-35	3	33.6	36.5	39.4	42.3	45.2	48.1	51.0	53.9	56.9	59.8	62.7	65.6	68.5	71.4	74.3	77.3	80.2	83.1	86.0	88.9
830403	2	113	900	-35	3	14.4	17.0	19.6	22.2	24.9	27.5	30.1	32.7	35.3	37.9	40.5	43.2	45.8	48.4	51.0	53.6	56.2	58.9	61.5	64.1
830403	3	148	1800	-35	3	38.8	41.8	44.9	47.9	50.9	53.9	56.9	59.9	63.0	66.0	69.0	72.0	75.1	78.1	81.1	84.1	87.1	90.2	93.2	96.2
830801	1	185	2000	-35	3	41.8	44.9	47.9	51.0	54.1	57.2	60.2	63.3	66.4	69.5	72.6	75.7	78.8	81.8	84.9	88.0	91.1	94.2	97.2	100.3
830801	2	103	400	-35	3	-0.9	1.2	3.4	5.5	7.6	9.8	11.9	14.1	16.2	18.4	20.5	22.7	24.8	26.9	29.1	31.2	33.4	35.5	37.7	39.8
830801	3	126	3000	-35	3	75.9	79.2	82.5	85.8	89.1	92.5	95.8	99.1	102.4	105.7	109.0	112.4	115.7	119.0	122.3	125.6	128.9	132.3	135.6	138.9
830801	4	170	1500	-35	3	28.3	31.2	34.1	37.1	40.0	42.9	45.8	48.7	51.6	54.5	57.4	60.4	63.3	66.2	69.1	72.0	74.9	77.8	80.8	83.7
840101	1	174	100	-23	1	1.7	3.0	4.3	5.7	7.0	8.3	9.8	11.0	12.4	13.7	15.0	16.4	17.7	19.1	20.4	21.7	23.1	24.4	25.8	27.1
840101	2	179	100	-23	1	1.5	2.8	4.2	5.5	6.8	8.2	9.5	10.9	12.2	13.5	14.9	16.2	17.5	18.9	20.2	21.6	22.9	24.2	25.6	26.9
840101	3	87	100	-23	1	4.6	6.0	7.3	8.7	10.0	11.3	12.7	14.0	15.4	16.7	18.0	19.4	20.7	22.1	23.4	24.7	26.1	27.4	28.7	30.1
840204	1	114	800	-24	2	18.6	21.2	23.7	26.3	28.8	31.4	33.9	36.5	39.0	41.6	44.1	46.7	49.2	51.8	54.3	56.9	59.4	61.9	64.5	67.0
840204	2	88	1200	-24	2	28.5	31.3	34.1	36.8	39.6	42.4	45.2	48.0	50.8	53.5	56.3	59.1	61.9	64.7	67.5	70.2	73.0	75.8	78.6	81.4
840604	1	107	1500	-25	2	31.8	34.7	37.6	40.5	43.4	46.3	49.3	52.2	55.1	58.0	60.9	63.8	66.7	69.6	72.5	75.5	78.4	81.3	84.2	87.1
840604	2	90	500	-25	2	12.5	14.8	17.0	19.3	21.6	23.9	26.2	28.4	30.7	33.0	35.2	37.5	39.8	42.1	44.3	46.6	48.9	51.2	53.4	55.7
840604	3	30	NA	-25	2	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
840604	4	35	1500	-25	2	37.2	40.1	43.0	45.9	48.8	51.7	54.6	57.5	60.5	63.4	66.3	69.2	72.1	75.0	77.9	80.9	83.8	86.7	89.6	92.5
850201	1	117	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
850201	2	73	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
850206	1	106	150	-30	1	-4.0	-2.4	-0.8	0.8	2.3	3.9	5.5	7.1	8.6	10.2	11.8	13.4	14.9	16.5	18.1	19.7	21.2	22.8	24.4	26.0
850206	2	63	150	-30	1	-2.2	-0.6	0.9	2.5	4.1	5.7	7.2	8.8	10.4	11.9	13.5	15.1	16.7	18.2	19.8	21.4	23.0	24.5	26.1	27.7
850601	1	122	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
850601	2	74	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
860501	1	55	50	-22	3	-1.5	-0.6	0.4	1.3	2.2	3.2	4.1	5.1	6.0	6.9	7.9	8.8	9.7	10.7	11.6	12.5	13.5	14.4	15.3	16.3
860501	2	85	50	-22	3	1.5	2.5	3.4	4.3	5.3	6.2	7.2	8.1	9.0	10.0	10.9	11.8	12.8	13.7	14.6	15.6	16.5	17.4	18.4	19.3
860501	3	41	50	-22	3	2.6	3.5	4.5	5.4	6.3	7.3	8.2	9.1	10.1	11.0	11.9	12.9	13.8	14.8	15.7	16.6	17.6	18.5	19.4	20.4
860603	1	86	50	-20	2	5.6	6.6	7.5	8.4	9.4	10.3	11.2	12.2	13.1	14.1	15.0	15.9	16.9	17.8	18.7	19.7	20.6	21.5	22.5	23.4
860603	2	80	50	-20	2	5.8	6.7	7.6	8.6	9.5	10.5	11.4	12.3	13.3	14.2	15.1	16.1	17.0	17.9	18.9	19.8	20.7	21.7	22.6	23.6
860603	3	34	50	-20	2	6.9	7.8	8.8	9.7	10.6	11.6	12.5	13.4	14.4	15.3	16.2	17.2	18.1	19.0	20.0	20.9	21.9	22.8	23.7	24.7
870102	1	95	100	-20	3	6.7	8.0	9.4	10.7	12.0	13.4	14.7	16.1	17.4	18.7	20.1	21.4	22.8	24.1	25.4	26.8	28.1	29.5	30.8	32.1
870102	2	46	200	-20	3	13.2	15.0	16.7	18.5	20.2	21.9	23.7	25.4	27.2	28.9	30.6	32.4	34.1	35.9	37.6	39.4	41.1	42.8	44.6	46.3
870504	1	31	800	-25	3	29.2	31.7	34.3	36.8	39.4	41.9	44.4	47.0	49.5	52.1	54.6	57.2	59.7	62.3	64.8	67.4	69.9	72.5	75.0	77.6
870504	2	61	1700	-25	3	55.0	58.0	61.0	64.0	67.0	70.0	73.0	75.9	78.9	81.9	84.9	87.9	90.9	93.9	96.9	99.8	102.8	105.8	108.8	111.8

Table B.2 Transverse Cracking Prediction Using Hajek Model (Page 2 of 2)

Test Site	Section	Thickness	Stiffness	Minimum	Subgrade	Cracking Index With Time (Years)[1]																			
		(mm)	(kg/cm^2)	Temperature (°C)		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
870505	1	75	60	-21	3	3.8	4.8	5.8	6.9	7.9	9.0	10.0	11.0	12.1	13.1	14.2	15.2	16.3	17.3	18.3	19.4	20.4	21.5	22.5	23.5
870505	2	73	NA	-21	3	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
870505	3	105	100	-21	3	5.1	6.4	7.8	9.1	10.5	11.8	13.1	14.5	15.8	17.1	18.5	19.8	21.2	22.5	23.8	25.2	26.5	27.9	29.2	30.5
870505	4	106	400	-21	3	16.4	18.5	20.7	22.8	25.0	27.1	29.3	31.4	33.6	35.7	37.8	40.0	42.1	44.3	46.4	48.6	50.7	52.9	55.0	57.1
870701	1	43	450	-25	3	16.6	18.9	21.1	23.3	25.5	27.7	29.9	32.1	34.4	36.6	38.8	41.0	43.2	45.4	47.6	49.8	52.1	54.3	56.5	58.7
870701	2	106	30	-25	3	8.0	9.9	11.9	13.9	15.9	17.8	19.8	21.8	23.8	25.8	27.7	29.7	31.7	33.7	35.6	37.6	39.6	41.6	43.6	45.5
880203	1	51	50	-20	2	6.5	7.4	8.3	9.3	10.2	11.1	12.1	13.0	14.0	14.9	15.8	16.8	17.7	18.6	19.6	20.5	21.4	22.4	23.3	24.3
880203	2	109	50	-20	2	5.1	6.0	6.9	7.9	8.8	9.8	10.7	11.7	12.6	13.5	14.4	15.4	16.3	17.2	18.2	19.1	20.0	21.0	21.9	22.9
880203	3	51	50	-20	2	6.5	7.4	8.3	9.3	10.2	11.1	12.1	13.0	14.0	14.9	15.8	16.8	17.7	18.6	19.6	20.5	21.4	22.4	23.3	24.3
880203	4	99	50	-20	2	5.3	6.3	7.2	8.1	9.1	10.0	10.9	11.9	12.8	13.7	14.7	15.6	16.5	17.5	18.4	19.4	20.3	21.2	22.2	23.1
890503	1	40	800	-27	3	26.1	28.6	31.2	33.7	36.3	38.8	41.4	43.9	46.5	49.0	51.5	54.1	56.6	59.2	61.7	64.3	66.8	69.4	71.9	74.5
890503	2	51	200	-27	3	4.3	6.0	7.7	9.5	11.2	13.0	14.7	16.4	18.2	19.9	21.7	23.4	25.2	26.9	28.6	30.4	32.1	33.9	35.6	37.4
890503	3	106	300	-27	3	5.5	7.4	9.4	11.4	13.4	15.3	17.3	19.3	21.3	23.3	25.2	27.2	29.2	31.2	33.1	35.1	37.1	39.1	41.1	43.0
890503	4	83	270	-27	3	5.5	7.4	9.4	11.3	13.2	15.1	17.0	18.9	20.9	22.8	24.7	26.6	28.5	30.4	32.4	34.3	36.2	38.1	40.0	41.9
890702	1	44	800	-28	2	18.2	20.8	23.3	25.9	28.4	31.0	33.5	36.1	38.6	41.2	43.7	46.2	48.8	51.3	53.9	56.4	59.0	61.5	64.1	66.6
890702	2	85	1200	-28	2	23.7	26.5	29.3	32.1	34.8	37.6	40.4	43.2	46.0	48.8	51.5	54.3	57.1	59.9	62.7	65.5	68.2	71.0	73.8	76.6
900402	1	86	130	-35	3	28.5	31.4	34.2	37.0	39.8	42.7	45.5	48.3	51.2	54.0	56.8	59.7	62.5	65.3	68.2	71.0	73.8	76.6	79.5	82.3
900402	2	126	NA	-35	3	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
900802	1	55	160	-35	3	40.0	42.9	45.9	48.8	51.8	54.7	57.7	60.6	63.6	66.5	69.5	72.4	75.4	78.3	81.3	84.2	87.2	90.1	93.1	96.0
900802	2	67	NA	-35	3	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
900802	3	104	NA	-35	3	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
900803	1	158	1200	-35	3	20.3	23.1	25.9	28.7	31.5	34.2	37.0	39.8	42.6	45.4	48.2	50.9	53.7	56.5	59.3	62.1	64.9	67.6	70.4	73.2
900803	2	108	NA	-35	3	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
Lamont	1	120	2200	-40	3	51.5	54.7	57.8	60.9	64.1	67.2	70.3	73.5	76.6	79.7	82.9	86.0	89.1	92.3	95.4	98.6	101.7	104.8	108.0	111.1
Lamont	2	97	1200	-40	3	30.9	33.8	36.7	39.6	42.6	45.5	48.4	51.3	54.2	57.1	60.0	62.9	65.9	68.8	71.7	74.6	77.5	80.4	83.3	86.3
Lamont	3	121	300	-40	3	-8.8	-6.8	-4.8	-2.9	-0.9	1.1	3.1	5.0	7.0	9.0	11.0	13.0	14.9	16.9	18.9	20.9	22.8	24.8	26.8	28.8
Lamont	4	105	3000	-40	3	76.1	79.4	82.7	86.1	89.4	92.7	96.0	99.3	102.6	106.0	109.3	112.6	115.9	119.2	122.5	125.9	129.2	132.5	135.8	139.1
Lamont	5	104	650	-40	3	4.3	6.8	9.2	11.6	14.1	16.5	18.9	21.3	23.8	26.2	28.6	31.0	33.5	35.9	38.3	40.7	43.2	45.6	48.0	50.5
Lamont	6	115	300	-40	3	-8.1	-6.2	-4.2	-2.2	-0.2	1.7	3.7	5.7	7.7	9.7	11.6	13.6	15.6	17.6	19.5	21.5	23.5	25.5	27.5	29.4
Lamont	7	110	450	-40	3	-2.8	-0.6	1.6	3.9	6.1	8.3	10.5	12.7	14.9	17.1	19.4	21.6	23.8	26.0	28.2	30.4	32.6	34.8	37.1	39.3
Hearst	AA	54	50	-35	1	-4.9	-3.9	-3.0	-2.0	-1.1	-0.2	0.8	1.7	2.6	3.6	4.5	5.4	6.4	7.3	8.2	9.2	10.1	11.1	12.0	12.9
Hearst	A	50	60	-35	1	-4.3	-3.3	-2.2	-1.2	-0.1	0.9	1.9	3.0	4.0	5.1	6.1	7.2	8.2	9.2	10.3	11.3	12.4	13.4	14.4	15.5
Hearst	B	59	80	-35	1	-4.0	-2.8	-1.6	-0.3	0.9	2.1	3.3	4.5	5.7	6.9	8.1	9.3	10.5	11.7	13.0	14.2	15.4	16.6	17.8	19.0
Hearst	BB	100	80	-35	1	-3.4	-2.2	-1.0	0.2	1.4	2.6	3.8	5.0	6.2	7.4	8.6	9.9	11.1	12.3	13.5	14.7	15.9	17.1	18.3	19.5
Sherbrooke	A	120	450	-35	3	-3.1	-0.9	1.4	3.6	5.8	8.0	10.2	12.4	14.6	16.9	19.1	21.3	23.5	25.7	27.9	30.1	32.3	34.6	36.8	39.0
Sherbrooke	B	120	2000	-35	3	45.4	48.5	51.6	54.7	57.8	60.8	63.9	67.0	70.1	73.2	76.2	79.3	82.4	85.5	88.6	91.6	94.7	97.8	100.9	104.0
Sherbrooke	C	120	1000	-35	3	14.9	18	20	23	26	28	31	34	36.3	39	41.6	44.3	47	49.7	52.3	55	57.7	60.4	63.1	65.7
Sherbrooke	D	120	1200	-35	3	21.1	24	27	29	32	35	38	41	43.4	46.1	48.9	51.7								

Table B.3 ANOVA Comparison of Hajek Prediction versus Actual Cracking

Using C-LTPP Sites and C-SHRP Sites

Anova: Two-Factor Without Replication

SUMMARY	Count	Sum	Average	Variance
Row 1	2	27.6	13.8	0.5
Row 2	2	11.2	5.6	2.8
Row 3	2	14.4	7.2	80.5
Row 4	2	14.7	7.4	58.4
Row 5	2	13.3	6.7	75.6
Row 6	2	9.7	4.9	47.0
Row 7	2	10.0	5.0	50.0
Row 8	2	8.8	4.4	38.7
Row 9	2	9.2	4.6	42.3
Row 10	2	4.7	2.4	11.0
Row 11	2	125.4	62.7	154.8
Row 12	2	61.7	30.8	6.8
Row 13	2	90.8	45.4	420.5
Row 14	2	152.8	76.4	524.8
Row 15	2	37.3	18.7	81.1
Row 16	2	192.4	96.2	0.3
Row 17	2	124.0	62.0	524.8
Row 18	2	9.8	4.9	48.0
Row 19	2	9.5	4.8	45.1
Row 20	2	13.4	6.7	72.0
Row 21	2	45.7	22.9	244.2
Row 22	2	68.0	34.0	250.9
Row 23	2	119.2	59.6	212.2
Row 24	2	84.8	42.4	524.9
Row 25	2	129.8	64.9	212.2
Row 26	2	10.7	5.4	0.0
Row 27	2	27.0	13.5	79.4
Row 28	2	4.1	2.1	8.4
Row 29	2	7.2	3.6	25.9
Row 30	2	8.2	4.1	33.6
Row 31	2	11.2	5.6	62.7
Row 32	2	11.4	5.7	65.0
Row 33	2	12.5	6.3	78.1
Row 34	2	112.4	56.2	3863.7
Row 35	2	244.4	122.2	20120.2
Row 36	2	75.7	37.9	32.8
Row 37	2	124.0	62.0	128.0
Row 38	2	9.0	4.5	40.5
Row 39	2	17.3	8.7	18.8
Row 40	2	50.9	25.5	5.4
Row 41	2	88.2	44.1	537.8
Row 42	2	72.4	36.2	677.1
Row 43	2	13.8	6.9	74.4
Row 44	2	11.7	5.9	68.4
Row 45	2	15.5	7.8	55.1
Row 46	2	11.9	6.0	70.8
Row 47	2	73.6	36.8	42.3
Row 48	2	24.4	12.2	12.5
Row 49	2	31.3	15.7	5.4
Row 50	2	32.0	16.0	2.0
Row 51	2	44.4	22.2	255.4
Row 52	2	63.1	31.6	156.6
Row 53	2	85.4	42.7	82.7
Row 54	2	98.9	49.5	248.6
Row 55	2	66.2	33.1	89.8
Row 56	2	79.2	39.6	1523.5
Row 57	2	63.4	31.7	380.9
Row 58	2	1.1	0.6	0.6
Row 59	2	110.8	55.4	2782.6
Row 60	2	23.6	11.8	44.2
Row 61	2	4.6	2.3	0.7
Row 62	2	9.3	4.7	28.6
Row 63	2	0.6	0.3	0.2
Row 64	2	1.2	0.6	0.2
Row 65	2	2.7	1.4	1.1
Row 66	2	4.2	2.1	0.5
Row 67	2	7.0	3.5	10.6
Row 68	2	64.6	32.3	1300.5
Row 69	2	30.5	15.3	214.2
Row 70	2	33.8	16.9	488.2
Column 1	70	1732.7	24.8	500.1
Column 2	70	1550.9	22.2	1300.8

ANOVA

Source of Variation	SS	df	MS	F	P-value	F crit
Rows	87309.21	69	1265.35	2.3621	0.0002	1.4900
Columns	236.08	1	236.08	0.4407	0.5090	3.9798
Error	36962.42	69	535.69			
Total	124507.71	139				

Table B.4 Power Calculations for Cracking Models (Page 1 of 1)

Power Calculations for Hajek Model and Observed Cracking

a	Sum of Squares	K	Degrees of Freedom	Variance	Number of Columns	Phi	Tables
0.018	100000	32.4000	1	535.69	69	1.4445	0.52
0.020	100000	40.0000	1	535.69	69	1.6050	0.62
0.022	100000	48.4000	1	535.69	69	1.7655	0.7
0.024	100000	57.6000	1	535.69	69	1.9260	0.77
0.026	100000	67.6000	1	535.69	69	2.0865	0.81
0.028	100000	78.4000	1	535.69	69	2.2470	0.88
0.030	100000	90.0000	1	535.69	69	2.4075	0.91
0.032	100000	102.4000	1	535.69	69	2.5680	0.94
0.034	100000	115.6000	1	535.69	69	2.7285	0.958
0.036	100000	129.6000	1	535.69	69	2.8891	0.982
0.038	100000	144.4000	1	535.69	69	3.0496	0.99
0.040	100000	160.0000	1	535.69	69	3.2101	0.99
0.042	100000	176.4000	1	535.69	69	3.3706	0.99
0.044	100000	193.6000	1	535.69	69	3.5311	
0.046	100000	211.6000	1	535.69	69	3.6916	
0.048	100000	230.4000	1	535.69	69	3.8521	
0.050	100000	250.0000	1	535.69	69	4.0126	
0.052	100000	270.4000	1	535.69	69	4.1731	

Power Calculations for Airport Model and Observed Cracking

a	Sum of Squares	K	Degrees of Freedom	Variance	Number of Columns	Phi	Tables
0.000	5000	0.0000	1	118.19	59	0.0000	
0.030	5000	4.5000	1	118.19	59	1.0598	
0.035	5000	6.1250	1	118.19	59	1.2364	0.36
0.040	5000	8.0000	1	118.19	59	1.4131	0.51
0.045	5000	10.1250	1	118.19	59	1.5897	0.58
0.050	5000	12.5000	1	118.19	59	1.7663	0.64
0.055	5000	15.1250	1	118.19	59	1.9430	0.77
0.060	5000	18.0000	1	118.19	59	2.1196	0.83
0.065	5000	21.1250	1	118.19	59	2.2962	0.86
0.070	5000	24.5000	1	118.19	59	2.4729	0.91
0.075	5000	28.1250	1	118.19	59	2.6495	0.95
0.080	5000	32.0000	1	118.19	59	2.8262	0.98
0.085	5000	36.1250	1	118.19	59	3.0028	0.989
0.090	5000	40.5000	1	118.19	59	3.1794	0.99
0.095	5000	45.1250	1	118.19	59	3.3561	
0.100	5000	50.0000	1	118.19	59	3.5327	
0.105	5000	55.1250	1	118.19	59	3.7093	
0.110	5000	60.5000	1	118.19	59	3.8860	
0.115	5000	66.1250	1	118.19	59	4.0626	
0.120	5000	72.0000	1	118.19	59	4.2392	
0.125	5000	78.1250	1	118.19	59	4.4159	
0.130	5000	84.5000	1	118.19	59	4.5925	

Table B.5 Airport Transverse Cracking Prediciton (Page 1 of 1)

Test Site	Section	PVN	Thickness (mm)	Minimum Temperature (°C)	Cracking Index Using Various Coefficient of Thermal Contraction					Cracks (/150m) Using Various Coefficient of Thermal Contraction				
					1.2	1.5	2.0	2.2	2.5	1.2	1.5	2.0	2.2	2.5
810404	1	-0.2179	61	-30	71.7	53.7	23.7	11.7	-6.3	2.1	2.8	6.3	12.9	-23.7
	2	-0.2936	103	-30	74.8	56.8	26.8	14.8	-3.2	2.0	2.6	5.6	10.2	-46.5
	3	-0.1239	94	-30	78.7	60.7	30.7	18.7	0.7	1.9	2.5	4.9	8.0	210.1
	4	-0.1158	55	-30	74.0	56.0	26.0	14.0	-4.0	2.0	2.7	5.8	10.7	-37.2
820205	1	-1.2162	42	-15	77.1	59.1	29.1	17.1	-0.9	1.9	2.5	5.2	8.8	-164.8
	2	-1.0576	83	-15	87.1	69.1	39.1	27.1	9.1	1.7	2.2	3.8	5.5	16.5
820502	1	-1.0167	104	-15	91.0	73.0	43.0	31.0	13.0	1.6	2.1	3.5	4.8	11.5
	2	-0.7761	118	-15	100.0	82.0	52.0	40.0	22.0	1.5	1.8	2.9	3.7	6.8
820605	1	-1.1493	50	-20	67.5	49.5	19.5	7.5	-10.5	2.2	3.0	7.7	19.9	-14.3
	2	-1.0404	73	-20	73.7	55.7	25.7	13.7	-4.3	2.0	2.7	5.8	10.9	-35.1
830403	1	-0.3470	100	-35	60.2	42.2	12.2	0.2	-17.8	2.5	3.6	12.3	789.5	-8.4
	2	-0.3454	113	-35	61.9	43.9	13.9	1.9	-16.1	2.4	3.4	10.8	78.9	-9.3
	3	-0.3722	148	-35	65.6	47.6	17.6	5.6	-12.4	2.3	3.2	8.5	26.9	-12.1
830801	1	-0.3786	185	-35	70.1	52.1	22.1	10.1	-7.9	2.1	2.9	6.8	14.8	-19.0
	2	-0.4103	103	-35	58.7	40.7	10.7	-1.3	-19.3	2.6	3.7	14.1	-113.2	-7.8
	3	-0.7134	126	-35	52.5	34.5	4.5	-7.5	-25.5	2.9	4.3	33.1	-20.1	-5.9
	4	-0.3594	170	-35	68.8	50.8	20.8	8.8	-9.2	2.2	3.0	7.2	17.1	-16.3
840101	1	-0.8537	174	-23	84.7	66.7	36.7	24.7	6.7	1.8	2.2	4.1	6.1	22.4
	2	NA	179	-23	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
	3	-1.0117	87	-23	68.8	50.8	20.8	8.8	-9.2	2.2	3.0	7.2	17.0	-16.3
840204	1	-0.7389	114	-24	77.9	59.9	29.9	17.9	-0.1	1.9	2.5	5.0	8.4	-2727.3
	2	-0.9298	88	-24	68.9	50.9	20.9	8.9	-9.1	2.2	2.9	7.2	16.9	-16.5
840604	1	-0.7329	107	-25	74.7	56.7	26.7	14.7	-3.3	2.0	2.6	5.6	10.2	-45.6
	2	-0.9828	90	-25	65.0	47.0	17.0	5.0	-13.0	2.3	3.2	8.8	29.8	-11.6
	3	NA	30	-25	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
	4	-0.4656	35	-25	73.5	55.5	25.5	13.5	-4.5	2.0	2.7	5.9	11.1	-33.4
850201	1	NA	117	-30	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
	2	NA	73	-30	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
850206	1	-0.4327	106	-30	71.0	53.0	23.0	11.0	-7.0	2.1	2.8	6.5	13.7	-21.4
	2	NA	63	-30	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
850601	1	NA	122	-30	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
	2	NA	74	-30	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
860501	1	-0.6978	55	-22	76.7	58.7	28.7	16.7	-1.3	2.0	2.6	5.2	9.0	-112.4
	2	-0.6978	85	-22	80.5	62.5	32.5	20.5	2.5	1.9	2.4	4.6	7.3	59.9
	3	-0.6978	41	-22	74.9	56.9	26.9	14.9	-3.1	2.0	2.6	5.6	10.1	-48.0
860603	1	-0.6978	86	-20	85.7	67.7	37.7	25.7	7.7	1.8	2.2	4.0	5.8	19.5
	2	-0.6832	80	-20	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
	3	-0.6832	34	-20	79.5	61.5	31.5	19.5	1.5	1.9	2.4	4.8	7.7	103.0
870102	1	-0.4065	95	-20	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
	2	-0.3920	46	-20	81.0	63.0	33.0	21.0	3.0	1.9	2.4	4.5	7.1	50.1
870504	1	-0.6100	31	-25	68.7	50.7	20.7	8.7	-9.3	2.2	3.0	7.3	17.3	-16.1
	2	-0.6960	61	-25	69.9	51.9	21.9	9.9	-8.1	2.1	2.9	6.8	15.1	-18.6
870505	1	0.6385	75	-21	121.8	103.8	73.8	61.8	43.8	1.2	1.4	2.0	2.4	3.4
	2	NA	73	-21	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
	3	-0.4945	105	-21	91.7	73.7	43.7	31.7	13.7	1.6	2.0	3.4	4.7	11.0
	4	-0.8812	106	-21	80.2	62.2	32.2	20.2	2.2	1.9	2.4	4.7	7.4	67.8
870701	1	NA	43	-25	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
	2	-0.5775	106	-25	79.2	61.2	31.2	19.2	1.2	1.9	2.4	4.8	7.8	120.7
880203	1	-1.1000	51	-20	69.1	51.1	21.1	9.1	-8.9	2.2	2.9	7.1	16.4	-16.9
	2	-0.6090	109	-20	91.3	73.3	43.3	31.3	13.3	1.6	2.0	3.5	4.8	11.3
	3	-0.6090	51	-20	83.9	65.9	35.9	23.9	5.9	1.8	2.3	4.2	6.3	25.6
	4	-0.6090	99	-20	90.0	72.0	42.0	30.0	12.0	1.7	2.1	3.6	5.0	12.5
890503	1	-0.4998	40	-27	68.1	50.1	20.1	8.1	-9.9	2.2	3.0	7.5	18.6	-15.1
	2	0.5482	51	-27	100.9	82.9	52.9	40.9	22.9	1.5	1.8	2.8	3.7	6.5
	3	-0.4998	106	-27	76.5	58.5	28.5	16.5	-1.5	2.0	2.6	5.3	9.1	-102.3
	4	0.5482	83	-27	105.0	87.0	57.0	45.0	27.0	1.4	1.7	2.6	3.3	5.5
890702	1	-0.4469	44	-28	67.7	49.7	19.7	7.7	-10.3	2.2	3.0	7.6	19.6	-14.5
	2	-0.4469	85	-28	72.9	54.9	24.9	12.9	-5.1	2.1	2.7	6.0	11.6	-29.5
900402	1	-0.7684	86	-35	45.8	27.8	-2.2	-14.2	-32.2	3.3	5.4	-66.8	-10.5	-4.7
	2	NA	126	-35	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
900802	1	-0.5432	55	-35	48.5	30.5	0.5	-11.5	-29.5	3.1	4.9	275.7	-13.1	-5.1
	2	NA	67	-35	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
	3	NA	104	-35	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
900803	1	-0.4545	158	-35	64.4	46.4	16.4	4.4	-13.6	2.3	3.2	9.2	34.2	-11.0
	2	NA	108	-35	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA

Notes: NA means not available

Table B.6 ANOVA Comparison of Airport Model and Observed Cracking (Page 1 of 2)

Anova Two-Factor Without Replication

SUMMARY	Count	Sum	Average	Variance
Row 1	2	27.2	13.6	1.0
Row 2	2	10.0	5.0	0.7
Row 3	2	3.7	1.9	0.0
Row 4	2	3.9	2.0	0.0
Row 5	2	2.4	1.2	1.0
Row 6	2	1.7	0.9	1.5
Row 7	2	1.6	0.8	1.4
Row 8	2	1.5	0.7	1.1
Row 9	2	2.2	1.1	2.5
Row 10	2	2.0	1.0	2.1
Row 11	2	83.8	41.9	1752.3
Row 12	2	39.8	19.9	165.6
Row 13	2	57.8	28.9	8.0
Row 14	2	107.4	53.7	3026.4
Row 15	2	39.5	19.8	63.8
Row 16	2	129.7	64.9	2016.1
Row 17	2	95.3	47.7	1866.6
Row 18	2	1.8	0.9	1.6
Row 19	2	2.9	1.5	1.1
Row 20	2	20.2	10.1	5.9
Row 21	2	39.7	19.8	17.6
Row 22	2	80.1	40.0	1782.2
Row 23	2	88.4	44.2	415.1
Row 24	2	36.3	43.2	2054.3
Row 25	2	11.7	5.9	0.8
Row 26	2	2.0	1.0	1.9
Row 27	2	1.9	0.9	1.7
Row 28	2	2.0	1.0	2.0
Row 29	2	1.8	0.9	1.5
Row 30	2	1.9	0.9	1.8
Row 31	2	116.9	58.4	138.6
Row 32	2	51.1	25.6	136.0
Row 33	2	69.1	34.6	756.3
Row 34	2	1.2	0.6	0.7
Row 35	2	10.2	5.1	0.3
Row 36	2	31.2	15.6	134.1
Row 37	2	175.3	87.7	2184.6
Row 38	2	3.0	1.5	1.0
Row 39	2	1.6	0.8	1.3
Row 40	2	4.8	2.4	0.0
Row 41	2	1.7	0.9	1.4
Row 42	2	50.8	25.4	92.5
Row 43	2	16.2	8.1	5.1
Row 44	2	23.1	11.6	12.0
Row 45	2	20.5	10.3	45.1
Row 46	2	18.5	9.3	5.4
Row 47	2	34.3	17.2	61.6
Row 48	2	42.5	21.3	502.4
Row 49	2	43.2	21.6	557.8
Row 50	2	60.6	30.3	30.4
Row 51	2	22.4	11.2	1.3
Row 52	2	31.0	15.5	11.5
Row 53	2	4.7	2.4	11.0
Row 54	2	24.9	12.5	63.8
Row 55	2	11.3	5.7	4.2
Row 56	2	-1.7	-0.9	28.1
Row 57	2	4.2	2.1	2.4
Row 58	2	7.9	4.0	22.4
Row 59	2	7.8	3.9	25.9
Row 60	2	9.6	4.8	35.3
Row 61	2	8.4	4.2	13.5
Row 62	2	3.9	2.0	1.1
Row 63	2	11.2	5.6	2.9
Row 64	2	7.5	3.8	2.6
Row 65	2	6.1	3.1	4.2
Column 1	65	669.3	10.3	280.8
Column 2	65	1215.9	18.7	663.7

ANOVA

ource of Variatio	SS	df	MS	F	P-value	F crit
Rows	44654.8	64	697.7	2.8267	2.53E-05	1.5133
Columns	2297.7	1	2297.7	9.3084	0.0033	3.9909
Error	15797.5	64	246.8			
Total	62750.03	129				

Table B.6 ANOVA Comparison of Airport Model and Observed Cracking (Page 2 of 2)

ANOVA Less Than 70 Cracks

Anova Two-Factor Without Replication

SUMMARY	Count	Sum	Average	Variance
Row 1	2	27.2	13.6	1.0
Row 2	2	10.0	5.0	0.7
Row 3	2	3.7	1.9	0.0
Row 4	2	3.9	2.0	0.0
Row 5	2	2.4	1.2	1.0
Row 6	2	1.7	0.9	1.5
Row 7	2	1.6	0.8	1.4
Row 8	2	1.5	0.7	1.1
Row 9	2	2.2	1.1	2.5
Row 10	2	2.0	1.0	2.1
Row 11	2	39.8	19.9	165.6
Row 12	2	57.8	28.9	8.0
Row 13	2	39.5	19.8	63.8
Row 14	2	1.8	0.9	1.6
Row 15	2	2.9	1.5	1.1
Row 16	2	20.2	10.1	5.9
Row 17	2	39.7	19.8	17.6
Row 18	2	80.1	40.0	1782.2
Row 19	2	88.4	44.2	415.1
Row 20	2	11.7	5.9	0.8
Row 21	2	2.0	1.0	1.9
Row 22	2	1.9	0.9	1.7
Row 23	2	2.0	1.0	2.0
Row 24	2	1.8	0.9	1.5
Row 25	2	1.9	0.9	1.8
Row 26	2	116.9	58.4	138.6
Row 27	2	51.1	25.6	136.0
Row 28	2	69.1	34.6	756.3
Row 29	2	1.2	0.6	0.7
Row 30	2	10.2	5.1	0.3
Row 31	2	31.2	15.6	134.1
Row 32	2	175.3	87.7	2184.6
Row 33	2	3.0	1.5	1.0
Row 34	2	1.6	0.8	1.3
Row 35	2	4.8	2.4	0.0
Row 36	2	1.7	0.9	1.4
Row 37	2	50.8	25.4	92.5
Row 38	2	16.2	8.1	5.1
Row 39	2	23.1	11.6	12.0
Row 40	2	20.5	10.3	45.1
Row 41	2	18.5	9.3	5.4
Row 42	2	34.3	17.2	61.6
Row 43	2	42.5	21.3	502.4
Row 44	2	43.2	21.6	557.8
Row 45	2	60.6	30.3	30.4
Row 46	2	22.4	11.2	1.3
Row 47	2	31.0	15.5	11.5
Row 48	2	4.7	2.4	11.0
Row 49	2	24.9	12.5	63.8
Row 50	2	11.3	5.7	4.2
Row 51	2	-1.7	-0.9	28.1
Row 52	2	4.2	2.1	2.4
Row 53	2	7.9	4.0	22.4
Row 54	2	7.8	3.9	25.9
Row 55	2	9.6	4.8	35.3
Row 56	2	8.4	4.2	13.5
Row 57	2	3.9	2.0	1.1
Row 58	2	11.2	5.6	2.9
Row 59	2	7.5	3.8	2.6
Row 60	2	6.1	3.1	4.2
Column 1	60	580.9	9.7	294.2
Column 2	60	801.8	13.4	334.2

ANOVA

ource of Variatio	SS	df	MS	F	P-value	F crit
Rows	30106.2	59	510.3	4.3175	3.9E-08	1.5400
Columns	406.4	1	406.4	3.4385	0.0687	4.0040
Error	6973.0	59	118.2			
Total	37485.58	119				

Table B.7 ANOVA For C-SHRP Test Sections (Page 1 of 1)
Thermal Contraction Coefficients ANOVA
Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	2	3.23	1.62	0.00
Row 2	2	3.25	1.63	0.00
Row 3	2	2.63	1.32	0.03
Row 4	2	3.45	1.73	0.03
Row 5	2	4.00	2.00	0.98
Row 6	2	2.40	1.20	0.72
Row 7	2	2.00	1.00	0.50
Row 8	2	3.20	1.60	0.02
Row 9	2	3.50	1.75	0.13
Row 10	2	2.00	1.00	0.50
Column 1	10	16.50	1.65	0.16
Column 2	10	13.16	1.32	0.33

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	2.05	9	0.2274	0.8722	0.5791	3.1789
Columns	0.56	1	0.5578	2.1393	0.1776	5.1174
Error	2.35	9	0.2607			
Total	4.95	19				

APPENDIX C

MODULE THREE: PERFORMANCE PREDICTION

Table C.1 Comparison of IRI Prediction Using Hein Equation (Page 1 of 1)

IRI	
	IRI
0.8973	1.6575
1.0263	1.4202
1.7650	1.3350
1.8157	1.1742
1.9879	1.7119
2.1855	1.3724
2.2336	0.7645
2.2437	1.4094
2.3824	1.4212
2.4170	1.7804
2.4347	1.0688
2.4459	1.8380
2.4845	1.1715
2.4878	1.4523
2.5092	1.1829
2.6497	1.3404
2.7024	1.3563
2.7300	1.5585
2.7541	1.7981
2.8204	1.1464
2.8371	0.9288
2.8541	1.2313
2.9534	1.4971
3.0271	1.5735
3.0640	1.0284
3.0669	1.5566
3.1377	1.2946
3.1468	1.1477
3.3460	0.9203
3.7422	0.9856
4.2804	1.1342
4.4153	1.8064
4.4796	1.0174
4.7307	1.6597
4.9517	1.4847
5.2058	1.4204

Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	2	2.55	1.28	0.29
Row 2	2	2.45	1.22	0.08
Row 3	2	3.10	1.55	0.09
Row 4	2	2.99	1.49	0.21
Row 5	2	3.70	1.85	0.04
Row 6	2	3.56	1.78	0.33
Row 7	2	3.00	1.50	1.08
Row 8	2	3.65	1.83	0.35
Row 9	2	3.80	1.90	0.46
Row 10	2	4.20	2.10	0.20
Row 11	2	3.50	1.75	0.93
Row 12	2	4.28	2.14	0.18
Row 13	2	3.66	1.83	0.86
Row 14	2	3.94	1.97	0.54
Row 15	2	3.69	1.85	0.88
Row 16	2	3.99	2.00	0.86
Row 17	2	4.06	2.03	0.91
Row 18	2	4.29	2.14	0.69
Row 19	2	4.55	2.28	0.46
Row 20	2	3.97	1.98	1.40
Row 21	2	3.77	1.88	1.82
Row 22	2	4.09	2.04	1.32
Row 23	2	4.45	2.23	1.06
Row 24	2	4.60	2.30	1.06
Row 25	2	4.09	2.05	2.07
Row 26	2	4.62	2.31	1.14
Row 27	2	4.43	2.22	1.70
Row 28	2	4.29	2.15	2.00
Row 29	2	4.27	2.13	2.94
Row 30	2	4.73	2.36	3.80
Row 31	2	5.41	2.71	4.95
Row 32	2	6.22	3.11	3.40
Row 33	2	5.50	2.75	5.99
Row 34	2	6.39	3.20	4.72
Row 35	2	6.44	3.22	6.01
Row 36	2	6.63	3.31	7.16
Column 1	36	104.21	2.89	0.98
Column 2	36	48.65	1.35	0.08

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	18.10	35	0.52	0.9482	0.5620	1.7571
Columns	42.88	1	42.88	78.6327	1.8E-10	4.1213
Error	19.09	35	0.55			
Total	80.07	71				

Table C.2 Comparison of IRI Prediciton Using Paterson Equation (Page 1 of 1)

IRI	
	IRI
1.1537	1.6575
1.3078	1.4202
2.2838	1.3350
2.3576	1.1742
2.6157	1.7119
2.9275	1.3724
3.0063	0.7645
3.0229	1.4094
3.2569	1.4212
3.3169	1.7804
3.3478	1.0688
3.3674	1.8380
3.4358	1.1715
3.4416	1.4523
3.4798	1.1829
3.7380	1.3404
3.8380	1.3563
3.8911	1.5585
3.9378	1.7981
4.0687	1.1464
4.1021	0.9288
4.1363	1.2313
4.3407	1.4971
4.4974	1.5735
4.5777	1.0284
4.5840	1.5566
4.7414	1.2946
4.7619	1.1477
5.2323	0.9203
6.3065	0.9856
8.2110	1.1342
8.8129	1.8064
9.1241	1.0174

Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	2	2.81	1.41	0.13
Row 2	2	2.73	1.36	0.01
Row 3	2	3.62	1.81	0.45
Row 4	2	3.53	1.77	0.70
Row 5	2	4.33	2.16	0.41
Row 6	2	4.30	2.15	1.21
Row 7	2	3.77	1.89	2.51
Row 8	2	4.43	2.22	1.30
Row 9	2	4.68	2.34	1.68
Row 10	2	5.10	2.55	1.18
Row 11	2	4.42	2.21	2.60
Row 12	2	5.21	2.60	1.17
Row 13	2	4.61	2.30	2.56
Row 14	2	4.89	2.45	1.98
Row 15	2	4.66	2.33	2.64
Row 16	2	5.08	2.54	2.87
Row 17	2	5.19	2.60	3.08
Row 18	2	5.45	2.72	2.72
Row 19	2	5.74	2.87	2.29
Row 20	2	5.22	2.61	4.27
Row 21	2	5.03	2.52	5.03
Row 22	2	5.37	2.68	4.22
Row 23	2	5.84	2.92	4.04
Row 24	2	6.07	3.04	4.27
Row 25	2	5.61	2.80	6.30
Row 26	2	6.14	3.07	4.58
Row 27	2	6.04	3.02	5.94
Row 28	2	5.91	2.95	6.53
Row 29	2	6.15	3.08	9.30
Row 30	2	7.29	3.65	14.16
Row 31	2	9.35	4.67	25.04
Row 32	2	10.62	5.31	24.55
Row 33	2	10.14	5.07	32.86
Column 1	33	135.22	4.10	3.27
Column 2	33	44.08	1.34	0.08

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	50.61	32	1.58	0.8923	0.6254	1.8045
Columns	125.86	1	125.86	71.0034	1.3E-09	4.1491
Error	56.72	32	1.77			
Total	233.20	65				

Table C.3 Comparison of IRI Prediction Using Al-Omari Equation (Page 1 of 1)

IRI	IRI
0.8032	1.6575
0.9104	1.4202
1.5899	1.3350
1.6412	1.1742
1.8210	1.7119
2.0380	1.3724
2.0928	0.7645
2.1044	1.4094
2.2673	1.4212
2.3091	1.7804
2.3306	1.0688
2.3442	1.8380
2.3918	1.1715
2.3959	1.4523
2.4225	1.1829
2.6022	1.3404
2.6719	1.3563
2.7088	1.5585
2.7413	1.7981
2.8325	1.1464
2.8557	0.9288
2.8795	1.2313
3.0218	1.4971
3.1309	1.5735
3.1868	1.0284
3.1912	1.5566
3.3007	1.2946
3.3150	1.1477
3.6425	0.9203
4.3903	0.9856
5.7161	1.1342
6.1351	1.8064
6.3518	1.0174
7.3399	1.6597
8.4867	1.4847

Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	2	2.46	1.23	0.36
Row 2	2	2.33	1.17	0.13
Row 3	2	2.92	1.46	0.03
Row 4	2	2.82	1.41	0.11
Row 5	2	3.53	1.77	0.01
Row 6	2	3.41	1.71	0.22
Row 7	2	2.86	1.43	0.88
Row 8	2	3.51	1.76	0.24
Row 9	2	3.69	1.84	0.36
Row 10	2	4.09	2.04	0.14
Row 11	2	3.40	1.70	0.80
Row 12	2	4.18	2.09	0.13
Row 13	2	3.56	1.78	0.74
Row 14	2	3.85	1.92	0.45
Row 15	2	3.61	1.80	0.77
Row 16	2	3.94	1.97	0.80
Row 17	2	4.03	2.01	0.87
Row 18	2	4.27	2.13	0.66
Row 19	2	4.54	2.27	0.44
Row 20	2	3.98	1.99	1.42
Row 21	2	3.78	1.89	1.86
Row 22	2	4.11	2.06	1.36
Row 23	2	4.52	2.26	1.16
Row 24	2	4.70	2.35	1.21
Row 25	2	4.22	2.11	2.33
Row 26	2	4.75	2.37	1.34
Row 27	2	4.60	2.30	2.01
Row 28	2	4.46	2.23	2.35
Row 29	2	4.56	2.28	3.71
Row 30	2	5.38	2.69	5.80
Row 31	2	6.85	3.43	10.50
Row 32	2	7.94	3.97	9.37
Row 33	2	7.37	3.68	14.23
Row 34	2	9.00	4.50	16.13
Row 35	2	9.97	4.99	24.51
Column 1	35	109.96	3.14	2.93
Column 2	35	47.23	1.35	0.08

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	51.24	34	1.51	1.0011	0.4987	1.7721
Columns	56.23	1	56.23	37.3477	6.2E-07	4.1300
Error	51.19	34	1.51			
Total	158.66	69				

Table C.4 Comparison of IRI Prediction Using Hajek Road User Equation (Page 1 of 1)

IRI	
	IRI
0.7156	1.6575
0.8761	1.4202
1.7955	1.3350
1.8587	1.1742
2.0730	1.7119
2.3189	1.3724
2.3788	0.7645
2.3914	1.4094
2.5640	1.4212
2.6071	1.7804
2.6291	1.0688
2.6430	1.8380
2.6911	1.1715
2.6952	1.4523
2.7218	1.1829
2.8967	1.3404
2.9624	1.3563
2.9967	1.5585
3.0266	1.7981
3.1092	1.1464
3.1300	0.9288
3.1511	1.2313
3.2747	1.4971
3.3664	1.5735
3.4124	1.0284
3.4160	1.5566
3.5041	1.2946
3.5154	1.1477
3.7634	0.9203
4.2565	0.9856
4.9264	1.1342
5.0944	1.8064
5.1743	1.0174
5.4869	1.6597
5.7620	1.4847
6.0782	1.4204

Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	2	2.37	1.19	0.44
Row 2	2	2.30	1.15	0.15
Row 3	2	3.13	1.57	0.11
Row 4	2	3.03	1.52	0.23
Row 5	2	3.78	1.89	0.07
Row 6	2	3.69	1.85	0.45
Row 7	2	3.14	1.57	1.30
Row 8	2	3.80	1.90	0.48
Row 9	2	3.99	1.99	0.65
Row 10	2	4.39	2.19	0.34
Row 11	2	3.70	1.85	1.22
Row 12	2	4.48	2.24	0.32
Row 13	2	3.86	1.93	1.15
Row 14	2	4.15	2.07	0.77
Row 15	2	3.90	1.95	1.18
Row 16	2	4.24	2.12	1.21
Row 17	2	4.32	2.16	1.29
Row 18	2	4.56	2.28	1.03
Row 19	2	4.82	2.41	0.75
Row 20	2	4.26	2.13	1.93
Row 21	2	4.06	2.03	2.42
Row 22	2	4.38	2.19	1.84
Row 23	2	4.77	2.39	1.58
Row 24	2	4.94	2.47	1.61
Row 25	2	4.44	2.22	2.84
Row 26	2	4.97	2.49	1.73
Row 27	2	4.80	2.40	2.44
Row 28	2	4.66	2.33	2.80
Row 29	2	4.68	2.34	4.04
Row 30	2	5.24	2.62	5.35
Row 31	2	6.06	3.03	7.19
Row 32	2	6.90	3.45	5.41
Row 33	2	6.19	3.10	8.64
Row 34	2	7.15	3.57	7.32
Row 35	2	7.25	3.62	9.15
Row 36	2	7.50	3.75	10.85
Column 1	36	115.26	3.20	1.53
Column 2	36	48.65	1.35	0.08

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	27.44	35	0.78	0.9571	0.5512	1.7571
Columns	61.64	1	61.64	75.2418	3.1E-10	4.1213
Error	28.67	35	0.82			
Total	117.75	71				

Table C.5 Comparison of IRI Prediction Using Hajek Truck User Equation (Page 1 of 1)

IRI	
	IRI
0.7337	1.6575
0.8642	1.4202
1.6116	1.3350
1.6629	1.1742
1.8371	1.7119
2.0370	1.3724
2.0857	0.7645
2.0959	1.4094
2.2362	1.4212
2.2712	1.7804
2.2891	1.0688
2.3005	1.8380
2.3395	1.1715
2.3428	1.4523
2.3645	1.1829
2.5067	1.3404
2.5600	1.3563
2.5880	1.5585
2.6123	1.7981
2.6794	1.1464
2.6963	0.9288
2.7135	1.2313
2.8139	1.4971
2.8885	1.5735
2.9259	1.0284
2.9288	1.5566
3.0004	1.2946
3.0096	1.1477
3.2112	0.9203
3.6120	0.9856
4.1565	1.1342
4.2931	1.8064
4.3581	1.0174
4.6121	1.6597
4.8358	1.4847
5.0928	1.4204

Anova: Two-Factor Without Replication

SUMMARY	Count	Sum	Average	Variance
Row 1	2	2.39	1.20	0.43
Row 2	2	2.28	1.14	0.15
Row 3	2	2.95	1.47	0.04
Row 4	2	2.84	1.42	0.12
Row 5	2	3.55	1.77	0.01
Row 6	2	3.41	1.70	0.22
Row 7	2	2.85	1.43	0.87
Row 8	2	3.51	1.75	0.24
Row 9	2	3.66	1.83	0.33
Row 10	2	4.05	2.03	0.12
Row 11	2	3.36	1.68	0.74
Row 12	2	4.14	2.07	0.11
Row 13	2	3.51	1.76	0.68
Row 14	2	3.80	1.90	0.40
Row 15	2	3.55	1.77	0.70
Row 16	2	3.85	1.92	0.68
Row 17	2	3.92	1.96	0.72
Row 18	2	4.15	2.07	0.53
Row 19	2	4.41	2.21	0.33
Row 20	2	3.83	1.91	1.18
Row 21	2	3.63	1.81	1.56
Row 22	2	3.94	1.97	1.10
Row 23	2	4.31	2.16	0.87
Row 24	2	4.46	2.23	0.86
Row 25	2	3.95	1.98	1.80
Row 26	2	4.49	2.24	0.94
Row 27	2	4.30	2.15	1.45
Row 28	2	4.16	2.08	1.73
Row 29	2	4.13	2.07	2.62
Row 30	2	4.60	2.30	3.45
Row 31	2	5.29	2.65	4.57
Row 32	2	6.10	3.05	3.09
Row 33	2	5.38	2.69	5.58
Row 34	2	6.27	3.14	4.36
Row 35	2	6.32	3.16	5.62
Row 36	2	6.51	3.26	6.74
Column 1	36	99.17	2.75	1.01
Column 2	36	48.65	1.35	0.08

ANOVA

Source of Variation	SS	df	MS	F	P-value	F crit
Rows	18.50	35	0.53	0.9487	0.5614	1.7571
Columns	35.45	1	35.45	63.6251	2.2E-09	4.1213
Error	19.50	35	0.56			
Total	73.45	71				

Table C.6 ANOVA For IRI Predicted Versus Observed IRI With Less Than 10 Cracks (Page 1 of 1)

Hein

Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	2	2.55	1.28	0.29
Row 2	2	2.45	1.22	0.08
Row 3	2	3.10	1.55	0.09
Row 4	2	2.99	1.49	0.21
Row 5	2	3.70	1.85	0.04
Row 6	2	3.56	1.78	0.33
Row 7	2	3.00	1.50	1.08
Row 8	2	3.65	1.83	0.35
Column 1	8	14.16	1.77	0.28
Column 2	8	10.85	1.36	0.09

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	0.81	7	0.1155	0.4554	0.8394	3.7871
Columns	0.68	1	0.6847	2.6991	0.1444	5.5915
Error	1.78	7	0.2537			
Total	3.27	15				

Al-Omari

Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	2	2.46	1.23	0.36
Row 2	2	2.33	1.17	0.13
Row 3	2	2.92	1.46	0.03
Row 4	2	2.82	1.41	0.11
Row 5	2	3.53	1.77	0.01
Row 6	2	3.41	1.71	0.22
Row 7	2	2.86	1.43	0.88
Row 8	2	3.51	1.76	0.24
Column 1	8	13.00	1.63	0.26
Column 2	8	10.85	1.36	0.09

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	0.76	7	0.1080	0.4456	0.8459	3.7871
Columns	0.29	1	0.2905	1.1980	0.3099	5.5915
Error	1.70	7	0.2424			
Total	2.74	15				

Hajek truck users

Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	2	2.39	1.20	0.43
Row 2	2	2.28	1.14	0.15
Row 3	2	2.95	1.47	0.04
Row 4	2	2.84	1.42	0.12
Row 5	2	3.55	1.77	0.01
Row 6	2	3.41	1.70	0.22
Row 7	2	2.85	1.43	0.87
Row 8	2	3.51	1.75	0.24
Column 1	8	12.93	1.62	0.29
Column 2	8	10.85	1.36	0.09

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	0.83	7	0.1180	0.4578	0.8378	3.7871
Columns	0.27	1	0.2712	1.0518	0.3392	5.5915
Error	1.80	7	0.2578			
Total	2.90	15				

Paterson

Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	2	2.81	1.41	0.13
Row 2	2	2.73	1.36	0.01
Row 3	2	3.62	1.81	0.45
Row 4	2	3.53	1.77	0.70
Row 5	2	4.33	2.16	0.41
Row 6	2	4.30	2.15	1.21
Row 7	2	3.77	1.89	2.51
Row 8	2	4.43	2.22	1.30
Column 1	8	18.68	2.33	0.54
Column 2	8	10.85	1.36	0.09

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	1.53	7	0.2189	0.5313	0.7884	3.7871
Columns	3.83	1	3.8320	9.3023	0.0186	5.5915
Error	2.88	7	0.4119			
Total	8.25	15				

Hajek carusers

Anova: Two-Factor Without Replication

SUMMARY	*Count*	*Sum*	*Average*	*Variance*
Row 1	2	2.37	1.19	0.44
Row 2	2	2.30	1.15	0.15
Row 3	2	3.13	1.57	0.11
Row 4	2	3.03	1.52	0.23
Row 5	2	3.78	1.89	0.07
Row 6	2	3.69	1.85	0.45
Row 7	2	3.14	1.57	1.30
Row 8	2	3.80	1.90	0.48
Column 1	8	14.41	1.80	0.44
Column 2	8	10.85	1.36	0.09

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	1.23	7	0.1761	0.5060	0.8056	3.7871
Columns	0.79	1	0.7934	2.2790	0.1749	5.5915
Error	2.44	7	0.3481			
Total	4.46	15				

APPENDIX D

MODULE FOUR: LIFE-CYCLE COSTING

Table D.1 Thickness Values Based On C-LTPP Sites (Page 1 of 1)

C-SHRP ID	Thickness Mean(mm)	Thickness Min(mm)	Thickness Max(mm)	Thickness of Tests	Thickness StdDev
860603	33	19	45	4	13.00
840604	33	25	40	4	7.64
840101	34	33	36	4	1.53
860603	34	26	42	4	8.00
870504	34	33	35	4	0.82
830403	35	33	36	4	1.70
840604	35	21	42	5	9.60
830403	35	30	45	4	8.40
820205	36	32	42	7	4.46
830403	36	36	36	4	0.00
840204	36	35	36	4	0.58
850601	36	22	45	4	13.00
900402	36	35	37	4	1.00
890702	37	40	35	4	2.36
850206	37	35	42	4	3.30
900402	37	36	38	4	1.00
900802	37	34	40	4	3.00
840604	37	34	39	4	2.89
870505	38	35	40	3	3.54
870505	38	35	40	3	3.54
870505	38	35	40	3	3.54
850201	38	33	43	3	7.00
850601	38	30	48	4	9.00
810404	39	36	42	4	3.10
890503	40	34	45	4	5.51
810404	40	40	41	4	0.60
850601	40	38	40	4	1.00
850601	40	39	41	3	1.40
870505	40	40	40	3	0.00
830403	40	40	41	4	0.60
860501	41	36	47	4	5.00
820205	42	38	44	7	2.34
840101	42	35	50	4	7.55
850601	42	34	54	4	11.00
860603	42	34	48	4	7.30
870701	43	39	48	5	4.04
830403	43	41	45	4	2.00
850206	43	40	45	4	2.20
860603	44	42	50	4	5.70
870505	44	35	65	5	14.40
840101	45	40	51	4	5.69
850206	45	45	46	4	0.50
900802	45	35	54	4	9.60
840101	46	45	48	4	1.53
870102	46	44	49	7	1.94
830403	47	42	53	4	5.70
820205	47	46	48	7	0.82
860603	47	44	52	4	4.30
850201	48	33	60	4	14.00
870102	48	45	50	4	2.88
890702	48	45	51	4	
830403	49	48	50	4	1.00
840101	49	33	60	4	10.00
890702	49	43	59	4	7.12
900402	49	48	50	4	0.60
820502	50	50	50	7	0.00
820502	50	50	50	7	0.00
820605	50	40	58	7	6.90
840101	50	42	60	4	9.20
890503	51	50	52	4	1.16
850201	51	46	56	3	7.00
880203	51	51	52	4	0.58
880203	51	46	56	4	5.03
830403	52	50	54	4	2.10
900402	53	51	56	4	2.60
820502	54	53	54	7	
810404	55	51	62	4	5.90
810404	55	51	58	4	5.30
860501	55	50	61	4	5.00
840604	57	52	63	4	5.69
840101	57	45	80	4	19.60
840101	59	48	67	4	10.00
810404	61	60	63	4	1.70
870505	63	45	85	5	18.50
810404	63	53	69	4	9.00
870701	64	51	76	3	17.68
870505	68	65	70	3	3.54
820502	68	67	68	7	
840604	70	57	80	4	11.80
890503	70	65	76	4	5.51
820605	73	71	74	7	1.33
860501	85	75	90	4	9.00
890503	91	86	95	4	4.58
880203	99	91	104	4	3.50
830801	103	90	120	4	15.00
880203	109	105	112	4	2.90
830801	126	115	138	4	12.00
830801	170	165	180	4	9.00
830801	185	180	190	4	5.00
890503	12	8	16	4	
900803	13	13	14	4	0.60
890503	15	14	15	4	
900803	18	17	19	4	1.20
900803	19	15	26	4	6.40
850206	20	15	25	4	4.10
900802	22	13	27	4	7.80
900803	22	22	23	4	0.60
900803	23	22	23	4	0.60
840204	23	20	25	4	2.89
830403	24	22	26	3	2.80
850206	24	18	27	4	4.20
850201	25	40	13	4	0.60
900803	27	27	28	4	0.60
870504	27	25	29	4	1.70
900802	28	22	32	4	5.10
900802	28	24	31	4	3.60
840101	28	23	33	4	5.00
850201	28	21	35	3	10.00
900802	28	24	33	4	4.60
840604	30	25	34	3	6.36
900802	30	29	31	4	1.20
840101	30	28	32	4	2.00
870504	31	30	33	4	0.82
870505	33	30	35	3	3.54
840204	33	32	36	4	2.30

Table D.2 Prices Granular A (Page 1 of 3)

CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE	CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE
95-67	1	95	11	40.79	93-35	2	93	20450	10.16
97-211	19	97	41	83.38	96-17	8	96	20521	9.5
94-67	6	94	48	21.88	96-16	9	96	21510	13.67
93-94	6	93	76	34.69	94-14	5	94	21538	11.94
94-36	6	94	96	29.86	93-26	9	93	21681	11.54
95-206	18	95	101	36.83	93-36	8	93	21819	12.1
93-46	3	93	147	22.9	97-05	1	97	21846	16.16
95-56	5	95	150	54	94-13	2	94	21952	8.99
95-55	6	95	172	36.31	96-59	9	96	22082	9.17
96-226	19	96	194	33.72	97-49	6	97	22246	9.76
97-17	6	97	202	19.33	93-39	6	93	22441	10.03
93-81	9	93	211	19.48	97-13	6	97	22544	9.87
96-208	19	96	244	33.08	96-07	9	96	22758	9.25
97-15	6	97	248	29.69	94-79	6	94	23091	19.2
93-90	9	93	301	22.6	96-27	1	96	23165	15.49
97-215	14	97	317	14.7	94-10	2	94	23625	7.4
96-62	6	96	332	27.48	93-42	9	93	23660	10.75
94-38	6	94	344	23.94	93-52	6	93	23706	11.2
97-210	19	97	349	41.99	97-11	6	97	23778	10.64
93-104	6	93	421	21.48	96-33	11	96	23844	10.76
94-15	5	94	425	9.98	94-224	18	94	24801	13.26
97-16	6	97	430	19.85	94-44	2	94	24982	9.62
93-307	2	93	487	21.88	97-28	6	97	25301	10.12
94-34	6	94	512	32.72	94-39	6	94	25325	16.03
93-08	2	93	589	21.69	93-78	11	93	25441	11.64
96-241	17	96	590	25.5	94-01	1	94	25671	17.86
96-73	2	96	602	9.51	96-38	6	96	25956	6.96
93-107	6	93	676	36	93-700	19	93	26038	14.47
96-54	6	96	745	16.25	96-31	5	96	26111	9.47
96-78	9	96	748	14.87	94-213	19	94	26270	14.5
97-31	6	97	766	14.18	96-57	6	96	26386	8.9
95-62	6	95	823	21.84	94-35	1	94	26443	17.95
93-34	3	93	1050	14.81	94-56	11	94	26473	12.47
95-66	5	95	1124	13.12	95-10	9	95	26561	9.52
93-68	1	93	1126	23.5	94-504	13	94	26784	13.06
94-215	19	94	1187	17.71	97-201	17	97	27208	8.57
93-232	11	93	1225	19.39	93-33	10	93	27727	12.97
96-224	19	96	1228	33.14	97-34	10	97	27796	12.13
97-46	9	97	1241	8.96	96-06	11	96	28471	8.12
93-91	2	93	1254	15.06	97-32	10	97	28694	8.63
93-21	2	93	1280	17.77	96-63	2	96	28806	7.69
93-104	6	93	1313	21.48	93-212	14	93	28821	8.07
94-47	9	94	1401	15.74	96-34	5	96	28839	9.47
94-202	14	94	1539	21.46	95-31	2	95	29034	8.74
97-51	6	97	1615	13.82	94-24	5	94	29176	10.56
95-70	6	95	1629	13.13	93-214	17	93	29658	9.79
93-102	6	93	1714	13.73	93-44	6	93	29684	12.12
96-43	8	96	1721	15.76	93-15	5	93	29725	10.94
93-74	8	93	1729	11.91	96-77	2	96	29929	7.03
97-04	6	97	1743	14.29	97-39	9	97	30773	10.34
96-01	6	96	1743	22.65	96-228	19	96	31004	13.11
93-53	7	93	1833	16.19	96-42	10	96	31361	9.2
95-220	11	95	1834	14.62	97-225	14	97	31542	12
94-68	2	94	1849	11.77	93-210	17	93	31579	10.72
93-83	5	93	1899	12.49	97-209	19	97	32030	11.67
94-51	17	94	1900	19.04	96-49	11	96	32443	9.12
93-101	7	93	1912	15.27	95-16	8	95	32549	12.55
94-69	6	94	1983	27.15	93-213	13	93	32719	8.2
96-03	6	96	1996	11.96	93-18	5	93	32785	10.46
97-223	17	97	2159	19.17	93-79	6	93	32880	11.48
96-05	6	96	2326	11.99	97-50	6	97	33306	13.19
94-59	6	94	2326	20.85	95-65	11	95	33503	10.96
96-21	9	96	2411	15.28	97-206	18	97	33841	9.67
97-29	6	97	2441	15.84	93-23	6	93	34284	12.79
95-510	17	95	2447	16.42	96-59	9	96	34667	9.17
95-68	1	95	2449	18.44	97-44	1	97	34960	12.55
95-46	6	95	2535	13.45	93-105	4	93	35085	11.33
97-08	5	97	2614	11.07	97-205	18	97	35358	10.39
94-60	10	94	2667	17.99	94-21	8	94	35367	13.24
96-50	6	96	2803	15.74	95-210	19	95	35754	6.89
93-97	6	93	2812	15.9	97-40	9	97	36249	9.61
97-200	14	97	2824	11.43	94-75	9	94	36461	9.66

Table D.2 Prices Granular A (Page 2 of 3)

CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE	CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE
94-16	5	94	2863	13.53	93-56	6	93	36834	8.96
93-50	4	93	2868	13.35	96-222	17	96	37289	9.18
93-69	4	93	2919	10.63	93-25	8	93	37476	11.16
93-05	2	93	3109	15.24	93-19	5	93	38182	8.98
97-27	6	97	3109	13.74	96-39	17	96	38639	10.73
95-42	6	95	3134	14.42	96-240	11	96	38983	8.7
96-506	17	96	3157	17.23	94-216	19	94	39415	13.67
96-75	6	96	3188	15.28	94-29	10	94	41292	10.63
95-63	6	95	3267	14.02	93-222	17	93	41539	12.94
93-22	5	93	3281	11.81	95-17	8	95	41675	10.5
97-14	5	97	3331	12.04	93-43	4	93	41706	13.2
96-80	6	96	3407	19.18	96-220	19	96	41823	11.51
97-506	11	97	3413	12.63	94-62	9	94	42239	11.21
96-211	14	96	3458	15.22	96-218	18	96	42709	10.69
95-27	10	95	3524	11.41	93-04	1	93	42949	16.81
93-31	9	93	3586	8.94	94-58	1	94	43077	15.96
93-37	3	93	3939	10.86	96-17	8	96	43233	9.5
97-507	17	97	3971	15.03	95-213	18	95	43378	10.34
96-205	19	96	4011	15.4	96-209	18	96	43483	7.81
93-98	6	93	4039	14.26	93-233	10	93	43725	8.88
93-32	8	93	4143	14.42	96-231	18	96	43945	9.99
96-22	6	96	4379	12.96	93-17	5	93	44686	7.35
96-72	1	96	4644	14.97	95-212	11	95	45113	11.26
93-218	14	93	4687	11.32	94-08	2	94	45463	10.59
93-64	4	93	4688	11.1	97-30	6	97	45888	10.72
93-75	9	93	4927	10.98	94-61	6	94	46553	9.43
96-56	6	96	4938	14.39	93-31	9	93	47094	8.94
93-04	1	93	4956	17.09	96-66	6	96	47293	8.79
95-13	6	95	4978	13.03	96-48	2	96	47457	7.94
94-350	17	94	5005	14.16	93-200	17	93	47468	8.46
96-505	17	96	5020	17.88	94-27	10	94	48272	9.8
93-67	6	93	5058	13.3	96-238	14	96	48300	8.13
94-25	9	94	5294	11.07	94-66	6	94	48502	11.31
94-71	2	94	5323	12.03	94-15	5	94	49675	9.98
93-401	6	93	5357	13.55	93-86	6	93	51335	12.14
96-11	5	96	5418	12.65	93-07	3	93	51608	8.46
96-239	19	96	5501	13.12	93-206	20	93	52180	7.31
94-18	5	94	5573	11.51	93-45	6	93	52534	10.84
93-28	9	93	5617	15.34	94-06	2	94	52800	8.7
93-305	1	93	5642	16.56	96-206	19	96	53035	10.99
93-108	9	93	5688	13.41	96-29	6	96	53346	10.06
95-29	2	95	5787	12.12	97-07	11	97	53555	7.86
93-204	17	93	6016	10.24	94-234	19	94	54683	8.86
95-506	14	95	6042	11.7	96-236	19	96	55654	9.07
95-23	1	95	6052	21.05	96-219	19	96	56160	8.47
95-61	6	95	6079	12.83	94-22	9	94	56581	9.84
95-501	18	95	6094	19.75	93-09	3	93	57055	6.65
95-36	6	95	6291	13.7	93-59	7	93	57128	9.13
97-47	1	97	6368	17.98	96-12	6	96	57607	11.26
97-11	6	97	6399	10.64	94-227	14	94	58102	11.21
97-38	8	97	6669	10.13	95-02	11	95	58692	11.01
94-03	1	94	6677	14.1	94-09	5	94	58956	8.39
96-59	9	96	6766	9.17	96-239	19	96	58985	13.12
96-235	11	96	6954	13.36	93-231	17	93	59537	9.9
96-17	8	96	6980	9.5	95-215	17	95	59961	7.64
96-70	6	96	6999	9.67	95-48	8	95	59991	11.15
97-12	2	97	7220	13.17	93-60	3	93	60151	8.35
96-36	6	96	7350	14.15	93-203	17	93	63139	9.96
93-88	6	93	7401	12.58	94-208	19	94	65209	8.42
93-24	6	93	7580	13.31	96-225	14	96	66883	8.65
95-47	6	95	7584	17.41	97-213	17	97	67686	10.51
95-09	2	95	7934	10.29	95-219	17	95	69150	10.54
93-93	6	93	7975	13.7	96-30	9	96	70865	10.03
94-02	1	94	8009	18.65	94-55	6	94	72062	10.34
96-239	19	96	8156	13.12	93-106	4	93	73082	10.25
94-31	8	94	8163	15.59	95-32	9	95	73443	9.76
96-58	6	96	8468	12.57	97-19	9	97	73801	7.48
96-69	6	96	8640	11.72	94-50	9	94	79225	9.52
93-16	5	93	9074	8.47	93-62	9	93	80159	9.14
96-67	9	96	9119	13.11	96-212	11	96	80642	10.65
96-40	6	96	9205	14.14	93-76	4	93	80644	11.99
96-207	19	96	9280	17.57	95-38	8	95	80762	9.03

CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE	CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE
95-14	2	95	9365	9.12	94-19	11	94	80849	9.6
93-30	5	93	9632	10.61	96-230	19	96	81845	9.51
94-220	11	94	9691	12.28	93-02	11	93	82312	10.07
93-63	5	93	9865	11.48	93-202	18	93	82882	8.16
94-401	2	94	9875	9.8	96-229	18	96	83000	10.14
97-03	2	97	9924	11.56	93-38	8	93	83520	8.96
93-03	1	93	10062	14.77	93-219	19	93	83532	14.47
94-213	19	94	10126	14.5	94-11	2	94	83725	7.36
96-507	17	96	10171	16.22	94-50	9	94	83935	9.52
94-17	2	94	10213	12.37	93-48	2	93	84933	7.85
97-217	17	97	10463	11.3	93-227	19	93	85070	8.76
96-35	11	96	10570	11.67	95-203	17	95	85606	7.5
93-55	5	93	10685	11.69	93-72	6	93	85708	12.24
96-25	9	96	10814	11.47	96-200	17	96	86014	9.49
93-41	3	93	10922	12.62	97-204	11	97	86628	8.86
96-65	2	96	11199	8.46	93-209	19	93	86757	8.83
96-237	18	96	11209	12.47	93-99	4	93	86758	9.94
93-352	14	93	11224	17.77	95-03	10	95	87995	9.77
93-58	11	93	11394	15.87	93-207	20	93	88321	13.32
93-14	3	93	11489	9.07	94-04	2	94	88452	7.32
95-209	19	95	11712	9.16	95-217	19	95	88867	9.2
97-41	10	97	11934	13.48	97-220	19	97	90860	8.57
94-400	8	94	12453	12	96-32	2	96	92206	8.68
93-400	1	93	12553	15.36	94-219	11	94	93036	12.67
96-213	19	96	12627	17.48	96-02	6	96	93625	11.23
97-42	2	97	12819	12.32	95-208	19	95	95348	7.24
96-47	2	96	12994	8.75	94-80	6	94	97380	10.15
97-218	14	97	13018	12.18	94-218	19	94	97810	11.83
97-02	2	97	13138	8.62	94-78	8	94	98137	9.16
94-82	6	94	13326	11.23	97-214	14	97	99980	8.23
93-11	2	93	13361	10.79	93-221	14	93	100093	9.46
93-100	6	93	13511	11.4	96-38	6	96	100654	6.96
96-215	18	96	13873	13.08	95-201	19	95	109238	8.31
94-450	17	94	14252	16.01	94-53	6	94	110982	11.45
93-20	5	93	14526	6.9	97-01	11	97	111125	10.2
94-28	9	94	14756	11.34	93-01	11	93	114151	11.26
93-98	6	93	14887	14.26	94-12	2	94	119756	7.98
96-18	9	96	15006	7.52	95-214	17	95	119867	8.45
95-229	11	95	15008	13.69	94-212	19	94	121050	10.67
94-26	9	94	15044	11.04	93-66	9	93	121676	7.25
93-29	9	93	15052	9.92	97-203	17	97	122552	8.75
97-33	6	97	15136	12.05	96-203	14	96	123087	12.32
94-222	19	94	15153	8.52	93-51	3	93	124369	6.59
93-215	20	93	15339	13.22	93-205	19	93	125123	9.15
93-10	3	93	15557	9.43	95-24	2	95	127215	10.01
94-207	14	94	15710	11.21	96-09	2	96	127830	6.98
96-13	8	96	15770	9.6	96-08	11	96	128717	7.96
96-37	2	96	16658	7.62	94-217	17	94	131983	7.61
93-06	2	93	16830	9.99	93-223	19	93	136004	10.02
95-15	9	95	17010	13.21	96-214	19	96	141103	7.08
94-57	6	94	17223	12.87	94-203	14	94	149792	8.04
96-28	5	96	17276	8.75	93-201	11	93	156640	8.55
97-21	2	97	17436	9.21	93-225	18	93	158382	7.38
96-14	2	96	17528	8.24	93-27	10	93	159384	8.59
96-41	1	96	17683	17.13	93-71	4	93	163434	10.69
93-350	17	93	17818	12.76	96-20	6	96	172564	7.15
95-39	6	95	18039	12.48	96-38	6	96	172750	6.96
97-202	17	97	18797	9.08	94-07	2	94	175759	8.21
94-76	1	94	19042	17.63	94-32	11	94	178854	8.03
93-450	14	93	19068	8.11	94-05	2	94	195222	7.84
96-204	14	96	19096	11.08	94-221	17	94	202710	11.52
93-211	20	93	19109	10.63	95-34	6	95	223900	14.32
95-202	11	95	19167	12.86	93-85	4	93	226477	10.08
93-13	3	93	19788	10.27	93-89	4	93	231105	10.78
97-52	6	97	20236	10.3	94-84	4	94	233027	10.23
97-43	1	97	20240	16.71	94-83	6	94	351639	12.74
97-45	1	97	20366	17.56	95-45	6	95	491799	11.42
94-30	2	94	20404	12.79					

CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE	CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE
94-15	5	94	238	9.42	94-50	9	94	20672	5.74
93-08	2	93	268	16.2	93-24	6	93	20938	8.12
93-41	3	93	308	9.5	94-401	2	94	20960	6.5
93-10	3	93	449	8.11	94-82	6	94	21218	8.38
93-352	14	93	496	10.91	96-37	2	96	21613	5.86
93-98	6	93	701	13.55	93-35	2	93	22302	7.12
96-73	2	96	759	7.54	93-43	4	93	22714	10.24
96-239	19	96	823	8.81	93-66	9	93	23195	5.2
96-28	5	96	924	8.55	93-93	6	93	23201	10.04
94-67	6	94	1404	16.47	93-400	1	93	23292	10.91
93-21	2	93	1480	10.92	93-17	5	93	23997	5.39
93-53	7	93	1535	9.48	94-220	11	94	24221	7.79
93-34	3	93	1547	11.18	94-213	19	94	24229	8.95
94-51	17	94	1703	11.93	93-210	17	93	25180	6.93
94-68	2	94	1706	9.13	94-29	10	94	25754	6.41
97-38	8	97	1778	8.83	95-506	14	95	26322	6.24
97-209	19	97	1800	7.84	94-450	17	94	26642	9.22
93-03	1	93	1903	15.23	93-100	6	93	26732	8.66
95-23	1	95	1945	16.24	97-21	2	97	27123	6.54
96-43	8	96	1980	14.43	96-35	11	96	27337	6.52
95-14	2	95	1985	7.56	93-27	10	93	28034	7.02
96-239	19	96	2088	8.81	93-233	10	93	28066	4.72
96-47	2	96	2140	7.63	96-203	14	96	28340	7.85
95-66	5	95	2158	9.57	94-44	2	94	29337	7.55
97-217	17	97	2253	6.69	94-222	19	94	29512	5.73
94-22	9	94	2393	8.9	93-11	2	93	30059	5.85
93-45	6	93	2488	12.85	97-34	10	97	30811	6.43
97-51	6	97	2689	9.17	97-52	6	97	31164	7.26
97-40	9	97	2843	7.56	97-28	6	97	31735	7.72
93-39	6	93	3012	9.34	96-18	9	96	32423	4.2
93-98	6	93	3094	13.21	93-221	14	93	32550	7.22
95-10	9	95	3094	8.59	94-76	1	94	33603	12.72
93-91	2	93	3178	9.93	96-38	6	96	33930	5.27
96-231	18	96	3195	6.97	93-204	17	93	34313	5.12
93-36	8	93	3531	9.38	96-06	11	96	34894	5.54
93-83	5	93	3603	9.92	93-209	19	93	35162	6.82
97-200	14	97	3754	6.15	94-27	10	94	36168	6.53
93-22	5	93	3794	8.41	96-204	14	96	36985	7.49
93-97	6	93	3849	11.39	93-213	13	93	37289	5.15
93-108	9	93	3907	12.89	93-86	6	93	38191	8.96
97-223	17	97	4039	14	96-206	19	96	38294	6.83
96-12	6	96	4104	12.03	93-450	14	93	38989	4.45
97-14	5	97	4132	8.24	93-23	6	93	39823	10.04
95-501	18	95	4674	11.05	95-02	11	95	42042	6.7
94-31	8	94	4802	11.27	96-57	6	96	43488	6.67
94-18	5	94	4839	8.11	97-07	11	97	44377	4.38
93-33	10	93	4930	9.58	94-13	2	94	44920	5.92
93-20	5	93	5027	5.43	96-219	19	96	45652	4.67
94-08	2	94	5058	9.95	95-48	8	95	46074	8.89
95-29	2	95	5330	9.71	95-217	19	95	46750	5.26
95-220	11	95	5479	9.12	96-222	17	96	47463	6.46
96-17	8	96	5528	8.4	93-59	7	93	49296	5.6
93-64	4	93	5558	7.75	93-52	6	93	49456	8.67
96-215	18	96	5559	8.43	97-11	6	97	49523	8.07
93-05	2	93	5605	8.71	97-45	1	97	51860	11.95
97-220	19	97	5625	5.65	94-30	2	94	51943	9.71

Table D.3 Prices Granular B (Page 2 of 3)

CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE	CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE
96-27	1	96	5652	11.07	96-17	8	96	52700	8.4
93-18	5	93	5738	6.93	94-218	19	94	52933	6.28
96-22	6	96	5775	8.76	96-49	11	96	53384	4.63
93-211	20	93	5903	6.47	94-224	18	94	53768	6.9
95-201	19	95	6004	5.73	94-212	19	94	54711	5.9
93-37	3	93	6063	7.99	93-223	19	93	55418	5.76
95-210	19	95	6108	4.67	93-206	20	93	57424	4.02
93-31	9	93	6483	6.22	96-59	9	96	57473	4.98
96-17	8	96	6571	8.4	97-01	11	97	58189	6.05
93-55	5	93	6767	9.59	93-25	8	93	61406	8.97
95-212	11	95	6796	7.73	96-240	11	96	63860	5.63
94-14	5	94	6876	9.11	97-204	11	97	65359	6.29
96-41	1	96	6900	11.12	93-56	6	93	66095	6.78
96-229	18	96	7011	7.41	96-29	6	96	66139	7.71
97-206	18	97	7130	7.62	95-38	8	95	68109	6.56
94-16	5	94	7298	8.3	95-31	2	95	69450	6.13
93-14	3	93	7403	5.97	94-207	14	94	69460	5.48
96-67	9	96	7547	6.16	94-61	6	94	69911	7.78
94-26	9	94	7548	6.56	94-09	5	94	72765	5.71
97-50	6	97	7803	8.42	96-238	14	96	75199	5.01
93-222	17	93	7831	7.79	96-220	19	96	75473	7.07
93-88	6	93	7855	8.56	94-208	19	94	76576	4.88
93-700	19	93	8094	8.36	94-219	11	94	79468	7.74
93-75	9	93	8295	6.96	94-234	19	94	81604	5.17
93-30	5	93	8600	7.59	93-38	8	93	82410	5.64
97-225	14	97	8919	8.53	93-202	18	93	83267	5.7
96-11	5	96	9125	6.67	93-48	2	93	84493	6.26
94-213	19	94	9302	8.95	94-80	6	94	86525	7.17
97-202	17	97	9363	4.86	94-78	8	94	88553	7.97
96-59	9	96	9483	4.98	93-203	17	93	91892	4.66
96-213	19	96	9562	9.89	93-227	19	93	92885	5.7
96-211	14	96	9597	10.16	93-44	6	93	94482	9.15
94-56	11	94	9965	6.65	93-02	11	93	95704	5.16
95-13	6	95	10835	8.84	93-01	11	93	100853	6.07
95-17	8	95	10835	9.71	96-236	19	96	101262	5.27
94-03	1	94	10842	11.21	95-203	17	95	103102	4.76
95-32	9	95	10847	8.89	95-215	17	95	103904	4.25
97-13	6	97	10992	7.63	97-19	9	97	105920	4.44
97-12	2	97	11526	9.85	95-208	19	95	115666	4.47
94-17	2	94	11609	9.26	93-212	14	93	118113	5.31
94-19	11	94	11671	7.6	93-07	3	93	119911	6.09
93-67	6	93	11676	9.53	97-214	14	97	120561	5
94-75	9	94	11710	6.58	94-203	14	94	126440	4.71
94-36	6	94	11918	10.97	96-32	2	96	129708	5.9
95-61	6	95	12014	7.88	93-72	6	93	131420	9.29
96-225	14	96	12240	6.34	97-32	10	97	133351	4.58
97-47	1	97	12400	12.77	94-06	2	94	133763	6.42
93-401	6	93	12673	9.19	96-38	6	96	136415	5.27
93-42	9	93	12860	8.82	93-225	18	93	136956	4.09
97-201	17	97	13340	4.8	93-15	5	93	137701	6.3
97-33	6	97	13362	9.51	94-66	6	94	137747	9.06
93-207	20	93	13414	8.48	96-209	18	96	138526	4.39
93-218	14	93	13433	7.05	93-09	3	93	144270	4.61
97-11	6	97	13448	8.07	93-231	17	93	154280	4.24
97-205	18	97	13466	7.16	94-04	2	94	156896	6.16
93-16	5	93	13487	5.56	95-03	10	95	158314	6.02

Table D.3 Prices Granular B (Page 3 of 3)

CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE	CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE
93-29	9	93	13578	6.05	96-02	6	96	169227	7.26
93-102	6	93	13646	11.62	93-60	3	93	169763	5.43
94-36	6	94	14252	5.76	93-51	3	93	176159	4.86
96-07	9	96	14633	5.51	94-217	17	94	195229	4.4
96-214	19	96	14933	4.95	93-205	19	93	196526	6.74
96-42	10	96	15703	6.53	94-227	14	94	199839	6.72
95-09	2	95	15836	8.44	93-201	11	93	201347	4.88
93-215	20	93	16415	6	94-07	2	94	213718	5.99
94-28	9	94	16516	8.83	95-24	2	95	233543	4.9
94-35	1	94	16553	12.67	96-59	9	96	238101	4.98
97-08	5	97	16675	7.35	94-12	2	94	240619	4.84
94-55	6	94	18301	6.01	96-20	6	96	243716	5.56
97-02	2	97	18302	7.15	96-38	6	96	261401	5.27
96-77	2	96	18565	4.89	96-09	2	96	321264	5.58
93-214	17	93	18890	6.13	95-214	17	95	408706	4.84
93-63	5	93	18909	6.97	95-34	6	95	423215	9.57
96-218	18	96	19394	5.79	94-05	2	94	525099	4.54

CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE
93-98	6	93	14	150.93
93-08	2	93	32	214.72
97-14	5	97	192	83.8
94-70	9	94	206	212.48
96-37	2	96	241	67.67
93-107	6	93	285	76.29
95-23	1	95	297	103.46
93-74	8	93	322	86.38
93-34	3	93	356	82.97
93-94	6	93	386	82.57
94-68	2	94	470	88.8
97-17	6	97	477	80.95
96-57	6	96	479	63.41
95-34	6	95	486	64.14
96-78	9	96	514	82.38
94-72	2	94	556	100.99
93-68	1	93	646	112.58
94-79	6	94	757	69.12
96-36	6	96	868	61.27
93-83	5	93	980	57.41
95-39	6	95	1073	67.99
93-37	3	93	1129	63.94
97-08	5	97	1148	59.29
95-14	2	95	1193	60.18
97-12	2	97	1220	97.67
94-16	5	94	1278	53.53
95-61	6	95	1634	58.75
93-63	5	93	1685	64.15
93-44	6	93	1748	53.14
93-55	5	93	1911	63.67
97-47	1	97	1924	58.77
94-400	8	94	2194	63.99
93-306	2	93	2610	67.5
94-76	1	94	3377	68.42
95-31	2	95	4055	59.5
95-24	2	95	4733	53.79
97-05	1	97	4985	54.21
94-17	2	94	5226	70.18
93-11	2	93	5376	58.76
96-41	1	96	6096	53.39
93-307	2	93	6235	76.08
95-17	8	95	6401	62.37
97-31	6	97	6493	50.58
93-15	5	93	6528	52.29
96-09	2	96	7675	49.3
95-15	9	95	8316	48.51
97-49	6	97	8331	45.13
96-70	6	96	8393	49.95
94-78	8	94	8536	51.78
93-99	4	93	8751	60.57
93-73	9	93	9014	40.26
97-50	6	97	9411	49.05
94-12	2	94	11747	52.88
94-35	1	94	13258	57.5
93-60	3	93	13709	51.47
93-18	5	93	14109	44.15
93-81	9	93	14316	46.13
97-20	10	97	15111	41.92
97-32	10	97	15432	40.62
94-75	9	94	15931	49.39
94-14	5	94	16260	48.05
94-09	5	94	17612	47.42
94-24	5	94	18348	45.49
95-27	10	95	18906	45.09
96-31	5	96	25151	41.62
93-108	9	93	25582	46.9
93-29	9	93	28396	44.29
93-30	5	93	36575	46.46
96-16	9	96	45895	40.88

Table D.5 Prices HL3 (Page 1 of 1)

CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE
97-27	6	97	4	446.46
94-67	6	94	8	126.47
97-26	6	97	11	245.37
94-36	6	94	48	71.9
93-46	3	93	54	104.78
94-39	6	94	62	82.67
96-02	6	96	64	48.86
93-97	6	93	71	86.17
96-80	6	96	89	53.1
94-48	8	94	91	214.11
96-12	6	96	100	61.75
93-90	9	93	104	85.38
93-98	6	93	105	108.87
93-93	6	93	106	73.43
93-100	6	93	110	44.53
97-51	6	97	112	49.27
97-11	6	97	116	41.51
93-44	6	93	121	40.53
93-24	6	93	126	73.52
96-75	6	96	154	143.48
93-45	6	93	160	49.61
94-80	6	94	206	59.4
93-50	4	93	209	61.75
94-201	17	94	212	151.02
93-85	4	93	344	56.37
93-52	6	93	346	59.33
96-37	2	96	368	50.56
95-34	6	95	392	50.4
94-53	6	94	415	57.21
93-102	6	93	419	49.77
94-202	14	94	445	255.08
93-71	4	93	486	54.71
93-69	4	93	495	50.46
93-70	4	93	498	69.3
93-72	6	93	504	56.56
94-47	9	94	537	84.16
96-38	6	96	570	38.93
96-211	14	96	584	113.23
94-66	6	94	620	46.54
96-01	6	96	777	56.67
93-99	4	93	817	53.99
96-29	6	96	841	58.94
94-50	9	94	869	40.67
94-25	9	94	926	49.12
93-218	14	93	956	100.45
97-223	17	97	988	98.13
93-76	4	93	991	47.31
95-31	2	95	1083	44.73
94-38	6	94	1097	56.34
97-11	6	97	1119	41.51
93-21	2	93	1123	54.6
94-79	6	94	1129	59.77
93-86	6	93	1185	50.23
94-61	6	94	1304	49.73
93-401	6	93	1324	43.17
96-38	6	96	1326	38.93
96-54	6	96	1374	40.34
93-43	4	93	1498	50.51
94-55	6	94	1624	54.54
93-56	6	93	1691	52.69
97-02	2	97	1877	40.56
93-23	6	93	1941	48.4
94-62	9	94	2043	42.88
94-22	9	94	2438	43.26
94-83	6	94	2461	54.49
97-217	17	97	2473	67.19
94-50	9	94	2660	40.67
93-66	9	93	2902	32.36
95-45	6	95	2937	47.83
94-82	6	94	3040	44.72
93-05	2	93	3810	52.91
93-89	4	93	4736	49.33
93-64	4	93	5761	43.85
97-502	17	97	5967	55.16
96-48	2	96	7853	42.45
95-32	9	95	8077	40.27
93-20	5	93	8183	36.44
96-502	17	96	8947	75.87
96-47	2	96	9530	37.36
93-10	3	93	10001	38.64
93-106	4	93	10611	45.15
93-48	2	93	11985	38.68
93-38	8	93	12742	44.08
94-08	2	94	15056	44.79
97-40	9	97	19786	35.22

CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE	CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE
95-67	1	95	6	384.07	94-17	2	94	6709	56.51
94-15	5	94	124	35.4	93-11	2	93	6970	44.88
94-233	18	94	125	198.83	96-59	9	96	7336	41.47
94-210	19	94	138	138.44	93-55	5	93	7417	55.32
97-211	19	97	141	219.75	95-501	18	95	7453	76.25
93-20	5	93	158	60.98	93-305	1	93	7750	43.12
97-223	17	97	161	108.63	93-62	9	93	8002	36.28
93-75	9	93	163	100.62	96-33	11	96	8049	43.09
93-229	18	93	183	162.21	93-400	1	93	8311	47.69
93-101	7	93	203	110.82	93-14	3	93	8319	40.27
96-350	19	96	230	168.81	93-215	20	93	8339	63.84
93-233	10	93	267	106.26	97-19	9	97	8569	37.49
95-206	18	95	299	124.39	94-27	10	94	8652	50.64
97-210	19	97	299	181.72	94-44	2	94	8780	41.27
97-215	14	97	341	173.25	93-78	11	93	8956	42.06
96-226	19	96	349	121.19	96-16	9	96	8963	48.61
96-241	17	96	358	102.2	96-206	19	96	9234	68.97
93-228	19	93	361	241.66	93-38	8	93	9237	38.59
93-63	1	93	377	81.65	93-212	14	93	9388	57.17
95-66	5	95	392	91.81	94-401	2	94	9602	45.34
93-34	3	93	397	65.17	97-44	1	97	9812	48.17
94-13	2	94	410	63.78	94-30	2	94	10218	44.45
96-73	2	96	413	56.71	94-12	2	94	10456	39.55
94-68	2	94	417	61.39	93-10	3	93	10582	36.82
93-205	19	93	493	63.87	96-49	11	96	10624	40.86
94-215	19	94	510	101.23	96-77	2	96	10888	36.31
93-63	5	93	617	59.65	93-58	11	93	11131	61.15
96-208	19	96	660	86.56	94-208	19	94	11579	64.99
94-60	10	94	737	96.67	95-10	9	95	12672	38.49
93-91	2	93	779	62.01	96-32	2	96	13255	35.36
96-43	8	96	793	75.64	96-215	18	96	13263	59.9
97-212	19	97	802	113.97	96-25	9	96	13289	44.45
93-224	18	93	848	193.08	97-21	2	97	13392	46.56
96-224	19	96	905	99.45	97-05	1	97	13433	43.7
93-83	5	93	978	48.99	93-352	14	93	14309	64.9
93-41	3	93	990	50.55	96-28	5	96	14535	35.63
95-68	1	95	1035	53.35	93-02	11	93	14609	47.84
93-31	9	93	1100	40.89	95-65	11	95	14662	44.91
93-37	3	93	1226	45.25	93-231	17	93	15031	43.81
96-34	5	96	1295	51.27	93-202	18	93	15139	55.02
96-78	9	96	1334	57.23	97-504	17	97	15560	48.51
96-59	9	96	1338	41.47	96-42	10	96	15862	50.15
97-217	17	97	1381	66.41	95-24	2	95	16145	41.6
96-37	2	96	1455	43.74	94-14	5	94	16594	40.71
95-29	2	95	1492	62.17	93-27	10	93	16884	39.13
93-53	7	93	1495	66	93-48	2	93	17273	36.82
93-04	1	93	1622	59.27	95-03	10	95	17297	41.26
94-04	2	94	1680	49.71	96-09	2	96	17510	37.28
95-506	14	95	1695	79.29	93-59	7	93	17511	40.36
95-218	18	95	1759	179.33	95-210	19	95	17745	44.46
96-72	1	96	1767	70.24	94-35	1	94	17818	49.38
94-228	18	94	1809	83.97	93-25	8	93	17963	42.53
95-23	1	95	2012	72.78	93-700	19	93	17998	61.8
94-01	1	94	2035	61.42	96-200	17	96	18052	38.71
94-51	17	94	2049	81.05	95-219	17	95	18176	42.46
94-58	1	94	2081	67.16	94-217	17	94	18245	41.48
94-28	9	94	2131	62.93	93-57	9	93	18474	38.76
93-204	17	93	2155	66.33	94-213	19	94	19196	57.24
93-22	5	93	2160	52.66	93-17	5	93	19995	34.78
97-08	5	97	2454	52.88	93-06	2	93	20012	39.92
93-66	9	93	2473	32.69	96-233	19	96	21094	48.12
93-35	2	93	2516	44.08	94-213	19	94	21916	57.24
93-04	1	93	2526	58.83	96-203	14	96	22597	45.9
95-212	11	95	2588	58.62	94-216	19	94	23212	43.29
94-71	2	94	2634	56.04	97-07	11	97	23460	44.2
97-12	2	97	2652	86.67	93-09	3	93	23508	40.38
95-215	17	95	2690	41.55	93-13	3	93	23731	35.01
96-41	1	96	2700	46.33	94-19	11	94	24682	44.25
97-02	2	97	2737	38.55	94-234	19	94	24969	55.62
97-14	5	97	2807	50.48	93-214	17	93	25274	37.26
95-509	17	95	2823	89.56	93-01	11	93	25431	43.47
94-16	5	94	2872	49.23	93-33	10	93	28566	40.59

Table D.6 Prices HL4 (Page 2 of 2)

CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE
93-15	5	93	3125	50.9
96-18	9	96	3231	41.97
96-213	19	96	3249	69.42
97-43	1	97	3399	44.43
97-47	1	97	3411	47.2
93-213	13	93	3443	56.62
96-209	18	96	3469	64.94
95-14	2	95	3476	42.07
94-03	1	94	3557	51.49
93-506	17	93	3722	85.99
96-07	9	96	3920	36.08
94-76	1	94	3972	57.75
94-508	17	94	3982	67.43
96-27	1	96	4020	47.87
94-207	14	94	4254	71.26
95-09	2	95	4366	58.2
96-67	9	96	4400	44.72
93-31	9	93	4497	40.89
93-16	5	93	4559	50.8
95-510	17	95	4763	60.8
93-28	9	93	4893	50.03
95-31	2	95	4916	40.33
96-06	11	96	5102	43.53
94-26	9	94	5430	47.07
94-18	5	94	5467	59.98
94-21	8	94	5518	49.86
94-450	17	94	5520	87.01
97-45	1	97	5537	66.6
94-400	8	94	5810	53.38
97-507	17	97	6027	52.73
94-222	19	94	6280	66.16
93-450	14	93	6629	74.83
93-207	20	93	28570	44.64
97-204	11	97	29111	41.14
94-219	11	94	30370	46.77
96-230	19	96	30922	43.41
93-42	9	93	31924	37.96
96-212	11	96	31972	38.39
97-01	11	97	32733	41.15
95-209	19	95	32836	40.05
94-10	2	94	34018	33.55
97-218	14	97	34601	44.95
97-214	14	97	35685	44.52
96-231	18	96	35940	40.79
93-221	14	93	36114	42.2
97-225	14	97	36550	54.26
94-29	10	94	39729	38.78
96-39	17	96	39778	42.16
93-200	17	93	40644	50.88
94-15	5	94	42375	35.4
93-219	19	93	43393	60.46
95-38	8	95	45148	39.68
96-240	11	96	46943	37.31
93-19	5	93	51472	33.94
94-11	2	94	52816	34.03
95-203	17	95	53767	37.63
93-210	17	93	55314	49.45
95-214	17	95	56447	40.96
93-201	11	93	62624	42.13
94-07	2	94	67244	34.5
93-203	17	93	75001	38.83
94-203	14	94	80558	40.64
94-05	2	94	82381	39.16

Table D.7 Prices HL8 (Page 1 of 1)

CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE
96-62	6	96	32	119.57
93-97	6	93	121	81.74
96-53	6	96	127	67.1
96-66	6	96	140	52.62
93-98	6	93	176	105.16
97-17	6	97	279	70.39
93-69	4	93	290	45.24
93-70	4	93	453	58.63
93-102	6	93	473	48.99
95-46	6	95	650	52.85
93-74	8	93	651	67.04
93-24	6	93	672	64.99
94-34	6	94	696	89.74
94-66	6	94	754	44.95
93-56	6	93	769	49.01
95-13	6	95	811	45.03
96-54	6	96	1184	38.54
94-38	6	94	1281	50.55
96-36	6	96	1345	48.7
97-11	6	97	1494	35.37
96-57	6	96	1602	43.24
94-25	9	94	1648	47.7
94-82	6	94	1709	39.76
93-105	4	93	1893	49.77
93-76	4	93	2232	43.44
93-100	6	93	2367	37.73
97-26	6	97	2745	40.95
96-80	6	96	2799	59.3
94-55	6	94	2937	50.14
95-39	6	95	2979	45.51
93-401	6	93	3094	40.16
94-57	6	94	3341	43.93
94-79	6	94	3668	51.66
93-43	4	93	4314	46.57
96-38	6	96	5663	32.81
96-02	6	96	5743	38.76
93-86	6	93	5752	40.19
97-27	6	97	6027	43.62
97-31	6	97	6459	36.82
93-44	6	93	6986	38.14
93-50	4	93	7210	39.01
96-70	6	96	8063	37.22
97-46	9	97	8747	32.74
96-38	6	96	9838	32.81
94-53	6	94	10216	41.86
93-71	4	93	10577	42.18
96-48	2	96	10693	39.88
93-79	6	93	11524	37.33
94-62	9	94	12060	41.07
96-20	6	96	12110	32.26
96-38	6	96	12606	32.81
93-45	6	93	12740	39.68
94-61	6	94	12754	37.92
93-85	4	93	13256	42.72
94-80	6	94	13547	43.32
94-22	9	94	16043	39.76
97-49	6	97	16436	38.3
93-23	6	93	16450	40.4
95-06	2	95	16481	32.89
94-83	6	94	17399	44.47
95-34	6	95	18378	41.96
96-12	6	96	18842	33.52
97-30	6	97	19300	34.21
94-84	4	94	23129	45.17
94-50	9	94	24698	34.43
95-45	6	95	29916	41.78
93-51	3	93	30431	33.51
93-72	6	93	30860	38.31
93-07	3	93	40883	40.85
94-50	9	94	40887	34.43
93-66	9	93	43736	29.81
93-89	4	93	47101	42.84
94-06	2	94	52074	39.32

Table D.8 Prices Heavy Duty Binder Course (Page 1 of 1)

CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE	CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE
93-53	7	93	125	86.54	93-107	6	93	14246	57.15
97-15	6	97	148	89.85	97-33	6	97	14406	41.55
94-400	8	94	188	87.74	96-52	6	96	14531	46.68
94-55	6	94	195	60.45	97-52	6	97	14940	37.56
95-42	6	95	211	70.39	94-01	1	94	15172	57.53
96-53	6	96	215	64.26	96-59	9	96	15202	39.24
93-76	4	93	445	52.58	96-66	6	96	15318	41
93-100	6	93	493	43.61	93-89	4	93	15534	44.78
93-94	6	93	523	68.63	97-11	6	97	15977	38.76
97-200	14	97	858	79.54	96-07	9	96	16230	33.81
96-22	6	96	996	56.27	97-30	6	97	16386	38.23
95-47	6	95	1012	61.12	97-13	6	97	16770	40.21
96-21	9	96	1100	98.68	97-51	6	97	17410	41.67
95-62	6	95	1138	59.5	95-57	9	95	17649	40.12
97-16	6	97	1272	54.61	94-227	14	94	19055	63.43
95-13	6	95	1626	50.51	96-59	9	96	19399	39.24
93-69	4	93	1742	43.44	96-56	6	96	19619	39.29
96-37	2	96	2131	49.72	94-80	6	94	19735	39.83
95-17	8	96	2200	40.07	96-19	6	96	19859	38.63
93-88	6	93	2291	48.67	97-45	1	97	20077	71.02
97-39	9	97	2350	53.05	93-85	4	93	20176	42.86
93-67	6	93	2566	44.08	96-69	6	96	20577	41.08
94-57	6	94	2582	50.82	96-27	1	96	20810	44.12
96-38	6	96	2728	34.39	96-40	6	96	21037	39.11
93-97	6	93	2798	51.59	93-50	4	93	21196	40.5
97-26	6	97	3185	41.97	93-62	9	93	21694	39.97
94-30	2	94	3498	56.8	94-56	11	94	22443	42.09
96-05	6	96	3534	38.82	96-38	6	96	23198	34.39
94-34	6	94	3611	60.99	95-02	11	95	23626	47.53
97-21	2	97	3748	51.14	97-38	8	97	23908	36.18
93-104	6	93	3900	48.73	95-215	17	95	24028	40.85
94-69	6	94	3907	67.73	96-29	6	96	24854	43.49
93-86	6	93	3984	46.17	95-41	6	95	24872	48.04
93-44	6	93	4028	43.79	94-79	6	94	24941	54.68
93-23	6	93	4159	47.32	96-14	2	96	25320	37.57
96-13	8	96	4274	54.6	96-65	2	96	26255	37.07
93-43	4	93	4460	49.88	96-12	6	96	27424	37.65
97-04	6	97	4644	41.16	93-79	6	93	27696	40.84
93-105	4	93	4646	47.9	95-06	2	95	27807	37.33
94-82	6	94	4822	45.88	94-83	6	94	28080	48.36
97-03	2	97	5011	52.18	93-56	6	93	28170	44.06
96-11	5	96	5115	49.61	97-43	1	97	28579	53.79
97-301	8	97	5227	41.26	96-58	6	96	28870	38.6
93-77	6	93	5271	60.83	96-80	6	96	29161	48.55
95-70	6	95	5370	46.48	96-17	8	96	29386	40.07
97-42	2	97	5566	58.29	94-50	9	94	29585	38.34
93-66	9	93	6538	37.16	94-66	6	94	30015	39.47
96-57	6	96	6641	43.13	93-04	1	93	30307	54.87
94-53	6	94	6658	44.4	94-58	1	94	30549	42.47
96-35	11	96	6667	39.5	93-28	9	93	31147	44.26
94-22	9	94	6973	48.24	97-19	9	97	31477	36.47
95-16	8	95	7396	54.9	95-45	6	95	32469	44.69
97-46	9	97	7746	35.93	96-20	6	96	34278	34.87
96-17	8	96	7900	40.07	93-31	9	93	34786	37.56
95-36	6	95	8456	49.07	96-38	6	96	35189	34.39
93-24	6	93	8737	41.54	94-27	10	94	35249	44.75
93-213	13	93	9827	56.62	97-29	6	97	38081	42.96
93-45	6	93	9952	44.37	96-02	6	96	38480	41.66
97-27	6	97	10319	42.92	93-72	6	93	38775	43.81
93-52	6	93	10558	42.95	94-221	17	94	39168	41.47
94-44	2	94	10610	48.64	94-39	6	94	40188	46.57
93-71	4	93	10777	42	97-44	1	97	41500	49.44
93-104	6	93	10849	48.73	94-50	9	94	43096	38.34
93-231	17	93	11074	45.94	93-32	8	93	46029	38.01
94-84	4	94	11769	46.53	97-203	17	97	55581	40.92
94-62	9	94	12364	46.47	94-06	2	94	58458	43.33
94-21	8	94	12470	47.18	96-63	2	96	61137	39.19
97-201	17	97	12636	39.84	94-07	2	94	65247	39.56
93-35	2	93	12682	46.12	93-51	3	93	75537	38.11
94-61	6	94	12747	41.23	94-05	2	94	77183	40.78
93-214	17	93	13121	50.1	93-07	3	93	79622	46.97
96-03	6	96	13576	40.25	95-34	6	95	81081	50.05
93-41	3	93	13690	49.44	96-16	9	96	105911	35.07
96-54	6	96	13793	40.16					

Table D.9 Prices Dense Friction Course (Page 1 of 1)

CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE
94-55	6	94	88	80.53
94-400	8	94	105	102.96
96-53	6	96	178	64.82
93-401	6	93	195	55.17
95-36	6	95	318	78.37
94-73	9	94	367	155.4
93-53	7	93	406	106.99
94-47	9	94	476	83.71
93-76	4	93	489	81.65
94-85	6	94	511	125.05
97-16	6	97	678	61.7
97-15	6	97	715	85.7
96-02	6	96	809	56.57
94-69	6	94	844	83.22
95-13	6	95	995	67.22
94-34	6	94	1028	84.39
95-62	6	95	1037	67.49
95-41	6	95	1073	67.97
93-67	6	93	1134	66.77
94-62	9	94	1224	66.09
93-100	6	93	1293	63.71
96-37	2	96	1389	68.32
94-57	6	94	1432	74.98
93-104	6	93	1487	69.79
93-88	6	93	1533	67.2
93-44	6	93	1629	63.3
93-107	6	93	1646	78.91
93-69	4	93	1761	72.63
96-21	9	96	1911	68.58
96-22	6	96	1944	55.15
94-79	6	94	1984	74.27
96-38	6	96	2025	48.71
97-30	6	97	2101	53.98
97-42	2	97	2140	76.02
93-97	6	93	2216	69.62
96-12	6	96	2345	51.86
96-80	6	96	2561	68.31
93-43	4	93	3045	71.29
93-104	6	93	3064	69.79
96-57	6	96	3209	59.76
93-105	4	93	3261	71.81
93-23	6	93	3438	68.34
93-86	6	93	3526	64.68
97-04	6	97	3776	49.71
93-66	9	93	3890	49.75
95-34	6	95	4061	66.64
94-30	2	94	4091	84.83
96-13	8	96	4151	64.11
97-26	6	97	4193	59.65
93-77	6	93	4454	70.87
96-05	6	96	4633	47.82
94-22	9	94	4871	58.09
93-24	6	93	4913	56.52
95-70	6	95	5425	56.12
93-52	6	93	5646	64.48
93-45	6	93	6843	62.6
94-53	6	94	6943	67.34
96-14	2	96	7003	55.85
95-16	8	95	7139	67.02
97-11	6	97	7158	51.06
97-13	6	97	7423	51.98
93-35	2	93	7818	71.92
94-39	6	94	7997	78.6
94-44	2	94	10957	73.44
94-61	6	94	11239	58.29
94-50	9	94	11276	49.35
93-41	3	93	11522	73.66
93-89	4	93	11580	65.9
93-71	4	93	11619	67.65
97-27	6	97	11841	61.19
94-80	6	94	12019	57.6
94-84	4	94	12501	64.65
97-52	6	97	12849	49.97
97-33	6	97	13080	52.01
94-82	6	94	13559	60.25
96-66	6	96	13938	52.95
96-65	2	96	14172	53.13
96-27	1	96	14736	56.57
96-29	6	96	14754	51.89
96-69	6	96	14861	53.77
93-62	9	93	14907	48.54
94-01	1	94	15445	72.23
97-03	2	97	15501	77.18
96-38	6	96	15587	48.71
94-66	6	94	15610	56.78
96-03	6	96	15964	56.33
94-58	1	94	16227	60.61
97-45	1	97	16797	83.93
95-42	6	95	16809	57.11
95-06	2	95	17601	56.47
94-52	9	94	17736	49.35
96-56	6	96	18208	55.15
93-56	6	93	18339	63.11
93-50	4	93	19372	67.6
97-19	9	97	19540	42.71
94-83	6	94	19882	67.74
93-31	9	93	19908	48.22
96-20	6	96	19925	54.74
97-51	6	97	20221	51.82
97-21	2	97	20254	66.84
96-38	6	96	20314	48.71
96-40	6	96	20432	56.75
93-85	4	93	20739	65.53
96-54	6	96	21451	56.42
97-43	1	97	24421	65.67
93-72	6	93	24930	60.49
93-04	1	93	25049	92.29
93-79	6	93	29654	56.18
94-27	10	94	31457	50.82
96-19	6	96	31467	47.43
93-07	3	93	32193	66.96
96-58	6	96	33732	47.93
96-63	2	96	34674	52.54
93-28	9	93	35389	51.02
95-45	6	95	35648	56.13
97-44	1	97	37229	65.07
94-06	2	94	38117	61.13
94-07	2	94	39902	60.5
93-51	3	93	41938	57.59
94-05	2	94	47278	61.62
96-11	5	96	48137	49.91
94-21	8	94	49924	48.89
97-29	6	97	65341	58.24

Table D.10 Prices Asphalt Removal Partial Depth (Page 1 of 2)

CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE	CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE
96-43	8	96	26	32.85	93-71	4	93	14674	1.73
95-66	5	95	59	26.94	93-44	6	93	14685	3.54
95-67	1	95	104	19.78	93-89	4	93	15474	2.88
94-25	9	94	162	26.74	97-26	6	97	15611	0.98
96-62	6	96	164	13.56	93-23	6	93	15651	4.59
97-16	6	97	183	13.12	93-69	4	93	16103	1.75
94-72	2	94	184	10.88	96-38	6	96	16807	1.53
93-91	2	93	217	14.49	93-97	6	93	17333	2.38
93-101	7	93	230	16.92	93-307	2	93	20250	4.49
97-211	19	97	238	15.54	93-306	2	93	20636	3.29
95-23	1	95	254	12.63	96-70	6	96	21282	1.25
93-04	1	93	293	15.65	95-45	6	95	22365	2.89
93-09	3	93	307	10.05	97-04	6	97	22882	1.17
93-53	7	93	334	14.74	93-04	1	93	23257	6.09
94-11	2	94	398	13.93	97-11	6	97	23346	1.45
94-62	9	94	430	13.77	94-58	1	94	23596	7.02
93-200	17	93	515	10.23	96-42	10	96	24386	1.76
97-11	6	97	527	3.54	93-48	2	93	24677	1.72
95-47	6	95	541	10.18	94-29	10	94	25057	1.62
93-66	9	93	560	8.46	93-41	3	93	26203	2.81
96-37	2	96	571	4.32	96-27	1	96	26297	2.79
97-17	6	97	574	7.33	93-52	6	93	28410	1.78
95-62	6	95	635	6.53	94-17	2	94	28863	1.11
95-506	14	95	688	11.18	94-84	4	94	29024	1.03
94-73	9	94	712	10.73	96-69	6	96	29455	2.04
96-30	9	96	712	6.34	94-66	6	94	33864	1.67
94-34	6	94	759	5.89	97-39	9	97	34064	1.43
94-76	1	94	818	5.42	96-05	6	96	36130	0.98
97-12	2	97	849	7	94-79	6	94	38669	1.79
93-78	11	93	882	8.29	93-77	6	93	40501	2.89
93-203	17	93	901	6.48	93-85	4	93	42487	1.9
94-50	9	94	913	4.76	94-80	6	94	43908	2.47
96-67	9	96	929	7.08	96-80	6	96	44301	1.84
93-83	5	93	1157	7.24	93-107	6	93	45111	2.03
94-32	11	94	1178	4.77	95-34	6	95	50324	1.89
94-85	6	94	1184	6.64	93-42	9	93	50469	1.32
95-13	6	95	1251	4.23	93-35	2	93	52146	1.32
93-401	6	93	1268	5.23	94-44	2	94	52285	2.39
93-352	14	93	1278	9.09	93-72	6	93	54041	1.75
97-225	14	97	1450	8.17	93-33	10	93	54600	1.38
94-60	10	94	1507	7.08	96-12	6	96	56791	1.1
97-13	6	97	1519	3.12	97-30	6	97	57898	1.72
93-68	1	93	1672	6.49	97-31	6	97	61465	1.1
94-30	2	94	1749	6.41	97-20	10	97	63854	1.38
93-400	1	93	1754	4.89	93-73	9	93	64437	1.28
94-69	6	94	1772	12.25	93-31	9	93	64932	1.06
93-34	3	93	1794	5.15	96-51	6	96	65102	1.26
96-41	1	96	1902	4.02	94-21	8	94	72335	2.2
93-11	2	93	1912	4.17	95-27	10	95	78141	0.88
93-214	17	93	1917	4.27	96-29	6	96	82602	0.91
96-53	6	96	2026	4.09	96-25	9	96	88133	1.36
95-14	2	95	2046	2.77	97-33	6	97	88358	0.79
93-105	4	93	2092	6.58	97-21	2	97	89068	1.37
95-61	6	95	2365	2.87	97-46	9	97	89776	1.06

Table D.10 Prices Asphalt Removal Partial Depth (Page 2 of 2)

CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE	CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE
94-47	9	94	2493	8.38	96-19	6	96	95457	1.61
93-59	7	93	2928	3.11	97-03	2	97	95839	1.41
93-62	9	93	3535	4.17	97-32	10	97	99244	0.79
96-22	6	96	3746	2.97	94-75	9	94	105093	1.11
96-21	9	96	3950	4.93	93-81	9	93	108891	1.21
93-104	6	93	4016	3	97-52	6	97	111500	1
95-31	2	95	4171	3.39	94-82	6	94	114927	1.39
95-36	6	95	5087	4.6	96-66	6	96	115672	1.4
94-22	9	94	5173	3.33	97-45	1	97	117589	1.83
93-14	3	93	5433	1.98	93-30	5	93	119603	0.98
95-02	11	95	5453	4.08	96-56	6	96	120868	0.99
93-10	3	93	5874	2.66	96-02	6	96	123984	1.14
93-37	3	93	5910	2.8	96-52	6	96	124786	1.62
93-05	2	93	6410	2.29	97-40	9	97	130553	1.02
93-24	6	93	7029	1.99	95-38	8	95	132307	0.71
94-04	2	94	7530	1.82	95-57	9	95	134566	0.84
97-47	1	97	7552	3.78	96-03	6	96	137017	0.74
94-01	1	94	7759	4.38	97-41	10	97	137458	1.02
94-38	6	94	7823	2.82	93-79	6	93	152992	1.28
93-88	6	93	8097	2.16	96-205	19	96	165144	1.31
93-55	5	93	8105	4.79	96-233	19	96	166361	1.17
95-09	2	95	8300	1.64	96-40	6	96	171240	0.96
94-18	5	94	8330	2.53	96-54	6	96	174104	1
94-57	6	94	8340	3.63	93-108	9	93	177013	1.22
97-28	6	97	8437	1.84	97-51	6	97	177656	0.57
93-104	6	93	8534	3.16	96-16	9	96	187734	0.77
93-100	6	93	8859	1.85	96-17	8	96	194983	1.15
97-02	2	97	9014	1.29	96-71	6	96	196761	1.37
96-38	6	96	9299	1.29	93-29	9	93	214603	1.03
94-16	5	94	9319	2.03	94-39	6	94	215548	2.73
93-18	5	93	9490	0.99	94-27	10	94	223605	1.01
97-43	1	97	10479	2.47	95-41	6	95	230295	2.38
97-44	1	97	11023	3.29	97-38	8	97	235946	0.9
96-17	8	96	11229	1.19	96-58	6	96	288418	1.06
95-24	2	95	12581	2.23	93-28	9	93	305029	1.4
97-27	6	97	13058	2.84	93-32	8	93	311269	1.16
97-42	2	97	13407	2.37	96-11	5	96	320724	0.76
95-70	6	95	13616	1.94	96-50	6	96	333110	1.5
96-48	2	96	14604	1.9	97-29	6	97	473693	0.98

Table D.11 Prices Miscellaneous Maintenance Items (Page 1 of 1)

COLD IN-PLACE RECYCLED MIX M2

CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE
94-02	1	94	27687	4.55
97-206	18	97	256874	2.9
96-239	19	96	117391	3.33
97-209	19	97	173435	5.22

ROUT AND SEAL CRACKS IN ASPHALT PAVEMENT M

CONTRACT #	DISTRICT#	YEAR	QUANTITY	AVERAGE PRICE
96-38	6	96	127	9.31
94-69	6	94	137	6.03
96-40	6	96	150	8.29
93-104	6	93	205	5.1
93-79	6	93	370	12.24
94-34	6	94	502	4.63
93-104	6	93	732	5.1
94-79	6	94	1150	7.48
93-56	6	93	14880	1.05
93-351	18	93	107000	1.31
93-507	17	93	213200	0.98
96-501	17	96	236875	0.97
95-504	17	95	250420	0.95
97-501	17	97	312000	1.29

Table D.12 Summary of Distributions For Material Costs (Page 1 of 3)

Material	Quantity (tonnes)	Distribution	χ Squared	Parameter		
				One	Two	Three
Granular A	100 - 1000	Extreme Value	2.56	20.57	8.18	
		Lognormal	2.56	25.33	10.85	
		Logistic	3	24.55	5.77	
		Normal	3	25.25	10.3	
		Gamma	2.56	4.04	5	4.24
	1000 - 10000	Gamma	6.05	7.67	1.95	3.46
		Extreme Value	7.59	12.78	2.74	
		Lognormal	9.36	14.38	3.46	
		Logistic	17.07	14.01	1.97	
		Normal	17.95	14.4	3.77	
	10000 - 100000	Lognormal	11.08	10.78	2.39	
		Extreme Value	11.22	9.67	1.9	
		Gamma	13.38	6.18	1.36	3.4
		Weibull	16.56	6.44	4.89	1.8
		Beta	26.64	13.2	34.25	38.78
	100000+	Lognormal	4.3	9.21	1.83	
		Weibull	3.43	6.3	3.25	1.57
		Beta	3.86	15.68	33.53	28.93
		Extreme Value	6.03	8.34	1.47	
		Triangular	5.16	6.29	15.64	6.96
Granular B	1000 - 10000	Weibull	6.9	4.09	5.52	2.05
		Beta	8.28	9.19	24.87	33.27
		Extreme Value	12.68	7.85	1.99	
		Normal	12.95	8.98	2.49	
		Logistic	13.23	8.77	1.36	
	10000 - 100000	Extreme Value	2.71	6.29	1.48	
		Gamma	5.33	3.68	1.09	3.2
		Weibull	5.74	3.79	3.79	1.82
		Lognormal	7.14	7.16	1.86	
		Triangular	11.98	3.7	13.46	5.54
	100000+	Extreme Value	3.2	5.03	0.84	
		Weibull	4	4.03	1.64	1.17
		Gamma	5.2	4.07	1.11	1.36
		Lognormal	6.8	5.57	1.17	
		Pareto	7.2	4.06	3.4	
HL1	100 - 1000	Logistic	0.667	82.88	16.29	
		Lognormal	1.111	86.91	25.79	
		Extreme Value	2.44	75.67	17.76	
		Pareto	4.22	56.17	2.54	
		Normal	6.44	87.59	34.63	
	1000 - 10000	Lognormal	2.13	58.93	9.86	
		Extreme Value	3.44	54.34	8.22	
		Logistic	3.88	58.18	5.56	
		Gamma	3	32.81	3.88	6.74
		Normal	5.63	58.96	10.64	
	10000+	Extreme Value	0.18	44.48	3.44	
		Logistic	0.17	46.17	2.49	

Table D.12 Summary of Distributions For Material Costs (Page 2 of 3)

Material	Quantity (tonnes)	Distribution	χ Squared	Parameter		
				One	Two	Three
HL1	10000+	Lognormal	0.18	46.48	4.25	
		Normal	1.11	46.49	4.49	
		Pareto	3	40.29	7.2	
HL3	100 - 1000	Weibull	6.03	38.53	29.07	0.8
		Extreme Value	12.51	58.92	20.24	
		Pareto	12.51	38.39	1.93	
		Gamma	13.38	38.92	39.55	0.82
		Lognormal	15.54	70.12	30.05	
	1000+	Lognormal	4.95	46.81	8.3	
		Normal	4.95	46.83	8.85	
		Logistic	5.37	46.04	4.82	
		Beta	5.37	19.13	43.47	153.26
		Gamma	7.05	28.96	4.12	4.33
HL4	100 - 1000	Triangular	4.8	22.04	60.98	277.18
		Gamma	5.2	30.44	37.37	2.11
		Extreme Value	6.8	85.65	38.89	
		Lognormal	7.6	109.42	54.56	
		Normal	9.6	109.35	53.02	
	1000 - 10000	Gamma	6.87	31.55	10.3	2.42
		Extreme Value	9.02	49.43	11.34	
		Lognormal	9.83	56.27	15.17	
		Weibull	13.34	31.82	26.84	1.33
		Logistic	23.04	54.25	8.86	
	10000+	Gamma	10.56	33.08	5.37	1.96
		Extreme Value	13.24	40.39	5.23	
		Lognormal	14.58	43.6	6.95	
		Weibull	14.31	33.05	11.64	1.44
		Logistic	23.71	42.66	4.08	
HL8	100 - 1000	Exponential	15.6	0.01		
		Extreme Value	2.8	55.17	12.3	
		Pareto	1.6	43.39	3.1	
		Logistic	1.2	60.96	10.14	
		Gamma	1.6	44.94	33.68	0.53
	1000 - 10000	Extreme Value	1.57	39.32	5.31	
		Logistic	2.43	41.97	3.71	
		Lognormal	2.43	42.32	6.28	
		Normal	2.43	42.33	6.49	
		Gamma	1.57	28.49	3.14	4.41
	10000+	Triangular	1.83	26.95	46.64	41.96
		Weibull	3.07	22.19	17.91	4.42
		Lognormal	5.55	38.52	4.25	
		Gamma	5.55	14.14	0.74	32.84
		Extreme Value	7.6	40.48	3.41	
HDBC	100 - 1000	Exponential	8.9	0.01		
		Extreme Value	1.27	76.36	13.03	
		Pareto	3.45	41.75	2.08	

Table D.12 Summary of Distributions For Material Costs (Page 3 of 3)

Material	Quantity (tonnes)	Distribution	χ Squared	Parameter		
				One	Two	Three
HDBC	100 - 1000	Logistic	0.18	69.21	9.05	
		Gamma	0.18	0	3.28	21.09
	1000 - 10000	Extreme Value	2.21	45.28	7.15	
		Lognormal	3.36	49.44	9.14	
		Logistic	4.13	48.55	5.14	
		Gamma	3.74	31.98	5.18	3.38
		Weibull	5.66	32.77	18.71	1.65
	10000+	Gamma	4.71	32.8	3.48	2.96
		Extreme Value	6.87	40.44	4.4	
		Weibull	11.45	33.2	11.04	1.61
		Lognormal	14.69	43.07	5.8	
		Logistic	17.38	42.33	3.29	
DFC	1000 - 10000	Extreme Value	6	69.47	8.69	
		Triangular	6.7	43.68	88.81	67.02
		Normal	10.88	64.97	9.18	
		Logistic	11.25	65.19	5.3	
		Beta	10.13	20.1	14.15	110.7
	10000+	Extreme Value	6.33	54.61	7.05	
		Gamma	6.33	39.93	4.39	4.29
		Lognormal	13	58.73	8.83	
		Normal	15.33	58.75	9.37	
		Weibull	15.33	40.05	21.12	2.1
Removal of Asphalt Pavement Partial Depth	100 - 1000	Extreme Value	5.81	8.55	3.88	
		Lognormal	5.81	10.86	5.38	
		Normal	6.71	10.82	5.09	
		Beta	5.35	3.42	13.64	54.01
		Weibull	5.35	2.42	9.42	1.7
	1000 - 10000	Lognormal	2.69	4.11	2.36	
		Gamma	4.77	0.78	1.52	2.18
		Extreme Value	7.54	3.12	1.6	
		Beta	6.85	2.6	13.09	24.75
		Weibull	6.85	0.74	3.73	1.52
	10000 - 100000	Extreme Value	6.85	1.56	0.68	
		Lognormal	7.15	1.98	0.98	
		Gamma	12.08	0.77	0.95	1.3
		Weibull	13.61	0.77	1.27	1.07
		Logistic	20.08	1.82	0.57	
	100000+	Extreme Value	4.89	1.01	0.29	
		Gamma	5.78	0.48	0.22	3.26
		Lognormal	7.56	1.18	0.38	
		Beta	8.44	5.78	21.1	5.51
		Weibull	8.89	0.49	0.78	1.67