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LOW-TEMPERATURE PERFORMANCE OF
ASPHALTIC CONCRETE PAVEMENTS WITHIN
THE REGIONAL MUNICIPALITY OF OTTAWA-CARLETON

by

Robert J. Singleton, B.Sc.(Eng.), P. Eng.

A thesis submitted to the Faculty of
Graduate Studies in partial fulfillment
of the requirements for the degree of
Master of Engineering

Faculty of Engineering
Carleton University
Ottawa, Ontario

1978

THE REGIONAL MUNICIPALITY OF OTTAWA-CARLETON 1978 REGIONAL ROAD SYSTEM - RURAL -

LEGEND

REGIONAL ROADS
HIGHWAYS
MONITORING STATION

Map made by the T. L. H. A. in 1978, 1:50,000 scale

The undersigned recommend to the Faculty of
Graduate Studies acceptance of the thesis
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ASPHALTIC CONCRETE PAVEMENTS WITHIN
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ABSTRACT

The results of an investigation of asphaltic concrete pavements within the rural areas of the Regional Municipality of Ottawa-Carleton with respect to changes in the character of existing transverse pavement cracks at low temperatures for both newly constructed roadways and roadway surfaces which received an asphaltic concrete overlay are presented.

Results are compared with conclusions drawn from other investigations in the field of low-temperature asphaltic concrete pavement performance.

Recommendations are made suggesting additional investigations to study the performance of asphaltic concrete overlays with respect to reflection cracking and low-temperature conditions.

The frequency of occurrence of transverse pavement cracks is related to traffic volumes, age of asphaltic concrete, type of asphalt cement used in the pavements and recovered penetration values of the asphalt cement used.

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CHAPTER 1

INTRODUCTION

1.1 GENERAL

The funding and construction of asphalt pavements are major priorities of the road building authorities of Canada and the United States today. Pavement structures are designed such that for a given set of criteria and constraints, some of which are fixed (i.e. roadway widths, subgrades) and others which are variable (traffic volumes, environmental conditions), the product will have a theoretical life expectancy which relates directly to the design criteria. Life expectancy of an asphalt pavement structure is normally measured in years and can be loosely defined as the number of years from the date of completion of construction and subsequent traffic movement, that the pavement provides a satisfactory and safe ride for the passage of persons and goods when being transported by motorized vehicles. The point in time at which the asphalt pavement is no longer fulfilling its designed purpose may be determined either by the pavement performance rating established by RTAC or a similar version of this method as revised by local municipalities, or by a maintenance performance system through the experience of personnel assigned to establishing repair requirements of pavement structures.

As the life expectancy date of a pavement structure approaches, funding again becomes a requirement to enable the road building authority to replace or repair the outlived pavement. In many instances, because of lack of funds or rearrangement of priorities, a pavement structure

is forced to endure beyond its life expectancy and usually with a very poor performance in so doing. All too often, however, asphalt pavement structures are failing in one way or another well in advance of the design life of the roadway. In some cases this is due to substantial increases in traffic volumes or loads during the design life, in other instances to inferior construction, the use of poor quality materials (and inadequate quality control).

In the design and construction of asphalt pavements, one of the most important components of the pavement mix is the asphalt cement. An improper selection of the grade of asphalt cement to be used in the design mix can lead to a substantial decrease in the life of an asphalt pavement.

The primary objective of this thesis is to evaluate the performance of asphaltic concrete pavements constructed under the jurisdiction of the Regional Municipality of Ottawa-Carleton (RMOC) by monitoring the changes in character of existing transverse pavement cracks over the range of temperatures occurring during the period November 1, 1976 to May 30, 1977, for both newly constructed roadways and roadways which had received an asphaltic concrete overlay.

In addition the frequency of occurrence of transverse pavement cracking in the road surfaces under consideration will be investigated to establish relationships with:

- a) traffic volumes
- b) age of asphaltic concrete
- c) type of asphalt cement used in the mix design
- d) recovered penetration values of asphalt cement
- e) type of roadway reconstruction

1.2 LITERATURE REVIEW

Asphalt pavements are usually referred to as flexible pavements in that they have the ability to deform over a wide range of temperature and load conditions without the use of man-made expansion joints. Past experience has proven that the performance and life expectancy of flexible pavements when in an environment where the ambient temperatures are above the freezing mark are both superior to the condition wherein a flexible pavement must perform under extremes of temperature both above and below the freezing point. As the ambient temperature continues to drop, an accumulation of tensile stresses and strains leads to failure of the asphalt pavement in the form of a transverse crack. This condition, transverse pavement cracking associated with low temperatures, represents one of the most common defects in flexible pavements in cold climates. Initially the cracks are microscopic in nature, however the cracks eventually cause a major concern in that deterioration in and around the cracks results in a pavement surface which must be repaired. Haas (42) has suggested that the maintenance cost for repairing pavement cracks in the Province of Saskatchewan was approximately \$2,000. per mile annually in the mid 1960's. Extrapolated to the total surface miles of highways in that province at the time, the annual maintenance cost was approximated at about 2.5 million dollars.

Along with repair costs another major aspect is the reduced service life of the pavement surface. Anderson et al (39) have pointed out extreme examples where pavements have had to be resurfaced after approximately one-fourth of the expected design life; this was due in part to riding quality but mainly to a vast amount of extreme cracking and patching on the pavement surface. The increasing costs of

providing asphalt surfaces today, which seem to be directly related to the price of oil in today's world markets, are now requiring that research and design methodology provide a pavement surface that will give a satisfactory life expectancy of 15 to 20 years.

Transverse pavement cracking did not become a major problem in Canada until the introduction of hot mixed pavements in the early 1950's. It has been under examination since the 1950's with extensive work being carried out in the 1960's and continuing into the 1970's. The leading experts in the field of transverse pavement cracking appear to agree that the problem is definitely due to low temperatures.

(McLeod; 3) Research is continuing along the following lines.

1. -to determine the exact mechanisms for the cracks that do occur;
2. -to predict the extent of cracking through modelling techniques;
3. -as a result of 1. and 2., to update asphalt specifications to eliminate or reduce as much as possible the low temperature cracking problem in the field when the asphalt pavements are constructed. The leaders in the field of investigation of the above problem areas in Canada are the Departments of Highways of the Provinces of Alberta, Saskatchewan, Manitoba and Ontario together with several consulting firms and asphalt cement manufacturers. Extensive field and laboratory investigations by the above-mentioned Highway Departments have verified that the single most outstanding factor contributing to the cracking of asphalt pavements is the grade of asphalt cement used in the design mix. *

* - Assuming that all other phases of construction are performed in accordance with accepted practice and specifications.

Hajek (40) has reported the following: "Observation and research has shown that several variables influence low temperature cracking: materials of asphalt concrete, age and thickness of asphalt concrete layers, environmental conditions and subgrade type. The greatest research attention has been devoted to the bituminous component of the asphalt concrete because this was identified as one of the most important factors influencing low temperature cracking. Moreover, asphalt cement is the easiest factor to change, without extensive additional expenditures, when it appears that an intolerable amount of cracking may occur."

Initially it would imply that transverse pavement cracking could be reduced or avoided with the use of a softer grade of asphalt cement in the asphalt mix. From the results of research conducted to date it would appear that this statement has been verified in most instances.

Work conducted in the field and laboratory by several agencies is outlined in the following sections.

1.2.1 Saskatchewan

In 1963 a test road was constructed in Saskatchewan to investigate the causes of regularly spaced transverse cracks in flexible pavements. The objectives of this particular test road were the following:

- (1) To determine if the crude oil source and refining methods of the penetration asphalt cement were significant variables.

- (2) To determine if the penetration grade of asphalt cement used was too low, or the asphalt too "hard" for the purpose for which it was being used.
- (3) To determine if the bond provided by a prime coat provides excessive restraint to the expansion-contraction behaviour of the surface course.

This particular study and its results are reported by Culley (43). The most significant result of this study was to establish a relationship between asphalt source * and the amount of transverse cracking that occurred at low temperatures in the field. A lesser degree of cracking occurred in pavements using a softer asphalt cement. The interpretation of the results of this experiment led to a new specification based on more accurate and stringent viscosity limits for asphalt cement and asphalt usage programs based on the viscosity criteria. A part of the new specification was a minimum penetration of 150 required for all asphalt cements.

1.2.2 Manitoba

The loss in serviceability of asphalt pavements in the Province of Manitoba, due in part to transverse cracking and its side effects of spalling and heaving or settlement, led to a joint venture between Shell Canada Limited and the Manitoba Department of Highways in constructing the St. Anne test road in 1967 to evaluate the transverse pavement cracking problem as it relates to asphalt cement types. (Deme; 11, Young et al; 14)

- * - Asphalt source refers to the manufacturer of the asphalt cement. Initially, specifications for asphalt cement were insufficiently stringent to obtain an identical product from several manufacturers.

Asphalt cements used in the construction of various sections of the St. Anne test road were of the four following types:

- (1) low viscosity 150-200 penetration;
- (2) high viscosity 150-200 penetration;
- (3) low viscosity 300-400 penetration;
- (4) high viscosity SC5.

A survey of the pavement test sections made with the asphalt cements listed above after two years of service showed the number of type 1 transverse cracks per mile as listed in the table below. (A type 1 transverse crack is one that extends completely across one lane of the pavement surface.)

<u>Binder Contained in Pavement Test Sections</u>	<u>Number of Type 1 Transverse Cracks Per Mile</u>
Type 150-200 LVA	260
Type 300-400 LVA	81
Type 150-200 HVA	0
Type SC-5	0

Several interesting conclusions have been drawn from the results of this extensive test program. Firstly it was concluded that the frequency of transverse crack occurrence was not significantly related to the differences in traffic volumes, the asphalt content, and the incorporation of cement fillers into the mixes. The asphalt cement grade proved to be significant in that the softer penetration grade asphalt exhibited approximately 30% of the transverse cracks observed in the harder grade asphalt cements. Thirdly, the asphalt cement type was

significant in that the high viscosity type of penetration grade asphalt pavements showed no signs of transverse cracking and, over particular clayey or sandy subgrades, were infinitely superior in resisting the formation of transverse cracking when compared with the low viscosity type of penetration grade asphalt pavements. Fourthly, the high viscosity SC-5 asphalt pavement which was both of high viscosity and soft grade asphalt cement showed no signs of transverse cracking on all types of subgrades.

The subgrade type had a definite influence on the low viscosity penetration grade asphalt. The low viscosity 100-200 penetration grade asphalt exhibited much greater transverse cracking frequency in the pavements with sand subgrades than in the pavements with clay subgrades, however, the reverse was found with a low viscosity 300-400 penetration grade asphalt pavements. Also the depth of asphalt pavement had a definite relation to the transverse cracking that occurred, the thicker the pavement depth the less transverse cracking had occurred. It should also be noted that the results of this particular field investigation showed that the temperature susceptibility of the asphalt cement, i.e. high viscosity and low viscosity, as well as the penetration grade of the asphalt cement to be both factors which definitely influence the transverse pavement cracking frequency.

1.2.3 Alberta

In 1966 the Province of Alberta constructed a four-lane divided freeway in central Alberta to determine the influence of asphalt source on low temperature transverse cracking. The Department incorporated within one contract

three different sources of 200-300 penetration grade asphalt cement. The three sources of asphalt cement represent high, intermediate, and low viscosity materials as measured at 140°F. The results of this particular experiment as reported by Shields (4) were that the viscosity-temperature sensitivity of the asphalt cements supplied which range from low to high resulted in asphalts with the greatest temperature sensitivity tending to crack first. An overall conclusion from the above field tests is that the frequency of transverse pavement cracking can be reduced by the use of a softer grade of asphalt cement. The Alberta experiment also concluded that the subgrade conditions play a significant role in the formation of transverse pavement cracks.

1.2.4 Some Laboratory Results

Anderson and Shields (26) have shown in experiments in Alberta that low viscosities of asphalt at 140°F are particularly susceptible to transverse cracking at low winter temperatures. Results of tensile splitting tests on three 200-300 penetration grade asphalt pavement samples have led them to the following conclusions.

1. An increase in cracking frequency is accompanied by a decrease in failure strain and an increase in failure stress and failure stiffness.
2. An increase in density with service life is accompanied by a decrease in failure strain and an increase in failure stress, failure stiffness, and cracking frequency.
3. With respect to service life the low viscosity asphalt pavement exhibits the greatest change in density, failure stress, failure strain, and failure stiffness

and the highest cracking frequency per mile. From the results of these tests the Province of Alberta adopted a specification in 1967 incorporating a minimum penetration of 250 at 77°F and a minimum viscosity of 275 poises at 140°F for asphalt cements. This has resulted in a more uniform product and elimination of tender mixes and a reduction in early transverse cracking on studies to date. In previous work Shields and Anderson(41) have indicated transverse cracking results from temperature variations in the asphalt pavement. They have suggested the following as a possible means of the crack initiation in the pavement. A contraction of the asphalt surface course due to low temperature, which in turn is restrained by an underlying base course, if followed by a rise in ambient temperature will cause a weakening of the highly stressed asphalt surface course and, if a contraction of the subgrade occurs then a pavement transverse crack will commence at the surface.

Laboratory studies conducted at the University of Saskatchewan by Domaschuk, Skarsgard, and Christianson (37) on the stress-strain characteristics of asphalt mixtures concluded that asphaltic concrete may crack due to thermal contraction under one of the following two conditions:

- 1) a prolonged period of air temperatures in the order of -4°F;
- 2) depressing temperatures beyond -4°F followed by sudden rises in temperatures above -4°F.

Laboratory studies carried out by Monismith et al (1) have verified that cracking occurs because thermal stresses reach the breaking strength of the asphaltic concrete surface at temperatures significantly below 0°F. The

stresses developed at the low temperatures because of restraint of movement will be less than the breaking strength of the asphalt concrete at a specific temperature. A sudden rise in temperature and warming of the surface may cause a considerable reduction in the fracture strength of the asphalt concrete thereby allowing it to develop a crack.

A relationship between low-temperature cracking of asphalt pavements and softness of the asphalt cement was suggested by Hills and Brien (38). From their laboratory tests they conclude that low-temperature cracking decreased as the penetration grade of asphalt cement increased.

1.2.5 Ontario

Extensive studies and experiments have been conducted in Ontario by both private industry and the Ministry of Transportation and Communications to determine the causes of transverse pavement cracking and to find a solution to this particular problem. The conclusions and recommendations of the majority of the work done in Ontario have been summarized in various papers by McLeod. (3, 19, 30, 32)

Three Ontario test roads were constructed in 1960 in an effort to investigate transverse pavement cracking at low temperatures. Three different 85-100 penetration asphalt cements were used from three different suppliers in each test road. The three 85-100 penetration asphalt cements were widely different in temperature susceptibility and this was chosen particularly to determine the influence of asphalt temperature susceptibility on the performance of

the pavement. These test roads were monitored yearly and after nine years of service, surface core samples were extracted from each of the test roads and completely analysed by Imperial Oil's Research Department. In an analysis of these results, McLeod (32) has shown that the Pfeiffer and Van Doormaal penetration index (3), which is a measure of temperature susceptibility, was not adequately sufficient to describe the causes of the transverse pavement cracking with respect to the asphalt cement used. As a result of this, McLeod developed a variation in the temperature susceptibility of asphalt cement measurements by developing the pen-vis (penetration-viscosity) number to replace the penetration index number for asphalt cements. This method of measurement of temperature susceptibility is based on the penetration of an asphalt cement at 77°F and on its viscosity in centistokes at 275°F, thus the term pen-vis number. The results of a transverse pavement crack survey after nine years on all three test roads showed that the number of type I transverse cracks per lane mile increased with a decrease in pen-vis number of the original asphalt cement. The transverse pavement crack surveys for these roads were conducted in the 8th, 9th, 10th and 11th years of service life and it was found that there was a substantial annual increase in the number of type I transverse cracks per mile for each of the four years of service for which data had been collected, thereby indicating that the number of transverse pavement cracks increased with the age of the asphalt pavement. (McLeod; 32)

From the analysis of the samples taken in the field it was shown that the modulus of stiffness increases with the age of the asphalt and also with the number of transverse

pavement cracks. The general conclusion drawn from this particular experiment was that the 85-100 penetration asphalt cement was too hard to rely on a 20-year life of a pavement without substantial damage by transverse pavement cracking.

In another field study conducted in 1961 by the Ontario Department of Highways a nine mile test pavement was constructed with the only variable being the penetration grade of asphalt cement. Both 150-200 penetration asphalt cement and 85-100 penetration asphalt cement were used with pen-vis numbers of -1.5 and -1.4 respectively. After four years of service the pavement with 150-200 penetration asphalt cement showed no transverse pavement cracking whereas the pavement constructed with 85-100 penetration asphalt cement showed four hundred (400) Type 1 transverse cracks per lane mile. From laboratory tests conducted on the asphalt cement it was determined that the modulus of stiffness of the 85-100 penetration pavement was approximately ten times the modulus of stiffness of 150-200 penetration pavement. It was concluded from this experiment that for this particular condition of climate, the 85-100 penetration asphalt cement was too hard to be employed in the pavement mixes.

On the basis of the performance of the Ontario test roads, the St. Anne test road, laboratory studies on pavement samples, and many other observations of asphalt pavements in Canada, the United States and Norway, McLeod has concluded that low-temperature transverse pavement cracking can be expected if the modulus of stiffness of a pavement attains a value of 1,000,000 psi (70,300 k/sq.cm.) at a pavement depth of 2" due to any critical

combination of chilling of the pavement at low temperatures, hardness of asphalt cement, and other controlling factors. It must be noted however, that this modulus of stiffness value is based on Van der Poel's charts for a loading time of 20,000 seconds and applies to a pavement mixture with a Cv* value of 0.88, 14.5 per cent VMA (voids in mineral aggregate) and 3 per cent air voids. From his analysis, McLeod has drawn the following three conclusions respecting transverse pavement cracking at low temperatures.

When all other factors are equal:

- (a) the higher the penetration at 77°F of the asphalt cement, the lesser is the low-temperature cracking that develops;
- (b) the higher the pen-vis number of the asphalt cement, the lesser is the transverse cracking that occurs; and
- (c) the harder the asphalt cement in a pavement the greater is the degree of transverse cracking that develops.

Detailed discussions of the above may be found in the articles by McLeod (5, 19, 32).

Of particular interest to the central theme of this thesis is the results of the test program conducted on transverse pavement cracking by the Department of Highways of Ontario, the results of which are reported by Fromm and Phang. (15) During 1966 and 1967 a series of samples were extracted from the pavements of 33 contracts in the province of Ontario. Samples were extracted from

$$* \text{ Cv} = \frac{\text{Vol. of Aggregate}}{\text{Vol. of Bitumen + Aggregate}} = \frac{100 - \text{VMA}}{100 - \text{air voids}}$$

pavements with varying degrees of transverse cracking severity. The recovered asphalt cements from these samples were rigorously tested for consistency and other quality control characteristics. To correlate these data with transverse pavement cracks, a cracking index was established by Fromm and Phang. This cracking index is defined as the number of multiple and full transverse cracks together with one-half of the number of half transverse cracks occurring in a 500 ft. stretch of two-lane pavement. Realizing that tensile stresses developed in the asphalt concrete resulted in transverse cracking, Fromm and Phang assessed the flow properties of the asphalt concrete by equating the temperature at which the viscous flow in one hour equalled the temperature shrinkage of the asphalt pavement in one hour. This temperature they called the critical temperature T_c . Through experimentation they have concluded that at higher temperatures the viscous flow exceeds the shrinkage rate, but at lower temperatures the shrinkage rate exceeds the viscous flow and tensile stresses build up within the asphalt concrete if the temperature continues to drop. The tensile stresses will exceed the strength of the asphalt if this drop in temperature continues and a transverse crack will result.

An attempt was made to establish a transverse cracking mathematical model through a step-wise regression analysis using forty variables which evolved from the sampling of the asphalt pavements. The model attempted to describe transverse cracking over the entire province in terms of the cracking index. From the results of this modeling technique the following conclusions were drawn. Cracking is a temperature dependent phenomenon for the

most part with cracking being more severe in areas of high freezing index. Transverse cracking can be reduced by using softer asphalts or asphalts of lower temperature susceptibility, i.e. higher pen-vis numbers. Asphalts with good flow properties at low temperatures display fewer transverse cracks. The stiffness modulus of the asphalt concrete at low winter temperatures is the major factor governing transverse cracking with the gradation of the base and subbase materials having a minor effect.

The report by Fromm and Phang (28) contains a detailed outline of the criteria of the current Department of Highways of Ontario asphalt cement specifications. Through such test programs it was determined that the temperature susceptibility of the various asphalt cements being used in the province was a major factor in the performance of pavements with respect to low temperatures. A study of viscosity at 275°F of all the asphalt cements was undertaken with the result that the temperature susceptibility of the various types and grades of asphalt cements was determined. As a result of this investigation specifications for viscosity were included in the Ontario Department of Highways specifications for the commonly used 85-100 penetration asphalt cement, the minimum kinematic viscosity being set at 280 centistokes. For the 150-200 penetration asphalt cement the minimum viscosity was set at 200 centistokes. In 1968 when the 300-400 penetration cement was introduced the minimum viscosity was established at 140 centistokes at 275°F but was reduced in 1970 to 120 centistokes due to problems of suppliers not being able to meet the previous 140 specification.

Stiffness modulus criteria are also being investigated to

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predict at what temperature pavement cracking will occur. With regard to the stiffness tests, Fromm and Phang (28) have suggested the following recommendations. By using a stiffness modulus value of 20,000 psi for asphalt cement as a safe crack-free figure as determined by Young et al (14) and a loading time of 10,000 seconds and with the knowledge of the corrected penetration indices of the available asphalt cements, McLeod's modified charts (30) for stiffness modulus and P.I. can be used in reverse to yield a penetration value for the required asphalt cement. The penetration value is that of the aged asphalt cement, i.e. after it has passed through the hot mixed plant, and the knowledge of thin-film oven test characteristics of the asphalt cement will make it possible to select the correct grade for the environmental conditions. In conclusion Fromm and Phang (28) indicate that the Department's present two-point consistency specification appears to be limiting the asphalt cements to those that are giving close to the desired performance under the temperature prevailing throughout the province of Ontario. Also the pen-vis numbers of the asphalts being supplied to Ontario range from 0.0 to approximately -1.5 at the present time.

1.2.6 Objectives and Methodology applied in the present investigation

Based on knowledge gained from academic studies, and the author's field experience, an investigation of asphalt pavement performance at low temperatures has been selected as the subject of this thesis. The author has investigated the current performance of asphaltic concrete pavements within the rural district of the Regional Municipality of Ottawa-Carleton (R.M.O.C.) with respect to low-temperature associated transverse pavement cracking and the penetration grade of asphalt cement contained in the asphaltic concrete.

The specific objectives of the study are:

- 1) To monitor the surface movement of the vertical faces of existing transverse pavement cracks in pavements of various ages over the range of temperature occurring during the 1976-77 winter months.
- 2) To evaluate the movement of the transverse pavement cracks for both totally reconstructed roadways and roadways which received only an asphaltic concrete overlay.
- 3) To determine the extent of transverse crack lengthening and the recovery ability of the asphaltic concrete with respect to closing of transverse cracks after the winter months.
- 4) To evaluate the extent of transverse pavement cracking on rural roadways within the RMOC and investigate any relationships between number and type of crack and
 - a) traffic volumes
 - b) age of asphaltic concrete :
 - c) type of asphalt cement used in the mix
 - d) type of roadway reconstruction
- 5) To determine the penetration value of recovered asphalt cement from roadways of various ages and relate this to the original penetration grade of asphalt cement used in the mix.

Information and data pertaining to the above was gathered from personal field investigations, laboratory analyses, and R.M.O.C. construction records.

Results and conclusions are compared to current knowledge and reports on this topic.

Reference to results of previous work reported in this field in Ontario and, in particular by N.W. McLeod and the Ministry of Transportation and Communication of Ontario are made.

All of the asphaltic concrete surfaces reported herein have been laid under the jurisdiction of R.M.O.C. and basically in accordance with M.T.C. specifications.

CHAPTER 2
FIELD INVESTIGATION PROGRAM

2.1 PAVEMENT

2.1.1 Steel-Pin Monitoring Stations

With the predominance of transverse cracks in the asphalt pavements that were surveyed and reported in this report, it was obvious that movement of the asphaltic concrete in the longitudinal direction (direction of traffic movement) was occurring. The following procedure was adopted to monitor the movement of the asphaltic concrete during the winter months.

Asphaltic concrete surfaces were selected according to the field age of the wearing surface from one year to five years old. Ten surfaces were chosen for monitoring of which six were asphalt overlays and four were the product of total reconstruction of the particular roadway. (Fig. 2.1 to 2.7)

On each of nine of the ten roadway surfaces, an existing transverse crack was selected, basically at random, as the site for a monitoring station. One reference station was established where no transverse crack was in evidence. (Fig. 2.6)

Six of the nine transverse cracks were straight line cracks extending continuously across both lanes of the roadway surface and at ninety degrees to the direction of traffic movement (Fig. 2.2). Three of the nine transverse cracks were discontinuous, in that they extended only partially across the total width of the roadway surface

but again at ninety degrees to the direction of traffic movement. (Reference Fig. 2.3, 2.4, 2.5)

2.1.1.a Function of Installation

In any study of transverse pavement cracking, it is logical to assume that movement of the pavement structure has occurred if a transverse crack is in evidence. Theories have been presented indicating that transverse pavement cracking is caused by expansion and contraction of subgrade materials thereby reflecting a crack in the stiffer (less flexible) asphaltic concrete overlying the subgrade materials. (Krukar & Cook; 8) Still other theories postulate (McLeod; 5, 19, 32), (Hills & Brien; 38) that transverse pavement cracks are a result of failure of the asphaltic concrete to remain sufficiently flexible at below freezing ambient temperatures to contract without exceeding the tensile bonding strength of the asphalt cement in the mix. On the basis of case histories and results of field studies reviewed by the author, the latter theory is favoured as the primary temperature associated cause of transverse pavement cracking. It is felt that, in some instances, specific types of subgrades may contribute to a lesser degree to transverse pavement cracking. (Young et al; 14)

The objective of the steel pin installations was to establish a magnitude for the longitudinal movement of the surface of the asphaltic concrete. It was felt that this could best be accomplished by selecting a monitoring location where movement had obviously already taken place,

i.e. at an existing transverse crack. Due to the random pattern of transverse cracking, it would be virtually impossible to predict a precise location at which a transverse crack would occur. By measuring the distances between opposite pairs of steel pins across an existing crack over the range of winter ambient temperatures, some conception of the magnitude of movement could hopefully be obtained. Substantial changes in crack width would affect the riding quality and eventually the structural capacity of the roadway itself.

A visual analysis of the width of existing transverse pavement cracks indicated to the author that such measurements could be taken with a millimeter scale as opposed to attempting field measurements with micro-measuring equipment.

Results presented by some researchers indicated that transverse cracking initiated at an ambient temperature of between 0°F and -4°F. (Anderson et al; 39) Also there is evidence that cracking is initiated by a sudden warming in temperature after a prolonged period of below 0°F minimum temperatures. (Domaschuk et al; 37) Assuming that the above are valid assessments, movement across the existing transverse cracks could be anticipated, during a typical Ottawa winter.

Field measurements were taken on the steel pin installations in an effort to verify the above two statements regarding pavement movement.

By installation of monitoring stations in asphalt pavement surfaces constructed between 1970 and 1975 with either

85/100 or 150/200 penetration grade asphalt cement, it was hoped to obtain sufficient data for analysis to determine if a correlation existed between magnitude of movements across the steel pin installation, and

- (1) age of asphaltic concrete,
- (2) type of asphalt cement in the mix.

2.1.1.b Method of Installation

At each of the nine stations, movement of the pavement surfaces on each side of the transverse crack was to be monitored. To accomplish this, two parallel stringlines were established across the pavement surface, one on each side of the transverse crack. (Appendix B, photo #1) Galvanized spiral steel survey pins*, 1/4" dia. by 2-1/2" in length were driven flush with the pavement surface along the stringlines in diametrically opposite pairs at four locations as shown in figure 2.1. (Appendix B, photo #2) The pairs of pins nearest the edge of pavement were established beyond the outer wheel path whereas the pairs of pins adjacent to the centreline of the two-lane roadway were established approximately 12 inches from the centreline of the roadway, but in all cases avoiding the longitudinal pavement joint. (Appendix B, photo #4)

* - Parker-Kalon Masonry nails.

All pins were located a minimum of five(5) inches from the edge of the transverse crack. By doing so, the presence of the 1/4" diameter pin would have a negligible effect on the movement of the edge of pavement of the crack.
(Selvadurai, A.P.; 44)

Because the centreline of the roadway was a construction joint, and its location could not be accurately determined in all cases, the transverse spacing of the pairs of pins adjacent to the centreline was established at approximately 15 inches. It was initially assumed that movement would take place across the construction joint.

A set of pins was established, in a similar manner to the above, at the control station where no transverse crack existed. (Reference fig. 2.6)

2.1.1.c Data Collection

Initial measurements were taken in millimeters between diametrically opposite pairs of pins using a measuring tape graduated in one-half millimeters. Future measurements on these pins would establish any longitudinal movement across the transverse crack.

Initial measurements were also taken between the two pairs of pins ~~at~~ the centreline of the roadway paralleling the transverse crack. Future measurements would establish any transverse movement across the centreline of the roadway where no pavement cracking and no separation of the centreline joint was initially evident.

The date of installation of each monitoring station was recorded and the temperature of the pavement surface was taken immediately following the measurements across the pins. Temperature readings were taken using a standard Fahrenheit thermometer. This procedure was followed for each subsequent monitoring of these installations.

Although monitoring of the steel pin installations was weather dependent, it was intended to take measurements as previously described over as wide a temperature range as was possible throughout the winter and spring months. Also as suggested by Anderson et al (39) measurements were taken when temperatures were near or at -4°F and when a prolonged period of cold (below 0°F) was followed by a sudden warming.

During the month of January, 1977, blowing snow and ice conditions made monitoring of the pin installations extremely difficult.

2.1.2 Transverse Crack Lengthening

In addition to monitoring movements longitudinally across the steel pins, the length of existing pavement cracks at three locations was monitored to ascertain any change in length of these cracks which initially extended only partially across the full width of the roadway surface. These measurements were of secondary importance to the purpose of the steel pin installation, but it was felt that by taking such measurements, some indication of the point in time at which changes in crack length may occur could be determined. Also the magnitude of crack lengthening over one winter season, if appreciable, could be used as a basis for further study.

2.1.3 Vertical Pavement Displacement

Studies of frost action and "spring thaw" conditions in roadways have verified that roadway surfaces tend to rise during the spring thaw period and subside following this period each year in climates similar to that of Ottawa.

To establish to what extent this is applicable to the roads in this study, elevations were taken once monthly after the initial steel pin installation on each of the steel pins. Measurements were taken using a level and standard rod with direct reading accurate to one-hundredth and interpolation to one-thousandth of one foot.

Vertical displacements would be used to provide a magnitude for the frost susceptibility of the underlying pavement structure. It is suggested that if large vertical displacements occur at any particular location, any changes in asphalt pavement cracks at that location must be analyzed with a view to the vertical movement being a major contributing factor (as opposed to characteristics of the asphalt concrete only).

2.2 TRANSVERSE CRACK SURVEY

2.2.1 Precise Survey

2.2.1.a Function of Survey

The objective of this precise transverse crack survey was to establish a correlation between the number of

transverse cracks per unit length of pavement and the variables that might be contributing to the cracking of the pavement. These variables were the following:

- 1) Age of the pavement surface.
- 2) Type of pavement improvement:
 - a) asphalt overlay
 - b) total reconstruction
- 3) Traffic volumes.
- 4) Type of asphalt cement used in the mix.
- 5) Penetration values of the recovered asphalt cement.
- 6) Type of subgrade for completely reconstructed roadways.

2.2.1.b Method of Data Collection

A precise transverse pavement crack survey was conducted by the author on one-half mile sections of roadways, each section of which contained one of the previously mentioned steel-pin monitoring stations. Within each half-mile section, the exact location, length and description of all transverse and longitudinal cracks in the asphalt pavement surface were recorded. To accomplish this, a digital measuring wheel was rolled along the pavement surface parallel to the edge of pavement. For each crack encountered in the pavement surface, a footage reading was recorded from the measuring wheel, the length of the crack was measured, its location established within the pavement width, and a description of the crack noted. A consistent description of the transverse cracks was accomplished by categorizing the cracks into three types:

- A Type I transverse crack was one which extended totally across both lanes of the two lane pavement surface.
- A Type II transverse crack was one which extended from the edge of pavement to at least the centreline of the roadway but not completely across both lanes of the roadway.
- A Type III transverse crack was one which began and ended within the pavement surface and was thereby distinct from both type I and II cracks.

The lengths and locations of all longitudinal cracks in the asphalt surfaces were measured and recorded.

The width of the pavement, the age of the surface course asphaltic concrete and the penetration grade of asphalt cement used in the mix were noted on the transverse pavement crack inventory sheets.

Transverse cracking in the shoulder of the roadways adjacent to types I and II cracks was not found in this survey.

The information gathered from the precise transverse crack survey was plotted to a scale of 1" to 100 feet longitudinally for each of the one-half mile sections of roadway and is shown in figures 2.8 to 2.16. The corresponding steel pin installation station contained within each half-mile section is noted on each figure for ease of comparison later in the report.

It will be noted that no precise survey figure containing steel pin station G is shown. No cracking in the one-half mile section of roadway was found at this location.

2.2.2 Windshield Survey

A windshield survey is a survey in which a vehicle is driven along a roadway at slow speed and details recorded from observation by the passengers. It is intended as a rapid means of assembling easily attainable information.

2.2.2.a Function of Survey

Additional transverse crack surveys were conducted to supplement the precise surveys in enabling comparisons and conclusions to be made with other variables from the field data. Because the majority of the additional surveys were conducted on the same roadways on which the precise surveys were made, a higher degree of reliability of comparisons between variables in the data analysis could be anticipated. Furthermore, it was recognized by the author when conducting the precise surveys that misleading conclusions could be drawn from the short sections of roadways under survey particularly with respect to any comparisons being made between transverse cracking, age of asphaltic concrete and traffic volumes. This will be expanded later in the report under analysis of field data.

2.2.2.b Method of Data Collection

The precise transverse crack survey provided an accurate record of the number and location of all cracks within one-half mile sections of the roadways surveyed.

Additional crack surveys were conducted on approximately 33 miles of Regional roads where either overlay work or new construction of the asphalt wearing surface was done over the five year period, 1970 to 1975, using either 85/100 or 150/200 pen grade asphalt cement in the mix.

Each of the three previously described types of transverse cracks were recorded per half mile sections of roadway. This information was gathered by a windshield survey in which a vehicle was driven (stop and go method) along the shoulder of the roadway with the driver noting the type of transverse crack and advising an assistant who recorded this information as well as the mileage location for each section of roadway. No information was recorded regarding cracking in the shoulders of the roadways. Where other pavement defects (i.e. alligatoring, frost heaves) occurred at a transverse crack, no transverse crack was recorded because it was felt that the nature of the crack could not be attributed solely to factors under consideration in this report.

2.3 ASPHALT CONCRETE CORE SAMPLING

2.3.1 Function of Core Sampling

Field testing and studies have shown that the rate of transverse pavement cracking at low temperatures increases with increases in the hardness of the asphalt cement in the mix. (Culley;24) (McLeod; 3, 19, 20) It has been suggested by some authorities that the hardest grade of asphalt cement to be used in an environment such as the Ottawa area is 150/200 pen. (McLeod; 3, 32) It has been shown that the stiffness of an asphalt cement increases with age under field conditions. (Culley; 17) As the asphalt cement age-hardens, the asphalt pavement becomes more susceptible to low temperature transverse cracking.

This author favours the method of specification proposed by McLeod (3, 5, 30) and others in that the grading of asphalt cement is best accomplished by using the penetration test. Commonly used grades of asphalt cement are as follows: (60-70, 85-100, 150-200, 300-400 pen grades).

To investigate the penetration characteristics of the asphalt cement used in the pavement mixes for roadways in this report, a program of recovery and testing of asphalt cement from core samples was undertaken. Through extraction of the asphalt cement from the core samples, taken from roadway surface constructed between 1970 and 1975, and subsequent testing for penetration values, an indication of age-hardening of the asphalt cement could be obtained.

Although the actual original penetration value for each asphalt cement was not known, it was felt that the range of penetration in the known penetration grade, in this case 85/100 and 150/200 pen, for each pavement could be used as a basis for comparison with the penetration values from the recovered samples.

An analysis could also be made between the age-hardening characteristics of both 85/100 and 150/200 pen grade asphalt cements and the frequency of either one or all types of transverse pavement cracks.

2.3.2 Method of Data Collection

Core samples were extracted using a rotary drill from the full depth of asphaltic concrete on roadways where total reconstruction had been carried out.

These were the one-half mile sections of roadways referred to in section 2.2.1.b of this report and containing steel pin stations C, E, F and G.

On roadways which had received an asphaltic concrete overlay, core samples were taken similar to the above. It was necessary however to separate the overlay asphalt from the remainder of the core sample in these cases because only the age of the asphalt overlay was known and was to be analyzed.

The overlaid sections of roadways referred to here were those in section 2.2.1.b of this report containing steel pin stations A, B, D, H, J, and K.

Samples were taken in duplicate from each of the above-mentioned ten roadway surfaces and carefully transported to the laboratory for further testing.

2.3.3 Laboratory Analyses

2.3.3.a Sample Preparation

All asphaltic concrete samples were cleaned of any foreign matter prior to testing. The overlay asphalt was separated using hand tools from the total field core for the six locations previously mentioned.

2.3.3.b Asphalt Cement Extraction

Asphalt cement was extracted from each of the twenty samples using the Abson Recovery Method, A.S.T.M.

D-1856-75 and collected in separate containers for further analysis.

A detailed outline of the above procedure may be found in the A.S.T.M. Standards for testing.

2.3.3.c Penetration Tests

Each sample of the recovered asphalt cement was tested for penetration at 77°F in accordance with A.S.T.M. D5-73. The results of each test were recorded and correlated to the section of roadway from which the asphalt concrete sample had been extracted.

2.4 TRAFFIC VOLUME DATA

For all of the roadways on which transverse pavement crack surveys were taken, automatic traffic counts were obtained from the Department of Traffic Engineering Services of the R.M.O.C. The traffic counts were 24 hour total volume counts taken within eighteen months of the other field data assembled in this report. No information was available regarding either mix of traffic or wheel loads for those roadways surveyed.

2.5 SUPPLEMENTARY FIELD DATA

2.5.1 Roadway Cross-sections

A complete cross-section of the roadway was taken at each of the steel-pin monitoring stations. These were plotted to scale to aid the reader in visualizing the similarity

of sections of roadways being compared in this report. (Reference fig. 2.17 to 2.26 inclusive). For each of the cross-sections (fig. 2.17, 2.18, 2.20, 2.24, 2.25, 2.26) where an asphalt overlay was employed, the thickness and type of asphalt concrete used has been shown. For each of the cross-sections (fig. 2.19, 2.21, 2.22, 2.23) where total reconstruction of the roadway was carried out, the entire pavement structure type and thickness has been shown.

This information was obtained from "as-built" construction drawings of the R.M.O.C.

2.5.2 Subgrade Data

The type of subgrade for each of the four totally reconstructed roadways was obtained from soils reports of R.M.O.C. and is shown on the corresponding cross-sections. It is to be noted that the subgrade type as shown is applicable only to the immediate vicinity of the cross-section.

No additional subgrade information is presented in this report as it was not this author's intent to establish any definite correlations between pavement performance and the type and characteristics of subgrade and subbase materials.

2.6 ENVIRONMENTAL DATA

An evaluation of performance of pavements of different asphalt cement grades is temperature dependent. This was

referenced earlier in this report to the monitoring of the steel pin installations and behaviour of transverse cracks in chapter 2.1. To supplement this information, a complete meteorological summary of environmental conditions was obtained from Environment Canada for the months of October, 1976 to May, 1977.

This data is shown in Appendix A.

CHAPTER 3

TEST RESULTS AND DISCUSSIONS

3.1 PAVEMENT MOVEMENT

3.1.1 Longitudinal Movement

3.1.1.a Movement vs Temperature

Maximum displacement of the steel pins across a crack was measured at the end of January (26th) immediately following the coldest days of the winter. Section 3.1.1.d comments additionally on Station F.

It can be seen from the figures 3.1 to 3.10 that the increase in displacement follows a pattern (curve) quite similar to the plot of minimum daily temperature for the period Nov. 1/76 to May 1/77.

Maximum displacements of the steel pins were recorded during the month of greatest number of degree-days, i.e. January (427) (Fig. 2.0). The average minimum temperature for the month of January was -17.4°C (0.7°F). Since maximum displacements were measured at or below this average temperature, it may be suggested that initiation of transverse cracks may prevail at the temperatures which result in maximum displacements. This would substantiate the theory that transverse pavement cracking initiates at temperatures approximately 0°F to -4°F (Anderson et al; 39).

During the first 14 days of December (Dec. 1 to 14), the average minimum temp. was $(-273.3/14) = -19.5^{\circ}\text{C}$.

On Dec. 15 the daily minimum temperature rose to -2.9°C , on Dec. 16 was -6.4°C with the maximum daily temperature rising to slightly above freezing for both days (i.e. 2.5°C).

Eight of the ten steel pin stations were monitored on Dec. 4, 8, and 16 and the results are plotted and shown on the appropriate figures 3.1 through 3.10. Of the eight stations monitored, stations A, B, E, H and J produced consistent reaction to temperature changes. In these six instances, a definite increase in displacement between opposite pairs of steel pins from the previous measurements was found on Dec. 4 and an additional increase on Dec. 8. On Dec. 16 after approximately 36 hours of substantially increased temperatures, as previously outlined, displacement between opposite pairs of steel pins was measured and found to have decreased. These measurements of displacements provide an indication of the time response of the asphalt with respect to temperature changes in this viscoelastic material. The time-temperature relationship is a function of the stiffness of the mix which in turn is dependent on the grade of asphalt cement in the mix. (McLeod, 5; Haas and Anderson, 23)

It can be concluded that displacement across transverse cracks is highly temperature dependent, increasing as the temperature falls and decreasing as the temperature rises, that this reaction occurs over relatively short periods of time. This substantiates in part the theory of crack propagation by a sudden warming after a prolonged period of below 0°F (-18°C) minimum temperatures. (Domaschuk et al; 37)

In addition to this, a definite correlation between minimum temperature fluctuations and displacements between opposite pairs of steel pins exists as shown in the figures 3.1 to 3.10. Since the steel pins were rigidly fixed in only the surface course asphaltic concrete layer for each of the six locations where an asphalt overlay was encountered, it can be assumed that displacement between the steel pins is an acceptable approximation of the maximum displacement of the edges of the existing transverse crack between the pins. Theoretically the precise increase (or decrease) in displacement across a crack can only be made by monitoring the movement at the extreme faces of the crack and would be accurate for the surface movement only. (Appendix C)

Previous experiments by others (Burgess et al; 16) (Anderson et al; 39) have suggested that initial cracks start microscopically at the surface and progress downward to limited depth. These observations are based on the assumption that the crack is a result of contraction within the asphaltic concrete.

It is the author's opinion that the two or more layers of asphaltic concrete are sufficiently bonded together that they do not move relative to each other and therefore the measurements across existing transverse cracks are consistent with those taken on the overlay pavements.

At the four locations where total roadway reconstruction had taken place, the asphalt cement was the same grade for all layers of asphalt. The steel pins at these locations penetrated the surface course layer of asphalt completely and were embedded in the underlying asphalt layer. This

can be verified from the asphalt thicknesses shown in figures 2.18, 2.20, 2.21, 2.22.

A similar trend between low-temperature fluctuations and maximum displacements between opposite pairs of steel pins exists in two of the three locations monitored where a transverse crack existed. (Fig. 3.3 and 3.5)

The results shown on Fig. 3.6 for station F are somewhat inconsistent in comparison, although the temperature-displacement correlation is evident.

3.1.1.b Maximum Displacement vs Age of Asphaltic Concrete

Table 3.1 is a display of the maximum displacement recorded between opposite pairs of steel pins as compared to the age of the asphaltic concrete at each station location.

The table is in two parts, the above showing field data for roadways which were completely reconstructed and the lower showing field data for roadways which had received an asphalt overlay. The information in the table is tabulated in chronological order in each part beginning with the newest (youngest) asphalt surfaces. The age of these surfaces was taken from the month and year in which the surface was constructed to November 1, 1976 (i.e. the beginning of data collection for this survey):

The results indicate a random pattern of movement across existing transverse cracks with respect to the age of the asphalt surfaces.

It can be concluded that maximum widening of existing transverse pavement cracks at the surface will occur during extreme low temperatures (approximately -20°C) and that the extent of the widening appears to be independent of the age of the asphaltic concrete.

Based on the measurements taken at the control station "G", where no transverse pavement crack was evident, differential movement between opposite pairs of pins was a minimum in comparison to locations where movement across existing cracks was monitored. At station "D", where no existing transverse crack was initially in evidence between pin pairs "ab", "cd", "ef", and "gh" (figure 2.4), movement between opposite pairs of pins was again a minimum. The age of the asphalt at stations "G" and "D" was 15 months and 74 months respectively.

Appreciating that the measurements in all cases represent differential movements only, it can nevertheless be postulated that if differential movement within the surface asphalt can be minimized to an order of 10 mm maximum, transverse pavement cracking may not occur. Conversely, if the thermal and elastic properties of the asphaltic concrete mix are such that the mix, when laid as a road surface, can accommodate contractions of up to 10 mm in the longitudinal direction, transverse pavement cracking may be minimized.

It can also be concluded that after the asphalt pavement has initially developed a transverse pavement crack, substantial longitudinal movement of the asphalt surface begins to occur and, together with contamination of the

crack opening with dirt particles and precipitation and multiple freeze-thaw cycles within the pavement surface, rapid deterioration at the faces of the crack will occur.

3.1.1.c Maximum Displacement vs Grade of Asphalt Cement in Mix

Table 3.1 also outlines the two grades of asphalt cement used in the asphalt pavements surveyed at each of the ten stations.

From the data available, no correlation could be established between the grade of asphalt cement used in the mix and the maximum displacement recorded across existing transverse cracks.

Effects of the grade of asphalt cement with respect to transverse pavement cracking is discussed again later in this report.

3.1.1.d A Comment on Results at Station "F"

The steel pin layout and existing transverse crack pattern at Station "F" can be seen from figure 2.5. A record of displacement between opposite pairs of pins is shown on figure 3.6 with the maximum displacement values repeated in Table 3.1.

Of all monitoring stations, Station "F" recorded the maximum differential movement of the steel pins. This station also produced the maximum crack lengthening and vertical pavement displacement, both to be discussed later in this chapter. From figure 3.6, it can be seen that the maximum movement across the existing crack (i.e. between

pins "cd" and "ef") was first recorded on February 14/77 following a rapid rise in temperature. Maximum movement across pins "gh" was recorded on March 3/77 prior to which the transverse pavement crack had extended between "gh" to the pavement edge as well as extending to the pavement edge between "ab."

It is believed that movements recorded at Station "F" are, in large part, due to subgrade conditions. This statement is made on the basis of subgrade soil investigations carried out in 1974 prior to and during reconstruction of this roadway by the R.M.O.C. testing laboratory. At the time it was concluded that a plastic flow condition in the subgrade on this roadway was responsible for heaving and cracking in the granular "B" materials during construction. After thorough investigation, it was recommended that quarry crushed granular "B" material be used in place of the normal granular "B" to minimize the movement. It was also suggested that some surface cracking of the asphalt could be anticipated after completion of the work due to continued but limited movement within the subgrade.

Based on the above information, this author has elected not to use the results of pavement movements and crack lengthening at Station "F" in comparison with other station results in this report.

3.1.1.e Permanent Crack Widening

Minimum temperatures below 0°C did not occur between April 29/77 and the last date of monitoring of the steel pin installations on May 29/77. Each steel pin station was

monitored twice during the month of May (May 3 and May 29) after freezing temperatures had ceased.

Table 3.2 has been prepared to show the permanent increase in transverse pavement crack widths from the initial measurements taken in November 1976. This information is shown graphically in figures 3.1 through 3.10.

The numbers shown for "permanent crack width increase" in table 3.2 are the averages of the two readings taken in the month of May for each pair of steel pins.

From table 3.2 the following observations have been made:

- (a) Permanent crack width increase is greater for totally reconstructed pavements than for overlay pavements of similar ages (Compare results at Stations C and E with those of Station B and H).
- (b) Permanent crack width increase is greater for asphaltic concrete of similar age containing 85/100 penetration asphalt cement than those containing 150/200 penetration asphalt cement (Note: the different type of construction may be a contributing factor).
- (c) For both totally reconstructed roadways and those which received an asphalt overlay, permanent crack width increase is greater across pairs of pins located adjacent to the roadway centreline than those located in the outer wheel path of the pavement.
- (d) For asphalt overlay construction, permanent crack

width increase is not related to the age of the asphaltic concrete overlay.

3.1.2 Transverse Crack Length

Monitoring the development of the lengths of existing transverse pavement cracks was of secondary importance in this study, however the results did provide for some interesting observations.

At stations "C", "D", and "F" (reference fig. 2.3, 2.4 and 2.5 respectively) the lengths of existing transverse cracks were measured and referenced to the locations of the steel pins and the edge of pavement at the time of installation of the steel pins. The accuracy in measuring the lengths of these cracks was $\pm 1/2$ inch of the length of the crack visible to the naked eye and was deemed adequate for this survey.

From the accompanying Tables 3.3a, 3.3b, 3.3c, crack lengthening at the three stations can be seen. In all three cases, the maximum amount of crack lengthening was observed in the measurements taken on January 26/77, immediately following the coldest days of the winter. Continuation of crack lengthening after this time was minimal in comparison to that which took place between the time of the initial measurements and January 26/77.

The cracks which developed at stations "C" and "D" were hairline but quite visible whereas at station "F", although initially probably a hairline crack, when observed on January 26/77 were very discernible and obviously not

solely a result of thermal contractions within the asphalt surface.

At station "D", the presence of a wide longitudinal crack paralleling the centreline and approximately 12" right of centreline at the steel pin installation may have had an influence on the transverse crack pattern that developed, however this could not be verified. The transverse crack lengthened to the left until it intercepted the longitudinal crack. Movement across the longitudinal crack was measured and found to have increased 19 mm from the initial measurements between "ce" and "df" (reference fig. 2.4)

From the limited field data it can be suggested that existing transverse pavement cracks terminating within the pavement edge (type II and III cracks) will propagate to the pavement edges with time and develop into type I cracks as defined in this report. The propagation of the cracks is definitely temperature dependent with maximum lengthening to be expected during the coldest part of the winter season when temperatures fall to the -20°C range.

It is evident that a transverse pavement crack, if a type II or III at first, will propagate in a relatively straight line across the pavement surface.

3.1.3 Vertical Pavement Displacement

Crack propagation has three modes.

1. Longitudinal displacement, as previously discussed.
2. In-plane shearing - unlikely because of the restraint afforded by the roadway construction.

3. Out-of-plane shearing as discussed below.

Table 3.4 displays the elevation changes of each steel pin for all stations as recorded on four separate occasions from Dec. 8/76 to March 10/77. The bench mark elevations (B.M. El.) and steel pin elevations are referenced to those pavement elevations shown on figures 2.17 through 2.26.

Maximum displacement for each steel pin is shown in both feet (ft.) and millimetres(mm). It must be noted that the maximum displacement shown is applicable only to the duration over which readings were taken. It is quite conceivable that some vertical displacement could have occurred prior to Dec. 8/76 because frost conditions were present during the months of October and November. It is also conceivable that the maximum vertical displacement of the steel pins did not occur on the dates that measurements were taken regardless of the fact that subsequent readings on the pins showed a drop in elevation.

The author's primary purpose for taking pavement elevations at the steel pin stations was to establish at what approximate time of the year the maximum vertical displacement of the pavement surface occurred in comparison to the time at which the maximum horizontal movement was recorded across the steel pins. From the results shown in table 3.4 it can be seen that the maximum vertical displacement was recorded in the February, 1977 measurements taken at seven of the ten stations. At the remaining three stations, D, E and F, increase in vertical

movement was recorded in the readings taken in March /77.

From these results it can be estimated that vertical displacement of the pavement surface will occur and that it will be a maximum sometime after the most severe low temperature conditions have passed.

Table 3.5 has been produced to show the average maximum vertical displacement recorded at each station for both the overlay pavements and the newly constructed roadways. Considerably greater movements were recorded on those sections of roadway where total reconstruction had taken place when compared with the movements recorded on the overlay pavements. No obvious correlation exists between average maximum vertical displacement of the pavement surface and the age of the surface.

Vertical displacements were greater at stations "G" and "F" than at stations "C" and "E". The subgrade at stations "G" and "F" was basically clay whereas the subgrade at stations "C" and "E" was of the sandy silt type.

The presence of a substantially wide centreline crack at station "D" indicated to the author that movements had previously occurred prior to this survey. (See Appendix B, photo #6) The magnitude of vertical displacements measured indicates movement within the subgrade of this roadway. It is suggested that the freedom of movement allotted by the large longitudinal crack curtailed any substantial changes in the transverse crack and the distances between the steel pins.

The magnitude of the vertical displacement of the pavement

surface at station "F" undoubtedly contributed to both the considerable and rapid crack lengthening at this station and the large longitudinal increases between the pairs of steel pins. (See Appendix B, photo #5)

3.1.4 Summary of Vertical Displacements

1. Vertical displacements of pavement surfaces does occur during freezing temperatures. Displacements will reach a maximum sometime after the coldest part of the winter has passed and the roadway surface will subside.
2. At the four locations where measurements were taken, less vertical displacement was observed over subgrades of silty sand than those of clay materials.
3. Transverse crack lengthening attributed to relative vertical pavement displacements is minimal due to a minimum of out-of-plane shearing as shown in Table 3.4.

3.1.5 Transverse Movement Across Pavement Centreline

Because it was anticipated that movement of the pavement surface would occur across the centreline construction joint, field measurements were taken between pin pairs "ce" and "df" at each station, except "F" because of its unusual geometric layout, each time the steel pin station was monitored.

With the exception of station "D", the maximum displacement recorded across the centreline joint was one(1) millimetre as shown in table 3.6.

As stated previously, a large longitudinal crack was present along the centreline at station "D" at the time of

installation of the station. The results of measurements taken across this crack between steel pins "ce" and "df" are shown in table 3.7. It is noted that the maximum displacement was measured on March 3/77 whereas the maximum horizontal movement between steel pins across the transverse cracks was recorded on Jan. 26/77. These large displacements have led this author to conclude that frost action within the pavement structure is responsible for the majority of the movement recorded.

3.2 TRANVERSE PAVEMENT CRACKING AND AGE OF ASPHALTIC CONCRETE

3.2.1 Precise Survey Results

Transverse pavement cracks were categorized as to "type" as defined in Chapter 2, Section 2.2.1b. Table 3.8 shows the results of the precise survey. The number of each type of crack per one-half mile of roadway is listed with a summary of the age of the asphaltic concrete pavement and the grade of asphalt cement used in the pavement.

Table 3.9 presents a summation of the three types of cracks listed with the two variables of table 3.8. In both tables the type of roadway constructed is separated to be consistent in comparison of variables throughout this report.

The precise survey of one-half mile sections of roadways provided details of locations and extent of transverse pavement cracking in the pavement surfaces. The information generated from this survey was to be used for providing 1" to 100' scale sketches showing extent of cracking and was not intended for analysis to draw conclusions between cracking frequency and either

- 1) the age of the asphaltic concrete surface,
- 2) the grade of asphalt cement in the mix.

3.2.2 Windshield Survey Results

Due to limited data obtained from the precise survey of transverse cracks, it was anticipated that these findings would prove inconclusive.

The analysis of data obtained from the survey of an additional 33 miles of roadway surfaces where both overlay and new construction of the asphalt pavement had been conducted over the same five year period as that of the precise survey data could more reliably define any trends between the number and type of transverse cracks and other variables of this study.

Table 3.10 displays the field data obtained from the transverse crack count survey of approximately 33 miles of asphaltic concrete roadways. The data is arranged in ascending order of age of the asphaltic wearing surface for both totally reconstructed roadways and roadways that have been overlayed. The grade of asphalt cement used in the mix is also shown for future reference.

This data was analyzed to ascertain any definite relationships between the number and type of transverse cracks and the age of the asphaltic concrete. No comparisons were made between the data and the type of asphalt cement used in the mix because of the difference in construction methods (total reconstruction vs asphalt overlay).

3.2.2.a Totally Reconstructed Roadways

For those roadways that were totally reconstructed, a plot of transverse crack data vs age of the asphaltic concrete is shown in figure 3.11.

Four graphs are shown:

- (1) Age of the asphaltic wearing surface vs type I cracks per mile of two lane roadway.

- (2) Age of the asphaltic wearing surface vs the summation of type I and type II cracks per mile of two lane roadway.
- (3) Age of the asphaltic wearing surface vs the total of all types of cracks per mile of two lane roadway.
- (4) Age of the asphaltic wearing surface vs McLeod * type I cracks per mile of two lane roadway.

In very general terms it can be said that the number of each type of transverse crack per mile of two lane roadway increased with increasing age of the asphaltic concrete. One could not reliably predict the number of transverse cracks to occur in a new pavement based on the results of the field data. In particular the three (3) pavements of the same age (52 months) displayed markedly varied cracking patterns.

The general conclusion that transverse cracks per mile on totally reconstructed roadways increases with the age of the asphalt concrete (no other variables being considered) is consistent with conclusions drawn from studies conducted by others. (McLeod, 32) By definition the summation of twice the number of type I cracks plus the number of type II cracks per mile of two lane roadway is equal to McLeod's type I transverse crack, hence the reason for showing a graphical plot of $2(\text{type I}) + \text{type II}$ cracks vs age of the asphaltic concrete.

Table 3:11 displays a further refinement of the transverse crack count data in an attempt to correlate

- * - McLeod's type I crack per two lane roadway per mile is equivalent to $2(\text{Type I}) + \text{Type II}$ cracks per two lane roadway per mile of this author by definition.

- (1) age of asphaltic concrete vs ratio of total cracks per mile to type I;
- (2) age of asphaltic concrete vs ratio of total cracks per mile to type I and II;
- (3) age of asphaltic concrete per mile vs ratio of total cracks to McLeod's type I.

From the results of this analysis it may be concluded that the ratio of total cracks to type (I + II) cracks compares favourably with the ratio of total cracks to the type I cracks as defined by McLeod, the ratio of the former approximating 2.0 and that of the latter 1.6.

Using this criteria, for two lane totally reconstructed roadways in the Regional Municipality of Ottawa-Carleton, one could predict a roadway surface to contain twice the number of transverse cracks as would be counted by either McLeod's definition of type I cracks or the author's definition of type (I + II) transverse cracks.

The ratios shown in the table 3.11, i.e. A, B and C, when combined with the age of the asphaltic concrete provided no conclusive information which would assist in predicting the number of transverse cracks per mile of two lane roadway.

3.2.2.b Asphaltic Overlay Construction

The data from figure 3.10, for roadways where asphaltic overlay construction was surveyed, was analyzed using the same methods as that used for the totally reconstructed roadways.

Figure 3.12 displays four graphs where combinations of transverse crack types per mile of two lane roadway vs age of the asphaltic concrete wearing surfaces are plotted.

As was the case with the totally reconstructed roadways, in general terms the number of transverse cracks per mile of two lane roadway increased with the age of the asphaltic concrete overlay.

The spread of data is such that a reliable prediction of the number of transverse cracks per mile of two lane roadway cannot be defined. In particular the three(3) pavements of age 32 months displayed markedly varied cracking patterns as did the two(2) pavements of age 52 months.

Again a plot of the equivalent McLeod type I transverse crack vs age is shown for comparison.

Table 3.12 shows a further refinement of the transverse crack count data in an attempt to correlate the age of the asphaltic concrete with

- (1) the ratio of $\left(\frac{\text{total cracks per mile}}{\text{Type I cracks per mile}} \right)$
- (2) the ratio of $\left(\frac{\text{total cracks per mile}}{\text{Type (I + II) cracks per mile}} \right)$
- (3) the ratio of $\left(\frac{\text{total cracks per mile}}{\text{McLeod type I cracks per mile}} \right)$

From this analysis it is concluded that the ratio of total cracks to type (I + II) cracks compares closely with the ratio of total cracks to the type I cracks as defined by McLeod, the ratio of the former approximately 1.9 and that of the latter 1.4.

It is reasonable to predict the total number of transverse cracks per mile of two lane roadway by counting the types I and II cracks as defined by the author and applying the factor of 1.9. Alternatively McLeod's type I transverse cracks when multiplied by the factor 1.4 would produce similar results for total number of cracks in the wearing surface.

It is interesting to note that the ratio of total cracks to the sum of types I and II cracks per mile of 2 lane roadway is practically equal for the totally reconstructed roadways (2.0) and the asphalt overlay roadways (1.9).

The ratios in table 3.12, i.e. A, B and C, when combined with the age of the asphaltic concrete produced no additional information which could be used to predict cracking frequency in the asphalt overlay wearing surfaces.

3.3 ASPHALTIC CONCRETE CORE ANALYSIS

Asphaltic concrete samples were extracted from those roadway surfaces which contained a steel-pin monitoring station for the purposes outlined in Chapter 2, Section 2.3. Asphalt cement was recovered from each of the twenty samples using the Abson recovery method (ASTM D-1856-75).

A Standard Penetration test (ASTM D5-73) was carried out on each of the recovered samples of asphalt cement firstly by the testing laboratory of the R.M.O.C. and secondly by the Materials and Laboratory Services Section of the Ministry of Transportation & Communications (Ontario).

3.3.1 Penetration Test Results

Table 3.13 outlines the penetration values obtained on the recovered asphalt cement samples from both R.M.O.C. and M.T.C. testing laboratories. Also shown is the original penetration grade asphalt cement used in the mix according to R.M.O.C. construction specification records, and the age of each pavement surface at the time of sample extraction.

In general the penetration test result values from the M.T.C. laboratory are slightly higher than those of the R.M.O.C. laboratory, however both laboratories indicate no substantial difference in penetration values between the 85/100 and 150/200 original pen grade asphalt cements.

Because only that asphaltic concrete for which the original penetration grade asphalt cement was known was

subjected to laboratory analysis, the recovered asphalt cement penetration values can be compared regardless of the type of roadway construction (i.e. total reconstruction and overlay reconstruction).

The results of data analysis for the information contained in table 3.13 are detailed further in table 3.14.

The original penetration value of the asphalt cement used in each case is unknown but the penetration grade is known. For comparison purposes, it has been assumed that:

1. the original penetration value of the asphalt cement for the 85/100 grade is 90.
2. the original penetration value of the asphalt cement for the 150/200 grade is 180.

The percentage retained penetration is the ratio

$$\frac{\text{recovered penetration value}}{\text{assumed penetration value}} \times 100$$

The average percentage retained penetration is the combined average of the results of the R.M.O.C. and M.T.C. tests.

Based on the above assumptions, the average percentage retained penetration value for the asphalt cements of original 85/100 penetration value decreases with an increase in age of the asphaltic concrete. (Table 3.14) These results are very consistent with recommended specifications of M.T.C. respecting anticipated recovered pen values. M.T.C. suggest a recovered pen value of approximately 47% of the original 85/100 pen grade for

samples from new asphaltic concrete with a decrease in recovered pen value with age. In this analysis 47% of the original asphalt cement pen value yields a value of 42.3.

For those asphaltic concrete samples containing asphalt cement with an original pen grade of 150/200, the assumed original pen value for each sample was 180. The M.T.C. suggested percent recovered pen value for asphalt cement from newly layed asphaltic concrete is 42, yielding an expected pen value of $\left(\frac{42}{100} \times 180\right) = 75.6$.

As can be seen from table 3.14, the average percent retained pen value for 7 roadways ranged from 22.3% to 16.8% the latter being for the oldest sample (74 months). Compared to the anticipated value of 42%, these values obtained are unusually low especially for the newer asphalt cements. (2 test results at 15 months)

Although the test results show the lowest percent retained value for the oldest sample tested (an expected result), there is little variation in these results to satisfy the criteria of age-hardening of the asphalt cement.

With the exception of the results at station G, all of the original 150/200 pen value samples were taken from asphaltic concrete overlays of existing asphalt pavements.

3.3.2 Comments on the 150/200 pen grade asphalt cement results

It is the author's understanding that samples of asphalt cement being supplied for inclusion in asphaltic concrete

for the R.M.O.C. are no longer being taken and tested for physical properties. It is assumed, therefore, that the penetration grade of asphalt cement shown in the R.M.O.C. specifications is being supplied. Invariably the two grades of asphalt cement requested are 85/100 and 150/200.

Is it feasible that the asphaltic concrete samples taken and assumed to contain 150/200 pen grade asphalt cement actually contained 85/100 pen grade asphalt cement?

The laboratory tests results and analysis would then be as follows:

Station	Average Pen		% Ret. Pen *		Avg. % Ret. Pen.	Age (Mo.)
	RMOC	MTC	RMOC	MTC		
G	32.7	45.0	36.2	50.0	43.2	15
J	31.3	42.0	34.8	46.6	40.8	15
K	38.7	41.5	43.0	46.2	44.6	28
B	33.4	27.5	37.2	30.6	34.0	40
H	26.7	45.5	29.8	50.6	40.2	51
A	33.4	33.5	37.2	37.2	37.2	65
D	28.5	32.0	31.6	35.6	33.6	74

* - Original penetration grade is 85/100; original penetration value assumed as 90

The anticipated recovered penetration is 47% for 85/100 pen grade asphalt cement for newly constructed roadways.

The tabulated results shown above bear a close relationship to the anticipated retained pen values if the original asphalt cement was 85/100 pen grade.

In addition there exists a slight trend indicating a decrease in retained pen value with an increase in the age of the asphaltic concrete.

3.3.3 Retained Penetration Value and Transverse Crack Frequency

It has been established that transverse cracking increases with the age of the asphaltic concrete. It is also evident that the penetration value of recovered asphalt cement decreases with increasing age of the asphalt cement.

If the above two conditions are correct, then a decrease in the penetration value of recovered asphalt cement should be accompanied by an increase in transverse pavement cracking. In fact McLeod (3, 5, 20, 32) and others (11, 17) have shown this to be true.

In this particular study, insufficient data is available to confirm the above. Asphalt core samples were taken from those roadways where precise crack inventories for one-half mile sections of roadway were made. As noted in section 3.2.1, no conclusive comparisons between cracking frequency and other variables could be drawn from the one-half mile section counts.

Additional crack count inventories did however include five of the ten monitoring stations where asphaltic

concrete core sampling had been conducted. At these locations, a comparison of cracking frequency and retained penetration value of the asphalt cement was made and is shown in table 3.15.

The data is separated by type of roadway construction because of the differences in the nature of transverse crack occurrences in each. (Overlay construction produces reflection cracking whereas new construction does not show this characteristic).

The results from table 3.15 are inconclusive for the roadways on which an asphalt overlay was placed. For the two roadways listed under new construction, the data justifies the hypothesis that increased age of asphaltic concrete and asphalt cement produces greater frequency of transverse cracks and a lesser penetration value from the recovered asphalt cement.

3.4 TRANSVERSE PAVEMENT CRACKING AND TRAFFIC VOLUMES

Analysis of the field data of transverse pavement cracks for the one-half mile section of roadway surveyed showed no correlation with traffic volumes (Reference Table 3.10).

Traffic volumes were obtained for each of the roadways surveyed via the windshield method. In the analysis of the data, 12-hour volume counts (7:00 AM to 7:00 PM) were used because 24-hour counts were not available for all sections of roadway.

Separate data analyses were made for each of the totally

reconstructed roadways and those of asphaltic concrete overlay construction.

3.4.1 Totally Reconstructed Roadways

Table 3.16 and figure 3.13 provide the results of data analysis for the totally reconstructed roadways.

Figure 3.13 shows a plot of transverse pavement cracks per mile of two-lane roadway against traffic volumes for four combinations of crack types. The scatter of results is such that no correlation could be established for this data plot. The number of transverse pavement cracks per mile of two-lane roadway for the combination of crack types shown indicated that no consistent relationship could be produced with respect to traffic volumes when based solely on a volume vs crack type and frequency basis.

Traffic volumes were subsequently compared against the crack type and frequency, with the age of the asphaltic concrete included, to determine a relationship between crack frequency and traffic volumes. The results of this analysis are shown in table 3.16 under the heading "Volume-Crack Type-Age Ratios".

As was the case previously with the volume vs crack type and frequency analysis, no consistency in the data results could be obtained on a ratio basis for any of three comparisons. The table allows comparisons for several combinations of variables:

- a. Asphaltic concrete with the same age and different traffic volumes.

- b. Asphaltic concrete carrying approximately the same traffic volume but of different ages (i.e. Road #125, Volume 4209 and Road #12, Volume 4193).
- c. Asphaltic concrete of the same age with the same $\frac{\text{Volume}}{A \times \text{age}}$ ratio with different traffic volumes.
(i.e. Road #12, Volume 3041, and Road #12, Volume 4193)

3.4.2 Asphaltic Overlay Construction

Figure 3.14 and table 3.17 display the results of data analysis for those roadways which received an asphaltic overlay.

The scatter of results in figure 3.14 indicates a very inconsistent pattern in the plot of traffic volumes vs transverse cracks per mile of 2-lane roadway for the four combinations of crack types shown. The data results listed in table 3.17 under "volume-crack type ratios" substantiate the above graphical plot.

The age of the asphaltic concrete wearing surface was introduced into the volume-crack type ratios using data from table 3.12. The results of this analysis are shown in table 3.17 under the "volume-crack type-age ratios" heading. Again the random pattern of results is very evident indicating no apparent correlation between traffic volumes and either the number or types of transverse pavement cracks in the roadway surfaces.

3.4.3 Summary

The overall conclusion drawn from the analyses of traffic

volumes with types and frequency of occurrence of cracks in the asphaltic concrete wearing surface is that traffic volume is not a significant factor in this particular survey.

This conclusion is consistent with those of more in-depth studies conducted by others (McLeod; 32) for totally reconstructed (new) roadways.

The results of this report indicate that traffic volume is also not a significant factor in the above-mentioned type of analysis for two-lane rural roadways where asphaltic overlay construction has been undertaken.

The conclusions presented are consistent for both new and overlay construction for roadway surfaces of:

1. similar ages carrying different traffic volumes.
2. substantially different ages carrying approximately the same traffic volumes.

(Reference tables 3.13 and 3.14)

3.5 COMMENT ON TRANSVERSE PAVEMENT CRACKING AND TYPE OF ROADWAY CONSTRUCTION

Throughout this report, a distinction has been made between totally reconstructed roadways and roadways on which asphaltic concrete overlay work has been done.

To correctly assess the results of data analysis for temperature dependent transverse pavement cracking, it is essential to realize that all transverse pavement cracks are not necessarily as a result of the properties of the asphalt cement and asphalt mix. Some transverse cracking

may be attributed to the type of subgrade under the granular materials of the roadbed. Because of this, those roadways which have been reconstructed to the subgrade elevation have been categorized in this report under "totally reconstructed roadways" although no attempt has been made to identify which of the transverse cracks are attributed to the subgrade properties and which are essentially a result of the properties of the asphalt cement in the mix. It has been previously established by others that the single most important factor contributing to transverse pavement cracking in asphaltic concrete is the temperature dependent properties of the asphalt cement.

With respect to those roadways on which asphaltic concrete overlay work was carried out, the overlayed surfaces invariably contained transverse cracks as well as other structural pavement defects. Throughout the life of the asphaltic overlay, some of the previous structural defects will reflect through the new asphaltic concrete. This reflection cracking phenomenon is in part due to the properties of the asphaltic concrete mix but is also related to the physical condition of the overlayed asphalt and the type and number of vehicles on the roadway.

Again no attempt has been made in this report to differentiate the cause of transverse cracking in the overlayed asphalt between reflection cracking and first-time transverse pavement cracks related essentially to the properties of the asphalt cement.

For the reasons mentioned above, this author has chosen to separate the two methods of roadway construction in comparing results of data analysis.

CHAPTER 4
CONCLUSIONS AND RECOMMENDATIONS

4.1 CONCLUSIONS

4.1.1 Thermal Contraction of Asphaltic Concrete

Where transverse cracks exist in a flexible pavement, during subfreezing temperatures cracks will widen due to thermal contraction of the asphaltic concrete to a maximum of approximately 10 mm. It appears that this maximum coincides with the minimum daily temperatures of the winter months.

As winter conditions begin to subside and minimum daily temperatures increase, the transverse pavement cracks begin to close.

When freezing temperatures have completely ceased, the cracks will close to a minimum width which will be somewhat greater than the value prior to the beginning of freezing temperatures. (i.e. a permanent crack widening)

This permanent increase in transverse pavement crack width is due in part to the inability of the asphaltic concrete to retain its initial thermo-viscoelastic properties. It is evident from the literature (McLeod; 5, 19, 32; Deme; 10; Fromm and Phang; 15; Culley; 24) that this factor is the major cause for the initiation of the crack itself. Also contamination by soil particles and normal road debris such as sand, salt and water will have a contributing effect to the permanent increase in transverse pavement crack width. A knowledge of the maximum increase in pavement crack width can be extremely useful as a criterion for selecting materials which will

have the required thermo-viscoelastic properties to fill and repair existing cracks and yet remain sufficiently flexible so as not to allow cracking of the patching material to occur during the winter months of extreme cold temperatures. The field observations conducted in connection with this report suggest that these materials should be capable of thermal shrinkage to a minimum of 10 mm without failure in the filler material. Such materials, if found successful in reducing transverse crack reopening, could prove invaluable to minimizing reflection cracking through new overlay pavements, currently a serious asphalt associated problem. A knowledge of the magnitude of transverse pavement crack width increase during the winter months can provide valuable insight into current research to establish asphalt cement specifications to more adequately suit the climates similar to Ottawa for new transverse crack-free asphaltic concrete pavements. In addition, a knowledge of the magnitude of annual permanent increase in transverse pavement crack widths could be used as one of the major factors in modelling pavement deterioration with time to establish life expectancies of pavement surfaces.

Existing transverse pavement cracks terminating within the pavement surface (types II and III) will propagate to the pavement edges with time to produce type I cracks. The magnitude of such lengthening is dependent on the minimum temperature attained by the pavement with maximum crack lengthening occurring at the minimum temperature when increase in transverse crack width is also a maximum.

A minimum out-of-plane vertical shearing (0.2 mm average) occurred across transverse cracks as measured at each steel-pin location in this study.

Maximum displacements between steel pins across the centreline of the roadways at the monitoring stations were recorded at 1.0 mm where no longitudinal crack between the centreline pins was in evidence. These results substantiate that lateral restraint provided by the type of construction (2 lane pavements with granular shoulders) is sufficient to both minimize lateral movement of the pavement surface and temperature associated longitudinal pavement cracking.

4.1.2 Transverse Crack Frequency

For "totally reconstructed roadways", the following ratios were established.

$$\frac{\text{Total Cracks}}{\text{Type (I + II)* Cracks}} = 2.0 \quad ; \bar{X} = 2.0 , \sigma = 0.78$$

$$\frac{\text{Total Cracks}}{\text{McLeod Type 1 Cracks}} = 1.6 \quad ; \bar{X} = 1.6 , \sigma = 0.93$$

For existing pavements which were "overlaid with asphaltic concrete", the following ratios were established:

$$\frac{\text{Total Cracks}}{\text{Type (I + II) Cracks}} = 1.9 \quad ; \bar{X} = 1.9 , \sigma = 0.62$$

$$\frac{\text{Total Cracks}}{\text{McLeod Type I Cracks}} = 1.4 \quad ; \bar{X} = 1.4 , \sigma = 0.60$$

* as defined by this author

The total number of transverse pavement cracks per mile of two-lane roadway within the Regional Municipality of Ottawa-Carleton can be approximated by using either the author's definition of type I and II cracks or the definition of type I cracks of McLeod and applying the appropriate correction factor from the equations.

The frequency of transverse pavement cracking increases with the age of the asphaltic concrete wearing surface for these roadways surveyed for which the age ranged from one to six years.

For two-lane rural roadway pavements of the same age, the frequency of transverse cracking per mile is substantially greater on overlay surfaces than on totally reconstructed surfaces. This result is attributed in part to reflection cracking from the older overlaid pavements.

4.1.3 Asphalt Cement

The loss in penetration value with time of the recovered 85/100 pen grade asphalt cement is consistent with M.T.C. specifications.

The loss in penetration value with time of the recovered 150/200 pen grade asphalt cement is inconsistent with M.T.C. specifications. The test results suggest that the recovered asphalt cement penetration values contain a discrepancy factor of 2, being approximately twice as low as expected and showing little change in value with respect to age.

4.1.4 Traffic Volumes

No correlation between traffic volumes and either types or frequency of transverse pavement cracks was evident. This would indicate that traffic volume is not a significant factor in initiation of transverse pavement cracking on two-lane rural roadways in the area of this study.

Traffic volumes and wheel loads, however, are the most significant factors contributing to the deterioration of pavements at transverse crack locations. (Haas; 42, 45)

4.2 RECOMMENDATIONS

1. Materials used for transverse crack patching must possess sufficient thermo-viscoelasticity to accommodate a minimum longitudinal expansion of 10 mm at temperatures approximating -30°C to minimize deterioration at the transverse pavement crack due to a combination traffic loads and seasonal changes in the bearing strength of the pavement structures.
2. The grade of asphalt cement used in construction of asphaltic concrete pavements must be verified to ensure compliance with specifications prior to roadway construction being carried out.
3. Transverse pavement cracks should be repaired prior to placing asphaltic concrete overlays to reduce reflection cracking through the new asphaltic concrete layer.

4. Further study with respect to the actual width of transverse pavement cracks could be undertaken to establish the most advantageous time of year to conduct crack filling and patching to ensure a permanent and acceptable repair for the climatic conditions of the geographical location.
5. The mix of traffic should be investigated together with volume of traffic to establish if axle loads contribute to the frequency of transverse pavement cracking.

4.3 AUTHOR'S CLOSURE

The frequency of occurrence of transverse pavement cracking of flexible pavements is evident from the data presented. The service life and serviceability of the flexible pavements are initially lowered only marginally due to transverse cracking, however the resulting rapid deterioration of the pavement at these cracks results in accelerated loss in serviceability and, subsequently, service life. The control of this deterioration through reduction in transverse pavement cracking frequency can result in substantial savings in maintenance costs.

Although not a major factor in the initiation of transverse pavement cracks, traffic volumes and heavy wheel loads are the most significant factors contributing to the deterioration of pavements at transverse crack locations during the Spring of each year when pavement structures and subgrades are weakened by repeated freeze-thaw cycles.

Once transverse cracks exist in the pavement surface, the methods available to reduce the rate of deterioration at these cracks are limited to either surface treatments, crack cleaning and patching, or asphaltic concrete overlays.

These methods of pavement surface improvement are short term answers to retaining acceptable serviceability and extending the service life of the pavement to approximate the design life.

In 1976 and 1977 the maintenance cost for crack cleaning and patching on asphaltic concrete roadways within the R.M.O.C. approximated \$0.25/lineal foot of crack.

Based on the crack count inventory of Table 3.10, the costs for repairing transverse pavement cracks using the \$.25/lineal foot figure would be as follows:

(It is assumed that type I cracks are 22 feet in length; type II cracks are 11 feet in length; 25% of type III cracks are sufficiently long to warrant repair and the average length is 10 feet.)

Totally Reconstructed Roadways:

<u>Age</u> (Months)	<u>Ashphalt Cement</u> <u>Grade</u>	<u>Crack Repair</u> <u>Cost Estimates</u> \$/Mile
40	85/100	\$ 27.00
52	85/100	322.00
57	85/160	.97.00

Asphaltic Overlay Construction

<u>Age</u> (Months)	<u>Ashphalt Cement</u> <u>Grade</u>	<u>Crack Repair</u> <u>Cost Estimates</u> \$/Mile
8	150/200	\$146.00
18	150/200	220.00
32	150/200	338.00
44	150/200	785.00
55	150/200	735.00
76	150/200	685.00

The average crack repair cost per mile for totally reconstructed roadways is \$152.00 with a maximum of \$322.00 after approximately five years of service or one-third of the design service life of the asphalt pavement.

The crack repair costs per mile for asphaltic concrete overlays varies from \$146.00 to a maximum of \$785.00 after approximately 4 years of service (Fig 4.1).

Over the 500 miles of paved secondary roadways under the jurisdiction of the R.M.O.C., the crack repair costs could conservatively be estimated at \$250,000.00.

Using 1977 cost figures, this expenditure is the equivalent of constructing 8 miles of asphaltic concrete overlays.

The foregoing is based on repair costs of transverse pavement cracks only and has not considered other pavement defects which require maintenance expenditures. It is evident that low-temperature cracking is a major cause for concern in the area of this study.

Results of studies of the California State University at Fullerton, U.S.A., on the rate of reflection cracking through bituminous overlays suggest a theoretical time of 280 days to produce a reflection crack in a 3" thick overlay specific conditions of temperature and vehicular traffic ($\Delta T = 50^{\circ}F$, traffic equivalent = 137, 18 KIP axle loads/day).

Results in this report show reflection cracking in pavements at age 8 months (Table 3.10).

The combination of low-temperature transverse pavement cracking and reflection cracking in asphaltic concrete overlays was found to be a maximum at an age of between 44 and 55 months. If we assume an average age of 50 months and a cost/mile of \$750.00 (from page 72), a comparison with the worst case of low-temperature cracking of the totally reconstructed roadways at age 52 months (reflection cracking phenomenon eliminated) shows the cost/mile of repair as approximately \$320.00/mile. From the analysis it can be suggested that the crack repair cost attributed to reflection cracking alone on asphalt pavements of age 50 months is \$430.00/mile. On a two-lane rural roadway, this is equivalent to the crack repair cost for 78 type I cracks/mile at \$0.25/lineal foot. This number of type I cracks/mile is greater, with one exception, than the frequency of type I cracks/mile found on any of the sixteen (16) lengths of roadways listed in Table 3.10.

It is, therefore, suggested that adequate repair of existing transverse pavement cracks prior to asphalt overlays to reduce reflection cracking could produce a savings of \$430.00/mile for repair of reflection cracking that may occur after approximately 4 years of service of the new overlay, while substantially increasing the service life of the overlay.

Extrapolated to the 500 miles of paved secondary roadways under the jurisdiction of the R.M.O.C., the potential cost saving approximates \$225,000.00 over a minimum four year cycle.

FIG. 2.0

TYPICAL MONITORING STATION ARRANGEMENT
APPROX. SCALE: 1" = 4' HORIZ.

FIG. 2.1

TYPICAL MONITORING STATION ARRANGEMENT
FOR STATIONS 'A','B','E','H','J'
SCALE: 1" = 4' HORZ.

FIG. 2.2

STATION 'C' LAYOUT: NOV. 3, 1976
SCALE: 1" = 4' HORZ.

STATION 'D' LAYOUT: NOV. 5, 1976
 SCALE: 1" = 4' HORZ.

FIG. 2.4

STATION 'F' LAYOUT : NOV. 5, 1976
 SCALE: 1" = 4' HORIZ.

FIG. 2.5

CONTROL REFERENCE STATION
STATION 'G' LAYOUT, NOV. 5, 1976
SCALE: 1" = 4' HORZ.

FIG. 2.6

STATION 'K' LAYOUT : NOV. 20, 1976
SCALE: 1" = 4' HORZ.

FIG. 2.7

TRANSVERSE PAVEMENT CRACK SURVEY
 DATE OF SURVEY: OCT. 1976
 REGIONAL ROAD 19: ASPHALT OVERLAY 1971; STA. 'A'
 ASPHALT CEMENT 150/200 PEN.

FIG. 2.8

TRANSVERSE PAVEMENT CRACK SURVEY

DATE OF SURVEY: OCT. 1976

REGIONAL ROAD 19: ASPHALT OVERLAY 1973; STA. 'B'

ASPHALT CEMENT 150/200 PEN.

FIG. 2.9

2 2
OF/DE

TRANSVERSE PAVEMENT CRACK SURVEY

DATE OF SURVEY: OCT. 1976

REGIONAL ROAD 13; NEW CONSTRUCTION 1973; STA. 'C'

ASPHALT CEMENT: 85/100 PEN.

FIG. 2.10

SCALE 1" = 100' LONG.

TRANSVERSE PAVEMENT CRACK SURVEY
 DATE OF SURVEY: OCT. 1976
 REGIONAL ROAD 12: ASPHALT OVERLAY 1970; STA. 'D'
 ASPHALT CEMENT: 150/200 PEN.

FIG. 2.11

SCALE 1" = 100' LONG.

TRANSVERSE PAVEMENT CRACK SURVEY

DATE OF SURVEY: OCT. 1976

REGIONAL ROAD 12: NEW CONSTRUCTION 1972; STA. 'E'

ASPHALT CEMENT: 85/100

FIG. 2.12

SCALE 1" = 100' LONG

TRANSVERSE PAVEMENT CRACK SURVEY
 DATE OF SURVEY: OCT. 1976
 REGIONAL ROAD 28: NEW CONSTRUCTION 1974; STA. 'F'
 ASPHALT CEMENT 85/100 PEN.

FIG. 2.13

SCALE 1" = 100' LONG

TRANSVERSE PAVEMENT CRACK SURVEY

DATE OF SURVEY: OCT. 1976

REGIONAL ROAD 11 : ASPHALT OVERLAY 1972 ; STA. 'H'

ASPHALT CEMENT 150/200 PEN.

FIG. 2.14

TRANSVERSE PAVEMENT CRACK SURVEY
 DATE OF SURVEY: OCT. 1976
 REGIONAL ROAD 5: ASPHALT OVERLAY 1975; STA. 'J'
 ASPHALT CEMENT 150/200 PEN.

FIG. 2.15

SCALE 1" = 100' LONG

TRANSVERSE PAVEMENT CRACK SURVEY
 DATE OF SURVEY: OCT. 1976
 REGIONAL ROAD 5 : HIGHWAY 7 NORTHWARD, ASPHALT OVERLAY 1974, STA. 'K'
 ASPHALT CEMENT 150/200 PEN.

FIG. 2.16

CROSS-SECTION : STATION 'A'

LOCATION : REGIONAL ROAD 19

4.3 MI. SOUTH OF REGIONAL ROAD 8

SCALE : 1" = 10' HORIZ.

1" = 1' VERT.

FIG. 2.17

CROSS-SECTION : STATION 'B'

LOCATION : REGIONAL ROAD 19

3.2 MI. SOUTH OF REGIONAL ROAD 8

SCALE : 1" = 10' HORZ.

1" = 1' VERT.

FIG. 2.18

CROSS-SECTION : STATION 'C'

LOCATION : REGIONAL ROAD 13

1.0 MI. NORTH OF REGIONAL ROAD 4

SCALE: 1" = 10' HORZ.

1" = 1' VERT.

FIG. 2.19

CROSS-SECTION : STATION 'D'

LOCATION : REGIONAL ROAD 12

0.7 MI. WEST OF REGIONAL ROAD 10

SCALE : 1" = 10' HORIZ.

1" = 1' VERT.

FIG. 2.20

CROSS-SECTION : STATION 'E'

LOCATION : REGIONAL ROAD 112

1.1 MI. WEST OF REGIONAL ROAD 11

SCALE : 1" = 10' HORIZ.

1" = 1' VERT.

FIG. 2.21

CROSS-SECTION : STATION 'F'

LOCATION : REGIONAL ROAD 28

0.3 MI. SOUTH OF INTERSECTION OF REG. RDS. 28 & 31

SCALE : 1" = 10' HORZ.

1" = 1' VERT.

FIG. 2.22

LOCATION : REGIONAL ROAD 28

0.3 MI. N.-E. OF INTERSECTION OF REG. RDS. 28 & 31

SCALE : 1" = 10' HORZ. &

1" = 1' VERT.

FIG. 2.23

CROSS-SECTION : STATION 'H'

LOCATION : REGIONAL ROAD 11

0.4 MI. NORTH OF REGIONAL ROAD 12

SCALE : 1" = 10' HORIZ.

1" = 1' VERT.

FIG. 2.24

CROSS SECTION STATION 'J'
LOCATION : REGIONAL ROAD 5

0.4 MI. NORTH OF REGIONAL ROAD 12
SCALE : 1" = 10' HORIZ.
1" = 1' VERT.

FIG. 2.25

CROSS-SECTION : STATION 'K'

LOCATION : REGIONAL ROAD 5

0.7 MI. NORTH OF HWY. 7

SCALE : 1" = 10' HORIZ.

1" = 1' VERT.

FIG. 2.26

REL. LONG. MOVEMENT (MM.)

MIN. DAILY TEMP. DEG. C

FIG. 3.1

FIG. 3.2

FIG. 3.4

FIG. 3.5

REL. LONG. MOVEMENT (MM.)

LOT

MIN. DAILY TEMP. DEG. C

FIG. 3.6

FIG. 3.7

REL. LONG. MOVEMENT (MM.)

60T

MIN. DAILY TEMP. DEG. C

FIG. 3 R.

FIG. 3.9

FIG. 3.10

NEW CONSTRUCTION

- - TYPE I
- △ - TYPE I + II
- - TYPE I + II + III
- - McLEOD TYPE I

TRANSVERSE CRACKS PER MILE VS. AGE OF ASPHALTIC CONCRETE

□

TRANSVERSE CRACKS PER MILE - 2 LANES

134.6
100
80
60
40
20
0

AGE (MONTHS)

60

50

40

30

FIG. 3.11

ASPHALT OVERLAY CONSTRUCTION

TRANSVERSE CRACKS PER MILE VS. AGE OF ASPHALTIC CONCRETE OVERLAY

- O - TYPE I
- Δ - TYPE I + II
- - TYPE I + II + III
- - McLEOD TYPE I

FIG. 3.12

TRANSVERSE CRACKS PER MILE VS. TRAFFIC VOLUME

TOTAL RECONSTRUCTION

□

- - TYPE I
- △ - TYPE I + II
- - TYPE I + II + III
- - MCLEOD TYPE I

TRANSVERSE CRACKS PER MILE - 2 LANES

114

TRAFFIC VOLUME (12 HR. TOTAL) X 100

FIG. 3.13

ASPHALTIC OVERLAY CONSTRUCTION

TRANSVERSE CRACKS PER MILE VS. TRAFFIC VOLUME

- O - TYPE I
- Δ - TYPE I + II
- - TYPE I + II + III
- - McLEOD TYPE I

FIG. 3.14

CRACK REPAIR COSTS PER MILE VS. NUMBER OF WINTERS "IN - SERVICE "

- ASPHALTIC CONCRETE OVERLAY
- x TOTALLY RECONSTRUCTED ROADWAYS

COST OF REPAIRS PER MILE

NUMBER OF WINTERS "IN - SERVICE "

FIG. 4.1

TABLE 3.1

STATION	MAXIMUM DISPLACEMENT BETWEEN PINS (mm)					ASPHALTIC CONCRETE		
	ab	cd	ef	gh	xy	a ₁ a	AGE (MONTHS) ¹	GRADE OF ASPHALT CEMENT
TOTAL RECONSTRUCTION								
G ²	0.5	1.0	0.5	0.5			15	150/200
F	2.0 ³	8.0	9.5	9.5 ³			25	85/100
C	4.0	D	5.0	0.5 ³			37	85/100
E	3.0	4.5	4.0	3.5			49	85/100
ASHPALT OVERLAY CONSTRUCTION								
J	6.0	5.5	4.0	4.5			15	150/200
K	2.5	2.5	D	3.0		0.5	28	150/200
B	3.5	5.5	6.0	5.0			40	150/200
H	4.0	3.5	3.0	3.0			51	150/200
A	5.0	5.0	5.0	5.5			65	150/200
D	1.5 ³	0.5 ³	0.5 ³	0.0 ³	0.5		74	150/200

Maximum displacement vs age of asphaltic concrete.

- 1 - As of November 1976
- D - Damaged (Probably by snow removal equipment)
- 2 - Control Station - no crack between pins.
- 3 - Initially no crack between pins.

TABLE 3.2

STATION ³	PERMANENT CRACK WIDTH INCREASE				ASPHALTIC CONCRETE	
	ab	cd	ef	gh	AGE (MONTHS)	GRADE OF ASPHALT CEMENT
<u>TOTAL RECONSTRUCTION</u>						
C	1.0	-	1.75	0.0 ¹	37	85/100
E	0.25	2.5	2.0	0.5	49	85/100
<u>ASPHALT OVERLAY CONSTRUCTION</u>						
J	1.0	1.0	0.25	0.0	15	150/200
K	0.25	-0.5 ²	-	0.0	28	150/200
B	0.5	1.0	1.5	1.0	40	150/200
H	0.0	0.0	1.0	0.0	51	150/200
A	1.0	1.5	1.0	1.0	65	150/200

Permanent Crack Width Increase

- 1 - No crack between pins
- 2 - Negative result
- 3 - Results at Station F not recorded (Seed 3.1.1.)
- No crack at Control Station G and D for steel pins shown.

TABLE 3.3a

STATION	DATE	CRACK LENGTH		LENGTH INCREASE	DIRECTION OF INCREASE	
					LEFT	RIGHT
	Nov.03/76	* 33½"	64"	INITIAL MEASUREMENT		
C	Dec.04/76	33½"	65"	1"	X	
	Dec.16/76	33½"	65½"	1/2"		Uncertain
	Jan.26/77		153½"	88"		X
		73½"		40"		X*
NOTE: Both cracks are now one for 227 inches.						
	Feb.24/77		228"	1"		
	Mar.03/77		228"	NIL		
	Mar.27/77		228"	NIL		
	Apr.07/77		228"	NIL		
	May 03/77		228"	NIL		
	May 29/77		228"	NIL		
	TOTAL INCREASE IN LENGTH:			130½" ±		

Crack lengthening at Station "C"

- * It is unknown as to which of the two cracks lengthened firstly and by how much, however, on January 26, 1977, a continuous crack existed from the left edge of pavement at pin ab for a total length of 227 inches.

TABLE 3.3b

STATION	DATE	CRACK LENGTH	LENGTH INCREASE	DIRECTION OF INCREASE	
				LEFT	RIGHT
	Nov.05/76	56½"	INITIAL MEASUREMENT		
D	Dec.04/76	59½"	3"	1½"	1½"
	Dec.08/76	61"	1½"		1½"
	Jan.26/77	79"	18"	15"	3"
	Feb.14/77	90"	11"		11"
	Mar.03/77	94"	4"		4"
	Mar.27/77	94"	NIL		
	Apr.07/77	94"	NIL		
	May 03/77	94"	NIL		
	May 29/77	94"	NIL		
	TOTAL INCREASE IN LENGTH:		37½"		

Crack lengthening at Station "D"

TABLE 3.3c

STATION	DATE	CRACK LENGTH		LENGTH INCREASE	DIRECTION OF INCREASE	
					LEFT	RIGHT
F ¹	Nov. 05/76	78"	99"	INITIAL MEASUREMENT		
	Dec. 16/76	82"			4"	
			105"			6"
	Jan. 26/77 ²	149"			67"	
			148"			43"
TOTAL INCREASE IN LENGTH:				120"		

Crack lengthening at Station "F"

- 1 Crack lengthening at this station is attributed partially to subgrade movement (Ref. 3.1.1.d)
- 2 Both initial cracks extended to the edge of pavement and joined together along the construction joint.

TABLE 3.4

VERTICAL DISPLACEMENT OF ROADWAY
SURFACE AT STEEL-PIN LOCATION

STATION	DATE OF OBSERVATION	STEEL PIN ELEVATION (FT.)									
		a	b	c	d	e	f	g	h		
A B.M. EL 400.000	Dec.09/76	0.601	.600	.900	.882	.955	.959	1.020	1.021	Initial Observation	
	Jan.07/77	.607	.608	.907	.894	.965	.969	1.022	1.022		
	Feb.09/77	.612	.618	.911	.902	.923	.975	1.025	1.023		
	Mar.10/77	.585	.578	.881	.868	.938	.938	1.018	1.018		
	Maximum Displacement(ft) (mm)	.026 0.7	.040 1.0	.030 0.8	.026 0.7	.048 1.2	.037 0.9	.007 0.2	.005 0.1		
B B.M. EL 300.000	Dec.09/76	8.592	8.593	8.236	8.232	8.212	8.211	7.950	7.953	Initial Observation	
	Jan.07/77	.636	.640	.284	.282	.260	.258	.992	.993		
	Feb.09/77	.646	.653	.295	.293	.270	.270	.996	.998		
	Mar.10/77	.653	.658	.283	.283	.253	.258	.980	.980		
	Maximum Displacement(ft) (mm)	.054 1.4	.065 1.7	.059 1.5	.061 1.5	.068 1.7	.059 1.5	.046 1.2	.045 1.1		
C B.M. EL 500.000	Dec.09/76	1.837	1.831	2.024	2.021	2.027	2.025	1.957	1.960	Initial Observation	
	Jan.07/77	1.902	1.895	2.084	2.086	2.084	2.082	2.016	2.021		
	Feb.09/77	1.950	1.938	2.132	2.132	2.129	2.130	2.062	2.067		
	Mar.09/77	1.948	1.940	2.115	2.105	2.115	2.112	2.062	2.067		
	Maximum Displacement(ft) (mm)	.113 2.9	.109 2.8	.108 2.8	.111 2.9	.102 2.6	.105 2.7	.105 2.7	.102 2.6		

(cont'd)

TABLE 3.4

VERTICAL DISPLACEMENT OF ROADWAY
SURFACE AT STEEL-PIN LOCATION

STATION	DATE OF OBSERVATION	STEEL PIN ELEVATION (FT.)									
		a	b	c	d	e	f	g	h	x	y
D	Dec.09/76	8.139	8.141	8.440	8.442	8.445	8.444	8.423	8.423	8.483	8.487
B.M. EL											Initial
300.000	Jan.06/77	.244	.248	.604	.605	.615	.615	.533	.534	.625	.628
	Feb.07/77	.308	.308	.730	.730	.750	.745	.605	.603	.730	.735
	Mar.09/77	.332	.332	.755	.760	.762	.765	.633	.633	.758	.765
Maximum Displacement(ft)		.193	.191	.315	.318	.317	.321	.210	.210	.275	.278
(mm)		4.9	4.9	8.0	8.1	8.1	8.2	5.3	5.3	7.0	7.1
E	Dec.08/76	9.251	9.259	9.479	9.490	9.468	9.480	9.275	9.283	Initial	
B.M. EL										Observation	
200.000	Jan.06/77	.217	.221	.446	.460	.437	.450	.256	.265		
	Feb.08/77	.232	.243	.482	.498	.475	.492	.315	.333		
	Mar.09/77	.285	.295	.528	.538	.525	.540	.388	.400		
Maximum Displacement(ft)		.068	.074	.082	.078	.088	.090	.132	.135		
(mm)		1.7	1.9	2.1	2.0	2.2	2.3	3.3	3.4		
F	Dec.09/76	9.266	9.254	9.506	9.503	9.495	9.505	9.247	9.258	Initial	
B.M. EL										Observation	
200.000	Jan.06/77	.360	.355	.633	.624	.625	.618	.370	.375		
	Feb.08/77	.477	.474	.766	.756	.761	.746	.514	.520		
	Mar.09/77	.535	.525	.792	.785	.792	.892	.560	.462		
Maximum Displacement(ft)		.269	.271	.285	.282	.297	.387	.313	.204		
(mm)		6.8	6.9	7.3	7.2	7.5	9.8	8.0	5.2		

(cont'd)

TABLE 3.4

VERTICAL DISPLACEMENT OF ROADWAY
SURFACE AT STEEL-PIN LOCATION

STATION	DATE OF OBSERVATION	STEEL PIN ELEVATION (FT.)									
		a	b	c	d	e	f	g	h		
G B.M. EL 100.000	Dec. 09/76	5.492	5.503	5.395	5.402	5.369	5.380	5.215	5.223	Initial	
	Jan. 06/77	.571	.588	.461	.469	.433	.445	.290	.297	Observation	
	Feb. 07/77	.611	.628	.503	.511	.481	.491	.353	.363		
	Mar. 09/77	.618	.628	.503	.508	.478	.488	.348	.360		
	Maximum Displacement (ft) (mm)	.126 3.2	.125 3.2	.108 2.7	.106 2.7	.109 2.8	.108 2.7	.133 3.4	.137 3.5		
H B.M. EL 100.000	Dec. 08/76	7.875	7.854	8.002	7.996	8.007	7.992	7.846	7.822	Initial	
	Jan. 06/77	.877	.859	.017	8.001	.011	.997	.850	.826	Observation	
	Feb. 08/77	.890	.870	.025	8.015	.020	8.008	.850	.825		
	Mar. 09/77	.884	.863	.010	7.995	.010	7.980	.845	.820		
	Maximum Displacement (ft) (mm)	.015 0.4	.016 0.4	.024 0.6	.020 0.5	.013 0.3	.028 0.7	.005 0.1	.006 0.1		
J B.M. EL 400.000	Dec. 08/76	9.271	9.249	9.453	9.429	9.442	9.418	9.255	9.227	Initial	
	Jan. 06/77	.281	.256	.456	.431	.443	.421	.266	.237	Observation	
	Feb. 07/77	.280	.258	.457	.432	.445	.420	.268	.237		
	Mar. 09/77	.270	.247	.454	.424	.439	.417	.247	.217		
	Maximum Displacement (ft) (mm)	.011 0.3	.009 0.2	.004 0.1	.008 0.2	.006 0.2	.004 0.1	.021 0.5	.020 0.5		

(cont'd)

TABLE 3.4

VERTICAL DISPLACEMENT OF ROADWAY
SURFACE AT STEEL-PIN LOCATION

STATION	DATE OF OBSERVATION	STEEL PIN ELEVATION (FT.)							
		a	b	c	d	e	f	g	h
K B.M. EL 500.000	Dec. 08/77	8.525	8.524	8.574	8.573	8.574	8.565	8.284	8.289
	Jan. 15/77	.524	.522	.577	.576	.578	.570	.292	.294
	Feb. 07/77	.428	.525	.585	.583	.587	.578	.297	.302
	Mar. 09/77	.534	.534	.570	.569	.572	.562	.277	.282
Maximum Displacement (ft)		.106	.011	.015	.014	.015	.016	.020	.020
(mm)		2.7	0.3	0.4	0.4	0.4	0.5	0.5	0.5
									0.2

TABLE 3.5

STATION	AVERAGE MAXIMUM VERTICAL DISPLACEMENT (mm)	DATE MEASURED	AGE OF ASPHALTIC CONCRETE (MONTHS)
TOTAL RECONSTRUCTION			
G	3.0	Feb. 7	15
F	7.3*	Feb. 7	25
C	2.8	Feb. 9	37
E	2.4*	Feb. 9	49
ASPHALT OVERLAY CONSTRUCTION			
J	0.3	Feb. 7	15
K	0.6	Feb. 7	28
B	1.4	Feb. 9	40
H	0.4	Feb. 8	51
A	0.6	Feb. 9	65
D	6.7*	Feb. 7	74

Average maximum vertical displacement vs roadway construction type.

* - Continuing vertical displacement recorded last date of measurement, March 9, 1977.

TABLE 3.6

STATION	MAXIMUM DISPLACEMENT ACROSS CENTRELINE (mm)	
	ce	df
A	0.0	1.0
B	0.5	0.5
C	0.0	*
D	18.5	19.5
E	0.0	1.0
G	1.0	1.0
H	0.5	0.5
J	0.5	0.5
K	0.0	**

Maximum displacement across centreline joint.

* Steel pin d was destroyed

** Steel pin f was destroyed

TABLE 3.7

STATION 'D'		MEASUREMENTS (mm)	
DATE OF OBSERVATION	ce	df	
Nov. 05/76	297	283	Initial Observation
Dec. 04/76	297½	285	
Dec. 08/76	298	285½	
Dec. 16/76	298	285½	
Jan. 26/77	312	299	
Feb. 14/77	314	302	
Mar. 03/77	315½	302½	
Mar. 27/77	312	299½	
Apr. 07/77	310	297½	
May 03/77	303	290½	
May 29/77	301	289	

Measurements across centreline joint at Station "D"

TABLE 3.8

Station	Transverse Cracks Counts				Asphaltic Concrete	
	Type I	Type II	Type III	Total	Age (Months)	Grading of Asphalt Cement
<u>TOTAL RECONSTRUCTION</u>						
G	0	0	0	0	15	150/200
F	4	0	30	34	25	85/100
C	0	2	6	8	37	85/100
E	55	10	32	87	49	85/100
<u>OVERLAY CONSTRUCTION</u>						
J	3	2	2	7	15	150/200
K	15	35	103	153	28	150/200
B	3	5	22	30	40	150/200
H	29	21	80	130	51	150/200
A	13	6	20	39	65	150/200
D	18	26	86	129	74	150/200

Transverse Crack Type vs (i) Age of Asphaltic Concrete
(ii) Grade of Asphalt Cement in Mix

TABLE 3.9

Station	Transverse Crack Count by Type Per 1/2 Mi. Section			Asphaltic Concrete	
	I	I + II	I + II & III	Age (Months)	Grade of Asphalt Cement
<u>TOTAL RECONSTRUCTION</u>					
G	0	0	0	15	150/200
F	4	4	34	25	85/100
C	0	2	8	37	85/100
E	55	65	87	49	85/100
<u>OVERLAY CONSTRUCTION</u>					
J	3	5	7	15	150/200
K	15	50	153	28	150/200
B	3	8	30	40	150/200
H	29	50	130	51	150/200
A	13	19	39	65	150/200
D	18	43	129	74	150/200

Summation of Crack Type vs (i) Age of Asphaltic Concrete.
(ii) Grade of Asphalt Cement in Mix.

TABLE 3.10

Regional Road No.	Length Surveyed (Mi)	Transverse Crack Count			Total	Asphaltic Concrete		Traffic Volumes 12 Hr. Totals
		Type I	Type II	Type III		Age * (Mo.)	Grade of Asphalt Cement	
NEW CONSTRUCTION **								
13	4.3	9	10	18	37	40	85/100	1,890
125	1.1	4	7	15	26	40	85/100	4,209
12	1.3	77	21	29	127	52	85/100	2,418
12	1.2	40	44	20	104	52	85/100	3,041
12	1.3	15	12	27	54	52	85/100	4,193
9	1.7	6	30	82	118	57	85/100	2,081
OVERLAY CONSTRUCTION **								
9	1.3	20	25	15	60	8	150/200	1,069
5	2.9	96	33	26	155	18	150/200	1,118
5	4.5	184	274	316	774	32	150/200	2,709
9	1.2	7	33	95	135	32	150/200	1,213
9	2.4	46	142	163	351	32	150/200	1,659
9	2.8	190	309	466	965	44	150/200	2,081
11	2.6	264	190	402	856	55	150/200	1,944
5	0.9	24	90	66	180	55	150/200	1,527
125	2.0	150	122	341	613	76	150/200	2,772
9	3.8	43	42	77	162	78	150/200	1,410
TRANSVERSE CRACK FIELD DATA								

* Age was computed from the date of construction to March 1, 1977.

** Reference Table 3.18 for locations of Regional Roads surveyed.

TABLE 3.11

Regional Road No.	Length Surveyed (Mi)	Transverse Cracks Data			Total = C	C x age 12	Age Mo.	
		Cracks		B x age 12				
		Total Type I	Total Type I + II					
New Construction								
13	4.3	4.1	13.7	1.9	6.3	1.3	4.4	40
125	1.1	6.6	21.7	2.4	8.0	2.7	9.1	40
12	1.3	1.6	7.1	1.3	5.6	0.7	3.1	52
12	1.2	2.6	11.3	1.2	5.2	0.8	3.6	52
12	1.3	3.6	15.6	2.0	8.7	1.3	5.5	52
9	1.7	19.8	93.4	3.3	15.7	2.8	13.4	57

Ratios of type of transverse crack per mile of two lane roadway to total number of cracks vs age of the asphaltic concrete wearing surface.

TABLE 3.12

Regional Road No.	Length Surveyed (Mi)	T r a n s v e r s e			C r a c k s			Age Mo.
		Total Type I	= A A x age 12	Total Type I + II	= B B x age 12	Total McLeod Type I	= C C x age 12	
Asphalt Overlay								
9	1.3	3.0	2.0	1.3	✓	0.9	0.6	8
5	2.9	1.6	2.4	1.2		1.8	1.1	18
5	4.5	4.2	11.2	1.7		4.5	3.2	32
9	1.2	19.3	51.5	3.4		9.0	7.7	32
9	2.4	7.6	20.3	1.9		5.1	4.0	32
9	2.8	5.1	18.7	1.9		6.9	5.1	44
11	2.6	3.2	14.7	1.9		8.7	5.5	55
5	0.9	7.5	34.4	1.6		7.3	6.0	55
125	2.0	4.1	25.9	2.3		14.6	9.5	76
9	3.8	3.8	24.7	1.9		12.4	8.5	78

Ratios of type of transverse crack per mile of two lane roadway to total number of cracks vs age of the asphaltic wearing surface.

TABLE 3.13

Station	Penetration Values		Original Pen Grade Asphalt Cement	Age (Months)
	RMOC Lab	MTC Lab		
TOTAL RECONSTRUCTION				
G	34.3	53	150/200	15
	31.0	37		
F	29.3	45	85/100	25
	29.0			
C	32.1	34	85/100	37
	35.0	34		
E	24.7	37	85/100	49
	23.3	30		
ASPHALT OVERLAY CONSTRUCTION				
J	37.5	42	150/200	15
	31.0	42		
K	40.3	43	150/200	28
	37.0	40		
B	39.7	33	150/200	40
	27.0	22		
H	26.0	39	150/200	51
	27.7	52		
A	38.5	35	150/200	65
	28.2	32		
D	26.3	32	150/200	74
	30.7	33		

Penetration test results on recovered asphalt cement.

TABLE 3.14

Station	Average Pen		Original * Penetration	% Ret. Pen		Average % Ret. Pen	Age (Mo.)
	RMOC	MTC		RMOC	MTC		
F	29.2	45.0	90	30.2	50.0	40.1	25
C	33.6	34.0	90	37.3	37.8	37.5	37
E	24.0	33.5	90	26.7	37.2	31.9	49
G	32.7	45.0	180	18.1	25.0	21.6	15
J	31.3	42.0	180	17.4	23.3	20.4	15
K	38.7	41.5	180	21.5	23.1	22.3	28
B	33.4	27.5	180	18.6	15.3	17.0	40
H	26.9	45.5	180	14.9	25.3	20.1	51
A	33.4	33.5	180	18.6	18.6	18.6	65
D	28.5	32.0	180	15.8	17.8	16.8	74

Average % retained penetration vs age of asphaltic concrete.

* - Assumed to be 90 for 85/100 grade and 180 for 150/200 grade.

TABLE 3.15

Station	Transverse Cracks Per Mile (All Types)	% Retained Penetration	Asphaltic Concrete Grade of Asphalt Cement	Age (Mo.)
<u>OVERLAY CONSTRUCTION</u>				
J	53	20.4	150/200	15
K	172	22.3	150/200	28
H	329	20.1	150/200	51
<u>NEW CONSTRUCTION</u>				
C	9	37.5	85/100	37
E	75	31.9	85/100	49

Transverse crack frequency and asphalt cement characteristics.

TABLE 3.16

Regional Rd. No.	Length Surveyed	Traffic Volume 12 Hr. Total ²	Volume - Crack Type - Age Ratios ¹			Volume - Crack Type Ratios			Age (Mo.)	
			Vol. A x age	Vol. B x age	Vol. C x age	Vol. I	Vol. I + II	Vol. Total		Vol. McLeod I
TOTAL RECONSTRUCTION										
13	4.3	1,890	12	25	36	900	429	220	291	40
125	1.1	4,209	16	44	39	1,169	421	178	309	40
12	1.3	2,418	29	36	66	41	32	25	18	52
12	1.2	3,041	21	49	73	91	43	35	29	52
12	1.3	4,193	21	40	62	365	202	101	131	52
9	1.7	2,081	17	11	13	595	99	30	84	57

Ratios of type of transverse cracks per mile of 2-lane roadway vs traffic volume for totally reconstructed roadways.

1 - A, B and C are taken from table 3.11.

2 - 12 Hr. Volumes are from 7:00 AM to 2:00 PM.

TABLE 3.17

Regional Rd. No.	Length Surveyed	Traffic Volume 12 Hr. Total ²	Volume - Crack Type - Age Ratios ¹			Volume - Crack Type Ratios				Age (Mo.)
			Vol.	Vol.	Vol.	Vol.	Vol.	Vol.		
			A x age	B x age	C x age	I	I + II	Total	McLeod I	
ASPHALT OVERLAY CONSTRUCTION										
9	1.3	1,069	45	103	149	70	31	23	21	8
5	2.9	1,118	39	52	89	34	25	21	15	18
5	4.5	2,709	20	50	70	66	27	16	20	32
9	1.2	1,213	2	11	13	208	36	11	31	32
9	2.4	1,659	7	27	35	86	21	11	17	32
9	2.8	2,081	9	25	34	31	12	6	9	44
11	2.6	1,944	11	19	29	19	26	6	7	55
5	0.9	1,527	4	17	21	57	12	8	10	55
125	2.0	2,772	9	16	24	37	20	9	13	76
9	3.8	1,410	5	10	14	125	63	33	43	78

Ratios of types of transverse cracks per mile of 2-lane roadway vs Traffic Volumes for asphalt overlay construction.

1 - A, B and C are taken from Table 3.12.

2 - 12 Hr. Volumes are from 7:00 AM to 7:00 PM.

TABLE 3.18

LOCATIONS FOR CRACK COUNT INVENTORY

Regional Road No.	Length Surveyed (Mi)	RMOC Contract No.	Approximate Limits of Contract
NEW CONSTRUCTION			
13	4.3	70-526	Century Rd to Reg. Rd 4
125	1.1	73-521	Walkley Rd to Ottawa City Limit
12	1.3	70-544	Reg Rd 10 to Reg Rd 11
12	1.2	70-544	Reg Rd 11 to Reg Rd 23
12	1.3	70-544	Reg Rd 23 to Reg Rd 13
9	1.7	71-503	Riddell Road to 0.7 mi south of C.N.R. crossing
OVERLAY CONSTRUCTION			
9	1.3	76-509	Reg Rd 20 to Constance Bay Rd
5	2.9	75-540	Reg Rd 12 to 0.8 Mi north of Reg Rd 10
5	4.5	74-502	Highway #17 to 1.5 Mi north of Highway 7
9	1.2	74-502	0.5 Mi south of Reg Rd 129 to Dunrobin
9	2.4	74-502	Reg Rd 20 to 0.5 Mi south of Reg Rd 129
9	2.8	73-513	0.7 Mi south of C.N.R. crossing to Dunrobin
11	2.6	72-508	Reg Rd 10 to 0.5 Mi north of Reg Rd 12
5	0.9	72-508	Highway #17 to Village of Carp
125	2.0	71-506	Ottawa City Limits to Highway #31
9	3.8	71-504	Reg Rd 20 to Reg Rd 22

A P P E N D I X " A "

MONTHLY METEOROLOGICAL SUMMARY

MONTHLY METEOROLOGICAL SUMMARY SOMMAIRE MÉTÉOROLOGIQUE MENSUEL

MONTH/MOIS OCTOBER/OCTOBRE

19 70

AT/A OTTAWA INTERNATIONAL AIRPORT/AÉROPORT INTERNATIONAL D'OTTAWA

LAT 45 19 N		LONG 75 40 W		ELEVATION ALTITUDE 374		FEET (ASL) PIEDS (MER)		STANDARD TIME USED HEURE NORMALE UTILISÉE		EASTERN DE L'EST									
DATE	TEMPERATURE TEMPERATURE			DEGREE DAYS DEGRÉS-JOURS		REL HUMIDITY HUMIDITÉ REL.		THUNDERSTORM ORAGE	PRECIPITATION PRÉCIPITATIONS			SNOW ON GROUND NEIGE AU SOL	WIND VENT			BRIGHT SUNSHINE INSOLATION EFFECTIVE			
	MAXIMUM MAXIMALE	MINIMUM MINIMALE	MEAN MOYENNE	HEATING DE CHAUFFÉE	GROWING DE CROISSANCE	MAXIMUM MAXIMALE	MINIMUM MINIMALE		RAINFALL PLUIE (HAUTEUR)	SNOWFALL NEIGE (HAUTEUR)	TOTAL PRECIP PRÉCIP TOTALE		AVERAGE SPEED VITESSE MOYENNE	PREVAILING DIRECTION DIRECTION DOMINANTE	MAXIMUM SPEED AND DIRECTION VITESSE MAXIMALE ET DIRECTION				
	°C	°C	°C	BASE 18.0°C	BASE 5.0°C	%	%		mm	cm	mm		km/h		km/h		HOURS HEURES		
1	19.4	3.5	11.5	6.5	6.5	84	43						7.6	E	NE 16.7	10.6			
2	21.1	5.3	13.2	4.8	8.2	89	48						2.8	SE	NNE 13.0	10.1			
3	20.7	8.2	14.5	3.5	9.5	84	31						7.8	N	NNW 14.8	10.3			
4	19.4	4.2	11.8	6.2	6.8	79	35						9.9	E	ENE 16.7	9.8			
5	21.9	4.9	13.4	4.6	8.4	82	45						7.8	E	S 20.4	10.2			
6	21.3	10.1	15.7	2.3	10.7	81	56		0.3		0.3		13.9	S	S 20.4	6.4			
7	11.2	6.8	11.5	6.5	6.5	90	58		3.8		3.8		14.5	NW	SSW 24.1	0.0			
8	9.4	5.9	7.7	10.3	2.7	87	60		TR	TR	TR		9.3	NNE	NNE 16.7	0.0			
9	7.2	0.1	3.7	14.3		89	76		22.9	5.1	27.7		20.8	NNE	W 42.6	0.0			
10	7.1	-0.4	3.4	14.6		82	44		TR	TR	TR		25.6	WNW	WNW 42.6	2.5			
11	10.8	-1.1	4.9	13.1		80	39						8.1	WNW	WNW 16.7	9.5			
12	14.0	-1.1	6.5	11.5	1.5	80	38		0.5		0.5		8.3	S	S 16.7	7.3			
13	19.4	8.2	13.8	4.2	8.8	72	43		TR		TR		13.2	SW	SW 25.9	4.9			
14	8.8	-0.7	4.1	13.9		71	36		TR		TR		25.9	WNW	NW 37.0	4.9			
15	17.4	0.9	9.2	8.8	4.2	73	41		TR	TR	TR		15.2	SW	SW 27.8	4.6			
16	7.6	1.9	4.8	13.2		77	48						20.4	WNW	WNW 29.6	5.1			
17	4.4	-2.8	0.8	17.2		78	44			0.5	0.5		11.0	WNW	W 18.5	5.5			
18	4.5	-2.3	1.1	16.9		73	38			TR	TR	TR	19.2	NW	NW 31.5	6.8			
19	4.6	-4.6	0.0	18.0		75	38			TR	TR		5.6	NE	NE 16.7	4.4			
20	4.9	-0.7	2.1	15.9		87	76		9.4	3.8	13.2	1	21.0	ENE	ENE 37.0	0.0			
21	7.7	0.3	4.0	14.0		86	53		21.3		21.3		26.1	W	WNW 35.2	1.3			
22	7.4	0.4	3.9	14.1		80	39						21.5	WSW	WSW 37.0	7.2			
23	6.5	-3.5	1.5	16.5		79	38						18.4	WSW	WNW 35.2	8.3			
24	2.6	-4.8	-1.1	19.1		78	57			0.3	0.3		11.7	ENE	NE 22.2	0.0			
25	5.7	-0.6	2.6	15.4		86	53			0.8	0.8		12.0	NE	NNE 18.5	0.0			
26	2.4	-2.9	-0.3	18.3		6	40					TR	17.6	N	N 29.6	9.1			
27	3.6	-5.2	-0.8	18.8		76	42						15.9	WNW	WNW 25.9	5.0			
28	5.4	-6.5	-0.6	18.6		75	48			TR	TR		17.3	SW	WSW 26.0	0.2			
29	12.5	2.8	7.7	10.3	2.7	83	50						19.1	SW	SW 29.6	7.6			
30	10.4	2.9	6.7	11.3	1.7	89	60		1.8		1.8		5.6	SW	WSW 13.0	0.5			
31	5.9	-1.1	2.4	15.6		93	66		10.7	0.5	11.2		18.2	NNW	N 27.8	0.0			
MEAN MOYENNE	10.7	0.9	5.8	378.3	78.2	81	48	0	70.7	11.0	81.4		14.6	WNW DOMINANTE	N 42.6	152.1			
NORMAL NORMALE	13.8	3.0	8.7	288	98.3			0	63.8	1.8	65.8		15.0	SW		198			
DEGREE-DAY SUMMARY - SOMMAIRE DE DEGRÉS-JOURS																			
BELOW 18°C AU-DESSUS DE 18°C		THIS YEAR ANNEE EN COURS	PREVIOUS YEAR ANNÉE PRÉCÉDENTE	NORMAL NORMALE	ABOVE 5°C AU-DESSUS DE 5°C		THIS YEAR ANNEE EN COURS	PREVIOUS YEAR ANNÉE PRÉCÉDENTE	NORMAL NORMALE	DAYS WITH TOTAL PRECIPITATION JOURS AVEC PRÉCIPITATIONS TOTALES									
										0.2 OR MOINS	1.0 OR PLUS	2.0 OR PLUS	10.0 OR PLUS	10.0 OR PLUS	0.2 OR MOINS	1.0 OR PLUS	2.0 OR PLUS	10.0 OR PLUS	10.0 OR PLUS
TOTAL FOR MONTH TOTAL DU MOIS		378.3	291.0	288	TOTAL FOR MONTH TOTAL DU MOIS		78.2	126.3	98.3	0	6	5	4	0	6	2	2	0	0
ACCUMULATED SINCE JULY 1 ACCUMULÉ DEPUIS LE 1 ^{er} JUILLET		567.3	431.7	466.9	ACCUMULATED SINCE APRIL 1 ACCUMULÉ DEPUIS LE 1 ^{er} AVRIL		1981.9	2194.2	1853.9	10	6	5	4	0	6	2	2	0	0

UDC 561.508.11 713.84

indicates more than one occurrence of same speed but not necessarily the same direction.
indique qu'il y a eu plus d'une fois des vents de la même vitesse qui ont soufflé de directions différentes.

MONTHLY METEOROLOGICAL SUMMARY SOMMAIRE MÉTÉOROLOGIQUE MENSUEL

MONTH/MOIS NOVEMBER/NOVEMBRE

1976

AT/A OTTAWA INTERNATIONAL AIRPORT/AÉROPORT INTERNATIONAL D'OTTAWA

LAT 45 - 19 'N		LONG 75 - 40 'W		ELEVATION 374		FEET (ASL) 374		STANDARD TIME USED		EASTERN											
				ALTITUDE		PIEDS (MER)		HEURE NORMALE UTILISÉE.		DE L'EST											
DATE	TEMPERATURE			DEGREE-DAYS		REL. HUMIDITY		THUNDERSTORM	PRECIPITATION			SNOW ON GROUND	WIND			BRIGHT SUNSHINE					
	TEMPERATURE			DEGRÉS-JOURS		HUMIDITÉ REL.			PRÉCIPITATIONS				VENT								
	MAXIMUM	MINIMUM	MEAN	HEATING	GROWING	MAXIMUM	MINIMUM		RAINFALL	SNOWFALL	TOTAL PRECIP.		AVERAGE SPEED	PREVAILING	MAXIMUM SPEED						
	MAXIMALE	MINIMALE	MOYENNE	DE CHAUFFEE	DE CROISSANCE	MAXIMALE	MINIMALE		PLUIE (HAUTEUR)	NEIGE (HAUTEUR)	PRÉCIP. TOTALE	NEIGE AU SOL	VITESSE MOYENNE	DIRECTION	AND DIRECTION	ET DIRECTION	INSOLATION				
	°C	°C	°C	BASE 18°C	BASE 5°C	%	%		mm	cm	mm	cm	km/h	DOMINANTE	MAXIMALE	HEURES					
1	2.4	-4.1	-0.9	18.9		67	39					TR	20.0	NNW	NNW 33.3	9.4					
2	5.7	-7.3	-0.8	18.8		85	45		0.5	0.3	0.8		9.7	NNW	SSW 18.5	5.9					
3	11.7	0.6	6.2	11.8	1.2	90	44		TR		TR		9.1	SSW	S 18.5	4.4					
4	6.0	1.7	3.9	14.1		93	85		0.5		0.5		11.5	ESE	ESE 20.4	0.0					
5	6.1	-1.0	2.6	15.4		92	54						5.8	SSW	S 14.8	3.5					
6	5.1	-1.7	1.7	16.3		86	65		TR	0.3	0.3		16.9	WSW	NW 37.0	1.3					
7	2.4	-3.6	-0.6	18.6		80	51						15.7	NNW	NW 25.9	4.7					
8	-0.5	-7.9	-4.2	22.2		69	43			TR	TR	TR	18.3	NW	NNW 24.1	4.8					
9	-0.6	-11.3	-6.0	24.0		89	48			4.1	3.8		9.5	W	E 16.7	3.3					
10	-0.3	-5.0	-2.7	20.7		90	70			2.3	2.0	5	11.1	ESE	W*24.1	0.1					
11	-2.3	-10.6	-6.5	24.5		79	56					8	15.4	W	W 29.6	7.3					
12	-0.6	-9.2	-4.9	22.9		82	59			TR	TR	5	10.2	W	W 22.2	4.6					
13	4.3	-8.4	-2.1	20.1		79	58					5	13.0	WSW	WSW*24.1	6.3					
14	4.2	-1.3	1.5	16.5		81	64					5	16.3	WSW	WSW 25.9	0.1					
15	2.5	-2.2	0.2	17.8		90	76			TR	TR	TR	5.4	NE	SW* 9.3	0.0					
16	7.4	-2.8	2.3	15.7		88	46					TR	10.2	SW	W 27.8	7.3					
17	7.0	-1.8	2.6	15.4		83	53		TR		TR	TR	14.8	SW	SW 27.8	5.5					
18	5.3	0.4	2.9	15.1		92	62		TR	0.5	0.5		15.0	SW	NW 25.9	1.4					
19	7.6	-1.3	3.2	14.8		92	60		0.5	1.5	2.3		19.4	WSW	WSW 31.5	0.9					
20	0.0	-7.6	-3.8	21.8		67	40			TR	TR		22.1	NNW	NNW 29.6	5.5					
21	-1.7	-8.7	-5.2	23.2		80	53			TR	TR		19.2	W	W*27.8	2.7					
22	0.4	-7.4	-3.5	21.5		73	57			TR	TR	TR	19.4	W	W*27.8	0.1					
23	0.6	-5.6	-2.5	20.5		76	49			TR	TR		21.0	W	WNW 33.3	2.6					
24	1.1	-4.3	-1.6	19.6		88	65			1.0	0.8		4.4	WSW	SW 9.3	0.0					
25	1.1	-5.7	-2.3	20.3		100	62			2.5	2.5	TR	12.0	SW	SW 18.5	2.1					
26	9.3	-1.2	4.1	13.9		100	87		12.7	2.5	15.2	3	15.4	SSW	SSW 22.2	0.0					
27	13.2	1.4	7.3	10.7	2.3	100	68		5.1		5.1		18.7	S	WSW 35.2	1.2					
28	3.8	-2.6	0.6	17.4		93	63		0.3	0.8	1.0		9.5	WSW	NW 20.4	0.0					
29	-2.2	-10.3	-6.3	24.3		86	51			0.5	0.3	TR	15.5	NW	W 33.3	0.5					
30	-3.4	-12.6	-8.0	26.0		83	39					TR	16.7	WSW	WSW 27.8	5.2					
MEAN MOYENNE	3.2	-4.7	-0.8	562.8	3.5	85	57	TOTAL	19.6	16.3	35.1		14.0	WSW	NW 37.0	90.7					
NORMAL NORMALE	5.1	-2.4	1.4	496	18				56.6	22.4	78.5		18.4	E		76.2					
DEGREE-DAY SUMMARY - SOMMAIRE DE DEGRÉS-JOURS												DAYS WITH TOTAL PRECIPITATION: JOURS AVEC PRÉCIPITATIONS TOTALES					DAYS WITH SNOWFALL: JOURS AVEC CHUTE DE NEIGE				
BELOW 18°C	THIS YEAR	PREVIOUS YEAR	NORMAL	ABOVE 5°C	THIS YEAR	PREVIOUS YEAR	NORMAL	0.2	1.0	2.0	10.0	50.0	0.2	1.0	2.0	10.0	50.0				
AU-DESSUS DE 18°C	ANNEE EN COURS	ANNEE PRECEDENTE	NORMALE	AU-DESSUS DE 5°C	ANNEE EN COURS	ANNEE PRECEDENTE	NORMALE	ou PLUS	ou PLUS	ou PLUS	ou PLUS	ou PLUS	ou PLUS	ou PLUS	ou PLUS	ou PLUS	ou PLUS				
TOTAL FOR MONTH	562.8	428.0	496	TOTAL FOR MONTH	3.5	60.5	18	13	7	6	1	0	11	6	4	0	0				
ACCUMULATED SINCE JULY 1	1130.1	880.2	962.9	ACCUMULATED SINCE JULY 1	1985.4	2254.7	1871.9														
DEPUIS LE 1 ^{er} JUILLET				DEPUIS LE 1 ^{er} JUILLET																	

UDC 561.508.1: 713.84

* Indicates more than one of the same speed.
Indique plusieurs vitesses identiques.

MONTHLY METEOROLOGICAL SUMMARY SOMMAIRE MÉTÉOROLOGIQUE MENSUEL

MONTH/MOIS DECEMBER/DECEMBRE

1976

AT/A OTTAWA INTERNATIONAL AIRPORT/AÉROPORT INTERNATIONAL D'OTTAWA

LAT. 45° 19' N		LONG. 75° 40' W		ELEVATION ALTITUDE: 374		FEET (ASL) PIEDS (MER)		STANDARD TIME USED: HOURS NORMALE UTILISÉE		EASTERN DE L'EST													
DATE	TEMPERATURE TEMPÉRATURE			DEGREE-DAYS DEGRÉS-JOURS		REL. HUMIDITY HUMIDITÉ REL.		THUNDERSTORM ORAGE	PRECIPITATION PRÉCIPITATIONS			SNOW ON GROUND NEIGE AU SOL	WIND VENT			BRIGHT SUNSHINE INSOLATION EFFECTIVE							
	MAXIMUM MAXIMALE	MINIMUM MINIMALE	MEAN MOYENNE	HEATING DE CHAUFFE	GROWING DE CROISSANCE	MAXIMUM MAXIMALE	MINIMUM MINIMALE		RAINFALL PLUIE (HAUTEUR)	SNOWFALL NEIGE (HAUTEUR)	TOTAL PRECIP PRÉCIP TOTALE		AVERAGE SPEED VITESSE MOYENNE	PREVAILING DIRECTION DIRECTION DOMINANTE	MAXIMUM SPEED AND DIRECTION VITESSE MAXIMALE ET DIRECTION								
																	°C	°C	°C	BASE °F/°C	BASE °F/°C	%	%
1	-2.7	-13.1	-7.9	25.9		69	48			0.3	0.3		17.3	S	SSW 25.9	4.1							
2	-2.3	-21.6	-12.0	30.0		82	34			2.5	1.5	TR	23.9	WNW	WNW 37.0	0.1							
3	-12.1	-25.6	-18.9	36.9		55	30			TR	TR	3	16.3	WSW	WNW 27.8	8.0							
4	-10.6	-17.2	-13.9	31.9		62	36			TR	TR	3	13.0	NE	ENE 22.2	0.2							
5	-4.5	-15.0	-9.8	27.8		79	50			1.8	1.3	5	6.0	SSE	S 11.1	0.0							
6	-2.3	-13.4	-7.9	25.9		96	52			0.3	0.3	5	11.5	E	E 20.4	0.0							
7	1.5	-20.6	-9.6	27.6		100	61			8.6	8.6	10	16.1	WNW	WNW 27.8	2.2							
8	-14.6	-22.9	-18.8	36.8		75	58			TR	TR	8	9.7	NW	WNW 24.1	6.6							
9	-16.7	-26.3	-21.5	39.5		74	50			TR	TR	8	13.9	W	WNW 29.6	7.8							
10	3.4	-25.0	-10.8	28.8		90	61		TR	0.3	0.3	8	14.8	E	E 22.2	0.0							
11	2.2	-13.4	-5.6	23.6		98	38			0.6	0.6	8	17.6	WNW	WNW 46.3	5.2							
12	-2.4	-10.4	-6.4	24.4		97	76			5.8	4.1	8	7.6	WSW	NW 20.4	0.0							
13	-10.4	-24.6	-17.5	35.5		76	51			TR	TR	15	21.5	WNW	NW 35.2	7.5							
14	3.1	-24.2	-10.6	28.6		85	53			1.5	0.8	15	16.3	SW	SSW 27.8	0.0							
15	2.3	-2.9	-0.3	18.3		82	59			TR	TR	10	13.9	SW	SW 27.8	0.0							
16	-2.5	-6.4	-4.5	22.5		89	72			TR	TR	10	11.8	ENE	ENE 16.7	0.5							
17	-1.4	-6.9	-4.2	22.2		94	73			1.3	1.0	10	5.0	WNW	NW 22.2	0.9							
18	-2.9	-16.6	-9.8	27.8		82	50			TR	TR	10	15.4	WNW	NW 27.8	7.2							
19	-8.0	-14.5	-11.3	29.3		74	52					10	9.3	ENE	NE 14.8	0.3							
20	-3.3	-11.9	-7.6	25.6		94	64		3.8	4.1	10.7	8	15.0	ENE	WNW 24.1	0.0							
21	-11.6	-19.3	-15.5	33.5		70	53			TR	TR	10	27.8	WNW	WNW 35.2	1.2							
22	-13.9	-21.7	-17.8	35.8		81	54			1.0	1.0	10	12.4	ENE	WNW 31.5	1.4							
23	-2.7	-15.1	-8.9	26.9		90	75			2.3	1.3	15	11.1	S	S*18.5	1.5							
24	-8.1	-18.7	-13.4	31.4		89	56			0.5	0.3	15	12.6	W	NW 27.8	5.6							
25	-1.0	-13.4	-7.2	25.2		94	60			1.5	1.5	15	7.1	S	NE*11.1	0.0							
26	-2.3	-16.0	-9.2	27.2		98	61			4.8	4.8	18	11.1	WNW	WNW*18.5	4.6							
27	-15.9	-23.3	-19.6	37.6		65	49					18	17.3	WNW	WNW 25.9	7.9							
28	-16.7	-25.1	-20.9	38.9		70	46			TR	TR	18	9.1	ENE	ENE 20.4	5.2							
29	-13.3	-16.9	-15.1	33.1		81	59			1.5	1.0	18	9.3	ENE	ENE 20.4	0.1							
30	-12.8	-19.6	-16.2	34.2		78	58			0.3	TR	18	13.0	W	W 25.9	7.9							
31	-17.7	-24.2	-21.0	39.0		71	46			1.0	1.3	18	14.8	W	WNW 22.2	7.7							
MEAN MOYENNE	-6.5	-17.6	-12.1	931.7	TOTAL	82	52	TOTAL	TOTAL	3.8	39.9	TOTAL	13.6	W	WNW 46.3	93.9							
NORMAL NORMALE	-3.7	-11.6	-7.7	791					28.7	51.6	77.0		15.3	W		78							
DEGREE-DAY SUMMARY - SOMMAIRE DE DEGRÉS-JOURS																							
BELOW 10°C AU-DESSUS DE 10°C								ABOVE 5°C AU-DESSUS DE 5°C								DAYS WITH TOTAL PRECIPITATION: JOURS AVEC PRÉCIPITATIONS TOTALES:							
THIS YEAR ANNEE EN COURS								PREVIOUS YEAR ANNEE PRECEDENTE								DAYS WITH SNOWFALL: JOURS AVEC CHUTE DE NEIGE:							
NORMAL NORMALE								NORMAL NORMALE								NORMAL NORMALE							
TOTAL FOR MONTH TOTAL DU MOIS								TOTAL FOR MONTH TOTAL DU MOIS								TOTAL FOR MONTH TOTAL DU MOIS							
931.7								863.8								791							
2061.8								1723.5								1732							
2254.7								1872								18							
4								1								0							
19								13								6							
0								0								0							

UDC 61.506.11 713.R4

*Indicates more than one of the same speed.
Indique plusieurs vitesses identiques.

MONTHLY METEOROLOGICAL SUMMARY

SOMMAIRE MÉTÉOROLOGIQUE MENSUEL

MONTH/MOIS JANUARY/JANVIER

1977

AT/A OTTAWA INTERNATIONAL AIRPORT/AÉROPORT INTERNATIONAL D'OTTAWA

LAT 45° 19' N		LONG 75° 40' W		ELEVATION ALTITUDE 374		FEET (ASL) PIEDS (MER)		STANDARD TIME USED EASTERN		MEURE NORMALE UTILISÉE DE L'EST									
DATE	TEMPERATURE TEMPERATURE			DEGREE-DAYS DEGRÉS-JOURS		REL HUMIDITY HUMIDITÉ REL		THUNDERSTORM ORAGE	PRECIPITATION PRÉCIPITATIONS			SNOW ON GROUND NEIGE AU SOL	WIND VENT			BRIGHT SUNSHINE INSOLATION EFFECTIVE			
	MAXIMUM MAXIMALE	MINIMUM MINIMALE	MEAN MOYENNE	HEATING DE CHAUFFE	GROWING DE CROISSANCE	MAXIMUM MAXIMALE	MINIMUM MINIMALE		RAINFALL PLUIE (HAUTEUR)	SNOWFALL NEIGE (HAUTEUR)	TOTAL PRECIP PRÉCIP TOTALE		AVERAGE SPEED VITESSE MOYENNE	PREVAILING DIRECTION DIRECTION DOMINANTE	MAXIMUM SPEED AND DIRECTION VITESSE MAXIMALE ET DIRECTION				
	°C	°C	°C	BASE 18°C	BASE 5°C	%	%		mm	cm	mm		cm	km/h			km/h	HOURS HEURES	
1	-7.0	-18.0	-12.5	30.5		81	69			3.0	2.4	22	20.6	WNW	WNW 25.9	2.8			
2	-6.4	-11.5	-9.0	27.0		87	67			TR	TR	22	11.1	W	WNW 25.9	7.1			
3	-4.0	-9.4	-6.7	24.7		87	77			TR	TR	22	3.0	SVRL	E 9.3	0.0			
4	-2.6	-6.5	-4.6	22.6		90	74			4.2	3.0	24	9.9	W	W 22.2	2.0			
5	-5.1	-17.2	-11.7	29.7		85	64			TR	TR	27	8.9	WNW	WNW 16.7	7.9			
6	-13.5	-19.7	-16.6	34.6		76	60			0.2	0.2	26	11.3	E	ENE*14.8	1.6			
7	-6.1	-17.2	-11.7	29.7		86	63			3.4	3.1	27	13.7	SVRL	WNW*27.8	0.0			
8	-15.8	-22.2	-19.0	37.0		73	52			0.4	0.3	29	10.6	WNW	WNW 24.1	7.0			
9	-15.8	-21.8	-18.8	36.8		76	49			0.2	0.2	30	9.7	WNW	WNW 24.1	7.4			
10	-12.0	-20.0	-16.0	34.0		82	56			23.2	21.1	31	27.8	ENE	ENE 46.3	0.0			
11	-14.6	-20.6	-17.6	35.6		79	50			2.4	2.1	53	20.0	W	W*27.8	7.6			
12	-15.4	-22.7	-19.1	37.1		65	45					54	11.1	W	W 20.4	6.8			
13	-15.2	-26.4	-20.8	38.8		72	51			0.6	0.6	52	4.1	E	W 11.1	5.9			
14	-12.9	-17.9	-15.4	33.4		76	69			2.2	1.8	53	13.4	ENE	ENE 22.2	1.6			
15	-12.6	-15.7	-14.2	32.2		79	67			0.2	TR	54	9.3	E	ENE*18.5	0.1			
16	-12.6	-21.1	-16.9	34.9		75	59			TR	TR	54	8.7	SVRL	WNW*18.5	7.2			
17	-20.8	-27.9	-24.4	42.4		67	45			TR	TR	54	12.0	WNW	WNW*20.4	7.2			
18	-10.9	-27.7	-19.3	37.3		82	52			1.6	1.2	53	18.9	W	W 27.8	1.4			
19	-6.4	-11.3	-8.9	26.9		79	60			TR	TR	55	19.8	WNW	WNW*25.9	4.9			
20	-4.6	-12.6	-8.6	26.6		77	57			0.2	TR	54	17.8	WNW	W 27.8	7.8			
21	-7.0	-16.4	-12.2	30.2		81	52			TR	TR	53	11.1	WNW	WNW 20.4	1.5			
22	-11.2	-18.8	-15.0	33.0		61	52			TR	TR	53	18.5	NW	WNW*25.9	1.8			
23	-11.6	-19.8	-15.7	33.7		73	61			TR	TR	53	8.3	SVRL	WNW*14.8	2.7			
24	-4.5	-12.4	-8.5	26.5		80	65		TR	1.0	0.8	52	7.3	E	NE *11.1	0.0			
25	-4.0	-9.0	-6.5	24.5		83	73			5.0	4.4	57	4.8	ENE	ENE*13.0	0.0			
26	-6.5	-11.1	-8.8	26.8		81	59			TR	TR	56	10.0	W	W 22.2	5.3			
27	-6.2	-16.2	-11.2	29.2		77	40			0.6	0.6	53	23.3	SW	WSW 37.0	5.1			
28	-11.4	-19.0	-15.2	33.2		77	55			8.0	7.4	52	20.6	WSW	SW 33.3	0.0			
29	-14.6	-17.6	-16.1	34.1		73	52			TR	TR	60	27.6	WSW	WSW*33.3	0.2			
30	-11.1	-18.3	-14.7	32.7		62	50			TR	TR	60	29.5	WSW	WSW*37.0	7.2			
31	-8.6	-14.4	-11.5	29.5		66	57			TR	TR	60	35.0	SW	WSW 44.4	2.7			
MEAN MOYENNE	-10.1	-17.4	-13.8	985.2	nil	77	58	nil	TR	56.4	49.2		14.8	WNW	WSW 44.4	112.8			
NORMAL NORMALE	-6.4	-15.6	-10.9	894	nil			nil	14.0	48.5	59.9		19.1	W		96			
DEGREE-DAY SUMMARY - SOMMAIRE DE DEGRÉS-JOURS										DAYS WITH TOTAL PRECIPITATION JOURS AVEC PRÉCIPITATIONS TOTALES					DAYS WITH SNOWFALL JOURS AVEC CHÛTE DE NEIGE				
BELOW 18°C AU-DESSOUS DE 18°C		THIS YEAR ANNÉE EN COURS	PREVIOUS YEAR ANNÉE PRÉCÉDENTE	NORMAL NORMALE	ABOVE 5°C AU-DESSUS DE 5°C		THIS YEAR ANNÉE EN COURS	PREVIOUS YEAR ANNÉE PRÉCÉDENTE	NORMAL NORMALE	0.2 OR MORE mm	1.0 OR MORE mm	2.0 OR MORE mm	10.0 OR MORE mm	50.0 OR MORE mm	0.2 OR MORE cm	1.0 OR MORE cm	2.0 OR MORE cm	10.0 OR MORE cm	50.0 OR MORE cm
TOTAL FOR MONTH TOTAL DU MOIS		985.2	1010.1	894	TOTAL FOR MONTH TOTAL DU MOIS		nil	nil	nil	15	9	7	1	0	17	10	8	1	0
ACCUMULATED SINCE JULY 1 ACCUMULÉE DEPUIS LE 1 ^{er} JUILLET		3047.0	2754.1	2523	ACCUMULATED SINCE APRIL 1 ACCUMULÉE DEPUIS LE 1 ^{er} AVRIL		1985.4	2254.7	1872										

UDC 961.508.1 (713.84)

SVRL signifie plusieurs
SVRL stands for several

*Indicates more than one of the same speed
*Indique plusieurs vitesses identiques.

Prix/Price (\$1.00)

MONTHLY METEOROLOGICAL SUMMARY SOMMAIRE MÉTÉOROLOGIQUE MENSUEL

MONTH/MOIS FEBRUARY/FÉVRIER

1977

AT/A OTTAWA INTERNATIONAL AIRPORT/AÉROPORT INTERNATIONAL D'OTTAWA

LAT 45° 19' N			LONG 75° 40' W			ELEVATION: 374 FEET (ASL) ALTITUDE 374 PIEDS (MER)		STANDARD TIME USED: EASTERN HEURE NORMALE UTILISÉE: EST								
DATE	TEMPERATURE			DEGREE-DAYS		REL HUMIDITY		THUNDERSTORM ORAGE	PRECIPITATION			SNOW ON GROUND NEIGE AU SOL	WIND			BRIGHT SUNSHINE INSOLATION EFFECTIVE
	TEMPÉRATURE			DEGRÉS JOURS		HUMIDITÉ REL.			PRÉCIPITATIONS				VENT			
	MAXIMUM MAXIMALE	MINIMUM MINIMALE	MEAN MOYENNE	HEATING DE CHAUFFE	GROWING DE CROISSANCE	MAXIMUM MAXIMALE	MINIMUM MINIMALE		RAINFALL PLUIE (HAUTEUR)	SNOWFALL NEIGE (HAUTEUR)	TOTAL PRECIP PRÉCIP TOTALE		AVERAGE SPEED VITESSE MOYENNE	PREVAILING DIRECTION DOMINANTE	MAXIMUM SPEED AND DIRECTION VITESSE MAXIMALE ET DIRECTION	
	°C	°C	°C	BASE 18.0°C	BASE 5.0°C	%	%	mm	cm	mm	cm	km/h		km/h	HOURS HEURES	
1	-9.0	-18.8	-13.9	31.9		74	44			TR	TR	60	25.9	WNW	WNW 37.0	8.2
2	-9.2	-20.9	-15.1	33.1		71	48					58	11.1	W	W 22.2	8.2
3	-2.7	-11.5	-7.1	25.1		81	67			6.2	6.2	56	15.7	S	SSW 24.1	0.0
4	-4.6	-9.6	-7.1	25.1		75	57			TR	0.2	59	9.9	W	W 22.2	8.5
5	-8.3	-15.6	-12.0	30.0		77	63			3.8	3.2	58	16.1	WNW	WNW 29.6	0.0
6																
7	-12.8	-20.9	-16.9	34.9		67	53			0.4	0.4	60	26.8	W	WNW 31.5	1.9
8	-9.4	-16.3	-12.9	30.9		60	46					59	23.7	WNW	WNW 29.6	5.0
9	-5.5	-18.5	-12.0	30.0		72	46			TR	TR	55	13.2	SVRL	WNW 25.9	8.8
10	-4.4	-8.8	-6.6	24.6		77	66			6.4	6.8	54	17.6	SSW	SSW 27.8	0.0
	-3.0	-11.4	-7.2	25.2		83	60			TR	TR	57	5.4	SW	WNW 11.1	3.9
11																
12	1.1	-5.5	-2.2	20.2		77	67		0.2	TR	0.2	46	11.7	E	W 24.1	0.0
13	2.3	-0.3	1.0	17.0		96	70		TR	TR	TR	35	9.1	S/W	W 22.2	0.0
14	3.5	-3.5	0.0	18.0		99	64		TR	4.0	4.2	32	22.2	E	E 38.9	1.0
15	-1.6	-9.6	-5.6	23.6		95	63			0.8	0.7	33	18.4	SW	WSW 27.8	1.6
	-9.1	-17.9	-13.5	31.5		93	53			TR	TR	31	19.8	WNW	WNW 27.8	8.1
16																
17	-14.4	-21.8	-18.1	36.1		65	49			TR	TR	31	15.2	WNW	WNW 22.2	6.4
18	-8.2	-19.6	-13.9	31.9		74	51			TR	TR	31	14.5	W	W 24.1	7.0
19	-3.9	-16.6	-10.3	28.3		86	50			0.3	0.3	30	10.2	S	S 22.2	3.1
20	-1.8	-7.1	-4.5	22.5		96	73			3.0	3.0	32	6.0	WNW	WNW 13.0	1.2
	-2.5	-9.1	-5.8	23.8		92	56					32	11.6	NE	W 22.2	0.4
21																
22	-6.9	-14.2	-10.6	28.6		89	39					30	20.4	WNW	W 33.3	9.1
23	-2.6	-14.1	-8.4	26.4		86	50			0.4	0.4	30	12.6	SVRL	SSW 25.9	0.6
24	-7.1	-14.4	-10.8	28.8		76	31		TR	1.4	1.2	30	20.8	ENE	E 37.0	4.3
25	-0.1	-13.0	-6.6	24.6		97	73		20.0	TR	20.0	31	32.9	E	E 44.4	0.1
	0.8	-2.0	-0.6	18.6		100	80		0.4	2.6	3.0	30	15.2	E/W	ESE 27.8	0.0
26																
27	-0.2	-2.7	-1.5	19.5		85	69			TR	TR	31	13.5	WSW	WSW 25.9	0.0
28	5.0	-3.0	1.0	17.0		94	63		TR	0.6	0.6	30	24.5	SVRL	E 37.0	0.8
29	-1.8	-8.9	-5.4	23.4		73	51			TR	TR	24	27.8	WSW	WSW 37.0	2.9
30																
31																
MEAN MOYENNE	-4.2	-12.0	-8.3	730.6	nil	100	31	nil	20.6	29.9	50.4		16.9	WNW	E 44.4	90.7
NORMAL NORMALE	-4.7	-14.4	-9.5	773	nil			nil	11.2	47.8	56.9		18.5	W		115.3

DEGREE-DAY SUMMARY - SOMMAIRE DE DEGRÉS-JOURS								DAYS WITH TOTAL PRECIPITATION JOURS AVEC PRÉCIPITATIONS TOTALES					DAYS WITH SIGNIFICANT SNOWFALL JOURS AVEC CHÔTE DE NEIGE				
BELOW 18°C AU-DESSOUS DE 18°C	THIS YEAR ANNÉE EN COURS	PREVIOUS YEAR ANNÉE PRÉCÉDENTE	NORMAL NORMALE	ABOVE 5°C AU-DESSUS DE 5°C	THIS YEAR ANNÉE EN COURS	PREVIOUS YEAR ANNÉE PRÉCÉDENTE	NORMAL NORMALE	0.2 OR MORE 0.2 PLUS	1.0 OR MORE 1.0 PLUS	2.0 OR MORE 2.0 PLUS	10.0 OR MORE 10.0 PLUS	50.0 OR MORE 50.0 PLUS	0.2 OR MORE 0.2 PLUS	1.0 OR MORE 1.0 PLUS	2.0 OR MORE 2.0 PLUS	10.0 OR MORE 10.0 PLUS	50.0 OR MORE 50.0 PLUS
TOTAL FOR MONTH TOTAL DU MOIS	730.6	726.3	773	TOTAL FOR MONTH TOTAL DU MOIS	nil	nil	nil	15	8	7	1	0	12	7	6	0	0
ACCUMULATED SINCE JULY 1 ACCUMULÉE DEPUIS LE 1 ^{er} JUILLET	3777.6	3480.4	3399	ACCUMULATED SINCE APRIL 1 ACCUMULÉE DEPUIS LE 1 ^{er} AVRIL	1985.4	2254.7	1872										

UDC 541.585:1 713.84

SVRL signifie plusieurs
SVRL stands for several

* Indique plusieurs vitesses identiques.
* Indicates more than one of the same speed

Prix/Price (\$1.00)

MONTHLY METEOROLOGICAL SUMMARY SOMMAIRE MÉTÉOROLOGIQUE MENSUEL

MONTH/MOIS March/Mars

1977

AT/A Ottawa International Airport/Aéroport International d'Ottawa

LAT 45 19 N LONG 75 40 W			ELEVATION ALTITUDE 374		FEET (ASL) PIEDS (MER)		STANDARD TIME USED: Eastern de l'est		HEURE NORMALE UTILISÉE							
DATE	TEMPERATURE TEMPERATURE			DEGREE DAYS DEGRÉS JOURS		REL HUMIDITY HUMIDITÉ REL		THUNDERSTORM ORAGE	PRECIPITATION PRÉCIPITATIONS			SNOW ON GROUND NEIGE AU SOL	WIND VENT			BRIGHT SUNSHINE INSOLATION EFFECTIVE
	MAXIMUM MAXIMALE	MINIMUM MINIMALE	MEAN MOYENNE	HEATING DE CHAUFFE	GROWING DE CROISSANCE	MAXIMUM MAXIMALE	MINIMUM MINIMALE		RAINFALL PLUIE (HAUTEUR)	SNOWFALL NEIGE (HAUTEUR)	TOTAL PRECIP PRECIP TOTALE		AVERAGE SPEED VITESSE MOYENNE	PREVAILING DIRECTION DIRECTION DOMINANTE	MAXIMUM SPEED AND DIRECTION VITESSE MAXIMALE ET DIRECTION	
				BASE 18.0°C	BASE 5.0°C				mm	cm	mm	cm	km/h		km/h	HOURS HEURES
1	-4.3	-10.0	-7.2	25.2		77	55					24	17.8	WSW	WSW 31.5	5.9
2	1.1	-10.5	-4.7	22.7		82	58					24	15.4	W	W 24.1	6.3
3	2.1	-6.1	-2.0	20.0		81	48					24	9.9	WSW	W 18.5	9.7
4	3.5	-4.9	-0.7	18.7		97	59		9.4	0.5	9.9	24	25.0	SVRL	E* 37.0	0.0
5	4.0	-0.3	1.9	16.1		95	73		0.4	1.2	1.4	23	22.9	SSW	W 33.3	0.1
6	1.9	-1.0	0.5	17.5		80	63					22	15.4	W	W 24.1	0.1
7	1.6	-4.3	-1.4	19.4		86	63					20	9.5	W	W* 14.8	0.4
8	4.1	-5.6	-0.8	18.8		81	64					20	13.0	SSW	SSW 22.2	7.3
9	9.9	-1.1	4.4	13.6		86	48					19	11.1	SSW	SSW 25.9	4.7
10	12.7	1.9	7.3	10.7	2.3	87	51					16	7.6	SW	SW 16.7	9.1
11	10.5	-0.9	4.7	13.3		96	59					10	5.4	E	ENE 13.0	9.2
12	12.1	0.7	6.4	11.6	1.4	80	53		TR		TR	6	9.3	NE	NE* 14.8	5.2
13	5.9	3.1	4.5	13.5		100	88		38.1		38.1	4	13.7	E	E* 25.9	0.0
14	5.6	2.7	4.2	13.8		100	94		12.5		12.5	TR	6.2	WNW	WNW 14.8	0.0
15	4.3	1.1	2.7	15.3		100	94		TR		TR	TR	3.9	E	E 13.0	0.0
16	7.3	0.1	-3.7	14.3		100	47		1.9		1.9	TR	20.0	W	W 33.3	2.0
17	0.7	-5.1	-2.2	20.2		75	42			TR	TR	TR	24.5	WNW	W 37.0	9.5
18	-1.6	-6.0	-3.8	21.8		58	37					TR	13.0	ENE	ENE* 24.1	0.0
19	2.2	-6.4	-2.1	20.1		80	32					TR	16.3	SVRL	WNW 27.8	11.1
20	4.9	-6.3	-0.7	18.7		95	38			1.1	1.1	TR	8.5	SVRL	E* 22.2	4.6
21	5.2	-2.8	1.2	16.8		94	59		0.2	TR	0.2	1	9.1	W	WSW 20.4	3.7
22	-0.3	-3.8	-2.1	20.1		98	65			21.5	20.8	TR	25.0	ENE	E* 33.3	0.0
23	2.2	-5.3	-1.6	19.6		95	78			2.9	2.7	24	13.2	N	N* 25.9	1.5
24	-0.2	-7.6	-3.9	21.9		93	47					25	24.3	WNW	WNW 37.0	8.7
25	1.4	-11.2	-4.9	22.9		63	42					21	28.3	NNW	NNW 38.9	8.2
26	3.7	-6.5	-1.4	19.4		76	52					18	15.0	NNW	NNW 25.9	11.4
27	7.4	-6.3	0.6	17.4		93	50					14	5.2	E	SSW* 11.1	7.1
28	10.4	0.8	5.6	12.4	0.6	97	49		8.8		8.8	7	8.9	ENE	E 16.7	0.6
29	19.6	3.0	11.3	6.7	6.3	100	59		1.4		1.4	TR	8.3	E	SW 14.8	5.3
30	24.3	0.2	12.3	5.7	7.3	97	32	1	1.4		1.4	TR	8.3	S	WNW* 24.1	8.3
31	10.2	-0.9	4.7	13.3		85	38						22.2	WNW	WNW 37.0	7.7
MEAN MOYENNE	5.6	-3.2	1.2	521.5	17.9	85	54	1	74.1	27.2	100.2		14.1	W	NNW 38.9	147.7
NORMAL NORMALE	1.3	-7.5	-3.1	654				0.2	25.9	35.1	61.0		22.1	E		149.9

DEGREE-DAY SUMMARY - SOMMAIRE DE DEGRÉS-JOURS

BELOW 18°C AU-DESSOUS DE 18°C				ABOVE 5°C AU-DESSUS DE 5°C				DAYS WITH TOTAL PRECIPITATION JOURS AVEC PRÉCIPITATIONS TOTALES					DAYS WITH SNOWFALL JOURS AVEC CHÊTE DE NEIGE				
THIS YEAR ANNÉE EN COURS	PREVIOUS YEAR ANNÉE PRÉCÉDENTE	NORMAL NORMALE		THIS YEAR ANNÉE EN COURS	PREVIOUS YEAR ANNÉE PRÉCÉDENTE	NORMAL NORMALE		0.2 OR MORE	1.0 OR MORE	2.0 OR MORE	10.0 OR MORE	50.0 OR MORE	0.2 OR MORE	1.0 OR MORE	2.0 OR MORE	10.0 OR MORE	50.0 OR MORE
TOTAL FOR MONTH TOTAL DU MOIS	521.5	677.0	654	TOTAL FOR MONTH TOTAL DU MOIS	17.9	6.7	n11	DU PLUS	DU PLUS	DU PLUS	DU PLUS	DU PLUS	DU PLUS	DU PLUS	DU PLUS	DU PLUS	DU PLUS
ACCUMULATED SINCE JULY 1 ACCUMULÉE DEPUIS LE 1 ^{ER} JUILLET	4299.1	4157.4	4077	ACCUMULATED SINCE APRIL 1 ACCUMULÉE DEPUIS LE 1 ^{ER} AVRIL	2003.3	2261.4	1872	9	7	4	2	0	5	4	2	1	0

UDC 551.506:1 713.84 SVRL signifie plusieurs
SVRL stands for several

*Indique plusieurs vitesses identiques Prix/Price (\$1.00)
*Indicates more than one of the same speed

MONTHLY METEOROLOGICAL SUMMARY SOMMAIRE MÉTÉOROLOGIQUE MENSUEL

MONTH/MOIS APRIL / AVRIL

1977

AT/A OTTAWA INTERNATIONAL AIRPORT / AEROPORT INTERNATIONAL D'OTTAWA

LAT 45 1' N		LONG 75 40 W		ELEVATION ALTITUDE 374		FEET (ASL) PIEDS (MER)		STANDARD TIME USED HEURE NORMALE UTILISEE		EASTERN OF L'EST						
DATE	TEMPERATURE TEMPERATURE			DEGREE DAYS DEGRES JOURS		REL HUMIDITY HUMIDITE REL		THUNDERSTORM ORAGE	PRECIPITATION PRECIPITATIONS			SNOW ON GROUND NEIGE AU SOL	WIND VENT			BRIGHT SUNSHINE INSOLATION EFFECTIVE
	MAXIMUM MAXIMALE	MINIMUM MINIMALE	MEAN MOYENNE	HEATING DE CHAUFFEE	GROWING DE CROISSANCE	MAXIMUM MAXIMALE	MINIMUM MINIMALE		RAINFALL PLUIE (HAUTEUR)	SNOWFALL NEIGE (HAUTEUR)	TOTAL PRECIP PRECIP TOTALE		AVERAGE SPEED VITESSE MOYENNE	PREVAILING DIRECTION DIRECTION DOMINANTE	MAXIMUM SPEED AND DIRECTION VITESSE MAXIMALE ET DIRECTION	
	C	C	C	BASE 18°C	BASE 5°C				mm	cm	mm	cm	km/h		km/h	HOURS HEURES
1	6.9	-4.3	1.3	16.7		62	23						13.0	WNW	NNW	27.8
2	4.2	-2.3	1.0	17.0		100	49		13.4	TR	13.4		19.2	E	E	31.5
3	10.9	-2.7	4.2	13.8		97	28		TR		TR		30.6	WNW	WNW	59.3
4	2.2	-4.1	-1.0	19.0		92	37		0.3	3.0	3.4		21.3	E	E	40.7
5	4.0	-1.3	3.8	14.2		97	49		5.1	TR	5.1	1	22.2	E	E	40.7
6	0.7	-6.3	-2.8	20.8		75	39			TR	TR	TR	30.2	WNW	SW	40.7
7	1.5	-9.0	-3.8	21.8		96	30			5.0	3.8		12.0	S	S	22.2
8	-2.1	-10.5	-6.3	24.3		97	38			2.2	2.0	7	20.6	WNW	WNW	42.2
9	3.7	-10.4	-3.4	21.4		74	27					3	13.9	WNW	WNW	27.8
10	10.3	-4.7	2.8	15.2		71	32					TR	3.7	SE	SE	13.0
11	20.3	-1.0	9.7	4.3	4.7	79	39						14.5	S	S	27.8
12	17.4	9.3	13.4	4.6	8.4	67	22						10.0	NNW*	NNW	25.9
13	20.8	4.8	12.8	5.2	7.8	94	44		5.5		5.5		14.3	WNW	WNW	37.0
14	10.5	2.1	6.3	11.7	1.3	91	40						21.3	NNW	NNW	31.5
15	13.6	-1.0	6.3	11.7	1.3	82	22						16.3	NNW	NNW	35.2
16	15.4	1.0	8.2	9.8	3.2	52	18						14.8	NW	WNW	25.9
17	17.5	2.0	9.8	8.2	4.8	52	22						8.3	NNW	NNW	13.0
18	21.2	2.4	11.8	6.2	6.8	62	21						3.6	SVRL	NW	11.1
19	23.7	5.3	14.5	3.5	9.5	73	26						5.8	ESE	E	18.5
20	23.8	7.3	15.6	2.4	10.6	77	40		0.2		0.2		9.9	SSW	SSW	22.2
21	25.4	14.3	19.9		14.9	87	43	1	1.2		1.2		15.4	SSW	SW	27.8
22	19.3	9.1	14.2	3.8	9.2	94	62		7.6		7.6		13.2	SW	NW	25.9
23	12.4	6.9	9.7	8.3	4.7	69	47						16.5	ENE	ENE	22.2
24	10.3	4.2	7.3	10.7	2.3	94	53		17.3		17.3		23.9	ENE	E	33.3
25	5.1	3.7	4.9	13.1		92	88		6.8		6.8		15.4	NE	E	25.9
26	14.1	4.0	9.1	8.9	4.1	94	43						5.9	S	S	18.5
27	14.4	1.9	8.2	9.8	3.2	90	45		TR		TR		17.3	SSW	SSW	27.8
28	10.9	-2.6	4.2	13.8		89	38		0.6	TR	0.6		27.8	NA	NNW	40.7
29	13.4	-3.5	5.0	13.0		73	29						21.5	W	WNW	42.6
30	19.8	1.0	10.4	7.6	5.4	67	24						11.7	WSW	S	20.4
31																
MEAN MOYENNE	12.6	0.5	6.1	344.8	102.2	81	37	1	58.0	10.2	67.1		16.0	WNW	NNW	59.3
NORMAL NORMALE	10.9	0.3	5.6	368	67.0			0.9	58.4	7.9	67.6		16.7	W		175.2

DEGREE-DAY SUMMARY - SOMMAIRE DE DEGRES-JOURS						DAYS WITH TOTAL PRECIPITATION JOURS AVEC PRECIPITATIONS TOTALES						DAYS WITH SNOWFALL JOURS AVEC CHUTE DE NEIGES					
BELOW 18°C AU-DESSOUS DE 18°C			ABOVE 5°C AU-DESSUS DE 5°C			mm						cm					
THIS YEAR ANNEE EN COURS	PREVIOUS YEAR ANNEE PRECEDENTE	NORMAL NORMALE	THIS YEAR ANNEE EN COURS	PREVIOUS YEAR ANNEE PRECEDENTE	NORMAL NORMALE	0.2 OR MORE PLUS	1.0 OR MORE PLUS	2.0 OR MORE PLUS	10.0 OR MORE PLUS	50.0 OR MORE PLUS	0.2 OR MORE PLUS	1.0 OR MORE PLUS	2.0 OR MORE PLUS	10.0 OR MORE PLUS	50.0 OR MORE PLUS		
TOTAL FOR MONTH TOTAL DU MOIS	344.8	320.0	368	TOTAL FOR MONTH TOTAL DU MOIS	102.2	10	7	6	2	0	3	3	3	0	0		
ACCUMULATED SINCE JULY 1 DEPUIS LE 1 ^{er} JUILLET	4463.9	4477.4	4416	ACCUMULATED SINCE APRIL 1 DEPUIS LE 1 ^{er} AVRIL	102.2	10	7	6	2	0	3	3	3	0	0		

UDC 551 506 1/13, 54 SVRL SIGNIFIE PLUSIEURS * INDIQUE PLUSIEURS VITESSES IDENTIQUES PRIX/PRICE (\$1.00)
 S/M/L STANDS FOR SEVERAL * INDICATES MORE THAN ONE OF THE SAME SPEED

MONTHLY MÉTÉOROLOGICAL SUMMARY

SOMMAIRE MÉTÉOROLOGIQUE MENSUEL

MONTH/MOIS MAY/MAI

1977

AT/A OTTAWA INTERNATIONAL AIRPORT / AÉROPORT INTERNATIONAL D'OTTAWA

LAT 45 • 19' N		LONG 75 • 40' W		ELEVATION ALTITUDE 374		FEET (ASL) PIEDS (MER)		STANDARD TIME USED HEURE NORMALE UTILISÉE		EASTERN DE L'EST													
DATE	TEMPERATURE TEMPERATURE			DEGREE DAYS DEGRÉS JOURS		REL HUMIDITY HUMIDITÉ REL		THUNDERSTORM ORAGE	PRECIPITATION PRÉCIPITATIONS			SNOW ON GROUND NEIGE AU SOL	WIND VENT			BRIGHT SUNSHINE INSOLATION EFFECTIVE							
	MAXIMUM MAXIMALE	MINIMUM MINIMALE	MEAN MOYENNE	HEATING DE CHAUFFE	GROWING DE CROISSANCE	MAXIMUM MAXIMALE	MINIMUM MINIMALE		RAINFALL PLUIE (HAUTEUR)	SNOWFALL NEIGE (HAUTEUR)	TOTAL PRECIP PRÉCIP TOTALE		AVERAGE SPEED VITESSE MOYENNE	PREVAILING DIRECTION DIRECTION DOMINANTE	MAXIMUM SPEED AND DIRECTION VITESSE MAXIMALE ET DIRECTION								
	°C	°C	°C	BASE 18°C	BASE 5°C	%	%		mm	cm	mm		cm/h		km/h								
1	22.2	5.7	14.0	4.0	9.0	68	25			0.4			17.8	SSW	SW	35.2	12.1						
2	17.2	4.8	11.0	7.0	6.0	95	27			0.4			16.1	NNW	NNW	35.2	9.1						
3	12.6	1.0	7.1	10.9	2.1	72	33						12.0	NNW	NNW	25.9	11.8						
4	17.0	0.8	8.9	9.1	3.9	78	30						3.4	E	SSE	0.3	13.2						
5	21.9	4.9	13.4	4.6	8.4	94	50			1.0			10.0	SSW	SSW	22.2	9.9						
6	25.3	3.9	14.6	3.4	9.6	96	19						25.6	SW	W	46.3	13.4						
7	10.5	1.0	5.8	12.2	0.8	64	29						21.0	N	NNW	27.8	13.2						
8	6.6	0.8	3.7	14.3		94	44		1.8	0.8	2.6		14.3	E	ESE	27.8	2.4						
9	11.7	1.0	6.4	11.6	1.4	68	40		TR		TR		21.1	N	N	29.6	6.0						
10	13.9	6.5	10.2	7.8	5.2	57	36						17.6	N	N	29.6	4.3						
11	22.1	5.7	13.9	4.1	8.9	68	26						13.0	WNW	WNW	22.2	13.7						
12	20.3	8.7	14.5	3.5	9.5	77	40		TR		TR		11.5	W	W	18.5	3.3						
13	17.2	6.3	11.8	6.2	6.8	52	21						21.3	NNW	NNW	35.2	14.0						
14	16.6	3.8	10.2	7.8	5.2	51	24						10.9	NNW	NNW	33.3	14.3						
15	21.2	6.1	13.7	4.3	8.7	61	21						10.4	NW	W	18.5	13.9						
16	28.3	8.6	18.5		13.5	66	24						14.1	SW	WSW	22.2	13.1						
17	32.4	14.2	23.3		18.3	64	26						17.1	N	WNW	29.6	10.6						
18	24.5	13.6	19.1		14.1	74	43						8.3	E	E	22.2	8.9						
19	26.4	10.1	18.3		13.3	79	29						7.8	E	ENE	14.8	11.5						
20	26.4	15.4	20.9		15.9	83	38						5.2	E	E	14.8	7.8						
21	31.0	14.9	23.0		18.0	92	35						8.9	SSW	SSW	18.5	13.2						
22	31.1	16.4	23.6		18.6	86	32		TR		TR		8.0	SSW	WSW	24.1	11.7						
23	30.2	16.3	23.3		18.3	85	37	1	19.0		19.0		8.9	SSW	NNW	33.3	10.4						
24	28.4	16.5	22.5		17.5	93	44						5.6	SSW	SW	16.7	4.3						
25	22.0	12.6	17.3	0.7	12.3	92	47						18.0	NW	NW	29.6	11.8						
26	24.1	10.1	17.1	0.9	12.1	75	26						12.8	WNW	N	27.8	14.2						
27	21.9	13.0	17.5	0.5	12.5	71	36		TR		TR		7.1	WNW	ENE	14.8	5.6						
28	14.0	6.2	10.1	7.9	5.1	95	71	1	19.8		19.8		7.1	N	NE	27.8	2.5						
29	18.7	4.3	11.5	6.5	6.5	89	43						7.1	ENE	E	16.7	14.5						
30	22.6	8.0	15.3	2.7	10.3	88	37						2.8	S	SSE	9.3	13.1						
31	25.5	11.6	18.6		13.6	76	30						6.7	SSW	SSW	14.8	14.1						
MEAN MOYENNE	21.4	8.2	14.8	130.0	305.4	80	34	2	51.8	0.8	52.6		12.2	S	N	46.3	321.9						
NORMAL NORMALE	18.3	6.5	12.4	181.0	233.0			2.2	68.8	0.8	70.1		18.2	SW			230.5						
DEGREE-DAY SUMMARY - SOMMAIRE DE DEGRÉS-JOURS																							
BELOW 18°C AU-DESSOUS DE 18°C				ABOVE 5°C AU-DESSUS DE 5°C				DAYS WITH TOTAL PRECIPITATION JOURS AVEC PRÉCIPITATIONS TOTALES mm					DAYS WITH SNOWFALL JOURS AVEC CHÛTE DE NEIGE cm										
								THIS YEAR ANNÉE EN COURS	PREVIOUS YEAR ANNÉE PRÉCÉDENTE	NORMAL NORMALE	0.2 OR MORE	1.0 OR MORE	2.0 OR MORE	10.0 OR MORE	50.0 OR MORE	0.2 OR MORE	1.0 OR MORE	2.0 OR MORE	10.0 OR MORE	50.0 OR MORE			
TOTAL FOR MONTH TOTAL DU MOIS				TOTAL FOR MONTH TOTAL DU MOIS				130.0	203.8	181	305.4	208.9	233.0	6	5	4	2	0	1	0	0	0	0
ACCUMULATED SINCE JULY 1 DEPUIS LE 1 ^{er} JUILLET				ACCUMULATED SINCE APRIL 1 DEPUIS LE 1 ^{er} AVRIL				4773.9	4681.2	4629	407.6	324.1	298.5	6	5	4	2	0	1	0	0	0	0

UOC 551 508.1 (713.84)

SVRL SIGNIFIE PLUSIEURS
SVRL STANDS FOR SEVERAL

* INDIQUE PLUSIEURS VITESSES IDENTIQUES
* INDICATES MORE THAN ONE OF THE SAME SPEED

PRIX/PRICE (1.00)

A P P E N D I X " B "

PHOTOGRAPHS

PHOTO #1

Installation of Stringline
Parallel to Transverse
Pavement Cracks

PHOTO #2

Installation of Steel Pins

PHOTO #3

Measurement Between
Steel Pins

PHOTO #4

Steel Pin Arrangement
at Road Centreline

PHOTO #5

Transverse Pavement Crack
at Station F - June, 1977

PHOTO #6

Transverse & Longitudinal
Pavement Cracking at
Station D

A P P E N D I X " C "

THEORETICAL THERMAL CONTRACTION OF ASPHALTIC CONCRETE

APPENDIX C THEORETICAL THERMAL CONTRACTION OF ASPHALTIC CONCRETE

It is assumed that a finite length of pavement on each side of a transverse pavement crack will contract under low temperatures in accordance with the formula $\Delta = l \alpha t$ where:

- Δ = change in length of the finite length of pavement
- l = length of pavement
- α = linear coefficient of thermal contraction for asphaltic concrete
- t = temperature differential

For length a, $\delta_a = a \alpha t$
 For length b, $\delta_b = b \alpha t$

The total pavement contraction can be written as

$$\delta_T = \delta_1 + \delta_2 + \alpha t (a + b)$$

For purposes of this report $\alpha t (a + b)$ has been assumed to be negligible in that the degree of accuracy of measurement of shrinkage is ± 0.5 mm.

Taking the worst case of

a) a frictionless boundary between asphaltic concrete and granular base materials.

b) a maximum temperature variation of 100°F .

and using $\alpha = 1.5 \times 10^{-5}/^{\circ}\text{F}$ as suggested by Phang (46),

and assuming a distance between steel pins as 10 inches

$$(254 \text{ mm}), \text{ then } \alpha t(a + b) = 1.5 \times 10^{-5} \times 100 \times 254 \\ = 0.381 \text{ mm}$$

Since the accuracy of measurements is $\pm 0.5 \text{ mm}$, it is justified

to assume that $\alpha t(a + b)$ is therefore negligible

(i.e. $0.381 < 0.5$).

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