THE UNIVERSITY OF ALBERTA

A STUDY OF LOW TEMPERATURE TRANSVERSE CRACKING IN ALBERTA

by

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A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES AND RESEARCH

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE

OF MASTER OF SCIENCE

DEPARTMENT OF CIVIL ENGINEERING

EDMONTON, ALBERTA

FALL 1986

ISBN 0-315-32284-5

THE UNIVERSITY OF ALBERTA

RELEASE FORM

NAME OF AUTHOR: David Paul Palsat

TITLE OF THESIS: A Study of Low Temperature Transverse Cracking in Alberta

DEGREE: Master of Science

YEAR THIS DEGREE GRANTED: Fall 1986

....

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Date: October 10, 1986

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ABSTRACT

Low temperature transverse cracking of asphalt concrete pavements is associated with large areas of Canada and can result in reduced pavement riding quality, increased pavement maintenance costs and premature pavement rehabilitation. In this investigation, the performance of asphalt concrete pavements constructed by Alberta Transportation was studied in an attempt to identify the major factors that are influencing low temperature transverse cracking.

The investigation was separated into two distinct phases. Phase I included field identification of transverse cracking behavior on a large number of projects and the development of a data bank of significant parameters and characteristics. The data was analyzed using multiple linear regression techniques to identify the major factors that were influencing the observed cracking behavior. Phase II was a more detailed investigation into three projects exhibiting peculiar transverse cracking behavior. This phase included a field sampling and laboratory testing program designed to identify in-place materials characteristics that could be correlated with cracking performance.

Resulting from Phase I, the three most significant variables identified that had the greatest influence on the frequency of transverse cracking were pavement thickness, original asphalt stiffness predicted using McLeod's method and based on site specific temperature conditions and pavement age. A critical original asphalt stiffness was identified.

Using the cracking frequency model, a design map was developed for Alberta that can be used in selecting the appropriate asphalt grade and type that would optimize the low temperature performance of the pavement.

Phase II identified that subgrade effects were a large factor in influencing the differences in observed cracking. Asphalt characteristics appeared to have the greatest influence in governing the behavior of the sections that did not exhibit any transverse cracking or exhibited only very low frequencies of transverse cracking.

ACKNOWLEDGEMENTS

I would like to acknowledge the financial support provided by Alberta Transportation during my educational leave to attend University.

This research has been performed as part of a research contract between Alberta Transportation and the Department of Civil Engineering, University of Alberta.

I would like to extend my sincere thanks to:

My wife, Frances, and my family, Bryan and Kevin, for their support, understanding and sacrifices made,

Mr. Chuck McMillan, Special Projects Engineer, for his assistance and support, particularly in setting up the computer data base system,

Professor K.O. Anderson for his guidance and suggestions throughout the course of the work,

Ms. Joyce Kiyooka for typing the manuscript,

Many individuals within Alberta Transportation for their assistance and cooperation.

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CHAPTER 1

INTRODUCTION

1.1 Introduction

Considerable efforts and resources by researchers and highway engineers have been expended to design asphalt concrete pavements that exhibit acceptable performance at both high and low in-service temperatures. The major consideration relating to high in-service temperatures is the plastic deformation of the pavement under traffic loadings resulting in rutting or channelization in wheelpath locations. At low in-service temperatures, the major concern is with non-load associated, thermally induced transverse cracking.

Within Alberta Transportation, asphalt cement grade selection is the only variable considered in designing asphalt concrete mixes and pavement structures to optimize low temperature pavement performance in terms of thermally induced transverse cracking. As witnessed by extensive transverse cracking throughout the pavement system, the ultimate goal of designing crackfree pavements has not been realized.

However, there are some pavements within the Department's network that have been identified as exhibiting little or no transverse cracking. About 600 km of asphalt concrete pavement on about 60 projects constructed between about 1974 and 1979 have been observed to have transverse cracking frequencies between 0 and 5 transverse cracks per kilometre. Of these, 341 km exhibited no transverse cracking.

These low transverse cracking frequencies are significant in terms of their ultimate effects on pavement riding quality, pavement structural integrity and pavement maintenance costs and warrant further research.

1.2 Objectives of the Investigation

The overall purpose of this investigation was to review the performance of asphalt concrete pavements constructed by Alberta Transportation with respect to low temperature transverse cracking. More specifically, the objectives were:

1. To identify the major factors that are influencing the low temperature performance of asphalt concrete pavements.

2. To provide more definitive design guidelines to the highway engineer to minimize or negate low temperature transverse cracking.

3. To assess the impact of asphalt cement specification changes made by Alberta Transportation in 1967 as they relate to low temperature transverse cracking.

4. To assess the impact of asphalt cement specifications made in the late 1970's and the use of "harder" asphalt cements as they relate to the longer term low temperature performance of pavements

designed by Alberta Transportation.

1.3 Scope of the Investigation

The scope of the investigation was limited to asphalt concrete pavements within the primary highway and secondary road network in Alberta. The investigation focused mainly on conditions which are unique to Alberta:

- -climate
- -soils and aggregates
- -asphalt cement sources
- -pavement design methodology and policy.

In general, only newly constructed pavements were included in the investigation. Pavements that were subsequently overlaid were not studied unless it was felt that the observed transverse cracking behavior was governed by the underlying, previously constructed, pavement.

Attempts were made to use the broad historical data base of information that was available. This data included routine test results of asphalt cement supplied to all paving projects and the results of quality control testing carried out during construction on all projects. A pragmatical approach was taken to try to maximize practical implementation of the results of the investigation.

1.4 Organization of the Thesis

As the first step of the investigation, described in Chapter 2, low temperature transverse cracking and its effects on pavement is discussed. Major factors identified in the literature that influence low temperature transverse cracking and Alberta Transportation's methodology for selection of asphalt cement grades for asphalt concrete pavements is reviewed. The development of asphalt cement specifications by Alberta Transportation is briefly presented.

Chapter 3 describes the evolution of the investigation from one project identified in 1981 that exhibited no transverse cracking to a larger scale field inspection program. The research has been separated into two phases. Phase I includes field identification of transverse cracking behavior on a large number of projects, the development of a data bank of significant parameters and characteristics and the analyses of this data using statistical techniques. Phase II focuses on the particular behavior of a limited number of projects.

Chapter 4 describes the development of the data bank used in Phase I. The source and significance of the data is discussed. Chapter 5 presents the analyses of the data. Multiple linear regression analyses are used to develop models which identify the major factors influencing cracking performance.

Chapter 6 presents three case studies. Selected projects were investigated in detail. Field sampling and laboratory testing was carried out to characterize the materials present to attempt to explain the observed field performance. The last chapter is a summary of the principal findings of this investigation.

CHAPTER 2

LOW TEMPERATURE TRANSVERSE CRACKING

2.1 Low Temperature Transverse Cracking

Low temperature transverse cracking can be defined as regular, "across the pavement" cracks, orientated perpendicular to the longitudinal direction of the asphalt concrete pavement. The types of transverse cracking that have been addressed in this investigation are those cracks resulting from low temperature stresses induced in the pavement by the cooling of a longitudinally restrained pavement exceeding the tensile strength of the pavement. The investigation will also consider the effects of low temperature thermal shrinkage of the underlying subgrade materials which may reflect through the pavement surface. Cracking resulting from the shrinkage of granular or cement treated bases or frost heaving will not be addressed.

Transverse cracking, as perceived by highway engineers in Alberta, is a widespread problem and is regarded as a "fact of life" in the performance of asphalt concrete pavements. A survey of District Transportation Engineers carried out by the author in 1983 to identify uncracked pavements identified only two projects with Alberta's network that exhibited no transverse cracking after a minimum 5 years performance. Kathol (1) reported in 1969 that there were only approximately 50 km of some 6000 km of asphalt pavement constructed over granular bases in the Alberta primary highway network that exhibited transverse crack frequencies less than 6 per km. Results of a 1968 survey showed an average crack frequency of 47 per km of some 3160 km counted.

2.2 Why is Transverse Cracking a Highway Engineering Problem?

The initial occurrence of fine transverse cracks may have little substantial effects on pavement performance. With time, as the cracks widen, the intrusion of water into the pavement and subsurface materials can result in significant losses of riding quality. With clay subgrades, especially those with substantial amounts of montmorillonite clay, localized swelling in the vicinity of cracks can result in bumps up to 25 or 50 mm high (2). On sandy subgrades, washing away of fines can cause dips. Spalling of the asphalt concrete at the crack face can further widen the crack which may eventually result in potholes. All of these conditions can seriously reduce pavement riding quality which results in increased costs to the user of the facility and may require premature rehabilitation in the form of an overlay.

Considerable resources are also expended in the maintenance of these cracks by patching and crackfilling. In general, crackfilling is ineffective in providing a permanent seal of the crack and must be repeated on an annual basis. Further, crackfilling results in an unsightly appearance of the road surface and if excessive amounts are applied, construction problems are created when the next asphalt overlay is applied. To complete the process, the existing cracks will reflect through the new overlay, often within one year, requiring continued long term maintenance.

2.3 Factors Effecting Low Temperature Transverse Cracking

There are many factors that affect the low temperature performance of asphalt pavements. The major factors identified in the literature are briefly discussed.

1) Climate

Low temperature transverse cracking is generally associated with large areas of Canada and the more northern regions of the United States where winter temperatures drop below -23°C (3). The lower in-service winter temperatures are, the greater the magnitude of the induced thermal stresses and the greater the incidence of cracking that can be expected. Correlations between the initiation of pavement transverse cracking and the lowest ambient air temperatures have been identified (4) (5).

2) Asphalt Characteristics

The relationship between asphalt consistency, both penetration and viscosity, and asphalt temperature susceptibility on the occurrence of transverse cracking has long been apparent (2) (4) (5) (6). For a given consistency (penetration or viscosity), an increase in the temperature susceptibility will result in a greater incidence of transverse cracking.

3) Asphalt Stiffness

~~Van der~~ Poel (7) introduced the concept of asphalt stiffness as a means of characterizing asphalt consistency over a range of temperatures and loading times. Stiffness, which is a function of loading time and temperature, is defined as:

$$S_{(t,T)} = (\text{stress/strain})_{(t,T)}$$

Asphalt stiffness can be evaluated at the low temperatures at which cracking occurs and can account for both asphalt consistency and and temperature susceptibility.

Correlations have been identified between low temperature asphalt stiffness and pavement transverse cracking (2) (5) (8) (9). Higher asphalt stiffness at low temperatures result in greater incidences of transverse cracking, all other conditions (pavement thickness, age, etc.) being equal.

4) Pavement Structure

The thickness of the asphalt concrete layer has been shown to affect the occurrence and frequency of transverse cracking (5) (8). Generally, the thicker the pavement section, the lower the frequency of cracking. This may result from the insulating properties of the asphalt concrete resulting in a stiffness gradient through the pavement structure.

5) Age

The age of a pavement has an effect on the occurrence and frequency of transverse cracking. Asphalt hardening due to oxidation will increase asphalt stiffness with time. Also the probability of the occurrence of extreme low temperatures will increase with pavement age. These effects will tend to increase the probability of the initiation of transverse cracking or increase the frequency of transverse cracking with pavement age.

6) Traffic

The Ste. Anne Test Road (10) demonstrated that traffic effects influenced the severity of transverse cracking with significantly more transverse cracking experienced in the traffic lane as compared to the passing lane within some sections.

7) Subgrade

The effects of subgrade type on pavement transverse cracking have been recognized (1) (6) (8) (10). Pavements constructed over sand subgrades exhibit higher cracking frequencies than pavements over clay subgrades. The author has observed that the frequency of transverse cracking is often significantly reduced for sections of road constructed through low lying muskegs or sloughs compared to sections constructed through well-drained terrain.

8) Mix Properties

Some mix properties such as density, air voids and asphalt content have an influence on mix stiffness and the rate of asphalt aging or embrittlement. It is recognized that mix properties could therefore influence transverse cracking frequencies. More extensive research is required in this area to correlate these properties with low temperature performance.

2.4 Alberta Transportation's Asphalt Cement Specifications

The influence of asphalt cement properties on low temperature transverse cracking was briefly discussed in the previous section. Asphalt cement specifications used by Alberta Transportation have been developed over the last two decades in an attempt to optimize the low and high temperature performance of asphalt concrete pavements.

Prior to 1967, asphalt cements were graded only by penetration at 25°C. Two grades were in use: 150-200 pen and 200-300 pen. A minimum quantity of SC3000 was also used.

The Alberta Test Road (4), constructed in 1966, used three different sources of 200-300 penetration grade asphalt cement in one contract with uniform subgrade conditions. Each source represented a major supplier and a different viscosity at 60°C - a low, a medium and a high viscosity asphalt. Field performance showed that the "low viscosity" or more temperature susceptible source tended to crack earlier in its service life than the medium or higher viscosity sources.

As a result of the preliminary findings of this test road and other field surveys (2) , a minimum viscosity at 60°C of 275 poise and a minimum penetration at 25°C of 250 dmm was adopted to control both the consistency and temperature susceptibility of the asphalt. This asphalt cement grade was referred to as AC 275 (Figure 2.1).

In 1978, two new asphalt cement designations were introduced to replace AC 275. It was recognized that the minimum viscosity specified for AC 275 was too low to result in acceptable high temperature performance of more heavily trafficked highways. The two new grades were designated as AC 27.5 and AC 60 (Figure 2.2) and were essentially "high viscosity" 200-300 pen and 150-200 pen asphalt cements respectively. The minimum and maximum viscosity limits paralleled PVN (Penetration-Viscosity Number) lines as defined by McLeod (11).

In 1980, asphalt cement specifications were further revised to include a total of 5 grades - a high viscosity 150-200 pen, a high and medium viscosity 200-300 pen and a high and medium viscosity 300-400 pen asphalt cement (Figure 2.3). These grades were designated by their minimum and maximum penetration limits and their relative viscosity levels with the high viscosity materials suffixed "A" and the medium viscosity materials suffixed "B". Again the viscosity limits parallel McLeod's PVN lines. These five grades were adopted to provide maximum flexibility in selecting the appropriate asphalt grade in terms of high, medium or low traffic volumes and recognized that more temperature susceptible asphalt cements ("B" grades) could be incorporated into the pavement structure in lower lifts. Surface courses would only use "A" grades. A comparison of 1979 and 1984 asphalt test data is being evaluated in a current investigation at the University of Alberta by Leung.

2.5 Alberta Transportation's Design Methodology

Currently, asphalt cement selection is the only design factor used to address low temperature pavement performance. The general approach is to select the softest (penetration) grade of asphalt cement that can be used and ensure acceptable high temperature performance. Acceptable high temperature performance is a function of both geographical location and traffic volumes. Generally 150-200A is used on all heavily trafficked highways throughout the Province and on medium trafficked highways in the southern regions; 200-300A is used on medium trafficked highways in the northern regions and on low trafficked highways throughout the Province; 300-400A is restricted to use on community airports. There is no definitive criteria to separate low, medium or high traffic highways or northern and southern regions. Selections are made based on general rules-of-thumb and experience.

Two asphalt suppliers capable of producing "B" grade asphalts are located in the northwest region of the Province. In this region, when more than one lift of pavement is designed for a project, "A" grades are specified for the surface course and "B" grades would be specified as an alternate for lower lift construction. For example, if 150-200A was specified for top lift, 150-200A or 200-300A or 200-300B could be used on bottom lifts. Selection would be governed by economics.

The final selection of an asphalt grade and type is a compromise between the anticipated low temperature cracking performance and the high temperature performance of the asphalt concrete pavement relating to the safety of the travelling public (rutting and hydro-planing under wet weather conditions).

2.6 Other Design Methods to Minimize Low Temperature Cracking

Two different design methodologies to minimize low temperature cracking have been developed.

Haas (8) has presented a model that predicts low temperature cracking frequency. Inputs to the model are stiffness of the original asphalt cement as determined by McLeod's method, thickness of the bituminous layer, design life of the pavement, winter design temperature and subgrade type. The model is used to predict cracking at some time in the future given certain design input values. The designer would then make a judgement on the acceptability of the estimated cracking frequency and if unacceptable, vary the inputs until an acceptable cracking frequency is obtained.

Another design procedure, based on cracking temperatures predicted from asphalt penetrations at 25°C and at 5°C, 100 g, 5 s test conditions, has been proposed more recently (3) as an interim guide to design pavements to resist low temperature transverse cracking. This predicted cracking temperature, based upon the asphalt cement proposed for use, would be compared to the mininum anticipated pavement surface temperature to determine if an asphalt grade change is required.

It is not known how widespread these two methods are being used in North America.

Other indirect and direct procedures have been developed for predicting pavement cracking temperatures from both asphalt and mix stiffnesses (3) but have not been considered in this study.

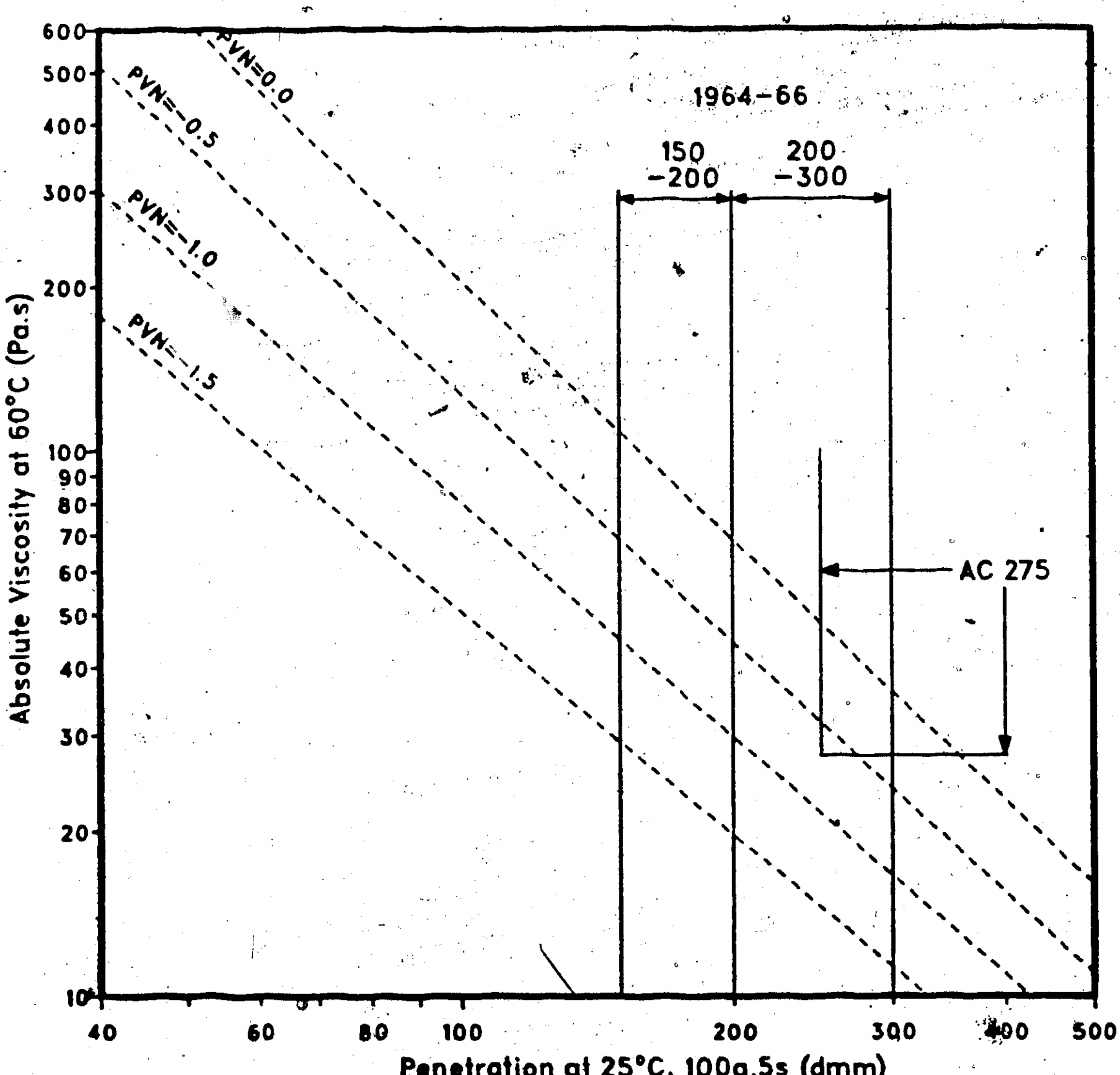


Fig. 2.1 AC 275 Specifications

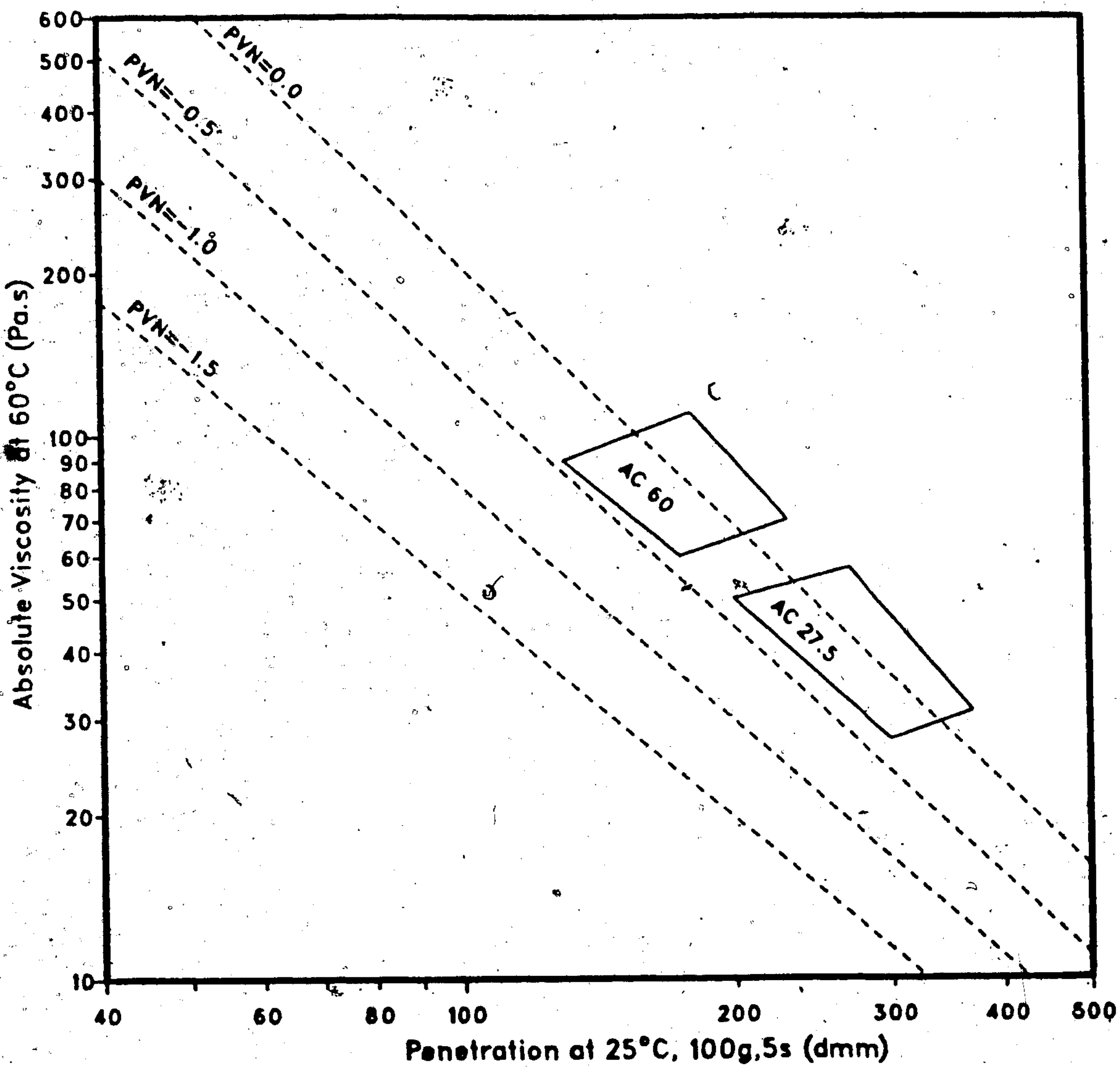


Fig 2.2 AC 27.5 and AC 60 Specifications

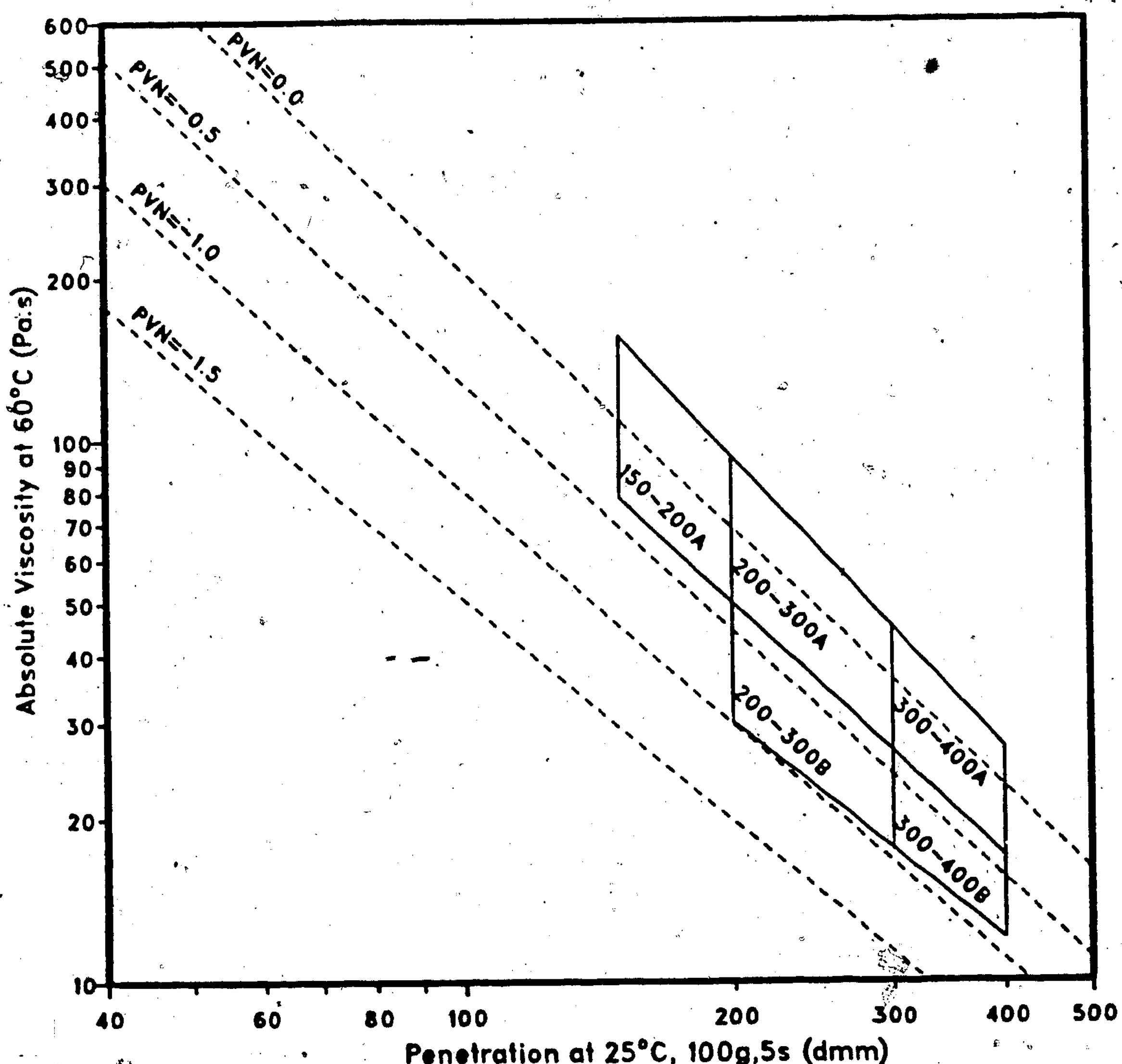


Fig. 2.3 Alberta Transportation Asphalt Cement Specifications

CHAPTER 3

RESEARCH PROGRAM

3.1 Initial Investigation

In the summer of 1981, the author inspected project SR 771 located about 70 km southwest of Edmonton. This 150 mm full depth pavement, constructed in 1975 did not exhibit any transverse cracking over its total length of 10 km. This low temperature performance was recognized as untypical. In order to identify projects exhibiting low frequencies of transverse cracking, the Department's 15 District Engineers were surveyed. Only two projects were identified as exhibiting no cracking; one was SR 771.

Alberta Transportation carries out visual condition ratings of all pavements. Inventory sec[illegible] [illegible]ated on a three year frequency. The visual condition rating [illegible] estimation of transverse cracking frequency. Sections are rated as follows:

Estimating Unit		Transverse Cracking Frequency (cracks/km)
none	= 4	3
minor	= 3	3-30
moderate	= 2	30-79
major	= 1	79-131
severe	= 0	131+

This data base was reviewed to determine if it could be used to identify pavements exhibiting low cracking frequencies. It was felt that this data was not precise enough nor timely enough to use to quantify transverse cracking frequencies.

A program of informal field inspections was initiated during the summer of 1984 to try to identify the extent of non- or low frequency transverse cracking pavements. An initial review indicated this behavior was correlated to full depth pavement structures so inspection efforts were concentrated on projects incorporating this type of pavement structure. These inspections identified that uncracked highways were not as rare as originally perceived. At this point, this informal investigation was upgraded to a research project as part of Alberta Transportation's program with the University of Alberta.

3.2 Transverse Cracking Evaluation

The purpose of the on-site inspections was to try to identify the extent of pavements exhibiting relatively low transverse cracking frequencies and to also identify projects exhibiting various degrees of cracking so that the significant factors influencing cracking could be ascertained.

Inspections were carried out on total projects. A project was defined as a pavement constructed at one time under one contract, built by one Contractor and generally incorporating the same aggregate and asphalt materials. Generally only projects constructed prior to 1979 were inspected which means they would be at least 5-6 years old. It was felt that if any significant transverse cracking was to occur, it would have occurred by then. Performance of the Alberta Test Road (4) showed extensive transverse cracking was experienced after 2 years performance on all sections. Significant transverse cracking of some sections of the Ste. Anne Test Road (5) occurred after the first winter; during the fifth winter, transverse cracking occurred on some previously uncracked sections built on sand subgrades (10). Projects constructed prior to 1970 were not included due to a lack of available data relating to the original materials used during construction.

Initial work indicated the full depth pavement structures tended to exhibit lower cracking frequencies. Virtually all full depth pavements of significant length meeting the age constraints previously identified were inspected. Full depth pavements, although not constructed by the Department anymore, were considered important in the investigation because there would not be any effects of a granular base course influencing cracking frequency. McLeod (6) identified that thick granular bases influenced pavement transverse cracking. These pavements included asphalt cements supplied by six different refineries. Asphalts meeting the AC 275, AC 27.5 and AC 60 specifications were represented. Some low to medium viscosity 150-200 pen asphalts were also included. A few full depth projects that had been subsequently overlaid were also inspected.

In addition, all new pavement constructed in 1978 and 1979 and placed over granular bases were inspected. These projects would have been supplied with asphalt cements meeting AC 27.5 and AC 60 specifications. Complete information and data on materials used was available for these projects. Further, asphalt cement characteristics of the products supplied for this period would be very similar to the products being currently supplied to the Department.

3.3 Measurement of Transverse Cracking Frequency.

For this investigation, only full width transverse cracks were counted. If the crack did not extend at least one-half way across the total pavement width it was ignored. Cracks extending over one-half of the pavement width were considered full width. It was assumed that these cracks were low temperature related and would extend across the total pavement in the future. This approach was taken to simplify the crack counting exercise and would result in less discrepancies in crack counts between different people. Inspections confirmed that over ninety percent of the transverse cracks extended the full width of the pavement.

Alberta Transportation has started to log the entire highway network on a regular frequency using a truck mounted video camera. In some cases where available, these video tapes were used to determine transverse cracking frequencies where crackfilling operations highlighted cracking patterns. Where no cracking was observed, it could not be established if cracks did not exist or if they simply had not been crackfilled. In these cases a follow-up field inspection was carried out.

Transverse cracking frequency in Alberta has been historically quantified by the number of transverse cracks per kilometre (or mile). Frequency, as used in this investigation, has been defined as such. Transverse cracks were counted for each kilometre of the project. Transverse crack frequency for each project was summarized by the number of kilometres exhibiting 0, 1, 2, 3, 4, 5 and 5+ cracks per kilometre. It was felt that if the cracking frequency exceeded 5 cracks per kilometre, ie. a spacing of less than 200 metres, the frequency was significant and further detail was not required. The frequency of 5 per km was assumed to differentiate significant from non-significant cracking. The average frequency per kilometre for the total project was also calculated.

Ontario Ministry of Transportation and Communications (12) expresses transverse cracking frequency in terms of a Cracking Index (C.I.) which is calculated as the number of full transverse cracks plus one-half of the half transverse cracks per 500 ft of roadway length. The transverse cracks less than one-half of the roadway width are not included. Taking the method of counting cracks used for this investigation and assuming no half width cracks, then cracking frequency in numbers of transverse cracks per kilometre divided by 6.6 would be the equivalent Cracking Index.

A review of this cracking frequency information was used to separate projects from further analysis. Only projects with relatively consistent cracking frequencies from kilometre to kilometre were considered. This screening was necessary to ensure that the factors influencing the observed cracking frequency were consistent over the project which would make these factors more easily identifiable. Projects exhibiting significantly different cracking frequencies, but where these differences represented significant consecutive kilometres, were subdivided into sections and each section was considered separately. Examples of a project removed, SR 792:02, and a project included in by dividing it into two sections, 3:12-1 and 3:12-2, are presented in Tables 3.1 and 3.2 respectively.

A total of almost 1800 km of pavement representing about 130 projects were inspected. Excluding projects that did not meet the age constraints, were overlaid or were not included, resulted in 55 sections of full depth pavements totalling 642 km and 22 sections of pavements constructed over granular bases totalling 349 km available for further analyses. The total of 991 km of selected pavements represented:

242 km of 0 crack/km

66 km of 1 crack/km

46 km of 2 cracks/km

32 km of 3 cracks/km

26 km of 4 oracks/km

31 km of 5 cracks/km

548 km of 5+ cracks/km.

3.4 Research Approach

The approach of this investigation was separated into two distinct phases. Phase I involved the large scale transverse crack survey. Information and data pertaining to the pavement structure, its age, as-supplied asphalt characteristics, climate, surficial geology, cracking frequency and as-built pavement characteristics for selected sections were gathered and entered into a data bank. This data was analyzed using multiple regression computer programs to identify the major factors influencing the observed cracking frequency.

Phase II was a more detailed investigation of a small number of projects exhibiting peculiar transverse cracking behavior; for example, significant cracking and non-cracking sections within one project. This detailed investigation included a field sampling and laboratory testing program designed to identify the in-place asphalt, aggregate and subgrade characteristics in order to try to correlate cracking performance with both original and in-place material characteristics.

Table 3.1 Example of Project Screened Out of Further Analysis - SR 792:02

km - km	Transverse Crack Count (Cracks/km)
22.7 - 21.7	1
21.7 - 20.7	0
20.7 - 19.7	21
19.7 - 18.7	22
18.7 - 17.7	0
17.7 - 16.7	0
16.7 - 15.7	3
15.7 - 14.7	1
14.7 - 13.7	4
13.7 - 12.7	28
12.7 - 11.7	36
11.7 - 10.7	34
10.7 - 9.7	3
9.7 - 8.7	20
8.7 - 7.7	17
7.7 - 6.7	32
6.7 - 5.7	19
5.7 - 4.7	35
4.7 - 3.7	33
3.7 - 2.7	15
2.7 - 1.7	2
1.7 - 0.7	2
0.7 - 0	0

Ave Frequency (22 km) = 14.9

Std Dev. = 13.7

Table 3.2 Example of Project Screened in by Dividing Into Sections - Hwy 3:12

3:12-1	km - km	Transverse Crack Count (cracks/km)
	12.0 - 13.0	4
	13.0 - 14.0	9
	14.0 - 15.0	0
	15.0 - 16.0	0
	16.0 - 17.0	4
	17.0 - 18.0	0
	18.0 - 19.0	2
	19.0 - 20.0	0
	20.0 - 21.0	2
	21.0 - 22.0	5

Ave Frequency (10 km) = 2.6

Std Dev. = 3.0

3:12-2		
	22.0 - 23.0	19
	23.0 - 24.0	9
	24.0 - 25.0	8
	25.0 - 26.0	12
	26.0 - 27.0	7
	27.0 - 28.0	7
	28.0 - 29.0	6
	29.0 - 30.0	17
	30.0 - 31.0	17
	31.0 - 32.0	20

Ave Frequency (10 km) = 12.2

Std Dev. = 5.5

CHAPTER 4

PHASE I - DEVELOPMENT OF DATA BANK

4.1 Introduction

Phase I of this investigation was the development and analysis of a data bank of information relating to 77 sections of pavement exhibiting various degrees of transverse cracking. This chapter identifies the major inputs to the data bank, how or where they originated and the form in which they were quantified.

4.2 Transverse Cracking Frequency

As discussed in Section 3.3, the number of kilometres of pavement having transverse crack frequencies of 0, 1, 2, 3, 4, 5 and 5+ cracks/km, the total length of the section inspected and the average crack frequency for the section were entered into the data bank. Projects or sections that were screened out were not included.

4.3 Asphalt Cement Properties

Alberta Transportation has historically specified, supplied and paid for directly, the asphalt cement to be used on a paving project. It is the Department's quality control policy (13) to routinely sample, on a frequency of about one sample per 200 tonnes, the asphalt cement delivered to the project. These samples are tested to ensure specification compliance and to measure other, unspecified characteristics. Historical records were reviewed and the average test results for each characteristic of the samples actually submitted from each project were summarized and entered. There were a few isolated projects where samples were not taken or tested. In these cases, the average test results for samples submitted from other projects being supplied by the same asphalt supplier and refinery for the appropriate time period of construction were used. The information relating to the supplier, asphalt cement grade and materials characteristics included in the data bank are presented in Table 4.1. In general the data is complete for all projects except that kinematic viscosities were only available for samples tested in 1978 and 1979.

The penetration viscosity numbers, PVN, as defined be McLeod (11) were calculated for both the original asphalt and for the residue after the Thin Film Oven Test (TFOT). PVNs using viscosities at 60°C were designated PVN' to distinguish them from PVNs using viscosities at 135°C. PVNs were calculated using the formulas (14):

$$PVN' = \frac{6.489 - 1.590(\log P) - \log X'}{1.050 - 0.2234(\log P)} \quad (-1.5)$$

$$PVN = \frac{4.258 - 0.7967(\log P) - \log X'}{0.7951 - 0.1858(\log P)} \quad (-1.5)$$

where P = penetration at 25°C, dmm

X = viscosity at 135°C, mm²/s

X' = viscosity at 60°C, poise.

4.4 Pavement Structural Data

Information and data relating to the pavement structure was taken from the Department's Pavement Management System. This data originated from project final details submitted at the end of each project.

Information entered were the years of construction and thicknesses of the asphalt concrete and granular base layers. In cases where construction was staged, ie. the granular base was placed in one year and the asphalt concrete placed usually 1 or 2 years later, the granular base thickness would include 50 mm of a liquid asphalt bound temporary wearing course.

4.5 Climate

It was recognized that project or section specific data relating to low temperature climatic conditions was a required input. These low temperature climatic conditions can be described in terms of the Freezing Index and/or the minimum ambient air temperature experienced by the pavement during its life. Freezing Index gives a measure of the severity and length of cold weather conditions and varies significantly over the Province from about 800°C days in the south (Lethbridge) to about 2800°C days in the north (High Level). Minimum low temperatures are required for stiffness predictions and values should be used that reflected the lowest temperature the pavement has been exposed to in the case of sections exhibiting no cracking. Ideally, for sections exhibiting some degree of transverse cracking, values should be used that reflected the temperature the pavement was exposed to when it initially cracked. Minimum low ambient air temperatures also vary significantly across the Province. A review of climate records indicates that even extreme southern areas of the Province have been exposed to absolute minimum temperatures less than -40°C. It was decided to include both Freezing Index and some value of minimum low ambient temperature in the data bank.

Freezing Indices were interpolated for each section from a map (8) showing Freezing Index contours for Canada. There were no iso-temperature contour maps available for Alberta presenting historical minimum lows or expected minimum lows based on some probabilistic basis. A map of iso-temperature contours for Canada on a one percent recurrence interval basis presented in Reference (2) was considered too coarse to allow reasonably precise interpolations. Environment Canada (15) publishes weekly minimum temperatures for various percent risk levels for selected locations. This data was available for too few stations to be used.

A map included in the National Building Code (16) showing iso-temperature contours on a 2.8°C interval for January design temperatures (1 percent basis) was compared to local weather station data for the period of 1975-1984 for 8 locations across Alberta. The lowest experienced temperature that was equalled or exceeded only once in the 10 year period was compared with the map temperature. The data on the map correlated well with locally measured low temperatures in terms of relative differences between localities but appeared to underestimate actual experienced low temperatures by about 4°C. The values on this map were reduced 4°C and then the modified map was used to select low minimum winter temperatures for all sections. This map is presented in Figure 4.1.

4.6 Asphalt Stiffness

Asphalt cement stiffness can be determined using van der Poel's nomograph or nomographs modified by Heukolom or McLeod depending upon the measurement of temperature susceptibility being used. The required inputs to determine asphalt stiffness using these nomographs are:

- some measure of asphalt consistency at two temperatures
- some measure of asphalt temperature susceptibility
- temperature
- time of loading.

Because only penetration at 25°C and absolute viscosity at 60°C were available for the asphalt supplied for all sections, McLeod's method (11) for determining asphalt stiffness was used. The Asphalt Institute report (3) commented that McLeod's PVN system is reasonable for selected non-waxy, non-air blown asphalts only. Some asphalts supplied to the Department in the seventies may have had wax contents in excess of two percent. Wax contents of selected asphalts were determined by Alberta Transportation in the early seventies (17) following a procedure developed by Kromm (18). The results of this testing indicated that wax contents between one and eight percent were measured on selected asphalt samples. It is recognized that some errors in stiffness determination might be expected.

Temperature is a major input into the determination of asphalt stiffness. The determination of the minimum low ambient temperature at each section location was discussed in Section 4.5. To determine the asphalt stiffness, the pavement temperature at a 50 mm depth was determined using a chart based upon the Ste. Anne Test Road (19) that correlates air temperature with pavement temperature at various depths. Following McLeod's method (11), a loading time of 20 000 s (5-6 h) corresponding to a stress being applied as a result of slow chilling was used.

In summary, the following were used as inputs into all asphalt stiffness determinations:

- penetration at 25°C, 100 g, 5 s (dmm)
- PVN' based upon penetration at 25°C and absolute viscosity at 60°C
- time of loading of 20 000 s
- pavement temperature at 50 mm depth.

Stiffnesses were determined for both original and aged asphalt after the thin film oven test (TFOT). Low-temperature conditions for each section were used. In addition the stiffness was also calculated for the original asphalt using a constant pavement temperature of -29°C (equivalent to an ambient air temperature of -38°C) for all sections.

The reason for this additional determination will be discussed in Chapter 5.

The nomograph used gave asphalt stiffnesses in units of kg/cm². These values were in the range of about 5 - 200 kg/cm² which were convenient values to use in the analysis phase and resulted in regression coefficients between zero and one. Although this unit is not a recognized SI unit, it was used to simplify the analysis and model development.

The Figures used to determine asphalt stiffnesses are included in Appendix D.

4.7 Subgrade Materials

Although there is not a very wide range of soils generally encountered throughout the Province, it was felt that it was necessary to try to characterize subgrade soils to try to determine their effect on cracking frequency. It was recognized that it would be difficult to identify subgrade soil characteristics as well as to quantify them in some manner that would allow their inclusion into the analysis phase. Further it was recognized that the vast majority of sections would have been constructed over medium plastic clay subgrades.

One source of this data investigated was the results of preliminary soil surveys normally carried out by the Department prior to the design and construction of the subgrade. These results may or may not indicate the type of material incorporated into the subgrade but would normally represent the general soil types of the area. Many of the subgrades were designed and constructed by other jurisdictions and the preliminary design and construction testing was not available. Agricultural soil survey maps were investigated as another possible source of soil classification information but were not considered due to the non-engineering based soil classifications used.

A detailed study of a few select projects identified that transverse cracking behavior was highly influenced by the location of surficial sand dune deposits. In a couple of cases, the boundaries between sections of significantly different cracking frequencies coincided almost exactly with the boundaries of different surficial geological deposits. Surficial geology maps produced by the Geological Survey of Canada were used to identify the broad surficial geology of each section. For sections where maps were not available, air photos available through Alberta Energy and Natural Resources were used to classify the surficial geology deposits. These surficial deposits were broadly classsified as:

- tills (clays, silts)
- lacustrine deposits (silts, clays)
- lacustrine deposits (silts, sands)
- outwash deposits (gravel, sand)
- aeolian deposits (sand, silt)
- alluvial deposits (gravel, sand, silt, clay).

It was recognized that these classifications were not precise enough to be considered as an independent variable in further analyses, but could possibly be used to separate fine-grained from coarse-grained subgrades.

The only way to definitely classify subgrade soils would be a large scale field sampling program which could not be economically justified.

4.8 As-built Pavement Characteristics

Quality of construction is routinely monitored through on-site field testing by Alberta Transportation on all construction projects. Mix characteristics that would be measured on a paving project include:

- asphalt content
- formation of Marshall specimens and calculation of air voids and voids in the mineral aggregate (VMA) following Asphalt Institute procedures (20)
- aggregate gradation
- as-built pavement density (percent compaction) and air voids

This data was assembled for a select 32 sections that were full depth pavements built at one time and that had a structural thickness of 150 mm or greater.

Although Marshall mix designs were carried out for each project, the design data were not included in the data bank as it was considered that this data would not be significant in influencing low temperature performance.

Table 4.1 Asphalt Cement Information
Entered Into Data Bank

1. Supplier eg Imperial Oil
2. Refinery Location eg. Edmonton
3. Grade and Type eg. AC 275
4. Penetration @ 25°C, 100 g, 5 s (dmm)
5. Penetration @ 4°C, 200 g, 60 s (dmm)
6. Absolute Viscosity @ 60°C (poise)
7. Kinematic Viscosity @ 135°C (mm^2/s) (Note 1)
8. PVN' (Note 2)
9. PVN (Note 3)
10. Thin Film Oven Test (TFOT) Loss
11. Penetration @ 25°C, 100 g, 5 s (dmm) on Residue After TFOT
12. Penetration @ 4°C, 200 g, 60 s (dmm) on Residue After TFOT
13. Absolute Viscosity @ 60°C (poise) on Residue After TFOT
14. PVN' of Residue After TFOT (Note 2)

Note 1 - These results are only available for samples tested after and including 1978.

Note 2 - PVN' calculated using penetration @ 25°C and absolute viscosity @ 60°C.

Note 3 - PVN calculated using penetration @ 25°C and kinematic viscosity @ 135°C.

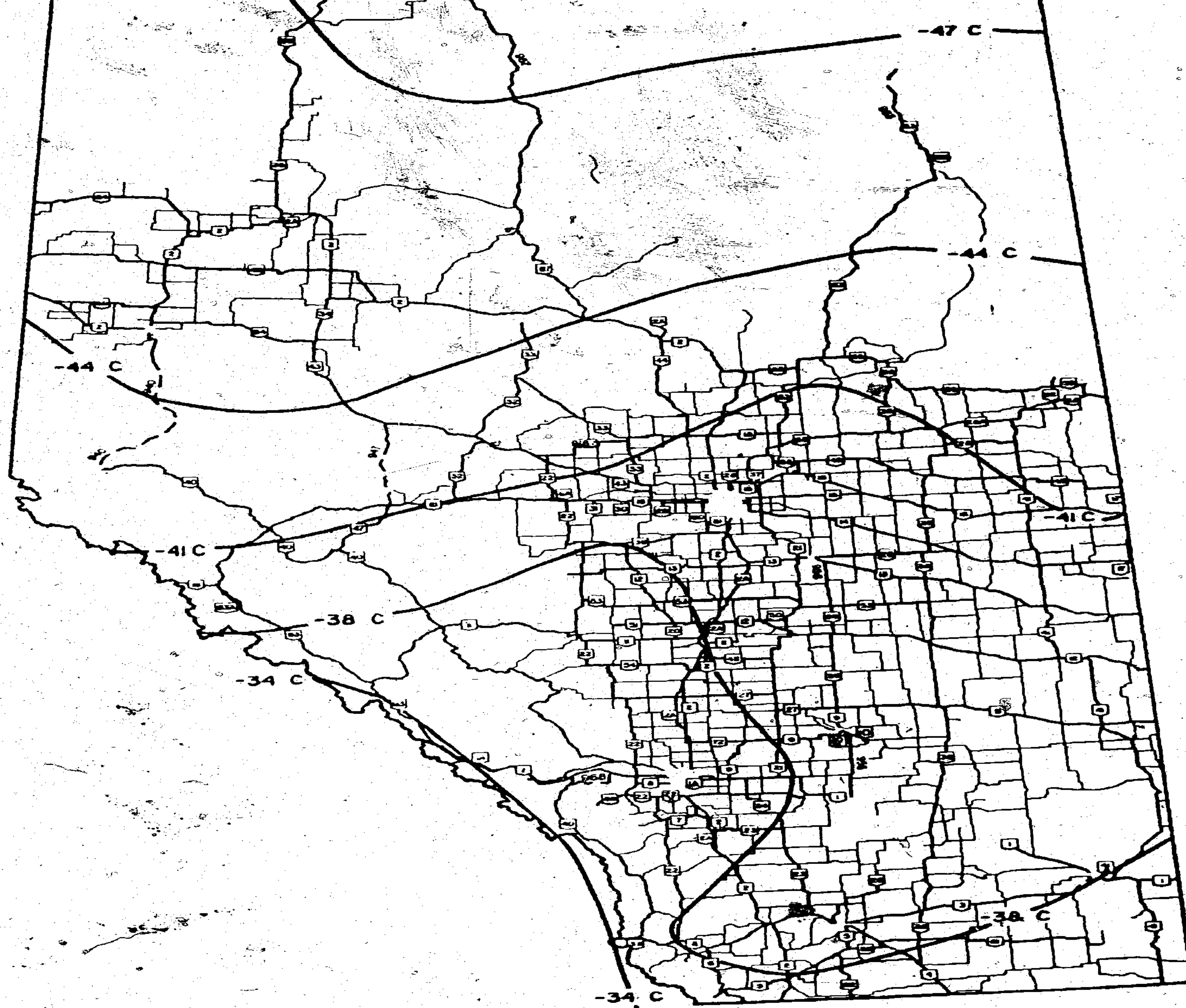


Fig. 4.1 Minimum Low Temperatures

CHAPTER 5

PHASE 1 - ANALYSIS

5.1 Introduction

As presented in Chapter 3, a total of 1800 km of pavement representing about 130 projects was inspected. Screening out projects that were constructed before 1970 or after 1979, projects that have been overlaid and projects exhibiting highly variable cracking frequencies from kilometre to kilometre, resulted in 55 sections of full depth pavement totalling 642 km and 22 sections of pavement constructed over granular base totalling 349 km available for further analysis.

The objective of the analysis stage of the investigation was to identify relationships between the observed transverse cracking behavior and the variables identified and accumulated in the data bank. Mathematical models developed could be used to identify the singular effects of each significant variable identified on cracking performance. Quantification of the effects of each major variable could then be used as part of the pavement design procedure to optimize low temperature pavement performance.

5.2 Statistical Approach

All data was entered into the Alberta Government's mainframe IBM computer which allowed convenient analysis using available statistical analyses packages. Stepwise regression (STEPR) and multiple regression analysis programs (MULTR) on FOCUS (21) were used as the primary programs to assist in identifying the most significant effects (independent variables) influencing transverse cracking frequency (dependent variable).

The STEPR analysis program performs stepwise multiple regressions for any choice of dependent and independent variable(s) from the data file. Each step of the analysis looks at the reduction of the sum of squares for each variable and includes the next independent variable which explains the greatest amount of variance between it and the dependent variable (the variable sharing the highest partial correlation with the dependent variable). The program will also allow any variable to be deleted or forced into the regression equation regardless of its partial correlation with the dependent variable. The program uses a forward selection procedure (22) to govern the selection of independent variables into each step of the analysis. A major drawback of this procedure is that the program does not check each independent variable left at the end of each step with the independent variables already entered into the regression equation. Without this check, independent variables that are highly correlated with each other will be entered into the final regression equation. For example, for typical asphalts utilized in Alberta, there is a high correlation between PVN, based on absolute viscosity at 60°C and PVN based on kinematic viscosity at 135°C. This STEPR program, in an appropriate situation, could select both variables as significant where, in fact, only the most significant of the two would be desirable to define temperature susceptibility.

As a result of this constraint in the program, large numbers of multiple regression analyses were carried out using judgement in selecting the independent variables to be used in each case.

The multiple regression programs available could only fit the data to models that were linear in the parameters. Natural log and log10 transformations and cross-products between significant variables were evaluated to try to improve the significance of the models developed.

A number of techniques were used to evaluate the significance or validity of each regression equation generated. Table A-11 in Reference (23) was used to evaluate the significance of r, the correlation coefficient. The F value for analysis of variance, defined as the ratio of the mean square attributable to the regression to the mean square deviation from the regression, was used to evaluate the significance of the regression equation. The t value, defined as the ratio of the regression coefficient to the standard error of the regression coefficient, was used to evaluate the significance of each regression coefficient. The sign of each regression coefficient was checked against general relationships known to exist between each independent variable and cracking frequency.

5.3 Summary of Analyses

The projects or sections were grouped into three classes and regression analyses as previously described were carried out on each class:

- all full depth pavements
- all granular base pavements
- all pavements combined
- all full depths with thicknesses greater or equal to 150mm using construction quality control data.

5.3.1 All Full Depth Pavements

The analysis of full depth pavements as a separate group was advantageous as there would not be any effects of a granular base course on cracking frequency. The transverse cracking data for the 55 full depth sections is presented in Table 5.1. The soil characteristics interpreted from surficial geology maps are presented in Table 5.2.

Altogether, a total of 16 independent variables related to the pavement structure and age, climatic conditions, asphalt cement and asphalt stiffness were considered as potential independent variables for the model. The values of these independent variables for each section are presented in Table 5.3. The simple correlation coefficients were calculated by the computer and the correlation coefficient matrix is presented in Table 5.4.

A number of interesting observations can be made from the correlation coefficient matrix. In order to interpret the significance of the correlation coefficients, Table A-11 in Reference (23) was used. Using this table, for 55 observations and 2 degrees of freedom, a minimum correlation coefficient of 0.325 is required to reject the null hypothesis that there is no association between the variables at the one percent level of significance. Obvious correlations between penetration at 25°C, penetration at 4°C and absolute viscosity at 60°C before and after the TFOT are expected. The correlation coefficient between the pavement age and viscosity at 60°C of the original asphalt supplied is -0.409. This shows that as pavement age increases, absolute viscosity at 60°C used for construction decreases. This suggests that higher viscosity asphalts have been used for newer pavements which is confirmed by the introduction and greater use of higher viscosity asphalts by the Department during the 70's. The positive correlation coefficient of 0.3724 between penetration at 25°C and freezing index suggests that as freezing index increases the penetration at 25°C of the asphalt used for construction increases which reflects the Department's tendency to use softer asphalts in more northern regions.

Numerous stepwise regressions were run to evaluate a large variety of combinations of these independent variables describing asphalt characteristics (penetration at 25°C, absolute viscosity at 60°C, etc.), temperature susceptibility (PVN',PR) and climatic conditions (freezing index, minimum low temperature).

The final, most statistically significant model developed was:

$$\text{Frequency (cracks/km)} = 49.40 + 3.09 \text{ (pavement age in years)} + 0.36 \text{ (original asphalt stiffness in kg/cm}^2\text{)} - 5.60 \text{ (pavement thickness in mm)}^{0.5}$$

This is referred to as Model 1. The computer output for this regression equation is presented in Table 5.5.

The statistical significance of this model can be evaluated in a number of ways. The correlation coefficient, r, of 0.80, using Table A-11 (23) is significant at the one percent level. The multiple correlation coefficient, r^2, of 0.64 indicates that 64 percent of the total variance is explained by the model. Comparing the F value for F (3, 51, 0.1 percent) = 6.407 to F = 30.244 from the analysis of variance indicates the model is adequate at the 0.1 percent level. (There is less than a 0.1 percent chance that an F value greater than 4.211 would be achieved by chance alone). The t value for 51 degrees of freedom is 3.523 at the 0.1 percent level of significance. Comparison of the calculated t statistics for the independent variables of 4.479, 6.846 and -4.726 to this value confirms the significance of the regression coefficients at the 0.1 percent level (there is less than 0.1 percent chance that a t value of 3.523 would be achieved by chance alone).

5.3.2 All Granular Base Pavements

A total of 22 granular base sections were included in this analysis. One additional independent variable, granular base thickness, was included. The transverse cracking data and soil characteristics, interpreted from surficial geology maps are presented in Tables 5.6 and 5.7.

The values of the independent variables for each section are presented in Table 5.8. A similar analysis approach as for the full depth sections was taken. A review of the values of the independent variables asphalt pavement thickness and asphalt pavement age shows very little variation in values due to the way these projects were selected for inclusion into the investigation. As a result both these variables did not enter the final model. The effect of granular base thickness was also not significant. The final most statistically significant model developed was:

Frequency (cracks/km) = -0.47 + 0.37 (original asphalt stiffness in kg/cm²)

$$\text{Frequency (cracks/km)} = -0.47 + 0.37\ (\text{original asphalt stiffness in kg/cm}^2)$$

This is referred to as Model 2. The multiple correlation coefficient, r^2, of 0.39 indicates that 39 percent of the total variation is explained by the effects of original asphalt stiffness. A review of the calculated F and t statistics indicates significance of these values at the 0.1 percent level. This model does not explain as much of the total variation as the full depth model. But it is significant to note that stiffness of the original asphalt cement had the greatest correlation with cracking frequency. Further, the coefficients of the stiffness independent variable are virtually identical in both models.

5.3.3 All Projects Combined

As previously noted, the selection of the 22 granular base sections were biased. When these sections were aggregated with the 55 full depth sections, a similar, slightly less significant model was developed:

$$\text{Frequency (cracks/km)} = 6.59 + 3.64\ (\text{Pavement age in years}) + 0.37\ (\text{Original asphalt stiffness in kg/cm}^2) - 2.91\ (\text{Pavement thickness in mm})^{0.5}$$

This is referred to as Model 3. The multiple correlation coefficient, r^2, of this model (77 sections) was 0.52 which is less than 0.64 for the full depth model (55 sections).

5.3.4 Full Depth Pavements with As-built Pavement Characteristics

An initial evaluation of cracking frequencies of full depth pavements indicated that high cracking frequencies were occurring on all sections with pavement thicknesses less than 150 mm. These sections were also generally older than sections with pavement thicknesses greater than 150 mm. In order to reduce the effects on cracking by including these sections with relatively thin pavement thickness, 32 full depth sections with pavement thickness greater than or equal to 150 mm were selected and mix and pavement characteristics at the time of construction were assembled. This data was summarized from construction quality control records. In addition to the 16 independent variables used for the analysis of all full depth pavements, 9 additional variables relating to mix and pavement characteristics at the time of construction were included. The values of these variables are presented in Table 5.9.

The final, most statistically significant model developed was:

Frequency (cracks/km) = 153.28 + 2.65 (pavement age in years)
+ 0.40 (original asphalt stiffness in kg/cm^2)
+ 3.32 (field Marshall V.M.A. in percent)
- 2.37 (percent compaction)

This is referred to as Model 4. The computer output for the regression equation is presented in Table 5.10. The multiple correlation coefficient, r^2, is 0.60 indicating that 60 percent of the total variation is explained by the model. The calculated F value indicates the model is adequate at the 0.1 percent level. The computed t values indicate that the regression coefficients for pavement age, original asphalt stiffness, field Marshall V.M.A. and percent compaction are significant at the 5 percent, 0.1 percent, 1 percent and 5 percent levels respectively.

5.3.5 Other Analyses

Further analyses were carried out to try to identify if asphalt source or refinery had an influence on transverse cracking frequency. There was a total of six different refineries that supplied asphalt to the sections included in the investigation which resulted in sample sizes of 1 to 18 sections having a unique supplier. A bias was introduced as there was a tendency for refineries to supply projects located in their general geographical area. Further, because of the small sample sizes, there was a lesser range of pavement thicknesses, ages and asphalt stiffnesses. All of these factors resulted in models of less statistical significance than when all sections of common pavement structure were aggregated.

Attempts were made to quantify surficial geology data to determine the influence of subgrade effects on transverse cracking. Codes were assigned to broad surficial geology classifications. Over ninety percent of the subgrade types were identified as fine grained clays or silts. Exclusion or inclusion of the less than ten percent of the sections with subgrade soils other than clay or silt did not significantly affect the models developed.

5.4 Discussion of Models

5.4.1 All Full Depth Pavements

The final, most significant model developed to explain the observed cracking frequency of full depth pavement sections was:

$$\begin{aligned}\text{Frequency (cracks/km)} = {} & 49.40 + 3.09\,(\text{Pavement age in years}) \\ & + 0.36\,(\text{Original asphalt stiffness in kg/cm}^2) \\ & - 5.60\,(\text{Pavement thickness in mm})^{0.5}\end{aligned}$$

The independent variables in the model will be discussed in the order in which they were entered by the stepwise regression program.

Pavement Thickness (in mm) - The square root of the pavement thickness exhibited the highest correlation coefficient with cracking frequency. The negative sign of the regression coefficient shows that as pavement thickness increases, cracking frequency reduces which is consistent with the findings of others (5) (8). The value of the coefficient suggests for example, that an increase in pavement thickness from 150 mm to 175 mm will reduce cracking frequency on average by 5.5 cracks/km. Using the 95 percent confidence interval for this coefficient suggests that cracking frequency will be reduced between 2.4 and 8.7 cracks/km with a 1 in 20 chance of being wrong.

Original Asphalt Stiffness (in kg/cm²) - The original asphalt stiffness had a slightly lower correlation coefficient with cracking frequency (0.4832 compared to 0.4878) and as a result was entered into the equation second. The positive sign of the coefficient shows that as the stiffness of the original asphalt supplied increases, cracking frequency increases, which is consistent with the findings of others (2) (5) (8) (9). The determination of original asphalt stiffness is a function of original asphalt consistency, temperature susceptibility and pavement temperature. This suggests:

1. as penetration at 25°C increases, stiffness reduces and cracking reduces (with temperature susceptibility and temperature constant)
2. as temperature susceptibility decreases, stiffness decreases and

cracking reduces (with penetration at 25°C and temperature constant)

3. as temperature increases, stiffness decreases and cracking reduces (with penetration at 25°C and temperature susceptibility constant).

To determine the effects of asphalt cement grade and type on cracking frequency, it is necessary to separate the effects of asphalt characteristics and temperature in the value of original asphalt stiffness. Another regression analysis was carried out using the independent variables of pavement age, pavement thickness and original asphalt stiffness based on an ambient temperature of -38°C for all sections. The multiple correlation coefficient, r^2, of this model was 0.46 compared to 0.64 from Model 1 which used original asphalt stiffnesses based on site specific temperature conditions. This significant reduction in r^2 means that ambient temperature is affecting, in real terms, the cracking frequency observed.

Assuming a minimum ambient temperature of -43°C, which results in a pavement temperature at a 50 mm depth of -33°C, and typical characteristics of asphalt cements used by Alberta Transportation, asphalt stiffness using McLeod's procedure and a loading time of 20 000 sec were determined. This is summarized below:

Asphalt Grade Designation	Penetration at 25°C (dmm)	PVN' (based on absolute viscosity at 60°C)	Predicted Asphalt Stiffness (kg/cm^2)
150-200A	162	0.0	100
200-300B	265	-0.8	80

200-300A	265	0.0	30
300-400A	335	0.0	20

Assuming constant effects of pavement thickness and age, then the effects of each of the above grades on transverse cracking and the range of effects at the 95 percent level of confidence are presented below:

Asphalt Grade Designation	Expected Transverse Freq (cr/km) due to the Effect of Asphalt Stiffness (from Model 1)	Range of Transverse Cracking at the 95 percent Confidence Level
150-200A	36.0	25.1 - 45.9
200-300B	28.8	20.1 - 36.7
200-300A	10.8	7.5 - 13.8
300-400A	7.2	5.0 - 9.2

These results indicate for example, that the singular effects of using 150-200A result in transverse cracking between 25.1 - 45.9 cracks/km at the 95 percent confidence level. Further, an average decrease in cracking frequency of about 25.2 cracks/km (36.0-10.8 = 25.2) could be expected by using 200-300A.

Pavement Age in Years - The regression coefficient of 3.09 for this variable shows that an increase of one year in pavement age was associated with an increase in cracking frequency of 3.1 cracks/km. For example, at the 95 percent confidence interval, cracking frequency will increase between 1.7 and 4.5 cracks/km per year.

The model developed for full depth pavements is considered significant although only 64 percent of the total variation is explained. It cannot be expected to explain cracking frequency definitely for a wide variation of projects based only upon the age and thickness of the pavement and the stiffness of the original asphalt supplied determined for estimated site specific climatic conditions. To be able to improve the model, the pavement stiffness characteristics at the time of initiation of the first transverse cracks would have to be measured. Use of this actual critical pavement stiffness would then take into account the effects of actual temperature conditions, aggregate characteristics, and asphalt aging due to initial construction effects and time.

5.4.2 Full Depth Pavements with As-Built Pavement Characteristics

The model developed for 32 selected full depth sections with pavement thicknesses greater than 150 mm and consideration of as-built pavement characteristics was:

$$\begin{aligned}\text{Frequency (cracks/km)} = {} & 153.28 + 2.65\ \text{(Pavement age in years)} \\ & + 0.40\ \text{(Original asphalt stiffness in kg/cm}^2\text{)} \\ & + 3.32\ \text{(Field Marshall V.M.A. in percent)} \\ & - 2.37\ \text{(percent compaction)}.\end{aligned}$$

The computer output for this model is presented in Table 5.10. These regression coefficients are not as statistically significant as those in the model developed for all full depth pavements. The values and signs of the regression coefficients for the pavement age and original asphalt stiffness independent variables are similar to those in the full depth model. The third variable entered into the model by the stepwise regression program was the field Marshall V.M.A. The sign of the regression coefficient showed that an increase in field Marshall V.M.A. was associated with an increase in cracking. Van der Poel (24) developed a figure that related asphalt stiffness and pavement stiffness as a function of mix V.M.A. and air voids. This figure shows that for a given asphalt stiffness and air void content, an increase in V.M.A. resulted in a decrease in mix stiffness. This should result in a decrease in the potential for low temperature cracking, contrary to the model prediction.

The last independent variable entered was percent compaction (pavement density/field Marshall density x 100). The sign of the regression coefficient indicates that as percent compaction increases, cracking frequency decreases. Using van der Poel's (24) relationship between asphalt and pavement stiffness, for a given asphalt content, an increase in pavement density results in an increase in pavement stiffness which should result in an increase in the potential for low temperature cracking which again conflicts with the model.

Although the influences of the field Marshall V.M.A. and percent compaction as predicted by the model do not correspond with relationships presented by van der Poel, it would appear that mix characteristics may also be influencing the frequency of transverse cracking.

5.5 Design Approach to Minimize Transverse Cracking

In order to establish definitive design guidelines to minimize low temperature transverse cracking it is necessary to identify a critical stiffness that separates acceptable from non-acceptable cracking. This critical asphalt stiffness based upon a review of the data available for the 77 sections previously presented and on the case studies to be discussed in Chapter 6 was selected to be 30 kg/cm² or 2.9×10^{6} Pa. (using original asphalt characteristics and the temperature at a pavement depth of 50 mm).

Knowing the typical characteristics of asphalt cements used by Alberta Transportation and using this critical asphalt stiffness value, critical pavement temperatures can be determined using van der Poel's stiffness nomograph as modified by McLeod. For example for 150-200A, penetration at 25°C = 162 dmm and PVN' = 0.0, the critical temperature at a 50 mm pavement depth is -28°C. This is equivalent to a critical ambient temperature of -37°C. In other words, at an ambient air temperature of -37°C, the asphalt stiffness of a 150-200A asphalt cement at a pavement depth of 50 mm will reach 2.9×10^{6} Pa which would be equivalent to a mix stiffness of 1.7×10^{9} Pa. McLeod (25) suggests that cracking would be eliminated if the mix stiffness at a 50 mm pavement depth and a temperature of -28°C was less than about 1.9×10^{9} Pa. This value compares favourably with the critical stiffness assumed based on Alberta conditions.

Similar calculations can be carried out for other typical asphalt grades. These calculations can be extended to determine ambient air temperatures that correspond to an asphalt critical stiffness at a 100 mm pavement depth. The ambient air temperatures that will result in the asphalt binder at a 50 mm depth and at a 100 mm depth reaching the critical stiffness for asphalt grades used by Alberta Transportation are:

Asphalt Grade	Ambient Air Temperature in °C. Resulting in a Critical Asphalt Stiffness of 2.9 x 10⁴ at a Pavement Depth of:	
	50 mm	100 mm
150-200A	-37°C	-43°C
200-300B	-38°C	-44°C
200-300A	-43°C	-49°C
300-400A	-45°C	-51°C

These values indicate, that for an expected low ambient temperature of -43°C, that 200-300A or 300-400A would have to be used for the surface pavement course if transverse cracking is to be avoided. This chart also shows that any of the grades could be used for the construction of lower lifts without the asphalt binder critical stiffness being exceeded. However this use of stiffer asphalt grades in lower lifts would only apply if the total pavement structure was built during one construction season. Construction of lower lifts and surface lifts cannot be staged over a few years. If transverse cracking is initiated in the pavement prior to the surface lift being placed, the cracks may reflect through the surface lift regardless of the original stiffness of the asphalt being used.

The same map that was developed to determine minimum expected ambient temperatures for each pavement section included in this investigation can be used to identify climatic regions that would correspond to the minimum ambient temperatures identified for each asphalt grade. A design map has been developed to assist in the selection of asphalt grade and type based upon climatic region. This map is presented in Figure 5.1.

These maps show that north of about Peace River, corresponding to the 22nd base line, that all grades will crack to some unacceptable level. In this northern most region, the use of 300-400A would minimize low temperature transverse cracking.

In summary, these maps can be used as a design aid in the selection of asphalt cement grades to minimize low temperature transverse cracking. Final asphalt grade selection for a particular project would be determined by considering high temperature pavement performance in terms of expected rutting, given traffic volumes and expected maximum summer temperatures.

The constraints of using these design maps for the selection of asphalt cement grades to minimize low temperature transverse cracking are:

1. Asphalt grade selection for surface and bottom lifts applies only

to new construction. The entire pavement structure must be built during one construction season and cannot be staged.

2. The maps won't apply to asphalt grade selection of overlays over previously cracked pavements. The reflection of existing transverse cracks through a new overlay has a different failure mechanism than the initiation of low temperature cracks in a new pavement.

3. Original asphalt stiffness is only one factor effecting transverse cracking frequency. To result in an absolute minimum frequency of low temperature transverse cracks, the thickness of the asphalt pavement must be maximized.

Table 5.1 Transverse Cracking Data for All Full Depth Sections

SECTION	NO. OF KILOMETRES WITH INDICATED FREQUENCY 0	1	2	3	4	5	5+	TOTAL (KM)	FREQUENCY (CR/KM)
1:10-1	2	1	2	1	0	1	0	7	1.8
1:10-2	1	1	2	0	0	0	2	6	4.7
1:10-3	1	2	1	0	0	0	4	8	4.8
1:12-1	0	0	0	0	0	0	7	7	45.0
1:12-2	5	0	1	1	1	0	0	8	1.3
1A:06	6	6	2	4	1	1	0	20	1.6
1A:08	1	3	0	0	2	0	3	9	3.9
12:08	1	1	0	0	0	2	0	4	2.8
14:06	0	0	0	0	0	0	9	9	11.0
2:12-1	7	1	0	0	0	1	0	9	.7
2:64	0	0	0	0	0	0	13	13	92.0
2A:06	0	2	1	0	0	0	0	3	1.3
20:02-1	0	0	0	0	0	0	5	5	53.0
20:02-2	0	0	0	0	0	0	11	11	26.0
21A:10	4	1	0	0	0	0	0	5	.2
22:16	0	0	0	0	0	1	11	12	11.0
3:02	7	0	0	0	0	0	0	7	.0
3:12-1	4	0	2	0	2	1	1	10	2.6
3:12-2	0	0	0	0	0	0	10	10	12.0
36:02-1	0	0	0	0	0	0	10	10	33.0
36:02-2	1	3	0	1	1	3	4	13	5.8
501:06-1	0	0	0	0	0	0	17	17	29.0
501:06-2	0	0	0	0	0	0	15	15	40.0
512:02	10	0	0	0	0	0	0	10	.0
519:02	0	0	0	1	1	0	17	19	29.0
519:04-1	0	0	0	0	0	0	3	3	74.0
526:02	0	0	0	0	0	0	16	16	18.0
53:12	5	0	0	0	0	0	0	5	.0
543:02	12	1	0	0	0	0	0	13	.1
544:02	1	2	0	1	0	2	5	11	4.8
583:02	12	1	0	0	0	0	0	13	.1
590:02-1	0	0	0	0	0	1	16	17	34.0
590:02-2	0	0	0	0	0	0	5	5	27.0
655:02	0	0	0	0	0	0	10	10	50.0
661:02-1	0	0	0	0	0	0	3	3	30.0
661:02-2	7	2	0	1	0	0	0	10	.7
661:06	16	0	0	0	0	0	0	16	.0
684:02-1	0	0	0	0	0	0	14	14	26.0
72:10-2	0	0	0	0	0	0	18	18	34.0
743:02	4	0	0	0	1	1	3	9	4.0
769:02	25	0	2	0	0	0	0	27	.1
771:06	11	0	0	0	0	0	0	11	.0
805:02-1	0	0	0	0	0	0	17	17	25.0
805:02-2	0	0	2	1	0	4	9	16	8.4
806:02	0	0	0	0	0	0	9	9	42.0
813:02	0	0	0	0	0	0	6	6	43.0
817:04	0	0	0	0	0	0	12	12	40.0
842:02	0	0	0	0	0	0	4	4	15.0
855:08	0	0	0	0	2	0	18	20	20.0
857:04-1	6	1	3	0	2	1	2	15	2.8
857:04-2	0	0	0	0	0	0	2	2	13.0
872:04	0	0	0	0	0	0	17	17	32.0
879:04	0	0	0	0	0	0	41	41	63.0
881:12	7	4	1	1	1	1	1	16	1.5
887:04	0	0	0	0	2	0	17	19	17.0

Table 5.2 Surficial Geology Soil Types for All Full Depth Sections

SECTION	SURFICIAL GEOLOGY
1:10-1	TILL
1:10-2	TILL
1:10-3	TILL
1:12-1	TILL
1:12-2	TILL
1A:06	TILL
1A:08	TILL
12:08	TILL
14:06	TILL-MIXED CLAY,SILT,SAND
2:12-1	TILL;LACUSTRINE SAND,SILT
2:64	LACUSTRINE CLAY,SILT,SAND
2A:06	TILL
20:02-1	TILL-CLAYEY TO SANDY
20:02-2	TILL-CLAYEY TO SANDY
21A:10	TILL
22:16	TILL
3:02	OUTWASH-GRAVEL,SAND,MINOR SILT
3:12-1	LACUSTRINE SAND,SILT,CLAY
3:12-2	OUTWASH-GRAVEL,SAND,SOME SILT
36:02-1	TILL;LACUSTRINE SILT,MINOR CLAY
36:02-2	TILL;LACUSTRINE SILT,MINOR CLAY
501:06-1	TILL
501:06-2	TILL
512:02	LACUSTRINE SILT;TILL
519:02	LACUSTRINE SILT;TILL
519:04-1	LACUSTRINE SILT
526:02	TILL;LACUSTRINE CLAY
53:12	TILL
543:02	TILL
544:02	LACUSTRINE SILT AND CLAY,SAND AND SILT
583:02	TILL;LACUSTRINE SILT,CLAY
590:02-1	TILL
590:02-2	TILL
655:02	TILL-CLAY,SILT,FINE SAND;TILL-SILT,SAND;LACUSTRINE SILT,CLAY
661:02-1	LACUSTRINE SILT,CLAY
661:02-2	LACUSTRINE SILT,CLAY;TILL-CLAY,SILT,FINE SAND
661:06	LACUSTRINE SILT,CLAY
684:02-1	LACUSTRINE CLAY,SILT,SAND
72:10-2	TILL
743:02	LACUSTRINE CLAY,SILT,SAND
769:02	TILL
771:06	TILL-CLAYEY TO SANDY
805:02-1	TILL;FINE OUTWASH-SAND,SILT
805:02-2	TILL
806:02	TILL;LACUSTRINE SAND,SILT
813:02	OUTWASH-GRAVEL,SAND;(SAND SUBGRADE)
817:04	LACUSTRINE SAND,SILT
842:02	LACUSTRINE SILT,SAND AND SILT
855:08	TILL
857:04-1	TILL-CLAY,SILT,SAND
857:04-2	AEOLIAN SAND
872:04	TILL-CLAY,SILT,SAND
879:04	LACUSTRINE SAND,SILT,CLAY
881:12	TILL
887:04	TILL

Table 5.3 Independent Variables for All Full Depth Sections

SECTION	ACP THICKNESS (MM)	AGE (YEARS)	TESTS ON ORIGINAL ASPHALT PEN@25C 100G 5S (DMM)	PEN@4C 200G 60S (DMM)	VISC@60C (PA.S)	PVN	PEN RATIO	TESTS ON RESIDUE AFTER TFOT PEN@25C 100G 5S (DMM)	PEN@4C 200G 60S (DMM)	VISC@60C (PA.S)	PVN	PEN RATIO	ASPHALT STIFFNESS (KG/CM)	FREEZING INDEX C-DAYS	MINIMUM TEMP C
1:10-1	255	11	243	68	291	-.67	27	96	35	1071	-.76	36	30	1111	-37
1:10-2	255	16	277	80	318	-.31	28	149	48	669	-.55	32	9	1111	-37
1:10-3	225	14	256	68	281	-.62	26	127	41	670	-.82	32	25	1111	-37
1:12-1	255	11	190	53	600	-.24	27	112	37	1195	-.39	33	28	1111	-37
1:12-2	255	11	318	100	299	-.11	31	182	61	586	-.35	33	7	1111	-37
1A:06	175	13	267	99	454	.08	37	152	55	1054	.01	36	7	1000	-36
1A:08	200	13	256	62	272	-.66	24	119	35	671	-.93	29	20	1000	-37
12:08	255	9	269	103	434	.03	38	131	51	1193	-.12	38	11	1333	-38
14:06	250	11	270	70	303	-.42	25	156	42	579	-.64	26	25	1500	-39
2:12-1	300	7	182	56	620	-.28	30	104	38	1350	-.38	36	30	1055	-36
2:64	175	14	251	62	277	-.70	24	117	34	654	-.97	29	190	1666	-46
2A:06	225	11	275	106	441	.01	38	142	57	1108	-.06	40	7	1055	-36
20:02-1	125	15	258	58	284	-.59	22	153	37	496	-.85	24	30	1333	-38
20:02-2	125	14	271	101	438	.05	37	145	55	1044	-.09	37	10	1333	-38
21A:10	200	10	264	100	477	.12	37	143	57	1114	-.04	39	13	1444	-39
22:16	175	12	291	90	336	-.14	30	160	53	701	-.37	33	8	1055	-36
3:02	250	7	182	56	620	-.28	30	104	38	1350	-.38	36	30	777	-37
3:12-1	225	8	168	53	839	-.07	31	95	37	1916	-.15	38	50	1111	-38
3:12-2	225	8	168	53	839	-.07	31	95	37	1916	-.15	38	50	1111	-38
35:02-1	175	7	174	53	480	-.67	30	88	33	1380	-.63	37	90	944	-38
36:02-2	175	7	196	70	630	-.13	35	99		1760	-.17	0	25	944	-38
501:06-1	125	12	270	101	450	.09	37	144	56	1081	-.06	38	8	1166	-37
501:06-2	125	10	314	97	309	-.09	30	165	55	677	-.36	33	7	1166	-37
512:02	175	9	295	92	345	-.07	31	170	57	721	-.23	33	8	833	-38
519:02	175	7	173	51	480	-.68	29	83	53	1450	-.67	39	90	833	-38
519:04-1	125	15	285	87	332	-.19	30	165	53	649	-.41	32	13	833	-38
526:02	150	8	162	59	876	-.09	36	93	36	2024	-.13	38	60	888	-38
53:12	200	9	266	101	459	.08	37	139	58	1111	-.09	41	16	1611	-39
543:02	175	7	349	105	176	-.62	30	171	55	514	-.62	32	7	1000	-37
544:02	175	10	272	104	437	.07	38	150	56	1023	-.05	37	15	1166	-39
583:02	165	8	263	92	431	-.02	34	139	53	1046	-.16	38	12	1222	-38
590:02-1	150	12	271	104	429	.04	38	143	57	1089	-.97	39	12	1333	-38
590:02-2	150	11	328	104	275	-.15	31	196	64	541	-.31	32	7	1333	-38
655:02	180	7	178	42	432	-.77	23	109	29	870	-.79	26	150	1388	-41
661:02-1	175	7	285	92	358	-.10	32	148	49	863	-.27	33	25	1444	-42
661:02-2	175	7	285	92	358	-.10	32	148	49	863	-.27	33	25	1444	-42
661:06	230	7	301	96	336	-.07	31	155	55	773	-.32	35	15	1500	-41
684:02-1	175	8	288	96	345	-.12	33	152	53	812	-.29	34	60	1722	-45
72:10-2	125	15	248	55	290	-.64	22	150	37	502	-.87	24	28	1111	-37
743:02	175	7	285	98	340	-.16	34	159	54	790	-.24	33	70	1777	-46
769:02	150	10	302	107	355	.01	35	156	53	922	-.10	33	11	1444	-41
771:06	150	11	270	69	303	-.42	25	152	41	612	-.62	26	19	1388	-38
805:02-1	100	11	324	98	297	-.08	30	180	62	599	-.35	34	10	1277	-38
805:02-2	150	11	324	98	297	-.08	30	180	62	599	-.35	34	10	1277	-38
806:02	125	14	251	71	262	-.74	28	123	42	609	-.98	34	30	1222	-38
813:02	150	10	287	96	369	-.04	33	155	54	855	-.20	34	20	1666	-42
817:04	175	7	175	52	470	-.68	29	87	29	1360	-.66	33	90	1111	-38
842:02	100	8	172	71	787	-.10	41	96	36	2025	-.08	37	45	888	-38
855:08	175	13	269	98	448	.08	36	145	55	1050	-.08	37	15	1611	-39
857:04-1	175	7	258	104	440	-.03	40	141	54	1120	-.06	38	19	1611	-40
857:04-2	175	7	258	104	440	-.03	40	141	54	1120	-.06	38	19	1611	-40
872:04	150	13	273	101	431	.06	36	144	54	1058	-.09	37	15	1555	-39
879:04	100	13	265	97	455	.06	36	141	54	1085	-.10	38	10	1111	-38
881:12	175	8	261	90	450	.02	34	135	60	1135	-.12	44	20	1722	-41
887:04	175	8	262	86	433	-.02	32	125	45	1194	-.29	35	11	1166	-38

Table 5.4 Correlation Coefficient Matrix for All Full Depth Sections

	FREQ	ACPA_THI	SACPA_TH	ASTIFF	AAGE	APEN25	APEN4
FREQ	1.0000	-.4641	-.4878	.4832	.3892	-.1028	-.2453
ACPA_THI		1.0000	.9838	.0245	-.2263	-.2283	-.2381
SACPA_TH			1.0000	.0545	-.2699	-.2378	-.2303
ASTIFF				1.0000	-.2056	-.5274	-.6006
AAGE					1.0000	.3180	.0707
APEN25						1.0000	.8007
APEN4							1.0000
AVISC60							
APVN							
APR							
A_APEN25							
A_APEN4							
A_AVISC6							
A_APVN							
A_APR							
FR_INDEX							
MIN_TEMP							

	AVISC60	APVN	APR	A_APEN25	A_APEN4	A_AVISC6	A_APVN
FREQ	-.1273	-.3042	.3232	-.0655	.0843	-.1651	-.3458
ACPA_THI	.1515	-.1046	.1309	-.2694	.0123	.1351	-.0822
SACPA_TH	.1726	-.0934	.1254	-.2858	-.0016	.1628	-.0604
ASTIFF	.1576	-.5621	.4115	-.5336	.0208	.1864	-.4563
AAGE	-.4090	-.1087	.1804	.3333	.1566	-.4912	-.2901
APEN25	-.7762	.3002	-.0786	.9308	.1738	-.7792	.1164
APEN4	-.3674	.7097	-.6065	.7345	.0904	-.3224	.6281
AVISC60	1.0000	.3016	-.3344	-.6640	-.1882	.9464	.4568
APVN		1.0000	-.7483	.3616	-.0373	.2467	.9535
APR			1.0000	-.0552	.1495	-.4131	.8347
A_APEN25				1.0000	.1923	-.7607	.1776
A_APEN4					1.0000	-.2650	-.0851
A_AVISC6						1.0000	.4439
A_APVN							1.0000
A_APR							
FR_INDEX							
MIN_TEMP							

	A_APR	FR_INDEX	MIN_TEMP	
FREQ	.3469	.0490	-.2214	FREQUENCY
ACPA_THI	-.0689	-.0681	.1294	PAVEMENT THICKNESS
SACPA_TH	-.0868	-.0389	.0858	SQUARE ROOT OF PAVEMENT THICKNESS
ASTIFF	.1393	.0959	-.4892	ORIGINAL ASPHALT STIFFNESS
AAGE	.3932	-.0456	.2620	PAVEMENT AGE
APEN25	.1623	.3740	-.1432	PEN@25C 100G,5S
APEN4	-.2686	.4027	-.1488	PEN@4C 200G,60S
AVISC60	-.4419	-.2996	.1585	VISC@60C
APVN	-.4986	.2403	-.0401	PVN'
APR	.6642	-.1966	.0162	PEN RATIO
A_APEN25	.2314	.3365	-.0994	PEN@25C 100G,5S AFTER TFOT
A_APEN4	.4995	.1584	-.0420	PEN@4 200G,60S AFTER TFOT
A_AVISC6	-.5546	-.3004	.1357	VISC@60C AFTER TFOT
A_APVN	-.5946	.1644	-.0277	PVN' AFTER TFOT
A_APR	1.0000	.0213	.0312	PEN RATIO AFTER TFOT
FR_INDEX		1.0000	-.7393	FREEZING INDEX
MIN_TEMP			1.0000	MINIMUM TEMPERATURE

Table 5.5 Computer Output for Model 1

MULTIPLE REGRESSION.....

INTERCEPT	49.40269
MULTIPLE CORRELATION	.80010
STD. ERROR OF ESTIMATE	13.04828
R-SQUARED	.64017

VARIABLE NO.	MEAN	STD. DEV.	CORRELATION X VS Y	REGRESSION COEFF.	STD. ERROR OF REG.COEF.	COMPUTED T VALUE
3	12.982	1.5574	-.4878	-5.595426	1.1841	-4.7256
4	30.309	34.9835	.4832	.3550581	.0519	6.8459
5	10.109	2.7262	.3892	3.091548	.6902	4.4794
DEPENDENT 1	18.778	21.1393				

FREQ = 49.40269 + -5.595426*SACPA_THICK + .3550581*ASTIFF + 3.091548*AAGE

ANALYSIS OF VARIANCE FOR THE REGRESSION

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARES	F VALUE
ATTRIBUTABLE TO REGRESSION	3	15447.9297	5149.3086	30.2442
DEVIATION FROM REGRESSION	51	8683.1445	170.2577	
TOTAL	54	24131.0742		

WHERE FREQ IS FREQUENCY IN CRACKS/KM
SACPA_THICK IS THE SQUARE ROOT OF PAVEMENT THICKNESS IN MM
ASTIFF IS ORIGINAL ASPHALT STIFFNESS IN KG/CM^2
AAGE IS PAVEMENT AGE IN YEARS

Table 5.6 Transverse Cracking Data for All GBC Sections

SECTION	NO. OF KILOMETRES WITH INDICATED FREQUENCY 0	1	2	3	4	5	5+	TOTAL (KM)	FREQUENCY (CR/KM)
11:06	5	3	3	1	2	2	6	22	5.1
11:06&08	6	5	5	2	1	1	1	21	1.9
11:08	4	2	8	3	1	3	5	26	3.0
12:20-1	2	3	1	0	0	1	7	14	5.6
12:20-2	0	0	0	0	0	0	16	16	43.0
16:06	0	0	0	0	0	0	5	5	7.4
33:10-1	2	1	0	0	0	0	9	12	9.7
33:10-2	12	6	1	2	0	0	0	21	.2
35:16-1	0	0	0	0	0	0	6	6	35.0
35:16-3	0	0	0	0	0	0	15	15	28.0
36:06	0	0	0	0	0	0	20	20	25.0
36:10	0	0	0	0	2	2	16	20	12.0
40:12	12	6	1	4	1	0	3	27	1.7
45:08	1	4	4	3	0	0	5	19	3.4
531:02	0	0	1	0	1	0	16	18	17.0
663:08	9	1	1	0	0	0	4	15	3.1
668:02	0	0	0	0	0	0	3	3	24.0
67:18	0	0	0	0	0	0	20	20	39.0
750:02	14	2	0	0	0	0	0	16	.1
770:06	3	1	0	1	0	0	1	6	2.0
776:02	13	0	0	1	1	0	0	15	.5
794:02-1	3	0	2	3	1	2	1	12	2.8

Table 5.7 Surficial Geology Soil Types for All GBC Sections

SECTION	SURFICIAL GEOLOGY
11:06	TILL-STONEY,SANDY
11:06&08	TILL-STONEY,SANDY
11:08	TILL-MODERATELY STONEY,SANDY;LACUSTRINE SILT,CLAY,SAND
12:20-1	TILL
12:20-2	TILL
16:06	LACUSTRINE CLAY,SILT,SAND
33:10-1	LACUSTRINE SILT,FINE SAND
33:10-2	TILL-CLAY,SILT,FINE SAND
35:16-1	LACUSTRINE SILT,CLAY
35:16-3	LACUSTRINE SILT,CLAY
36:06	TILL
36:10	TILL
40:12	ALLUVIAL AND COLLUVIAL GRANULAR DEPOSITS
45:08	TILL
531:02	TILL
663:08	TILL
668:02	AEOLIAN SILT,SAND
67:18	TILL
750:02	AEOLIAN SILT,SAND
770:06	LACUSTRINE SAND,SILT AND SAND,MINOR CLAY
776:02	TILL-CLAY,SILT,FINE SAND;LACUSTRINE SILT,CLAY
794:02-1	LACUSTRINE SILT,CLAY

Table 5.8 Independent Variables for All GBC Sections

SECTION	ACP THICKNESS (MM)	AGE (YEARS)	Tests on Original Asphalt: PEN@25C 100G 5S (DMM)	PEN@4C 200G 60S (DMM)	VISC@60C (PA.S)	PVN	PEN RATIO	Tests on Residue after TFOT: PEN@25C 100G 5S (DMM)	PEN@4C 200G 60S (DMM)	VISC@60C (PA.S)	PVN	PEN RATIO	ASPHALT STIFFNESS (KG/CM)	FREEZING INDEX C-DAYS	MINIMUM TEMP C
11:06	100	8	288	94	349	-.11	32	151	52	825	-.29	34	9	1166	-37
11:06&08	100	7	267	89	390	-.12	33	134	59	960	-.33	44	12	1166	-37
11:08	100	10	269	91	350	-.24	33	149	50	826	-.31	33	10	1222	-37
12:20-1	100	7	265	103	450	.05	38	135	56	1140	-.12	41	15	1611	-39
12:20-2	125	8	182	58	699	-.14	31	96	36	1799	-.20	37	50	1611	-39
16:06	150	8	170	72	781	-.14	42	82	32	2383	-.17	39	70	1277	-41
33:10-1	100	11	265	100	460	.08	37	141	57	1118	-.06	40	27	1444	-43
33:10-2	100	11	265	100	460	.08	37	141	57	1118	-.06	40	27	1444	-43
35:16-1	100	7	301	99	340	-.05	32	156	55	810	-.25	35	65	2777	-47
35:16-3	100	7	301	99	340	-.05	32	156	55	810	-.25	35	65	2777	-47
36:06	100	7	164	53	730	-.28	32	86	32	1820	-.37	37	40	1055	-39
36:10	125	7	300	102	330	-.10	33	157	54	800	-.25	34	13	1333	-39
40:12	100	7	268	85	320	-.36	31	154	56	750	-.36	36	7	944	-35
45:08	65	8	284	109	404	.05	38	150	65	996	-.08	43	14	1666	-41
531:02	100	8	165	64	845	-.10	38	99	42	1878	-.10	42	60	888	-38
663:08	125	8	295	109	340	-.09	36	165	65	754	-.23	39	15	1722	-41
668:02	200	8	255	89	453	-.02	34	120	44	1325	-.16	36	50	1444	-44
67:18	175	7	291	97	350	-.08	33	156	56	770	-.31	35	70	2509	-47
750:02	150	8	275	90	428	.06	32	133	53	1189	-.10	39	55	1666	-45
770:06	150	8	299	94	327	-.12	31	168	61	735	-.23	36	11	1388	-39
776:02	125	8	265	90	441	.02	33	140	52	1073	-.12	37	20	1444	-41
794:02-1	175	7	167	55	750	-.22	32	96	36	1820	-.19	37	60	1444	-40

SECTION	ASPHALT STIFFNESS (KG/CM)	FREEZING INDEX (C-DAYS)	MINIMUM TEMP (C)	GBC THICKNESS (MM)
11:06	9	1167	-37	300
11:06&08	12	1167	-37	300
11:08	10	1222	-37	300
12:20-1	15	1611	-39	125
12:20-2	50	1611	-39	100
16:06	70	1278	-41	450
33:10-1	27	1444	-43	275
33:10-2	27	1444	-43	275
35:16-1	65	2788	-47	350
35:16-3	65	2788	-47	250
36:06	40	1056	-39	250
36:10	13	1333	-39	150
40:12	7	944	-35	150
45:08	14	1677	-41	300
531:02	60	889	-38	100
663:08	15	1722	-41	125
668:02	50	1444	-44	125
67:18	70	2500	-47	100
750:02	55	1667	-45	200
770:06	11	1389	-39	200
776:02	20	1444	-41	150
794:02-1	60	1444	-40	100

Table 5.9 Independent Variables for Full Depth Sections
With As-built Pavement Characteristics

SECTION	ASPHALT CONTENT (%)	FIELD MST (KG/M)	MST AIR VOIDS (%)	MST VMA (%)	COMPACTION (%)	CORE DENSITY (KG/M)	CORE AIR VOIDS (%)	MINUS #4 (%)	MINUS #200 (%)
1:10-1	5.6	2352	3.0	13.5	94	2218	8.6	53	12.5
1:12-1	6.4	2317	3.7	15.8	94	2200	8.6	62	6.9
1:12-2	6.5	2329	3.4	15.4	97	2266	5.7	63	6.8
1A:06	5.1	2413	2.7	12.8	94	2283	7.9	53	7.7
1A:08	5.3	2411	2.4	13.3	96	2324	5.9	48	7.4
12:08	7.0	2276	4.9	17.2	97	2219	7.3	49	7.4
2:12-1	5.4	2395	2.8	11.9	95	2282	7.4	47	10.1
21A:10	6.4	2292	4.2	14.9	96	2212	7.6	58	5.0
3:02	5.7	2363	4.0	14.2	95	2260	8.1	58	7.2
36:02-1	5.9	2340	4.2	13.9	93	2190	10.3	63	9.8
36:02-2	5.6	2364	3.9	14.2	95	2265	7.9	51	5.9
512:02	5.3	2358	4.1	13.7	96	2283	7.1	45	5.5
519:02	5.3	2359	4.1	13.9	95	2242	8.8	51	8.5
53:12	6.2	2315	4.3	13.9	94	2181	9.9	44	5.9
543:02	5.9	2356	3.6	13.1	93	2212	9.4	53	12.5
544:02	6.[illegible]	2298	4.8	16.2	96	2228	7.7	46	5.7
583:02	6.7	2297	4.4	15.3	96	2217	7.7	58	7.9
590:02-1	6.3	2287	4.8	17.0	98	2249	6.4	54	3.9
590:02-2	5.8	2380	1.2	12.4	92	2207	8.4	61	8.9
655:02	4.7	2317	5.2	15.0	97	2248	8.1	44	6.8
661:02-1	5.7	2308	5.1	16.0	96	2232	8.3	49	8.9
661:02-2	5.7	2308	5.1	16.0	96	2232	8.3	49	8.9
684:02-1	5.9	2347	3.8	15.0	97	2281	6.5	47	4.5
743:02	5.1	2358	4.6	14.7	96	2264	8.4	57	7.2
769:02	7.2	2327	2.3	16.3	95	2229	6.4	51	7.5
771:06	7.1	2311	2.2	15.1	96	2225	5.9	54	11.3
805:02-2	5.9	2353	2.2	13.5	91	2158	10.3	63	9.8
813:02	8.3	2353	6.1	20.7	88	2086	13.0	69	10.7
817:04	5.3	2396	2.8	14.1	94	2266	8.1	48	8.6
872:04	6.3	2343	2.7	14.3	94	2221	7.8	60	6.7
881:12	5.6	2361	2.0	12.9	92	2179	9.6	62	7.2
887:04	6.1	2370	2.9	14.4	94	2248	7.9	51	9.5

Table 5.10 Computer Output for Model 4

```
MULTIPLE REGRESSION.....

INTERCEPT                     153.27858
MULTIPLE CORRELATION             .77590
STD. ERROR OF ESTIMATE         11.23816
R-SQUARED                        .60202
```

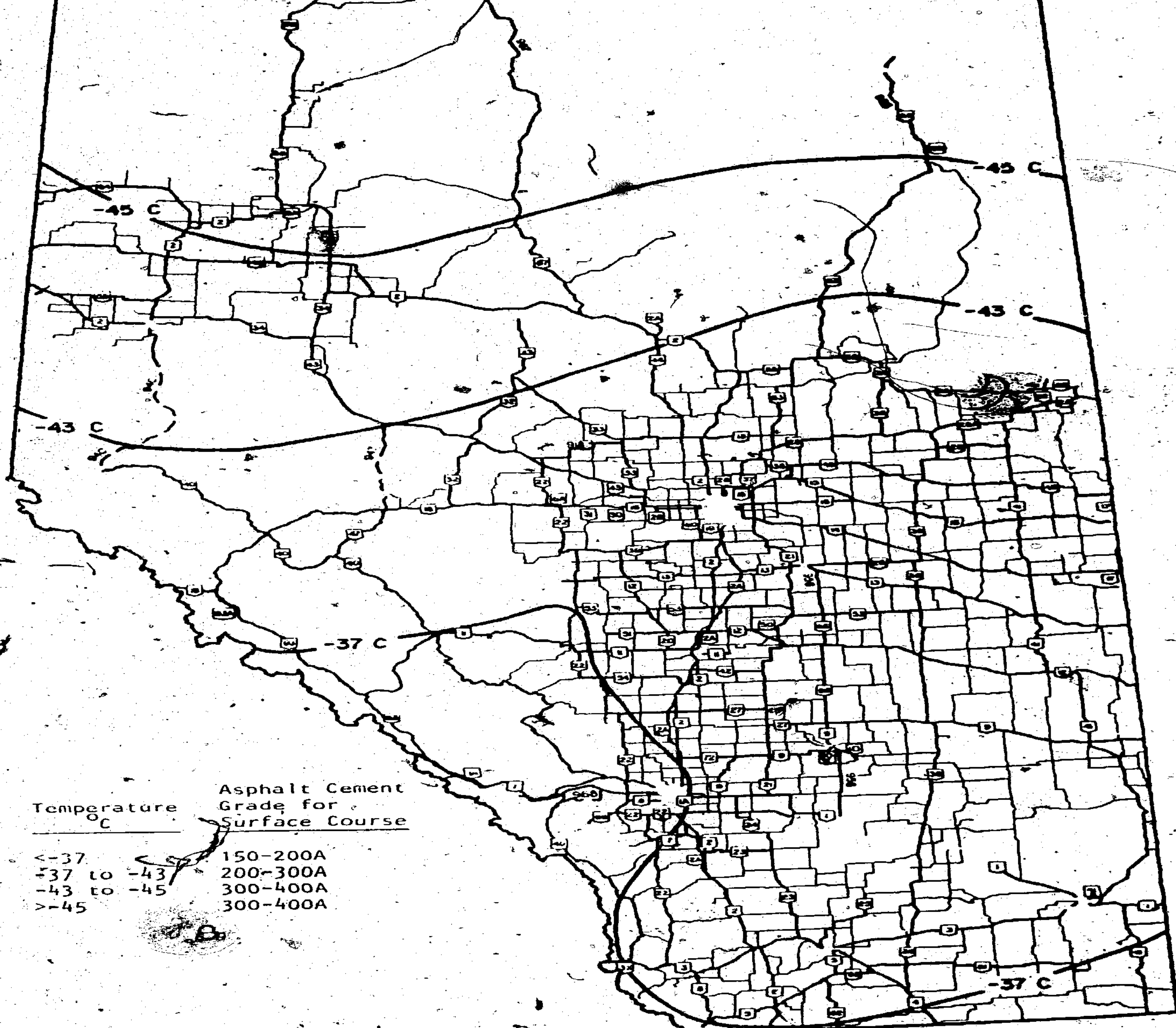
VARIABLE NO.	MEAN	STD. DEV.	CORRELATION X VS Y	REGRESSION COEFF.	STD. ERROR OF REG.COEF.	COMPUTED T VALUE
4	30.750	33.0435	.5634	.3966702	.0710	5.5845
5	9.156	2.0652	-.0209	2.649047	1.1288	2.3468
18	14.706	1.6999	.3107	3.323100	1.1907	2.7909
19	94.750	2.0478	-.2022	-2.373567	.9957	-2.3839
DEPENDENT						
1	13.706	16.6251				

```
FREQ    =    153.2786          +    .3966702*ASTIFF
        +    2.649047*AAGE     +    3.323100*FVMA
        +    -2.373567*COMP
```

ANALYSIS OF VARIANCE FOR THE REGRESSION

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARES	F VALUE
ATTRIBUTABLE TO REGRESSION	4	5158.2461	1289.5615	10.2106
DEVIATION FROM REGRESSION	27	3410.0000	126.2963	
TOTAL	31	8568.2461		

```
WHERE    FREQ IS FREQUENCY IN CRACKS/KM
         ASTIFF IS ORIGINAL ASPHALT STIFFNESS IN KG/CM²
         AAGE IS PAVEMENT AGE IN YEARS
         FVMA IS FIELD MARSHALL VMA IN %
         COMP IS AS-BUILT PAVEMENT PERCENT COMPACTION
```



Temperature °C	Asphalt Cement Grade for Surface Course
<-37	150-200A
-37 to -43	200-300A
-43 to -45	300-400A
>-45	300-400A

Fig. 5.1 Design Map for Selecting Asphalt Cement Grade for Surface Courses

CHAPTER 6

PHASE II - CASE STUDIES

6.1 Introduction

Phase II of the investigation focused on a select few projects exhibiting peculiar transverse cracking behavior. The objective of this phase was to relate original as-supplied and as-built materials characteristics and in-place materials characteristics after several years of aging to actual pavement performance in terms of low-temperature transverse cracking. These findings could be used to supplement the statistically based analyses of Phase I. Phase I could not adequately evaluate the influence of subgrade effects on transverse cracking due to limitations in both identifying and quantifying subgrade soil characteristics. Phase II was designed to provide more definitive relationships between cracking performance and subgrade materials.

Ideally, to correlate in-place asphalt binder or mix characteristics with transverse cracking performance, the pavement should be sampled and tested at the time transverse cracking is initiated. In this investigation this was not the case. The pavement was sampled, in the case of cracking sections, at some time after cracking had occurred and the materials characteristics identified would not reflect the critical characteristics at the inception of cracking. However this data could allow some conservative criteria to be established. Three projects were selected for a more extensive field sampling and laboratory testing and evaluation program. These three projects were:

1. S[illegible]rt Assiniboine to North of Fort Assiniboine
2. Hwy 33:10 South of Swan Hills to West of Fort Assiniboine
3. SR 845:02 Raymond to South of Coaldale

6.2 Field Sampling and Testing Program

A field sampling and laboratory testing program was designed such that the characteristics of the materials used in the pavement, granular base (if applicable) and underlying subgrade could be adequately identified and quantified.

Each project was broken into sections that represented significantly different cracking and non-cracking sections. In general, five randomly selected locations within each section were cored and the pavement and base course retrieved for laboratory testing. At each core location, a truck mounted drill rig was used to establish the subgrade soil profile to a 2 metre depth and to take representative soil samples of each horizon for laboratory classification. It was assumed that only the top 2 metres of subgrade would have any real influences on pavement cracking.

The samples were tested in the Alberta Transportation Laboratory as outlined in Table 6.1.

6.3 Discussion of Each Project

Detailed reports presenting all test results of the laboratory testing program, summaries of as supplied asphalt characteristics and construction quality control test results are included in Apendices A through C. A brief summary of each project and the significant factors that are thought to influence the observed cracking performance is included in this Chapter.

6.3.1 SR 661:02

This 175 mm full depth project was constructed on an existing subgrade in 1979. AC 275 was supplied to the project by Imperial Oil (Edmonton). The first 3 km of the project, SR 666:02-1, had an average frequency of 30 cracks/km compared to the last 9 kilometres which had a total of 6 cracks including five kilometres without any cracks at all. The actual transverse crack counts are shown in Table A.[illegible]. A review of construction records indicated no construction related influences that could account for the differences in cracking behavior.

A comparison of the results of testing of the pavement cores from both sections shows that pavement thicknesses and recovered asphalt characteristics are virtually identical. It can therefore be concluded that the observed differences in cracking behavior are not a function of the asphalt pavement. A review of the subgrade profiles of the two sections does not highlight any significant differences in soil types between the two sections although the clay soils in the non-cracking section tend to be slightly more plastic (higher PI) and wetter (field moisture content relative to the plastic limit of the soil). Field borings should likely have extended to a greater depth.

The results of the testing program were unable to identify significant differences between the two sections. However the asphalt characteristics, both as-supplied and recovered did highly influence the behavior of the non-cracking section. The stiffnesses of the original asphalt supplied and of the recovered asphalt was determined using van der Poel's nomographs as modified by Huekelom and by McLeod. These values have been compared to criteria developed by McLeod (25), Gaw (26) and Fromm and Phang (27). These data are presented in Appendix A and indicate that neither the stiffness of the mix using original asphalt characteristics nor the stiffness of the recovered asphalt exceed these critical stiffness values.

This would therefore suggest that the significant length of non-cracking pavement can be attributed to the asphalt cement originally used and that construction operations and in-place aging effects were not extreme enough to result in recovered asphalt stiffnesses exceeding some critical level. The more severe cracking performance of the cracking section can only be attributed to subgrade soil and moisture characteristics that were either not adequately measured by conventional soil classification parameters or resulted from influences below a 2 m depth.

6.3.2 Hwy 33:10

This 100 mm asphalt concrete pavement was constructed in 1975. The pavement was placed over two adjoining granular bases constructed in 1973 and 1974 which used different aggregate sources. AC 275 was supplied to the project by Husky Oil (Lloydminster). Originally it appeared that the observed cracking performance was related to the year of construction and/or aggregate source used for each granular base project. The pavement over the granular base constructed in 1973, 33:10-1, averaged 9.7 cracks/km whereas the pavement over the 1974 granular base, 33:10-2, averaged 0.2 cracks/km including 12 crackfree kilometres. In addition the general topography of the two sections was different. Section 33:10-2 shows level, fairly well drained topography and Section 33:10-1 is rolling with a higher frequency of cuts and fills. A review of the surficial geology map for the area (21) shows that the boundary between the two sections coincides with a boundary between surficial geology units.

Due to cold weather coring conditions, representative samples of the granular base course of the two sections could not be recovered. However the results of testing carried out during crushing operations in 1973 and 1974 did not indicate any significant differences in aggregate gradations.

A comparison of recovered asphalt characteristics from the two sections identified statistically significant differences although only one refinery supplied the asphalt cement and the same aggregate source and asphalt mix plant was used. Scrutinization of construction quality control tests and the sequence of paving operations did not offer any reason for the apparent differences in asphalt aging between the two sections. It should be noted that the "harder", more aged asphalt was recovered from the cracking section.

The boundary between the cracking and non-cracking sections coincides with the job split between granular base projects and with a boundary between surficial geology units. The cracking frequency of 33:10-1 is not consistent from kilometre to kilometre as shown in Table B.1. Field inspection indicates that there is a higher incidence of transverse cracking in cut sections and side-hill cuts. Hole number 3 was drilled in an area of high cracking frequency and the subgrade soil profile indicates sand is encountered below 1 m.

In summary the results of the testing program indicate significant but unexplained differences in recovered asphalt characteristics although both sections were constructed at one time. Comparison of nomograph stiffness for the two sections to published proposed stiffness limits and to values determined for 661:02 suggest that the pavement is not the cause of the observed cracking behavior of 33:10-1. The asphalt characteristics did, however, influence the behavior of the non-cracking section. It is concluded that subgrade characteristics are the primary factor influencing the observed cracking of 33:10-1.

6.3.3 SR 845:02

This 100 mm full depth pavement was originally constructed in 1973 and 1974 and overlaid with 60-100 mm of asphalt concrete in 1980. It was felt that the observed transverse cracking was not being influenced by the 1980 overlay and reflected the actual low temperature performance of the original pavement constructed. As shown in Tables C.1 and C.2 the cracking behavior appears to be a function of the year of construction of the original pavement with the section, 845:02-1, constructed in 1973 exhibiting no cracking and the pavement constructed in 1974 exhibiting significant cracking. The same refinery, Husky Oil (Lloydminster) supplied AC 275 to both years of construction and test results of as supplied asphalt indicate identical properties. Different aggregate sources were used in the two years of construction but a review of construction quality control testing records shows very similar gradations. The same Contractor using the same asphalt plant was used for the construction of both sections.

Recovered asphalt characteristics from the 1973 and 1974 pavements indicate that the pavement constructed in 1974 has aged significantly more than the 1973 pavement. A detailed review of quality control testing carried out during construction does not offer any explanation for the differences in recovered asphalt characteristics between the two sections. However, the results are consistent with the observed cracking performance with the section with the "harder" recovered asphalt cracking significantly.

Coring identified that the non-cracking section included an extra 100 mm of pavement that had been apparently placed prior to the 1973 construction. Given very similar as constructed materials characteristics between the two sections, it would be expected that the thicker structure of 845:02-1 would be more resistant to the initiation of transverse cracking. Nomograph stiffnesses of the original asphalt supplied to both years of construction are less than published proposed critical stiffness values and are similar to values determined for SR 661:02 and 33:10 suggesting that the asphalt was of low cracking potential. The relatively thin 100 mm pavement structure constructed in 1974 was likely the determining cause for the differences in cracking behavior between the two sections.

Within the portion of the project constructed in 1974, significant differences in cracking frequencies exist. Section 845:02-2 has an average cracking frequency of 21 cracks/km compared to 90 cracks/km for 845:02-3. This could be explained by the differences in pavement thickness between the medium, 845:02-2, and heavy, 845:02-3, cracking sections of 220 mm and 180 mm respectively. The boundary of the heavy cracking section coincide [illegible] split in surficial geology units. The heavy cracking section is constructed over post-glacial wind deposited sands and silts. This is confirmed by the subgrade soils profiles which show this section is constructed over clayey sands. The medium cracking section is constructed over medium plastic clay.

In summary, the main factor influencing the differences in cracking behavior between the 1973 and 1974 pavement is likely the thicker pavement structure of the non-cracking section. It is assumed that the 100 mm of pavement constructed prior to 1973 was not cracked when overlaid in 1973. The differences in recovered asphalt characteristics is also consistent with the cracking performance although why these differences would exist considering they were built by the same Contractor using apparently identical asphalt cements supplied by the same refinery and very similar aggregates cannot be ascertained.

The significant differences in cracking within the section constructed in 1974 are a result of subgrade influences with the pavement constructed over a granular sandy subgrade exhibiting about 4 1/2 times the cracking frequency of the section constructed over a medium plastic clay.

6.4 General Discussion

Phase I of this investigation developed a model that identified pavement age, pavement thickness and stiffness of the original asphalt supplied as the three most significant factors that were associated with the observed transverse cracking frequencies on 55 full depth pavement sections. Attempts were made to include the influence of the underlying subgrade within the model. Only a few sections could be identified as having sand or granular subgrades based on surficial geology maps. Due to the small number of sections, their inclusion or exclusion from the analysis did not significantly effect the final model developed.

Phase II focused in more detail on 3 projects. It appeared that subgrade effects were a large factor in influencing the difference in observed cracking behavior between sections. For Sections 33:10-2 and SR 845:02-3 sand subgrades were associated with higher frequencies of transverse cracking as compared to adjoining sections.

The association of transverse cracking of asphalt pavements with sand subgrades is well known. However the mechanism has not been extensively researched. Work by Hamilton (28) in the laboratory identified that compacted clays can undergo considerable freezing shrinkage depending upon the moisture and density conditions prior to freezing. This evaluation was confined to 5 Alberta clay soils and did not investigate freezing shrinkage of sandy soils. Rix (29) observed on three instrumented sections in Alberta that there was a relationship between crack width increases and decreases with increasing or decreasing thickness of the frozen layer. This suggests that subgrade soils will contract during cooling (freezing) conditions similarly to asphalt pavements and that the extent of contraction will be a function of soil type, moisture and density conditions and depth of frost penetration. If the tensile strength of the soil structure is exceeded, cracks will develop which will reflect through the overlying pavement structure.

In Alberta, cracks in an existing pavement reflect relatively quickly through a new overlay, often during the first winter. Various techniques have been investigated to try to control reflective cracking of asphalt overlays with little success. It is suggested that the mechanism of the reflective cracking of an asphalt overlay over a cracked highway would be similar to the cracking of a new pavement constructed over a sand subgrade and that the treatment of both conditions may be similar.

On the three projects investigated in Phase II, there were also significant sections of very similar pavements that did not crack at all. This suggests that the tensile strength of the pavement was never exceeded by induced thermal stresses. A critical stiffness of 2.9×10^6 Pa (30 kg/cm^2) based on original asphalt characteristics and a pavement temperature at a 50 mm pavement depth based on McLeod's method has been identified as a stiffness value separating acceptable from non-acceptable transverse cracking. A review of the stiffnesses of the original asphalt supplied for the non-cracking sections confirm this limit. However, pavement thicknesses and construction related or in-place aging and their effect on asphalt stiffness will govern overall cracking frequency. As shown, nomograph stiffness values increased with TFOT conditioning and in-place aging. This has been at least partially accommodated in the model developed by the inclusion of a pavement age variable. It is conceivable that even with original asphalt stiffnesses well below the critical limit identified, that asphalt pavements will crack to an unacceptable level if asphalt pavement structures are too thin or if excessive hardening due to construction effects increases the asphalt mix stiffness to an unacceptable level.

The performance of the cracking and non-cracking sections of SR 661:02 and Hwy 33:10 would indicate that subgrade soil influences tend to govern both the initiation of and frequency of transverse cracking even when original asphalt stiffnesses are sufficiently low that cracking wouldn't be expected. This would suggest that for the asphalt grades and types normally used by Alberta Transportation, excessive transverse cracking could be expected on asphalt pavements constructed over sand subgrades.

Table 6.1 Laboratory Testing Program

Asphalt Pavement Cores

Visual description
Measurement of lift thicknesses
Density measurement of each lift
Asphalt extraction of top lift and Abson recovery

Tests on recovered asphalt:

- absolute viscosity @ 60°C
- kinematic viscosity @ 135°C
- penetration @ 25°C, 100 g, 5 s
- penetration @ 4°C, 100 g, 5 s
- penetration @ 4°C, 200 g, 60 s

Calculation of temperature susceptibility parameters:

- PVN' based on penetration @ 25°C and absolute viscosity @ 60°C
- PVN based on penetration @ 25°C and kinematic viscosity @ 135°C
- PI based on penetration @ 25°C and penetration @ 4°C, 100 g, 5 s

Sieve Analysis of Extracted Aggregate

Subgrade Soils

Visual description
Sieve Analysis
Classification (Unified System as modified by P.F.R.A.)

CHAPTER 7

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

7.1 Summary

In Phase I of this investigation, a mathematical model has been developed that associates pavement characteristics with the observed low temperature transverse cracking of 55 full depth pavement sections. The three major, most statistically significant variables identified that had the greatest influence on the frequency of transverse cracking are:

- pavement thickness
- original asphalt stiffness predicted using McLeod's method and based on site specific temperature conditions
- pavement age.

The model developed took the form:

$$\begin{aligned}\text{Frequency (cracks/km)} = {} & 49.40 + 3.09 \text{ (pavement age in years)} \\ & + 0.36 \text{ (original asphalt stiffness in kg/cm}^2\text{)} \\ & - 5.60 \text{ (pavement thickness in mm)}^{0.5}\end{aligned}$$

A similar, less statistically significant, model has also been developed based on the cracking behavior of 77 sections (55 full depth and 22 granular base pavement sections). Analysis of 32 selected full depth projects including as-built pavement characteristics resulted in a model of less statistical significance but suggests that Marshall V.M.A. and pavement percent compaction may influence cracking frequency. Due to the difficulty of identifying and quantifying subgrade and soil characteristics, this variable was not included in the analysis although it was recognized that subgrade charatteristics can significantly affect transverse crack frequencies.

Based upon a review of the data available for the 77 sections used to develop the models and on the case studies of Phase II, a critical original asphalt stiffness of 2.9×10^6 Pa (30 kg/cm²) has been identified that is assumed to separate "acceptable" from "non-acceptable" transverse cracking. By separating the major inputs of asphalt characteristics and temperature that are used to predict asphalt stiffnesses, critical ambient air temperatures have been identified for each major grade and type of asphalt used by Alberta Transportation. A design map has been developed, based upon these critical temperatures, that can be used in selecting the appropriate asphalt grade and type for surface and lower pavement courses that would optimize the low temperature performance of the pavement. To minimize low temperature transverse cracking, it is identified that the thickness of the asphalt pavement structure should be maximized.

Phase II focuses in more detail on the particular cracking performance of three projects exhibiting significant lengths of both cracking and non-cracking within each project. Field sampling and laboratory testing identified that subgrade effects were a large factor in influencing the differences in observed cracking with pavement thicknesses also having some effect. It appeared that asphalt characteristics were of the greatest influence in governing the behavior of the sections that did not exhibit any transverse cracking or exhibited only very low frequencies of transverse cracking.

7.2 Conclusions

The following conclusions can be drawn based upon the results of this investigation:

1. The major factors influencing the low temperature transverse cracking behavior of the pavement sections studied are:
   - pavement thickness
   - original asphalt stiffness using McLeod's method and based on site specific temperature conditions
   - pavement age
   - subgrade soil characteristics.

   It has been also identified that asphalt mix and pavement characteristics may also influence cracking behavior.

2. A critical asphalt stiffness of 2.9×10^6 Pa (30 kg/cm^2), predicted using Mcleod's method and based on original asphalt characteristics and site specific temperature conditions, can be used to separate "acceptable" from "non-acceptable" transverse cracking behavior.

This value has been used to develop design maps for Alberta which can be used to select the most appropriate asphalt cement grade and type for both surface and lower pavement courses to optimize low temperature pavement performance. To minimize the frequency of transverse cracking, the thickness of the asphalt pavement should be maximized.

Further, subgrade soil influences will govern the initiation and frequency of transverse cracking. With the asphalt grades and types normally used by Alberta Transportation, excessive transverse cracking can be expected on pavements constructed over sand subgrades.

3. Although a full network survey was not carried out, the results of this investigation would suggest that the changes made to asphalt specifications in 1967 by introducing a minimum viscosity limit were beneficial and have resulted in pavements exhibiting lower transverse cracking frequencies than experienced by the Department prior to 1967.

4. Changes to asphalt specifications in 1980 that made minor modifications to penetration and viscosity limits and that introduced limits for medium viscosity or more temperature susceptible asphalts for use in lower pavement courses will allow continued flexibility in optimizing both high and low performance of asphalt pavements designed by Alberta Transportation.

7.3 Recommendations

1. The design map developed as part of this investigation should be used to aid in the selection of the appropriate asphalt cement grade and type to optimize the low temperature performance of asphalt pavements designed by Alberta Transportation.

2. In order to minimize the frequency of low temperature transverse cracking, the thickness of the asphalt pavement should be maximized.

3. The "softest" grade and type of asphalt cement that will minimize low temperature transverse cracking and still provide acceptable high temperature performance should be selected. This will provide maximum economic benefits in terms of pavement riding quality, maintenance costs and construction costs.

4. Further research is required to evaluate the high temperature performance characteristics of the asphalt cements used by Alberta Transportation to ensure that the design process used to select asphalt grade and type addresses both high and low temperature performance appropriately. The effects of aggregate and asphalt mix properties on high temperature performance should be included in the investigation.

5. Further research is required to identify the effects of subgrade influences on low temperature transverse cracking considering the thermal and moisture regimes and stiffnesses of underlying

subgrade materials.

6. Further research examining the mechanisms of low temperature transverse cracking and the prediction of pavement cracking temperatures should be undertaken.

7. Further research is required to identify the mechanism and appropriate treatments to control and minimize reflection cracking of new asphalt overlays placed over existing, cracked pavements.

REFERENCES

1. Kathol, B., "Transverse Cracking in Alberta", Western Association of Canadian Highway Officials Conference, 1968

2. Canadian Good Roads Association, Ad Hoc Committee on Low Temperature Behavior of Flexible Pavements, "Low Temperature Pavement Cracking in Canada: The Problem and Its Treatment", Annual Convention, Canadian Good Roads Association, 1970

3. Asphalt Institute Ad Hoc Committee "Design Techniques to Minimize Low-Temperature Asphalt Pavement Transverse Cracking", Asphalt Institute, Research Report No. 81-1, 1981

4. Shields, B.P., K.O. Anderson and J.M. Dacyszyn, "An Investigation of Low Temperature Transverse Cracking of Flexible Pavements", Proceedings, Canadian Good Roads Association, 1969

5. Young, F.D., I. Deme, R.A. Burgess and O. Kopvillem, "Ste. Anne Test Road--Construction Summary and Performance After Two Years Service", Proceedings, CTAA, 1969

6. McLeod, N.W., "Reduction of Transverse Cracking by Use of Softer Asphalts", Proceedings, CTAA, 1968

7. van der Poel, C., "A General System Describing the Viscoelastic Properties of Bitumen and Its Relation to Routine Test Data", Journal of Applied Chemistry, Vol. 4, Pt 5, 1954

8. Haas, R.C.G., "A Method for Designing Asphalt Pavements to Minimize Low Temperature Shrinkage Cracking", Asphalt Institute, Research Report No. 73-1, 1973

9. McLeod, N.W., "A 4-Year Survey of Low Temperature Transverse Pavement Cracking on Three Ontario Test Roads", Proceedings, AAPT, 1972

10. Gaw, W.J., R.A. Burgess and F.D. Young, "Road Performance After Five Years and Laboratory Predictions of Low Temperature Performance", Prodeedings, CTAA, 1974

11. McLeod, N.W., "Asphalt Cements: Pen-Vis Number and Its Application to Modulii of Stiffness", Journal for Testing and Evaluation, ASTM, Vol. 4, 1976

12. Fromm, H.J. and W.A. Phang, "A Study of Transverse Cracking of Bituminous Pavements", Proceedings, AAPT, 1972

13. Alberta Transportation, "Asphalt Concrete Pavement Manual", MEB 3

14. Finn, F.N., K. Nair and J.M. Hilliard, "Minimizing Premature Cracking in Asphaltic Concrete Pavement", NCHRP Report 195, 1978

15. Environment Canada, "Principal Station Data", Atmospheric Environment Service

16. National Research Council, Associate Committee on the National Building Code, "Climatic Information for Building Design in Canada-1975 -- Supplement to the National Building Code", NRC No. 13986.

17. Alberta Transportation, "Wax Contents of Asphalt Products - 1970-1973" Unpublished Report, 1973

18. Kromm, C.J., "Determination of the Wax Content of Bitumens", Journal of the Institute of Petroleum, August 1968

19. Deme, I., F.D. Young and R.W. Culley, "Low Temperature Performance of Composite Asphalt Concrete Pavements", Proceedings, CTAA, 1974

20. Asphalt Institute, "Mix Design Methods for Asphalt Concrete", Manual Series No. 2 (MS-2), May 1984

21. Information Builders Inc., "FOCUS Users Manual -- Release 4.5" July 1984

22. Draper, N.R. and H. Smith, "Applied Regression Analysis", John Wiley & Sons, Inc., 1966

23. Kennedy, J.B. and A.M. Neville, "Basic Statistical Methods for Engineers and Scientists", A Dun-Donnelley Publisher, 1976

24. van der poel, C., "Time and Temperature Effects on the Deformation of Asphaltic Bitumens and Bitumen-Mineral Mixtures", Society of Plastic Engineers Journal, Vol. 11, No. 7, 1955

25. McLeod, N.W., "Prepared Discussion on Ste Anne Test Road -- Construction Summary and Performance After Two Years Service" Proceedings, CTAA, 1969

26. Gaw, W.J., "Measurement and Prediction of Asphalt Stiffness and Their Use in Developing Specifications to Control Low-Temperature Pavement Transverse Cracking", ASTM Special Technical Publication 628, 1976

27. Fromm, H.J. and W.A. Phang, "Temperature Susceptibility Control in Asphalt Cement Specifications", Highway Research Board, HRR 350, 1971

28. Hamilton, A.B., "Freezing Shrinkage of Compacted Clays", Canadian Geotechnical Journal, Vol. 3, No. 1, February 1966

29. Rix, H.H., "Vertical Movements and Crack Width Changes on Highway Pavement Surfaces", Canadian Geotechnical Journal, Vol. 6, No. 3, August 1969

30. Alberta Environment, "Climate of Alberta -- Tables of Temperature, Precipitation and Sunshine"

31. Department of Energy, Mines and Resources, "Surficial Geology Whitecourt" Geological Survey of Canada, MAP 1367A

32. Department of Energy, Mines and Resources, "Surficial Geology Lethbridge (East Half)", Geological Survey of Canada, MAP 41-1962

APPENDIX A - CASE STUDY - PROJECT SR 661:02

Project SR 661:02

Fort Assiniboine to North of Fort Assiniboine

km 2.2 - km 14.4

A.1 Transverse Cracking Performance

This full depth project, constructed in 1979, is exhibiting non-uniform transverse cracking. The first 3 km (km 2.2-km 5.2), SR 661:02-1, has an average cracking frequency of 30 cracks/km. The last 9 km, SR 661:02-2, has a total of 6 transverse cracks. Five of these kilometres are crackfree. An initial review of construction records did not indicate any construction related influences eg. paving sequence, location of dead haul roads, etc., that could account for the differences in cracking behavior between the two sections. A review of construction quality control test data showed very similar materials characteristics over the length of the project. It was postulated that the cracking behavior was subgrade related although no noticeable change in topography or land use could be observed. The boundary separating the two sections is defined by a creek running through a low lying slough/muskeg.

A.2 Project Details

Location: The project is located in Central Alberta approximately 120 km northwest of Edmonton.

Local Climate: The average winter low temperature over the last 10 years was -43°C. The extreme record low was -48°C, occurring in 1980 (30). The freezing index is 1444°C days.

Surficial Geology: The topography is generally flat to gently rolling. Surficial materials are silt and clay glacial lake deposits with some sand deltaic deposits (31). Some localized low lying muskegs are evident. A study of the air photos for this does not identify a noticeable change in surficial geology at the interface between the two sections.

Traffic: AADT (1984) = 610

Equivalent 18 kip (80 kN) single axle applications = 23

Construction: This 175 mm full depth pavement structure was constructed in 1979 over an existing subgrade. The structure was made up of 50 mm of 16 mm topsize surface course over 125 mm of 20 mm topsize base pavement. AC 27.5 supplied by Imperial (Edmonton) was used for the entire project. Average results from the construction quality control testing program were:

- mix temperature at plant discharge = 116°C
- asphalt content = 5.7 percent (dry wt of aggregate)
- compaction = 96.7 percent (average core density of 2232 kg/m³)

Field Sampling and Laboratory Testing Program: Five 150 mm diameter cores at randomly selected locations were taken of the complete pavement structure in both the cracking and non-cracking sections. At each of these core locations, the underlying subgrade was sampled to a 2 m depth.

Only the top lift of each core was tested. Bulk densities were measured and extraction tests run. The asphalt cement was recovered using the Abson method and then tested for:

- penetration at 25°C, 100 g, 5 s
- penetration at 40°C, 100 g, 5 s
- penetration at 40°C, 200 g, 60 s
- absolute viscosity at 60°C
- kinematic viscosity at 135°C

Temperature susceptibility parameters, PVN', PVN and PI (based on penetrations at 25°C and 4°C, 100 g, 5 s) (14) were calculated.

The subgrade soil samples were tested for moisture content and routine classification testing was carried out.

A.3 Test Results

The results of the testing program and other observations are presented in the following Tables and Figures:

Table A.1	Transverse Crack Counts
Table A.2	Summary of Test Results of Asphalt Supplied in 1979
Table A.3	Summary of Test Results of Recovered Asphalt
Table A.4	Summary of Test Results of Pavement Cores

Figure A.1 Subgrade Soil Profiles - SR 661:02-1

Figure A.2 Subgrade Soil Profiles - SR 661:02-2

Table A.5 Summary of Soil Survey Test Results - SR 661:02-1

Table A.6 Summary of Soil Survey Test Results - SR 661:02-2

Table A.7 Summary of Stiffness Calculations

Figure A.3 Asphalt Penetration - Viscosity Relationships

A.4 Discussion

A comparison of the core thicknesses and of the test results of the recovered asphalt from cores shows that the characteristics of the pavement in the non-cracking and cracking sections are virtually identical. It can therefore be concluded that the observed differences in cracking behavior is not a function of asphalt pavement characteristics.

A review of the subgrade profiles of the two sections does not highlight any highly significant differences in soil types or characteristics. There is a tendency for the clay soils in the non-cracking section to be more plastic, ie. higher Plasticity Index, and wetter ie. field moisture content relative to the plastic limit of the soil.

The fact that a significant portion of the project did not crack indicates, that for those given subgrade conditions, the critical cracking stiffness of the pavement wasn't reached. Comparing the nomograph stiffnesses of the asphalt and mix for the on-site temperature conditions, and based on original and recovered asphalt characteristics, to critical stiffness limits proposed by McLeod (25), Gaw (26) and Fromm and Phang (27) would confirm that the critical stiffness of the asphalt and pavement haven't been exceeded. This is summarized below:

Criteria for Transverse Cracking to Occur	SR 661:02 Conditions
1. Stiffness of the pavement at a 50 mm depth > 6.9×10^9 to 1.4×10^{10} Pa (loading time = 20 000 s) (25)	1.1×10^9 Pa
2. Stiffness of the asphalt > 1×10^9 Pa (loading time = 1800 s) (26)	4.8×10^8 Pa
3. Stiffness of the asphalt > 1.4×10^8 Pa (loading time = 10 000 s) (27)	1×10^8 Pa

In summary, the results of the testing program confirmed that asphalt or pavement characteristics were not the cause for the difference in cracking behavior between the two sections. The asphalt characteristics however did highly influence the behavior of the non-cracking section. Comparison of nomograph in-place asphalt stiffness to limiting criteria developed by others shows that the critical asphalt stiffness required to initiate transverse cracking was not reached.

Unfortunately, the characteristics of the subgrade soils to a 2 m depth in the two sections did not differ significantly enough to make any conclusions regarding their influence on transverse cracking.

Table A.1 Project SR 661:02
Transverse Crack Counts

Section	km - km	No. of Transverse Cracks
661:02-1	2.2 - 3.2	37
	3.2 - 4.2	28
	4.2 - 5.2	26
661:02-2	5.2 - 6.2	0
	6.2 - 7.2	0
	7.2 - 8.2	1
	8.2 - 9.2	3
	9.2 - 10.2	1
	10.2 - 11.2	0
	11.2 - 12.2	0
	12.2 - 13.2	0
	13.2 - 14.4	1

Table A.2 Project SR 661:02
Summary of Test Results of
Asphalt Supplied in 1979

	Abs.Visc. @60°C (Pa.s)	Kin.Visc. @135°C (mm²/s)	Pen @25°C 100g,5s (dmm)	Pen @4°C 200g,60s (dmm)	PVN
Tests on Original Asphalt (N = 6)	35.8	212	285	92	-0.1
Tests on Residue After TFOT (N = 6)	86.3	-	148	49	-0.3

Table A.3 Project SR 661:02
Summary of Test Results of Recovered Asphalt

Section	Abs.Visc. @60°C (Pa.s)	Kin.Visc. @135°C (mm²/s)	Pen @25°C 100g,5s (dmm)	Pen @4°C 100g,5s (dmm)	Pen @4°C 200g,60s (dmm)	PVN'	PVN	PI
661:02-1 (N = 5)	157.9	338	99	12	35	-0.3	-0.5	-0.6
661:02-2 (N = 4)	167.2	341	97	12	36	-0.3	-0.5	-0.5
Average (N = 9)	162.0	340	98	12	34	-0.3	-0.5	-0.5

Table A.4 Project SR 661:02
Summary of Test Results of Pavement Cores (Top Lift)

Section	Thickness[1] (mm)	Density (kg/m³)	A.C. (%)	Air Voids (%)	V.M.A. (%)	Sieve Analysis - % Passing 16 000	10 000	5000	1250	630	315	160	80
661:02-1 (N = 4)	166	2278	5.5	6.6	17.0	99	76	53	39	35	26	12.4	7.9
661:02-2 (N = 5)	175	2269	5.4	7.1	17.2	98	71	48	36	33	25	12.2	7.9
Average (N = 9)	171	2274	5.5	6.9	17.1	99	74	51	38	34	26	12.3	7.9

[1]This value is for the total pavement structure

Table A.5 Project SR 661:02-1 (Cracking Section)
Soil Survey Test Results

Sample Depth (m)	% Passing Sieve Size (µm) 5000	400	160	80	LL (%)	PL (%)	PI (%)	Field M.C. (%)	LI	Est. Opt. M.C. (%)	Est. Max. Density (kg/m³)	Soil Class.
Hole #: 6	Station: 5.845		Location: 3.4 m rt ℄									
0.30	80	69	40.6	30.7	20.4	11.8	8.6	8.8	-0.34	N/A	N/A	SC
0.50	96	93		66.1	34.4	13.9	20.5	14.2	0.01	14	1850	CI
1.00	97	85	56.3	46.6	23.2	11.7	11.5	9.4	-0.20	N/A	1970	SC
1.50	100	100		90.2	47.6	21.4	26.2	24.9	0.13	23	1570	CI
1.80	No sieve test done				No test			345.5	N/A	N/A	N/A	Pt
2.60	100	91	28.7	19.8	Trace	High		23.8	N/A	N/A	N/A	SMd
Hole #: 7	Station: 6.060		Location: 0.6 m rt ℄									
0.27	80	79		48.3	30.8	13.2	17.6	14.8	0.09	13	1890	SC
0.50	99	97		71.8	39.8	14.8	25.0	16.5	0.06	16	1820	CI
0.90	86	85		59.5	34.2	14.3	19.9	16.9	0.13	15	1850	CI
1.30	100	93	47.8	34.6	15.9	12.5	3.4	8.9	-1.05	N/A	N/A	SMd
1.50	99	98		81.5	40.6	17.4	23.2	24.4	0.30	18	1720	CI
1.80	No sieve test done				No test			288.3	N/A	N/A	N/A	Pt
Hole #: 8	Station: 7.650		Location: 2.1 m lt ℄									
0.28	81	78		48.5	27.0	12.5	14.5	11.6	-0.06	12	1930	SC
0.70	96	96		83.9	40.6	17.1	23.5	17.7	0.02	18	1730	CI
1.20	100			95.2	31.9	20.5	11.4	20.1	-0.03	21	1620	CI-CL-
1.80	79	76		52.7	31.4	14.0	17.4	24.6	0.60	14	1880	CI-CL
Hole #: 9	Station: 8.551		Location: 1.5 m rt ℄									
0.24	99	98		87.2	27.1	18.3	8.8	14.9	-0.38	17	1720	CL
0.70	100	100		97.5	49.0	21.1	27.9	18.3	-0.10	22	1570	CI-CH
1.20	91	90		79.5	41.7	17.4	24.3	19.1	0.06	18	1720	CI
1.80	77	73		55.4	35.8	15.5	20.3	19.9	0.21	16	1810	CI
Hole #10	Station: 9.346		Location: 1.8 m rt ℄									
0.24	100	99		89.0	42.4	16.4	26.0	14.8	-0.06	17	1730	CI
0.70	99	96		82.3	39.7	16.8	22.9	17.3	0.02	17	1730	CI
1.20	100	99		88.4	25.9	18.9	7.0	18.2	-0.10	18	1690	CL-ML
1.70	98	98		91.3	50.0	20.2	29.8	24.0	0.12	21	1610	CH-CI

Table A.6 Project SR 661:02-2 (Non-cracking Section)
Soil Survey Test Results

Sample Depth (m)	% Passing Sieve Size (μm) 5000	400	160	80	LL (%)	PL (%)	PI (%)	Field M.C. (%)	LI	Est. Opt. M.C. (%)	Est. Max. Density (kg/m³)	Soil Class.
Hole #: 1	Station: 2.404		Location: 1.0 m lt ℄									
0.50	100			85.8	30.1	18.2	11.9	16.1	-0.17	18	1720	CI-CL
1.00	99	99		97.9	40.8	20.6	20.2	22.1	0.07	21	1610	CI
1.50	100			91.4	Trace	High		11.1	N/A	N/A	N/A	N/A
2.00	100			96.5	21.9	17.1	4.8	18.1	0.20	14	1810	CL-ML
Hole #: 2	Station: 2.489		Location: 3.4 m lt ℄									
0.50	100	100		77.1	26.2	18.0	8.2	16.5	-0.18	17	1730	CL
1.00	96	95		84.4	28.7	18.5	10.2	17.7	-0.07	18	1690	CL-CI
1.50	100	100		90.1	30.5	18.2	12.3	16.6	-0.13	18	1720	CI-CL
2.00	100			76.9	Trace	High		12.6	N/A	N/A	N/A	N/A
Hole #: 3	Station 3.677		Location: 3.2 m lt ℄									
0.50	100	99		84.8	31.8	16.3	15.5	17.1	0.05	16	1770	CI-CL
1.00	100	100		57.6	21.3	15.4	5.9	13.0	-0.40	13	1880	CL-ML
1.50	100			89.3	39.8	17.2	22.6	19.0	0.07	18	1730	CI
2.20	100			84.1	28.7	15.7	13.0	21.4	0.43	15	1820	CL-CI
Hole #: 4	Station: 4.217		Location: 3.2 m rt ℄									
0.50	100	99		80.7	30.5	17.5	13.0	16.0	-0.11	17	1730	CI-CL
1.20	100			97.8	42.4	19.7	22.7	33.2	0.59	21	1650	CI
1.50	100			99.5	36.1	22.0	14.1	31.6	0.68	23	1570	CI
2.00	100	100		88.5	22.3	17.2	5.1	20.7	0.68	15	1800	CL-ML
Hole #: 5	Station: 4.757		Location: 2.2 m lt ℄									
0.22	98	97		70.1	25.6	15.5	10.1	16.0	0.04	14	1850	CL
0.80	100			88.9	30.7	19.8	10.9	22.3	0.22	20	1650	CI-CL
1.20	100			93.4	34.2	18.4	15.8	22.6	0.26	18	1690	CI
1.70	100	100		92.2	29.3	20.9	8.4	23.2	0.27	21	1610	CL-CI

Table A.7 Project SR 661:02
Stiffness Calculations

Input required for stiffness calculations:

- characteristics of original asphalt: penetration @25°C = 285 dmm
  PVN' = -0.1
- characteristics of original asphalt after TFOT: penetration @25°C = 148 dmm
  PVN' = -0.3
- characteristics of recovered asphalt = penetration @25°C = 98 dmm
  penetration @ 4°C,100g,5s = 12 dmm
  PVN' = -0.3
  PI = -0.5
- minimum ambient air temperature = -42°C
  minimum pavement surface temperature = -35°C
  minimum pavement temperature @ 50 mm depth = -32°C
- in-place asphalt pavement characteristics: V.M.A. = 17.2%
  Air Voids = 2.1%

Asphalt	Original Supplied		Original After TFOT		Recovered	
Pvmnt Temp. °C	-35	-32	-35	-32	-35	-32
Stiffness(binder)[1] (Pa)	5.8×10^{6}	2.5×10^{6}	2.4×10^{7}	1.5×10^{7}	5.9×10^{7}	3.9×10^{7}
Stiffness(binder)[2] (Pa)	-	-	-	-	4×10^{8}	-
Stiffness(binder)[3] (Pa)	-	-	-	-	1×10^{8}	-
Stiffness(mix)[4] (Pa)	-	1.1×10^{9}	-	-	-	4.1×10^{9}

[1] using van der Poel's nomograph as modified by McLeod, time of loading = 20 000 s

[2] using van der Poel's nomograph, time of loading = 1 800 s

[3] using van der Poel's nomograph, time of loading = 10 000 s

[4] using van der Poel's nomograph, time of loading = 20 000 s

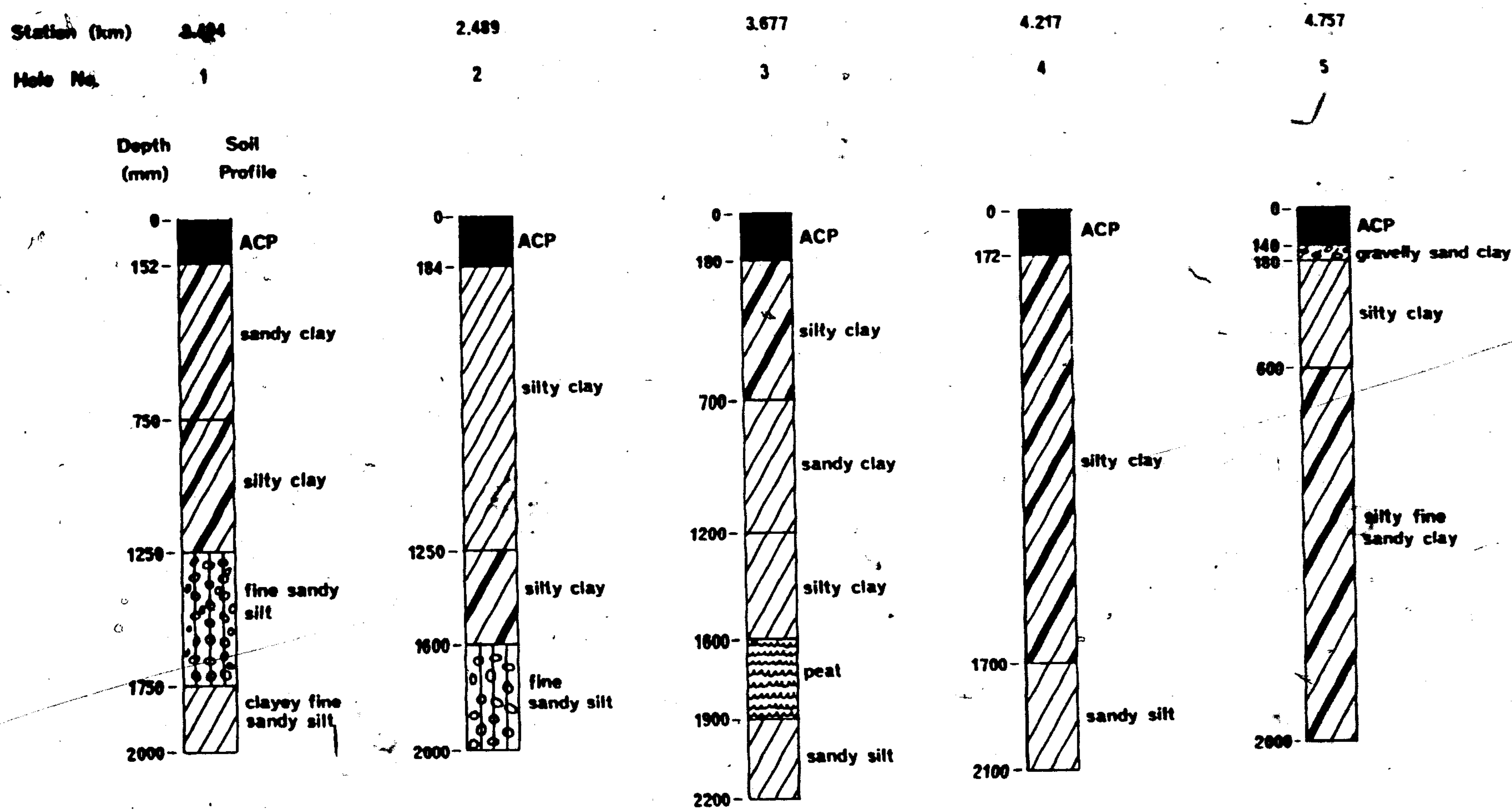


Fig. A.1 Project SR 661:02-1 (Cracking Section)
Subgrade Soil Profiles

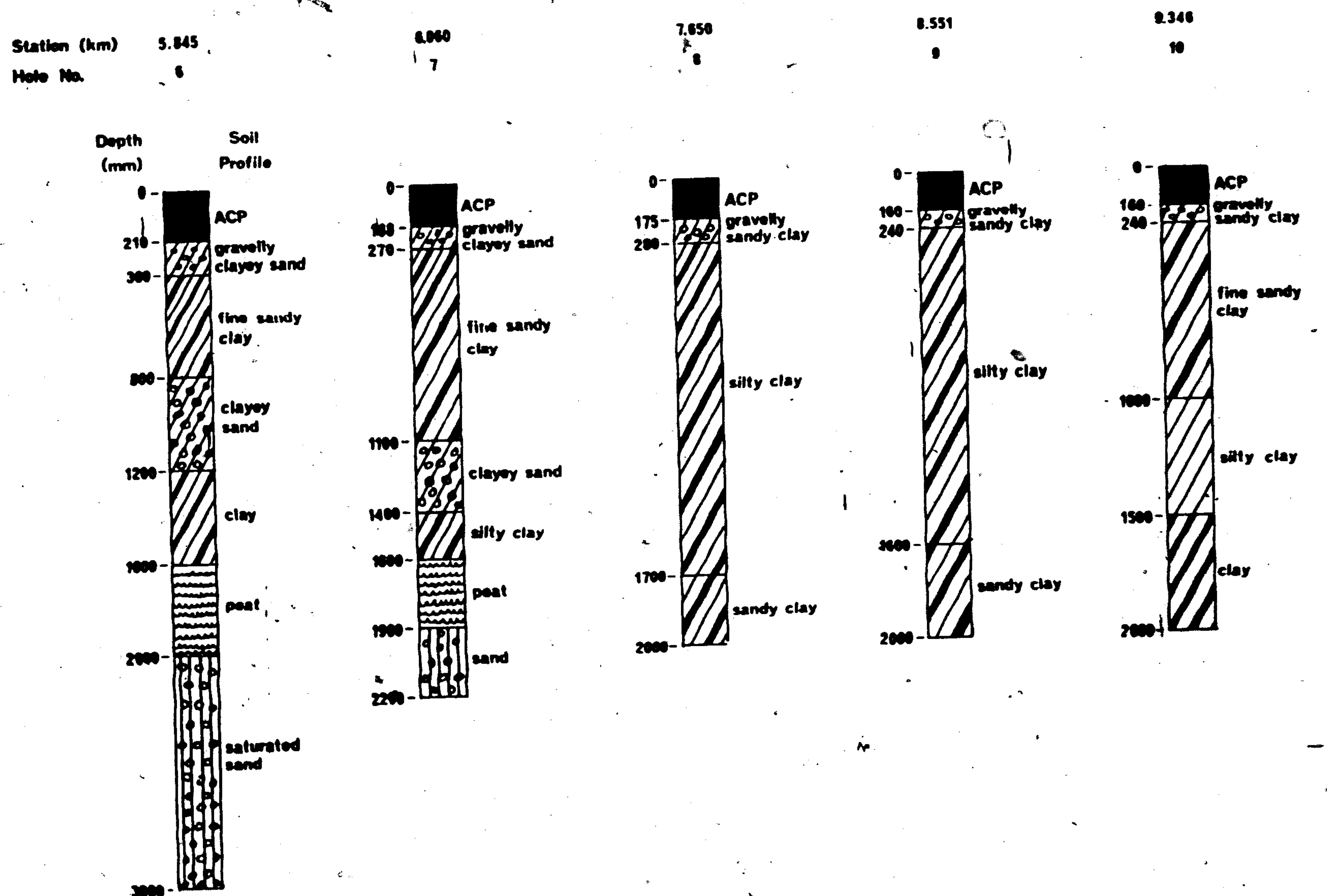


Fig. A-2 Project SR 661:02-2 (Non-cracking Section) Subgrade Soil Profiles

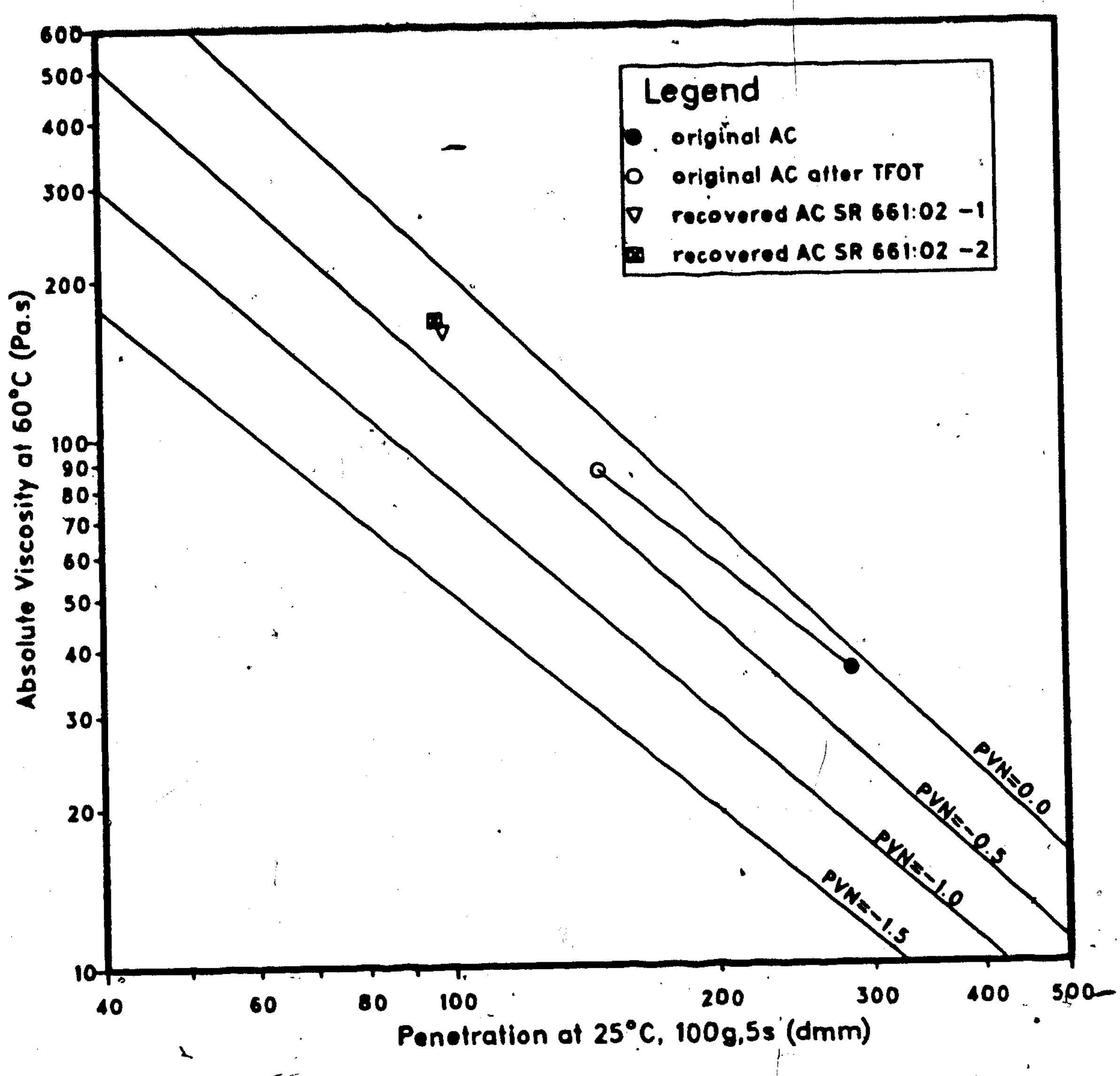


Fig. A.3 Project SR 661:02
Asphalt Penetration-Viscosity Relationships

APPENDIX B - CASE STUDY - PROJECT HWY 33:10

Project Hwy 33:10

South of Swan Hills to West of Fort Assiniboine

km 9 to km 42

B.1 Transverse Cracking Performance

This project was the first granular base pavement structure investigated that exhibited significant lengths of non-cracking pavement. Initially, the frequency of transverse cracking appeared to be a function of the year of construction of the granular base course. The granular base course of Section 33:10-1 was constructed in 1973 and for Section 33:10-2, in 1974. Both sections were paved in 1975 under one contract using one aggregate source and asphalt supplier. Section 33:10-1 exhibited cracking frequencies between 0 and 30 cracks/ per km with an average frequency of 9.7 cracks/per km. Section 33:10-2 had an average frequency of 0.2 cracks/km with a total of 12 crackfree kilometres.

The general topography of the two sections is different. Section 33:10-2 shows level, fairly well drained topography. Section 33:10-1 is rolling with a higher frequency of cuts and fills. The boundary between the two sections coincides with a boundary between two surficial geology units (31).

B.2 Project Details

Location: The project is located in Central Alberta and is approximately 150 km northwest of Edmonton and is about 30 km northwest of SR 661:02.

Local Climate: The average winter low temperature over the last 10 years was -43°C. The extreme record low was -48°C, occurring in 1980 (30). The freezing index is 1400 °C days.

Surficial Geology: A review of the surficial geology map of the area (31) shows that a distinct boundary between surficial geology units coincides with the split between the non-cracking and cracking sections. The surficial geology of Section 33:10-1 indicates silt and fine sand glacial lake deposits with a hummocky terrain topography; Section 33:10-2 indicates a till (mixture of clay, silt and fine sand) ground morraine. The topography, noted from air photos, is level.

Traffic: AADT (1985) = 1050

Equivalent 18 kip (80 kN) single axle applications = 120

Construction: The structural design for the two sections was the same: 100 mm ACP, 50 mm Asphalt Stabilized Base Course, and 225 mm granular base course. Different aggregate sources were used for the granular base for each section. The average gradations reported at the time of construction are presented in Table B.2. The asphalt pavement for both sections was constructed in one year by the same Contractor using the same aggregate source and asphalt cement. The asphalt cement used was AC 275 supplied by Husky (Lloydminster).

Field Sampling and Laboratory Testing Program: Five 150 mm diameter cores at randomly selected locations were taken of the pavement and underlying granular base course in both sections. Due to cold weather drilling conditions, representative samples of the base course could not be retrieved. At each of the core locations, the underlying subgrade was sampled to a 2 m depth.

The sealcoat was trimmed off the cores and the densities of both lifts were determined prior to extractions and Abson recoveries. The recovered asphalt was tested similarly as for SR 661:02.

The subgrade soil samples were tested for moisture content and routine classification testing was carried out.

B.3 Test Results

The results of the testing program and other observations are presented in the following Tables and Figures:

Table B.1 Transverse Crack Counts

Table B.2 Summary of Construction Quality Control Testing

Table B.3 Summary of Test Results of Asphalt Supplied in 1975

Table B.4 Summary of Test Results of Recovered Asphalt

Table B.5 Summary of Test Results of Pavement Cores

B.4 Discussion

The granular base course for each section was constructed in different years using different aggregate sources. The results of testing carried out during crushing operations, presented in Table B.2, indicate no major differences in gradations. It is concluded, therefore, that the granular base course is not significantly influencing the differences in observed cracking frequencies.

A comparison of the characteristics of the recovered asphalt from both sections indicates significant differences. Use of the t-test shows that the mean absolute viscosities at 60°C and the penetrations at 25°C of the two sections are significantly different at the 1.0 percent and 0.1 percent levels respectively. Scrutinization of the construction quality control test results and the sequence of paving operations does not offer any reasons for the differences in asphalt aging between the two sections. These data are presented in Table B.2. Based upon the available information, the reasons for the differences cannot be determined.

The nomograph asphalt and mix stiffnesses were determined for the on-site temperature conditions and were compared to limits proposed by McLeod (25), Gaw (26), Fromm and Phang (27) and values determined for SR 661:02-2 (non-cracking). This is summarized below:

Criteria for Transverse Cracking to Occur	33:10-1 (cracking)	33:10-2 (non-cracking)	661:02 (non-cracking)
1. Stiffness of the pavement at a 50 mm depth >6.9×10^{9}-1.4×10^{10} Pa (loading time = 20 000 secs) (25)	8.3×10^{8}	8.3×10^{8}	1.1×10^{9}
2. Stiffness of the asphalt > 1.0×10^{9} Pa (loading time = 1800 secs) (26)	1.7×10^{8}	1.0×10^{8}	4.8×10^{8}
3. Stiffness of the asphalt > 1.4×10^{8} Pa (loading time = 10 000 secs) (27)	9×10^{7}	5×10^{7}	1×10^{8}

These values indicate that the in-place nomograph stiffness for both the cracking and non-cracking sections are less than the proposed criteria and are also less than the values determined for SR 661:02 (non-cracking). This would suggest that the observed cracking behavior of 33:10-1 is not being governed by the characteristics of the asphalt pavement.

The boundary between the two sections coincides with the job split between the two granular base projects and with a boundary between surficial geology units. A review of the subgrade profiles does not highlight significant differences in soil types encountered. As shown in Table B.1, the cracking frequency of Section 33:10-1 is not consistent from kilometre to kilometre with frequencies ranging from 0 to 30 cracks/km. Field inspection indicates that there appears to be a higher incidence of cracking in cut sections or side-hill cuts. Only test hole number 3, km 12,063, was drilled in an area of high cracking frequency. This hole shows that sand is encountered below about 1 m.

In summary, the results of the testing program indicate significant but unexplainable differences in pavement characteristics between the two sections although the pavement was constructed at the same time. Comparison of nomograph stiffnesses to published proposed stiffness limits and to values determined for Project SR 661:02-2 suggest that the asphalt is not the cause of the observed cracking behavior of Section 33:10-1. The asphalt characteristics did, however influence the behavior of the non-cracking section. It is concluded that subgrade characteristics are the primary factor influencing the observed cracking of Section 33:10-1.

Table B.1 Project Hwy 33:10
Transverse Crack Counts

Section	km-km	No. of Transverse Cracks	Section	km-km	No. of Transverse Cracks
33:10-1	8- 9	1	33:10-2	21-22	0
	9-10	0		22-23	0
	10-11	5		23-24	0
	11-12	0		24-25	1
	12-13	30		25-26	0
	13-14	17		26-27	3
	14-15	9		27-28	1
	15-16	9		28-29	1
	16-17	7		29-30	1
	17-18	8		30-31	2
	18-19	25		31-32	0
	19-20	6		32-33	1
	20-21	11		33-34	0
				34-35	3
				35-36	0
				36-37	0
				37-38	0
				38-39	1
				39-40	0
				40-41	0
				41-42	0

Table B.2 Project Hwy 33:10
Summary of Construction Quality Control Testing

B.2.1 Granular Base Course

Section	Aggregate Source	N	2"	1½"	1"	3/4"	#4	#10	#40	#200
			Sieve Analysis - % Passing							
33:10-1	Horse Creek and Freeman River #2	27	-	100	-	51	31	27	19	6.2
33:10-2	Freeman River #2	39	100	-	63	-	33	-	-	4.7
	Telegraph #2	25	100	-	58	-	30	-	-	7.8

B.2.2 Asphalt Concrete Pavement

Section	[illegible]	Period of Construction	A.C. (%)	Compaction (%)	Core Density (kg/m³)	Core A.V. (%)	Pugmill Discharge Temp., °C	5/8"	3/8"	#4	#10	#40	#200
								Sieve Analysis - % Passing					
33:[illegible]	Top	[illegible]	6.6	93.8	2179	8.0	136	100	78	53	39	27	8.3
	Bottom	[illegible]-1	6.2	95.9	2228	6.8	133	100	77	58	45	30	9.0
33:10[illegible]	Top	[illegible]8-Oct15	6.6	95.5	2196	7.0	139	100	76	52	40	26	7.6
	Bottom	Aug1-14	6.7	96.5	2247	6.3	138	99	76	54	41	28	8.3

Table B.3 Project Hwy 33:10
Summary of Test Results of Asphalt Supplied in 1975

	Abs.Visc. @60°C (Pa.s)	Pen @25°C 100g,5s (dmm)	Pen @4°C 200g,60s (dmm)	PVN'
Tests on Original Asphalt (N = 4)	46.0	265	100	0.1
Tests on Residue After TFOT (N = 4)	111.8	141	57	-0.1

Table B.4 Project Hwy 33:10
Summary of Test Results of Recovered Asphalt

Section	Lift	Abs.Visc. @60°C (Pa.s)	Kin.Visc. @135°C (mm^2/s)	Pen @25°C 100g,5s (dmm)	Pen @4°C 100g,5s (dmm)	Pen @4°C 200g,60s (dmm)	PVN'	PVN	PI
33:10-1	Top (N=4)	229.3	406	84	11	33	-0.2	-0.4	-0.3
	Bottom (N=4)	257.0	450	89	11	32	0.1	-0.2	-0.5
33:10-2	Top (N=4)	178.0	374	103	13	38	-0.1	-0.3	-0.4
	Bottom (N=4)	170.1	378	113	13	38	0.0	-0.2	-0.7

Table B.5 Project Hwy 33:10
Summary of Test Results of Pavement Cores

Section	Lift	Thickness (mm)	Density (kg/cm^3)	A.C. (%)	Air Voids (%)	V.M.A. (%)	Sieve Analysis - % Passing 16 000	10 000	5000	1250	630	315	160	80
33:10-1	Top (N=5)	49	2268	6.1	6.2	18.0	99	81	53	36	31	21	12.4	8.8
	Bottom (N=5)	41	2244	5.8	7.5	18.7	98	77	51	34	30	21	11.4	7.5
33:10-2	Top (N=4)	59	2259	6.0	6.6	18.3	100	76	50	33	29	19	10.7	7.2
	Bottom (N=5)	57	2254	6.4	6.4	18.8	99	79	53	36	31	21	10.8	7.3

Table B.6 Project Hwy 33:10-2 (Non-cracking Section)
Soil Survey Test Results

Sample Depth (m)	% Passing Sieve Size (µm) 5000	400	160	80	LL (%)	PL (%)	PI (%)	Field M.C. (%)	LI	Est. Opt. M.C. (%)	Est. Max. Density (kg/m³)	Soil Class.
Hole #: 6	**Station: 22.300**			**Location: 1.3 m rt ℄**								
1.00	99	94		62.3	34.9	15.1	19.8	14.8	-0.01	15	1840	CI
1.40	97	92		66.8	38.9	14.9	24.0	16.7	0.07	16	1820	CI
1.80	100	93	54.2	42.9	20.9	15.4	5.5	16.9	0.27	13	1880	SC-SMd
2.10	100	99		74.9	61.9	41.5	20.4	73.1	1.54	N/A	N/A	OH
Hole #: 8	**Station: 25.000**			**Location: 3.5 m lt ℄**								
0.50	96	93		67.3	34.7	14.1	20.6	18.7	0.22	14	1850	CI
0.80	100	96		72.5	42.9	16.8	26.1	21.0	0.16	18	1730	CI
1.30	99	98		66.6	37.2	15.2	22.0	17.5	0.10	16	1810	CI
1.80	98	96		74.9	39.3	15.3	24.0	18.6	0.13	16	1810	CI
Hole #: 9	**Station: 26.200**			**Location: 0.4 m rt ℄**								
0.48	100	97		75.7	41.4	15.7	25.7	16.9	0.04	17	1770	CI
0.90	99	98		82.7	38.2	16.6	21.6	16.8	0.00	17	1760	CI
1.40	98	97		74.1	38.9	14.6	24.3	18.6	0.16	16	1840	CI
1.80	100	98		76.1	43.7	16.6	27.1	18.2	0.05	18	1730	CI
Hole #: 10	**Station: 29.494**			**Location: 4.2 m rt ℄**								
0.50	97	95		68.3	40.1	15.5	24.6	18.5	0.12	16	1770	CI
0.80	94	93		65.2	39.6	14.7	24.9	19.7	0.20	16	1840	CI
1.30	99	95		64.2	32.5	14.5	18.0	17.9	0.18	14	1850	CI
1.80	91	85		58.8	42.6	15.8	26.8	18.3	0.09	17	1770	CI
Hole #: 11	**Station: 30.350**			**Location: 2.9 m lt ℄**								
0.50	95	91		72.9	43.1	16.7	26.4	18.8	0.07	18	1730	CI
0.80	99	93		70.5	52.9	19.7	33.2	21.8	0.06	21	1620	CH
1.30	99	93		72.7	45.0	18.3	26.7	18.5	0.00	20	1690	CI
1.80	98	94		72.3	48.0	17.8	30.2	23.4	0.18	20	1690	CI-CH

Table B.7 Project Hwy 33:10-1 (Cracking Section)
Soil Survey Test Results

Sample Depth (m)	% Passing Sieve Size (μm) 5000	400	160	80	LL (%)	PL (%)	PI (%)	Field M.C. (%)	LI	Est. Opt. M.C. (%)	Est. Max. Density (kg/m³)	Soil Class.
Hole #: 1	Station: 8.260			Location: 4.8 m lt ℄								
0.43	99	97		79.7	42.7	17.6	25.1	23.2	0.22	19	1690	CI
0.80	94	93		80.9	43.2	16.5	26.7	18.8	0.08	18	1730	CI
1.20	98	97		83.9	39.9	15.3	24.6	18.8	0.14	16	1810	CI
1.80	95	93		70.8	41.0	16.3	24.7	18.7	0.09	17	1760	CI
Hole #: 2	Station: 8.910			Location: 4.3 m lt ℄								
0.435	82	81		60.3	36.1	14.9	21.2	17.7	0.13	15	1840	CI
0.90	100	99		82.0	43.5	16.4	27.1	24.6	0.30	18	1730	CI
1.20	100	99		64.2	34.6	13.6	21.0	18.6	0.23	14	1880	CI
1.40	100	99		63.8	21.9	15.5	6.4	16.6	0.17	13	1850	CL-ML
1.80	100	99		58.2	29.5	14.0	15.5	17.7	0.23	13	1890	CL-CI
Hole #: 3	Station: 12.063			Location: 4.1 m lt ℄								
0.475	97	96		73.7	44.2	16.7	27.5	23.4	0.24	18	1730	CI
1.40	100	99	31.0	15.6	Trace	Medium		6.7	N/A	N/A	N/A	SMd
1.90	100	94	29.8	19.0	Trace	Medium		7.7	N/A	N/A	N/A	SMd
Hole #: 4	Station: 17.230			Location: 4.0 m rt ℄								
0.50	98	96		78.8	39.3	16.4	22.9	16.2	0.00	17	1760	CI
0.90	100	100		84.0	41.2	18.2	23.0	21.4	0.13	20	1690	CI
1.40	96	96		88.7	45.6	18.3	27.3	28.5	0.37	20	1690	CI
1.90	100	100		79.9	34.3	19.0	15.3	19.5	0.03	20	1680	CI
Hole #: 5	Station: 19.780			Location: 2.8 m lt ℄								
0.35	No sieve test done				No test				N/A	N/A	N/A	
0.50	100	97		75.7	39.8	15.5	24.3	17.1	0.06	16	1770	CI
0.90	100	99		52.2	28.2	13.8	14.4	16.1	0.15	13	1890	CL-CI
1.90	100	98	68.7	46.4	25.5	14.5	11.0	14.2	-0.02	13	1890	SC

Table B.8 Project Hwy 33:10
Stiffness Calculations

Input required for stiffness calculations:

- characteristics of original asphalt: penetration @25°C = 265 dmm
  PVN' = 0.1
- characteristics of original asphalt after TFOT: penetration @25°C = 141 dmm
  PVN' = -0.1
- characteristics of recovered asphalt:

33:10-1 (cracking section)	- penetration @25°C = 84 dmm penetration @4°C,100g,5s = 11 dmm PVN' = -0.2 PI = -0.3
33:10-2 (non-cracking section)	- penetration @25°C = 103 dmm penetration @4°C,100g,5s = 13 dmm PVN' = -0.1 PI = -0.4

- minimum ambient temperature = -42°C
  minimum pavement surface temperature = -35°C
  minimum pavement temperature @ 50 mm depth = -32°C
- in-place asphalt pavement characteristics:

33:10-1	VMA = 18.0% Air Voids = 6.2%
33:10-2	VMA = 18.3% Air Voids = 6.6%

Asphalt	Original Supplied		Original After TFOT		Recovered 33:10-1		Recovered 33:10-2	
Pvmnt Temp. °C	-35	-32	-35	-32	-35	-32	-35	-32
Stiffness (binder)[1] (Pa)	4.9×10^{6}	2.7×10^{6}	2.2×10^{7}	1.4×10^{7}	5.9×10^{7}	3.9×10^{7}	3.9×10^{7}	1.6×10^{7}
Stiffness (binder)[2] (Pa)	-	-	-	-	1.7×10^{8}	-	1.0×10^{8}	-
Stiffness (binder)[3] (Pa)	-	-	-	-	9×10^{7}	-	5×10^{7}	-
Stiffness (mix)[4] (Pa)	-	8.3×10^{8}	-	-	-	3.8×10^{9}	-	2.1×10^{9}

[1] using van der Poel's nomograph as modified by McLeod, time of loading = 20 000 s
[2] using van der Poel's nomograph, time of loading = 1 800 s
[3] using van der Poel's nomograph, time of loading = 10 000 s
[4] using van der Poel's nomograph, time of loading = 20 000 s

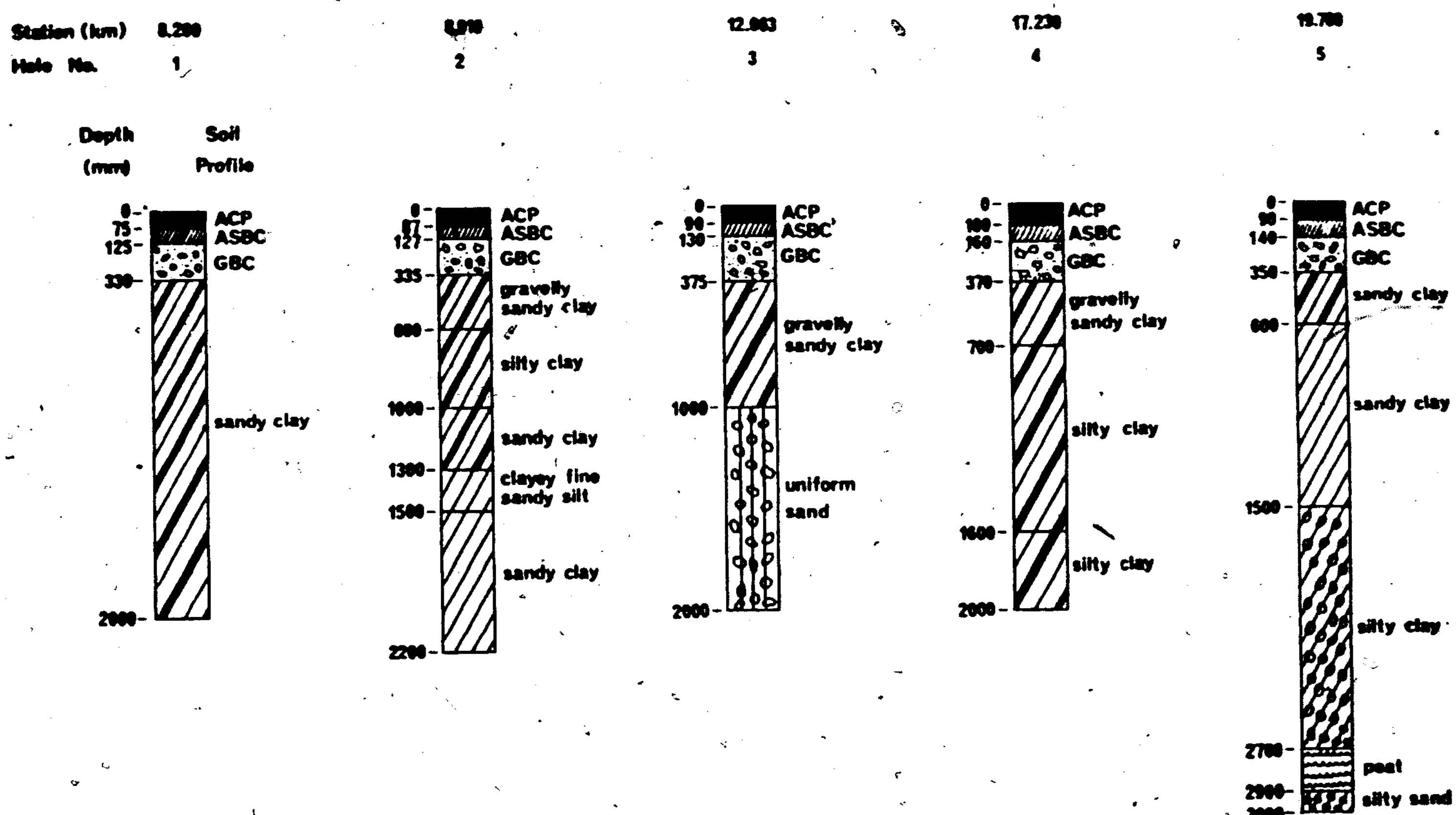


Fig. B.1 Project Hwy 33:10-1 (Cracking Section)
Subgrade Soil Profiles

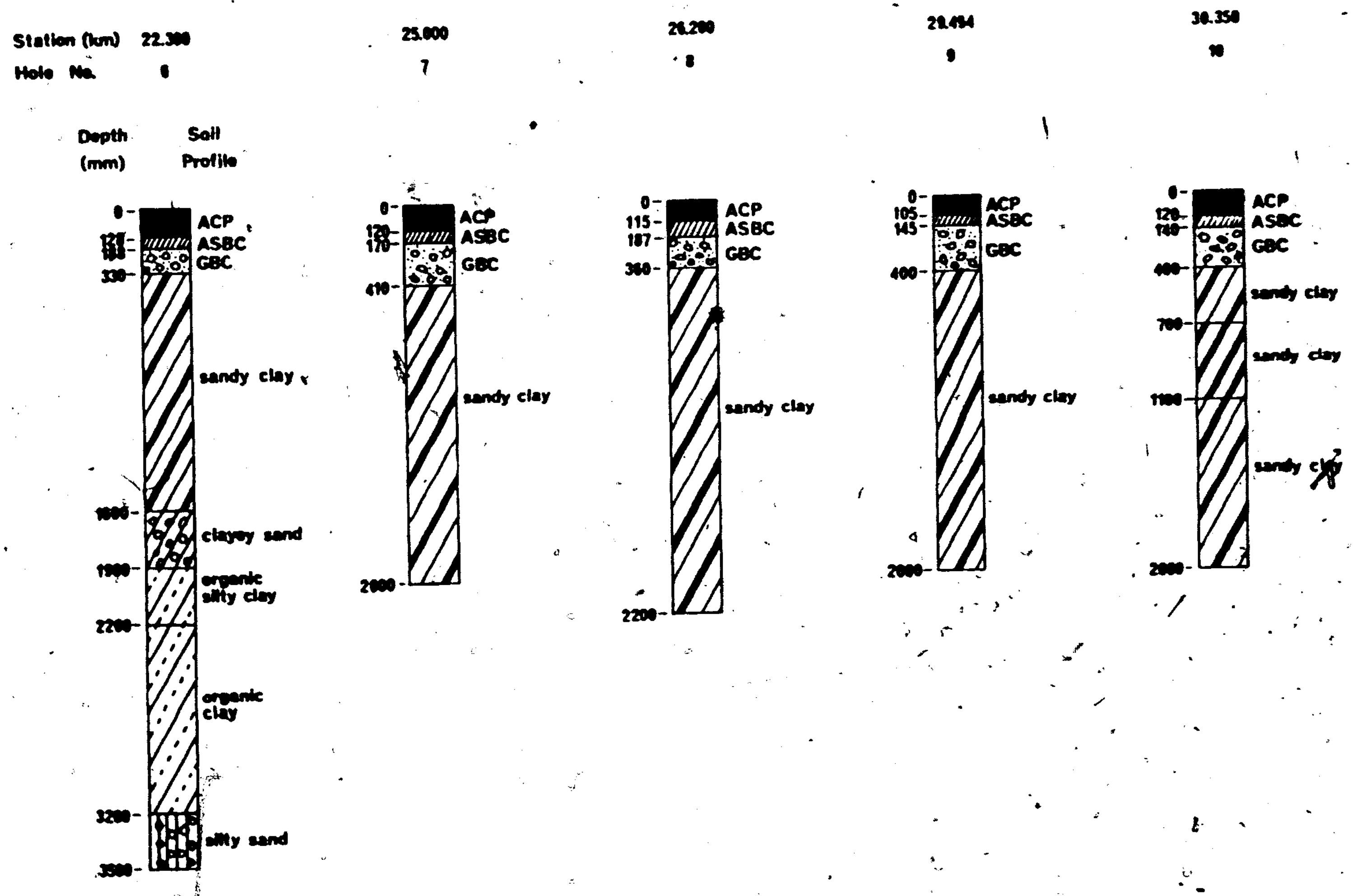


Fig. B.2 Project 33:10-2 (Non-cracking Section)
Subgrade Soil Profiles

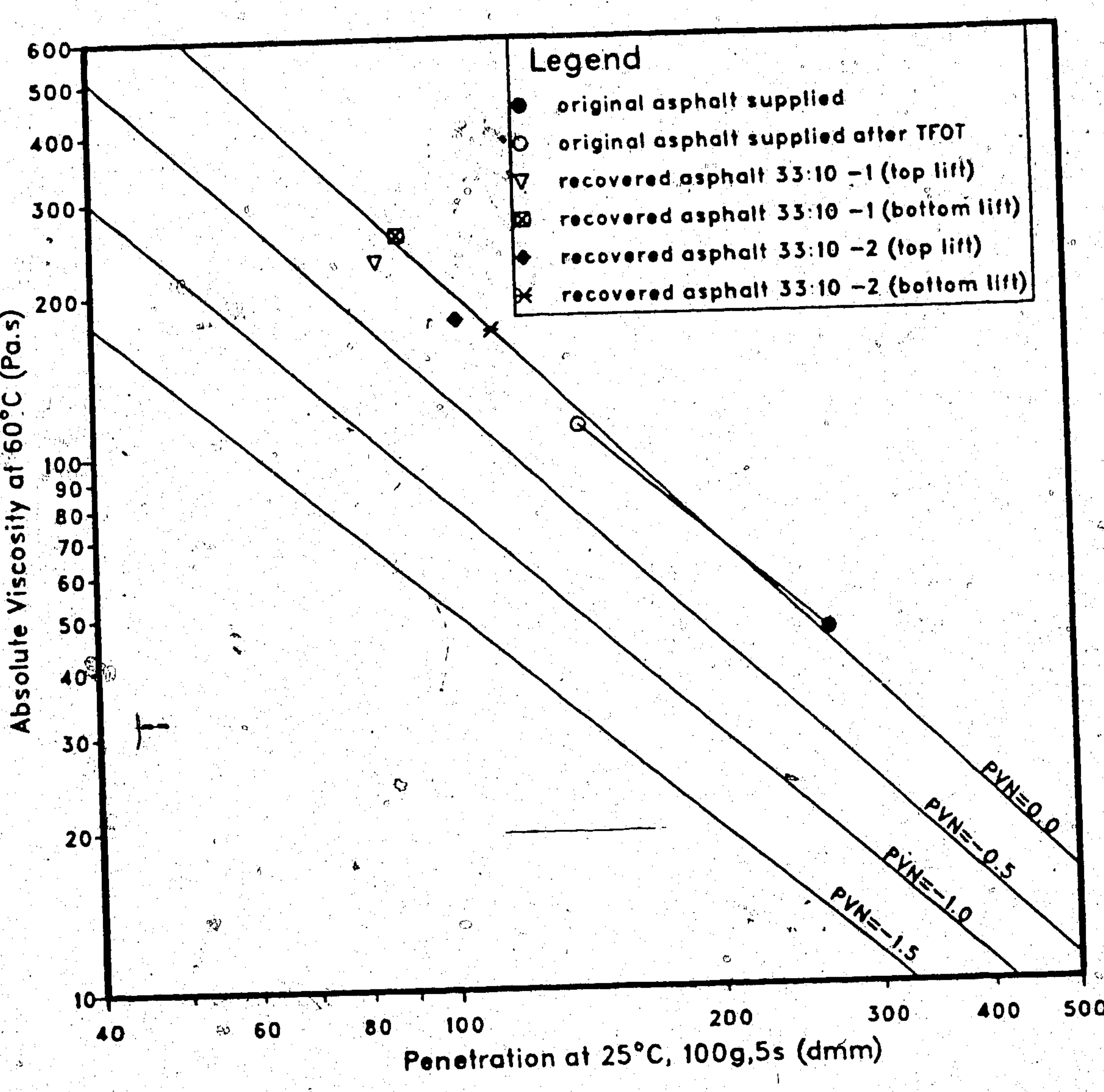


Fig. B.3 Project Hwy 33:10
Asphalt Penetration-Viscosity Relationships

APPENDIX C - CASE STUDY - PROJECT SR 845:02

Project SR 845:02

Raymond to South of Coaldale

km 9.6 to km 25.6

C.1 Transverse Cracking Performance

This full depth project can be broken up into 3 sections, each exhibiting very different frequencies of transverse cracking. Construction records indicated that 100 mm of asphalt pavement was constructed on prepared subgrade in 1973 and 1974 under the same Contract and overlaid with 60-100 mm of pavement in 1980. The AC 275 used in 1973 and 1974 and the AC 27.5 used in 1980 was supplied by Husky (Lloydminster). It was felt that the observed transverse cracking was not being influenced by the 1980 overlay and reflected the actual low temperature performance of the pavements constructed in 1973 and 1974. It was originally felt that the differences in cracking performance were related to the materials used in the two different years of construction. Summaries of the major differences between the 3 sections and the observed cracking frequencies are presented in Tables C.1 and C.2.

The general topography over the total length of the project can be described as level and well drained.

C.2 Project Details

Location: The project is located in Southern Alberta approximately 15 km southeast of Lethbridge and about 70 km north of the Canada/U.S. border.

Local Climate: This area could be described as semi-arid. The average low winter temperature over the last 10 years was -34°C (30). The extreme low since 1980 was -39°C occurring in 1983; the lowest temperature reached between 1973 and 1980 was -37°C occurring in 1977. The freezing index is 833°C days.

Surficial Geology: Surficial deposits are generally glacial tills (32). A small area of post-glacial wind deposited sands and silts corresponds exactly with the boundaries of the most heavily cracked section, 845:02-3.

Traffic: AADT (1984) = 967

Equivalent 18 kip (80 kN) single applications = 33

Construction: The 1980 overlay constructed under one Contract using one aggregate and asphalt source was continuous over all three sections. Different aggregate sources were used for the pavements constructed in 1973 and 1974. The results of quality control testing carried out during construction are summarized in Table C.3. Asphalt cement used in 1973 and 1974 was AC 275 supplied by Husky (Lloydminster).

Field Sampling and Laboratory Testing Program: Five 150 mm diameter cores at randomly selected locations were taken of the complete pavement structure in each of the three sections. At each of these core locations, the underlying subgrade was sampled to a 2 m depth.

Samples representing each of the three different years of construction were tested for density, asphalt content and gradation. The asphalt cement was recovered using the Abson method and tested for absolute viscosity at 60°C, kinematic viscosity at 135°C and penetration at 25°C and at 4°C, 100 g, 5 s and 200 g, 60 s. Temperature susceptibility parameters PVN', PVN and PI were calculated.

The subgrade soil samples were tested for moisture content and routine classification testing was carried out.

C.3 Test Results

The results of the testing program and other observations and data are presented in the following Tables and Figures:

Table C.1	Summary of Major Differences Between Sections
Table C.2	Transverse Crack Counts
Table C.3	Summary of Construction Quality Control Testing
Table C.4	Summary of Test Results of Asphalt Supplied
Table C.5	Summary of Test Results of Recovered Asphalt
Table C.6	Summary of Test Results of Pavement Cores
Table C.7	Soil Survey Test Results - 845:02-1

C.4 Discussion

The pavement constructed in 1973, 845:02-1, exhibited no transverse cracking compared to an average frequency of 21 cracks/km for Section 845:02-2. An initial review of construction quality control data showed very similar materials characteristics used in 1973 and 1974 which was constructed by the same Contractor using the same asphalt mix plant but different aggregate sources for each year of construction. It was only after the field coring program was completed that an additional 100 mm of asphalt pavement of unknown origin was discovered in the non-cracking section. It is assumed that this 100 mm of existing pavement was not cracked when overlaid in 1973. The recovered asphalt characteristics of this unknown pavement are very similar to those recovered from the 1973 pavement.

Given very similar as-constructed materials characteristics, it would be expected that the thicker pavement structure of 845:02-1 would be more resistant to transverse cracking. A review of the characteristics of the recovered asphalt shows that the asphalt recovered from the 1974 pavement has aged significantly more than the 1973 recovered asphalt. These results, presented in Table C.5 and Figure C.4 show recovered penetrations at 25°C of 51 and 80 respectively for the two sections. These results are also consistent with the cracking observed with the section with the harder asphalt cracking significantly. However, as shown in Table C.4 and Figure C.4, the as-supplied asphalt characteristics are virtually identical. A detailed review of construction quality control test results, presented in Table C.3, does not offer any explanation for the differences in asphalt aging between the two years of construction. Based upon the available information, the reasons for these differences cannot be explained. A review of the subgrade soil profiles indicates very similar characteristics between the two sections.

Within the portion of the project constructed in 1974, significant differences in cracking frequencies exist. This could be explained by the differences in pavement thicknesses between the medium, 845:02-2, and heavy, 845:02-3 cracking sections of 220 mm and 180 mm respectively. The boundaries of the heavy cracking section coincide identically with a split in surficial geology units. As shown in Figure C.3, the heavy cracking section is constructed over post-glacial wind deposited sands and silts. Comparison of the actual soil profiles for the two sections shows that the medium cracking section is constructed over medium plastic sandy clay whereas the heavy cracking section is constructed over more granular clayey sands. This is the main reason why only the 1980 overlay was reduced to a 60 mm thickness over the heavy cracking section. This section was stronger, as measured by the Benkelman beam, and required a thinner overlay to reduce rebound deflections to an appropriate level.

The nomograph stiffnesses of the original asphalt supplied in 1973 and 1974 were determined. These values are lower than the values determined for the original asphalts supplied to both SR 661:02 and 33:10. This suggests that the asphalt originally supplied was relatively resistant to low temperature transverse cracking.

In summary the difference in transverse cracking between sections can be explained as follows:

1. 845:02-1 (1973, non-cracking) and 845:02-2 (1974, medium cracking).

The thicker pavement structure is likely the main factor. The test results indicate that the pavement constructed in 1974 has aged significantly more than the 1973 project which would also be consistent with the cracking observed. Why the two pavements would age so differently considering they were built by the same Contractor using apparently identical asphalt cements supplied by the same refinery and very similar aggregates cannot be ascertained.

2. 845:02-2 (1974, medium cracking) and 845:02-3 (1974, heavy cracking).

The slightly thicker pavement has exhibited lower cracking frequencies. It is concluded however that subgrade effects are the major influence on cracking with the section constructed over the more granular sandy subgrade exhibiting greater transverse cracking frequencies than the section constructed over a medium plastic clay.

Table C.1 Project SR 845:02
Summary of the Major Differences Between Sections

Section	km - km	Ave Crack Frequency Cracks/km	Year of Construction	Asphalt Supplier	Design Pavement Thickness (mm)	"As-built" Pavement Thickness (mm)
845:02-1	6.4-15.6	0	1980	Husky (Lloyd)	100	90
			1973	Husky (Lloyd)	100	90
			19xx	Unknown	Unknown	100
						Total 280
845:02-2	15.6-19.5 22.7-25.6	21	1980	Husky (Lloyd)	100	120
			1974	Husky (Lloyd)	100	100
						Total 220
845:02-3	19.5-22.7	90	1980	Husky (Lloyd)	60	70
			1974	Husky (Lloyd)	100	110
						Total 180

Table C.2 Project SR 845:02
Transverse Crack Counts

Section	km - km	No. of Transverse Cracks
845:02-1	6.4-15.6	0
845:02-2	15.6-16.6	10
	16.6-17.6	23
	17.6-18.6	25
	18.6-19.6	36
845:02-3	19.6-20.6	100
	20.6-21.6	93
	21.6-22.7	87
845:02-2	22.7-23.7	25
	23.7-24.7	13
	24.7-25.6	13

Table C.3 Project SR 845:02
Summary of Construction Quality Control Testing

Section	Year of Construction	A.C. (%)	Compaction (%)	Core Density (kg/m³)	Core Air Voids (%)	Pugmill Discharge Temp. °C	Sieve Analysis - % Passing 5/8"	3/8"	#4	#10	#40	#200
845:02-1	1973	4.9	95.3	2279	7.4	143	100	75	52	35	21	6.7
845:02-2 & 845:02-3	1974	5.0	96.6	2301	6.4	144	100	74	54	40	21	6.1

Table C.4 Project SR 845:02
Summary of Test Results of
Asphalt Supplied in 1973, 1974 and 1980

Section	Abs.Visc. @60°C (Pa.s)	Pen @25°C 100g,5s (dmm)	Pen @4°C 100g,5s (dmm)	PVN'
SR 845:02-1 (1973)				
Tests on original asphalt	45.3	267	98	0.1
Tests on residue after TFOT	108.5	140	-	-0.1
SR 845:02-2&-3 (1974)				
Tests on original asphalt	45.0	273	100	0.1
Tests on residue after TFOT	112.1	141	55	-0.1
SR 845:02-1,-2&-3 (1980)				
Tests on original asphalt	45.0	260	-	0.0
Tests on residue after TFOT	113.0	131	-	-0.2

Table C.5 Project SR 845:02
Summary of Test Results of Recovered Asphalt

Section	Year of Construction	No. of Tests (N)	Abs.Visc. @60°C (Pa.s)	Kin.Visc. @135°C (mm^2/s)	Pen @25°C 100g,5s (dmm)	Pen @4°C 100g,5s (dmm)	Pen @4°C 200g,60s (dmm)	PVN'	PVN	PI
845:02-1	1973	5	227.6	402	80	10	30	-0.3	-0.5	-0.5
845:02-1	19xx	4	197.5	395	97	12	33	-0.1	-0.3	-0.5
845:02-2 &-3	1974	5	409.1	491	51	7	19	-0.4	-0.6	-0.2
845:02-1 -2 &-3	1980	5	204.3	396	95	12	33	-0.1	-0.3	-0.4

Table C.6 Project SR 845:02
Summary of Test Results of Pavement Cores

Section	Year of Construction	No. of Tests	Density (kg/m^3)	A.C. (%)	Air Voids (%)	V.M.A. (%)	Sieve Analysis - % Passing 16 000	10 000	5000	1250	630	315	160	80
845:02-1	1973	5	2317	4.9	6.2	15.5	100	76	47	27	23	16	9.8	6.6
845:02-1	19xx	5	2309	5.0	6.7	16.0	100	78	51	29	24	18	12.0	9.0
845:02-2 &-3	1974	5	2324	4.7	6.2	15.1	99	78	54	31	25	18	10.9	7.8
845:02-1 -2 &-3	1980	5	2345	5.1	5.4	15.0	100	80	53	34	29	17	10.4	7.3

Table C.7 Project SR 845-02-1 (Non-cracking Section)
Soil Survey Test Results

Sample Depth (m)	% Passing Sieve Size (µm) 5000	400	160	80	LL (%)	PL (%)	PI (%)	Field M.C. (%)	LI	Est. Opt. M.C. (%)	Est. Max. Density (kg/m³)	Soil Class.
Hole #: 1	Station: 10.175			Location: 0.4 m rt ℄								
0.35	99	95		83.1	39.5	15.4	24.1	17.3	0.07	16	1800	CI
1.40	95	95		84.4	53.1	22.2	30.9	26.6	0.14	23	1550	CH
1.60	99	99		86.4	45.2	19.2	26.0	25.1	0.22	20	1650	CI
Hole #: 2	Station: 10.800			Location: 2.1 m lt ℄								
0.38	100	96		86.0	41.4	18.8	22.6	20.2	0.06	20	1680	CI
0.90	100	99		90.4	44.5	21.4	23.1	28.4	0.30	22	1570	CI
1.50	99	97		84.5	43.9	22.6	21.3	28.2	0.26	23	1540	CI
Hole #: 3	Station: 12.[illegible]00			Location: 1.0 m rt ℄								
0.40	99	95		74.4	44.5	18.1	26.4	22.6	0.17	20	1690	CI
0.90	99	98		83.6	37.2	18.6	18.6	21.7	0.16	20	1690	CI
1.60	100	99		79.0	37.1	19.6	17.5	19.9	0.01	20	1650	CI
Hole #: 4	Station: 13.560			Location: 3.4 m rt ℄								
0.40	98	95		71.9	36.3	16.4	19.9	18.4	0.10	17	1770	CI
0.90	99	98		85.1	41.0	20.5	20.5	23.6	0.15	21	1610	CI
1.60	99	98		84.7	40.3	21.2	19.1	24.3	0.16	22	1580	CI
Hole #: 5	Station: 14.220			Location: 2.6 m lt ℄								
0.425	98	97		79.9	38.4	17.1	21.3	21.0	0.18	18	1730	CI
0.900	99	99		74.0	33.2	17.6	15.6	21.9	0.27	17	1730	CI
1.500	100	100		77.4	37.7	19.2	18.5	20.5	0.07	20	1660	CI

Table C.8 Project SR 845:02-2 (Medium Cracking Section)
Soil Survey Test Results

Sample Depth (m)	% Passing Sieve Size (μm) 5000	400	160	80	LL (%)	PL (%)	PI (%)	Field M.C. (%)	LI	Est. Opt. M.C. (%)	Est. Max. Density (kg/m³)	Soil Class.
Hole #: 6	Station: 18.440		Location: 0.5 m rt ℄									
0.325	100	99		77.6	36.0	18.4	17.6	18.6	0.01	19	1690	CI
0.900	100	100		80.7	38.4	20.6	17.8	20.2	-0.02	21	1610	CI
1.600	100	100		80.1	36.4	18.4	18.0	18.5	0.00	19	1690	CI
Hole #: 7	Station: 18.880		Location: 2.1 m lt ℄									
0.340	100	99		65.2	32.9	16.3	16.6	14.7	-0.09	16	1770	CI
0.850	100			69.7	34.6	17.0	17.6	18.1	0.06	17	1730	CI
1.400	100			73.1	35.7	20.2	15.5	19.9	-0.01	21	1640	CI
Hole #: 8	Station: 23.224		Location: 0.9 m rt ℄									
0.325	100	99		77.7	33.0	15.0	18.0	17.3	0.12	15	1840	CI
0.900	98	98		76.9	36.2	15.9	20.3	16.3	0.01	16	1770	CI
1.400	100			78.8	35.7	20.0	15.7	24.0	0.25	21	1650	CI
2.50	100	100		73.9	34.2	20.0	14.2	26.6	0.46	20	1650	CI
Hole #: 9	Station: 24.104		Location: 3.4 m rt ℄									
0.35	100	99		78.3	32.4	16.0	16.4	19.3	0.20	16	1810	CI
0.85	100	99		70.4	29.2	16.9	12.3	18.6	0.13	16	1770	CL-CI
1.40	100	99		63.5	26.4	15.6	10.8	16.3	0.06	14	1840	CL
Hole #: 10	Station: 24.632		Location: 2.7 m lt ℄									
0.35	100	99		79.9	35.4	15.8	[illegible].6	19.3	0.17	16	1800	CI
0.85	98	94		71.1	30.9	16[illegible]	14.8	18.0	0.12	15	1810	CI-CL
1.50	99	98		88.1	35.0	17.8	17.2	17.7	0.00	18	1720	CI

Table C.9 Project SR 845:02-3 (Heavy Cracking Section) Soil Survey Test Results

Sample Depth (m)	% Passing Sieve Size (µm) 5000	400	160	80	LL (%)	PL (%)	PI (%)	Field M.C. (%)	LI	Est. Opt. M.C. (%)	Est. Max. Density (kg/m³)	Soil Class.
Hole #: 11	Station: 19.820			Location: 3.1 m lt ℄								
0.265	100	97	67.6	45.6	20.9	13.4	7.5	9.0	-0.58	N/A	1960	SC
0.80	100	98	63.4	41.4	22.9	13.3	9.6	10.3	-0.31	11	1930	SC
1.40	100	98	57.8	33.9	22.2	15.8	6.4	11.2	-0.71	13	1850	SC-SMd
Hole #: 12	Station: 20.780			Location: 3.0 m rt ℄								
0.25	100	99	72.7	47.9	24.1	14.6	9.5	12.7	-0.20	13	1890	SC
0.75	100	99	69.8	43.9	23.1	14.6	8.5	14.7	0.01	13	1890	SC
1.40	100			53.7	27.5	15.8	11.7	21.7	0.50	14	1820	CL
Hole #: 13	Station: 22.065			Location: 0.7 m rt ℄								
0.27	99	95		49.5	25.0	14.7	10.3	12.9	-0.17	13	1880	SC
0.80	100	99		57.0	25.9	15.2	10.7	13.5	-0.15	13	1850	CL
1.40	100	100	79.5	49.5	24.2	16.1	8.1	13.5	-0.32	14	1820	SC
Hole #: 14	Station: 22.188			Location: 3.0 m lt ℄								
0.30	100	100		52.6	25.2	14.4	10.8	12.8	-0.14	13	1890	CL
0.80	100	98		51.6	26.4	15.2	11.2	16.8	0.14	13	1850	CL
1.40	100	99	78.6	50.6	25.7	15.9	9.8	17.8	0.19	14	1840	CL
Hole #: 15	Station: 21.500			Location: 2.1 m lt ℄								
0.30	98	93	55.8	30.1	17.3	14.1	3.2	6.8	-2.28	N/A	1970	SMd
0.80	100	100	64.8	36.9	21.3	13.8	7.5	10.1	-0.49	11	1930	SC
1.40	100	98	57.2	19.7	18.2	15.7	2.5	7.5	-3.28	12	1890	SMd

Table C.10 Project SR 845:02
Stiffness Calculations

Input required for stiffness calculations:

- characteristics of original asphalt:

  845:02-1 (1973) - penetration @25°C = 267 dmm
  PVN' = 0.1

  845:02-2 (1974) - penetration @25°C = 273 dmm
  PVN' = 0.1

- characteristics of original asphalt after TFOT:

  845:02-1 (1973) - penetration @25°C = 140 dmm
  PVN' = -0.1

  845:02-2 (1974) - penetration @25°C = 141 dmm
  PVN' = -0.1

- minimum ambient temperature = -38°C
  minimum pavement surface temperature = -32°C (prior to 1980 overlay)
  minimum pavement temperature @ 50 mm depth = -29°C (prior to 1980 overlay)

- in-place asphalt pavement characteristics:

  845:02-1 VMA = 15.5%
  Air Voids = 6.2%

  845:02-2 VMA = 15.1%
  Air Voids = 6.1%

	845:02-1 & -2			
Asphalt	Original Supplied		Original After TFOT	
Pvmnt Temp. °C	-32	-29	-32	-29
Stiffness (binder) Pa	2.2×10^6	9.8×10^5	1.4×10^7	5.9×10^6
Stiffness (mix) Pa	1.1×10^9	-	-	-

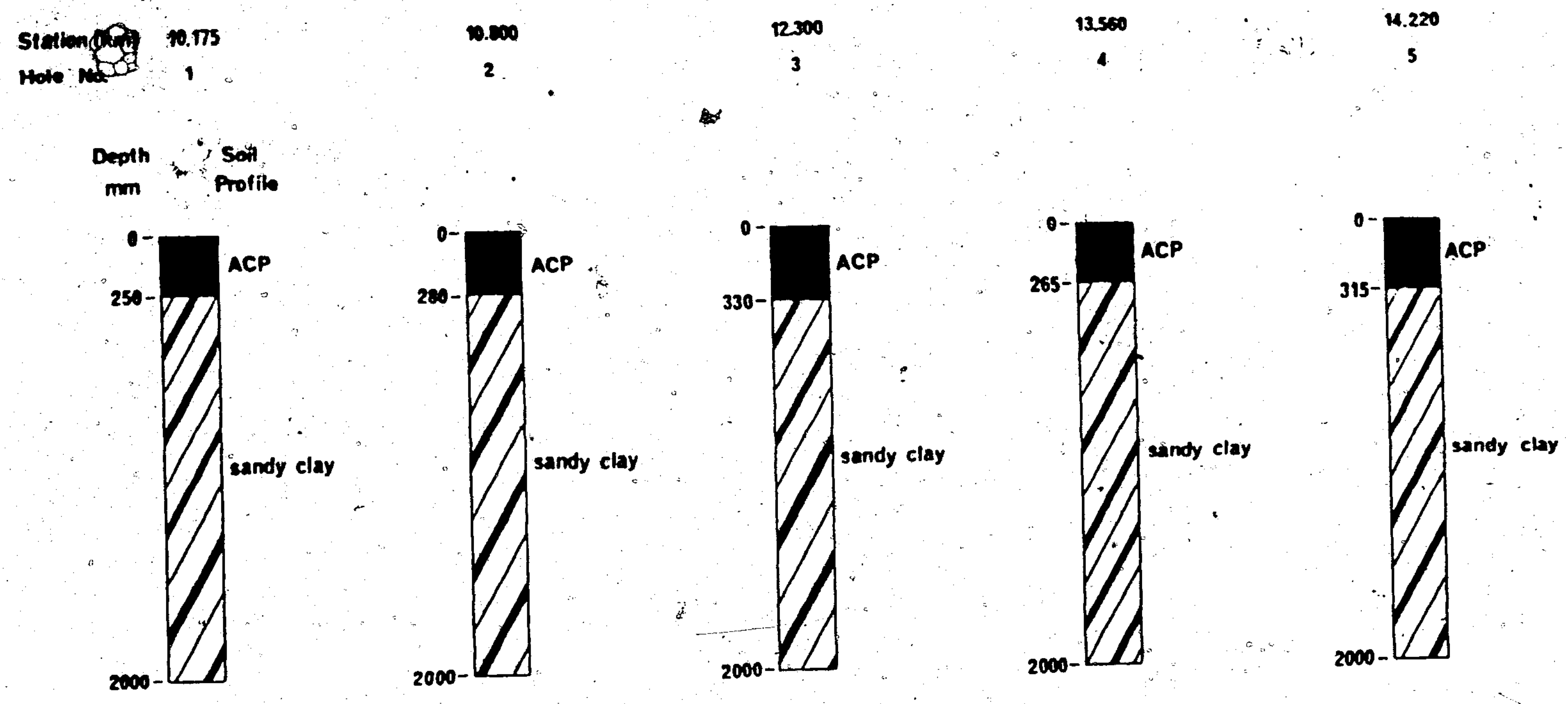


Fig. C.1 Project SR 845:02 (Non-cracking Section)
Subgrade Soil

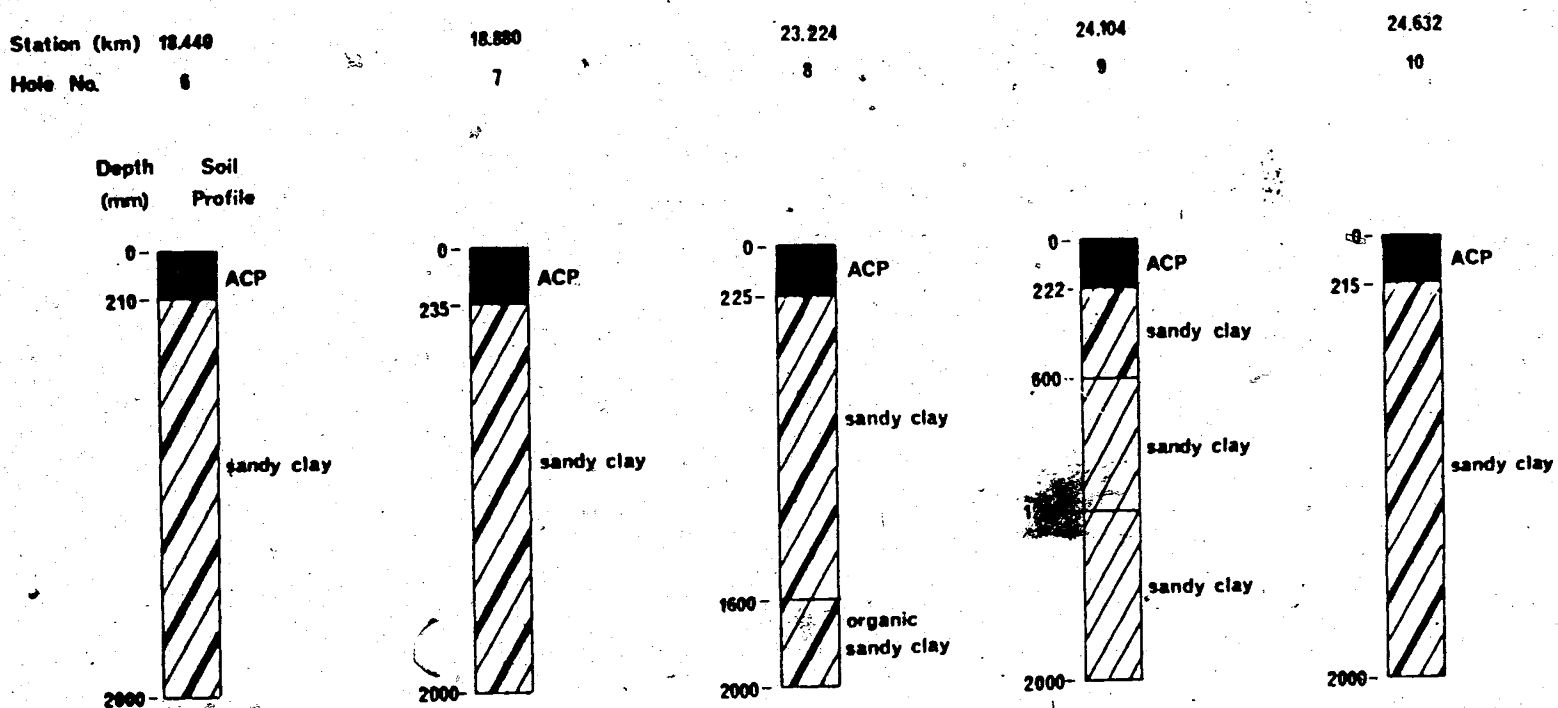


Fig. C.2 Project SR 845:02-2 (Medium Cracking Section)
Subgrade Soil Profiles

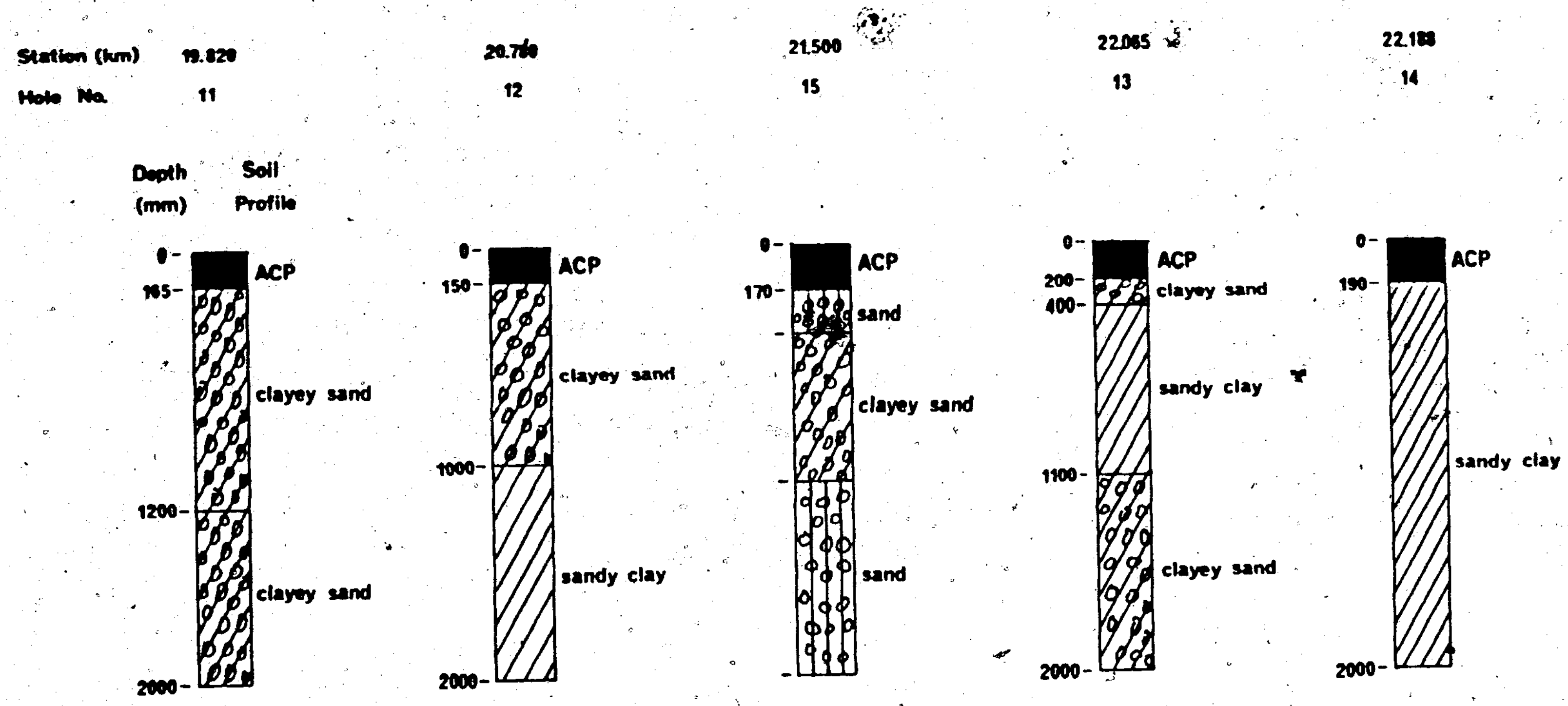


Fig. C.3 Project SR 845:02-3 (Heavy Cracking Section)
Subgrade Soil Profiles

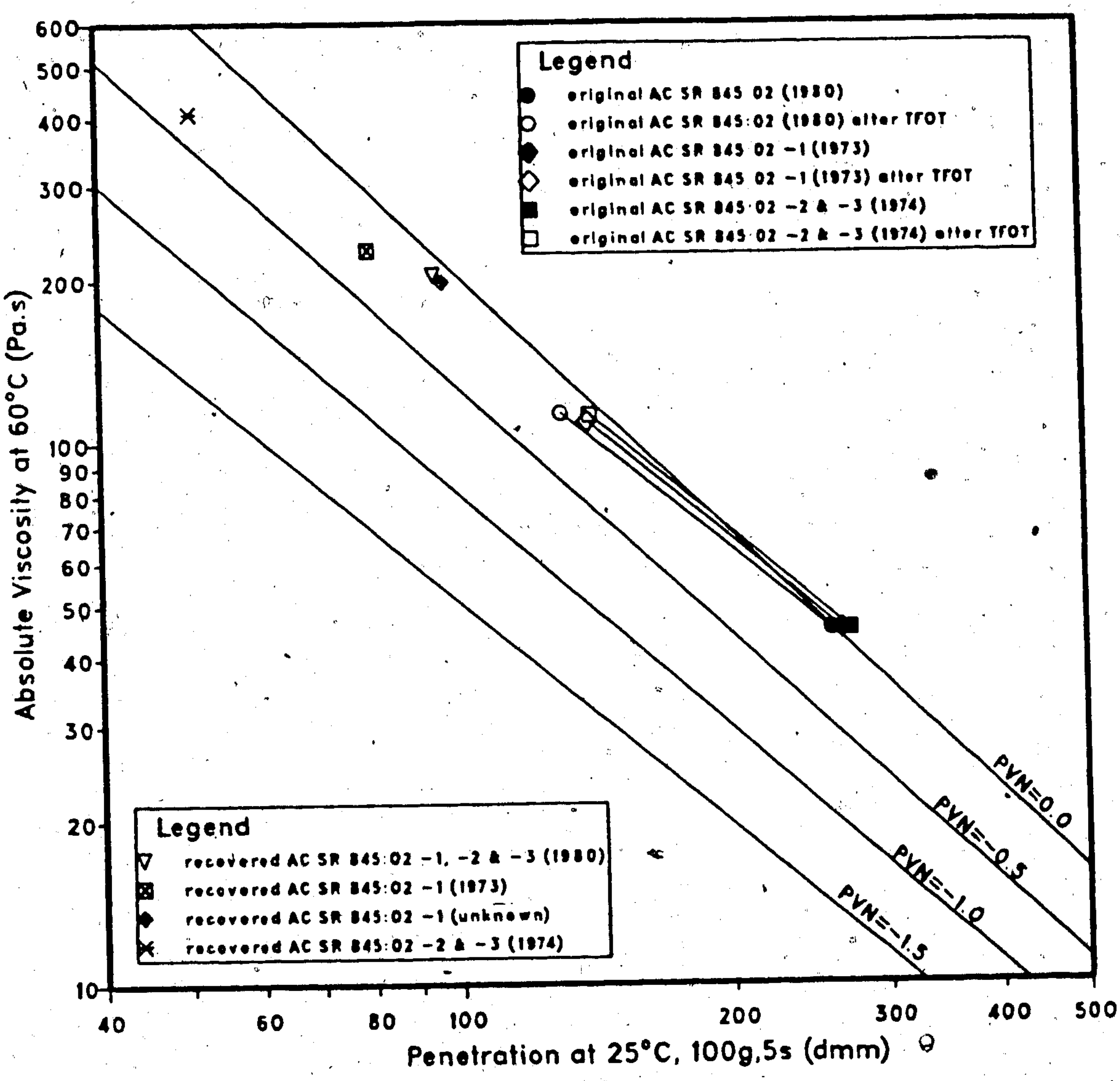


Fig. C.4 Project SR 845:02
Asphalt Penetration-Viscosity Relationships

APPENDIX D

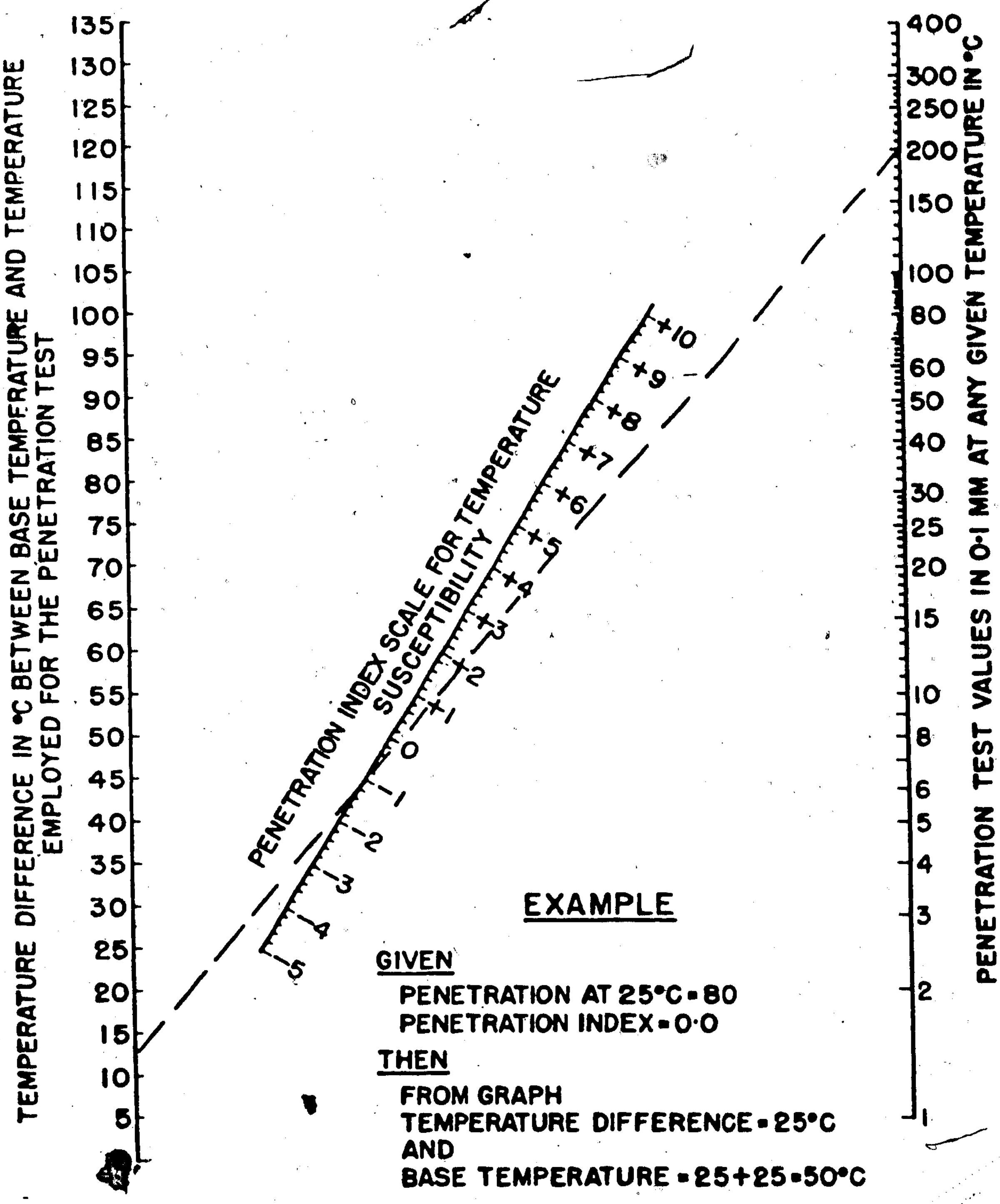


Fig. D.1 McLeod's Modification of Heukelom's Version of Pfeiffer's and van Doormal's Nomograph for Determining Base Temperature

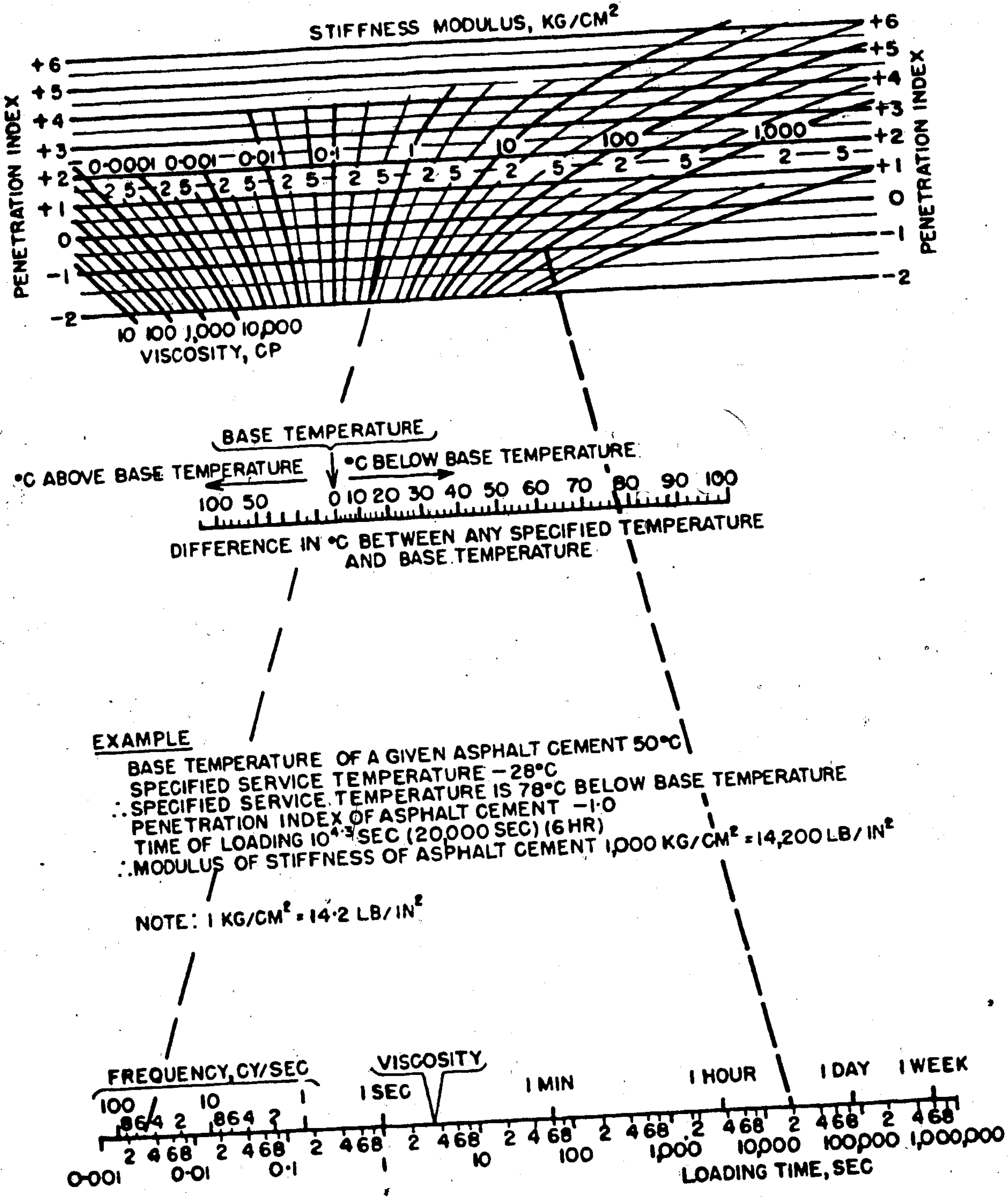


Fig. D.2 McLeod's Modification of Heukelom's and Klomp's Version of van der Poel's Nomograph for Determining the Stiffness of Asphalt Cements

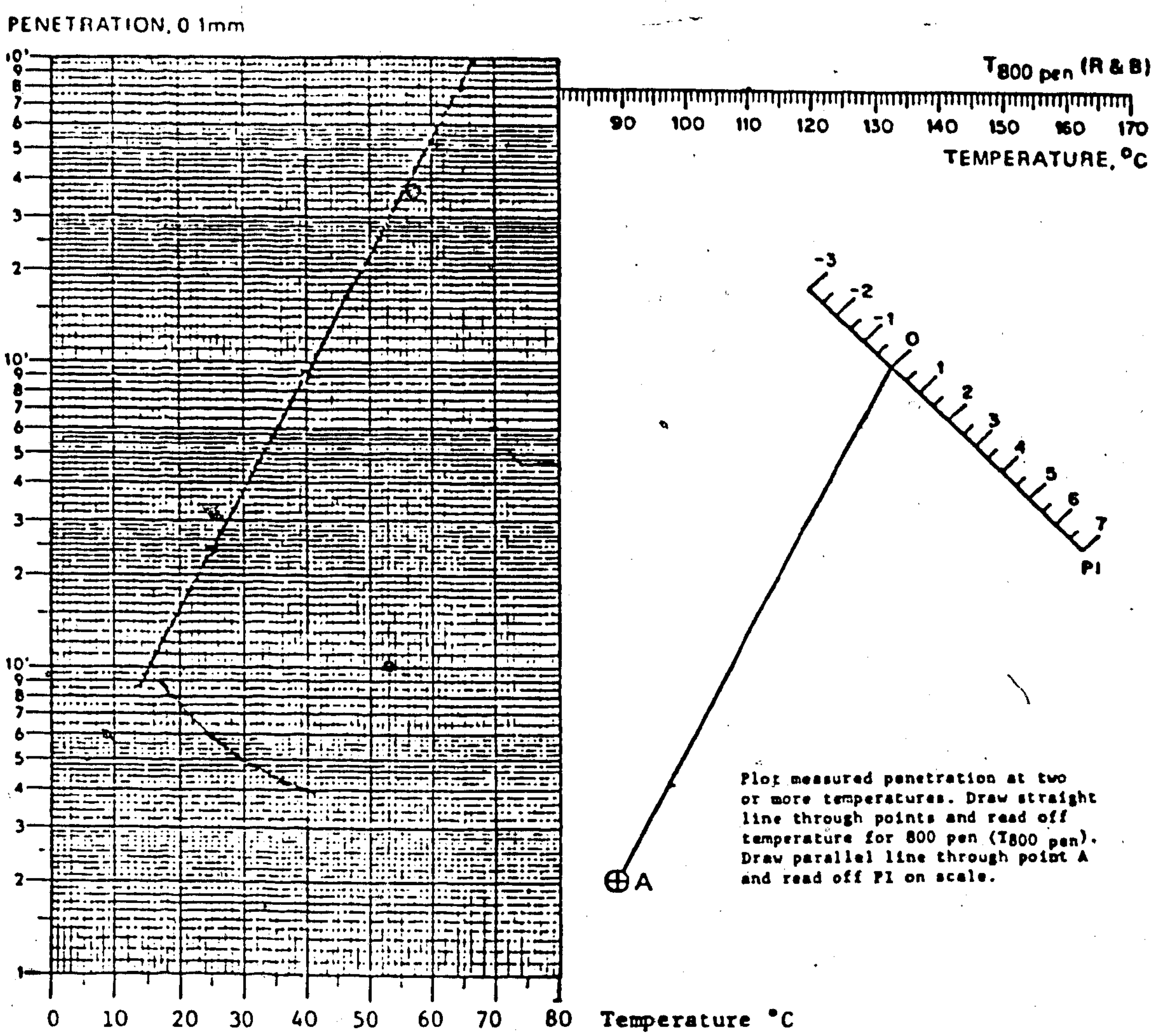


Fig. D.3 Chart for Determining Penetration Index (PI)

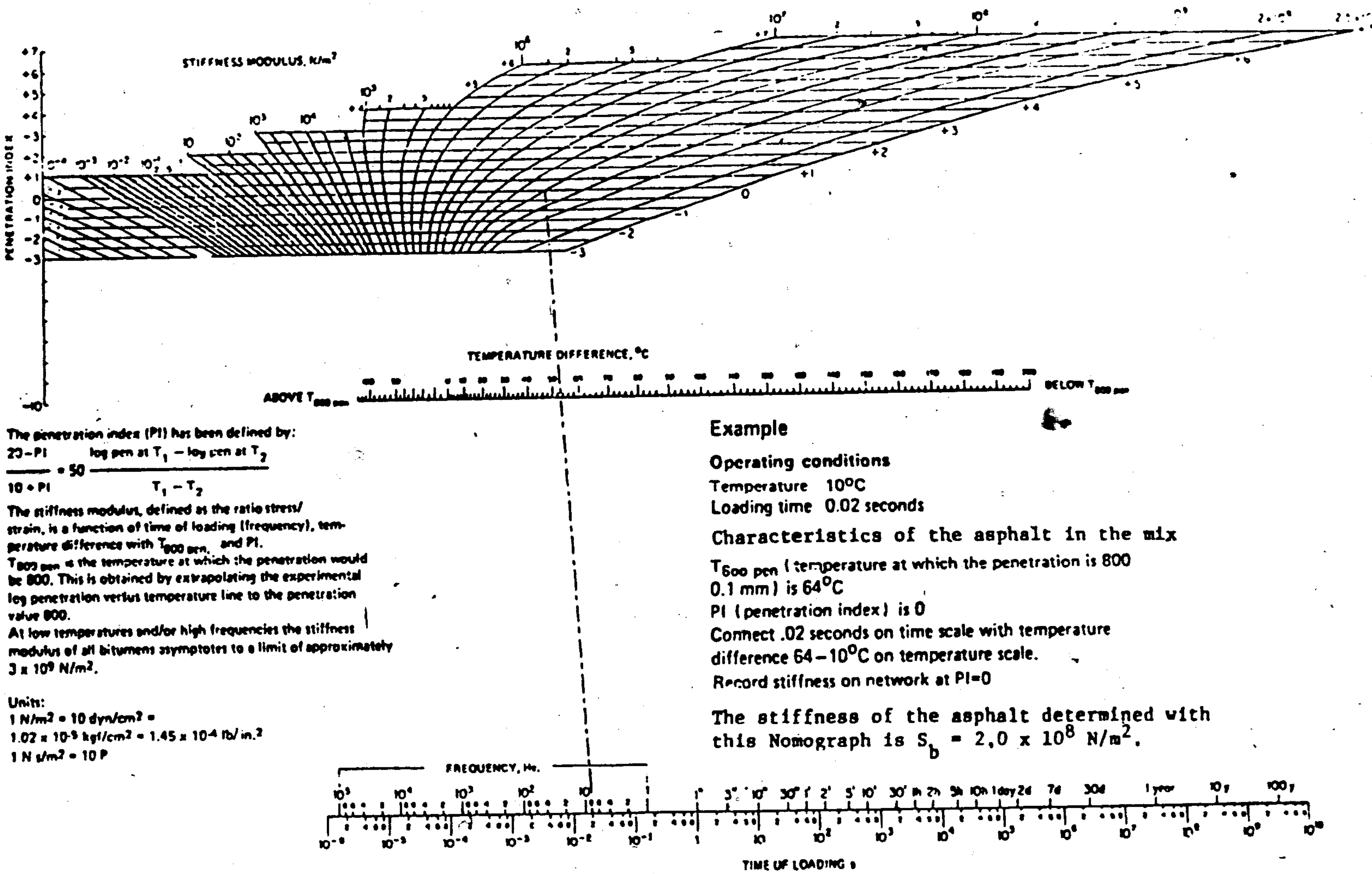


Fig. D.4 van der Poel's Nomograph for Determining the Stiffness of Asphalt Cements

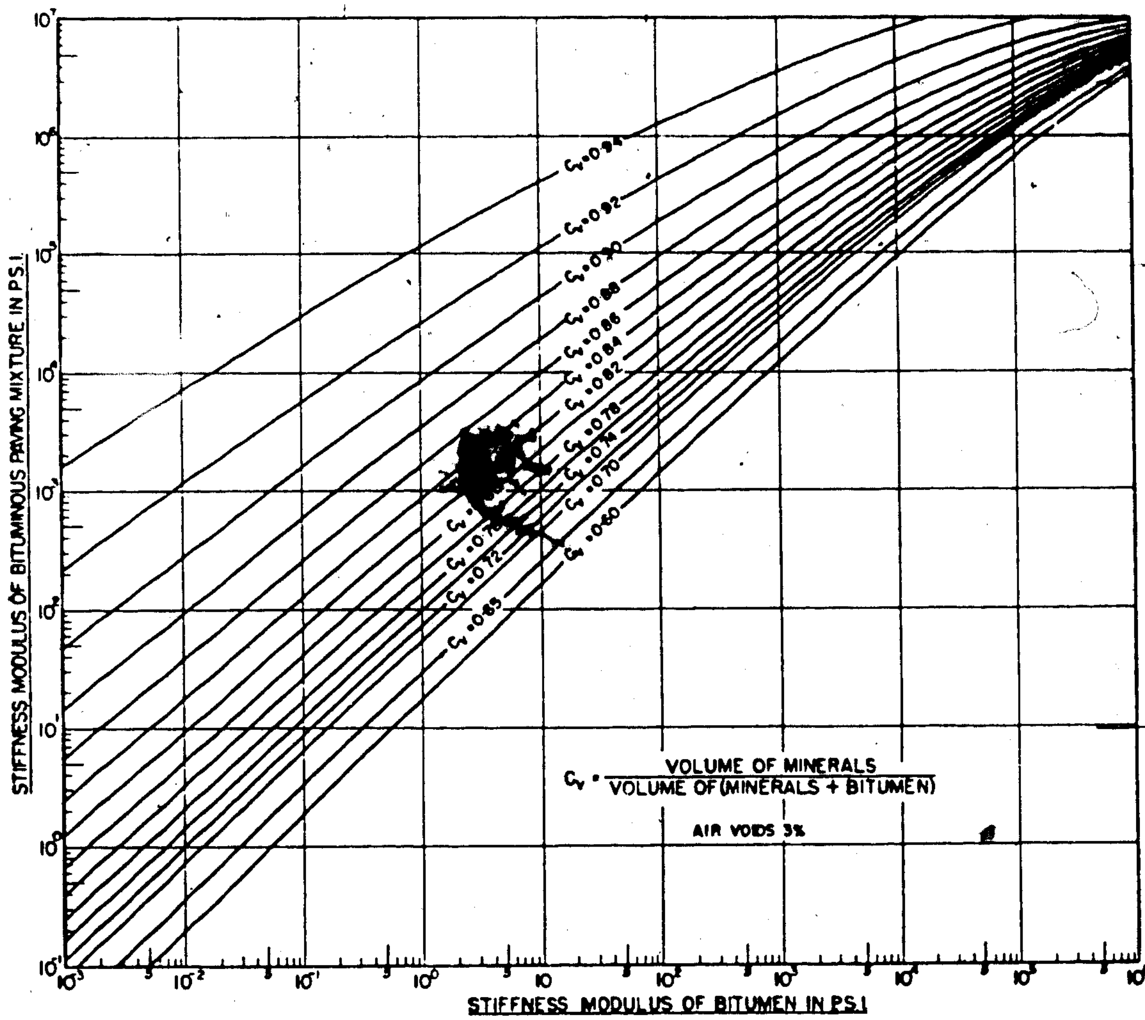


Fig. D.5 Relationship Between Moduli of Stiffness of Asphalt Cements and of Paving Mixtures Containing the Same Asphalt Cements