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LOW TEMPERATURE INDIRECT TENSILE TEST AND CRACKING TEMPERATURE PREDICTION FOR ASPHALT MIXES

by

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The undersigned certify that they have read, and recommended to the Faculty of Graduate Studies and Research, for acceptance, a thesis entitled **Low Temperature Indirect Tensile Test and Cracking Temperature Prediction for Asphalt Mixes** by Baoqin Bai in the partial fulfillment of the requirements for the degree of Master of Science.

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ABSTRACT

The indirect tensile test is one of the most extensively used test methods to evaluate low temperature properties of asphalt mixes. This thesis describes two testing programs utilizing the constant loading speed to failure indirect tensile tests on some recycled tire rubber modified asphalt mixes and the asphalt mixes used in the C-SHRP Lamont Test Road in Alberta. An revised outlier rejection method is introduced for the indirect tensile test data analysis.

Based on the asphalt mixes used in Lamont Test Road, the relationships between the results from this indirect tensile test method and the data from the generally accepted nomograph methods are analyzed. It is found that the tested asphalt mix stiffness compares reasonably well with the calculated stiffness from the nomographs. Generally, the tested mix stiffness is a little larger than the calculated stiffness, and the higher the asphalt mix stiffness, the smaller the difference between the tested and the calculated stiffnesses. It is also found that the curve through the tensile strength vs. mix stiffness from indirect tensile test is a little different from that of the generally accepted curves.

After reviewing previously used prediction methods, a cracking temperature prediction method called "Improved Theoretical Method" is presented based on an analysis of the thermal stress relaxation process. The cooling rate and changes of the asphalt stiffness with loading time, which are very important factors influencing cracking temperature, are emphasized in this method. Published information from the well-known Ste. Anne Test Road and the current C-SHRP Lamont Test Road is compared with the predicted cracking temperatures from four different methods. It is found that the Improved Theoretical Method is more accurate than the other methods as far as the data from Ste. Anne Test Road are concerned.

A computerized bitumen test data chart (BTDC) has been developed based on the Heukelom's version in 1969 using a commercially available program Scientific Graph System SigmaPlot™. This computerized version enables the BTDC to be produced with high accuracy and quality.

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It is gratefully acknowledged that both the laboratory tested and the field observed data for the asphalt cements and asphalt mixes from Lamont Test Road have been provided by Alberta Transportation and Utilities. The revised test data analysis procedure was based on data obtained in the Department of Civil Engineering Laboratory on recycled tire rubber asphalt mixes for a project undertaken by EBA Engineering Consultants Ltd. Appreciation for the use of these data is hereby given.

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CHAPTER ONE

INTRODUCTION

1.1 Background

Early cracking of asphalt pavement in low temperature climates usually occurs in the first or the second winter after construction of the pavement. This cracking is usually developed perpendicular to the center line of the road, and therefore, is commonly referred to as low temperature transverse cracking or simply low temperature cracking. It is generally agreed (Gaw, 1978) that when the thermal stress induced in the pavement surface layer material due to the temperature drops exceeds the tensile strength of the material, the failure or cracking of the material occurs.

Maintenance such as sealing is usually needed for this kind of cracking. Initially, the low temperature cracking itself does not influence the service quality much. However, if not properly sealed, the cracks permit the ingress of water. Water weakens the pavement structure resulting in various damages to the pavement. Swelling associated with clay subgrade soils can contribute to great losses in the service life of the pavement.

For the study of the low temperature cracking of asphalt mixes, some properties (such as failure stress, failure strain, mix stiffness, etc.) of the materials must be obtained. The indirect tensile test has been used at the University of Alberta since the 1960s for such characterization. The most recent study at the University of Alberta conducted by Hussain (1990) was a low temperature evaluation of the characteristics of some polymer modified asphalt mixes.

Several nomographs have been used in the study of asphalt binder and mix properties if direct tests are not available. Extensively used nomographs include those for the asphalt stiffness developed by van der Poel (1954), for the asphalt mix stiffness developed by Bonnaure et al. (1977), and for the asphalt mix strength developed by Heukelom (1966) or by Deme and Young (1987). These nomographs were produced from many direct tests and give an approximate estimation of the properties of the materials studied.

Although it is reasonable to assume that the indirect tensile test gives a good measurement of the tensile properties of asphalt mixes at low temperatures, because of the complicated biaxial stress state in a specimen, it is not clear whether the results measured from the indirect tensile test agree with the results estimated from the nomographs. It is also not clear if the results from the indirect tensile tests can be used in the study of the transverse cracking without significant error.

The Congress of the United States established a five year, $150 million research program in 1987. This program is known as the Strategic Highway Research Program (SHRP). By 1993, SHRP has just finished a major five-year research program with four areas targeted, one of which was asphalt, including asphalt pavement low temperature cracking. By benefiting from the SHRP of the United States, Canadian transportation authorities also started a research program known as the Canadian Strategic Highway Research Program (C-SHRP) in order to solve a set of unique Canadian problems which had been put forward by various experts across Canada.

In 1991, Alberta Transportation and Utilities constructed a test road east of Lamont, approximately 90 km northeast of Edmonton. This project was constructed as a part of the C-SHRP project entitled "Performance Correlation for Quality Paving Asphalt." The test road contained seven asphalt cements corresponding to the seven test sections with only one aggregate source. Low temperature indirect tensile tests were conducted as a complementary program by Alberta Transportation and Utilities and the University of Alberta. A detailed description of the study is presented in the report by Wang et. al. (1992).

By further analysis of the indirect tensile test data from the Lamont Test Road, the relationships between the results from the indirect tensile test and the data from the generally accepted nomographs are established in this thesis, and hopefully the results of the study will be helpful for the further research using the indirect tensile test method and its application to the study of low temperature cracking.

An additional opportunity to evaluate the low temperature properties of asphalt mixes was afforded when the University of Alberta contracted with EBA Engineering Consultants Ltd. to undertake the indirect tensile test of asphalt rubber specimens for the evaluation of low temperature performance which was a part of the research program, "The

Use of Recycled Tire Rubber in Asphalt Concrete Pavements". As the problem of waste tire disposal has become more and more serious over the past few years, interest in recycling waste tire rubber has increased. The research program carried out by EBA Engineering Consultants Ltd. (1993) was to document the technical and economic feasibility of incorporating recycled tire rubber into asphalt mixes in Alberta. One of the technical aspects of the study involved engineering properties of the asphalt rubber mixes as determined from laboratory testing programs to predict anticipated performance under various traffic and climate conditions typical in Alberta.

For the study of low temperature cracking in asphalt pavement, prediction of cracking temperature is one of the most important steps for design of asphalt mixes. The factors influencing the cracking temperature include the rheological properties of asphalt, the composition of the mix, the coefficient of thermal contraction of the mix, the cooling rate, and the thickness of the asphalt layer of the pavement, etc. Presently, many methods of cracking temperature prediction have been developed. Some of these methods fail to consider the influence of the cooling rate or changes of the stiffness of asphalt (or mix) with loading time, and others fail to consider the composition and (or) the tensile strength of the mix. Thus, in this thesis a prediction method has been developed to take account of almost all of the important influencing factors.

1.2 Purpose of the Thesis

This thesis will achieve the three main objectives:

a) Establish relationships between the results from the indirect tensile test on asphalt mixes from the Lamont Test Road and data estimated from the generally accepted nomographs.

b) Use the indirect tensile test to evaluate the low temperature failure properties of recycled tire rubber asphalt mixes, and introduce an outlier rejection method for the treatment of data from the indirect tensile test.

c) Develop a more accurate cracking temperature prediction method based on the thermal stress relaxation and superposition process analysis.

1.3 Organization of the Thesis

This thesis has six chapters:

Chapter One briefly presents the background, the purpose, and the organization of the thesis.

Chapter Two presents a general review on the test methods for low temperature evaluation of asphalt mixes with emphasis on the indirect tensile test.

Chapter Three discusses the low temperature property evaluation on the tire rubber modified asphalt mixes and introduces an outlier rejection method for the indirect tensile test.

Chapter Four analyzes the relationships between the results from the indirect tensile test and the data from generally accepted nomograph methods.

Chapter Five presents a brief review of cracking temperature prediction methods and develops an improved cracking temperature prediction method based on the thermal stress relaxation and superposition process analysis and finally, compares the observed cracking information from both Lamont and Ste. Anne test roads with the predicted cracking temperatures by four different methods.

Chapter Six summarizes the principal conclusions of the thesis and the directions for future study.

This thesis contains five appendixes:

Appendix I presents the grouping results for the specimens of the indirect tensile test on tire rubber asphalt mixes.

Appendix II provides the indirect tensile test results for tire rubber asphalt mix specimens both before and after the outlier rejection.

Appendix III explains the development of the computerized version of the Bitumen Test Data Charts (BTDC). The BTDCs for the asphalts used in the Ste. Anne Test Road and Lamont Test Road are presented.

Appendix IV presents the comparison of the indirect tensile test results for the Lamont Test Road with the estimates from the nomographs.

Appendix V presents the detailed input and output for the four different cracking temperature prediction methods.

CHAPTER TWO

LITERATURE REVIEW OF VARIOUS TEST METHODS FOR THE EVALUATION OF LOW TEMPERATURE PROPERTIES OF ASPHALT MIXES

2.1 Introduction

At low temperatures, the behavior of asphalt pavement depends mainly on the characteristics of the asphalt mix used in the pavement. Thus, it is important to evaluate the low temperature properties of asphalt mixes in an appropriate way for the purpose of successful low temperature design of asphalt pavement.

The evaluation methods for the low temperature behavior of asphalt mix generally can be classified into two categories:

i. Indirect methods, and
ii. Direct methods.

Indirect methods can be used to calculate some properties of asphalt mixes without direct tests. Examples of this approach are the methods suggested by Heukelom and Klomp (1964) and Bonnaure et al. (1977) to predict stiffness of asphalt mix by using the stiffness of asphalt. These indirect methods are also based on measured properties of asphalt mixes. In this chapter, the review on the direct methods are emphasized.

The direct evaluation methods are classified into five categories here based on the way the sample deforms:

i. Direct tensile test,
ii. Indirect tensile test,
iii. Bending beam test,
iv. Thermal contraction test, and
v. Other methods.

Different test methods are used to predict different types of response of asphalt pavement in order to closely represent both the pavement structure and the material behavior. Low temperature transverse cracking of asphalt pavement can be most closely represented by the direct tensile test, while cracking caused by traffic loads can be represented by the bending beam test, and rutting of asphalt pavement can be represented by the triaxial compression test, and so on.

Although the compressive test is also extensively used, it is not considered a separate class here because compressive properties of asphalt mix are not important for and are not related directly to the asphalt pavement low temperature cracking that this thesis mainly deals with.

Based on the way the load is applied, direct methods can also be classified as:

i. Creep test: The stress is kept constant, and the deformation or strain is measured,
ii. Relaxation test: The strain is kept constant, and the load is measured,
iii. Constant rate of strain test: The rate of strain is kept constant, and both load and deformation are measured,
iv. Dynamic test: Dynamic loading is applied, and both the load and the deformation are measured,
v. Others.

In the following discussion, the former classification system will be followed, and at the same time, the terms in the latter classification system will be mentioned or discussed as needed. Research on the indirect tensile tests will be reviewed last.

2.2 Direct Tensile Test

The direct tensile test is usually done under the condition of direct and uni-axial tensile loading on a cylindrical or prismatic specimen. The stress and strain are easily calculated or analyzed. This test most closely simulates the actual stress state and fracture mode that represents the thermally induced cracking in the field.

A lot of work has been carried out with this method since the 1960s. Following is a chronological review on some of the significant studies:

1). Tons and Krokosky (1963) conducted direct tensile tests at temperatures varying from -28.9 to 48.9°C (-20 to 120°F) and at different constant strain rates varying from 0.004 to 4 in/in/min. Cylindrical specimens were used to test the stress-strain characteristics of certain dense graded asphalt mixes with different asphalt contents (5.5, 6.5, and 7.5%) and different combinations of micro aggregate and asbestos:

a) 5% limestone and 0% asbestos,
b) 6% limestone and 5% asbestos, and
c) 7% limestone and 2.5% asbestos.

The asphalt used was 85-100 penetration grade. The curves of stress vs. strain at different temperatures and strain rates and the curves of failure stress vs. temperature at different strain rates were analyzed. The effects of air voids and micro aggregate were evaluated. From this study, the following important findings were obtained:

i. For the workable or practical asphalt mixes, the failure strain was around 1%, and the increase in asphalt content and temperature appeared to have relatively small effects on the failure stain.
ii. The loading rate had great influence on the tensile strength at higher temperatures but had little influence at lower temperatures.
iii. Mixes with asbestos filler showed considerably higher ultimate strength at low temperatures.
iv. The peak tensile strength occurred at about -6.7°C (20°F).

2). Domaschuk, Skarsgard, and Christianson (1964) undertook direct tensile tests at temperatures of -26.1, -20, and -4.4°C (-15, -4, and 24°F). The constant rates of extension of 0.203 and 0.0203 mm/min (0.008 and 0.0008 in./min) were used. The specimens were removed from an existing pavement. Two transverse grooves were cut along the center of the specimen to ensure failure at this section. The curves of stress vs. strain at different rates of extension and the curves of failure stress and failure strain vs. temperature were obtained and analyzed. It was found that the failure stress increased and the failure strain decreased with decreasing temperature for a given rate of loading and that the failure stress increased with the rate of loading.

3). Monismith, Secor, and Secor (1965) performed four kinds of tests with one single asphalt mix. These tests included:

- Creep tests in tension at constant temperature from -40 to 43°C (-40 to 110°F),
- Constant rate of strain tests in tension to failure at temperature from -40 to 43°C (-40 to 110°F),
- Thermal stress test (low temperature stress restrained specimen tests), and
- Creep test at constant stress but with variable temperature.

From the direct tensile creep tests, the tensile creep compliance of the mix was obtained. Using a time-temperature equivalence (superposition) technique, the master curve of the creep compliance was acquired under the assumption that the asphalt mix was a "thermorheologically simple" material. Furthermore, it was shown that the master curve could be used to predict the thermal stress in the asphalt pavement with reasonable accuracy for engineering purposes, based on the comparison between the predicted stress and the measured stress by using the restrained specimen thermal stress tests.

The data from both creep and constant rate of strain tests were used to plot a "failure envelope" chart, which was previously used to characterize polymers' failure strength. In a later study by Monismith et al. (1966), they found that the asphalt concrete could be assumed to be a linear visco-elastic material if the deformation to which it was subjected was less than 0.1%, and that the asphalt concrete could be assumed a thermorheologically simple material.

4). Burgess, Kopvillem, and Young (1971) conducted direct tensile creep tests to evaluate the low temperature stiffness and failure stress of the three asphalt mixes which were used in the Ste. Anne Test Road constructed in 1967 in Manitoba and incorporated 29 test sections. Three asphalt cements and one slow-curing liquid asphalt were used in the mixes. The elastic and viscous components of the asphalt mixes were analyzed. By using the test data and the theory of calculating thermal stress suggested by Hills and Brien (1966), the thermal stress was calculated, and then the cracking temperatures were obtained (called **laboratory predicted** fracture temperature). By using a "nomograph procedure", i.e. by using the van der Poel nomograph (1954) and Heukelom chart for tensile strength (1966), the so-called **predicted** fracture temperatures were calculated. It was concluded that the **laboratory predicted** and the **predicted** fracture temperatures correlated well; and both of them also correlated well with the pavement surface temperatures. It was found that the tendency of a mix to crack at low temperature could be indicated by a knowledge of the asphalt stiffness at low temperatures and long loading times.

5). Salam and Monismith (1972) tested different asphalt mixes by the direct tensile test (constant rate of strain to failure), fracture tests (single-edge-notched tension and bending), and the flexural (bending) fatigue test. The temperature at which the direct tensile tests were undertaken was from -28.9 to 20°C (-20 to 68°F). Fifteen different asphalt mixes were used with variables of:

(1) asphalt hardness (40-50, 60-70, 85-100, and 120-150 four penetration grades),
(2) asphalt content (6, 7, 8, and 9%),
(3) aggregate type,
(4) aggregate gradation,
(5) mineral filler type, and
(6) compaction degree.

It was found from the constant rate of strain to failure test that

i. The optimum strength asphalt content in tension and optimum asphalt penetration at low temperatures existed,
ii. The lower the air voids, the higher the tensile strength,
iii. The fine-graded mixes produced higher tensile strength than the coarse-graded ones at low temperatures, and
iv. The mix containing the granite exhibited a higher tensile strength than the mix with the limestone aggregate.

The data from the tests in this paper and the data from Heukelom (1966) were compared. The "mix factor", M, as described by Heukelom, for the studied mixes, was established.

6). Hignell, Hajek, and Haas (1972), by using the constant rate of strain direct tensile test method, evaluated the properties of different asphalt mixes including standard (conventional) asphalt concrete and mixes with different types and percentages of asbestos fibers and mineral fillers. The temperatures used in the test were -28.9, -17.8, and 21.1°C (-20, 0, and 70°F). The percent changes of the failure stress, strain, and stiffness vs.

temperature for the asbestos fiber modified asphalt mixes compared with the standard mixes were analyzed. It was concluded that at low temperatures, the properties of all types of mixes were primarily a function of asphalt type used, but at medium to high service temperatures, asbestos fiber modification could significantly improve the properties of the mixes. It was also stated that the properties of the asphalt mixes corresponded remarkably well with results reported from the Arkona Test Road and Ste. Anne Test Road, and that the concept of the limiting stiffness guidelines for the problem of low temperature cracking was supported by these comparisons.

7). Pavlovich and Goetz (1976) used ANOVA (Analysis of Variance) to evaluate the effects of the following four factors influencing failure strain obtained from direct tensile test:

i. Asphalt type (six levels),
ii. Aggregate gradation (three levels),
iii. Temperature from -27.5 to 60°C (-17.5 to 140°F), (six levels), and
iv. Strain rate from 0.3 to 159 micro strain per second, (four levels).

The results of the analysis were:

i. Temperature was the most significant factor,
ii. The strain rate in the range corresponding to the variation of the speed of ordinary vehicular loading was much less significant than the temperature,
iii. Fine graded mixes have higher limiting strain than coarse graded ones,
iv. Mixture type (asphalt type) had no significant effect.

8). Sugawara (1972) classified the breaking of asphalt mixes under loading into three categories (as shown in Fig.2.1):

i. Brittle type breaking (Type I),
ii. Yielding type breaking (Type II),
iii. Flowing type (Type III).

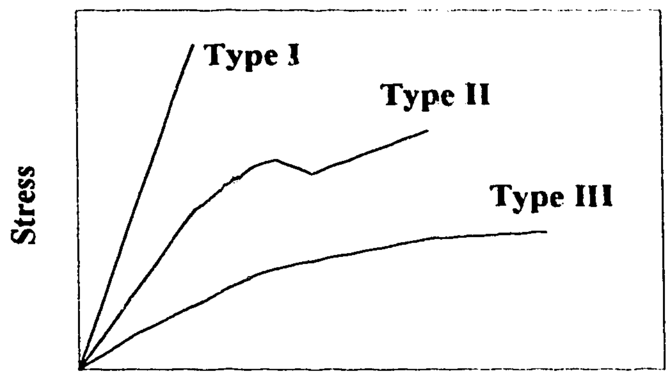


Fig.2.1-Failure Types of Asphalt Mix

The experiments included direct tensile, compressive, and bending beam tests und temperature from -30 to 30°C (-22 to 86°F). The principle findings were:

i. The failure stress increased as the temperature decreased to a certain point. At th point, the failure stress reached the maximum value and started to decrease as tl temperature decreased.
ii. The curves of failure stress vs. temperature, failure strain vs. temperature, and tl failure stiffness vs. temperature can be shifted in parallell fashion to the direction higher temperature as the rate of strain increased.
iii. The compressive strength was about 3-4 times as high as the bending strength, ai the bending strength was about 2 times as high as the direct tensile strength.
iv. In low temperature area (or brittle breaking area in which the failure stress decreas as temperature decreased), the failure strain of asphalt concrete was about 1×1C which was independent of the property of the binder.

9). Anderson and Epps (1983) reported direct tensile tests (constant rate of strai and indirect tensile tests (resilient modulus tests and constant loading speed to failu tests). The purpose was to establish the relationship between the properties of asphalt rr and its components and asphalt pavement cracking. Samples used in the tests were cored or cut from six highway test sections in West Texas. The direct tensile tests were performed at temperatures of 24, 0.6, and -23°C (75, 33, -9°F) and at constant rates of extension of 51, 5.1, and 0.51 mm/min (2, 0.2, and 0.02 in./min). The indirect tensile tests (constant loading speed to failure) were conducted at temperature of 23, 0.6, and -23°C (73, 33, and -9°F) and at loading speeds of 51, 5.1, and 0.51 mm/min (2, 0.2, and 0.02 in./min). It was found that the rates of extension (for direct tensile tests) and the loading speeds (for indirect tensile tests) had large influences on the behavior of the asphalt mixes at the highest temperature 22°C (72°F) in the tests, but had little (for direct tensile tests) or slight (for indirect tensile tests) influences at the lowest temperature -23°C (-9°F). The comparison of elastic modulus values determined by indirect tensile tests to those by the direct tensile tests showed that the moduli from the two tests were different, and the higher the moduli, the greater the difference. It was considered that although the reasons for the obvious deficiency of the indirect tensile test method and analysis were not readily apparent, the difficulties with the horizontal deformation measurement (exterior measurement method) might have contributed to part of the problem.

10). Haas, Meyer, Assef, and Lee (1987) carried out the laboratory tests on core samples from 26 airports including bulk density, coefficient of thermal contraction, low temperature stiffness, etc. in order to develop cracking prediction models for design and to assess the feasibility of using newly proposed asphalt specifications for controlling the cracking problems.

The low temperature stiffness of the samples was measured by the direct tensile test method developed at the University of Waterloo. The temperatures used in the test were -34, -17, and 0°C (-29.2, 1.4, and 32°F). The stiffness modulus was calculated as the ratio of failure stress to failure strain. It was found that the low temperature cracking at Canadian airports was significantly affected by the temperature susceptibility of the bitumen or mix, the thickness of the asphalt layer, the minimum temperature, and the coefficient of thermal contraction of the mix. The models for estimating low temperature cracking developed from the variables as mentioned above could operate either with the PVN (penetration-viscosity number) value or with the measured stiffness of the mixes directly.

11). Twenty years after the construction of the Ste. Anne Test Road, Deme and Young (1987) carried out a review study based on previous work. By direct tensile creep tests, the mix stiffness moduli were obtained at low temperatures and long loading times. The mix breaking stress was measured via direct tensile tests using constant rate of strain to failure in the temperature range from -40 to 25°C (-40 to 77°F). A relationship between the failure stress and stiffness modulus of asphalt mix was developed. It was concluded that the tests on the specimens cut from the test section showed an increase in mix stiffness with time, which was considered to be attributed to "age hardening" and "structural hardening" of the asphalt binder, and was judged to contribute to progressive transverse cracking with time.

12). Tam, Joseph, and Lynch (1990) evaluated low temperature susceptibility of recycled hot mixes (RHM) by the direct tensile test (constant rate of strain) and the thermal contraction test (contraction rate measurement). The specimens came from 5 recycling contracts and 5 different locations of highways which were built or recycled from 1981 to 1986. The temperatures were -35, -5, and 21°C (-31, 23, and 69.8°F). Different rates of extension were used for different temperature ranges (3×10^{-2} mm/min for -5°C and -35°C, and 5 mm/min for 21°C) in the tests. The limiting mix stiffness and pavement fracture temperature criteria were analyzed based on the data from the tests. Their conclusions included:

i. Recycled hot mixes were more susceptible to thermal cracking than conventional hot mixes.
ii. Hot mixes recycled at low recycling ratios or using high penetration virgin asphalt cement have better low temperature performance than those with high recycling ratios or using low penetration asphalt cement.
iii. The fracture temperature method was more suitable than limiting stiffness criteria.

13). Joseph, Dickson, and Kennepohl (1992) evaluated the low temperature properties of the polymer-modified asphalt mixes which were used in two test sections (Port Hope and Innisfil) by both direct tension tests (constant rate of strain) and thermal contraction tests at low temperatures. In the direct tensile test, the beam samples (48×48×63 mm) were sawed from Marshall briquettes, and the failure stress, strain, and stiffness of the mixes were obtained. It was found that at Innisfil the polymer Novophalt and Neoprene, and tire rubber used in the asphalt mixes had good resistance to the thermal cracking while the premium asphalt and conventional mix had moderate resistance based on cracking temperature. However, there was no significant difference tested among the stiffness moduli of the mixes used in the Port Hope test section at -35°C (-31°F), and it was suggested that more tests should be done for the temperature range from -35 to 0°C (-31 to 32°F) to properly evaluate the behavior of the mixes at moderate low temperatures.

2.3 Bending Beam Test

Bending beam test as shown in Fig.2.2 generally can be classified as:

- 2-point loading test,
- 3-point loading test, and
- 4-point loading test.

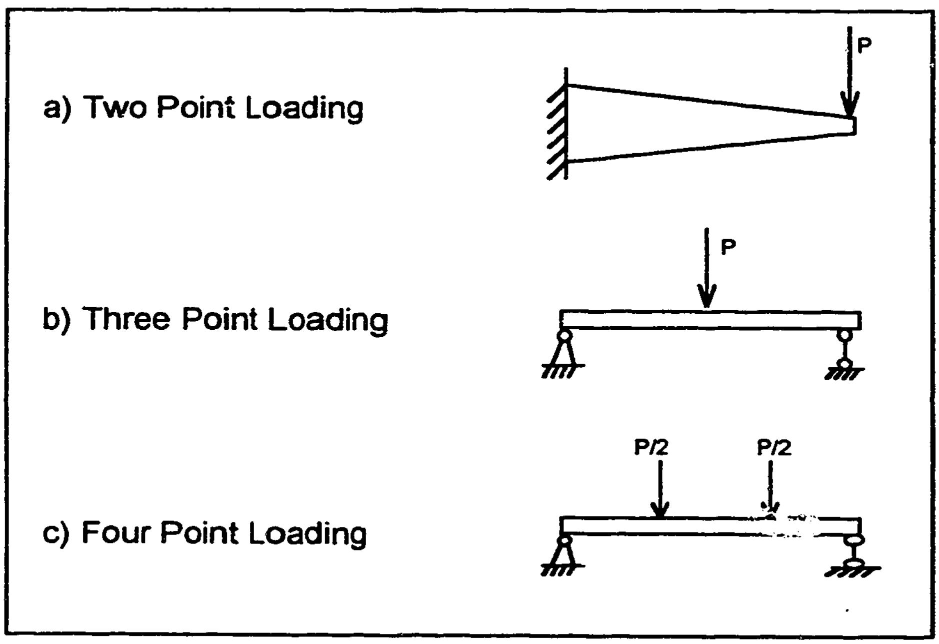


Fig.2.2-Illustration of Bending Beam Test

For the 2-point loading test, sometimes a specimen is used with a continuously increasing cross sectional area from the free end to the fixed end in order to achieve a uniform flexural stress along the beam. For 3-point loading, since the bending moment varies along the beam, the stress also changes. Usually, the middle point of the beam at which the load is applied is the analysis point. The 4-point loading shown in Fig.2.2

produces a uniform bending moment and consequently, uniform flexural stress between the two loading points.

The analyses for 2 and 4-point loading test results are easier than for the 3-point loading test, but the loading equipment for the latter is much simpler than for the former. Additionally, in the 2-point loading test, specimens with varied cross section areas also increases the difficulty of the test. The following is a review on some of the significant studies in bending beam tests:

1). Bahgat and Herrin (1968) attempted to characterize fracture behavior of asphalt mix using "fracture toughness" which was based on the stress analysis at the crack tip and "energy absorbed" by specimen when it broke. The fracture toughness was measured by using 3-point loading test. The load was a moment (an impact load) and the beam specimen was notched with a sharp edge. On the other hand, the energy absorbed was obtained by using a so-called Izod type impact machine, which actually was a 2-point bending beam machine with impact loading. Similarly the notched beam was made with a very sharp edge of the tip of the notch. Only sand sheet asphalt mixes were used in the tests to avoid the influence of the big size aggregates on the fracture property. It was found that temperature, asphalt consistency, asphalt content, and unit weight of mixes had a major effect on the brittle behavior of the asphalt mixes, and that a critical temperature existed below which mixes behaved in a brittle manner.

2). Busby and Rader (1972) adopted the 3-point bending beam to determine the low temperature stiffness modulus and the modulus of rupture (failure bending stress) of asphalt mix. The asphalts used in the mixes were 40-50, 85-100, and 200-300 penetration grades, and the temperatures at which the test was conducted were -37.2, -20.6, and -3.9°C (-35, -5, and 25°F). From the test, it was found that at higher temperatures with softer asphalts, failure came through the asphalt phase, but at lower temperatures with harder asphalts, failure was partly in the asphalt and partly in the aggregate. It was pointed out that the asphalt mix design methods or criteria at that time bore little relationship to the temperature and loading conditions of actual pavement. It was also stated that little attention had been paid to low temperature design criteria, and that the flexural stiffness and modulus of rupture at low temperature appeared to be suitable design criteria. It was found that the modulus of rupture at low temperature was a function of tensile strength of asphalt mix, and the mix with the hardest asphalt cement had the lowest modulus of rupture. A relationship among stiffness, asphalt grade, and temperature was

Furthermore, a chart was given to decide the minimum temperature for a given grade of asphalt or to decide the asphalt grade given the design temperature.

3). Salam and Monismith (1972) carried out fracture and fatigue tests by the bending beam method (4-point loading test) and the direct tensile method. Single-edge-notched specimens were used in the tests. The temperatures were from -28.9 to 20°C (-20 to 68°F) for the fracture test and -12.2 and 20°C (10 and 68°F) for the fatigue test. For the fracture test, four asphalt cements (40-50, 60-70, 85-100, and 120-150) and 15 different asphalt mixes were used with variables of:

(1) asphalt hardness,
(2) asphalt content,
(3) aggregate type,
(4) aggregate gradation,
(5) mineral filler type, and
(6) compaction degree.

The "fracture toughness", K_{IC}, was determined as the material property. For fatigue test, only one asphalt mix was used. It was found from the bending test at cold temperature of lower than -12.2°C (10°F), that

i. The K_{IC} increased with the asphalt content,
ii. The results indicated a peak value of K_{IC} corresponding to an original penetration of about 100,
iii. The fracture toughness increased with increased fineness of grading.
iv. The lower the air voids, the higher the toughness, but the sensitivity was reduced as the temperature increased.
v. The granite aggregate produced a mix with a higher fracture toughness than that containing the limestone aggregate.
vi. The value of K_{IC} in bending was about 15% larger than those in tension.
vii. The crack depth (notch to beam depth ratio) had a significant effect on fatigue life at -12.2°C (10°F) and a lesser effect at 20°C (68°F).

4). By using a 2-point bending beam method, Bonnaure, Gest, Gravois and Uge (1977) developed a so-called more accurate and more applicable method to predict the stiffness of the asphalt mixes from the asphalt stiffness and the volume percentages of asphalt and aggregates than the method suggested by Heukelom and Klomp (1964). In the tests, trapezoidal specimens were used. The load was dynamic (sinusoidal loads). "Twelve typical formulation of asphalt mixes were selected for the tests so as to cover the whole range of mixes for road, air field and hydraulic application. Nine mixes were laboratory prepared, and the other three were taken from the roads open for traffic for several years."

It was found that the temperature, the loading time, the hardness and temperature susceptibility of the bitumen, and the volume percentages of aggregates and bitumen (or air voids) in the mix had predominant influence, and the other parameters played only a secondary part. Based on the analysis of the influence of the variables, a computer program "MODULE" and then a nomograph for predicting the stiffness of bituminous mixes were established. The stiffness of bitumen could be obtained from the nomograph developed by van der Poel (1954). The prediction method was considered valid only when the stiffness of bitumen was larger than 5×10^6 N/m^2.

5). Goodrich (1991) studied fatigue life, high and low temperature properties of asphalt mixes, and rheological properties of asphalt cement. The 4-point loading test method was used to study the fatigue life of asphalt mixes. The loading cycle consisted of a 0.1-sec. pulse load followed by a 0.5 sec. rest period (100 cycles per minute). It was found from this test that "the elastic structure within an asphalt provides fatigue resistance in low strain rate conditions," and that "asphalt which maintains low temperature viscous flow properties provide fatigue resistance in high strain rate conditions." The asphalts or modified asphalts, which had lower loss tangents had good low-strain fatigue lives, and soft asphalts which had higher loss tangents had good high-strain fatigue lives.

2.4 Thermal Contraction Test

Thermal contraction tests include **thermal contraction (or expansion) rate measurement** and **thermal stress restrained specimen test** (TSRST). The result of thermal contraction rate test is very important and useful for the analysis of thermal cracking of asphalt mixes. A beam of asphalt mix is usually used as a specimen, and thermal contraction deformation is measured versus temperature. The thermal stress restrained specimen test is a little difficult to conduct. A specimen, usually a beam, is mounted in a frame which is not supposed to displace at all when the temperature decreases. The thermal stress in the specimen increases so that the load cell can measure the thermally induced load accurately. However, it is impossible that the frame has no displacement at all so that the accuracy of the test depends on the mechanical features of the test to minimize the displacement. Literally, this is also a direct tensile test. The difference is just that this test method uses "temperature loading" instead of mechanical loading. Following is a review on the two kind thermal contraction tests respectively.

2.4.1 Thermal Contraction (Expansion) Rate Measurement

Asphalt mixes volume decreases as the temperature decreases. As defined by Jones et al. (1968), the average cubic thermal coefficient of contraction is calculated as:

$$B = \Delta V / (\Delta T \cdot V_o)$$

where:

B = cubic coefficient of thermal contraction,
V = volume change due to temperature change $\Delta T = T - T_o$,
V_o = volume at reference temperature "To".

The linear thermal coefficient of contraction is defined by:

$$\alpha = \Delta L / (\Delta T \cdot L_o)$$

where:

α = linear coefficient of thermal contraction
ΔL = length change due to temperature change $\Delta T = T - T_o$
L_o = length at reference temperature "To"

If asphalt mixes are assumed as isotropic material, then

$$\alpha = B / 3$$

1). A thermal contraction rate test was done by Domaschuk, Skarsgard, and Christianson (1964). The specimens were obtained from an existing pavement. The temperature range in which the test was done was from -51 to 16°C (-60 to 60°F). It was found that the thermal contraction coefficient for the materials tested was constant at 3.15×10^{-5}/°C (1.75×10^{-5}/°F) in the temperature range.

2). Monismith, Secor, and Secor (1965) measured the thermal contraction coefficient for the asphalt mix at the density of 152 lb/ft^3 (2435 kg/m^3). The temperature range was from -18.7 to 18.9°C (-1.7 to 66°F). The result was that the coefficient of the contraction for this kind of asphalt mix was essentially constant, varying from about 2.16 to 2.52×10^{-5}/°C (1.2 to 1.4×10^{-5}/°F) with an average value 2.34×10^{-5}/°C (1.3×10^{-5}/°F).

3). Littlefield (1967) undertook a study on the thermal contraction and expansion characteristics of asphalt mix. He used asphalt cements from 5 sources of 120-150 AC, 6 sources of 85-100 AC, 5 sources of 60-70 AC, and 1 source of RT-11 tar which were from four states in the United States. The temperature range for the tests was from -17.8 to 54.4°C (0 to 130°F). The conclusions were:

i. Temperature range, grade of asphalt, and source of asphalt were the influencing factors on the coefficient of expansion and contraction of asphalt mixes.
ii. In a cycle of heating and cooling, the amount of shrinkage during cooling was more than the amount of expansion during heating, so that heating-cooling cycles caused densification of the specimen beams.
iii. About 70% to 80% of the total expansion of the specimen beams occurred between -17.8 to 15.6°C (0 to 60°F).
iv. The coefficient of the expansion for the specimen beams used in the tests was in the range of 2.38 to 2.93×10^{-5}/°C (1.32 to 1.63×10^{-5}/°F), which was calculated using the straight line portions of the expansion curves.

4). Jones, Darter, Littlefield (1968) conducted an investigation into the basic aspects of thermal expansion and contraction of asphaltic concrete. This study mainly investigated solid and fluid thermal coefficients, transition temperature, and the major factors influencing the coefficients (asphalt content and restraint condition). The materials used to fabricate the asphalt specimens were one source of aggregate and one source of asphalt (85-100 penetration grade). The specimens were fabricated with different asphalt contents from 4.25% to 6.5% (by weight of mix) with the optimum asphalt content 5.25%. By assuming that the air voids had no effect on the thermal contraction of asphalt mix, the theoretical linear thermal coefficient was developed as follows, and the calculated and the measured coefficients were compared.

$$\alpha_{mix} = (V_{ac} * B_{ac} + V_{agg} * B_{agg}) / (3 * V_{mix})$$

where

α_{mix} = theoretical linear thermal coefficient of asphalt mix,
V_{ac} = volume of asphalt in mix,
B_{ac} = cubic thermal coefficient of asphalt, 4.07×10^{-4}/°C (2.26×10^{-4}/°F) for the glassy state,
V_{agg} = volume of aggregates in mix,
B_{agg} = cubic thermal coefficient of aggregates, 3.33×10^{-5}/°C (1.85×10^{-5}/°F) for quartzite aggregate,
V_{mix} = total volume of mix.

The significant findings were:

i. From -23.3 to 60°C (-10 to 140°F), asphalt mix exhibited two different thermal coefficients of expansion which were called **solid** and **fluid** thermal coefficients. A transition temperature was found, above which asphalt concrete exhibited its fluid coefficient, and below which asphalt concrete exhibited its solid coefficient.
ii. The greater the asphalt content, the lower the transitional temperature of mix.
iii. The thermal coefficient of contraction was somewhat greater (4%) than the thermal coefficient of expansion in the solid state and significantly greater in the fluid state under free movement condition.
iv. In the fluid state, thermal expansion was less than thermal contraction so that permanent shrinkage resulted from heating-cooling cycles.
v. The thermal contraction coefficient in the solid state was in the range of 2.11 to 3.69 $\times10^{-5}$/°C (1.17 to 2.05×10^{-5}/°F) corresponding to the asphalt contents from 4.25% to 6.5% of the weight of mixture, and was 2.86×10^{-5}/°C (1.59×10^{-5}/°F) for the optimum asphalt content of 5.25%.
vi. The comparison between the calculated and the measured linear thermal coefficients showed that they were approximately the same for specimens containing optimum asphalt content, but for lean asphalt contents, the experimental coefficients were less

than the theoretical coefficients, and for rich asphalt contents, the experimental were greater than the theoretical.

5). Ellis, Jones, and Littlefield (1969) carried out an investigation to determine if there was any increase in bulk density of samples of asphalt mixes as a result of being exposed to temperature cycles. The test samples which consisted of test samples and control samples were made with twelve mixes. The three variables for the mixes were aggregate absorption, asphalt viscosity, and asphalt content. The test samples were subject to 300 temperature cycles ranging from 10 to 57°C (50 to 135°F), while the control samples remained at room temperature. Finally the bulk densities for all the samples were measured. A factorial design was used and an analysis of variance was conducted in the study. The important findings were:

i. Temperature cycles did densify the samples of asphalt mixes because the bulk densities of the test samples increased significantly while the ones of the control samples did not change much.
ii. Low viscosity asphalt cement, high absorptive aggregate, and high asphalt content caused greater increase in the bulk density of the samples than high viscosity asphalt cement, low absorptive aggregate, and low asphalt content.
iii. Early temperature cycles caused a greater increase in the bulk density of the samples than later temperature cycles.
iv. That asphalt cement was absorbed into the aggregates, and the air voids were reduced resulting in the densification during temperature cycling.

This paper was a continuing study based on previous work. It answered the questions about the densification of asphalt mix after heating-cooling cycles that were unanswered in previous studies.

6). Burgess, Kopvillem, and Young (1971) tested the thermal contraction coefficients for the three asphalt mixes used in the Ste. Anne Test Road. The temperature range was from -40 to 20°C (-40 to 68°F). The coefficients measured were 2.20, 1.90, and 2.03×10^{-5}/°C (1.22, 1.06, and 1.13×10^{-5}/°F) for asphalt mixes 150-200 LVA, 300-400 LVA, and 150-200 HVA respectively.

7). Anderson and Epps (1983) reported thermal expansion tests on the samples from six test sections in West Texas of the USA. The temperature range was from -18 to

21°C (0 to 70°F). The thermal expansion coefficient range tested in the study was from 2.48 to 6.30×10^{-5}/°C (1.38 to 3.5×10^{-5}/°F).

8). Deme and Young (1987) measured the thermal coefficient of the linear contraction of the mixes used in the Ste. Anne Test Road. The tests were undertaken in two laboratories, and the coefficients were found to be 1.8 to 1.9×10^{-5}/°C (1.0 to 1.05×10^{-5}/°F) and 1.5 to 1.7×10^{-5}/°C (0.83 to 0.94×10^{-5}/°F). The temperature range was from -40 to 25°C (-40 to 77°F). It was explained that the differences were attributable to differences in aggregate or mix characteristics since the asphalt thermal coefficient of contraction was independent of its source and type. It should be noted that these coefficients are a little lower than those reported in the previous study (Burgess, Kopvillem, and Young, 1971).

9). Haas, Meyer, Assaf, and Lee (1987) carried out laboratory test on core samples and crack surveys for 26 selected airports. One of the purposes of this study was to correlate cracking frequency with the factors including climate, asphalt property, asphalt mix property, pavement structure, and pavement age. The reported mean value of the contraction coefficients was 1.49 mm/m/°C with the maximum value 1.9 mm/m/°C and the minimum value 1.06 mm/m/°C. This result of 1.49 mm/m/°C (1.49×10^{-3}/°C) is much higher than those customarily reported which are usually on the order of 10^{-5}/°C.

10). Tam, Joseph, and Lynch (1990) carried out thermal contraction tests (contraction strain measurement under restraint condition) for the specimen materials which came from 5 recycling contracts and 5 different locations of highways which were built or recycled from 1981 to 1986. The temperature was from -38 to 20°C (-36 to 68°F). The coefficient ranges were found from 1.21 to 1.98 $\times10^{-5}$/°C (0.67 to 1.10×10^{-5}/°F) for the plant mixes and 1.47 to 2.57×10^{-5}/°C (0.82 to 1.43×10^{-5}/°F) for the laboratory mixes.

Table 2.1 summarizes the results of the asphalt mix contraction coefficients reviewed in this section.

Table 2.1-Summary of the Asphalt Mix Contraction Coefficients

Reference	Contraction (Expansion) Coefficient in Solid State ($10^{-5}/°C$)			Test Condition
	Average	Upper Limit	Lower Limit	
Domaschuk et al., 1964	3.15	/	/	Core sample; T. range: -51 to 16°C
Monismith et al., 1965	2.34	2.52	2.16	Density: 2435 kg/m³; T. range: -18.7 to 18.9°C
Littlefield, 1967	2.66	2.93	2.38	17 different asphalt binders; T. range: -17.8 to 54.4°C
Jones et al., 1968	2.90	3.69	2.11	T. range: -23.3 to 60°C; 1 agg. and 1 asphalt of 85-100 grade; Asphalt content range: 4.25 to 6.5%;
	2.86	/	/	Optimum asphalt content 5.25%;
Burgess et al., 1971	2.04	2.20	1.90	3 asphalt mixes; T. range: -40 to 20°C
Anderson et al., 1983	4.39	6.30	2.48	T. range: -18 to 21°C
Deme et al., 1987	1.7	1.9	1.5	3 asphalt mixes; T. range: -40 to 25° C; Two lab tests
Haas et al., 1987	1.49×10^2	1.90×10^2	1.06×10^2	Core samples from 26 airports
Tam et al. 1990	1.60	1.98	1.21	T. range:-38 to 20°C; For the plant mixes
	2.02	2.57	1.47	For the laboratory mixes

2.4.2 Thermal Stress Restrained Specimen Test

An asphalt mix specimen (e.g. a beam) contracts as the temperature drops. The strain of the specimen due to the thermal contraction can be expressed as

$$\varepsilon = \int_{T_f}^{T_o} \alpha \, dT$$

where:

ε = thermal strain

α = linear coefficient of thermal contraction

T_o = initial temperature

T_f = final temperature

If the specimen is restrained at its two ends, thermal stress is developed and can be expressed as

$$\sigma = \int_{T_f}^{T_o} \alpha\, S(T,t)\, dT$$

where

S(T,t) = stiffness of asphalt mix, a function of loading time "t" and temperature "T".

The major purpose of the thermal stress restrained specimen test is to measure the thermal stress developed in the specimen under various conditions.

1). Monismith, Secor, and Secor (1965) conducted the thermal stress restrained specimen test. The specimens were sawed with dimensions of 25.4×25.4×304.8 mm (1×1 ×12 in.). The frame was made of invar and was surrounded by a constant temperature cabinet. Another small cabinet which could provide temperature variations was so placed that only the specimen with 254 mm (10 in.) of length was encompassed. Thus the temperature change required in the test would not cause the deformation of the frame. It was claimed that "this apparatus is comparatively rigid and is thus capable of practically restraining any deformation that may develop in the asphalt concrete due to temperature change." It was found that practically no stress was developed in the specimen above about 50°F (10°C), and that the measured stresses were comparatively small even as low as approximately 30°F (-1.1°C) when the cooling rate was 25°F/hour (13.9°C/hour). The specimen temperature was compared with that of the surrounding air. It was found that some difference occurred initially between the air and specimen temperatures; however, the difference dissipated at longer time.

2). Hills and Brien (1966) developed a method to predict thermally induced stress in asphalt pavement. In order to validate the method, they performed a test to directly measure the fracture temperature of the beams of asphalt concrete and asphalt. At first, two asphalt mixes with one kind of aggregates, two kinds of asphalts, and only one asphalt content were used in the test, the cooling rate was 10°C/hour. The fused quartz, whose thermal expansion coefficient was so small that the deformation of the frame itself due to the change of temperature could be ignored, was used in the test equipment. The comparison between the predicted and measured results showed that although great precision could not be claimed, a reasonable agreement was achieved by using the stiffness of mix actually measured by constant load creep test for calculation of the cracking temperature. It was also claimed that the use of the quartz frame technique for the purpose of this test was satisfactory. Three more asphalt mixes were tested to investigate the effect of the variations in the binder content. The results of the cracking temperatures showed that the asphalt content had no significant effect on the cracking temperature of the asphalt mix. It was explained that an increase in binder content would increase the coefficient of thermal contraction for a mix but decrease its stiffness. The thermal stress depended on the product of the two values. Thus, a little change in one variable could be offset by a change in the other variable, leaving the thermal stress unaffected.

3). Tuckett, Jones, and Littlefield (1970) conducted an investigation to evaluate the effects of mixture variables on thermally induced stresses in asphalt mixes and to determine to what extent that these stresses changed with thermal cycling. The variables for the mixes which were used to comprise a complete factorial design were asphalt viscosity, asphalt content, and aggregate absorption. The samples were subject to 100 temperature cycles ranging from -9.4 to 46.1°C (15 to 115°F) with the cooling rate 11°C/hour (0.33°F/min) and heating rate 26°C/hour (0.79°F/min). Thermally induced tensile properties were measured. Some of the conclusions were:

i. The film thickness of asphalt in the samples of asphalt mixes at the optimum asphalt contents obtained by Marshall design method provided a bonding effect producing a high maximum stress or high internal resistance and a high rate of stress change with temperature, thereby reducing the susceptibility of the samples to thermal cracking.
ii. The low viscosity asphalt combined with absorptive aggregate produced a higher rate of stress change with temperature.
iii. The maximum thermal stress increased in the samples with temperature cycling.
iv. The samples with low asphalt content cracked significantly more than the samples with high asphalt content.

4). Fabb (1974) divided the methods for determining cracking temperature of asphalt mixes into two categories: indirect methods and direct methods. A indirect method was a method with which the cracking temperature could be obtained from the calculated thermal stress using nomographs and the tensile strength which might be calculated or directly measured. The direct method was a method where the cracking temperature and the thermal stress were both directly measured. The direct method was used in this study. The cooling rates used in the experiment were 5, 10, and 27°C/hour. A factorial experiment was conducted to measure the effect of the factors of mix type (two asphalt concrete mixes: dense-graded mix with 6% asphalt content and gap-graded mix with 8.4% asphalt content), asphalt consistency (two grades by penetration: 50-70 and 80-100), and asphalt rheological characteristics in terms of shear susceptibility. Additionally, three kinds of polymers were added to the 80-100 grade asphalt to test the effect of the polymers. From the study it was found that:

i. Thermal failure temperature was heavily influenced by the properties or rheological properties of the asphalt used in the mix. Low viscosity, low temperature susceptibility, and high shear susceptibility of asphalt were all factors conducive to reducing failure temperature of asphalt mix.
ii. Variations of aggregate and properties had little or no effect on the resistance of asphalt mix to thermally induced fracture.
iii. Increasing the asphalt content of the asphalt mix within practicable limits only slightly reduced its thermal fracture susceptibility.
iv. The failure temperature is independent of the rate of cooling. Then it was deduced that failure occurs when the asphalt attained a critical physical state, rather than when incremental stresses accumulated to exceed the fracture strength of the asphalt mixes.
v. The addition of the synthetic polymers to asphalt mix could reduce the fracture susceptibility.

5). Sugawara, Kubo, and Moriyoshi (1982) performed a thermal stress restrained specimen test in order to study the effects of various factors on the thermally induced stress and thermal fracture behavior of asphalt mixes. The factors included

i. asphalt type: twenty-three types of asphalts with penetration range from 34 to 470 and penetration index range from -1.3 to +3.5 (calculated with penetration at 25°C and ball and ring softening point),
ii. cooling rate: six cooling rates from 3 to 30°C/hour, and
iii. starting temperature: six starting temperatures from 20 to -10°C,

The specimens were 25×25×260 mm. The lowest measurable temperature in the test equipment was set at -39°C at a cooling rate of 12°C/hour. A considerable fluctuation of tested values were found in the study, and it was said that such fluctuations were not avoidable.

In this study, a so-called transition point (Tt), a critical temperature was defined. This critical temperature separated the stress-strain curve into two zones of relaxation and non-relaxation where the curve reached a straight line. The magnitude of the non-relaxation zone was found about 10°C in temperature and 2×10^{-4} in strain.

It was found that asphalts with higher penetration values and higher penetration indexes had lower cracking temperatures. The cooling rate and the starting temperature were found insignificant for the cracking temperatures.

In a later study by Sugawara and Moriyoshi (1984), the same equipment was used, and the effects of some more factors on the thermal properties of asphalt mixes were investigated. These factors were

i. air voids, composition, and type of mixtures: four types of mixes (air voids range from 2.3 to 6.7%) including dense graded, fine graded, and coarse graded asphalt concretes and a stabilized asphalt base material; three levels of asphalt contents (5.3, 5.8, and 6.8%) with air voids of 3%,
ii. three modes of temperature change: from the initial temperature of 10°C, a) cooling down to the appointed temperatures (-14.5, -20.5, -22.0, -26.0, and -27.5°C) and then maintaining this temperature for 20 hours; b) cooling down to the appointed temperatures (-10, -15, -18, -20, -25°C) and maintaining this temperature for 2 hours and then cooling down to the fracture point; c) cooling down to the appointed temperatures (-15, -20, and -25°C) and then warming up to the initial temperature (10°C) and finally cooling down to the cracking temperature,
iii. repeated cooling in various temperature levels and amplitudes: the maximum and minimum temperatures were chosen at three levels including below the transition point, between below and above the transition point, and above the transition point.

From this study, the main conclusions were

i. The higher the density of the mix, the higher the thermal stress developed in the mix.
ii. The asphalt content was not the critical factor.
iii. When a specimen was maintained at a constant low temperature, the thermal stress decreased. In some cases, cracking occurred during the suspending of the temperature at close to the cracking temperature. Compressive stress was generated in the specimen during the warming up of the temperature.

iv. The thermal stress gradually decreased with the increase of the repeated cooling number.

v. Fatigue type of fracture occurred when the minimum temperature was set at close to the cracking temperature; When the temperature was set between below and above the transition point, both fatigue and relaxation type changes of resulting stress were obtained; At temperature higher than the transition point, the relaxation type change was distinguished.

6). Arand (1987) studied the fatigue behavior of asphalt pavements by considering the superposition of thermally induced tensile stresses to the bending stresses caused by traffic. To investigate asphalt behavior at low temperatures, a process controlled testing machine was developed at the Institute for Highway Engineering of the Technical University of Braunschweig. Relaxation tensile tests and thermal stress restrained specimen tests could be done by using this high precision equipment. In the thermal stress restrained specimen test, the thermally induced tensile stress vs. temperature was obtained. It was stated that the fatigue life depended strongly on the hardness of bitumen because the harder bitumens caused greater thermal stresses at the same temperature and that harder bitumens delivered advantages for high temperatures and disadvantages for low temperatures.

Later, Stock and Arand (1993) studied the low temperature properties of the polymer modified binders by using the same equipment for the purpose of investigating the validity of the claim that polymer modified binders improved resistance to low temperature cracking. Another purpose of the study was to determine what measurements could be made on the individual binders in order to indicate their relative performance in relation to low temperature cracking.

The seven binders used in the research included two non-modified asphalt binders and five polymer modified binders with different ways. The penetration range of the binders was from 70-100 to 17-27. By using one gradation of the aggregates and one constant asphalt content of 4.7% by weight of the total mix, the specimens of the asphalt mix were fabricated with a roller compacting and sawing process. The target air void content was 4%. A very high degree of uniformity was achieved within the specimens. Three kinds of primary tests were conducted in the research:

i. The direct tensile test with a constant extension rate of 1 mm/min. within the temperature range -40 to 20°C.
ii. Thermal stress restrained specimen tests with a constant cooling rate of 10°C/hour,
iii. Relaxation test at a constant temperature -10°C.

It was found that the addition of polymers always improved the low temperature performance of a binder, but the effectiveness of the additive could be dependent on the characteristics of the base. It was concluded that the combination of fracture temperature and the tensile strength reserve, which was defined as the difference between the peak tensile strength and the thermal stress when they were plotted against temperature, was capable of differentiating between all the binders in this study. The only single measurement which ranked the binders in the same order as the two parameters was the fracture temperature.

7). King, King, Harders, and Chaverot (1988) reported use of this same thermal stress restrained specimen test to evaluate the low temperature properties of the polymer modified asphalts. The testing device could electronically maintain the specimen length within 0.00001 mm while cooling the sample at a rate of 10°C/hour. Five mixes were used in the tests. In the mixes, different grades of asphalts and both non-polymer modified and polymer modified binders were used. It was found that polymer modification reduced the fracture temperature by 2 to 5°C (3.6 to 9°F) in terms of the mixes studied in the research, and the polymer also enable the specimens to withstand higher tensile stresses before breaking. The low temperature stiffness by using the dynamic test method developed by King et al. (1986) and tensile strength by using the constant rate of strain direct tensile method, were also obtained. The significant results were that the stiffness moduli for the polymer modified mix showed approximately 10% less stiffness at low temperatures and 17% more stiffness at high temperatures than did the conventional mix and that the polymer modified specimens had significantly higher maximum tensile strengths and these maxima occurred at slightly lower temperatures than their unmodified counterparts.

8). Jung and Vinson (1992) conducted the thermal stress restrained specimen test to evaluate the thermal cracking resistance of asphalt concrete mixtures at Oregon State University under a SHRP Contract. Fourteen asphalts, two aggregate types, two degrees of aging, and two air void contents were used in the experiment. The cooling rate was kept at 10°C/hour. The test equipment could sense the contraction of the specimen while cooling, and a computer was able to control the equipment to stretch the specimen back to its original length. In this way, the contraction of the specimen would not be affected by the deformation of the frame of the equipment, and finally, accurate thermally-induced-stress and cracking temperature could be obtained. Based on statistical analysis, the following main conclusions were made:

i. Asphalt type, aggregate type, degree of aging, and air voids content were major factors which have a substantial effect on the low temperature characteristics of asphalt concrete mixtures.
ii. Fracture temperature was most affected by asphalt type, degree of aging, and air void content and by the interaction between asphalt type and degree of aging to a much lesser extent.
iii. Fracture strength was highly influenced by air void content and aggregate type. Asphalt type and the interaction between aggregate type and degree of aging had a minor influence on fracture strength.

9). By using the same equipment as that reported previously, King, King, Harders, Arand, and Chaverot (AAPT, 1993) studied several relationships of the "theoretical cracking temperatures" obtained from the thermal stress restrained specimen test versus the following:

i. the data from penetration (4°C, 60 sec, and 200 g) and ductility (4°C) tests.
ii. Fraass brittle points,
iii. the bending beam stiffness (-15°C and 60 sec) of the binders by using the Bending Beam Rheometer (Bahia et al., 1992)
iv. cracking temperatures predicted by the method from the SHRP Binder Specification (1993) which then used a critical stiffness of 200 MPa at 60 second loading time (The binder stiffness was measured with the Bending Beam Rheometer), and
v. the "direct tensile temperatures" defined as the temperatures at which the fracture strain is 1% and the rate of the extension is 1 mm/min in the direct tensile test on the binders.

The materials included four straight-run penetration grade asphalts (ranged from 40/50 to 180/200). Each of the four grades was modified with a polymer at three levels. Thus, there were a total of 16 binders including both polymer-modified and neat asphalts. A standard aggregate blend was adopted for the mixes used in the thermal stress restrained specimen test which was considered as a "proof test" for predicting thermal cracking temperatures in asphalt pavement.

It was found that the best single indicator of low temperature cracking was the cracking temperatures predicted by the SHRP critical stiffness method (200 MPa at 60 seconds) in which the binder stiffness was measured with the Bending Beam Rheometer. These predicted cracking temperatures were 15°C higher than the "theoretical cracking temperatures" obtained from the thermal stress restrained specimen test. It was also found that the polymer modifier they used significantly reduced the low temperature stiffness of an asphalt. The same test results and conclusions were also presented in a later paper by King and King (Pacific Rim, 1993).

Note: In the Final/3/20/93 SHRP Binder Specification, the maximum creep stiffness was changed to 300 MPa at 60 seconds. This specification has presently been submitted for approval as a Provisional Standard AASHTO Designation MP1 Edition 1A in September 1993.

10). Moriyoshi and Tokumitsu (1993) developed a new test method called the Moriyoshi Breaking Point (MBP) test and modified Fraass Breaking Point (MFBP) test to assess the low temperature behavior of asphalts. Good or consistent correlations of the MBP temperature and the MFBP temperature versus the thermal fracture temperature were obtained. The thermal fracture temperatures were obtained with the thermal stress restrained specimen test. The test involved 13 asphalt binders and 7 mixtures with different compositions. The cooling rate of -30°C/hour was used in the test. It was concluded that the MBP test and the MFBP test were very useful for predicting low temperature cracking of asphalt pavements.

2.5. Other Methods

1). Goodrich (1991) used a **torsion** Dynamic Mechanical Analysis **rheometer** to measure the rheology of the asphalt concrete mixtures. The strain was imposed on the specimen which was a rectangular bar as an oscillatory shear strain which was kept small at low temperatures and increased at higher temperatures, but was kept within the linear visco-elastic region. The temperature range used in the tests was from -40 to 120°C (-40 to 248°F), and the frequency was 0.1 radians/second (0.0159 Hz). Three kinds of gradations of aggregates were used with two dense-graded mixes and one open-graded mix. Both conventional asphalts and polymer-modified asphalts were used in the tests. It was concluded that **torsion** Dynamic Mechanical Analysis **rheometer** provided a revealing way to look at the binder properties within the aggregate mix. It was found that

i. The low temperature (< 10°C or 50°F) rheology of the mixes largely reflected the binder rheology: binders with higher loss tangents yielded mixes with higher loss tangents. However, although thermally induced cracking might be dominated by the asphalt binder, it was strongly affected by the aggregate-asphalt combination. Asphalt mixes had lower loss tangents (more brittle) than the binders used in the mixes.
ii. The middle temperature (10 to 50°C or 50 to 122°F) mix rheology was sensitive to unique properties of binders. Thus binder properties might be expected to have their best chance at improving mid-temperature properties of asphalt mixes.
iii. At high temperatures (>50°C or 122°F), the mix rheology is predominately influenced by the aggregate, and mix stability was best achieved through design rather than binder property changes.

In the DISCUSSION, when answering Zanzotto's question, Goodrich believed that the polymers themselves did not improve the low temperature rheological properties of asphalt cements, and the benefit of the polymers at low temperature was that it allowed the softer asphalts.

2). Valkering and Jongeneer (1991) studied the use of the Acoustic Emission technique, a non-destructive method, for evaluating the properties of asphalt mixes under thermal loading conditions. "Acoustic emission" (AE) is a term normally applied to the generation and propagation of transient waves in materials as they undergo deformation or fracture. The transient waves generated by rapid release of energy in the material could be detected by a sensitive device. The characteristics such as counts, amplitude, duration, shape, and distribution of the events could be determined by a data processing system and could be studied in an attempt to identify the sources of the signals. A large number of counts of the events obtained under heavy loading could mean that heavy loading resulted in damage in the form of an increased number of defect initiation sites corresponding to high stress. This paper reported the AE analysis in temperature cycling tests with restrained asphalt concrete specimens which were rectangular cylinders. The temperature range was from -60 to 90°C (-76 to 194°F). Five types of asphalts were used for asphalt mixes. The rate of the cooling and warming up was 10°C/hour. A quantitative interpretation of the AE data was made, and the results were compared with predictions of the low temperature strengths of the binders. It was concluded that:

i. The acoustic emissions recorded in temperature cycling tests on restrained specimens at low temperatures were ascribed to defect initiation in the brittle binder.
ii. The acoustic emission activity at a given temperature was independent-within limits of the temperature history of the specimen.
iii. The acoustic emission revealed differences between different binders in the dense mix used in the study.
iv. The acoustic emission activity correlated with predicted temperatures for thermal fracture for various binders, and this meant that acoustic emission allowed the determination of the relative strength of an asphalt mix with different binders in a non-destructive way.
v. Acoustic emission indicated that the thermal fracture behavior of asphalt mixes was largely determined by the binder properties.
vi. Acoustic emission in conjunction with temperature cycling gave a procedure to determine the low temperature strength of a binder in the actual mix configuration.

2.6 Indirect Tensile Test

In the indirect tensile test, cylindrical specimens are used, and the loading strips which are placed across the specimen's diameter are compressed under controlled conditions. Then the asphalt mix specimen is deformed and finally broken by the tensile stress which is along the diameter and is perpendicular to the load as shown in Fig.2.3.

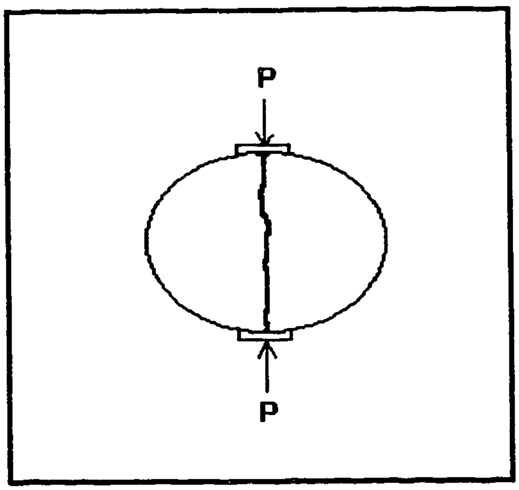
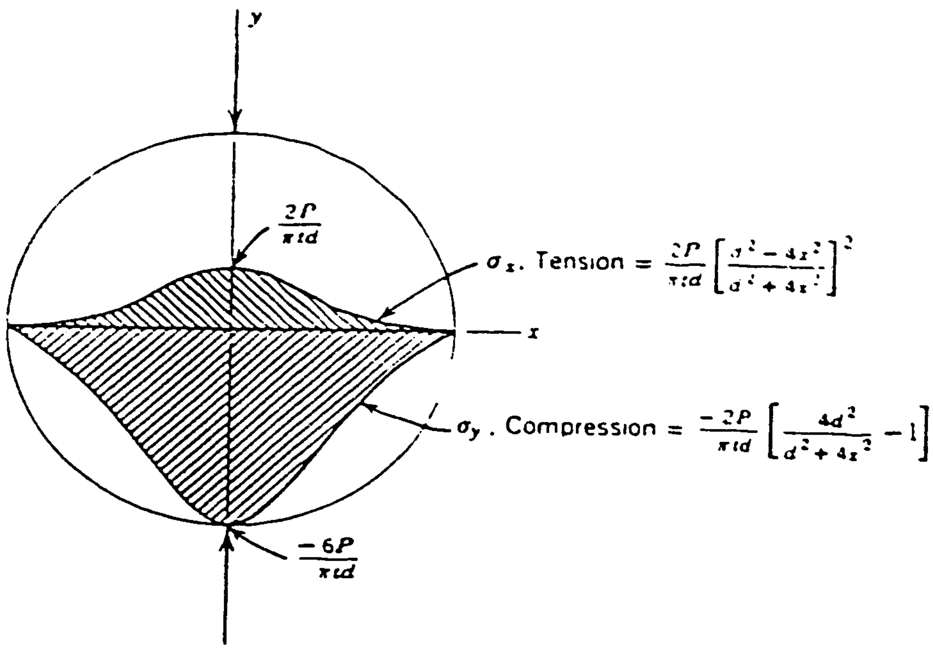
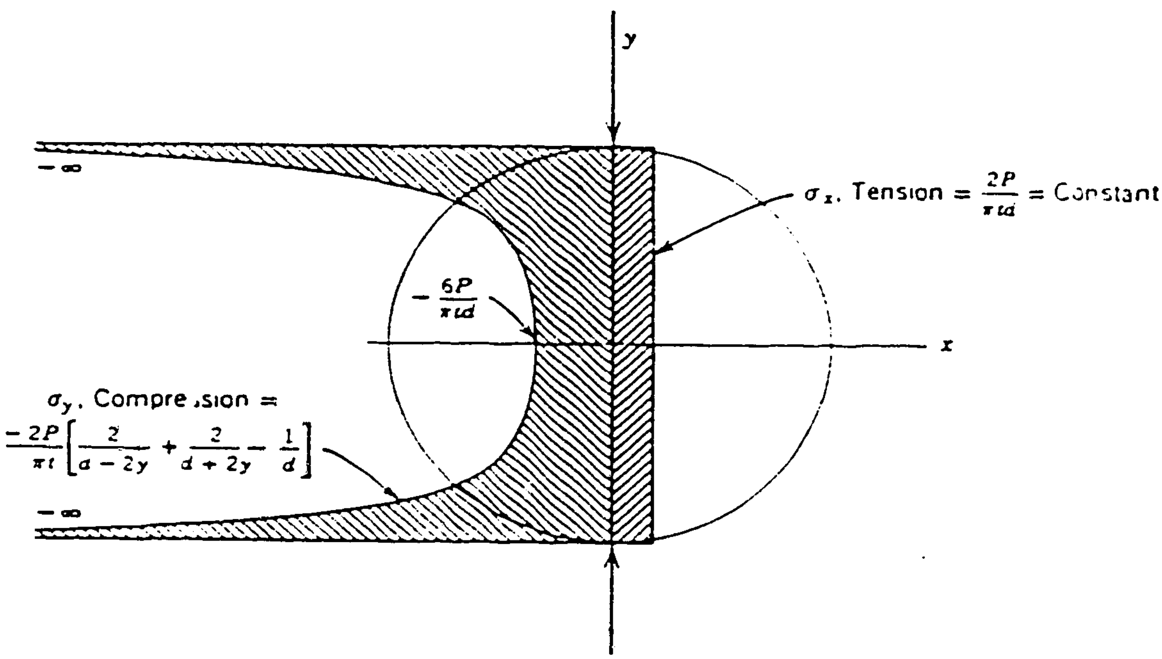


Fig.2.3 Illustration of Indirect Tensile Test

An early use of the indirect tensile test or tensile splitting test was reported by Carniero and Barcellus (1953) from Brazil to determine the tensile strength of cement concrete. The theoretical solution of tensile and compressive stresses for an elastic disk under two opposite concentrated diametrical loads was obtained by Frocht (1948). Yoder (1975) has presented a clear explanation of the theoretical distress distribution as shown in Fig.2.4. Because the concentrated line loading will cause local compressive failure of the specimen, Wright (1955) developed an analysis for the concrete cylindrical specimen under strip-distributed loads as shown in Fig.2.5. Hondros (1959) used this indirect tensile test to evaluate Poisson's ratio and elastic modulus of cement mortars and concrete. Since the study on the application of the indirect tensile test to asphalt mix done by Breen and Stephens (1966), the method has become one of the most important methods to evaluate various properties of asphalt mixes such as low temperature failure stress (failure strain, or failure stiffness), resilient modulus, creep, fatigue, etc.



STRESS DISTRIBUTION ON X-AXIS



HORIZONTAL STRESS DISTRIBUTION ON Y-AXIS

Fig.2.4-Stress Distribution along the Horizontal and Vertical Sections of Symmetry for a Disk under Diametrical Compression of Concentrated Line Loads (Yoder, 1975.)

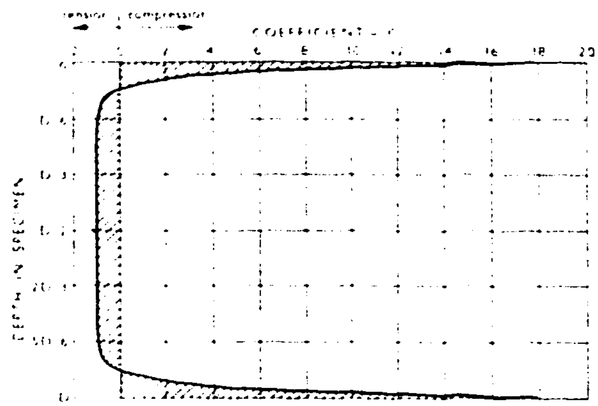


Fig.2.5-Horizontal Stress Distribution along the Vertical Section of Symmetry for a Disk under Diametrical Compression of Strip-Distributed Loads Strip Width = D/12, Stress = 2PC/(πDL) (Wright, 1955)

In the indirect tensile test, LVDTs are usually used to measure the deformations of the specimen. There are two ways to use LVDTs to measure deformations in the indirect tensile test: the interior measuring method and the exterior measuring method. In the interior measuring method, the measuring devices (e.g. LVDTs) are installed or fixed onto the face of the specimen. In the exterior measuring method, the devices are usually installed on a fixed frame which is not connected with the specimen in order to measure total diametrical deformations.

The following section is intended to review the literature in two parts: first, the research work using the indirect tensile test and second, various analysis methods of indirect tensile test data.

2.6.1 Research Work by Using Indirect Tensile Test

1). Breen and Stephens (1966) used the indirect tensile test method to try to evaluate the tensile strength of bituminous concrete. The temperature range used in the tests was from -17.8 to 4.4°C (0 to 40°F). Two types of asphalts (penetration grade 60-70 and 85-100) were used. Asphalt contents of the specimens ranged from 4.5% to 7.0% by weight of aggregate. Due to the uncertainty of the degree of the elastic behavior of the materials tested, the work to failure was studied in the research instead of the calculation of the stresses. It was found that some indentation occurred at 4.4°C (40°F) although this effect disappeared at low temperatures. It was recognized that this indentation could influence the results of the tests. It was also found that

i. The work to failure decreased as the temperature decreased.
ii. At low temperature (< -1°C or 30°F), the effect of asphalt content on work to failure was minor, and at high temperature (4°C or 40°F), the effect was significant.
iii. In contrast to load at failure, the work to failure was greater for 85-100 grade asphalt than for 60-70 grade asphalt.
iv. As asphalt became more brittle due to either lower temperature or harder grade, although the load to failure increased, the ultimate deflection and the work to failure decreased.

2). Anderson and Hahn (1968) developed an indirect tensile test method for measuring failure stress and failure strain of asphalt mixes. Failure strain measured by LVDTs interiorly was expected to be a possible criteria for the resistance of the low temperature cracking of asphalt mixes. Three sources of 200-300 penetration grade asphalts used in a particular project and eight different aggregates were utilized in the study. The specimens were from both laboratory fabrication and field coring. The testing temperature was -17.8°C (0°F), and the loading speed was 1.5 mm/min (0.056 in/min). The curves of failure stress and failure strain, the correlation of failure strain with cracking frequency, and the frequency distributions of the failure strains for different test series were analyzed. It was found that

i. The occurrence of cracking was found to increase as the failure strain decreased, and failure strain was considered the most significant parameter resulting from the test.
ii. Failure strain appeared to be primarily a function of the asphalt supply if aggregate and asphalt grade were same.
iii. Mixes that had high Marshall stability values at 60°C (140°F) generally had low failure strains at -17.8°C (0°F).

3). Hadley, Hudson, and Kennedy (1969) conducted indirect tensile tests and studied the factors influencing failure stress and horizontal deformation of asphalt treated materials. Deformations (both horizontal and vertical) were measured using the exterior method. The testing temperature was 25°C (77°F), and the loading speed was 50.8

mm/min (2 in./min). The loading strip used in the test was one inch wide with the middle 0.5 inch of the strip composed of a curved section with a radius of 2 inches and two 0.25 inches of tangent sections. A pre-loading procedure was used to prevent the impact of initial loading and to minimize the seating of the loading strip with the specimen. Eight factors were evaluated at two levels including:

- Aggregate type: crushed limestone and Seguin rounded gravel,
- Aggregate gradation: fine and coarse,
- Asphalt viscosity: AC-5 and AC-20,
- Asphalt content: 3.5% and 7.0%,
- Compaction type: impact and gyratory-shear,
- Mixing temperature: 121.1°C and 176.7°C (250°F and 350°F),
- Compaction temperature: 93.3°C and 148.9°C (200°F and 300°F),
- Curing temperature: 4.4°C and 43.3°C (40°F and 110°F).

The main effects for tensile strength were: aggregate type, asphalt viscosity, asphalt content, compaction type, mixing temperature, and compaction temperature. The main effects for horizontal deformation were: aggregate type, aggregate gradation, asphalt content, mixing temperature, and compaction temperature.

4). Hadley, Hudson, and Kennedy (AAPT 1970) studied the correlation between the indirect tensile properties and the stability values and Cohesiometer values from the Hveem Stabilometer and Hveem Cohesiometer respectively. Failure stress, failure strain, Poisson's ratio, and moduli of elasticity were studied while compared with both stability and Cohesiometer values. The temperature was 23.9°C (75°F) for the indirect tensile test and 60°C (140°F) for standard Hveem test. The study consisted of two major experiments. In experiment No.1, only one asphalt (AC-10) was used, and three factors (aggregate type, gradation, and asphalt content) with two levels were investigated. In experiment No.2, seven factors were investigated with two levels for each factor including aggregate type and gradation, asphalt viscosity and content, and the temperatures for mixing, compaction, and curing. It was found that "The correlation between the responses of the indirect tensile test and Hveem tests are dependent upon the confides of the study". It was also found that the following correlations were acceptable for a general range of test conditions:

- modulus of elasticity versus Cohesiometer value,

- tensile strength versus Cohesiometer value,
- tensile strain versus Cohesiometer value,
- Poisson's ratio versus stability, and
- tensile strain versus stability".

In the end of the paper, it was not recommended that further correlation be pursued. The author suggested that time and money could be better be spent on the development of a design method based on the indirect tensile test.

5). Schmidt (1972) developed a method to measure the resilient modulus of asphalt mix by indirect tensile test. This method measured the deformation exteriorly after the author established the theoretical relationship between the elastic strain at the center of the specimen and the deformations measured at the opposite ends of the specimen's horizontal diameter. Three different polymer specimens were subject to both direct tensile test and indirect tensile test. The comparison of resilient moduli from both tests showed that two of the three polymers gave nearly identical values by the two methods at the same stress levels, and the other one gave the values which agreed within about 12%. Furthermore, the four point loading bending beam test, compressive test, and indirect tensile test were performed on asphalt mix specimens. The comparison of the resilient moduli from these methods showed that:

i. The resilient moduli from both direct tensile and compressive tests agreed quite well,
ii. If Poisson's ratio was assumed 0.35, the values from the indirect tensile tests exhibited good agreement with the direct tensile test and compressive test, and
iii. Although the stress levels used in the bending beam test and the indirect tensile test could not overlap because of the equipment limitations, the data from projection of the bending beam test results which were at higher stress levels to the indirect tensile test results which were at the lower stress levels suggested that the agreement between the two test results was quite good.

6). Schmidt (1975) carried out an indirect tensile creep test to measure the stiffness of asphalt mix for the purpose of clarifying the extent to which ASTM tests could be used to predict thermally induced pavement cracking. Ten different asphalt cements with one aggregate and asphalt content of 6.2% were used in the "low void mixes".

ASTM routine tests were performed on the asphalts which were residua from the rolling thin film oven tests (RTFOT) and on the asphalts which were recovered from the specimens of the indirect tensile tests. Using the results of the routine tests with the Heukelom Bitumen Test Data Chart and the van der Poel nomograph modified by Heukelom, an equivalent-stiffness-temperature or limiting stiffness temperature (T_{L1}) at which the 10,000 second asphalt stiffness was 138MPa, was predicted for each asphalt.

The measured stiffness data were analyzed by time-temperature superposition technique, and then the curves of 10,000 second stiffness vs. temperature were plotted. The measured equivalent-stiffness-temperature, T_{L2}, the temperature at which the stiffness of asphalt mixes at 10,000 seconds equaled to 10,300 MPa (1.5×10^6 psi) was decided from the curves. The comparison between the measured T_{L2} and the predicted T_{L1} was made. It was concluded that:

i. T_{L1} estimated from the tests of penetration at 4°C along with penetration at 25°C or with the viscosity at 60°C correlated well with the measured T_{L2} ,
ii. T_{L1} estimated from viscosity at 60°C along with viscosity at 135°C showed no correlation with the measured T_{L2} ,
iii. Poor correlation was shown between T_{L1} calculated with penetration at 25°C along with softening point or with viscosity at 60°C and T_{L1} predicted with two penetration values at 4°C and 25°C, and
iv. There was no correlation between T_{L1} and ductility at 7°C.

7). Vila and Terrel (1975) evaluated the changes in the Poisson's ratio (or strain ratio as called in the paper) as a function of moisture and temperature cycling of asphalt mixes by using the indirect tensile test method. The deformation was measured exteriorly. All the specimens were fabricated by duplication of four field pavements which were selected from four different states of the USA. The temperatures used in the indirect tensile tests were 13°C (55°F) and 23°C (73°F). Before the indirect tensile tests, the specimens were subjected to two conditioning procedures: moisture conditioning and temperature conditioning. The moisture conditioning of specimens included three parts:

a) vacuum saturation in water: under condition of vacuum of 635 mm of mercury for 30 min and then under atmospheric pressure for 30 min,
b) wrapping and bagging the specimens in plastic in preparation for thermal cycling, and

c) temperature stabilizing and lost moisture restoring (after the temperature cycling and before the indirect tensile tests, keeping the specimens in water for 24 hours at the temperature at which they were going to be tested).

The temperature conditioning was the thermal cycling for which the maximum length of exposure was 18 cycles (-18°C to 49°C to -18°C). It was found that vacuum saturation and initial thermal cycling caused changes in the mix which resulted in a drop in tensile strength and embrittlement evidenced by a decrease in the strain ratio and that the strain ratio increased with increasing number of cycles. Therefore, the thermal and freeze-thaw cycling of asphalt concrete had a general degradation effect. It was concluded that the strain ratio as a measure of cohesiveness or integrity was a reasonably good indicator of quality.

8). Ruth (1977) performed indirect tensile creep tests and then developed creep prediction equation which established the relationship among asphalt viscosity, the stress ratio of applied tensile stress to maximum failure stress, and creep deformation rate. The indirect tensile creep tests were conducted using incremental increases in applied load. Two types of asphalts (straight-run and air-blown) which had the same penetration grade and two types of aggregates (highly absorptive and non-absorptive) were used to produce four asphalt mixes. It was found that indirect tensile creep strain rate was dependent on viscosity and stress ratio and did not appear to be influenced by asphalt content, aggregate type, or aggregate gradation. The predicted creep and measured creep were compared, and the developed creep prediction equation was found to give accurate estimates of creep for the mixes used in the study.

9). Noureldin and Manke (1978) studied transverse cracking in Oklahoma by means of indirect tensile tests for the laboratory testing part to evaluate the properties of field core samples. This study dealt primarily with the bituminous components of the pavement and their influence on transverse cracking. Nine test sites with various degrees of cracking were selected for comprehensive study. After the indirect tensile tests, the asphalt binder was recovered from the specimens, and routine tests were performed to evaluate the rheological properties of the asphalts. The indirect tensile tests were done at temperature 0, -5, and -10°C (32, 23, and 14°F), and some preliminary tests were conducted at -20°C (-4°F), but little or no deformation of the specimens was observed at -20°C. The relationships of cracking index vs. failure stress of mix, cracking index vs. failure strain of mix, cracking index vs. failure stiffness of mix, and cracking index vs.

recovered asphalt stiffness were analyzed. Failure strain of mix was recommended as a standard parameter in future mix design. It was found that

i. As temperature decreased, failure stress and failure stiffness remarkably increased, and failure strain remarkably decreased.
ii. The occurrence of transverse cracking increased as failure strain of mix decreased and failure stiffness of mix increased, and the correlation between the results of the indirect tensile tests and the observed degree of cracking was satisfactory.
iii. The recovered asphalt stiffness was significantly correlated with the cracking indexes of the test sites. The stiffer the asphalt, the greater the degree of transverse cracking.

10). Ruth, Schweyer, and Potts (1979) conducted both creep and dynamic indirect tensile tests to determine the rheological properties of ten different asphalt mixes with ten different asphalt cements. Both the exterior measuring method (LVDTs) and interior measuring method (strain gages) to measure the horizontal deformation were tried in the study, and it was found that the direct measurement by using the strain gages bonded to the specimens provided better results. The temperatures used in the tests were 25, 15, 5, and -5°C (77, 59, 41, and 23°F). All the specimens were stored for at least one month prior to testing. The dynamic loading of 0.1 second duration with 0.4 second rest was used in the dynamic indirect tensile tests. Some tests were also performed using loading duration's of 0.025, 0.5, and 1 second. Static creep tests were performed at four more stress levels. At each stress level, the specified loading duration (10, 100, or 1000+ seconds) was maintained. Upon removal of the load, the deformation and strain were monitored for a sufficient time to assure that complete recovery of elastic strain had been achieved prior to preceding with creep testing at the next stress level. A concept of **complex flow** was defined as the slope of the curve of log stress vs. log strain rate. The relationships of mix viscosity vs. asphalt viscosity, complex flow of mix vs. complex flow of asphalt, etc. were analyzed. The authors suggested that both temperature susceptibility and shear susceptibility be considered in the low temperature evaluation of thermal cracking potential of asphalt concrete pavement. The following findings were obtained:

i. Mix viscosity correlated with asphalt viscosity, and the controlling factors in the correlation were the shear rate and complex flow.
ii. Asphalt content and mix density did not appear to affect creep response if the asphalt content and air voids was in a reasonable range.
iii. The complex flows of mixes were essentially the same as those of the asphalts.

iv. The static elastic modulus determined from strain gage measurement correlated extremely well with the asphalt viscosity and well with mix viscosity.
v. The dynamic elastic modulus was related to both asphalt and mix viscosity.

11). Khosla and Goetz (1979) conducted indirect tensile tests to study the tensile characteristics of asphalt mixes. A nested factorial experiment was designed. The factors considered in the study were four asphalt types (with one partially air blown and one high float emulsion), eight temperature levels (from 60 to -23.3°C or from 140 to -10°F), and two loading speeds (50 micro-in. per second and 1,000 micro-in. per second). The analysis of variance was used to evaluate the effects of these factors. The exterior deformation measurement method was used in the tests. An acoustic emission technique was used to detect crack initiation and propagation at low temperatures. The primary conclusions were as follows:

i. The asphalt mix tensile properties were strongly influenced by the asphalt properties.
ii. Both high and low temperature properties of asphalts could be modified by the emulsification process and air-blown process.
iii. The stiffness was a better parameter than stress or strain to characterize the tensile behavior of asphalt mixes.
iv. Temperature was the most significant factor affecting the tensile properties of asphalt mixes.
v. The loading speed had a more pronounced effect on failure stress than failure strain of asphalt mixes.
vi. Acoustic emission techniques were useful in better defining failure point of asphalt mixes at temperatures below room temperature.

12). Dempsey, Ingersoll, Johnson, and Shahin (1980) conducted an investigation to determine the properties of asphalt mixes made with three grades of asphalts(AC-2.5, AC-5, and AC-20) and two types of aggregates (Crushed quarry stone and crushed gravel for coarse aggregates respectively) to project the performance of such mixes in resisting thermal cracking and traffic-associated distress (cracking and rutting). The laboratory testing included conventional tests on the asphalts and aggregates, Marshall mix design tests, and indirect tensile tests which included tests for failure properties and repeated-load tests for resilient modulus. The temperature range was from -40 to 30°C (-40 to 86°F). At different temperature ranges, the loading speed was different: for the higher temperature range from 5 to 30°C (41 to 86°F), higher loading speed of 49.8 mm/min

(0.83 mm/s) was used while for the lower temperature range from -40 to 5°C (-40 to 41°F), lower loading speed of 1.8 mm/min (0.03 mm/s) was used. The horizontal deformation was measured exteriorly in the study. The failure stress, failure strain, and total vertical deformation at failure were obtained and studied. It was found

i. Test temperature and asphalt grade had considerable effect on all the observed properties including failure stress and strain, and total vertical deformation at failure,
ii. The loading speed had a very significant effect on failure stress but not on failure strain or total vertical deformation at failure, and
iii. That aggregate source had no effect on any of these properties.

In the resilient modulus test, two loading times (0.05 and 0.1 seconds) were used. It was stated that the resilient modulus varied widely with temperature, loading time, asphalt grade, aggregate type, and compactive effort. The most significant factor was temperature.

13). Von Quintus, Rauhut, and Kennedy (1982) compared the resilient modulus results from three different test methods including indirect tensile test, unconfined compression test, and confined compression test. The specimens used in the tests were cored from 31 pavements in 5 states of the USA. The mixes varied from dense-graded (3% to 8% of air voids) to open-graded (15% to 20% of air voids). The asphalt types ranged from an AC-5 to an AC-40, and asphalt content ranged from 4% to approximately 8%. Seven different types of aggregates were included in the study. The temperature range for the indirect tensile tests was from -12 to 38°C (from 10 to 100°F). The principal conclusions were:

i. At lower temperature range (< 16°C or 60°F), similar resilient moduli were obtained by the three test methods (i.e. indirect tensile test, unconfined compression test, and confined compression test). However, at higher temperature range (> 16°C), significant difference occurred in the measured resilient moduli.
ii. At the temperature range of higher than 32°C (89.6°F), the indirect tensile test and the confined compression test produced similar moduli, with those from indirect tensile tests generally somewhat higher.

14). Gilmore, Lottman, and Scherocman (1984) used the indirect tensile tests including the measurements of tensile failure stress, resilient modulus, and fatigue life, to examine the effects of moisture and additives on the durability of asphalt mixes. The testing temperature was 12.8°C (55°F). Two different dense graded asphalt mixes and six different kinds of chemical additives were used in the study. Both controlled (no additives) and treated (with additives) specimens were used. Tensile strength ratio TSR (wet strength divided by dry strength), resilient modulus ratio, and comparison between wet and dry fatigue lives were analyzed. "High wet field fatigue lives were found for asphalt mixes having tensile strength ratios of 1.0 or greater". "Such high tensile strength ratio and high wet fatigue lives were provided by additives which act to decrease the moisture sensitivity of the asphalt, cement-to-aggregate bond, and strengthen the wetted asphalt binder matrix, i.e. promote both adhesion and cohesion retention". In order to develop a relationship for predictive purpose, adhesion and cohesion parameters were derived and then calculated from measurements of the tensile strength test. It was stated that the changes of these parameters from additive to additive could provide more information on what the additive had achieved and could give further clarification in the description of moisture damage or moisture resistance. In order to verify the predicted effects of additives on the durability of asphalt mixes under traffic, a number of test sections were constructed in eight states across the USA.

15). Anderson, Leung, Poon, and Hadipour (1986) utilized one aggregate with six asphalts at two grades (85-100 and 200-300 penetration) of three sources in the indirect tensile test. The testing temperatures were -30, -20, -10, and 0°C (-22, -4, 14, and 32°F), and the loading speed was 1.5 mm/min. The curves of failure stress vs. failure strain and the curves of failure stress or failure strain vs. temperature were analyzed after the test. It was found that:

i. Temperature, crude source, and asphalt grade had marked effects on the tensile properties of asphalt mixes.
ii. Temperature susceptibility of asphalts correlated with the low temperature tensile properties of asphalt mixes.
iii. Asphalts from the heavy sources were expected to perform better at low temperature than those from the lighter crude source studied in the paper.
iv. The failure strain of softer asphalts was generally higher than the one of harder asphalts.
v. The failure strain decreased as temperature decreased but at a decreasing rate. There was a critical temperature at which failure strain remained relatively unchanged.

16). Khosla (1986) used the indirect tensile creep test and indirect tensile resilient modulus test to investigate the effects of emulsified modifiers on the characteristics of recycled asphalt mixes. One asphalt (AC-20) was aged in the laboratory. Two emulsified modifiers with different concentrations (high and low) of "high float agent" were formulated from one modifier. The specimens were made from five different mixes which were designed by using five different binders, the virgin AC-20, the aged asphalt, the aged asphalt with the unemulsified modifier, the aged asphalt with high concentration of agent, and the aged asphalt with low concentration of agent. Each of the mixes was added equivalent of 15% of the modifier. The temperatures used for the creep tests were -6.7, 4.4, and 21.1°C (20, 40, and 70°F), and the temperature for the resilient modulus tests was 37.8°C (100°F). By using time-temperature superposition technique and the results of the creep tests, the master curves were obtained. The analyses of the calculated stiffnesses at 10,000 seconds loading time from the master curves and the resilient modulus data showed that the mix with the emulsified modifier with high concentration of agent had the lowest stiffness at low temperature of -17.8°C (0°F) and highest resilient modulus at high temperature of 37.8°C (100°F) in all the five mixes. This meant that this mix had best properties of both high temperature and low temperature. In order to evaluate the aging properties, the mixes except the one with the aged asphalt, were subjected to accelerated aging, and then the specimens were formed, and the creep test was performed on them to measure the viscosity of each mix at 21.1°C (70°F) and at a stable strain rate. The ratios of the viscosities after aging over the ones before aging was analyzed. It was found that the mix with the emulsified modifier with high concentration of agent had the lowest ratio which meant the largest resistance to aging.

17). Yao and Monismith (1986) studied the behavior of asphalt mixes with carbon black as a reinforcing agent by means of the indirect tensile test as well as direct tensile creep test and flexural fatigue test. Three types of asphalt cements (AR-2000, AR-4000, and AR-8000) and two types of aggregates (granite and gravel) were used in the study. The indirect tensile test was conducted to determine the tensile strength of the specimens with AR-2000, AR-8000, and AR-2000 plus 20% of microfiller (carbon black) at temperatures of -28, -17.8, and -6.7°C (-20, 0, and 20°F). The loading speed was 0.625 mm/min. (0.025 in/min.). The temperature range for the direct tensile creep test was from -17.8 to 65.6°C (0 to 150°F) and the temperature used in the flexural fatigue tests was 20° C (68°F). The characteristics of the specimens in terms of creep modulus, indirect tensile strength, and fatigue life were analyzed. Furthermore, the behavior of pavement with carbon black and without carbon black in terms of rutting depth and fatigue distress was analyzed by using the characteristics of the specimens. It was found from the study that at low temperatures, the creep properties of the specimens with carbon black were the same as those of the specimens without the carbon black, and that the fatigue resistance and fracture strength of the mixtures were not adversely affected by addition of carbon-black microfiller. It was concluded that a comparably soft asphalt may be used to mitigate low temperature cracking yet provide improved resistance to rutting at high pavement temperatures. Similar findings were obtained from a similar study done by Khosla and Zahran (1987). In this study, instead of tensile strength, resilient moduli of the specimens were obtained from the indirect tensile tests at temperature range from -17.8 to 60°C (0 to 140°F).

18). Lundy, Hicks, and Richardson (1987) evaluated coarse rubber asphalt mixes by using bulk specific gravity test, Hveem Stabilometer test, and indirect tensile tests including diametrical resilient modulus test and diametrical fatigue test, and indirect tensile constant loading speed to failure test. All of these tests were carried out at 22.5°C (72.5°F). The rubber content was 3%, and the asphalt contents were 7.8 and 5.5% by weight of total mix for the rubber asphalt mix and control mix respectively. The specimens were cored from test section constructed with rubber-asphalt mix and control mix. Field behavior of the test sections was also evaluated. The principal conclusions were:

i. The rubber modified mixes were hardening slightly faster than the control mixes although the moduli of both materials were increased with time.
ii. The expected fatigue life of rubber-asphalt mix would exceed that of the control for any given strain level although the laboratory fatigue lives of both materials were decreasing with time.
iii. The Hveem stability of the rubber asphalt mix was unacceptably low, but there was no evidence of rutting. This would indicate the Hveem stability test was not a valid indicator of field performance for the rubber asphalt mixes investigated.
iv. The rubber asphalt mixes had lower indirect tensile strength than the control mix.

19). Anderson, Hussain, and Jardine (1989) evaluated low temperature properties of some polymer modified asphalts by indirect tensile tests. Two asphalts which were utilized in a test road project were used to fabricate the specimens. One asphalt was conventional 150-200 grade asphalt, and the other was a polymer modified asphalt. The asphalt content was 6% for all the specimens. Because at high temperature, the modified asphalt was harder than the conventional asphalt, the specimens with the modified asphalt were compacted at a higher temperature than the ones with conventional binder so that same air voids and similar sample densities of the specimens with both binders were achieved. The testing temperatures were -30, -20, -10, and 0°C (-22, -4, 14, and 32°F), and the loading speed was 1.5 mm/min. The analysis on the curves of failure stress vs. temperature, failure strain vs. temperature, and failure stiffness vs. temperature showed that the specimens of the polymer modified asphalt mix exhibited higher failure stresses, higher failure strains, and lower failure stiffnesses.

20). Khosla and Zahran (1989) evaluated characteristics of asphalt mixes with and without polymer modified asphalts by using indirect tensile tests (indirect tensile resilient modulus test and indirect tensile fatigue test) and incremental static creep test. The temperature ranges were from 17.8 to 60°C (0 to 140°F) for resilient modulus test, from 28.9 to 60°C (-20 to 140°F) for creep test, and 21.1°C (70°F) for fatigue test. Three types of asphalts (AC-5, AC-10, and AC-20) were used in the study. A patented special vulcanizing process was adopted to polymerize the asphalts, and the polymer modified asphalts were identified as **Styrelf**. The analysis of the performance of a representative pavement such as rut depth, fatigue cracking, and PSI (present serviceability index) together with the test results were analyzed in the paper. It was found that the mixtures made with Styrelf provided higher resilient modulus at higher temperature, were more resistant to low temperature cracking, had less potential for rutting, and had longer fatigue life of the pavement than the ones made with the conventional asphalts.

21). Hugo and Nachenius (1989) studied properties of rubber-asphalt and the mixes with the rubber-asphalt. Experiments consisted of sliding plate rheometer tests for rubber-asphalt and indirect tensile tests before and after freeze thaw tests for rubber asphalt mixes. The area at maximum stress which was the area under the curve of indirect tensile stress vs. strain up to the point of the maximum stress, was calculated as one of the important parameters to evaluate the properties of the rubber asphalt mixes. At the beginning, the temperature of 25°C (77°F) was used in the indirect tensile tests, but it was considered too high to keep the specimen in the elastic state as the indirect tensile test theory required so that further tests were performed at temperature of 5°C (41°F). Both the dry method with which tire rubber crumbs were added directly to the hot aggregate as a non-mineral filler and the wet method with which the asphalt and rubber were pre-blended and added to the hot aggregate as a binder were investigated in the study. The relationships among the different test results were analyzed. It was found that the wet method type mix was notably more spongy than the dry method type mix.

In the indirect tensile test, study was done on the measurement of the tensile deformation. First, exterior measurement method was used to measure the horizontal deformation by using different gage lengths (34 and 10 mm) for the purpose of investigation of the effects associated with the tapering off of the indirect tensile stress towards the outsides of the briquette. Then interior measurement was done by using strain gauges. It was found that the strain gauges were unsuitable for this kind of material, and that the test results from gauge length of 34 mm in the exterior measurement were more consistent than from gauge length of 10 mm, which was considered as a result of the aggregate maximum size of 13 mm. It was concluded that although the indirect tensile tests with gauge length of 34 mm did not succeed in obtaining ex . t strain measurements, useful insights might be gained as to the tensile failure of different ιypes of asphalt mixes.

22). Von Quintus and Kennedy (1989) and Von Quintus (1989) used indirect tensile tests in a project entitled "Asphalt-Aggregate Mixture Analysis System" (AAMAS) which was sponsored by the NCHRP (National Cooperative Highway Research Program). The AAMAS was to develop a system for the laboratory evaluation of asphalt mix, based on criteria related with the performance of pavement which was defined by the distress types of fatigue cracking, thermal cracking, rutting, and moisture damage. In this way, the mixture design and structural design could be tied together and based on the same criteria and parameters. Fig.2.6 is from Von Quintus (1989) called AAMAS flow chart.

From the flow chart (Fig.2.6), it can be seen that the indirect tensile test was considered as a very important method to evaluate the properties of asphalt mix. The performance related properties of asphalt mixes concerned in AAMAS were resilient modulus, creep modulus, indirect tensile failure stress and strain, compressive strength, etc. The resilient modulus and creep modulus were measured in two ways, unconfined uniaxial compressive test which was used to evaluate rutting potential and indirect tensile test which was used to evaluate cracking potential. Each of these properties was given one of the five importance ratings of effect from 0 (no effect) to 5 (dominant effect) on the pavement performance of fatigue cracking, thermal cracking, rutting (permanent deformation), and moisture damage. In this way, the material design and structural design could be combined as an integral system of AAMAS.

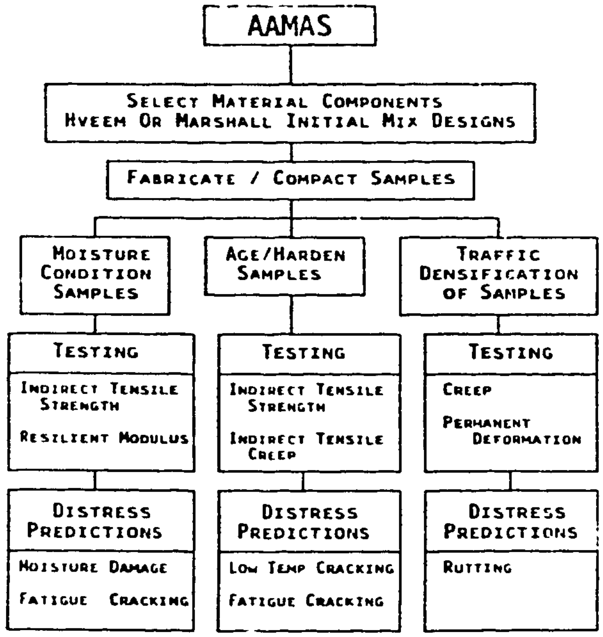


Fig.2.6-AAMAS Flow Chart

23). Ali, Chan, Theriault, Papagiannakis, and Bergan (1991) studied steel slag as an aggregate in asphalt mixes by using indirect tensile tests. The tests included indirect tensile resilient modulus at temperature of 0, 20, and 44°C (32, 68, and 111.2°F), indirect tensile failure stress tests at -30, -20, -10, and 0°C (-22, -4, 14, and 32°F), and indirect tensile creep tests at 0, 20, and 40°C (32, 68, and 104°F). Moisture damage evaluation was also done by indirect tensile resilient modulus tests and indirect tensile constant loading speed to failure tests before and after moisture conditioning of specimens. Eight asphalt mixes which had different percentages of steel slag and other conventional aggregates were designed for the study. Two of the mixes had the same aggregate composition but different asphalt contents. It was found that the mixes with 100% steel slag and Marshall asphalt contents had the highest resilient modulus at low temperature of 0°C (32°F), but at the high temperature of 44°C (111.2°F), there was no significant difference in the resilient modulus of various mixes tested. It was also found that the mixes with steel slag and optimum asphalt contents exhibited lower deformation or higher creep modulus at high temperatures and higher failure stress at low temperatures than did the conventional mixes.

24). Roque and Buttlar (1992) did an three dimension analysis of the indirect tensile test method for asphalt mix specimens and developed an accurate measurement system and analysis procedures to determine properties of asphalt mixes. Both horizontal and vertical deformations were measured and analyzed by LVDTs interiorly in the tests. All kinds of strain measurement methods were discussed in terms of the accuracy of the measurements. Based on three dimension finite element analyses on deformations, stresses, and strains of the indirect tensile specimens, it was found that:

i. There was a significant variation in horizontal tensile stress along the thickness direction.
ii. There was a significant and non-uniform specimen bulging on the face and edges of the specimen.
iii. Poisson's ratio had a significant effect on the stress distribution within the specimen.

Based on the analysis of the effects of local stress concentration near the steel loading strip on the stress states in the vicinity of the center of the specimen's face, it was concluded that significant stress state changes occurring near the loads (and localized damage and material non-linearity) had a negligible effect on the stresses (horizontal and vertical) occurring on the center of the specimen. Furthermore, a method was suggested in the paper to adjust the measured deformation for specimen bulging, to convert average strain determined from a specific gage length to point strains occurring at the center of the specimen face, and to correct stresses calculated from two dimension plane stress solution for three dimension effects.

2.6.2 Review of Various Analysis Methods of Indirect Tensile Test Data

For different purposes, various indirect tensile tests (e.g. resilient modulus test, creep test, and constant loading speed to failure test) were conducted so that the analysis methods are not the same. For different assumptions (e.g. two or three dimension stress state assumptions) the analysis methods are not alike. For various deformation measurement methods, the analysis methods are also different. LVDTs are most extensively used to measure the deformations in the indirect tensile test, so the following review deals with the test and analysis systems using LVDT deformation measurement. Generally speaking, most of the analysis methods of indirect tensile test data fall into following three categories:

(1) Two dimension stress state with exterior deformation measurement;
(2) Two dimension stress state with interior horizontal deformation measurement;

(3) Three dimension stress state with interior deformation measurement.

(1). Two dimension stress state with exterior deformation measurement

For these analysis methods, a two dimension stress state of elasticity is assumed, and horizontal and vertical deformations of asphalt mix specimens are measured exteriorly. Following is a discussion concerning the analysis methods for the resilient modulus test, creep test, and constant loading speed to failure test under this category.

Resilient Modulus Test

The indirect tensile resilient modulus tests was standardized by ASTM in 1982 and issued under designation D4123. It is stipulated in the method that the standard loading strip width is 13 mm (0.5 in) for 101.6 mm (4 in) diameter specimens and 19 mm (0.75 in) for 152.4 mm (6 in) diameter specimens. The temperatures and the loading frequencies used in the test are recommended at 5, 25, and 40°C (41, 77, and 104°F), and at 0.33, 0.5, and 1.0 Hz respectively. This recommended series will result in nine test values for one specimen which can be used to evaluate the overall resilient behavior of the asphalt mixes.

A haversine or other suitable wave form load should be used. Interpretation of the deformation data as shown in Fig.2.7 has resulted in two resilient modulus values being used. The instantaneous resilient modulus is calculated using the recoverable deformation that occurs instantaneously during the unloading portion of one cycle. The total resilient modulus is calculated using the total recoverable deformation which includes both the instantaneous recoverable and the time dependent continuing recoverable deformation during the in loading and rest-period portion of one cycle. The average recoverable horizontal and vertical deformations over at least three loading cycles (as shown in Fig.2.7) are measured after the repeated resilient deformation has become stable.

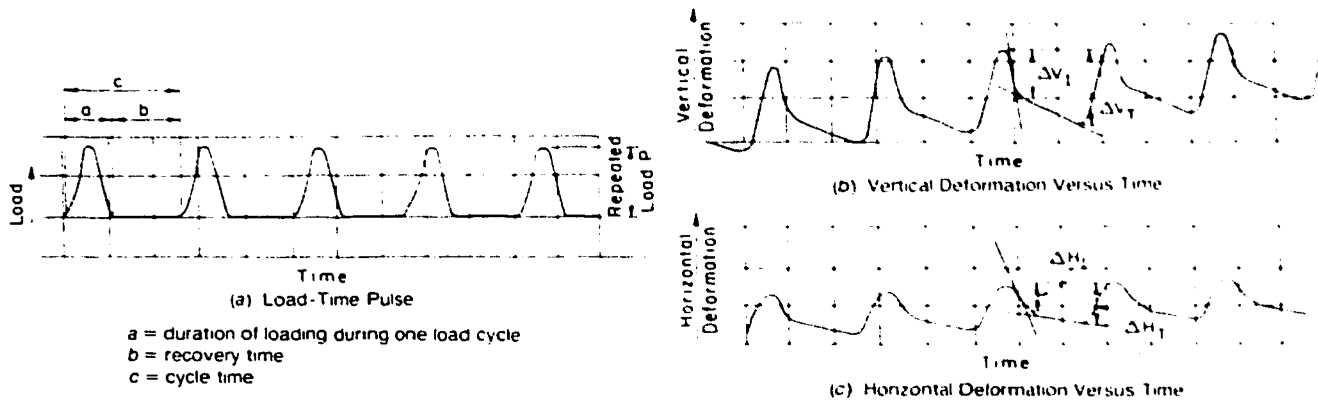


Fig.2.7-Typical Load and Deformation versus Time Relationships for Repeated-Load Indirect Tensile Test (ASTM D4123, 1982)

The following equations are used to calculate the resilient modulus of elasticity and Poisson's ratio:

$$E_{RI} = P\,(\nu_{RI}+0.27) \,/\, (t\,\Delta H_I)$$
$$E_{RT} = P\,(\nu_{RT}+0.27) \,/\, (t\,\Delta H_T)$$
$$\nu_{RI} = 3.59\,\Delta H_I \,/\, (\Delta V_I - 0.27)$$
$$\nu_{RT} = 3.59\,\Delta H_T \,/\, (\Delta V_T - 0.27)$$

where

E_{RI} = instantaneous resilient modulus, MPa (or psi)

E_{RT} = total resilient modulus, MPa (or psi)

ν_{RI} = instantaneous resilient Poisson's ratio

ν_{RT} = total resilient Poisson's ratio

P = repeated load, N (or lbf)

t = specimen thickness, mm (or in.)

H_I = instantaneous recoverable horizontal deformation, mm (or in.)

V_I = instantaneous recoverable vertical deformation, mm (or in.)

H_T = total recoverable horizontal deformation, mm (or in.)

V_T = total recoverable vertical deformation, mm (or in.)

Creep Test

1). Ruth (1977) developed a direct tensile creep test method and an analysis system for the test. In the indirect tensile creep test, loading strips of 0.5 in. width were used.

Load was incrementally increased, but the time duration of the load was decreased. The chart recording speed was increased with increasing stress levels to obtain greater accuracy in strain rate measurement. The Schmidt method (1972) to develop strain, deformation, and modulus by using elastic stress solutions was referred to in developing this analysis system.

By using creep test data and assuming that asphalt mix failure stress reached a constant maximum value when temperature was lower than some transitional temperature, a creep model was developed:

$$\log (d\delta / dt) = \log (1 / n) + 10 + 4 \log (\sigma_a / \sigma_f)$$

where

$d\delta / dt$ = creep deformation rate (in./hour)

n = asphalt viscosity (Pa·s)

σ_a = applied tensile stress (psi or kPa), and

σ_f = maximum failure stress (400 psi or 2758 kPa)

This creep model could be used to predict viscosities of asphalt mixes after creep data were obtained. It was assumed that the stress distribution was unaltered during the initial development of creep strains and the effect of stress on the creep strain was proportional to the creep deformation. The tensile stress and strain calculated by the solution of elastic theory were analyzed. It was found that most of the creep strain had occurred within the center 1.5 in. (38.1 mm) along the x-axis and within 1.2 in. (30.5 mm) along a plane that was displaced 0.5 in. (12.7 mm) from the x-axis. Thus the creep strain rate was predicted as follows by using the average estimated distance over which the creep strain developed:

$$d\varepsilon / dt = (d\delta / dt) / 1.35$$

where

$d\varepsilon / dt$ = creep strain rate

$d\delta / dt$ = creep deformation rate

Finally, the elastic modulus could be calculated from Schmidt's equation:

$$E_s = P\ (\nu + 0.2374)\ /\ (t\ \delta_h)$$

where

E_s = static stiffness modulus

ν= Poisson's ratio, (A value of 0.35 was used)

P = applied load

t = specimen thickness

δ_h = total elastic horizontal deformation

2). Khosla (1986) used a steady sustained load at a given temperature up to a loading time of 1,000 seconds in his indirect tensile creep test. The temperatures used for the creep tests were -6.7, 4.4, and 21.1°C (20, 40, and 70°F). The stress and strain were calculated by using elastic theory solutions. Time temperature superposition technique was used, and master curves of the asphalt mixes were developed. These master curves permitted the stiffness at much greater ranges of temperature and loading time to be determined.

Constant Loading Speed to Failure Test

1). The Transportation Research Board (1975) published a Test Procedure for indirect tensile test. In this procedure, the loading strip width is 12.7 mm (0.5 in.). The loading speed is 0.84 mm/s (2 in./min), which is supposed to simulate rapidly applied pavement loading. Horizontal deformations are measured by using a device basically consisting of two cantilevered arms with attached strain gauges. Deformation of the specimen or deflection of the arms at points of contact with the specimen are calibrated with the output from the strain gauges. Vertical deformations are measured by a LVDT. From this test, Poisson ratio, modulus of elasticity, and failure stress and failure strain can be estimated. Following are the simplified equations for the calculations suggested in the Test Procedure:

Tensile Property	4 in. Diameter Specimen	6 in. Diameter Specimen
Failure Stress, σ_R, (lbf/in^2)	$0.156\ P_{fail} / t$	$0.105\ P_{fail} / t$
Failure Strain, ε_R	$\dfrac{0.1185\ \upsilon + 0.03896}{0.2494\ \upsilon + 0.0673} X_{TF}$	$\dfrac{0.0793\ \upsilon + 0.02636}{0.1665\ \upsilon + 0.0452} X_{TF}$
Poisson's Ratio, υ	$\dfrac{0.0673\ DR + 0.8954}{0.2494\ DR + 0.0156}$	$\dfrac{0.04524\ DR + 0.6804}{0.16648\ DR + 0.00694}$
Elastic Modulus, E, (lbf/in^2)	$S_H\ (0.9976\ \upsilon + 0.2692) / t$	$S_H\ (0.9990\ \upsilon + 0.2712) / t$

where

P_{fail} = total load at failure

t = specimen thickness

X_{TF} = horizontal deformation at failure

DR = deformation ratio (Y_T / X_T) = the slope of line of best fit between vertical deformation Y_T and the corresponding horizontal deformation X_T up to P_{fail}

S_H = horizontal tangent modulus (P / X_T) = the slope of the line of best fit between load P and X_T for loads up to P_{fail}

2). Khosla and Goetz (1979) used LVDTs to measure the horizontal deformation, and used following equation to calculated the failure stress

$$\sigma_R = 2\ P\ [\sin(2\ \alpha) - a / (2\ R)] / (\pi\ a\ t)$$

where

σ_R = failure stress

P = total load at failure

α = angle subtended by one-half the width of loading strip

a = width of the loading strip

R = radius of the specimen

t = thickness of the specimen

For $a = 0.5$ in.; $R = 2$ in., $2\ \alpha = 14.29°$, the equation is simplified

$$\sigma_R = 0.1556\ P / t.$$

This equation is same as the one from the TRB Test Procedure mentioned above.

The failure strain (horizontal) and mix stiffness are calculated as follows

$$\varepsilon_R = X_{TF} / L = X_{TF} (0.1559 + 0.4742\ \upsilon) / (0.2692 + 0.9976\ \upsilon)$$
$$Smix(t,T) = \sigma(t,T) / \varepsilon(t,T)$$

where

ε_R = failure strain
X_{TF} = horizontal deformation at failure
L = gauge length
υ = Poisson's ratio
Smix(t,T) = mix stiffness
$\sigma(t,T)$ = indirect tensile stress
$\varepsilon(t,T)$ = indirect tensile strain

3). Hadley, Hudson, and Kennedy (Research Report 1970) at the University of Texas at Austin developed a testing and analysis system. The loading strip used in their indirect tensile test was 25.4 mm (1 in.) wide with the middle 12.7 mm (0.5 in.) of the strip composed of a curved section with a radius of 50.8 mm (2 inch) and two 6.4 mm (0.25 in.) of tangent sections. Testing temperature was 25°C (77°F). Loading speed was 50.8 mm/min (2 in./min). For this special loading strip and a specimen with 101.6 mm (4 in.) diameter, the **stresses at the center of the specimen** were calculated as follows:

$$\sigma_t = 0.14734\ P / t$$
$$\sigma_c = 0.46906\ P / t$$

where

σ_t = tensile stress at the specimen center
σ_c = compressive stress at the specimen center
P = load
t = thickness of the specimen

The **elastic modulus and Poisson's ratio** can be calculated as follows:

$$E = P\ (A_{rx} / P - \upsilon\ A_{0x} / P) / X_t$$
$$\upsilon = (A_{ry} + R\ A_{rx}) / (R\ A_{0x} + A_{0y})$$

where

E = elastic modulus

υ = Poisson's ratio

A_{rx}, $A_{\theta x}$, A_{ry}, and $A_{\theta y}$ = constants which can be obtained by integrations

P = load

$R = Y_t / X_t$

X_t = total tensile deformation at horizontal direction

Y_t = total compressive deformation at vertical direction

(2). Two dimension stress state with interior horizontal deformation measurement (University of Alberta Method)

A testing and analysis system has been reported at the University of Alberta. **Two dimension stress state of elasticity** is assumed. LVDTs are used to measure **horizontal** deformation **interiorly** with gauge length of 25.4 mm (1.0 in.) and vertical deformation exteriorly for standard Marshall 101.6 mm (4 in.) diameter specimens. Loading strip width is 12.5 mm (0.5 in.). The Constant loading speed was 1.5 mm/min. The temperature of -17.8°C (0°F) was used in the early work done by Anderson and Hahn (1968). More research work reported by Anderson et al (1986; 1989) used the temperatures of -30, -20, -10, and 0°C (-22, -4, 14, and 32°F). Failure stress, failure strain, and failure stiffness are obtained from the test.

Before starting the indirect tensile test, the grouping of the test specimens must be done according to their bulk specific gravities by a program called "Grouping". In this way, the differences in mean specific gravities among all groups of specimens which are going to be tested at different temperatures is minimized. The indirect tensile failure stress, failure stain, and failure stiffness are calculated as follows:

$$\sigma_R = 2\,P / (\pi\, t\, d)$$

$$\varepsilon_R = D / 25.4$$

$$S_R = 0.912\, \sigma_R / (0.5\, \varepsilon_R)$$

where

σ_R = indirect tensile failure stress, MPa

ε_R = indirect tensile failure strain averaged within the gauge length

S_R = indirect tensile failure stiffness, MPa

P = ultimate applied load required to fail a specimen, kN

t = specimen thickness, mm

d = specimen diameter, mm

D = horizontal deformation, mm

(3). Three dimension stress state with interior deformation measurement (Pennsylvania State University's Method)

Pennsylvania State University (Roque and Buttlar 1992) has expanded the testing system of the University of Alberta and has developed a three dimension stress state of elasticity testing and analysis system. This testing system measures both horizontal and vertical deformations interiorly by using LVDTs with gauge lengths of 25.4 mm (1.0 in) for 101.6 mm (4 in) diameter samples and 38.1 mm (1.5 in) for 152.4 mm (6 in) diameter samples. The three dimension stress state analysis system is a correction of the two dimension plane stress state analysis system based on a three dimension finite element analysis on the deformation, stress, and strain of indirect tensile specimens.

The calculation of the tensile stress, strain, elastic modulus, and Poisson's ratio can be done as follows:

$$\sigma_{xCORR} = 2\ P\ C_{\sigma xCTR} / (\pi\ t\ d)$$

$$\sigma_{yCORR} = 2\ P\ C_{\sigma yCTR} / (\pi\ t\ d)$$

$$\varepsilon_{CTRx} = X_M\ C_{Bx}\ C_{\varepsilon x} / GL$$

$$\varepsilon_{CTRy} = Y_M\ C_{By}\ C_{\varepsilon y} / GL$$

$$E = (\sigma_{xCORR} - \upsilon\ \sigma_{yCORR}) / \varepsilon_{CTRx}$$

$$\upsilon = \frac{(\sigma_{xCORR} - \sigma_{yCORR}\ \varepsilon_{CTRx} / \varepsilon_{CTRy})}{(\sigma_{yCORR} - \sigma_{xCORR}\ \varepsilon_{CTRx} / \varepsilon_{CTRy})}$$

where

σ_{xCORR} = corrected horizontal point stress occurring at the center of the specimen's face

σ_{yCORR} = corrected vertical point stress occurring at the center of the specimen's face

ε_{CTRx} = corrected horizontal point strain at the center of the specimen's face

ε_{CTRy} = corrected vertical point strain at the center of the specimen's face

E = Hooke's elasticity modulus

υ = Poisson's ratio

$C_{\sigma xCTR}$ = correction factor applied to the horizontal point stress occurring at the center of the specimen's face as predicted by two dimension stress solution to account for three dimension effects

$C_{\sigma yCTR}$ = correction factor applied to the vertical point stress occurring at the center of the specimen's face as predicted by two dimension stress solution to account for three dimension effects

$C_{\sigma xCTR}$ and $C_{\sigma yCTR}$ can be obtained from Table 1 in (Roque and Buttlar 1992)

$C_{Bx} = X / X_M = 1.01 - 0.12\,\upsilon - 0.05\, t / t_{STD}$

C_{Bx} = correction factor applied to the measured horizontal deformation to correct for specimen bulging

$C_{By} = Y / Y_M = 0.994 - 0.128\,\upsilon$

C_{By} = correction factor applied to the measured vertical deformation to correct for specimen bulging

$C_{\varepsilon x}$ = 1.07, correction factor applied to the average strain determined from the corrected horizontal deformation measurements to obtain the horizontal point strain occurring at the center of the specimen's face

$C_{\varepsilon y}$ = 0.98, correction factor applied to the average strain determined from the corrected vertical deformation measurements to obtain the vertical point strain occurring at the center of the specimen's face

X = corrected horizontal deformation

Y = corrected vertical deformation

X_M = measured horizontal deformation

Y_M = measured vertical deformation

GL = Gauge length

P = total load applied to specimen

t = measured specimen thickness

t_{STD} = 0.625 d; t_{STD} is standard specimen thickness: 63.5 mm (2.5 in) and 95.25 mm (3.75 in) for 101.6 mm (4 in) and 152.4 mm (6 in) diameter specimens respectively

d = specimen diameter

An iterative program must be used to solve the required parameters of Poisson's ratio, stress, strain, and elasticity modulus because the Poisson's ratio is needed to calculate the other parameters, and the other parameters are also needed to calculate the Poisson's ratio.

This testing and analysis system can be used for resilient modulus test, creep test, and constant loading speed to failure test. This system has now been standardized by SHRP and called SHRP Protocol (1020).

2.7 Summary

From the literature review, it is apparent that there are various ways and many parameters to evaluate the low temperature characteristics of asphalt mixes. But for the study of low temperature cracking of asphalt pavements, the most important parameters to characterize the asphalt mix behavior at low temperatures are thermal contraction (or expansion) coefficient, tensile strength, and tensile stiffness modulus.

From Table 2-1, it can be seen that the thermal contraction coefficient test results vary quite a bit. They cannot be compared directly because the test method has not been standardized. It is necessary and urgent to standardize the test method so that the accuracy of the test results from different sources can be examined.

The ideal method to obtain the tensile strength and stiffness modulus is the direct tensile test. The constant rate of strain direct tensile test can be used for obtaining both tensile strength and stiffness of asphalt mixes. In this test, the tensile strength is influenced by the rate of strain, and it is difficult to decide an appropriate rate of strain for the study of low temperature cracking. The curve of tensile strength vs. stiffness like the chart developed by Heukelom (1966) has avoided this problem. The direct tensile creep test and relaxation test can also be adopted to measure the tensile stiffness modulus, and the creep test is usually much easier to be carried out.

However, since the direct tensile test is not always available due to various reasons, the indirect tensile test and the bending beam test can also be used for the same purposes. Generally, the indirect tensile test is much easier to perform. Furthermore, if no direct tests can be done, the commonly accepted nomograph methods are the final choice.

If the indirect tensile test or the bending beam test is the choice, then care should be taken on how the test results are related to the direct tensile test data. If there are no direct tensile test data which can be used, the results from these tests should be compared with the ones from the commonly accepted nomograph methods. Sound engineering judgment must be used in these cases.

As for the indirect tensile test, the three dimensional stress state assumption with interior deformation measurement test and analysis system is the most accurate among all the systems discussed above but it is much more complicated in both test and analysis. Table 2.2 which is from Roque and Buttlar (1992) shows that the two dimensional stress state assumption with interior deformation measurement system is good enough for engineering use when considering that at low temperatures (< -10°C) the error of the deformation measurement is much more than the errors listed in the table.

Table 2.2-Two Dimension Analysis Method Compared with Three Dimension Analysis Method (Roque and Buttlar 1992)

		Input υ	Deflections (0.001 in.)		Assume υ = 0.35 (no vertical measurement)		Use H and V to Compute υ			
	Model Type*		HM	VM	E (ksi)	%ERR	υ	%ERR	E (ksi)	%ERR
Interior Measurements	1	0.20	1.05	-2.09	271	+36	.203	+1.5	201	+0.5
		0.35	1.42	-2.27	204	+2.1	.357	+2.1	201	+0.5
		0.45	1.66	-2.38	177	-11	.456	+1.3	201	+0.5
	2	0.20	1.05	-2.09	248	+24	.204	+2.1	195	-2.4
		0.35	1.42	-2.27	183	-8.3	.372	+6.3	189	-5.3
		0.45	1.66	-2.38	157	-21	.476	+5.8	186	-7.0

*Model Type — Description

1. Interior measurements, 2-D plane stress solution corrected for 3-D effects (Corrections involve applying factors for (1) bulging induced measurement errors, (2) conversion from 2-D to 3-D stress state at measurement location and (3) conversion from average strain across gage length to point strain at center of specimen.)
2. Interior measurement, 2-D plane stress assumptions used for analysis.

Presently, the Indirect Tensile Creep and Strength Test method (SHRP 1993) developed by Pennsylvania State University (Roque and Buttlar 1992) has been validated by SHRP researchers and has been submitted to AASHTO for review. This test method is an integral part of the Superpave system developed in SHRP. The calculation of the cracking temperature and frequency of a pavement by using this Superpave system needs the material properties (creep compliance curves and tensile strengths) obtained from this indirect tensile test.

Another validated test method by SHRP researchers is the Thermal Stress Restrained Specimen Test method (SHRP 1993). This method has also been submitted to AASHTO for review and has been published as a provisional standard. This test mimics the low temperature cracking of pavements, and the results of the test correlate closely with the cracking temperatures. Therefore, the asphalt mixes that can accommodate the local extreme low temperatures can be selected or designed based on the results of the test. The range of the cracking temperatures which can be measured by this test method is from -50 to 10°C. Usually, the curve of thermal stress vs. temperature and the curve of tensile strength vs. temperature can be obtained from this test.

CHAPTER THREE

INDIRECT TENSILE TEST AND ANALYSIS FOR RECYCLED TIRE RUBBER ASPHALT MIXES

The indirect tensile test procedure practiced at the University of Alberta is a constant loading speed to failure test. This procedure has been used most recently by Hussain (1990) and has also been used in the study of "Low Temperature Testing of Recycled Tire Rubber in Asphalt Concrete Pavements" (Anderson 1992, and EBA Engineering Consultants Ltd. 1993). A revised test data analysis procedure is presented in the following discussion by taking data from this Asphalt Rubber Project as an example.

3.1 Testing Equipment, Materials, and Conditions

3.1.1 Testing Equipment

The testing equipment is listed as follows and shown in Fig.3.1.

In the cold room:

- Compression Machine: Tristar 5,000 kg Stepless Compression Machine.
- Load Cell: F.W.L. Load Cell, Type: LC-5, Capacity: 5 ton.
- Horizontal LVDT: Trans-Tek-0025, Range: ±0.25 in.
- Vertical LVDT: 7DCDT-250.
- Loading Strips: Width = 12.5 mm (Details in Hussain 1990).
- Temperature Control System.

In the recording room:

- IBM compatible computer.
- Signal Conditioner and Digital Volt Meter.
- Thermal Electric Pyrometer Indicator and Digital Thermometer.

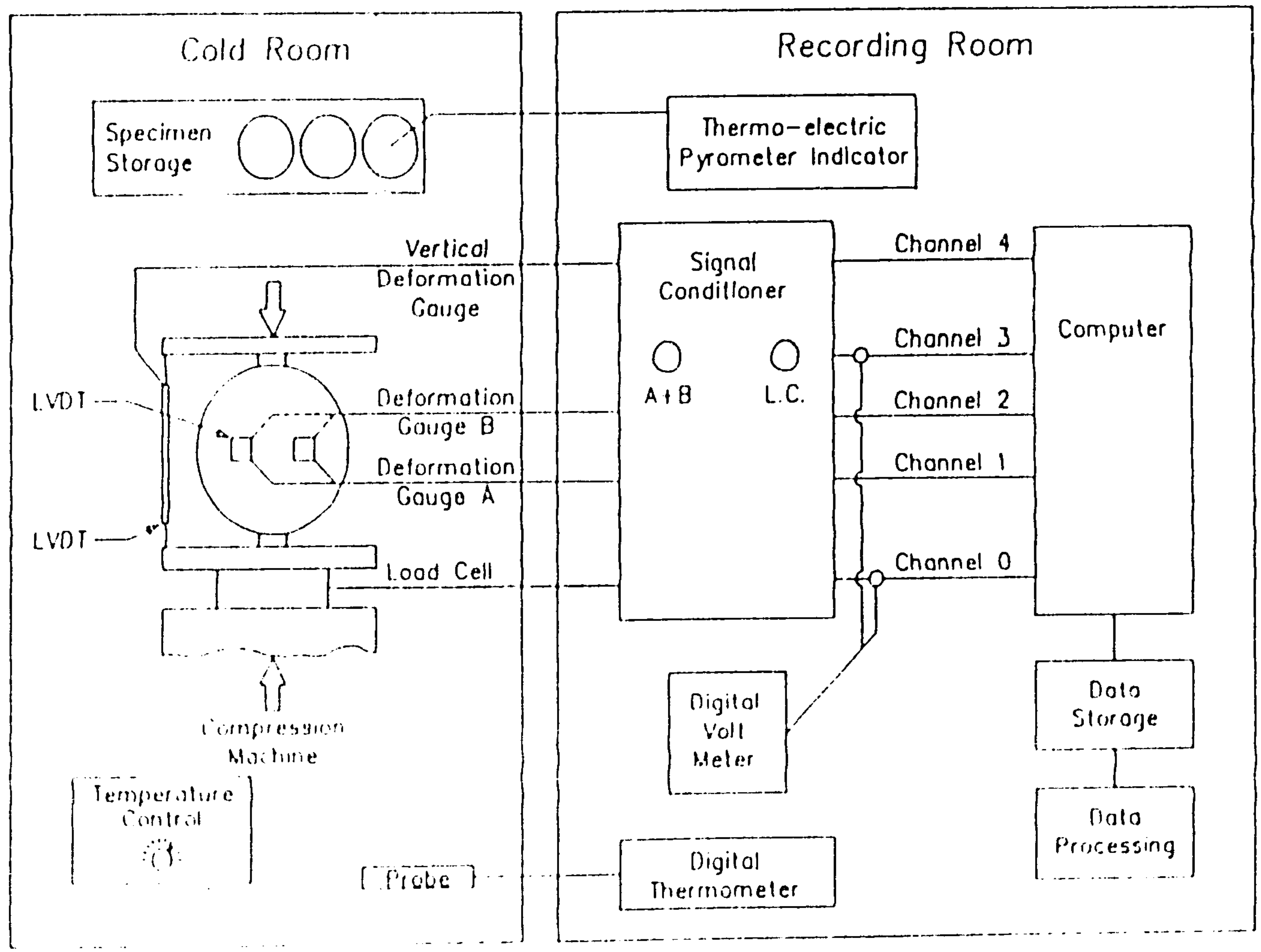


Fig.3.1-Schematic of Test Equipment Layout (Hussain 1990)

3.1.2 Testing Conditions

The temperatures normally used in the indirect tensile test are 0, -10, -20, and -30°C (32, 14, -4, and -22°F). The loading speed is 1.5 mm/min, although it may be changed if desired.

3.1.3 Materials

The indirect tensile test for the project of The Use of Recycled Tire Rubber in Asphalt Concrete Pavements consists of testing a control mix, a series with 10% rubber added to the asphalt cement using a "wet procedure" and a series with 1% by weight added to the aggregate as a "dry" procedure. The aggregate gradations used in the study are shown in Table 3.1, the tire rubber gradation is shown in Table 3.2, and the Marshall mix design parameters are shown in Table 3.3.

Table 3.1-Aggregate Gradation for Rubber Asphalt Mixes

Sieve Size (Approximate Equivalents)		Blackfalds Coarse Aggregate	Willow River Sand	Ball Blend Sand	Combined Gradation (73, 16, & 11%)	Combined Gradation (85, 15, & 0%)
Metric CGSB (μm)	AASHTO (M 92)	Percent Passing (%)				
16,000	5/8	100			100	100
10,000	3/8	66	100		75	71
5,000	#4	38	99		55	47
1,250	#16	21	84	100	39	30
630	#30	18	64	99	34	27
315	#50	13	26	93	24	15
160	#100	9.1	11.2	46.9	13.6	9.4
80	#200	6.4	6.3	9.9	6.8	6.4

Table 3.2-WTP-10 Tire Rubber Gradation

Sieve Size (μm)	5,000	2,000	1,250	800	400	250	160	063
% Passing	100	99.8	63.6	33.2	12.5	5.9	2.6	0.2

Note: WTP means Whole Tire Product and the numerical value (10) relates to U. S. standard sieve size (#10, 2 mm) of the maximum particle.

Table 3.3-Rubber Asphalt Mix Design Parameters

Parameters	1% WTP "Dry" Procedure	10% WTP "Wet" Procedure	Control
Coarse Aggregate (%)	85	73	73
Willow River Sand (%)	15	16	16
Ball Sand (%)	0	11	11
Rubber added to Asphalt (% of mass of asphalt)	0	10	0
Rubber added with Aggregate (% of mass of aggregate.)	1	0	0
Asphalt Content (% of mass of aggregate)	6.0	7.0	5.8
Density (kg/m^3)	2321	2288	2341
Stability (kN)	3.73	5.5	8.45
Flow (mm)	3.9	1.85	2.5
Air Voids (%)	\	4.7	3.8

Note: Only one asphalt binder, Husky 200-300 A, is used in the rubber asphalt mixes. The routine properties are: Pen(25°C) = 262 dmm; Vis(60°C) = 42.0 Pa.s; Vis(135°C) = 213 mm^2/s.

3.2 Analysis of the Test Data

There were three mix series with twenty briquettes of the rubber asphalt mixes in each mix series. They were sorted into four groups of five so that replicate tests could be performed at each of the four test temperatures. This sorting was done on the basis of compacted densities such that the average densities of each group would be as similar as possible. The grouping results are given in Appendix I, and the grouping program has been described by Hussain (1990).

The preliminary analyses on the indirect tensile test data from the project of "Low Temperature Testing of Recycled Tire Rubber in Asphalt Concrete Pavements" have been reported by Anderson (1992) and EBA Consultants Engineering Ltd. (1993). The actual tests and analysis were performed by the author as part of the project team. The following is a supplementary analysis of the test results with the introduction of a procedure for outlier rejection which had not been formalized in previous studies.

3.2.1 Test Results and Outlier Rejection

In a group of readings or in a supposedly homogeneous sample, one or more of the observations may be very different from the others, or **the observations deviate from the**

mean greater than expected. Such observations are defined as outliers by Kennedy and Neville (1986). If the outliers are not caused by some unidentified influencing factors, they must be caused by mistakes in readings or recording the measurements in the test. It is reasonable to reject these outliers only when we are **confident** in a certain degree that they are caused by mistakes in the test. The criteria for the rejection should be statistically sound instead of subjective judgment. An outlier rejection method called **Chauvenet's Criterion** is introduced in this thesis to process indirect tensile test data.

Chauvenet's Criterion is based on the assumption of a normal distribution. An observation in a sample of size n is rejected if it has a deviation from the mean greater than that corresponding to a probability of $1/(2n)$. For $n = 5$ (In our case, five replicates are used) the probability is 10%. In other words, A confidence interval corresponding to 90% probability is settled. If an observation is outside of the interval range as show below, it is considered as an outlier and is rejected:

$$(X - 1.64\ s) \leq x_i \leq (X + 1.64\ s)$$

where

x_i = an observation,
X = mean of the sample,
s = standard deviation of the sample.

In the indirect tensile tests, although there are three parameters to represent a specimen's properties including failure stress, failure strain, and failure stiffness, only the tensile failure stress is considered in the outlier rejection analysis. In other words, only if the tensile failure stress of a specimen is decided as an outlier and rejected, the specimen is rejected.

The original data from the test and the 90% probability confidence intervals for the tensile failure stresses are given in Table 3.4 for rubber asphalt mix (1% WTP, "Dry" Procedure). The test results after the outlier rejection are presented in Tables 3.5 for the same mix as in Table 3.4. The data before and after the outlier rejection for the other asphalt mixes are given in Appendix II.

Table 3.4-Indirect Tensile Test Data before Outlier Rejection for Rubber Asphalt Mix (1% WTP, "Dry" Procedure)

Temperature (°C)	Sample No.	Failure Stress (KPa)	Failure Strain (0.0001)	Failure Stiffness (MPa)	Sample Density (kg/m³)	90% Probability Confidence Interval for Failure Stress
0	A2	215	96	41.2	2299	
	A8	382	80	87.7	2316	
	A11	431	184	42.7	2316	
	A12	447	148	55.3	2327	(1.64*Std. Dev.)
	A19	496	95	95.5	2345	177
No. of Specimens		5	5	5	5	Lower Limit
Mean		394	120	64.5	2321	217
Std. Dev.		108	44	25.5	17	Upper Limit
Coef. Var.(%)		27.4	36.6	39.6	0.7	571
-10	A3	909	65	255	2290	
	A6	1194	97	226	2314	
	A13	1160	108	196	2321	
	A17	1146	128	163	2349	(1.64*Std. Dev.)
	A18	1137	121	171	2328	187
No. of Specimens		5	5	5	5	Lower Limit
Mean		1109	104	202	2320	922
Std. Dev.		114	25	38.2	21	Upper Limit
Coef. Var.(%)		10.3	23.9	18.9	0.9	1296
-20	A1	2423	28	1610	2303	
	A10	2576	29	1631	2311	
	A15	2753	36	1413	2330	
	A16	2707	27	1845	2320	(1.64*Std. Dev.)
	A20	2482	36	1266	2340	232
No. of Specimens		5	5	5	5	Lower Limit
Mean		2588	31	1553	2321	2356
Std. Dev.		142	4	222	15	Upper Limit
Coef. Var.(%)		5.5	14.3	14.3	0.6	2820
-30	A4	2680	38	1305	2299	
	A5	2375	19	2263	2322	
	A7	2732	7	7056	2323	
	A9	2747	29	1732	2317	(1.64*Std. Dev.)
	A14	2910	10	5483	2341	321
No. of Specimens		5	5	5	5	Lower Limit
Mean		2689	20	3568	2320	2368
Std. Dev.		196	13	2551	15	Upper Limit
Coef. Var.(%)		7.3	62.7	71.5	0.6	3010

Table 3.5-Indirect Tensile Test Results after Outlier Rejection for Rubber Asphalt Mix (1% WTP, "Dry" Procedure)

Temperature (°C)	Sample No.	Failure Stress (KPa)	Failure Strain (0.0001)	Failure Stiffness (MPa)	Sample Density (kg/m³)
0	A8	382	80	87.7	2316
	A11	431	184	42.7	2316
	A12	447	148	55.3	2327
	A19	496	95	95.5	2345
No. of Specimens		4	4	4	4
Mean		439	126	70	2326
Std. Dev.		47	48	25	14
Coef. Var.(%)		10.7	38.1	36.0	0.6
-10	A6	1194	97	226	2314
	A13	1160	108	196	2321
	A17	1146	128	163	2349
	A18	1137	121	171	2328
No. of Specimens		4	4	4	4
Mean		1159	113	189	2328
Std. Dev.		25	14	28	15
Coef. Var.(%)		2.2	12.4	14.9	0.6
-20	A1	2423	28	1610	2303
	A10	2576	29	1631	2311
	A15	2753	36	1413	2330
	A16	2707	27	1845	2320
	A20	2482	36	1266	2340
No. of Specimens		5	5	5	5
Mean		2588	31	1553	2321
Std. Dev.		142	4	222	15
Coef. Var.(%)		5.5	14.3	14.3	0.6
-30	A4	2680	38	1305	2299
	A5	2375	19	2263	2322
	A7	2732	7	7056	2323
	A9	2747	29	1732	2317
	A14	2910	10	5483	2341
No. of Specimens		5	5	5	5
Mean		2689	20	3568	2320
Std. Dev.		196	13	2551	15
Coef. Var.(%)		7.3	62.7	71.5	0.6

3.2.2 Test Result Analysis

In order to compare the rubber asphalt mix properties with the control mix properties, the test results are shown in Fig.3.2 to Fig.3.4 based on the data after outlier rejection as shown in Table 3.5. In Fig.3.2, the failure stress decreases slightly at all temperatures due to the addition of tire rubber when compared with the controlled specimens. This is likely due to the changed characteristics of the binder, although the somewhat lower density would account for some of this reduced failure stress. As seen in Table 3.6, the drop in density for the 1% WTP specimens is from 0.6 to 0.9% of that of control specimens, and for the 10% WTP specimens, is from 2.2 to 2.5% of that of control specimens.

As shown in Fig.3.3, failure strain is slightly improved with the addition of recycled rubber. This effect is more pronounced at the higher test temperatures of 0°C and -10°C. The combined effect of reduced failure stress and increased failure strain is reflected in the calculated stiffness values as shown in Fig.3.4. The failure stiffness of the mixes with addition of the recycled tire rubber is slightly lower at the lower temperatures of -20°C and -30°C. However, caution should be expressed in this observation due to the greater experimental error in the strain data at the lower temperatures.

Table 3.6-Density Comparison of Samples Tested

Temperature (°C)	Sample	Average Density (kg/m^3)	Percentage of Control Density (%)
0	CONTROL	2341	100
	1% WTP	2326	99.4
	10% WTP	2288	97.7
-10	CONTROL	2341	100
	1% WTP	2328	99.4
	10% WTP	2282	97.5
-20	CONTROL	2340	100
	1% WTP	2321	99.2
	10% WTP	2289	97.8
-30	CONTROL	2341	100
	1% WTP	2320	99.1
	10% WTP	2288	97.7

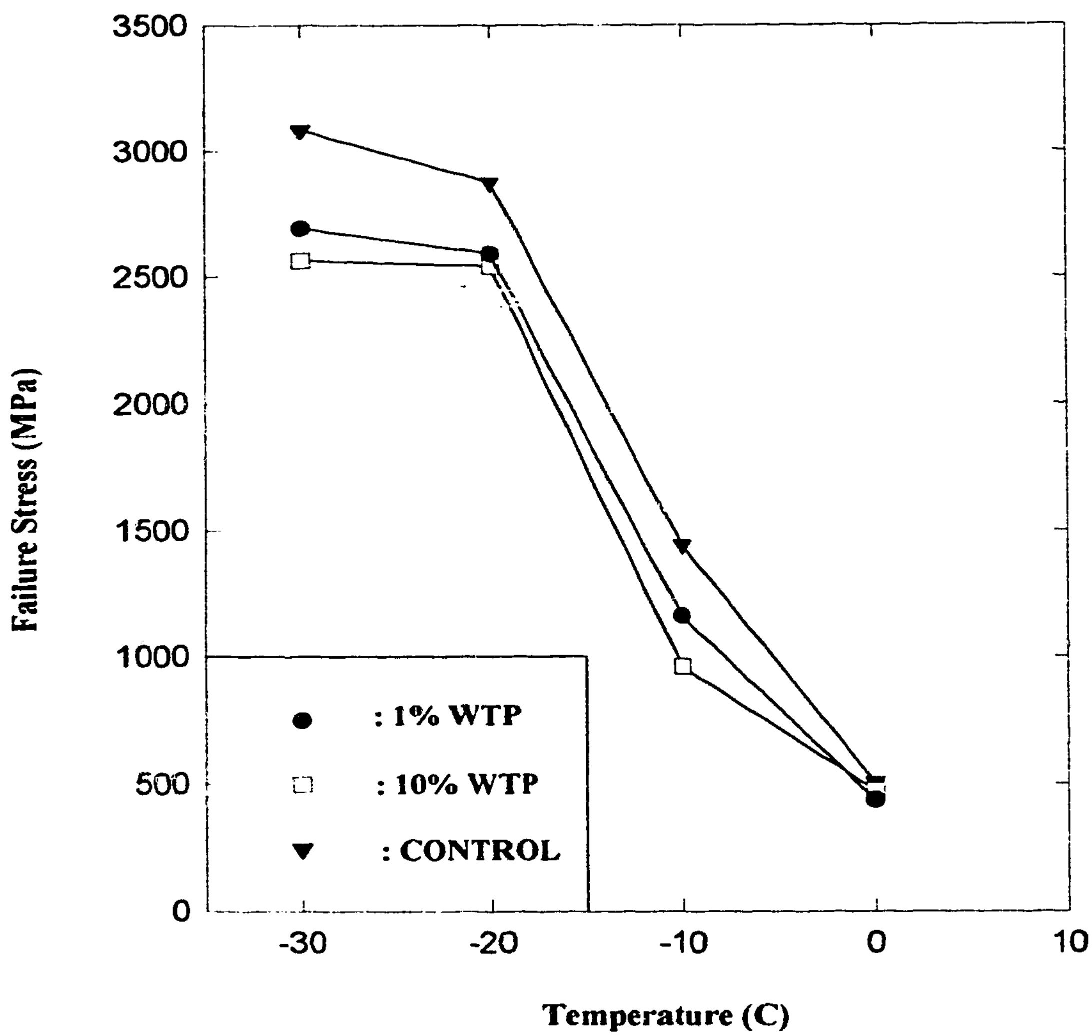


Fig.3.2-Failure Stress vs. Temperature From Indirect Tensile Test

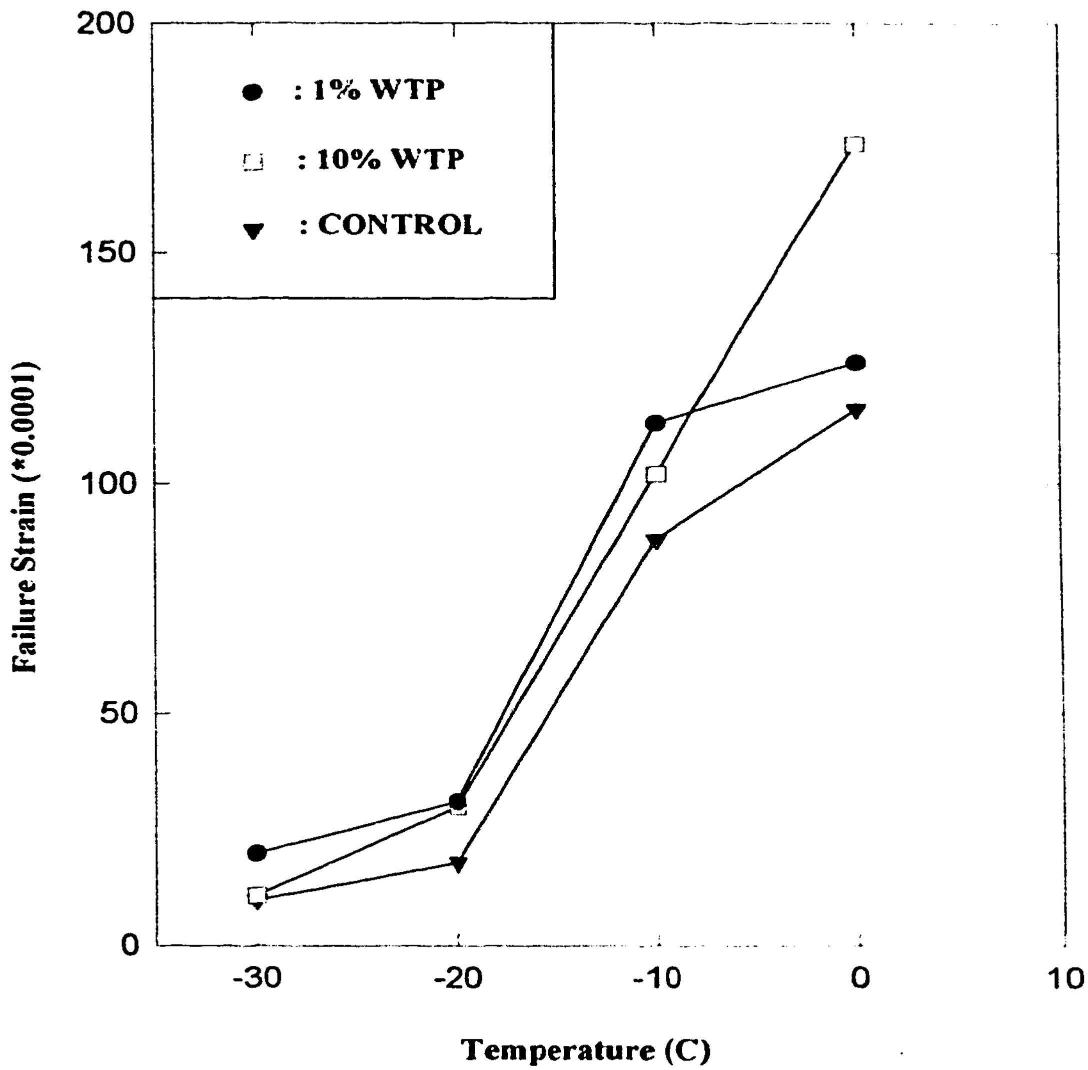


Fig.3.3-Failure Strain vs. Temperature from Indirect Tensile Test

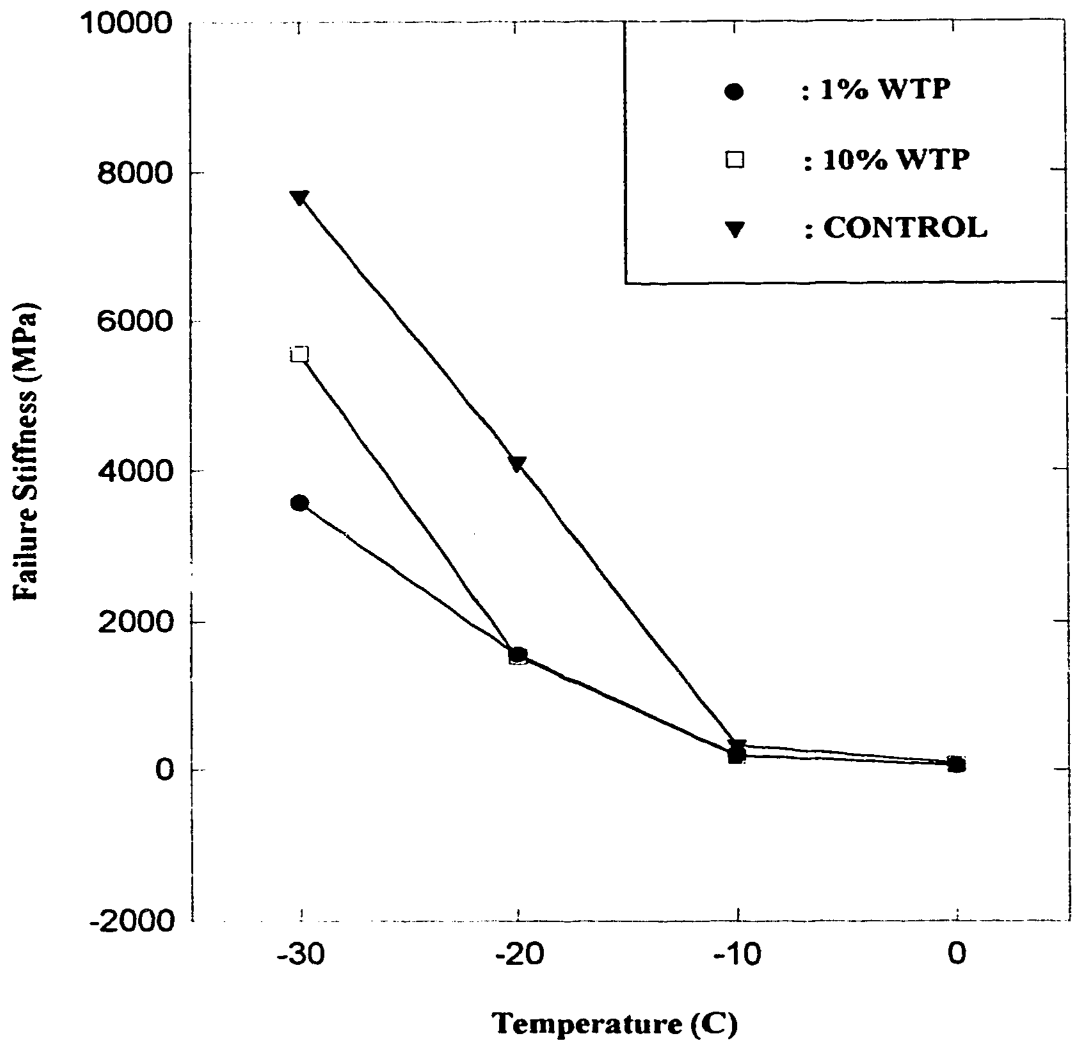


Fig.3.4-Failure Stiffness vs. Temperature from Indirect Tensile Test

3.3 Summary of Test Result Analysis

On the basis of this test program it was concluded by Anderson (1992) and EBA Engineering Consultants Ltd. (1993) that the addition of recycled tire rubber had a slight effect on the low temperature properties of asphalt concrete mixes. The addition of the rubber tire produced some improvement in the expected low temperature performance of pavements constructed with these mixes. The primary differences with respect to the control mixes with asphalt binder of Husky 200-300A are as follows based on these tested specimens:

1. The failure stress is slightly reduced with the addition of 1% WTP or 10% WTP at temperatures from -10°C to -30°C.

2. The failure strain is slightly improved with the addition of the recycled rubber and the improvement is more pronounced at the higher temperatures of 0°C to -10°C.

3. The failure stiffness of the compacted mixes with addition of the recycled tire rubber are slightly lower at the lower temperatures of -20°C and -30°C.

4. Despite a small reduction in compacted density with addition of recycled tire rubber, the mechanical properties at low temperatures have not been adversely influenced to a significant degree and may even show slight improvements.

5. Chauvenet's Criterion can be used as a standard outlier rejection method for the indirect tensile test (constant loading speed to failure test) to improve the precision of the data treatment.

3.4 Future Work

1) The test method used in the University of Alberta is a constant loading speed to failure test with two dimension stress state assumption and interior horizontal deformation measurement. The newly developed SHRP Protocol of the indirect tensile test involves both creep and fracture tests with three dimension stress state assumption and interior horizontal and vertical deformation measurements. Apparently, this newly developed equipment and analysis procedure is more accurate but it is considerably more expensive and the analysis is more complicated. Therefore, it would be desirable to compare the results from the two methods.

2) Application of the Chauvenet's Criterion can also be used as an outlier rejection method for future indirect tensile test data treatment.

CHAPTER FOUR

ASPHALT MIX INDIRECT TENSILE TEST COMPARED WITH NOMOGRAPH METHODS

When direct tests are not available, nomograph methods are used to obtain the needed asphalt mix properties. The nomograph methods generally used are the Van der Poel nomograph (1954) to calculate asphalt stiffness, the method developed by Bonnaure et al. (1977) to calculate mix stiffness, and the graphs developed by Heukelom (1966) and Deme et al. (1987) to calculate failure stress from mix stiffness.

The nomographs for prediction of asphalt and asphalt mix stiffness have been computerized by Koole et al. (1989) in software BANDS-PC (Bitumen and Asphalt Nomographs Developed by Shell for use on Personal Computers).

The direct tensile test has been used to obtain tensile strength and stiffness data of asphalt mixes for the study of low temperature transverse cracking in asphalt pavement because the stress state in this test is close to the field stress state. However, the indirect tensile test has two main advantages:

1) The test is much easier to conduct than the direct tensile test:

- **i)** The specimens used in the indirect tensile test are the same size as those used in the standard Marshall or Hveem tests and are easier to fabricate;
- **ii)** The attachment of loading devices to a specimen required for direct tensile test is not required for the indirect tensile test.

2) The location of the failure plane is defined.

Therefore, the indirect tensile test has been extensively used to evaluate low temperature properties of asphalt mixes. However, when the test is used for the analysis of transverse cracking, because of the biaxial stress state in a specimen when it is loaded in this test, the following two questions must be addressed:

1) Are there significant differences between the results from the indirect tensile test and the extensively used nomograph methods?

2) Can the results from the indirect tensile test be used to predict asphalt pavement cracking temperatures?

The relationships between the results from the indirect tensile test and the data from the generally accepted nomograph method are established in this chapter in order to answer these two questions.

The data used in the analysis are from Lamont Test Road provided by Alberta Transportation and Utilities (Wang et al. 1992, and Gavin 1992). In the indirect tensile test, five replicates were tested at each of the four testing temperatures (0, -10, -20, -30°C) and for each of the seven test sections. The analyses of the indirect tensile test data for the asphalt mixes used in Lamont Test Road have been reported by Wang et al. (1992). Although the previous analysis method was used without modification, a cursory examination shows that the reported data has been subjected to some informal outlier rejection method and it generally meets the proposed outlier rejection method described in Chapter Three.

4.1 Materials

The asphalt properties are shown in Table 4.1. The penetration indexes (PI) and the temperatures at which the penetration is 800 dmm (T_{800}) in this table are calculated by means of the linear regression of the logarithm penetration vs. temperature, and the conventional equations are described in Appendix III.

The asphalt classification is shown in Fig.4.1 which is a slightly modified version of Fig.1 from the CGSB (Canadian General Standard Board) specification CAN/CGSB-16.3-M90. From Fig.4.1, it can be seen that the asphalts used in test section No.3, 5, 6, and 7 are classified as Group A, the asphalt used in test section No.1 and 2 as Group B, and the one used in test section No.4 as Group C. The detailed CGSB classification results are shown in Table 4.2.

A computerized version of the Bitumen Test Data Chart (BTDC) has been developed. Fig.4.2 is an example for the asphalt used in test section No.7. Similar BTDCs for the other asphalts listed in Table 4.1 are presented in Appendix III. Detailed discussion concerning the development and the use of the computerized version are given in the same appendix.

The mix composition for each test section is shown in Table 4.2, and the aggregate gradation used in the asphalt mixes is shown in Table 4.3. The source of the aggregate is the Eugene David pit (SE 15-057-14-04) which is located on the banks of the North Saskatchewan River and contains a river deposited gravel. It was rated as a poor quality paving aggregate by a petrographic analysis. However, it was decided that this aggregate would not negatively effect the relative low temperature performance evaluation of the asphalts (Gavin 1992).

Table 4.1-Asphalt Properties Used in Lamont Test Road

Test Sec. No.	Asphalt Source	P(25°C) (dmm)	P(10°C) (dmm)	P(5°C) (dmm)	V(60°C) (Pa.s)	V(135°C) (mm^2/s)	T_{800} (°C)	PI
1	Esso (Air Blown)	100	22	13	96.0	277	45.4	-0.65
2	Montana	150	20	11	59.8	214	37.8	-2.22
3	Esso	333	58	36	31.3	163	32.9	-1.28
4	Esso	93	12	6	74.9	219	40.7	-2.45
5	Husky (Air Blown)	88	21	14	321	530	48.9	-0.05
6	Husky	176	28	17	83.8	280	37.9	-1.58
7	Esso	241	45	25	47.1	195	35.6	-1.32

Note:

1) CGSB uses the unit of Pa.s for absolute viscosity at 60°C and the unit of mm^2/s for kinematic viscosity at 135°C.
2) ASTM D 2170 defines Pa.s as the SI unit for absolute viscosity at 60°C and 1 Pa.s = 10 Poise. It also defines mm^2/s as the SI unit for kinematic viscosity at 135°C and 1 mm^2/s = 1cSt.
3) T_{800} and PI values are calculated from a regression of the three penetrations at 25, 10, and 5°C.

Table 4.2-Asphalt Mix Design Parameters from Lamont Test Road

Test Section No.		1	2	4	5	6	7
Asphalt Cement	Supplier	Esso	Montana	Esso	Husky	Husky	Esso
	Pen. Grade Group	80/100 B	150/200 B	80/100 C	80/100 A	150/200 A	200/300 A
	Spec. Gravity.	1.009	1.038	1.018	1.030	1.030	1.035
Asphalt Concrete	Asph. Content (%)	5.8	6.0	6.2	6.1	6.0	6.3
	Density (kg/m^3)	2360	2368	2354	2356	2364	2360
	Stability (kN)	18.1	11.3	11.4	14.0	10.9	10.4
	Flow (mm)	2.9	3.1	2.5	2.7	2.7	2.3
	VMA (%)	15.0	14.9	15.6	15.4	15.1	15.5
	Vg (%)	84.9	85.0	84.4	84.5	84.9	84.5
	Vb (%)	11.6	11.6	12.1	12.0	11.6	12.0
	Va (%)	3.5	3.4	3.5	3.5	3.5	3.5

Notes: 1) The design of the mixture for test section No.3 (Asphalt cement is Esso 300/400 A) is the same as for test section No.7.
2) VMA is voids in mineral aggregate.
3) Vg is volume percentage of aggregate in asphalt mixture.
4) Vb is volume percentage of asphalt in asphalt mixture.
5) Va is volume percentage of air voids in asphalt mixture.

Table 4.3-Aggregate Gradation Used in Lamont Test Road

Sieve Size (Approximate Equivalents)		Coarse Aggregate	Fine Aggregate	Sand	Combined Grade
Metric CGSB (μm)	AASHTO (M 92)	Percent Passing (%)			
12,500	1/2	100	100		100
10,000	3/8	80	99		88
5,000	#4	46	82		62
1,250	#16	24	46		38
630	#30	19	32	100	31
315	#50	14	22	56	25
160	#100	8.9	13	41.3	13.4
80	#200	5.3	8.3	7.7	6.4
Proportion		60%	30%	10%	100%

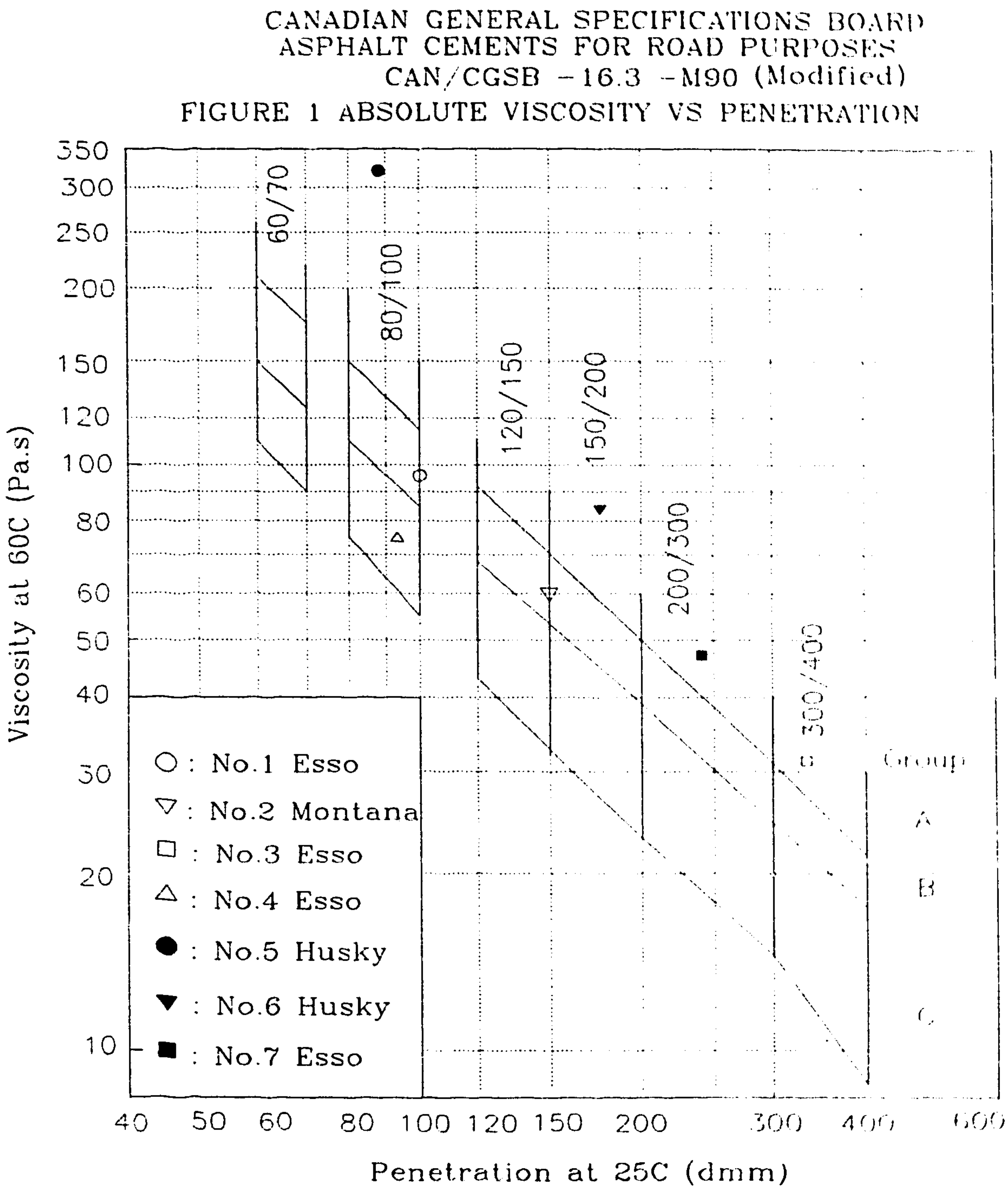


Fig.4.1-The Classification of the Asphalts Used in Lamont Test Road

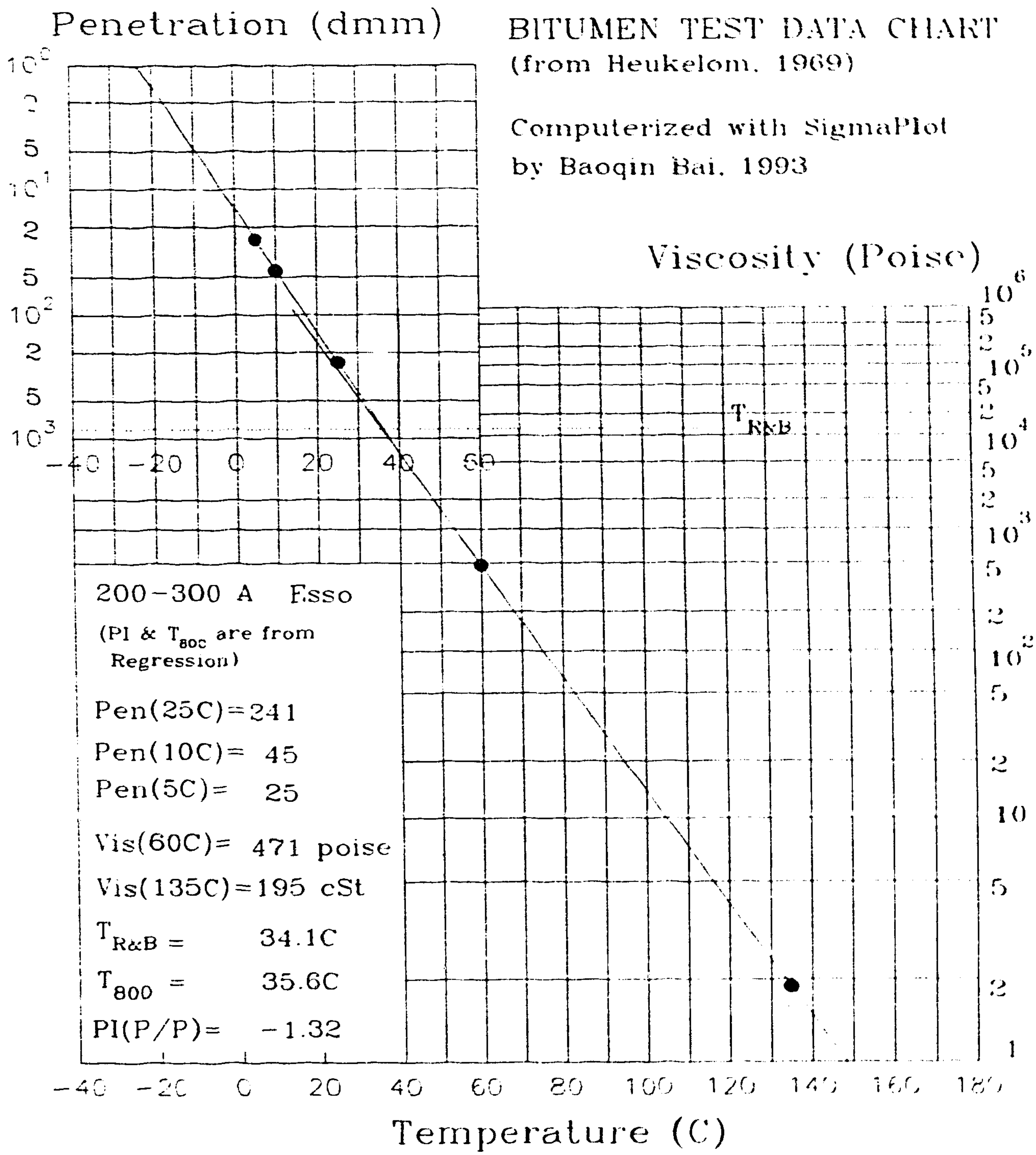


Fig.4.2-Asphalt Used in Lamont Test Section No.7

4.2 Tested Failure Stiffness Compared with Calculated Failure Stiffness

4.2.1 Calculation of mix stiffness from the nomograph method.

a). Determine loading time

The indirect tensile test procedure has been described by Hussain (1990). The nominal loading speed is 1.5 mm/min. A typical output of the indirect tensile stress test results from sample No.4 in Lamont test section No.1 is shown in Fig.4.3. The data collection program collects information at one second intervals. This is started some time before the actual loading begins.

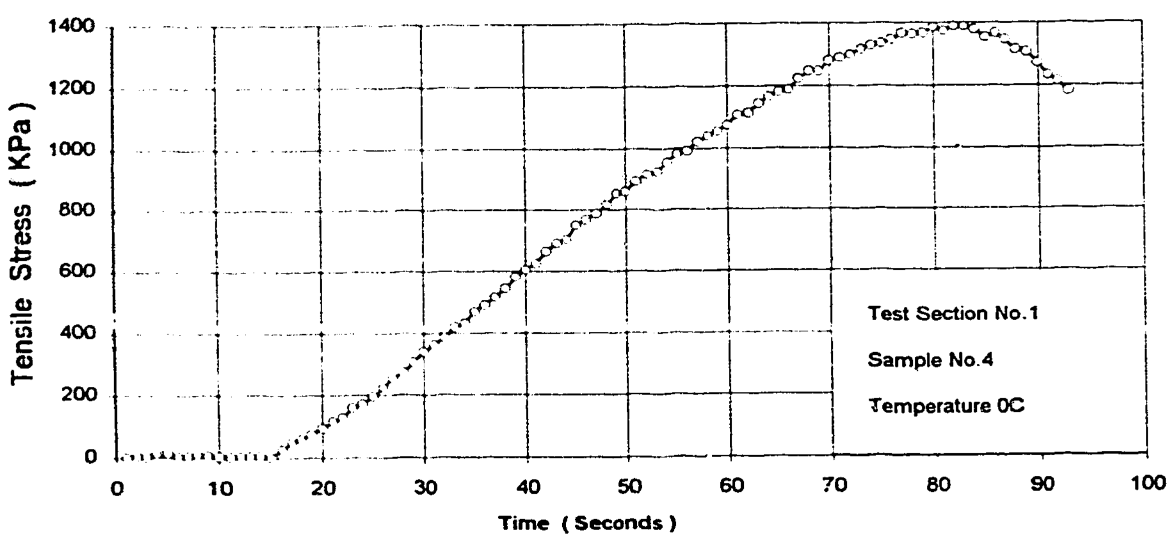


Fig.4.3-Indirect Tensile Stress vs. Time

From Fig.4.3, it can be seen that in this case the tensile stress was applied to the specimen from the 15 seconds after the starting of the data collection program, and the specimen failed at 82 seconds. This means that the loading time to failure of the specimen can be calculated as

Loading time = 82 - 15 = 67 (seconds).

b). Calculate stiffness of asphalt binder

The stiffness of asphalt binder was calculated by using the computer program BANDS-PC. For comparison, two sets of input parameters were used in the calculation, i. e., penetrations at 25 and 5°C and penetrations at 25 and 10°C. The other input parameters include the loading time determined as above and the temperature at which the specimen was tested. The calculated results for test section No.1 are shown in Table 4.4.

Table 4.4-Calculated and Tested Properties of the Materials Used in Lamont Test Section No.1

Asphalt Data: Esso 80/100, Air Blown, $P_{25°C}$ = 100, $P_{10°C}$ = 22, $P_{5°C}$ = 13 Mix Data: Vg = 84.9%, Vb = 11.6%, Va = 3.5%								
Temp (°C)	Specim. No.	Ldg Time (s)	Calculated Asphalt Stiffness (MPa)		Calculated Mix Stiffness (MPa)		Tested Mix Stiffness (MPa)	Tensile Strength (MPa)
			$P_{(25\&10)}$	$P_{(25\&5)}$	$P_{(25\&10)}$	$P_{(25\&5)}$		
0	TS1-4	67	1.85	1.38			4160	1.39
	TS1-5	79	1.60	1.20			696	1.90
	TS1-12	80	1.58	1.19			1090	2.12
	TS1-14	79	1.60	1.20			1160	2.03
	TS1-20	83	1.53	1.16			798	2.28
Average			1.63	1.23	687*	570*	1581	1.94
-10	TS1-1	82	20.8	11.3			6710	3.64
	TS1-3	82	20.8	11.3			3740	3.90
	TS1-8	77	21.7	11.7			4320	3.51
	TS1-13	97	18.4	10.1			3370	4.22
	TS1-19	87	19.9	10.8			2630	3.78
Average			20.3	11.0	3650	2430	4154	3.81
-20	TS1-7	78	172	71.0			12680	4.52
	TS1-10	83	167	68.7			29600	4.87
	TS1-11	69	183	75.6			12400	4.08
	TS1-15	74	177	72.9			9900	4.07
	TS1-17	77	173	71.4			5810	3.50
Average			174	71.9	13400	8440	14078	4.21
-30	TS1-2	86	646	267	24100	16300	34200	5.63

Note: $P_{(25\&10)}$ means penetrations at 25°C & 10°C, and the data in this column are the stiffness calculated by using the penetrations at 25°C & 10°C.

* Outside range of the nomograph

c). Calculate the mix stiffness

In the calculation of the mix stiffness, for convenience, a BASIC program has been produced based on the method developed by Bonnaure et al. (1977). Since BANDS-PC allows only keyboard input, the operation is very constrained and time-consuming if there is much data to be treated. A comparison of the results calculated with the two programs, shows that the BASIC program is valid since the results are the same (Table 4.5). The inputs needed for the BASIC program are asphalt stiffness (Sb) and volume contents of aggregate (Vg) and asphalt (Vb). The calculated results for test section No.1 are shown in Table 4.4. When the asphalt stiffness is less than 5 MPa, the nomograph (Bonnaure et al. 1977) does not hold valid any more. The data calculated by the BASIC program in such circumstances are also given in the Tables only for the purpose of providing a reference.

Table 4.5-Comparison of the Asphalt Mix Stiffnesses Obtained by Different Programs for the Mixes Used in Lamont Test Section No.1

Asphalt Mix Parameters: Vg = 84.9%, Vb = 11.6%, Va = 3.5%		
Asphalt Stiffness	Asphalt Mix Stiffness (MPa)	
(MPa)	by BASIC Program	by BANDS-PC
1.23	570	out of range
1.63	687	out of range
11.0	2430	2430
20.3	3650	3650
71.9	8440	8440
174	13400	13400
267	16300	16300
646	24100	24100

4.2.2 Acquisition of the tested mix stiffness from indirect tensile test.

Generally, the tested mix stiffness can be obtained from the indirect tensile test output. However, the measured tensile strain usually has a large error, especially at low temperatures. Fig.4.4 and Fig.4.5 show typical measured tensile strain changes versus time. Fig.4.4 is for sample No.5 in test section No.1 at 0°C. At this temperature, the measurement has very little error so that the tested mix stiffness can be used as read directly from the indirect tensile test output. However, in Fig.4.5, which is for sample No.2 from the same test section but tested at -30°C, the measurement is so poor that the strain value has to be adjusted subjectively based on the trend of the measurement. Therefore, to obtain the tested mix stiffness for each specimen, the plot of tensile strain vs. time must be visually checked and a smooth curve may be used to calculate the stiffness based on the adjusted tensile strain.

It should be noted that the scale of the tensile strain has been greatly magnified in Fig.4.5. The magnitude of the noise of the equipment is almost that of the measured strain.

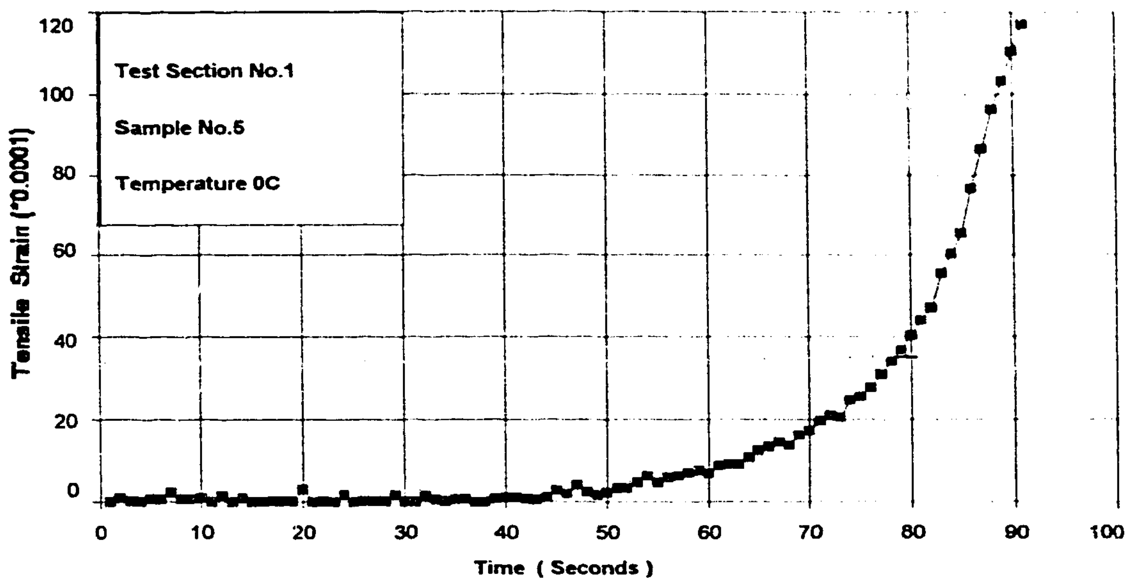


Fig.4.4-Indirect Tensile Strain vs. Time

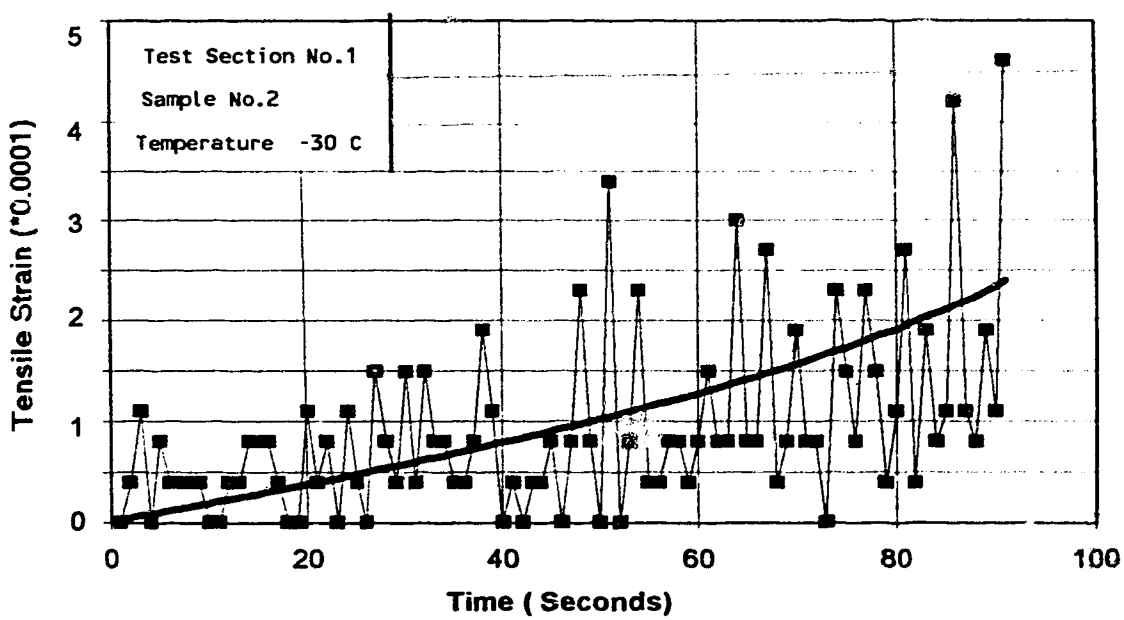


Fig.4.5-Indirect Tensile Strain vs. Time

Previous researchers using the University of Alberta test method (Hussain, 1990) have reported tensile stiffness modulus obtained from the indirect tensile test. These modulus values are based on an assumed value of Poisson's ratio and an average stress value over the interior gauge length. It was desired to compare tested values obtained in this way with calculated values based on nomographs.

4.2.3 Comparison between the calculated and the tested mix stiffnesses

As an example, Table 4.4 lists the calculated and tested mix stiffness and tensile strength of the specimens from test section No.1. The average values for each section at each temperature have been used for the analysis. The calculated and tested properties of the materials used in the other test sections are given in Appendix IV. The comparison of the mix stiffnesses from the indirect tensile test and from the nomograph method are shown in Fig.4.6. In Fig.4.6a, the calculated asphalt stiffness is based on penetrations at 25°C and 10°C and in Fig.4.6b, on penetrations at 25°C and 5°C.

From the regression lines shown, it can be seen that the tested mix stiffness is a little larger than that calculated, and the larger the stiffness, the smaller the difference. This can be explained that the larger the stiffness, the more elastic the material behaves, so that the elastic theory assumed in the indirect tensile test is more valid. It is also found that the wide range of penetration values for these materials studied has very little influence on the relationship between tested and calculated mix stiffnesses. Fig 4.7 presents the same data shown in Fig.4.6(a) for more direct comparison between the tested and calculated mix stiffnesses. Because other important factors such as asphalt wax contents, aggregate properties, etc. are not considered here, the agreement between the calculated and tested mix stiffness values is considered to be acceptable.

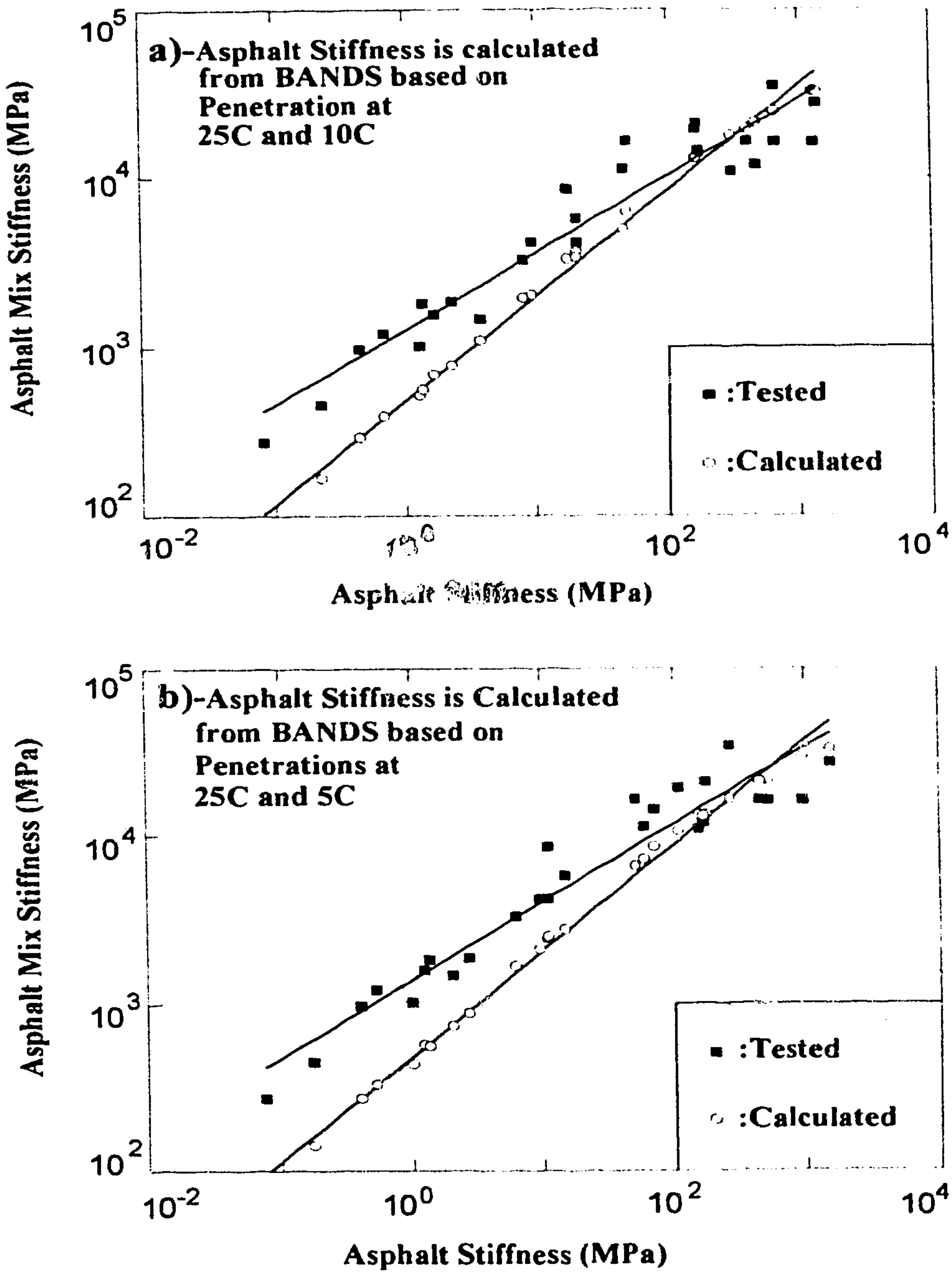


Fig.4.6-Comparison between the Tested and the Calculated Asphalt Mix Stiffnesses

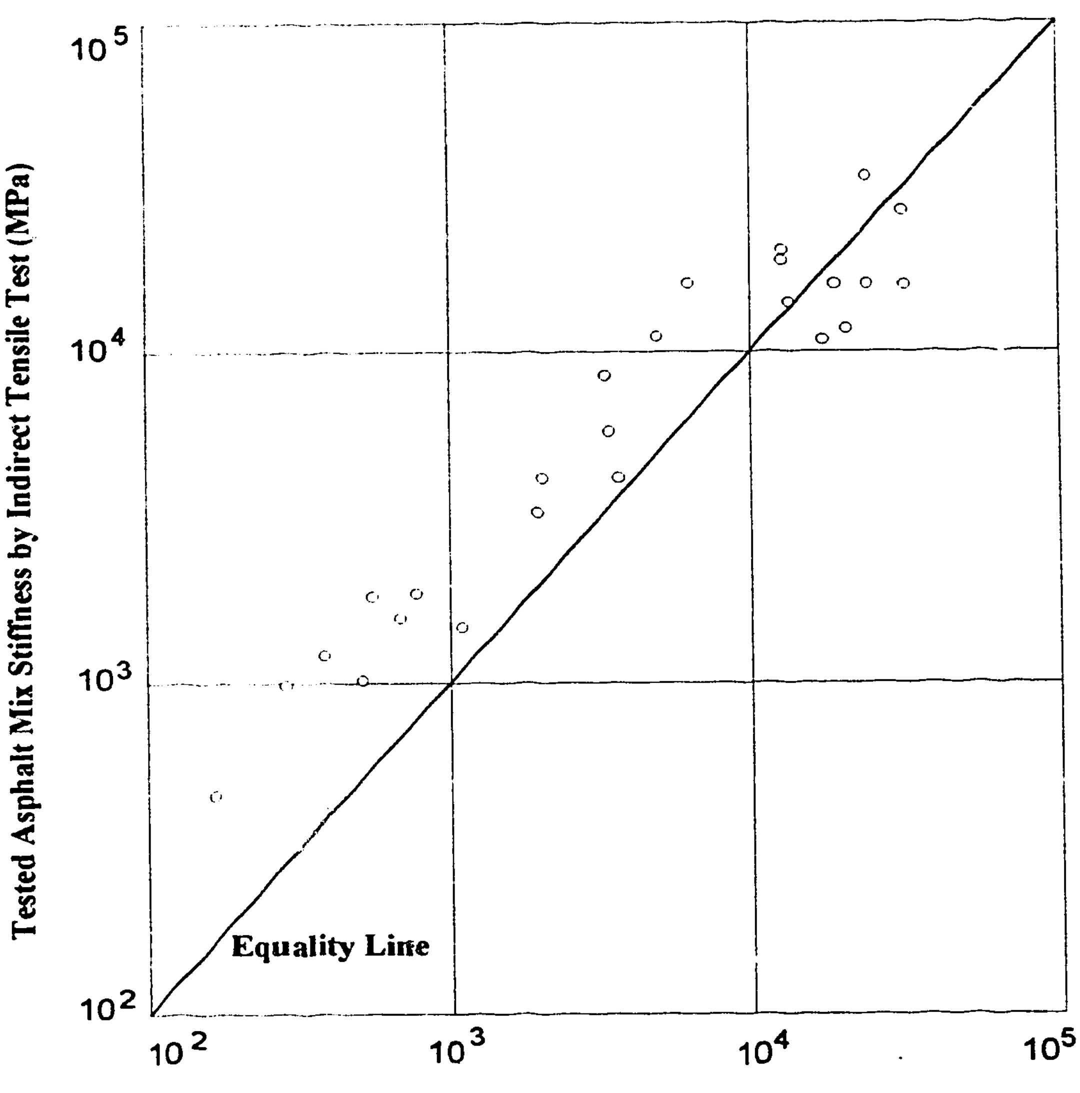


Fig.4.7-Asphalt Mix Stiffness Tested Compared with That Calculated by Nomograph Method

4.3. Relationship between Failure Stiffness and Failure Stress

Tested failure stress can be read directly from the indirect tensile test output with very good accuracy. The tested failure stiffness was obtained as mentioned in the previous section. Fig.4.8 is a plot of tensile strength vs. mix stiffness from the indirect tensile test. From this figure, it is obvious that the failure stress increases almost linearly as the stiffness at failure increases.

Fig.4.9 presents the same information compared with the published curves by Heukelom (1966) and Deme et al. (1987). The curves from Heukelom have been modified to show tensile strength as a function of mix stiffness as shown in Fig.4.8 and as reported by Deme. The Heukelom Type I curve was obtained from the poorly graded and/or compacted asphalt mixes, and Type II from well graded and/or compacted mixes. The tensile strength in these two curves were originally plotted as a function of asphalt stiffness and have been converted to mix stiffness.

From Fig.4.9, it can be seen that:

a) The curve from the indirect tensile test does not reach a peak at about 10,000 MPa of asphalt mix stiffness as do the other curves.

b) When asphalt mix stiffness is less than 10,000 MPa, the curve is approximately equal to the curve Type I from Heukelom and lower than the Heukelom's curve Type II but higher than the curve from Deme. In view of the scatter of the data points, the curve from the indirect tensile test is considered as not having significant differences from the Heukelom curve Type I before the point of 10,000 MPa stiffness.

c) When the stiffness is larger than 10,000 MPa, the three curves from Heukelom and Deme have similar shapes although the peak values are different.

A close examination for the test results as shown in Fig.4.8 has revealed a peak value being reached for the test section No.4 (#4 Esso) in which the most temperature susceptible asphalt was used. Similarly, peak value were also reported by Christison et al. (1972) using the indirect tensile test for the Ste. Anne test road materials which Deme et al. tested by using the direct tensile test. Based on these findings in Fig.4.8 and Fig.4.9, it is assumed in this thesis that when asphalt mix stiffness is less than 10,000 MPa, the indirect tensile test gives similar results to the Heukelom Type I curve. When the stiffness is larger than 10,000 MPa, the tensile strength vs. stiffness of asphalt mix curve has the same shape as the other curves shown in Fig.4.9. Therefore, a modified indirect tensile test curve has been developed and shown in Fig.4.10. This curve may be further modified in the event of more test data becoming available. However, in this thesis, it has been introduced to be consistent with published data.

Because Heukelom's nomograph was obtained using the bending beam test, and Deme's curve was from the direct tensile test, it can be said that the tensile strength from the indirect tensile test is approximately equal to the tensile strength from the bending beam test and slightly larger than the tensile strength from the direct tensile test.

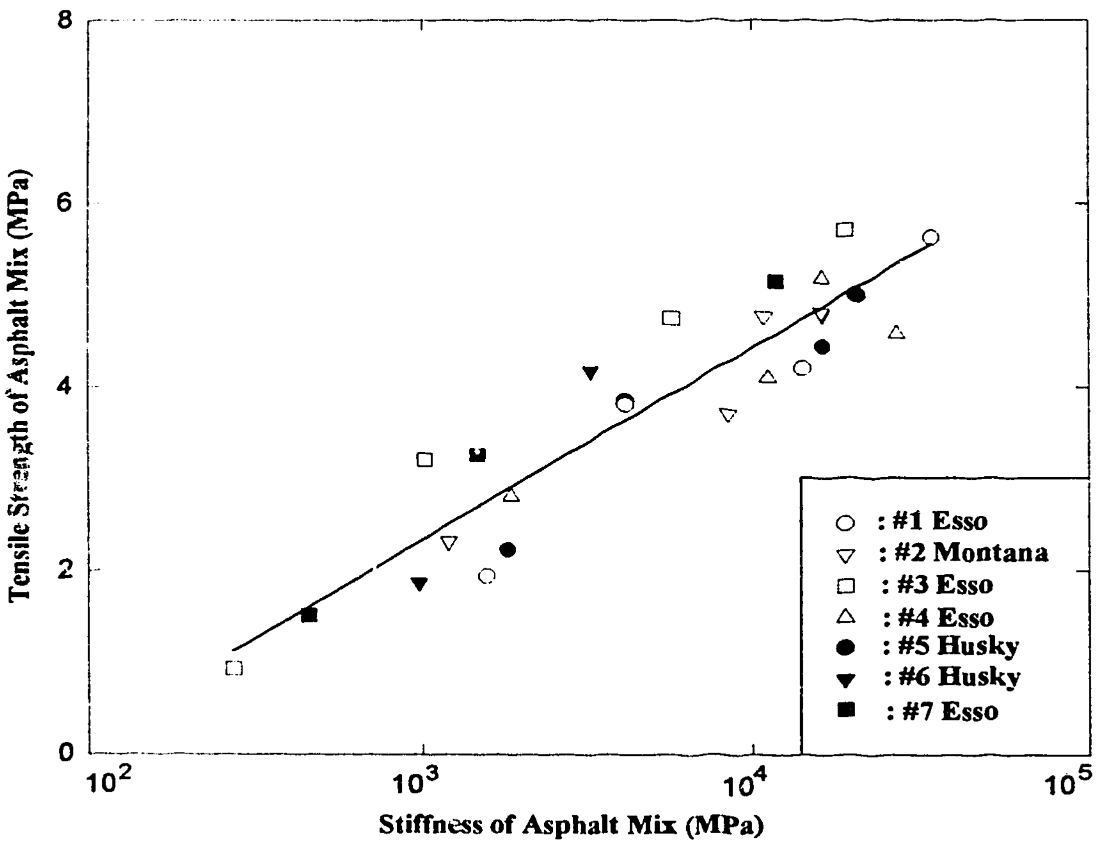


Fig.4.8-Indirect Tensile Strength vs. Asphalt Mix Stiffness

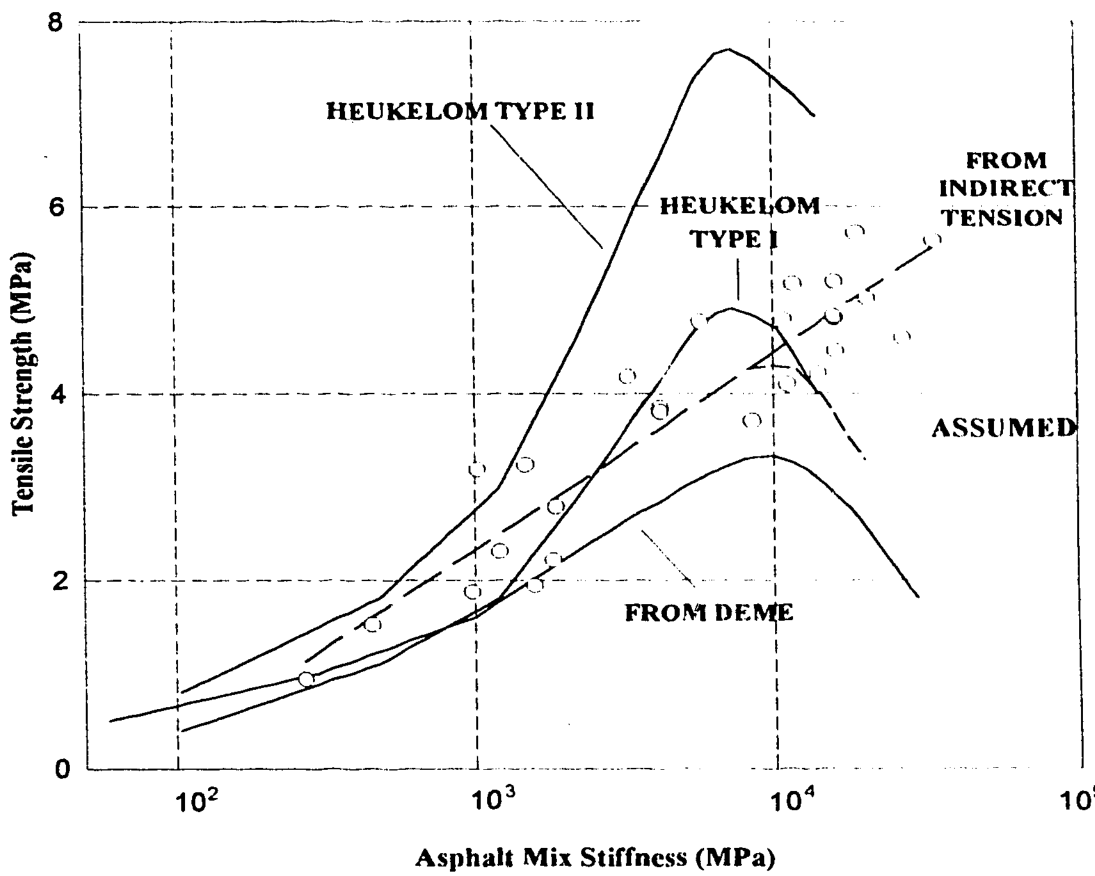


Fig.4.9-Tensile Strength vs. Asphalt Mix Stiffness from Different Sources

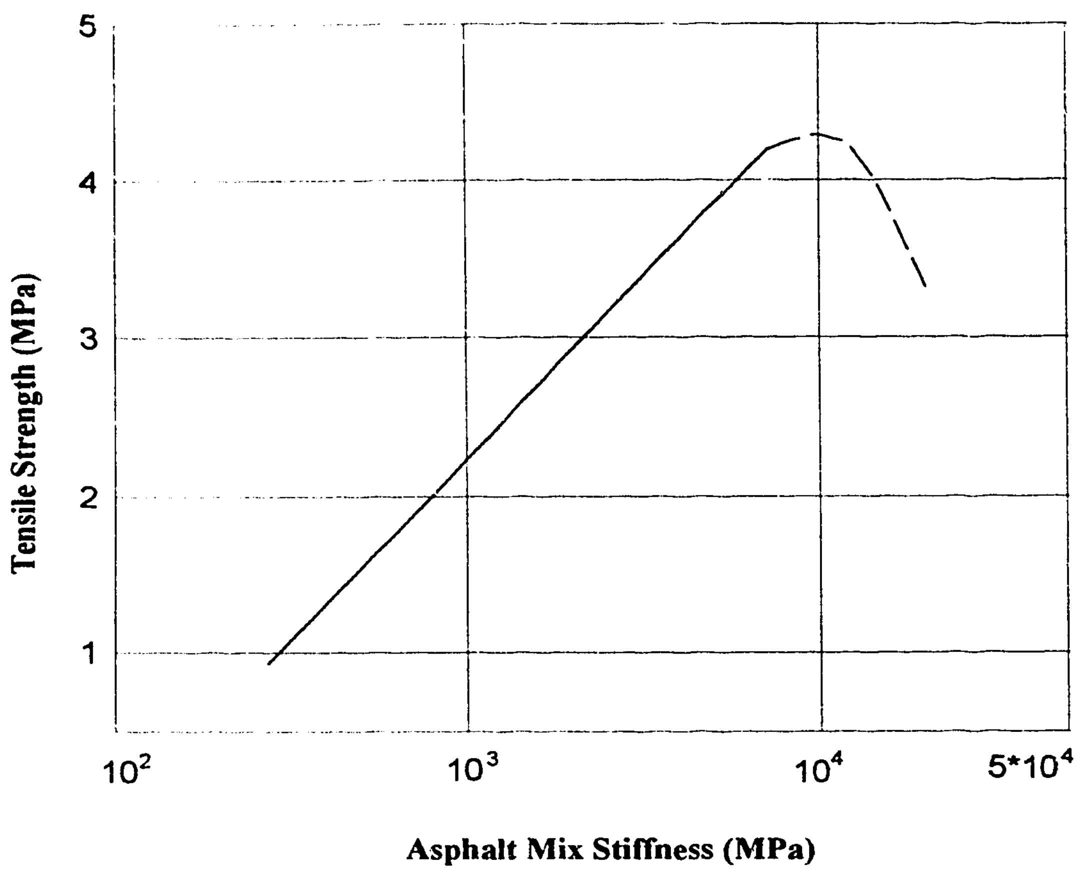


Fig.4.10-Modified Curve of Indirect Tensile Strength vs. Asphalt Mix Stiffness

4.4 Summary

1. For asphalt mix stiffness, there are no significant differences between the indirect tensile test values and the calculated data from nomographs. However, the tested values are a little larger than the calculated mix stiffness. There is a trend that the difference between the tested and the calculated mix stiffnesses tends to be smaller as the mix stiffness increases.

2. For asphalt mix tensile strength, when the mix stiffness is less than 10,000 MPa, there is no significant difference between the indirect tensile test and Heukelom's curve (asphalt mix Type I). However, the regression curve from test data does not reach a peak at approximately 10,000 MPa mix stiffness as do the other published curves. Therefore, the tensile strength from the indirect tensile test should not be used directly for the calculation of cracking temperatures in asphalt pavements.

3. The tensile strength from the indirect tensile test is similar to the tensile strength from bending beam tests and larger than the tensile strength from direct tensile tests.

4. The mix stiffness from the indirect tensile test could possibly be used directly in the calculation of cracking temperature in asphalt pavement. However, caution should be taken in using the tensile strength from the indirect tensile test because it is larger than the tensile strength from direct tensile tests. A modified curve for the mixes used in the Lamont test sections as shown in Fig.4.10 is recommended in this thesis for indirect tensile strength. This curve may be further modified in the event of more test data becoming available.

CHAPTER FIVE

PREDICTION OF CRACKING TEMPERATURES IN ASPHALT PAVEMENTS

5.1 Literature Review

Low temperature design of asphalt pavement to control the low temperature cracking to a desirable level is the main purpose of almost all the low temperature studies of asphalt pavement. In order to achieve this purpose, it is extremely important to develop a method which is accurate enough to predict the cracking temperature for the asphalt pavement that is to be designed under certain conditions. Thus, many studies have been done, and many cracking temperature prediction methods have been developed. Generally, these methods can be divided into two classes:

i. Empirical methods and
ii. Theoretical methods.

5.1.1 Empirical Methods

The empirical methods include **critical stiffness** methods (including asphalt critical stiffness methods and asphalt mix critical stiffness methods) and **modeling** or **correlation** methods. The critical stiffness methods are based on the assumption that the asphalt (or asphalt mix) low temperature properties are the most significant factor influencing the low temperature behavior of asphalt pavement, and low temperature cracking can be predicted by using only asphalt (or asphalt mix) low temperature stiffness. A summary of the critical stiffness methods is shown in Table 5.1.

Hajek and Haas (1972) developed a modeling method by correlating the cracking frequency of 32 pavements in Ontario and Manitoba with factors: i) asphalt stiffness at the design temperature and 20,000 second loading time, ii) winter climate, iii) pavement thickness, iv) pavement age, and v) subgrade characteristics.

Table 5.1-Critical Stiffness Methods.

Method	Loading Time (second)	Critical Stiffness (MPa)		
		Asphalt	Asphalt Mix	
			Min. Temp. at 50 mm Depth (°C)	Mix Critical Stiffness (MPa)
McLeod (1972)	20,000	/	-40 -32 -23 -12	3450 2,410 1,380 340
Fromm and Phang (1971)	10,000	140	/	
Readshaw (1972)	7,200	200	/	
Gaw et al. (1974)	1,800	1,000	/	
Deme and Young (1987)	1,800	1,000	18,000	

The model later was further modified by Haas (1973) after testing and evaluating a large number of alternative forms of the model.

Haas, Meyer, Assaf and Lee (1987) carried out crack surveys and laboratory tests on core samples from 26 selected airports. Regression models were developed to correlate transverse cracking space with minimum temperature recorded on site, PVN (asphalt penetration viscosity number), thermal contraction coefficient, asphalt layer's thickness, and asphalt mix stiffness.

Palsat (1986, 1988) carried out an investigation on transverse cracking in Alberta. Seventy-seven highway sections were studied. By the stepwise regression technique, pavement thickness, original asphalt stiffness, and pavement age were identified as the most significant variables on the transverse cracking frequencies. The original asphalt stiffness was calculated by using McLeod's method (1976).

Sugawara, Kubo, and Moriyoshi (1982) established a relationship among the cracking temperature of asphalt concrete, asphalt penetration, and penetration index. The penetration index was calculated from penetration and softening point at which penetration is assumed as 800 dmm. This relationship was based on the measured cracking temperature from thermal stress restrained specimen test at a constant cooling rate.

Moriyoshi and Tokumitsu (1993) developed a new test method called the Moriyoshi Breaking Point (MBP) test and modified Fraass Breaking Point (MFBP) test to assess the low temperature behavior of asphalts. Good or consistent correlations of the MBP temperature and the MFBP temperature versus the thermal fracture temperature were obtained. The thermal fracture temperatures were obtained with the thermal stress restrained specimen test. The test involved 13 asphalt binders and 7 mixtures with different compositions. The cooling rate of -30°C/hour was used in the test. It was concluded that the MBP test and the MFBP test were very useful for predicting low temperature cracking of asphalt pavements.

McLeod (1987) created a chart for selecting paving asphalts with various combinations of temperature susceptibilities (PVNs) and penetrations at 25°C to avoid low temperature transverse cracking at minimum winter pavement temperatures. This chart was based on test road data, the study conducted by Haas et. al. (1987), and the observations of the field performance of the highways.

The empirical methods are easy to use, but they have their limitations. If the conditions exceed the limitations, these methods will no longer hold valid.

5.1.2 Theoretical Methods

Theoretical methods are based on analyses of mechanisms of low temperature cracking in asphalt pavement. Generally, there are two mechanisms in use. One of these is based on **strain criteria** which states that in the asphalt mix used in the pavement, if the thermally induced strain due to temperature drop exceeds the failure strain, cracking of the pavement will occur. The other method is based on **stress criteria** which states that in the asphalt mix used in the pavement, if the thermally induced stress due to temperature drop exceeds the tensile strength of the material, cracking will occur. This is shown in Fig.5.1 which is Fig.3 in the paper presented by Hills and Brien (1966). Presently, all the theoretical methods published are limited in not being able to calculate the cracking frequency.

The research work done by Tam, Joseph, and Lynch (1990) is a typical application of the strain criteria. In this study, the direct tensile test was performed to determine the failure strain of the asphalt mixes (recycled and virgin mixes), and a thermal contraction test was conducted to measure the thermally induced strain. The cracking temperature was estimated by plotting the failure strain and thermally induced strain against temperature. The temperature at which the thermally induced strain exceeded the failure strain was considered as the required cracking temperature. It is apparent that the stress relaxation process was not considered in this method.

The most extensively used cracking mechanism is based on stress criteria. By this mechanism, Hills and Brien (1966) developed a method to calculate cracking temperature based on the asphalt stiffness concept from van der Poel (1954) and the assumption of an infinite bitumen beam. The stress calculation of Hills and Brien approach can be expressed by following equation:

$$\sigma = \sum_{T_o}^{T_f} \alpha \, S(T,t) \, \Delta T \qquad (1)$$

where

σ = thermally induced stress,

α = the coefficient of thermal expansion,

ΔT = temperature interval,

$S(T,t)$ = stiffness modulus,

T = temperature, the middle value of the temperature interval ΔT,

$t = \Delta T / CR$, loading time,

CR = cooling rate.

They used a tensile strength obtained from the Heukelom chart (1966). The calculation process is shown in Table 5.2 which is slightly modified from Table 1 in the paper presented by Hills and Brien (1966). The principle of cracking temperature prediction based on stress criteria is shown in Fig.5.1. In this figure, both stiffness values and strength values were plotted together against temperature so that the cracking temperature can be obtained by finding the temperature at which the two curves intercept.

Christison et. al. (1972) analyzed several methods for the calculation of the thermally induced stress. These methods include the analyses of a **i)** pseudo-elastic beam, **ii)** approximate pseudo-elastic slab, **iii)** viscoelastic beam, **iv)** viscoelastic slab, and **v)** approximate viscoelastic slab.

Table 5.2-Calculation of Thermal Stress by Hills and Brien Approach (1966)

From equation(1): $\sigma = \Sigma\, S\, \alpha\, \Delta T$

Penetration at 25 C: 100 | Penetration Index: -1.0

Softening point ($T_{R\&B}$): 44°C | Coefficient of linear thermal contraction: 2×10^{-4} °C^{-1}

T (°C)	$T_{R\&B}$ - T (°C)	S (kg/cm²)	$\Delta\varepsilon = \alpha\, \Delta T$	$\Delta\sigma = S\, \Delta\varepsilon$ (kg/cm²)	$\sigma = \Sigma\, \Delta\sigma$ (kg/cm²)	$\sigma_T = f(S)$ (kg/cm²)
0	44				0	
-5	49	2	0.002	0.004		6
-10					0.004	
-15	59	40	0.002	0.08		22
-20					0.084	
-25	69	500	0.002	1		53
-30					1.1	
-35	79	2500	0.002	5		42
-40					6.1	
-45	89	10000	0.002	20		38
-50					26.1	
-55	99	18000	0.002	36		37
-60					62.1	

Note: T is the temperature.
S is the stiffness modulus for a loading time of 1 hour, taken from van der Poel Nomograph.
$\Delta\varepsilon$ is the thermal strain for the temperature interval of 10°C.
α is the coefficient of linear thermal contraction.
$\Delta\sigma$ is the increment in thermal stress.
σ is the thermal stress.
σ_T is the tensile strength corresponding to S, taken from Fig.12 in Heukelom (1966).

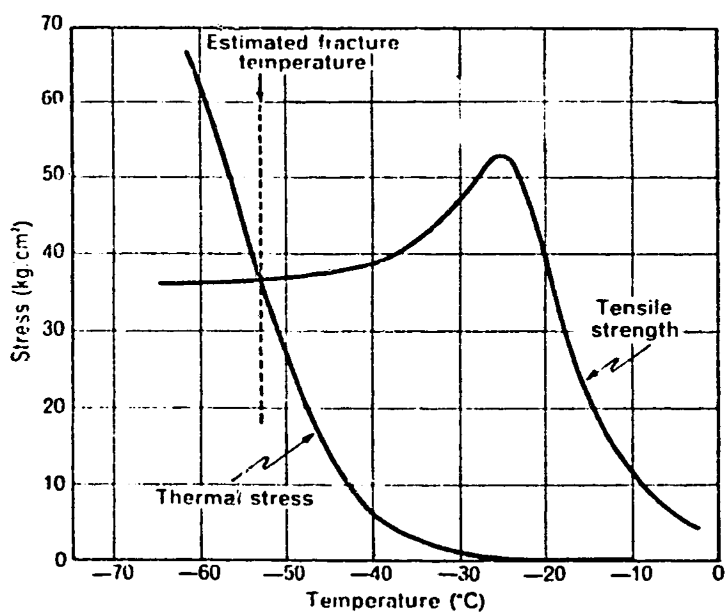


Fig.5.1-The Principle of Cracking Temperature Prediction

Based on the comparison of the results calculated from different analysis methods with the observed results in Ste. Anne test road, it was then concluded that methods more sophisticated than the pseudo-elastic beam method did not offer any advantages for the problem under consideration. However, the pseudo-elastic beam method developed by Hills and Brien had a problem in that the predicted stress from this method was dependent on the time interval used in the calculation. The maximum computed stress using the 15-minute time increment was approximately 50 percent greater than that computed using the 2-hour time increment (Christison et al. 1972).

In view of this shortcoming of the method suggested by Hills and Brien, Christison et al. (1972) empirically used a constant loading time of 7200 seconds which was independent of the temperature intervals. This selected loading time corresponded to the time interval of temperature input from Ste. Anne Test Road data.

Later, Finn et al. (1977, 1986) adapted Christison's method in the COLD program (Computation of Low Temperature Damage) under a NCHRP project (National Cooperative Highway Research Program). Recently, May and Witczak (1992) used this method in an asphalt concrete mix analysis program-CAMA (Computer Assisted Asphalt Mixture Analysis).

Robertson (1987) recognized the problem of the method from Hills and Brien and used a numerical integration technique to calculate the thermally induced stress. The loading time for the modulus at each temperature was taken as the time required to cool from that temperature to the final temperature. The author used a constant cooling rate of 10°C/hour after determining that the calculated stress was not very sensitive to the cooling rate. The fracture temperature was taken as the temperature required to develop a fracture stress value of 5×10^5 N/m^2 obtained by Hills for asphalt binders. Using this value and the experimentally determined asphalt stiffness, Robertson constructed a rational design chart for selecting asphalts for low temperature service. This rational design chart is shown in Fig.V4 in Appendix V.

Wang and Bai (1988) analyzed the relaxation process of the thermally induced stress in asphalt pavement. By using the Boltzmann superposition technique (Ward, 1971) under the assumption of linear behavior of asphalt mixes, the authors developed a method to predict thermally induced stress in an asphalt pavement. Any cooling rate or any process of temperature dropping could be used in the method. Both temperature susceptibility and time susceptibility of asphalt were considered in the calculation of thermal stress. However, there was no cracking temperature prediction method presented.

In this chapter, three typical existing methods as listed below are used to predict the cracking temperatures based on the information from Ste. Anne test road and the C-SHRP Lamont test sections in Alberta.

- Critical Stiffness Method from Deme and Young (1987) (empirical method),
- The Method used in CAMA (May and Witczak 1992) (theoretical method), and
- Robertson's Method (1987) (theoretical method),

Following this, an **Improved Theoretical Method** for cracking temperature prediction is presented. Finally, the observed cracking information will be compared with the predicted cracking temperatures, and the results from these different methods will also be compared.

5.2 The Materials Used in the Cracking Temperature Prediction Study

The reported properties of the materials in the Ste. Anne Test Road (Deme and Young, 1987) and Lamont Test Road (Gavin, 1992) are used in this study. The material properties from the Ste. Anne Test Road are shown in Table 5.3. The asphalt classification is shown in Fig.5.2 which is a slightly modified version of Fig.1 from the CGSB (Canadian General Standard Board) specification CAN/CGSB-16.3-M90. From Fig.5.2, it can be seen that the recovered asphalt HV150/200 is in the position of Group A, but it is too hard to fall in any of the categories. The original asphalt HV150/200 belongs to Group B, the original asphalts LV150/200 and LV300/400 are classified as Group C, and the recovered asphalts LV150/200 and LV300/400 are below Group C.

The aggregate for the hot-mix surface on some of the Ste. Anne test sections consisted of 100% crushed igneous aggregate (predominantly microcrystalline basalt and macrocrystalline aggregate). On other sections the aggregate consisted of 80% limestone/20% igneous glacial drift aggregate.

The properties of the materials used in the Lamont Test Road given earlier in Chapter Four are reorganized and presented in Table 5.4. The Vg, Vb, and Va in the tables are the volume percentages of aggregate, asphalt, and air voids in asphalt mixtures respectively.

Table 5.3-Properties of Asphalts and Mixes Used in Ste. Anne Test Road

Asphalts		Original (67)			Field Aged or Recovered (72)		
		HV150/200	LV150/200	LV300/400	HV150/200	LV150/200	LV300/400
Pen(25°C, dmm)		159	192	313	55	67	119
Pen(15°C, dmm)		/	/	/	20	20	34
Pen(5°C, dmm)		/	/	/	8.4	7.5	11
Pen(4°C, dmm)		14	10	14	/	/	/
Vis.(60°C, Pa.s)		59.1	25.3	14.1	259.0	81.3	33.5
Vis. (135°C, mm^2/s)		225	110	86	370	167	117
Sft. Pt.(R&B,°C)		39.0	35.0	31.2	53.7	48.8	41.7
T(800 pen.),(°C)		38.9	35.1	31.3	53.5	47.7	41.1
Pen. Index(P/P)		-1.50	-2.60	-2.90	-0.13	-1.10	-1.60
Mix Info. (%)	Vg	83.7	87.3	83.6	83.7	87.3	83.6
	Vb	11.4	9.7	11.4	11.4	9.7	11.4
	Va	4.9	3.0	5.0	4.9	3.0	5.0

Table 5.4-Properties of the Asphalts and Mixes Used in Lamont Test Road

Test Section No.		1	2	3	4	5	6	7
Asphalts Source		Esso (Blown)	Montana	Esso	Esso	Husky (Blown)	Husky	Esso
Pen(25°C, dmm)		100	150	333	93	88	176	241
Pen(10°C, dmm)		22	20	58	12	21	28	45
Pen(5°C, dmm)		13	11	36	6	14	17	25
Vis.(60°C, Pa.s)		96.0	59.8	31.3	74.9	321	83.8	47.1
Vis. (135°C, mm^2/s)		277	214	163	219	530	280	195
Sft. Pt.(R&B, °C)		/	41.9	30.7	/	49.8	36.5	/
T(800 pen.), (°C)		45.4	37.8	32.9	40.7	48.9	37.9	35.6
Pen.Index(P/P)		-0.65	-2.22	-1.28	-2.45	-0.05	-1.58	-1.32
Mix Info. (%)	Vg	84.9	85.0	84.5	84.4	84.5	84.9	84.5
	Vb	11.6	11.6	12.0	12.1	12.0	11.6	12.0
	Va	3.5	3.4	3.5	3.5	3.5	3.5	3.5

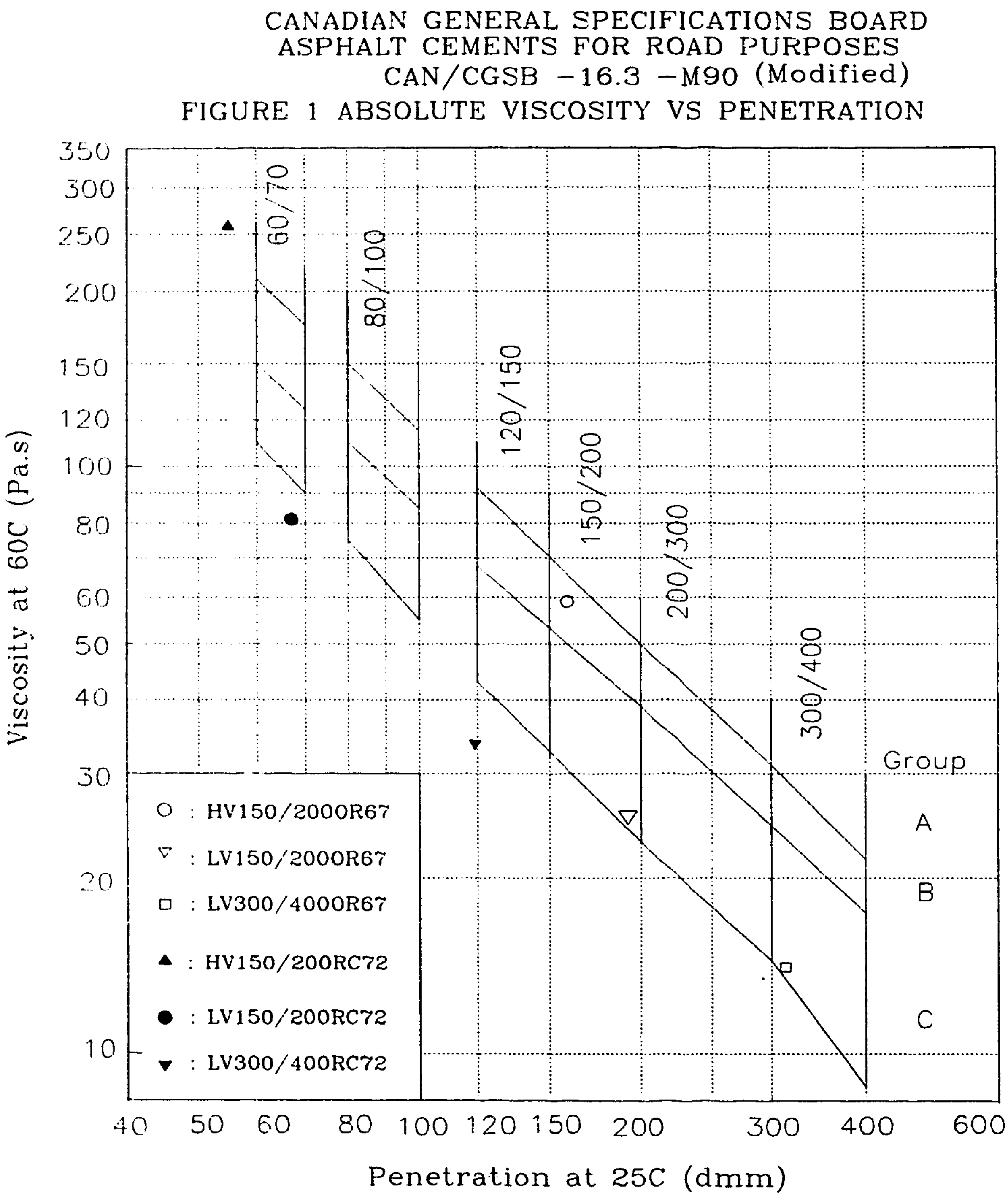


Fig.5.2-The Classification of the Asphalts Used in Ste. Anne Test Road

5.3 Prediction of Cracking Temperatures by Various Existing Methods

The following three typical existing methods are used to predict the cracking temperatures:

- Critical Stiffness Method from Deme and Young (1987) (empirical method),
- The Method used in CAMA (May and Witczak 1992) (theoretical method), and
- Robertson's Method (1987) (theoretical method),

The observed cracking information from the C-SHRP Lamont test sections in Alberta was obtained on March 30, 1993 as shown in Table 5.5 (Wang et al. 1993).

Table 5.5-Observed Cracking Information in Lamont Test Road

Test Section No.	1	2	3	4	5	6	7
Test Section Length (m)	428	491	417	435	467	419	500
Observed No. of Cracks	12	45	0	48	1	0	0
Cracking Frequency (Cracks /km)	28	92	0	110	2	0	0

From this table, we can see that in the Lamont Test Road the most serious cracking occurred in Test Section No.4. Following this section are Test Section No.2, 1, and 5 in the order of the cracking frequencies. There were no cracks found in sections 3, 6, and 7. It could be also reasoned that section 5 had no low temperature cracking since only one crack was found in the entire section.

Instrumentation for detection of cracking temperatures has been installed at the Lamont Test Road. However, this information is not yet available. Such observed cracking temperatures have been reported from the Ste. Anne Test Road (Deme and Young, 1987). This information were obtained after the first year (Original 67-68) and after 5 years (Recovered 71-72) of the construction. The data from both the Lamont Test Road and the Ste. Anne Test Road will be used to examine the validity of the various prediction methods.

5.3.1 Critical Stiffness Method from Deme and Young

A critical stiffness of 1×10^9 N/m^2 at 1800 second loading time is suggested by Deme and Young (1987) based on the correlation of the asphalt stiffness at 1800 seconds with road cracking observations. This empirical method assumes that the low temperature stiffness of asphalt at loading time of 1800 seconds is the dominant factor influencing low temperature cracking of asphalt pavement. The detailed calculation process is shown in Appendix V. Table 5.6 shows the results of the cracking temperatures predicted with this method and compared with the observed data.

Table 5.6-Cracking Temperature Prediction with Critical Stiffness Method from Deme and Young

Asphalt		Cracking Temperature (°C)	
Ste. Anne Test Road		Predicted	Observed
HV 150/200	Original (67-68)	-47	< -38
	Recovered (71-72)	-49	-34
LV 150/200	Original (67-68)	-36	-34
	Recovered (71-72)	-41	/
LV 300/400	Original (67-68)	-36	-37
	Recovered (71-72)	-42	-34
Lamont Test Road		Predicted	Cracking Frequency (Cracks/km)
TS 1	Esso 80/100B	-51	28
TS 2	Montana 150/200B	-37	92
TS 3	Esso 300/400A	-55	0
TS 4	Esso 80/100C	-32	110
TS 5	Husky 80/100A	-54	2
TS 6	Husky 150/200A	-46	0
TS 7	Esso 200/300A	-52	0

5.3.2 The Method Used in CAMA

Based on the Hills and Brien method and used by Finn et al. (1986) in the COLD program, the method used in CAMA (May and Witczak, 1992) adopts a constant loading time of 7200 seconds to calculate mix stiffness and resultant thermal stress. This method takes account of asphalt stiffness, asphalt mix composition, and asphalt mix tensile strength. Table 5.7 shows the cracking temperatures predicted with this method compared with observed data for the two test roads. The detailed input and output of the cracking temperature calculation by CAMA are given in Appendix V.

Table 5.7-Cracking Temperature Prediction with the Method Used in CAMA

Asphalt		Cracking Temperature (°C)	
Ste. Anne Test Road		Predicted	Observed
HV 150/200	Original (67-68)	-40.0	< -38
	Recovered (71-72)	-36.4	-34
LV 150/200	Original (67-68)	-30.0	-34
	Recovered (71-72)	-30.0	/
LV 300/400	Original (67-68)	-33.1	-37
	Recovered (71-72)	-34.2	-34
Lamont Test Road		Predicted	Cracking Frequency (Cracks/km)
TS 1	Esso 80/100B	-46.4	28
TS 2	Montana 150/200B	-39.7	92
TS 3	Esso 300/400A	-51.1	0
TS 4	Esso 80/100C	-35.3	110
TS 5	Husky 80/100A	-50.6	2
TS 6	Husky 150/200A	-49.2	0
TS 7	Esso 200/300A	-51.1	0

5.3.3 Robertson's Method

Table 5.8 shows the results of the cracking temperatures predicted by this method for both Ste. Anne and Lamont test roads. A detailed input of the cracking temperature calculation is given in Appendix V.

Table 5.8-Cracking Temperature Prediction with Robertson's Method

Asphalt		Cracking Temperature (°C)		
Ste. Anne Test Road		Design*	Predicted	Observed
HV 150/200	Original (67-68)	-26	-36	< -38
	Recovered (71-72)	-19	-29	-34
LV 150/200	Original (67-68)	-20	-30	-34
	Recovered (71-72)	-16	-26	/
LV 300/400	Original (67-68)	-29	-39	-37
	Recovered (71-72)	-19	-29	-34
Lamont Test Road		Design	Predicted	Cracking Frequency (Cracks/km)
TS 1	Esso 80/100B	-24	-34	28
TS 2	Montana 150/200B	-19	-29	92
TS 3	Esso 300/400A	< -45	< -55	0
TS 4	Esso 80/100C	> -15	> -25	110
TS 5	Husky 80/100A	-25	-35	2
TS 6	Husky 150/200A	-27	-37	0
TS 7	Esso 200/300A	-37	-47	0

Note: *Predicted Cracking Temperature (°C) = Design Temperature - 10.

As described previously this method utilizes a design chart developed by Robertson. This design chart was developed from the fracture temperatures taken as the temperature required to develop a tensile stress 5×10^5 N/m^2 in asphalt binders. Experimentally determined modulus values were used in the stress calculation on asphalt binders selected to represent wide range of wax contents and temperature susceptibilities.

5.4 Improved Theoretical Method for Cracking Temperature Prediction

5.4.1 Analysis on the Stress Calculation of the Hills and Brien Method

The stress calculation of Hills and Brien approach is expressed in Equation (1) mentioned previously in this chapter.

$$\sigma = \sum_{T_o}^{T_f} \alpha\, S(T,t)\, \Delta T \qquad (1)$$

In this formula, the time (t) is a cooling time through a temperature interval (ΔT); however, the cooling time is not the loading time. The following is an explanation of the Hills and Brien approach from three aspects: mathematical analysis of the aquation, thermal stresses obtained from different values of ΔT, and difference between cooling time and loading time. From these analyses, the method to calculate thermal stress as suggested by Wang and Bai (1988) will be developed.

1) Mathematical Analysis of the Equation

A specimen of an asphalt beam is cooled through a temperature interval (ΔT) in a period of time (t). The cooling rate (CR) is constant. Without restraint, the contraction strain of the asphalt beam would be

$$\Delta\varepsilon = \alpha\, \Delta T.$$

If the specimen is restrained entirely, the increment in thermal stress induced in the beam should be

$$\Delta\sigma = S(T,t)\,\Delta\varepsilon$$

$$= \alpha\, S(T,t)\,\Delta T$$

If the temperature from "T_0" drops down "n" steps of "ΔT" to "T_f", the final thermal stress will be

$$\sigma = \sum^{n} \Delta\sigma$$

$$= \sum^{n} \alpha\, S(T,t)\,\Delta T$$

or assuming α is a constant,

$$\sigma = \alpha \sum_{T_o}^{T_f} S(T,t)\,\Delta T$$

This is the Formula (1) which is an approximate formula. The smaller ΔT is and the larger n is, the more accurate the stress (σ) will be. Only when $\Delta T \rightarrow 0$ and $n \rightarrow \infty$, can the most accurate value of the stress (σ) be obtained:

$$\sigma = \alpha \int_{T_f}^{T_O} S(T,t)\,dT$$

However

$$\lim_{\substack{\Delta T \rightarrow 0 \\ n \rightarrow \infty}} (t) = \lim_{\substack{\Delta T \rightarrow 0 \\ n \rightarrow \infty}} (\Delta T / CR) = 0,$$

thus

$$\sigma = \alpha \int_{T_f}^{T_o} S(T,0)\,dT$$

According to van der Poel (1954), when loading time tends to zero, the stiffness of asphalt tends to be a constant (3×10^9 N/m^2 = 3.06×10^4 kg/cm^2), i.e.

$$\lim_{t\to 0} S(T,t) = S(T,0) = 3.06\times10^4 \text{ (kg/cm}^2\text{)}$$

Therefore,

$$\sigma = \alpha \int_{T_f}^{T_o} S(T,0)\, dT = \alpha \times 3.06\times10^4 \times (T_o - T_f)$$

In this equation, the stress is independent of the cooling rate and the properties of the asphalt. Obviously this is in error which means that formula (1) must also be fundamentally incorrect.

2) Thermal Stresses Obtained from Different Values of ΔT

Table 5.2 given on page 100 in this chapter shows the procedure to calculate the thermal stress by Hills and Brien approach. Table 5.9 shows the different thermal stress results with the same conditions as Table 5.2 when different values of ΔT are adopted.

Table 5.9-Thermal Stress Obtained with Different Values of ΔT by Hills and Brien approach

No.	ΔT (°C)	t = ΔT/CR* (hour)	σ (kg/cm^2)
1	20	2	42.4
2	10	1	62.1**
3	1	0.1	83.3
4	0.5	0.05	104.3
5	0.05	0.005	136.4
6	$\Delta T \to 0$	$t \to 0$	367.2

Note: *CR is the cooling rate of 10°C/hour.
**This result of 62.1 (kg/cm^2) is same as the result from Table 5.2.

From Table 5.9, it can be seen that the larger the ΔT is, the smaller the stress is, and vice versa. The stress from the smallest value (42.4 kg/cm^2) changes to the largest one (367 kg/cm^2) as ΔT changes from the largest (20°C) to the smallest (0°C). It is difficult to say which value of the stress is correct in the range. This means that formula (1) cannot give a uniquely correct result for the thermal stress value.

3) Difference between Cooling Time and Loading Time

The definitions of loading time and cooling time are basically simple in concept. The loading time is the period from the time at which the stress is exerted to the time at which the stress is calculated; The cooling time is just the duration of the drop in temperature over a controlled range. In order to analyze the difference between these two concepts in the thermal stress relaxation process, the Boltzmann superposition principle (Ward, 1971) has to be discussed first.

Like previous studies, the linear viscoelastic behavior of asphalt material is assumed. Therefore, application of the Boltzmann superposition principle is as follows:

i. The relaxed stress in a specimen is a function of the entire loading history,
ii. Each strain step makes an independent contribution to the final stress and the final stress can be obtained by the simple addition of each contribution.

According to this principle, the procedure of the relaxation and superposition of the thermal stress can be illustrated in Fig.5.3.

Practically, the temperature is a function of time:

$$T = f(\theta)$$

where

θ = time

T = temperature

In Fig.5.3, a linear relationship between temperature and time is assumed, i.e. cooling rate is a cor...ant. The reverse function can be expressed as:

$$\theta = \varphi(T)$$

When the temperature goes down from T_i to T_{i+1}, the temperature interval is ΔT_i. And it is assumed that no cooling time is needed for this very small and instantaneous temperature dropping. So the instantaneous (i.e. loading time, t = 0) thermal stress increment should be

$$\Delta\sigma_{oi} = \alpha\, S(T_i,0)\, \Delta T_i$$

Then the stress increment will relax until the time at which the thermal stress will be calculated. So the final relaxed stress increment should be

$$\Delta\sigma_i = \alpha\, S(T_i,t_i)\, \Delta T_i$$

where

t_i = loading time, to be calculated as follows:

$$t_i = \theta_f - \theta_i = \varphi(T_f) - \varphi(T_i)$$

If the relationship between temperature and time is linear,

$$T = CR \times \theta + T_o$$

or

$$\theta = (T - T_o) / CR$$

then

$$\theta_f = (T_f - T_o) / CR$$
$$\theta_i = (T_i - T_o) / CR$$

So

$$t_i = (T_f - T_i) / CR$$

In Fig.5.3, the loading time "t_i" stands for the period from the time θ_i at which the stress or the strain is exerted to the time θ_f at which the stress is calculated. On the other hand, "ΔT_i / CR" only stands for a cooling time interval through the temperature interval ΔT_i and has nothing to do with the loading time. So it is not surprising that if "ΔT_i / CR" is used as loading time, the calculated thermal stress will be incorrect.

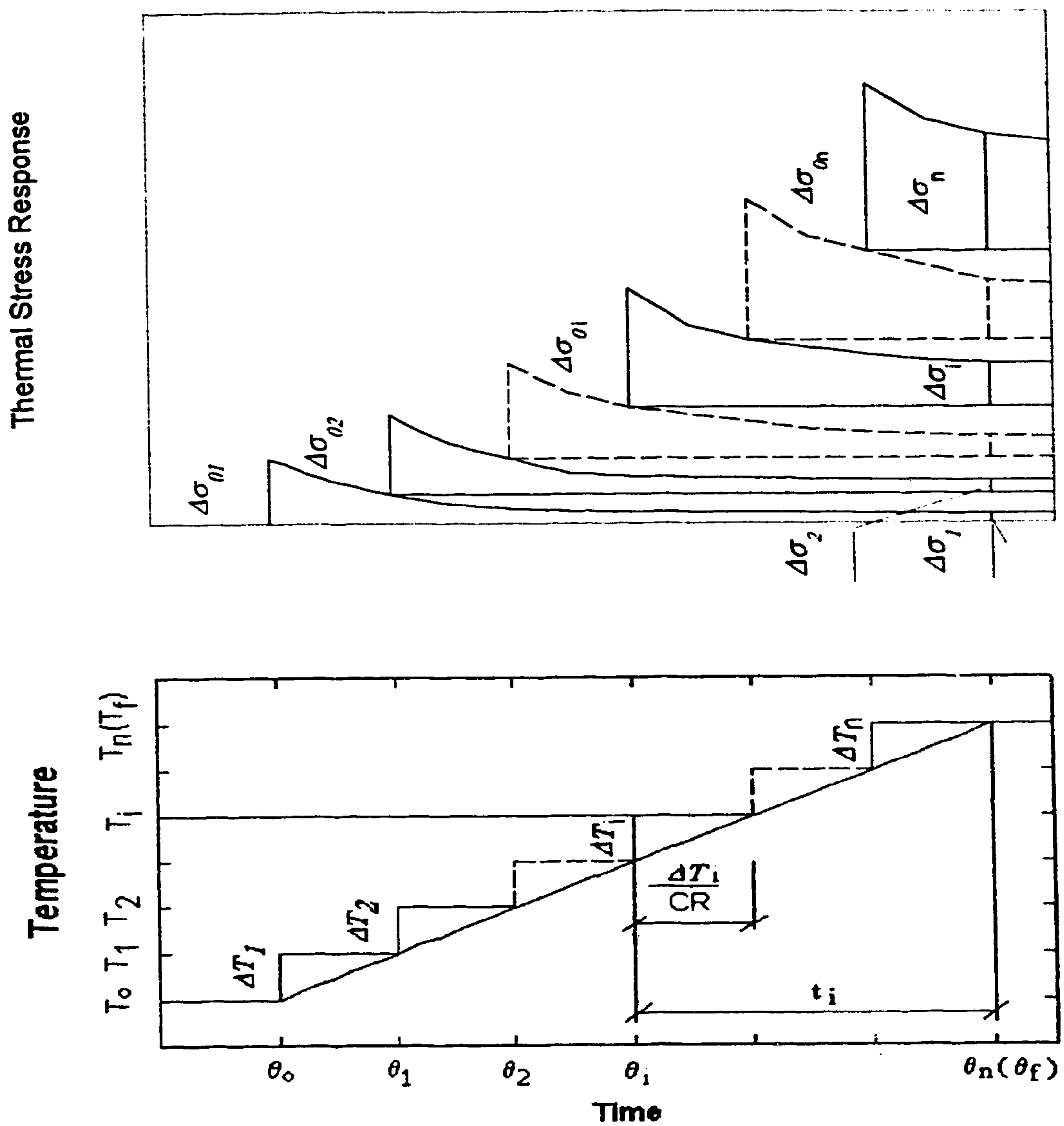


Fig.5.3-Relaxation and Superposition of Thermal Stress

According to the Boltzmann superposition principle, the final stress is a simple addition of each relaxed stress increment.

$$\sigma = \sum_{i=1}^{n} \Delta\sigma$$

$$\sigma = \alpha \sum_{i=1}^{n} S(T_i,t_i)\,\Delta T_i$$

$$\sigma = \alpha \sum_{i=1\,(T=T_o)}^{n\,(T_f)} S(T_i,\theta_f-\theta_i)\,\Delta T_i \qquad (2)$$

When $n \rightarrow \infty$ and $\Delta T_i \rightarrow 0$ ($i=1,2,3,\ldots\ldots,n$),

$$\sigma = \alpha \int_{T_f}^{T_o} S(T,t)\,dT \qquad (3)$$

where

$$t = \varphi(T_f) - \varphi(T)$$

Both Formula (2) and Formula (3) are the equations to calculate the thermally induced stress in an asphalt pavement. The procedure to calculate thermal stress by using Formula (2) is shown in Table 5.10. The input data are the same as in Table 5.2. The values of ΔT_i are obtained according to the following rules. When the temperature is relatively high, and the stiffness is so low that it does not influence the thermal stress much, the values of ΔT_i are chosen relatively large to facilitate the calculation. When the temperature is relatively low, the values of ΔT_i should be chosen relatively small, in order to obtain an accurate result. Certainly, the smaller the ΔT_i, the more accurate the result.

The regression of S = f(T) based on the data in Column (1) and Column (5) in Table 5.10 has been done. The result is

$$S = \begin{cases} 10^{(-0.1086\times T-0.6918)} & 0 > T \geq -40^\circ C. \\ -35{,}730-956.5\times T & -40 > T \geq -59^\circ C. \\ -357{,}800-6{,}413\times T & -59 > T \geq -60^\circ C. \end{cases}$$

where

S = The asphalt stiffness calculated in Column (5) in Table 5.10, (kg/cm^2),
T = The temperatures corresponding to Column (1) in Table 5.10, (°C).

Therefore, the thermal stress can be calculated by formula (3). The result is 51 kg/cm^2 which is slightly smaller than the result (57 kg/cm^2) which is calculated by using Formula (2). Theoretically, it is considered that the value of 51 kg/cm^2 is more accurate than the value of 57 kg/cm^2.

Table 5.10-Calculation of Thermal Stress Using Formula (2)

No.	(1) T_i (°C)	(2) ΔT_i (°C)	(3) t_i (s)	(4) $T_{R\&B} - T_i$ (°C)	(5) S_i (kg/cm^2)	(6) $\Delta\sigma = \alpha\, S_i\, \Delta T_i$ (kg/cm^2)
1	0		21600	44		0
2	-10	10	18000	54	2	0.004
3	-15	5	16200	59	8	0.008
4	-20	5	14400	64	35	0.035
5	-25	5	12600	69	130	0.13
6	-30	5	10800	74	450	0.45
7	-35	5	9000	79	1400	1.4
8	-40	5	7200	84	3200	3.2
9	-43	3	6120	87	5600	3.36
10	-45	2	5400	89	7600	3.04
11	-48	3	4320	92	10000	6
12	-50	2	3600	94	11200	4.48
13	-52	2	2880	96	12600	5.04
14	-54	2	2160	98	15600	6.24
15	-56	2	1440	100	18000	7.2
16	-57	1	1080	101	19000	3.8
17	-58	1	720	102	20200	4.04
18	-59	1	360	103	21500	4.3
19	-59.5	0.5	180	103.5	22900	2.29
20	-59.8	0.3	72	103.8	23600	1.42
21	-59.95	0.15	18	103.95	24800	0.744
22	-59.99	0.04	3.6	103.99	25800	0.206
23	-59.995	0.005	1.8	103.995	27500	0.027
24	-59.998	0.003	0.72	103.998	29000	0.017
25	-60	0.002	0	104	30600	0.012
$\sigma = \Sigma\, \Delta\sigma = 57$ (kg/cm^2), or $\sigma = \Sigma\, \Delta\sigma = 5.6$ (MPa)						

Note: 1. The conditions are same as in Table 5.2.
2. Loading time t_i (seconds) = $(T_f - T_i)$ / CR = 21600 + 360 × T_i
where, T_f = -60°C. and CR = -10°C/hour = (-1 / 360)°C/second.

Any relationship between temperature and time can be used in formula (2) and (3). In the two formulae, not only the properties of asphalts and the temperature but also the cooling rate has an important influence on the thermally induced stress in asphalt pavement. In addition to temperature susceptibility, this method also emphasizes the time susceptibility of the material which is defined as the amount of the change in the stiffness with loading time. The thermally induced stress in the asphalts which have high time susceptibilities tend to relax faster than the one in the asphalts which have low time susceptibilities.

In order to compare with the result from Hills and Brien (Table 5.2), the material used in the previous calculation of thermal stress has been the asphalt binder. For the asphalt mixes, the calculation procedure is the same except that the stiffness values of the asphalt mixes should be used rather than the stiffness values of the asphalt binders.

5.4.2 Analysis of the Method for Obtaining Failure Stress

Generally, there are two ways to obtain the failure stress or tensile strength: one is from direct test, another is from nomographs. The direct test is more accurate, but much more expensive and time consuming. Therefore, usually the nomographs are used to obtain the failure stress or tensile strength.

From the previous sections, it is reasoned that all the stiffness values in Table 5.2 are inaccurate because the cooling time is erroneously used as loading time. In a similar manner, the strength values are also not correct because they are obtained from Heukelom's chart (1966) based on the corresponding stiffness values in Table 5.2.

In order to obtain an alternatively estimated tensile strength corresponding to certain conditions of asphalt pavement from nomographs like Heukelom's chart (1966), a concept of "Practical Stiffness" is used. This term is defined as the stiffness corresponding to the same conditions existing in the asphalt pavement. The practical stiffness is difficult to measure and can not be predicted from the Van der Poel nomograph because the loading time is not known. However, it may be derived from the calculated thermal stress and thermal strain.

$$S_{pr} = \sigma / \varepsilon$$

where

S_{pr} = practical stiffness modulus

σ = thermally induced stress

ε = thermal strain

The method to calculate the thermal stress using Formula (2) and (3) has been discussed previously, and the thermal strain may be calculated as follows:

$$d\varepsilon = \alpha \, dT$$

$$\varepsilon = \int_{T_f}^{T_o} \alpha \, dT$$

where

α = coefficient of thermal contraction.

If α is a constant,

$$\varepsilon = \alpha \, (T_o - T_f)$$

where

T_o = The initial temperature

T_f = The final temperature

Finally, based on the calculated practical stiffness, the tensile strength may be obtained from one of the plots of tensile strength vs. mix stiffness as shown in Fig.4.9.

5.4.3 Cracking Temperature Prediction

After the acquisition of the thermally induced stress and tensile strength, a plot similar to Fig.5.1 may be plotted to predict cracking temperature. A computer program has been developed to calculate the cracking temperatures based on this Improved Theoretical Method. The detailed input and output of this program named "Asphalt Property Evaluation at Low Temperatures" (ASPELT) is presented in Appendix V.

Table 5.11a and Table 5.11b show the results of the cracking temperatures predicted with this Improved Theoretical Method.

Table 5.11-Cracking Temperature Prediction with the Improved Theoretical Method

5.11a-Asphalt Mixes Used in Ste. Anne Test Road			
Asphalt		Cracking Temperature (°C)	
		Predicted	Observed
HV 150/200	Original (67-68)	-39.5	< -38
	Recovered (71-72)	-36.1	-34
LV 150/200	Original (67-68)	-32.7	-34
	Recovered (71-72)	-31.6	/
LV 300/400	Original (67-68)	-37.0	-37
	Recovered (71-72)	-36.2	-34

Note: 1) Coefficient of contraction of asphalt mix is 2.04×10^{-5} which is the average value of the measured coefficients of the three asphalt mixes.
2) Cooling rate is 1.5°C/hour which is similar to the field cooling rate.

5.11b-Asphalt Mixes Used in Lamont Test Road						
Test Section	Asphalt Source	Predicted Cracking Temperature (°C)				**Cracking Frequency (Cracks/km)**
		Failure Stress from Deme & Young (1987)		Failure Stress from Indirect Tensile Test (Wang et al., 1992)		
		$\alpha1$	$\alpha2$	$\alpha1$	$\alpha2$	
TS 1	Esso 80/100B	-36.9	-30.8	-41.1	-35.4	28
TS 2	Montana 150/200B	-32.6	-27.6	-36.1	-31.1	92
TS 3	Esso 300/400A	-44.2	-37.7	-48.9	-42.5	0
TS 4	Esso 80/100C	-28.7	-23.8	-32.0	-27.0	110
TS 5	Husky 80/100A	-38.3	-31.6	-42.7	-36.4	2
TS 6	Husky 150/200A	-36.9	-31.4	-41.0	-35.7	0
TS 7	Esso 200/300A	-41.2	-35.3	-45.7	-39.6	0

Note: 1) The cooling rate used in the table is 3°C/hour.
2) $\alpha1 = 2.04\times10^{-5}$ /°C which is the average value of the contraction coefficients of the three asphalt mixes used in Ste. Anne Test Road.
3) $\alpha2 = (Vb \times Cb + Vg \times Cg) / 300$, where
Vb is volume percentage of asphalt in asphalt mixture.
Vg is volume percentage of aggregate in asphalt mixture.
$Cb = 6\times10^{-4}$ /°C is the volume contraction coefficient of asphalt.
$Cg = 3\times10^{-5}$ /°C is the volume contraction coefficient of aggregate.

5.5 Comparison among the Results Obtained from Different Methods

Fig.5.4 is plotted from the data in Table 5.6 for the Critical Stiffness Method from Deme and Young. From Fig.5.4b, a good correlation is obtained between the predicted cracking temperatures and the observed cracking frequencies in the Lamont Test Road. This means that the asphalt stiffness does have an important influence on the low temperature cracking of asphalt pavement as assumed by the critical stiffness method. However, from Fig.5.4a, the relationship can hardly be seen between the predicted cracking temperatures and the observed cracking temperatures in Ste. Anne Test Road, and the predicted temperature are usually lower than the observed cracking temperatures. This means this method is not accurate enough to predict cracking temperatures because many other important influencing factors are not considered such as the cooling rate, the change of the asphalt stiffness with loading time, the mix composition, etc.

Fig.5.5 is plotted from Table 5.7 for the Method Used in CAMA. From Fig.5.5b, a good correlation is achieved between the predicted cracking temperature and the observed cracking frequencies in the Lamont Test Road. However, from Fig.5.5a which is for Ste. Anne Test Road, the poor accuracy for the cracking temperature prediction is observed. One of the reasons for this should be that the cooling rate and the change of the stiffness with loading time are not considered because an arbitrary loading time of 7200 seconds is used in this method.

Fig.5.6 is the plot corresponding to Table 5.8 for the Robertson's Method. From Fig.5.6b which is for Lamont Test Road, the relationship between predicted cracking temperatures and the observed cracking frequencies is good. But from Fig.5.6a which is for Ste. Anne Test Road, the predicted cracking temperatures generally are warmer than the observed cracking temperatures. One of the reasons could be that a much higher cooling rate of 10°C/hour was used than the actual cooling rate of approximate 1.5°C/hour in the test road.

This method ignores the influences of the cooling rate and the mix composition on the stiffness and tensile strength of asphalt mixes, so that any changes in cooling rate and mix composition would not influence the results of the predicted cracking temperatures.

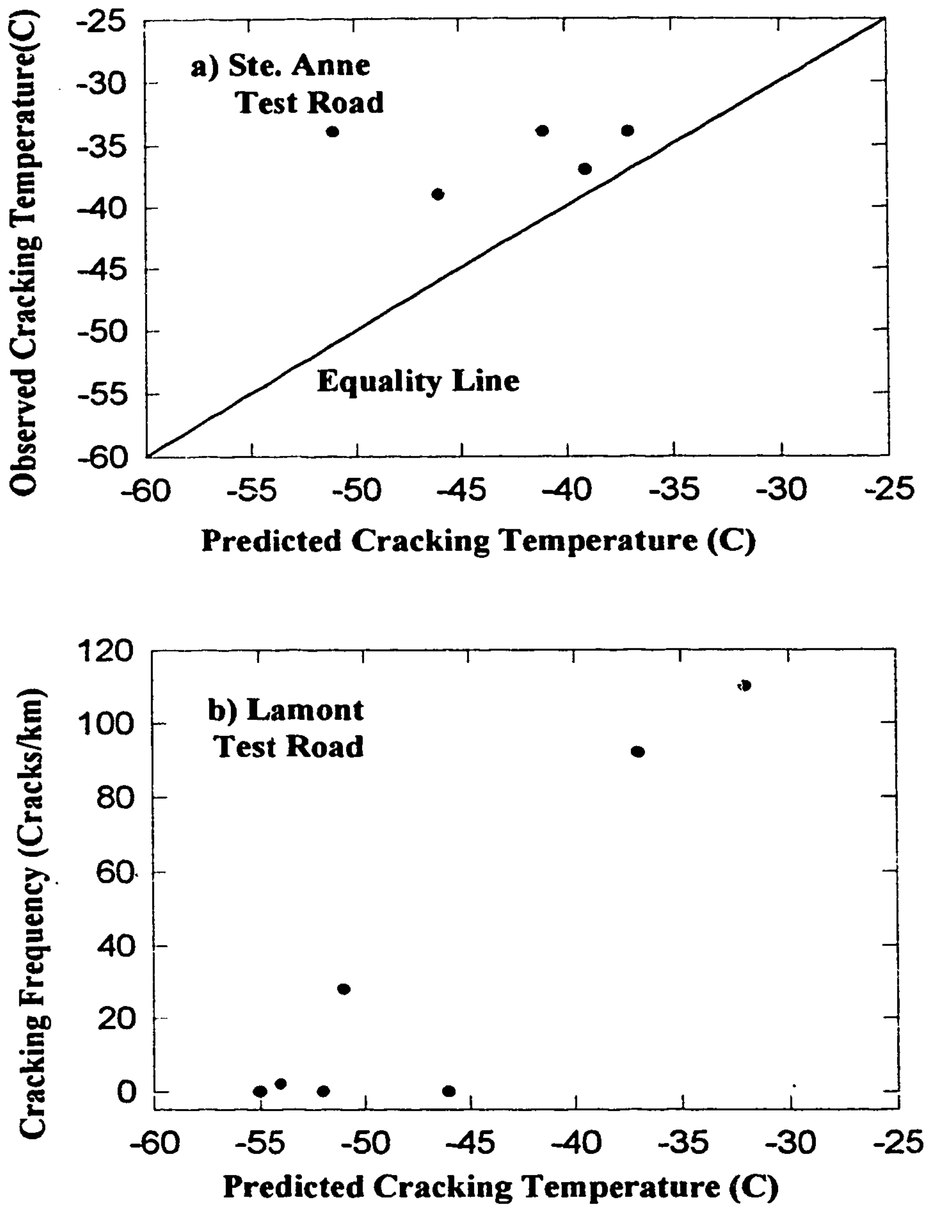


Fig.5.4-Predicted Cracking Temperature with the Critical Stiffness Method from Deme and Young Compared with the Observed Information

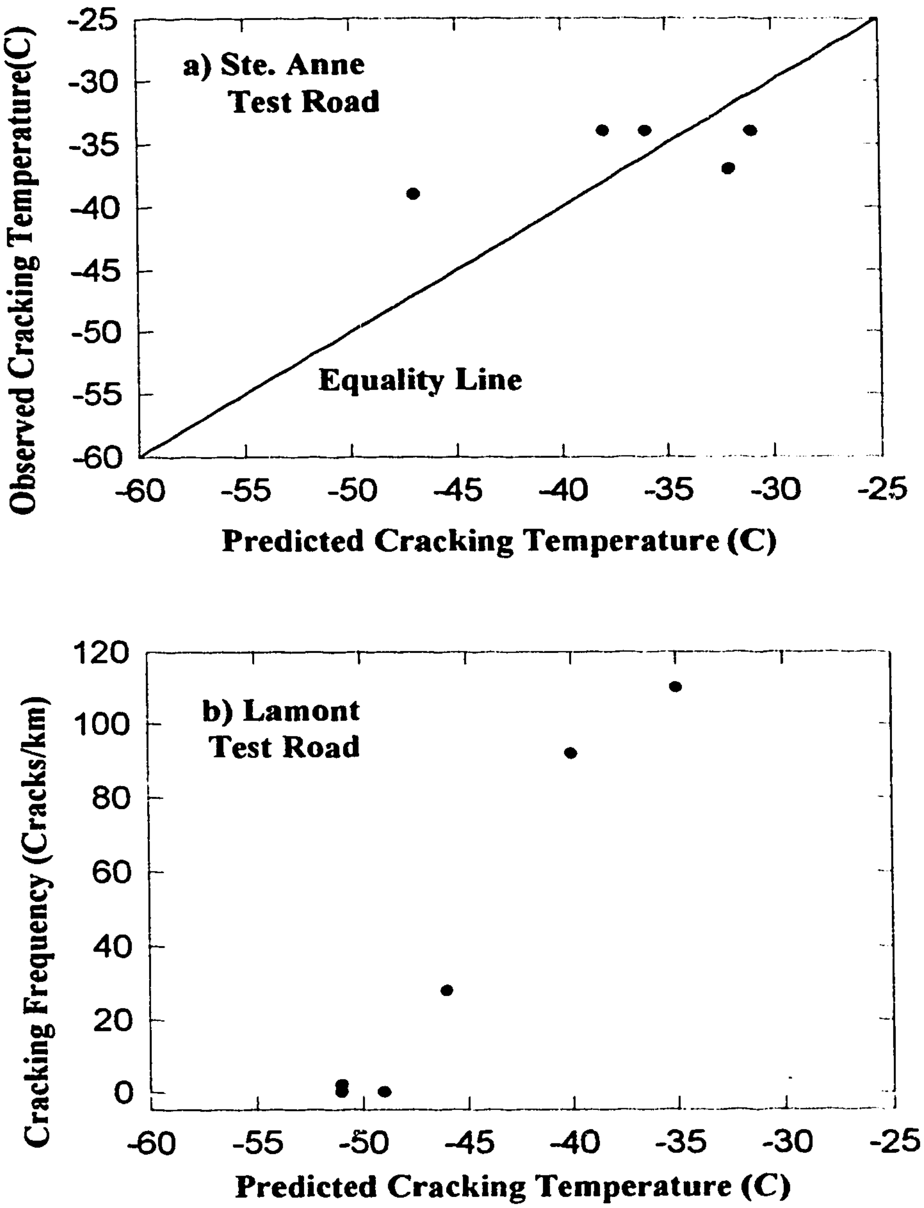


Fig.5.5-Predicted Cracking Temperature with the Method Used in CAMA Compared with the Observed Information

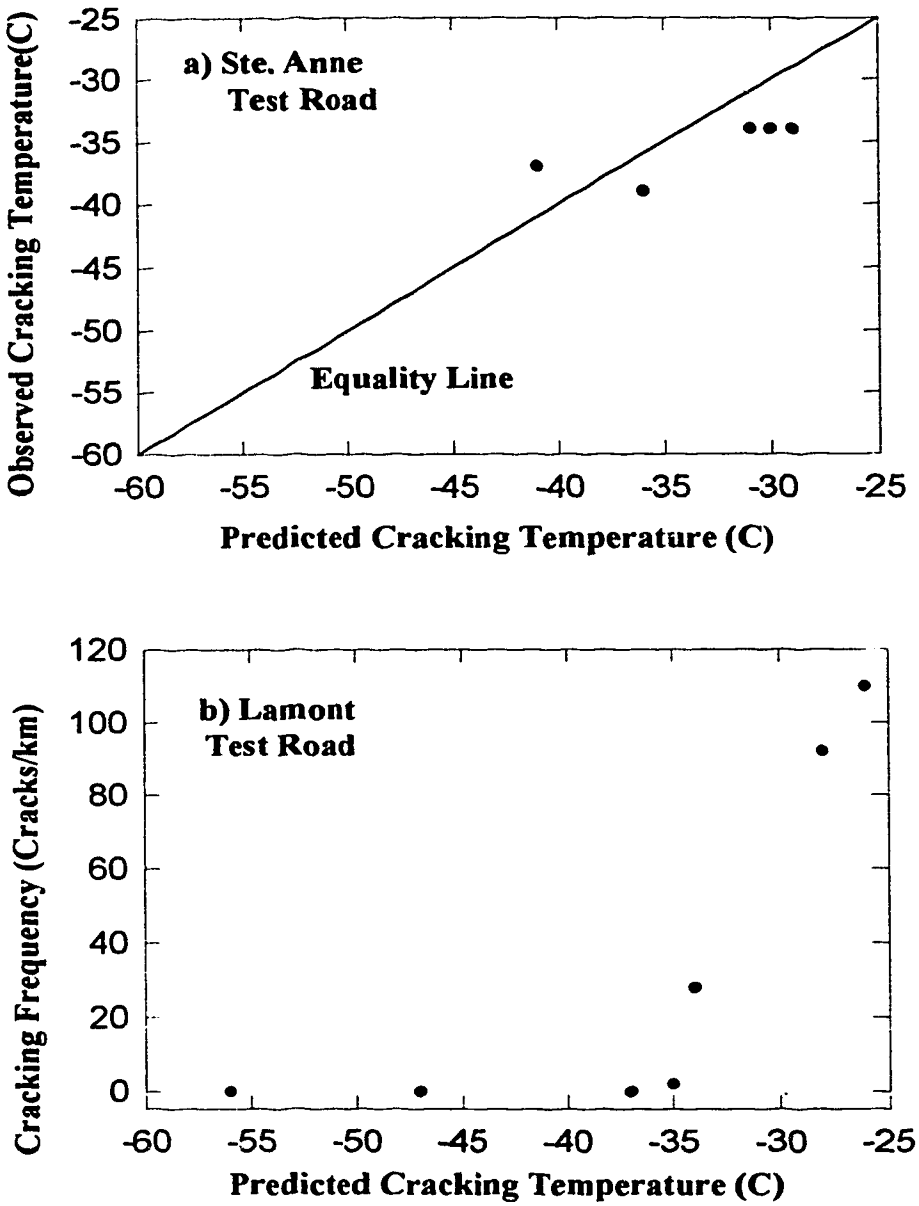


Fig.5.6-Predicted Cracking Temperature with Robertson's Method Compared with the Observed Information

The Improved Theoretical Method takes account of temperature, cooling rate, asphalt mix composition and contraction properties, asphalt and asphalt mix rheological characteristics (both temperature and time susceptibilities), and asphalt mix breaking properties. Fig.5.7a is from Table 5.11a, and Fig.5.7b is plotted from the results predicted by using the contraction coefficient $\alpha 1$ and the failure stress from direct tensile test conducted in Ste. Anne Test Road in Table 5.11b. It can be seen from Fig.5.7a that there is good agreement between the predicted and the observed cracking temperatures for Ste. Anne Test Road. From Fig.5.7b, the predicted cracking temperatures do correlate closely with the cracking frequencies. Because the calculated contraction coefficient $\alpha 2$ is a little larger than the tested coefficient $\alpha 1$ from Ste. Anne Test Road, the predicted cracking temperatures with $\alpha 2$ are also a little higher as shown in Table 5.11. Because the failure stress from indirect tensile test is a little larger than the failure stress from direct tensile test conducted in Ste. Anne Test Road, the predicted cracking temperatures with the failure stress from indirect tensile test are a little lower. The predicted cracking temperatures with different cooling rates for Lamont test road are provided in Appendix V. From the results with different cooling rates, it can be concluded that the cooling rate is a significant factor influencing cracking temperatures. The higher the cooling rate, the warmer the cracking temperatures; however, the effect becomes less significant as the cooling rate increases to a certain point.

Table 5.12 shows the deviations of the predicted from the observed cracking temperatures for Ste. Anne Test Road using the various prediction methods. From this table and previous analysis, it is shown that the Improved Theoretical Method can predict cracking temperatures of asphalt mixes more accurately than the other methods.

Table 5.12-Deviations of the Predicted from the Observe Cracking Teperatures for Ste. Anne Test Road with Various Methods

Prediction Method	D (°C)	$D = [(X_p - X_o)^2 / n]^{0.5}$ where
Critical Stiffness Method	8.5	D = Deviation (°C)
Method Used in CAMA	2.8	X_p and X_o = Predicted and observed
Robertson's Method	4.0	cracking temperatures (°C)
Improved Theoretical Method	1.5	n = observed cracking temp. number

Note: The Critical Stiffness method may have an unknown safety factor included.

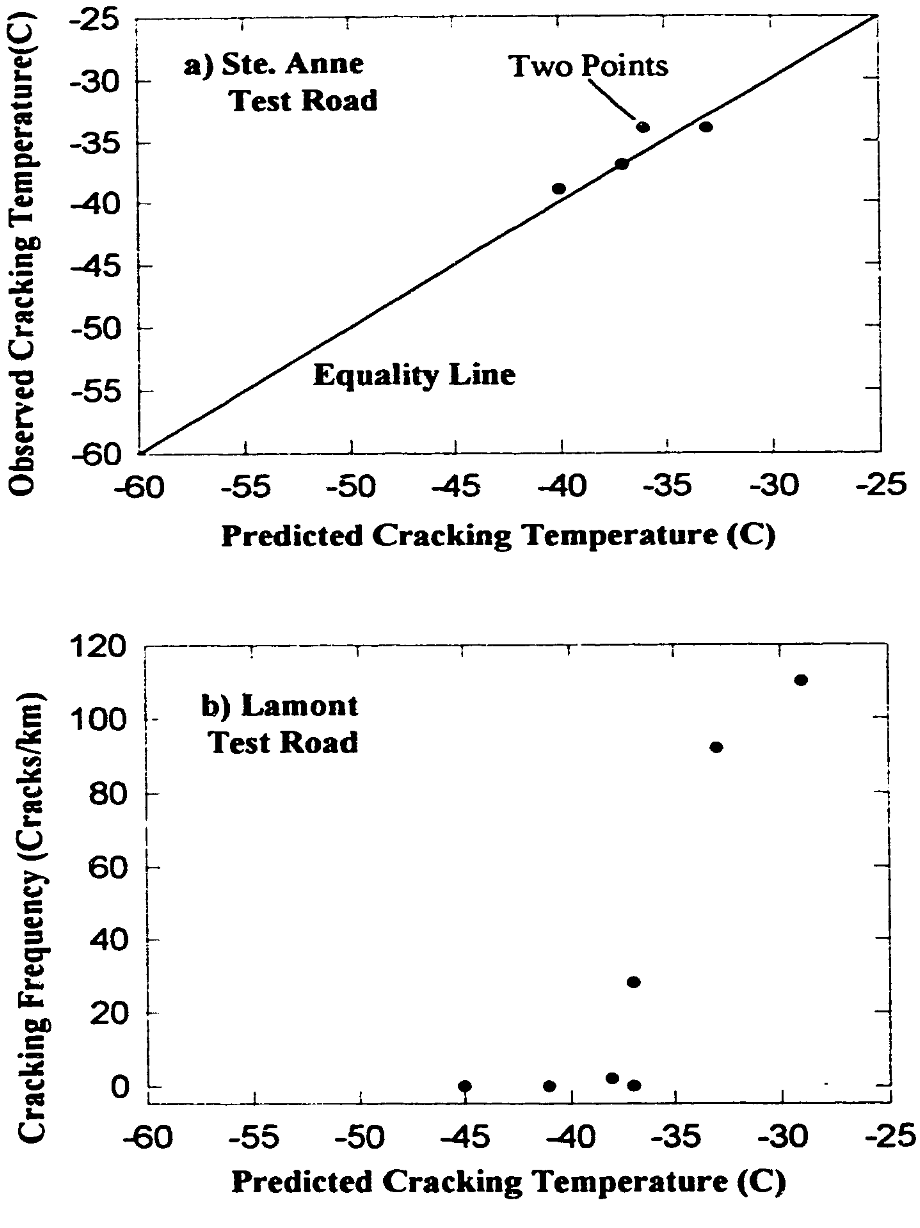


Fig.5.7-Predicted Cracking Temperature with the Improved Theoretical Method Compared with the Observed Information

To use the Improved Theoretical Method, the stiffness values of asphalts or asphalt mixes and the tensile strength of asphalt mixes from direct tensile test are required. If the direct tensile test is not available, other tests such as indirect tensile test or bending beam test can be used. In these cases, sound engineering analysis and judgment are necessary. In most circumstances, however, because of either budget or time constraints, there is no direct test available. Therefore, the nomographs have to be used for the acquisition of the material properties. The following nomographs are recommended in this case:

1) Original van der Poel (1954) nomograph for the stiffness of asphalt binders;
2) the nomograph developed by Bonnaure et al. (1977) for the stiffness of asphalt mixes;
3) the nomograph from Deme and Young (1987) for tensile strength.

5.6 Summary

Both time and temperature susceptibilities of asphalt are important for the low temperature behavior of asphalt pavements. The cooling rate is important. The Improved Theoretical Method developed in this chapter considered these factors using an analysis based on the thermal stress relaxation process. However, because this method fails to consider the structural parameters of asphalt pavement such as the thickness of asphalt layer of pavement, it cannot calculate the cracking frequencies.

The other methods discussed in the chapter consider one or more important factors influencing the low temperature cracking behavior of asphalt pavements, but some fail to take account of cooling rate and some do not consider time susceptibilities of asphalts or asphalt mixes while some others ignore the influence of the asphalt mix compositions on the stiffness and tensile strength. Therefore, these methods can provide qualitative estimates of cracking temperatures, but fail to predict the cracking temperatures accurately.

CHAPTER SIX

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

Based on the indirect tensile test results on the recycled tire rubber asphalt mixes, the addition of the recycled tire rubber produced some improvement in the expected low temperature performance of asphalt mixes. The primary conclusions are as follows:

1. Chauvenet's Criterion can be used as a standard outlier rejection method for the indirect tensile test (constant loading speed to failure test) to improve the precision of the data treatment.

2. The failure stress is slightly reduced with the addition of 1% WTP using a "dry procedure" or 10% WTP by a "wet procedure" at temperatures from -10°C to -30°C.

3. The failure strain is slightly improved with the addition of the recycled tire rubber and is more pronounced at the higher temperatures of 0°C to -10°C.

4. The failure stiffness of the compacted mixes with addition of the recycled tire rubber is slightly lower at the lower temperatures of -20°C and -30°C.

Based on the analysis on the relationship of data from the indirect tensile test and from the nomograph method, the following conclusions are found:

5. For asphalt mix stiffness, there are no significant differences between the indirect tensile test results and the data from nomograph method. However, the tested values are a little larger than the calculated mix stiffness from nomographs. There is a trend that the difference between the tested and the calculated mix stiffness tends to be smaller as the mix stiffness increases.

6. For asphalt mix tensile strength, when the mix stiffness is less than 10,000 MPa, there are no significant differences between indirect tensile test and the Heukelom's curve Type I. However, the tested curve from the indirect tensile test does not reach a peak at 10,000 MPa mix stiffness as do the curves from the other sources. Therefore, the tensile strength from the indirect tensile test should not be used directly for the calculation of cracking temperatures in asphalt pavements.

7. The tensile strength from the indirect tensile test is approximately equal to the tensile strength from bending beam tests and larger than the tensile strength from direct tensile tests.

8. The mix stiffness could possibly be used directly in the calculation of cracking temperatures in asphalt pavement. However, caution should be taken in using the indirect tensile strength because it is larger than the direct tensile strength. A modified curve for the mixes used in the Lamont test sections as shown in Fig.4.10 is recommended in this thesis for indirect tensile strength. This curve may be further modified in the event of more test data becoming available.

From the comparison of the cracking temperatures observed in the Ste. Anne Test Road and the cracking frequencies observed in the Lamont Test Road with the cracking temperatures calculated by using four different cracking temperature prediction methods, the following conclusions are obtained:

9. Both time and temperature susceptibilities of asphalt are important to cracking behavior of asphalt pavement. The cooling rate is not negligible. The Improved Theoretical Method considers these factors based on the thermal stress relaxation process analysis, so this method provides more accurate prediction of cracking temperatures of asphalt pavement than the others.

10. The other methods discussed in the thesis consider one or more factors influencing the cracking behavior of asphalts or asphalt mixes, but fail to take account of cooling rate and (or) time susceptibility of asphalt and (or) the composition of asphalt mixes. Therefore, all these methods can provide qualitative estimates of cracking temperatures, but fail to predict the cracking temperatures accurately.

6.2 Recommendations

1. Since the constant loading to failure indirect tensile test is not able to evaluate the stiffness of the asphalt mixes in a full range of loading time, the low temperature indirect tensile creep test is recommended in order to evaluate the rheological properties of asphalt mixes.

2. It is considered worthwhile to conduct the indirect tensile creep and strength tests, using the SHRP test method developed by Pennsylvania State University researchers and the University of Alberta method on the same materials at low temperatures in order to compare the two methods.

3. In Chapter Four, the discussion is based on the indirect tensile test values and the data predicted from nomographs. Because the data obtained by the nomographs are different from the data obtained by the indirect tensile tests, the indirect tensile test and direct tensile test should be conducted for the same materials in order to establish a reliable relationship between the two types of tests.

4. Although the Improved Theoretical Method is more accurate than other methods to predict cracking temperatures, it is not capable of calculating the cracking frequencies of asphalt pavements. More work should be done to try to develop a theoretical cracking frequency prediction method so that a practical low temperature design method for asphalt pavements can be achieved.

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Appendix I

GROUPING OF THE INDIRECT TENSILE TEST SPECIMENS OF THE TIRE RUBBER ASPHALT MIXES

A grouping program developed previously was used for grouping of the specimens. The weight of each specimen was determined by weighing each specimen in air, immersed in water, and then saturated surface dry weight, as discussed in ASTM D2726. These values were used as input in the program which then calculates the bulk specific gravity of each specimen. Sorting is then done into possible groups according to their bulk specific gravities so that the difference in mean specific gravity is minimized. Following is the outputs of the grouping program for the specimens of the tire rubber asphalt mixes.

Table I.1a-Specific Gravity Calculation for the Specimens

IDENTIFICATION OF THE PROJECT. RUBBER-ASPHALT-CONTROL
NUMBER OF SAMPLES = 20
THE MEAN SPECIFIC GRAVITY OF THE SAMPLES = 2.341
THE STANDARD DEVIATION = 0.009
THE COEFFICIENT OF VARIATION (%) = 0.378

Sample Name	Dry Weight g.	SSD. Weight g.	IM. Weight g.	Volume cc.	Specific Gravity.	No. of St. Dev. from Mean
C1	1197.80	1198.40	687.50	510.90	2.34	-0.45
C2	1193.10	1194.10	680.20	513.90	2.32	2.13
C3	1196.00	1196.60	684.30	512.30	2.34	0.67
C4	1196.00	1196.90	686.20	510.70	2.34	-0.16
C5	1192.50	1194.50	683.50	511.00	2.33	0.78
C6	1192.10	1192.50	683.50	509.00	2.34	-0.17
C7	1195.80	1196.10	685.30	510.80	2.34	-0.06
C8	1195.80	1196.10	686.90	509.20	2.35	-0.89
C9	1191.40	1192.20	684.00	508.20	2.34	-0.43
C10	1193.10	1193.80	689.60	504.20	2.37	-2.92
C11	1193.50	1194.20	683.00	511.20	2.34	0.66
C12	1192.20	1192.80	683.30	509.50	2.34	0.06
C13	1193.70	1194.50	685.60	508.90	2.35	-0.58
C14	1200.70	1201.60	688.50	513.10	2.34	0.05
C15	1198.50	1199.10	686.10	513.00	2.34	0.48
C16	1198.40	1199.60	688.10	511.50	2.34	-0.27
C17	1197.30	1198.60	685.40	513.20	2.33	0.85
C18	1198.40	1199.70	688.50	511.20	2.34	-0.43
C19	1197.20	1197.90	684.10	513.80	2.33	1.18
C20	1196.60	1196.90	686.60	510.30	2.35	-0.50

Table I.1b-Sorting into Groups Based on the Specific Gravities of the Specimens

IDENTIFICATION OF THE PROJECT, RUBBER-ASPHALT-CONTROL
NUMBER OF SAMPLES = 20
THE MEAN SPECIFIC GRAVITY OF THE SAMPLES = 2.341
THE STANDARD DEVIATION = 0.009
THE COEFFICIENT OF VARIATION (%) = 0.378
THESE ARE THE POSSIBLE GROUPS

Sample Name	Specific Gravity	Mean Specific Gravity	Group St. Dev.	Coeff. of Variation (%)
C10	2.366			
C2	2.322			
C9	2.344			
C11	2.335			
C15	2.336			
		2.341	0.0165	0.705
C8	2.348			
C19	2.33			
C18	2.344			
C12	2.34			
C14	2.34			
		2.341	0.0068	0.291
C13	2.346			
C17	2.333			
C16	2.343			
C7	2.341			
C4	2.342			
		2.341	0.0047	0.202
C1	2.344			
C3	2.335			
C5	2.334			
C6	2.342			
C20	2.345			
		2.34	0.0054	0.232

Table I.2a-Specific Gravity Calculation for the Specimens

IDENTIFICATION OF THE PROJECT, RUBBER-ASPHALT-1% WTP
NUMBER OF SAMPLES = 20
THE MEAN SPECIFIC GRAVITY OF THE SAMPLES = 2.321
THE STANDARD DEVIATION = 0.016
THE COEFFICIENT OF VARIATION (%) = 0.683

Sample Name	Dry Weight g.	SSD. Weight g.	IM. Weight g.	Volume cc.	Specific Gravity.	No. of St. Dev. from Mean
R1	1195.00	1196.30	677.40	518.90	2.30	1.11
R2	1192.30	1194.00	675.30	518.70	2.30	1.38
R3	1200.60	1202.00	677.80	524.20	2.29	1.91
R4	1197.20	1198.80	678.10	520.70	2.30	1.35
R5	1195.30	1196.40	681.60	514.80	2.32	-0.08
R6	1198.30	1199.10	681.30	517.80	2.31	0.40
R7	1194.80	1196.80	682.40	514.40	2.32	-0.14
R8	1197.30	1199.00	682.00	517.00	2.32	0.30
R9	1193.70	1194.30	679.20	515.10	2.32	0.20
R10	1191.60	1192.60	676.90	515.70	2.31	0.62
R11	1195.40	1197.00	680.90	516.10	2.32	0.27
R12	1195.30	1196.30	682.70	513.60	2.33	-0.43
R13	1192.20	1193.30	679.60	513.70	2.32	-0.02
R14	1194.60	1195.40	685.20	510.20	2.34	-1.32
R15	1193.30	1193.90	681.70	512.20	2.33	-0.58
R16	1190.60	1191.30	678.00	513.30	2.32	0.07
R17	1194.10	1195.30	687.00	508.30	2.35	-1.81
R18	1194.20	1194.80	681.80	513.00	2.33	-0.46
R19	1192.50	1193.10	684.60	508.50	2.35	-1.55
R20	1192.80	1193.10	683.30	509.80	2.34	-1.21

Table I.2b-Sorting into Groups Based on the Specific Gravities of the Specimens

IDENTIFICATION OF THE PROJECT, RUBBER-ASPHALT-1% WTP
NUMBER OF SAMPLES = 20
THE MEAN SPECIFIC GRAVITY OF THE SAMPLES = 2.321
THE STANDARD DEVIATION = 0.016
THE COEFFICIENT OF VARIATION (%) = 0.683
THESE ARE THE POSSIBLE GROUPS

Sample Name	Specific Gravity	Mean Specific Gravity	Group St. Dev.	Coeff. of Variation (%)
R17	2.349			
R3	2.29			
R18	2.328			
R6	2.314			
R13	2.321			
		2.32	0.0214	0.921
R19	2.345			
R2	2.299			
R12	2.327			
R8	2.316			
R11	2.316			
		2.321	0.0171	0.737
R14	2.341			
R4	2.299			
R7	2.323			
R9	2.317			
R5	2.322			
		2.321	0.0151	0.649
R1	2.303			
R10	2.311			
R15	2.33			
R16	2.32			
R20	2.34			
		2.321	0.0147	0.633

Table I.3a-Specific Gravity Calculation for the Specimens

IDENTIFICATION OF THE PROJECT. RUBBER-ASPHALT-10% WTP
NUMBER OF SAMPLES = 20
THE MEAN SPECIFIC GRAVITY OF THE SAMPLES = 2.288
THE STANDARD DEVIATION = 0.020
THE COEFFICIENT OF VARIATION (%) = 0.852

Sample Name	Dry Weight g.	SSD. Weight g.	IM. Weight g.	Volume cc.	Specific Gravity	No. of St. Dev. from Mean
R1	1192.8	1193.5	670.3	523.2	2.28	0.44
R2	1190.1	1191.7	661.1	530.6	2.243	2.33
R3	1193.1	1194	670.8	523.2	2.28	0.41
R4	1197.8	1200.9	679.8	521.1	2.299	-0.53
R5	1194.2	1194.9	677.7	517.2	2.309	-1.06
R6	1192.2	1193	677.4	515.6	2.312	-1.23
R7	1200.2	1200.9	681.8	519.1	2.312	-1.22
R8	1194.2	1195.8	679.6	516.2	2.313	-1.29
R9	1190.5	1191.9	668.4	523.5	2.274	0.73
R10	1186.9	1188.6	670.8	517.8	2.292	-0.2
R11	1190	1191.6	674.3	517.3	2.3	-0.62
R12	1196	1198.4	680.3	518.1	2.308	-1.03
R13	1187.8	1190.8	666.6	524.2	2.266	1.15
R14	1189.2	1190.9	666.1	524.8	2.266	1.15
R15	1184.7	1186.4	667.7	518.7	2.284	0.22
R16	1191.6	1193.3	676.7	516.6	2.307	-0.94
R17	1201.1	1202.6	677.3	525.3	2.287	0.09
R18	1181.7	1183.7	661.8	521.9	2.264	1.24
R19	1192.7	1194.2	672	522.2	2.284	0.22
R20	1193	1195.5	673.6	521.9	2.286	0.13

Table I.3b-Sorting into Groups Based on the Specific Gravities of the Specimens

IDENTIFICATION OF THE PROJECT, RUBBER-ASPHALT-10% WTP
NUMBER OF SAMPLES = 20
THE MEAN SPECIFIC GRAVITY OF THE SAMPLES = 2.288
THE STANDARD DEVIATION = 0.020
THE COEFFICIENT OF VARIATION (%) = 0.852
THESE ARE THE POSSIBLE GROUPS

Sample Name	Specific Gravity	Mean Specific Gravity	Group St. Dev.	Coeff. of Variation (%)
R8	2.313			
R2	2.243			
R16	2.307			
R1	2.28			
R4	2.299			
		2.288	0.0283	1.237
R6	2.312			
R18	2.264			
R11	2.3			
R3	2.28			
R19	2.284			
		2.288	0.0186	0.812
R7	2.312			
R13	2.266			
R10	2.292			
R15	2.284			
R17	2.287			
		2.288	0.0166	0.726
R5	2.309			
R9	2.274			
R12	2.308			
R14	2.266			
R20	2.286			
		2.289	0.0196	0.856

Appendix II

INDIRECT TENSILE TEST RESULTS OF THE RECYCLED TIRE RUBBER ASPHALT MIXES

Table II.1-Indirect Tensile Test Data before Outlier Rejection for Rubber Asphalt Mix (1% WTP, "Dry" Procedure)

Temperature (°C)	Sample No.	Failure Stress (KPa)	Failure Strain (0.0001)	Failure Stiffness (MPa)	Sample Density (kg/m³)	90% Probability Confidence Interval for Failure Stress
0	A2	215	96	41.2	2299	
	A8	382	80	87.7	2316	
	A11	431	184	42.7	2316	
	A12	447	148	55.3	2327	(1.64*Std. Dev.)
	A19	496	95	95.5	2345	177
No. of Specimens		5	5	5	5	Lower Limit
Mean		394	120	64.5	2321	217
Std. Dev.		108	44	25.5	17	Upper Limit
Coef. Var.(%)		27.4	36.6	39.6	0.7	571
-10	A3	909	65	255	2290	
	A6	1194	97	226	2314	
	A13	1160	108	196	2321	
	A17	1146	128	163	2349	(1.64*Std. Dev.)
	A18	1137	121	171	2328	187
No. of Specimens		5	5	5	5	Lower Limit
Mean		1109	104	202	2320	922
Std. Dev.		114	25	38.2	21	Upper Limit
Coef. Var.(%)		10.3	23.9	18.9	0.9	1296
-20	A1	2423	28	1610	2303	
	A10	2576	29	1631	2311	
	A15	2753	36	1413	2330	
	A16	2707	27	1845	2320	(1.64*Std. Dev.)
	A20	2482	36	1266	2340	232
No. of Specimens		5	5	5	5	Lower Limit
Mean		2588	31	1553	2321	2356
Std. Dev.		142	4	222	15	Upper Limit
Coef. Var.(%)		5.5	14.3	14.3	0.6	2820
-30	A4	2680	38	1305	2299	
	A5	2375	19	2263	2322	
	A7	2732	7	7056	2323	
	A9	2747	29	1732	2317	(1.64*Std. Dev.)
	A14	2910	10	5483	2341	321
No. of Specimens		5	5	5	5	Lower Limit
Mean		2689	20	3568	2320	2368
Std. Dev.		196	13	2551	15	Upper Limit
Coef. Var.(%)		7.3	62.7	71.5	0.6	3010

Table II.2-Indirect Tensile Test Data before Outlier Rejection for Rubber Asphalt Mix (10% WTP, "Wet" Procedure)

Temperature (°C)	Sample No.	Failure Stress (KPa)	Failure Strain (0.0001)	Failure Stiffness (MPa)	Sample Density (kg/m^3)	90% Probability Confidence Interval for Failure Stress
0	B3	411	160	46.8	2280	
	B6	575	87	120.6	2312	
	B11	491	159	56.3	2300	
	B18	382	279	25.0	2264	(1.64*Std. Dev.)
	B19	523	183	52.1	2284	131
No. of Specimens		5	5	5	5	Lower Limit
Mean		476	174	60.2	2288	346
Std. Dev.		80	69	35.9	19	Upper Limit
Coef. Var.(%)		16.7	39.8	59.6	0.8	607
-10	B1	1022	77	242	2280	
	B2	847	146	106	2243	
	B4	917	94	178	2299	
	B8	1483	131	207	2313	(1.64*Std. Dev.)
	B16	1063	93	208	2307	407
No. of Specimens		5	5	5	5	Lower Limit
Mean		1066	108	188	2288	660
Std. Dev.		248	29	51.3	28	Upper Limit
Coef. Var.(%)		23.3	26.7	27.2	1.2	1473
-20	B5	3064	32	1740	2309	
	B9	2870	35	1492	2274	
	B12	2250	29	1418	2308	
	B14	2428	26	1736	2266	(1.64*Std. Dev.)
	B20	2091	30	1254	2286	677
No. of Specimens		5	5	5	5	Lower Limit
Mean		2540	30	1528	2289	1863
Std. Dev.		413	4	210	20	Upper Limit
Coef. Var.(%)		16.3	11.8	13.7	0.9	3218
-30	B7	2929	07	7329	2312	
	B10	2460	15	3030	2292	
	B13	2699	17	2843	2266	
	B15	2155	14	2852	2284	(1.64*Std. Dev.)
	B17	2569	4	11700	2287	471
No. of Specimens		5	5	5	5	Lower Limit
Mean		2562	11	5551	2288	2092
Std. Dev.		287	6	3935	17	Upper Limit
Coef. Var.(%)		11.2	54.5	70.9	0.7	3033

Table II.3-Indirect Tensile Test Data before Outlier Rejection for Control Samples (No Rubber Added)

Temperature (°C)	Sample No.	Failure Stress (KPa)	Failure Strain (0.0001)	Failure Stiffness (MPa)	Sample Density (kg/m^3)	90% Probability Confidence Interval for Failure Stress
0	C8	392	111	64.4	2348	
	C12	533	113	86.0	2340	
	C14	594	105	103.5	2340	
	C18	529	150	64.4	2344	(1.64*Std. Dev.)
	C19	487	100	88.7	2330	122
No. of Specimens		5	5	5	5	Lower Limit
Mean		507	116	81.4	2340	384
Std. Dev.		75	20	16.9	7	Upper Limit
Coef. Var.(%)		14.7	17.0	20.7	0.3	629
-10	C2	1303	86	276	2322	
	C9	1462	103	260	2344	
	C10	1482	72	375	2366	
	C11	1274	116	201	2335	(1.64*Std. Dev.)
	C15	1690	61	504	2336	274
No. of Specimens		5	5	5	5	Lower Limit
Mean		1442	88	323	2341	1169
Std. Dev.		167	22	119.1	16	Upper Limit
Coef. Var.(%)		11.6	25.3	36.9	0.7	1716
-20	C1	2899	8	6538	2344	
	C3	2820	37	1385	2335	
	C5	2837	14	3786	2334	
	C6	3329	13	4556	2342	(1.64*Std. Dev.)
	C20	2941	11	4709	2345	343
No. of Specimens		5	5	5	5	Lower Limit
Mean		2965	17	4195	2340	2622
Std. Dev.		209	12	1867	5	Upper Limit
Coef. Var.(%)		7.1	69.4	44.5	0.2	3308
-30	C4	3129	7	8791	2342	
	C7	2631	16	2967	2341	
	C13	3137	20	2904	2346	
	C16	3409	5	12407	2343	(1.64*Std. Dev.)
	C17	3126	5	11377	2333	462
No. of Specimens		5	5	5	5	Lower Limit
Mean		3086	10	7689	2341	2624
Std. Dev.		282	7	4535	5	Upper Limit
Coef. Var.(%)		9.1	66.4	59.0	0.2	3548

Table II.4-Indirect Tensile Test Results after Outlier Rejection for Rubber Asphalt Mix (1% WTP, "Dry" Procedure)

Temperature (°C)	Sample No.	Failure Stress (KPa)	Failure Strain (0.0001)	Failure Stiffness (MPa)	Sample Density (kg/m³)
0	A8	382	80	87.7	2316
	A11	431	184	42.7	2316
	A12	447	148	55.3	2327
	A19	496	95	95.5	2345
No. of Specimens		4	4	4	4
Mean		439	126	70	2326
Std. Dev.		47	48	25	14
Coef. Var.(%)		10.7	38.1	36.0	0.6
-10	A6	1194	97	226	2314
	A13	1160	108	196	2321
	A17	1146	128	163	2349
	A18	1137	121	171	2328
No. of Specimens		4	4	4	4
Mean		1159	113	189	2328
Std. Dev.		25	14	28	15
Coef. Var.(%)		2.2	12.4	14.9	0.6
-20	A1	2423	28	1610	2303
	A10	2576	29	1631	2311
	A15	2753	36	1413	2330
	A16	2707	27	1845	2320
	A20	2482	36	1266	2340
No. of Specimens		5	5	5	5
Mean		2588	31	1553	2321
Std. Dev.		142	4	222	15
Coef. Var.(%)		5.5	14.3	14.3	0.6
-30	A4	2680	38	1305	2299
	A5	2375	19	2263	2322
	A7	2732	7	7056	2323
	A9	2747	29	1732	2317
	A14	2910	10	5483	2341
No. of Specimens		5	5	5	5
Mean		2689	20	3568	2320
Std. Dev.		196	13	2551	15
Coef. Var.(%)		7.3	62.7	71.5	0.6

Table II.5-Indirect Tensile Test Results after Outlier Rejection for Rubber Asphalt Mix (10% WTP, "Wet" Procedure)

Temperature (°C)	Sample No.	Failure Stress (KPa)	Failure Strain (0.0001)	Failure Stiffness (MPa)	Sample Density (kg/m³)
0	B3	411	160	46.8	2280
	B6	575	87	120.6	2312
	B11	491	159	56.3	2300
	B18	382	279	25.0	2264
	B19	523	183	52.1	2284
No. of Specimens		5	5	5	5
Mean		476	174	60.2	2288
Std. Dev.		80	69	35.9	19
Coef. Var.(%)		16.7	39.8	59.6	0.8
-10	B1	1022	77	242	2280
	B2	847	146	106	2243
	B4	917	94	178	2299
	B16	1063	93	208	2307
No. of Specimens		4	4	4	4
Mean		962	102	184	2282
Std. Dev.		99	30	58	29
Coef. Var.(%)		10.2	29.2	31.6	1.2
-20	B5	3064	32	1740	2309
	B9	2870	35	1492	2274
	B12	2250	29	1418	2308
	B14	2428	26	1736	2266
	B20	2091	30	1254	2286
No. of Specimens		5	5	5	5
Mean		2540	30	1528	2289
Std. Dev.		413	4	210	20
Coef. Var.(%)		16.3	11.8	13.7	0.9
-30	B7	2929	7	7329	2312
	B10	2460	15	3030	2292
	B13	2699	17	2843	2266
	B15	2155	14	2852	2284
	B17	2569	4	11700	2287
No. of Specimens		5	5	5	5
Mean		2562	11	5551	2288
Std. Dev.		287	6	3935	17
Coef. Var.(%)		11.2	48.6	70.9	0.7

Table II.6-Indirect Tensile Test Results after Outlier Rejection for Control Samples (No Rubber Added)

Temperature (°C)	Sample No.	Failure Stress (KPa)	Failure Strain (0.0001)	Failure Stiffness (MPa)	Sample Density (kg/m^3)
0	C8	392	111	64.4	2348
	C12	533	113	86.0	2340
	C14	594	105	103.5	2340
	C18	529	150	64.4	2344
	C19	487	100	88.7	2330
No. of Specimens		5	5	5	5
Mean		507	116	81.4	2340
Std. Dev.		75	20	16.9	7
Coef. Var.(%)		14.7	17.0	20.7	0.3
-10	C2	1303	86	276	2322
	C9	1462	103	260	2344
	C10	1482	72	375	2366
	C11	1274	116	201	2335
	C15	1690	61	504	2336
No. of Specimens		5	5	5	5
Mean		1442	88	323	2341
Std. Dev.		167	22	119.1	16
Coef. Var.(%)		11.6	25.3	36.9	0.7
-20	C1	2899	8	6538	2344
	C3	2820	37	1385	2335
	C5	2837	14	3786	2334
	20	2941	11	4709	2345
No. of Specimens		4	4	4	4
Mean		2874	18	4105	2340
Std. Dev.		56	10	2144	6
Coef. Var.(%)		1.9	75.2	52.2	0.2
-30	C4	3129	7	8791	2342
	C7	2631	16	2967	2341
	C13	3137	20	2904	2346
	C16	3409	5	12407	2343
	C17	3126	5	11377	2333
No. of Specimens		5	5	5	5
Mean		3086	10	7689	2341
Std. Dev.		282	7	4535	5
Coef. Var.(%)		9.1	66.4	59.0	0.2

Appendix III

BITUMEN TEST DATA CHART COMPUTERIZED WITH SIGMAPLOT™

Heukelom (1969) developed a Bitumen Test Data Chart (BTDC) for plotting the results of standard laboratory tests on asphalt against temperature. Four major purposes were identified:

i. distinguishing between different types of bitumen and obtaining a more complete understanding of their behavior,
ii. estimating whether certain performance requirements would be met,
iii. checking the mutual consistency of test data so that erroneous results may be discarded, and
iv. appraising the data used for entering van der Poel's stiffness nomograph.

The BTDC has a linear temperature scale, a logarithm penetration scale, and a viscosity scale which can be calculated by following equation:

$$C = 1310 \frac{\log \eta}{4.35 + \log \eta} \tag{III.1}$$

where

C = viscosity scale value,
η = viscosity in poise.

For a scale length of 1000 arbitrary units in the BTDC, the zero units corresponds to a viscosity of 1 poise, and 1000 units corresponds to a penetration of 1 dmm.

In the later work Heukelom (1973) modified the previous BTDC by adding a penetration index scale in the top of the chart. In this way, the Penetration Index (PI) based on the slope of the straight line in the penetration branch could be obtained graphically as shown in Fig.III.1, but the accuracy is very low.

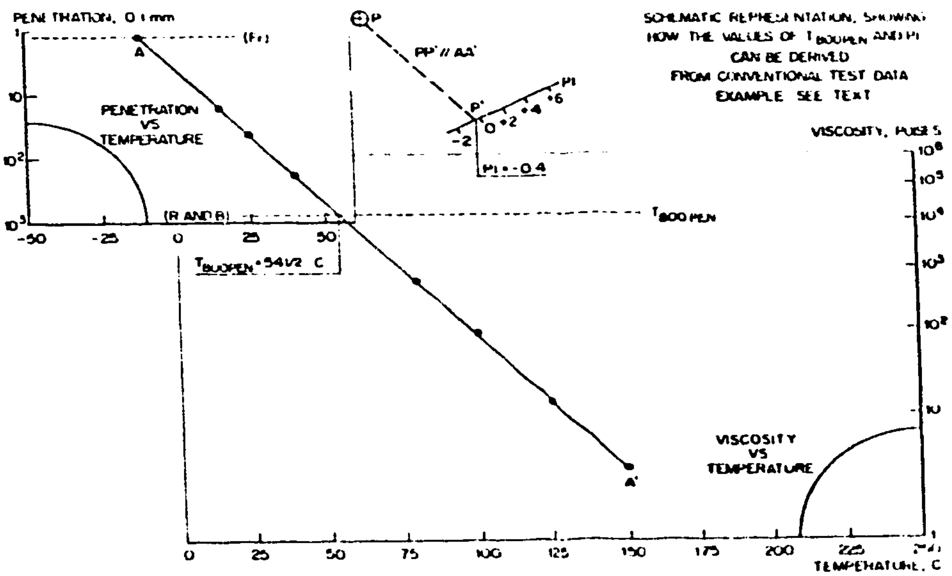

Fig.III.1-A Modified Version of the BTDC by Heukelom (1973)

A computerized version of the BTDC using commercially available Scientific Graph System SigmaPlot™ (Jandel Scientific 1990) has been developed. This plotting system enables the BTDC to be produced with high accuracy and quality. As an example, Fig.III.2, III.3, and III.4 have been plotted with the data presented by Heukelom (1969) for typical bitumens of Class S, B, and W. An updated discussion concerning the BTDC is given in the <u>Shell Bitumen Handbook</u> (1990)

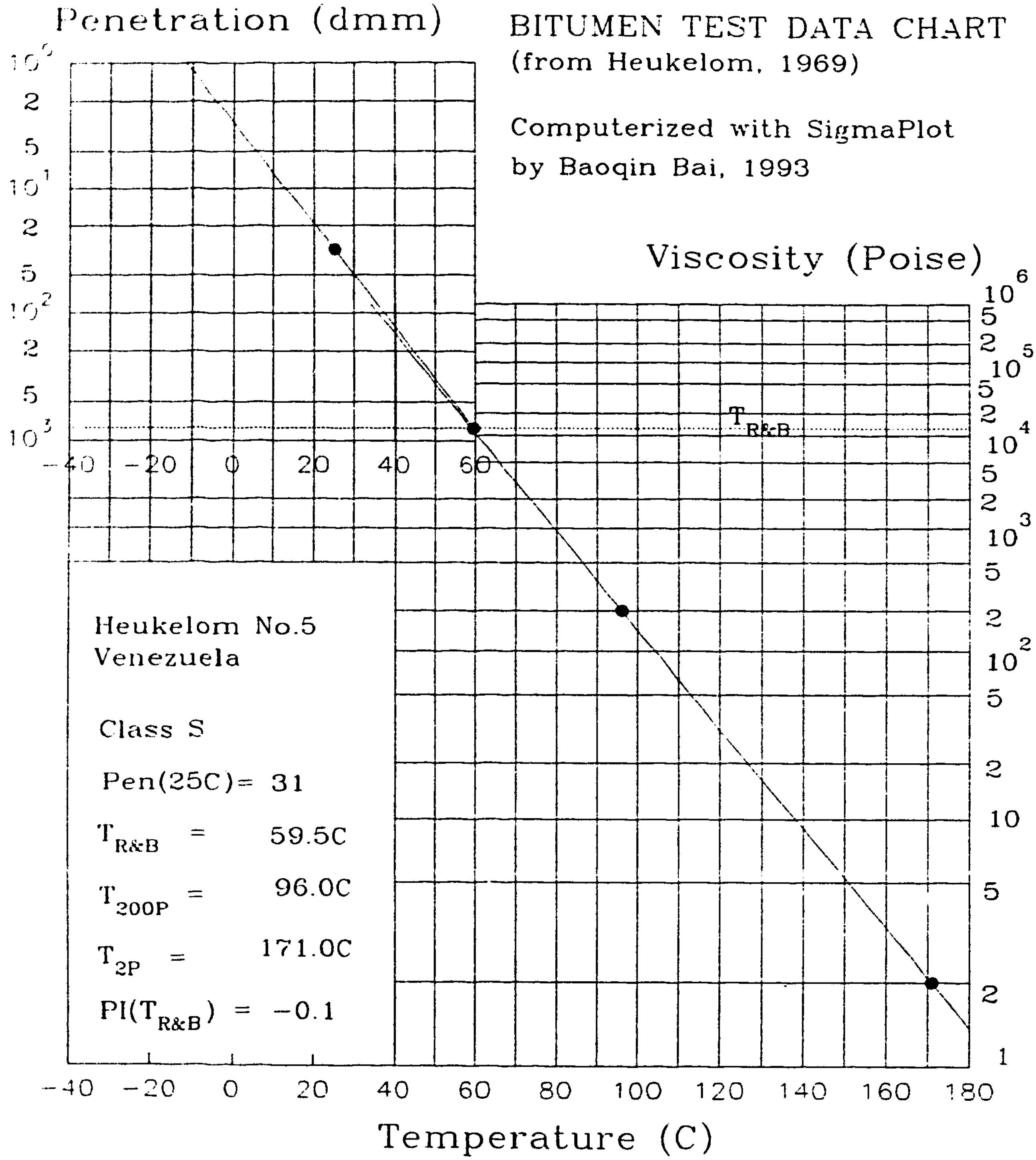

Fig.III.2-Asphalt No.5 (Class S) Used by Heukelom (1969)

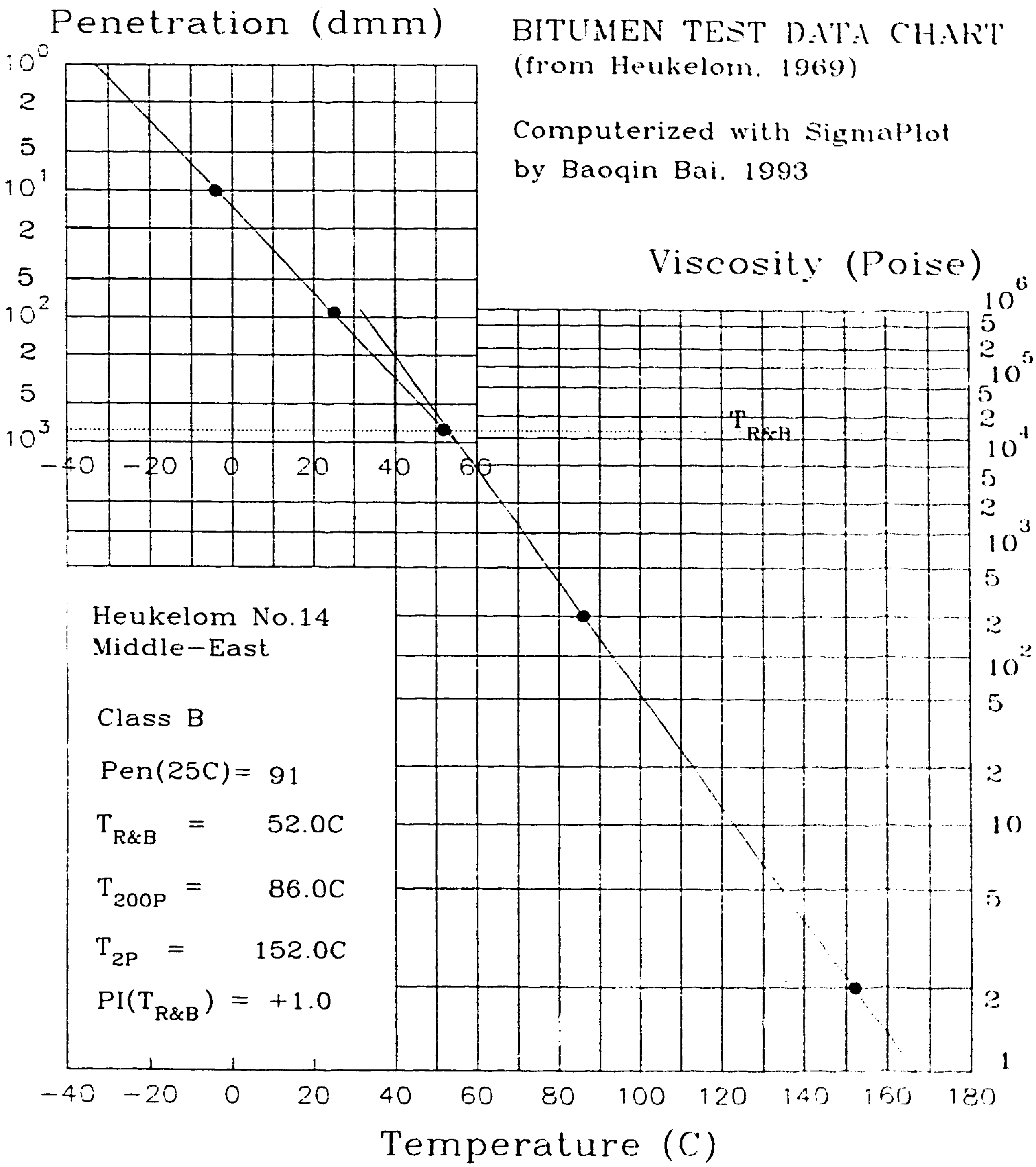

Fig.III.3-Asphalt No.14 (Class B) Used by Heukelom (1969)

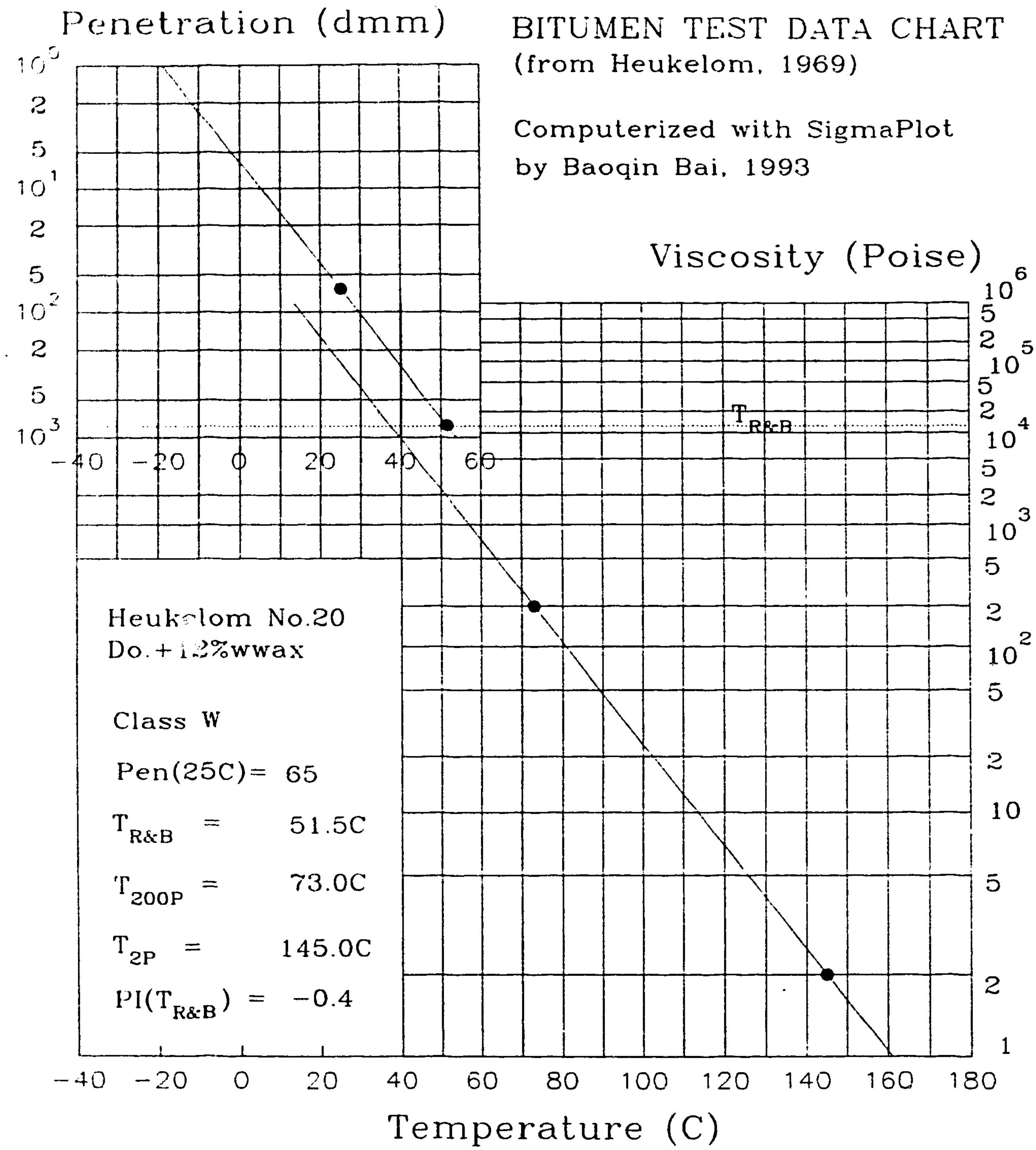

Fig.III.4-Asphalt No.20 (Class W) Used by Heukelom (1969)

How to Use the Computerized Version of BTDC

1) Data Preparation

i) penetration values (dmm) with the corresponding temperatures (°C) at which the penetration tests are conducted.

ii) viscosity values (poise) with the corresponding temperatures (°C) at which the viscosity tests are conducted.

Usually the kinematic viscosity value at 135°C is given in "cSt". The viscosity value in "poise" can be obtained from the viscosity value in "cSt" by following formula:

$$Vis(poise) = Vis(cSt) * D / 100 \qquad (III.2)$$

where

Vis(poise) = viscosity in poise,
Vis(cSt) = viscosity in cSt (mm^2/s),
D = density of the asphalt in the test temperature.

"D" can be assumed as 0.948 g/cm^3 if the tested data is not available. For detail, please refer to ASTM D2170, A1.3.

iii) the viscosity scale "C" values calculated with equation (III.1).

2) Data Input

For the details of the SigmaPlot™ software, please check SigmaPlot™ user's manual. Following description assumes that the reader has a basic knowledge of using the software.

i) Run SigmaPlot™ and then open file "BTDC.SPG" given as an example. The new data may be inputted by replacing the example data.

ii) View the "DataSheet" and then Find the columns with the labels of "TEMP.(C)-PEN." and "PEN.-(DMM)" (Columns 1 to 2).

iii) Input the temperatures in the column labeled with "TEMP.(C)-PEN." and the penetration values in the column labeled with "PEN.-DMM" correspondingly.

iv) Find the columns with the labels of "TEMP.(C)-VIS." and "VIS. (P)-C VALUE" (Columns 3 to 4)

v) Input the temperatures in the column labeled with "TEMP.(C)-VIS." and the viscosity scale "C" values in the column labeled with "VIS. (P)-C VALUE" correspondingly.

vi) Type any labels for identification following the data.

After the data input, click the "Graph on Page", and then you will see the chart.

2) Text Contents

The text in the chart can be designed in any way based on the user's interests. Following is a suggestion for the contents of the text:

i) the asphalt identification,
ii) the input data,
iii) ring and ball softening point ($T_{R\&B}$),
iv) temperature corresponding to penetration of 800 dmm (T_{800}),
v) penetration index, PI, and
vi) the title of the chart.

The T_{800} and PI can be calculated with following equations:

$$T_{800} = (-b + \log 800) / a$$

$$PI = 30 / (1 + 50\,a) - 10$$

where

a, b = regression constants from the line of

$$\log P = a\,T + b$$

where

P = penetration (dmm),

T = temperature (°C) at which the penetration test is conducted,

A regression program using QuickBasic has been developed for the regression of the logarithm penetration vs. temperature.

3) Save File and Obtain Hardcopy

After the text edition, use "Save As" to save the file in a different name with extension name "SPG" (e.g. BTDC1.SPG). Then click "File" and "Hardcopy". Finally the BTDC required is obtained.

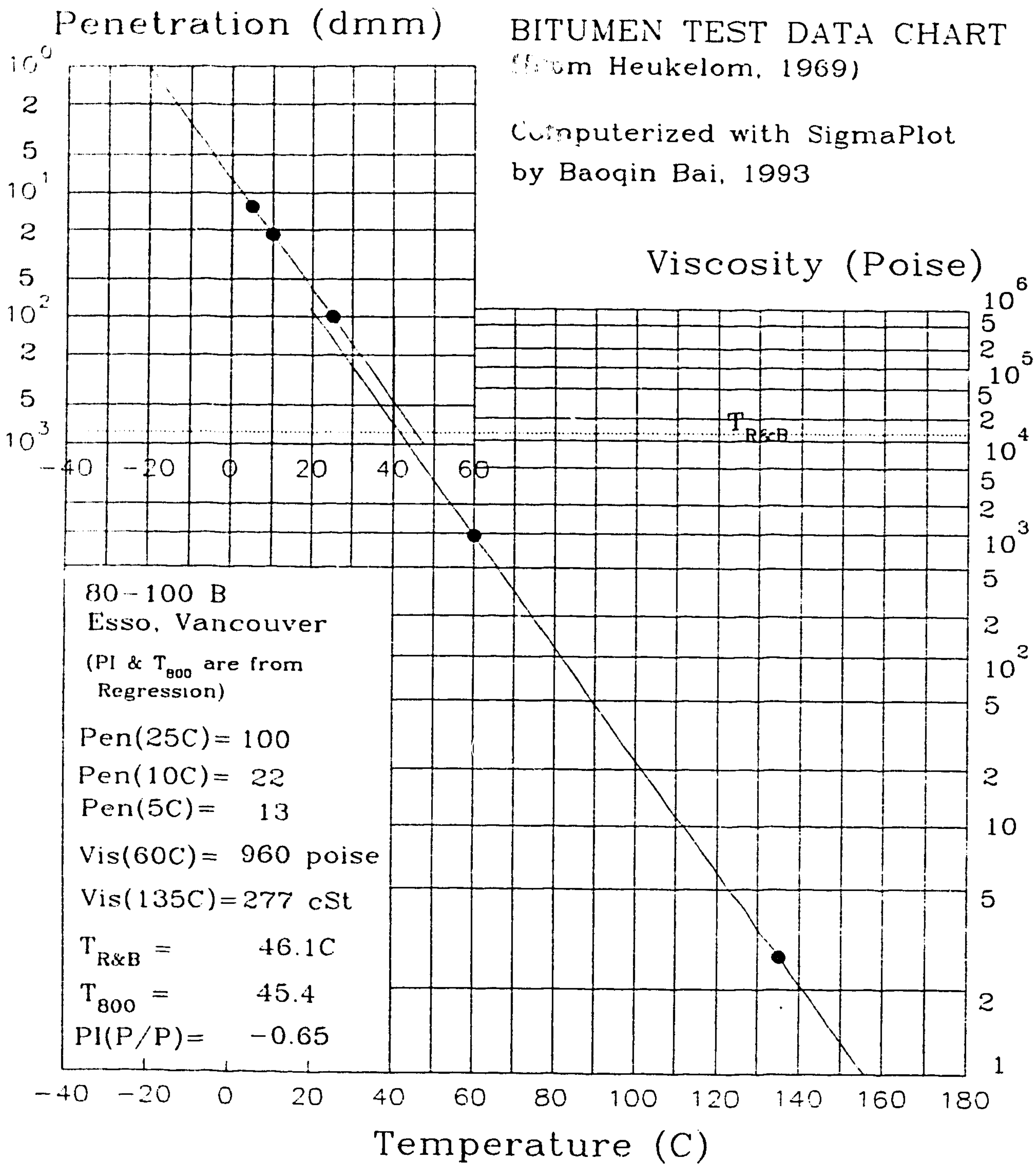

Fig.III.5-Asphalt Used in Lamont Test Section No.1

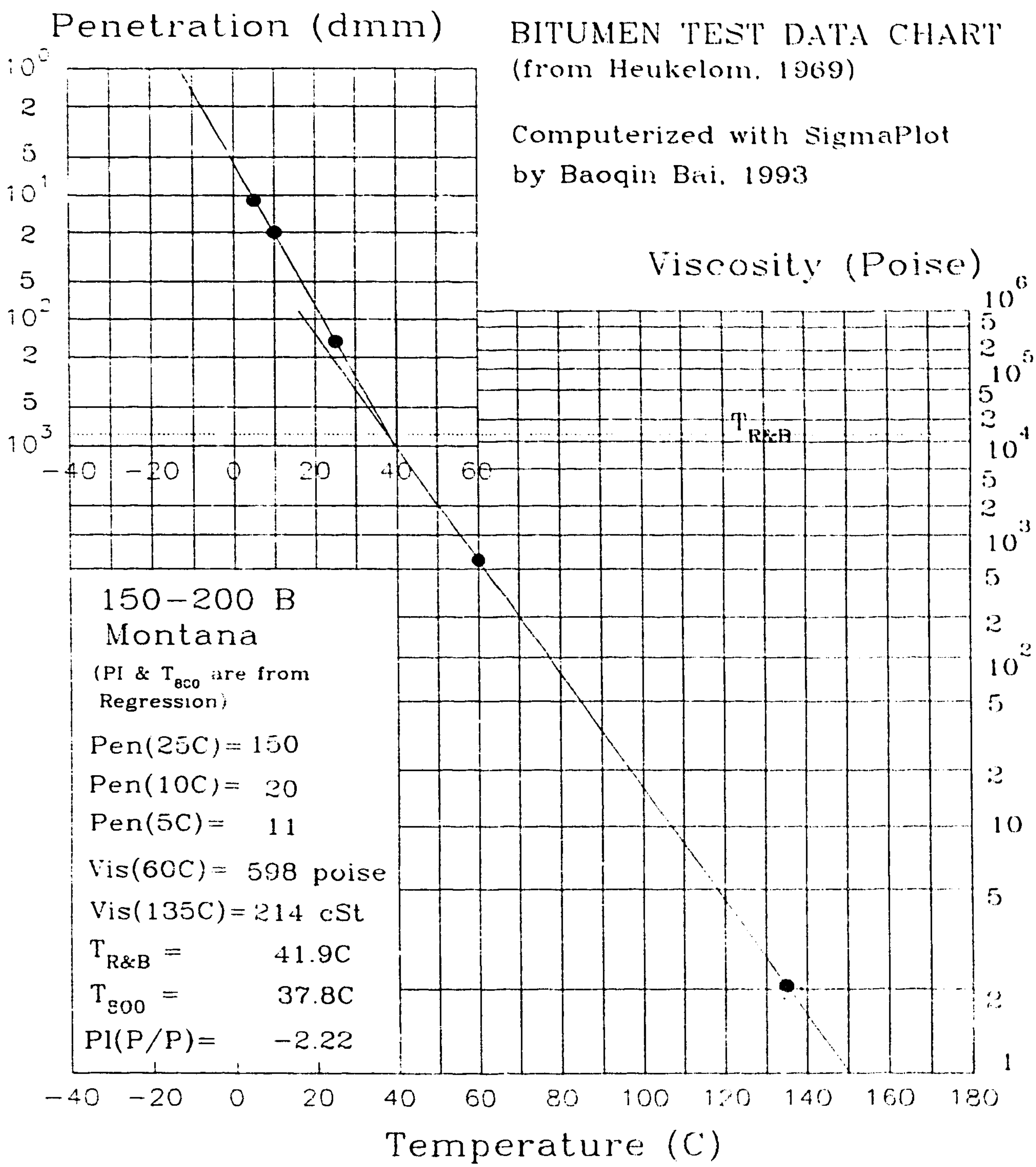

Fig.III.6-Asphalt Used in Lamont Test Section No.2

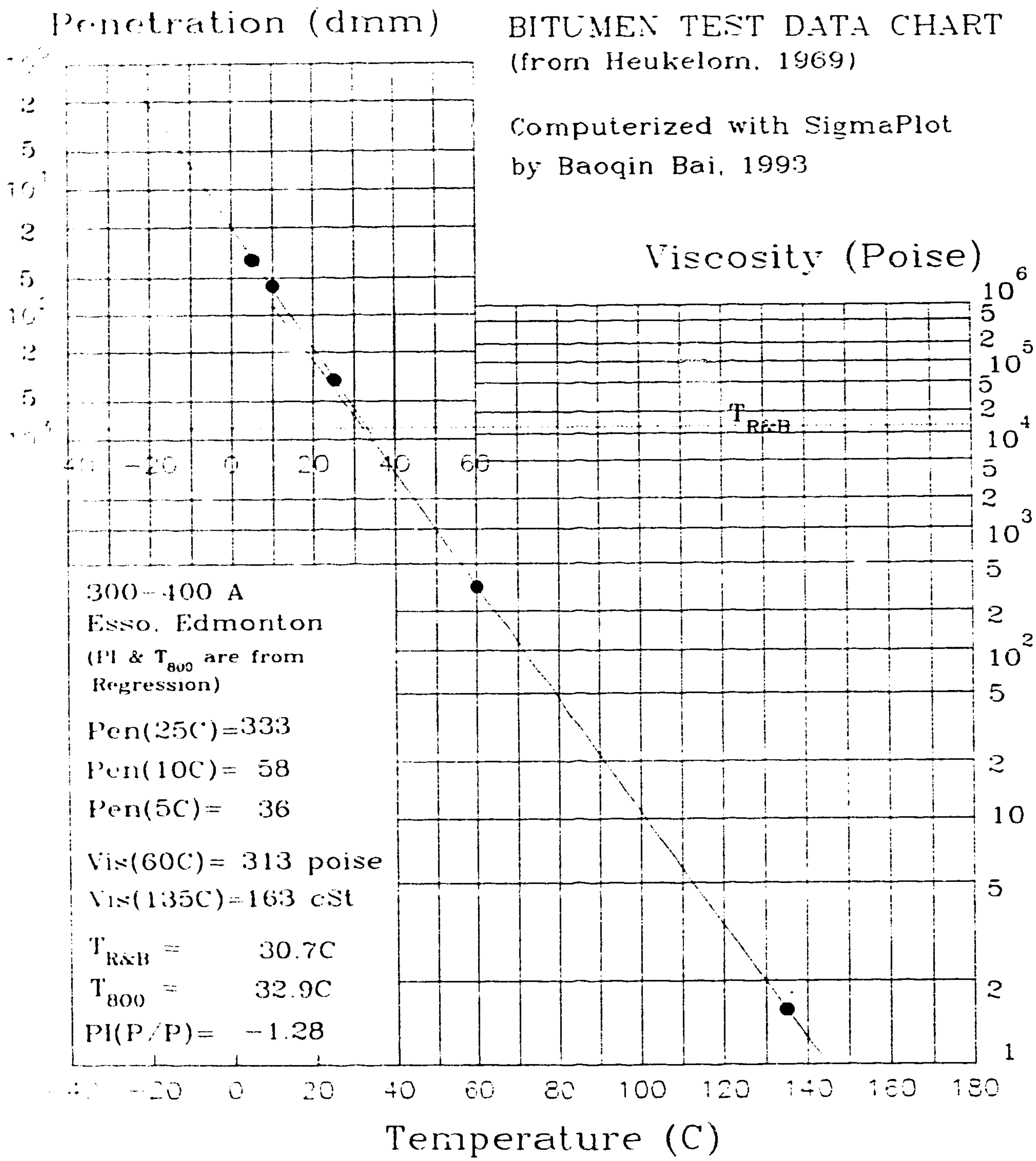

Fig.III.7-Asphalt Used in Lamont Test Section No.3

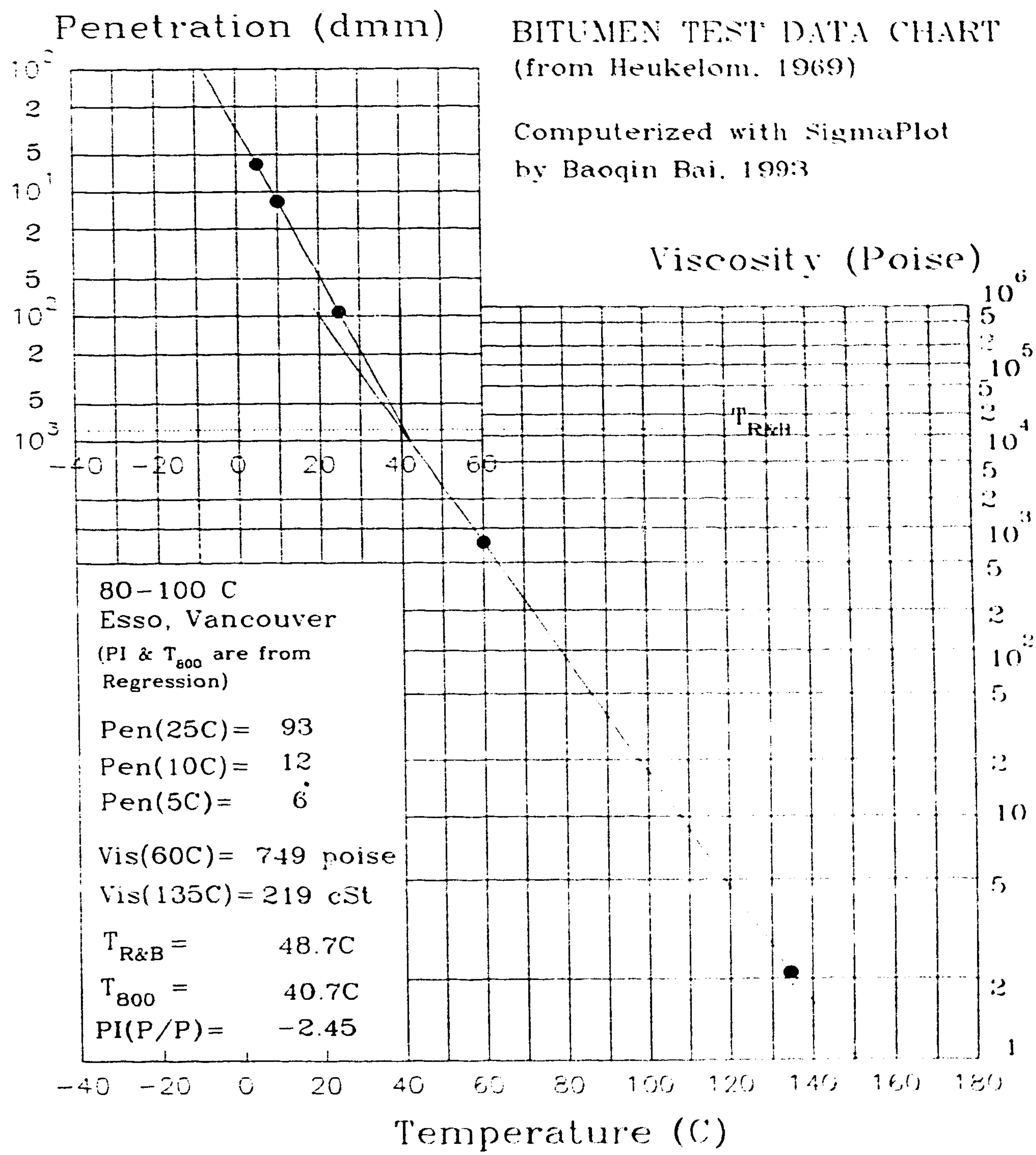

Fig.III.8-Asphalt Used in Lamont Test Section No.4

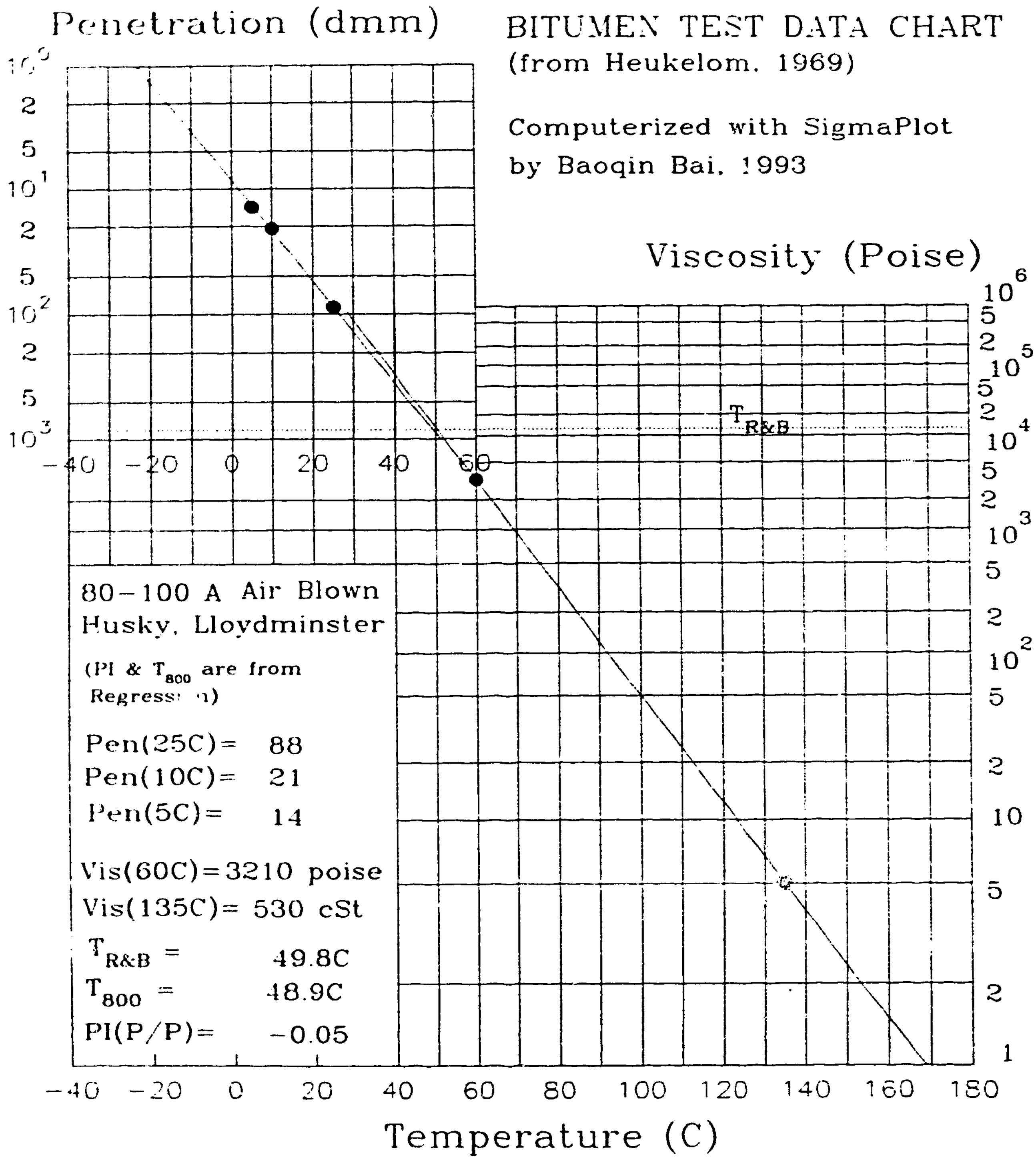

Fig.III.9-Asphalt Used in Lamont Test Section No.5

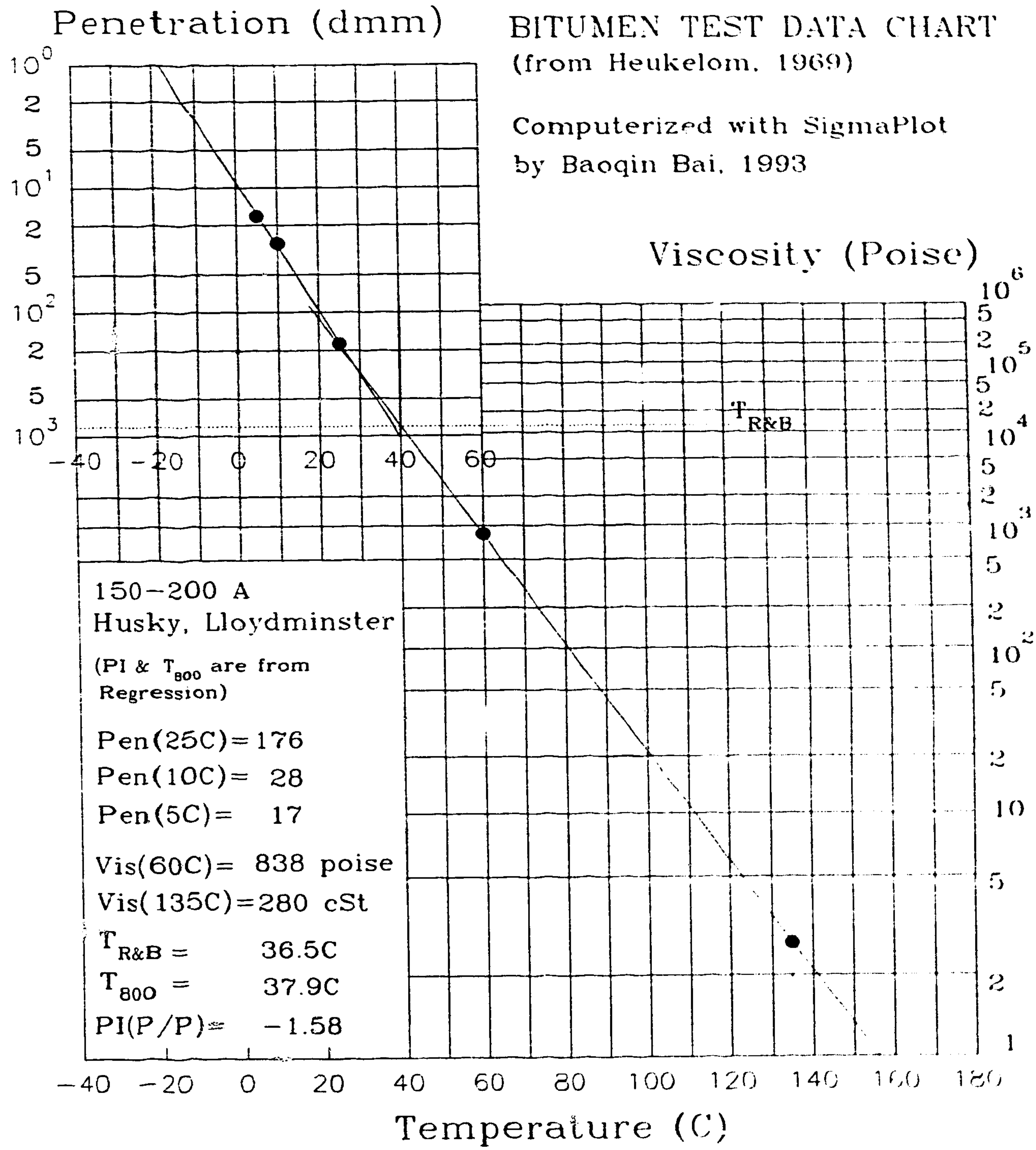

Fig.III.10-Asphalt Used in Lamont Test Section No.6

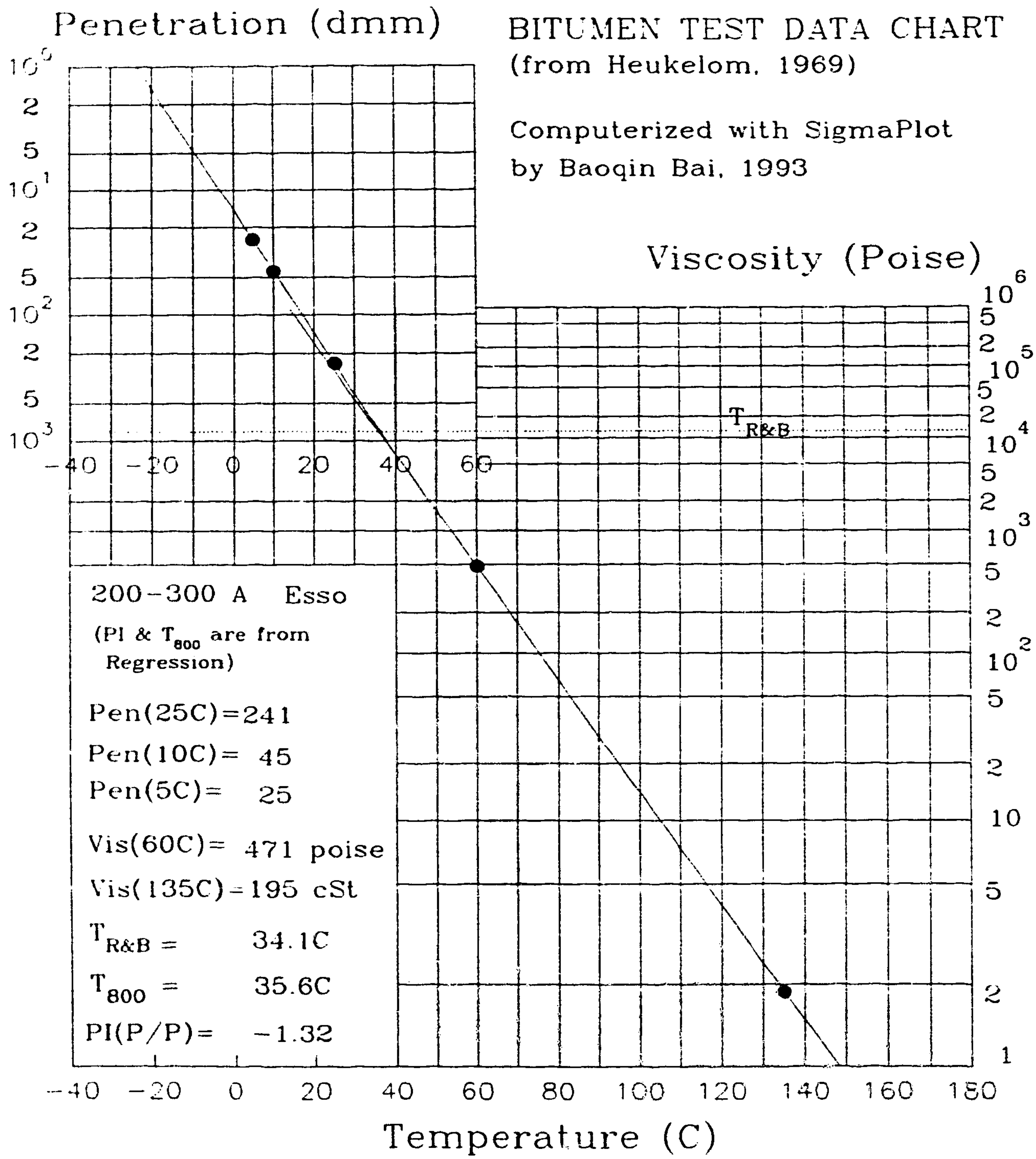

Fig.III.11-Asphalt Used in Lamont Test Section No.7

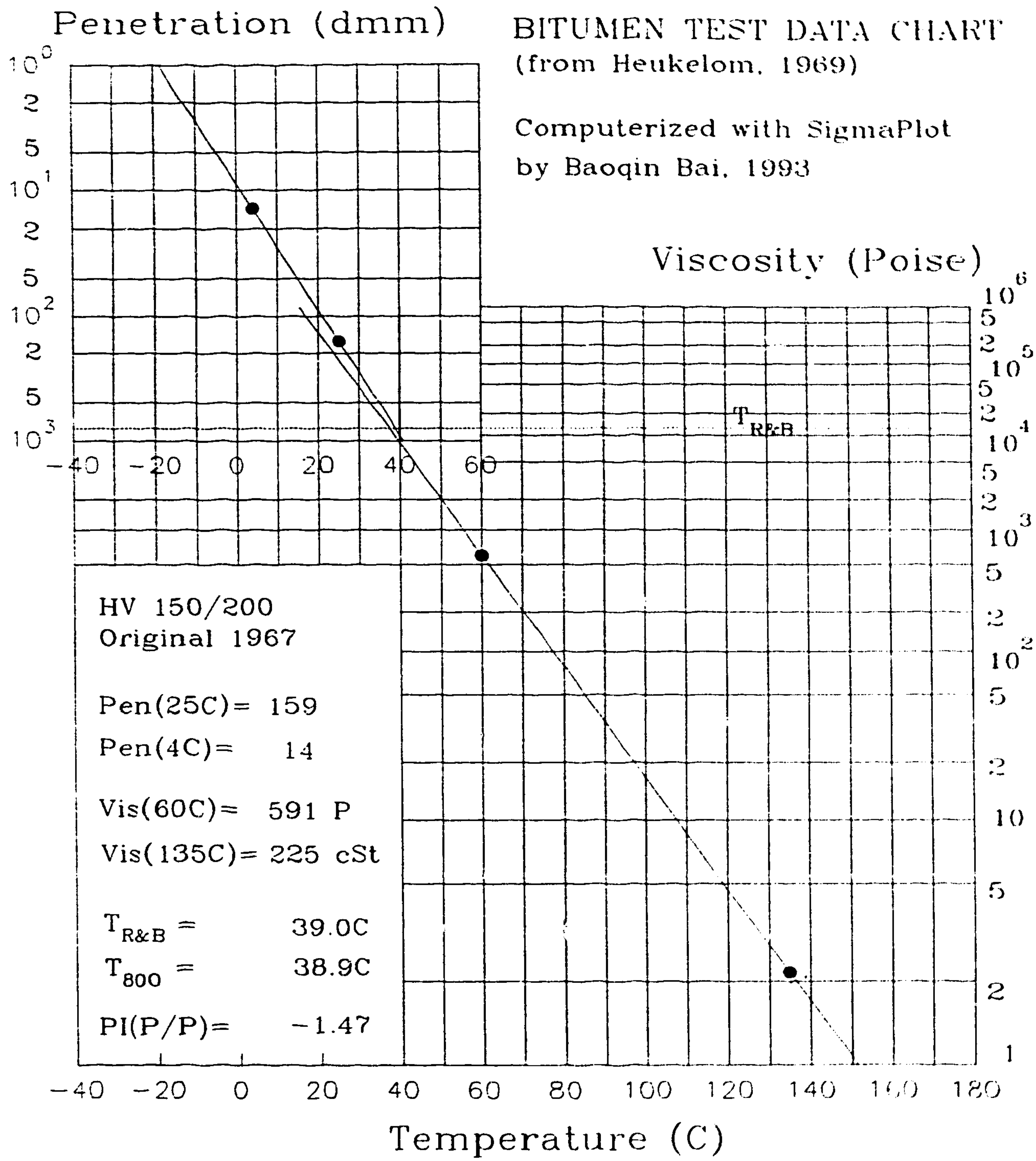

Fig.III.12-HV 150/200 Original Asphalt Used in Ste. Anne Test Road

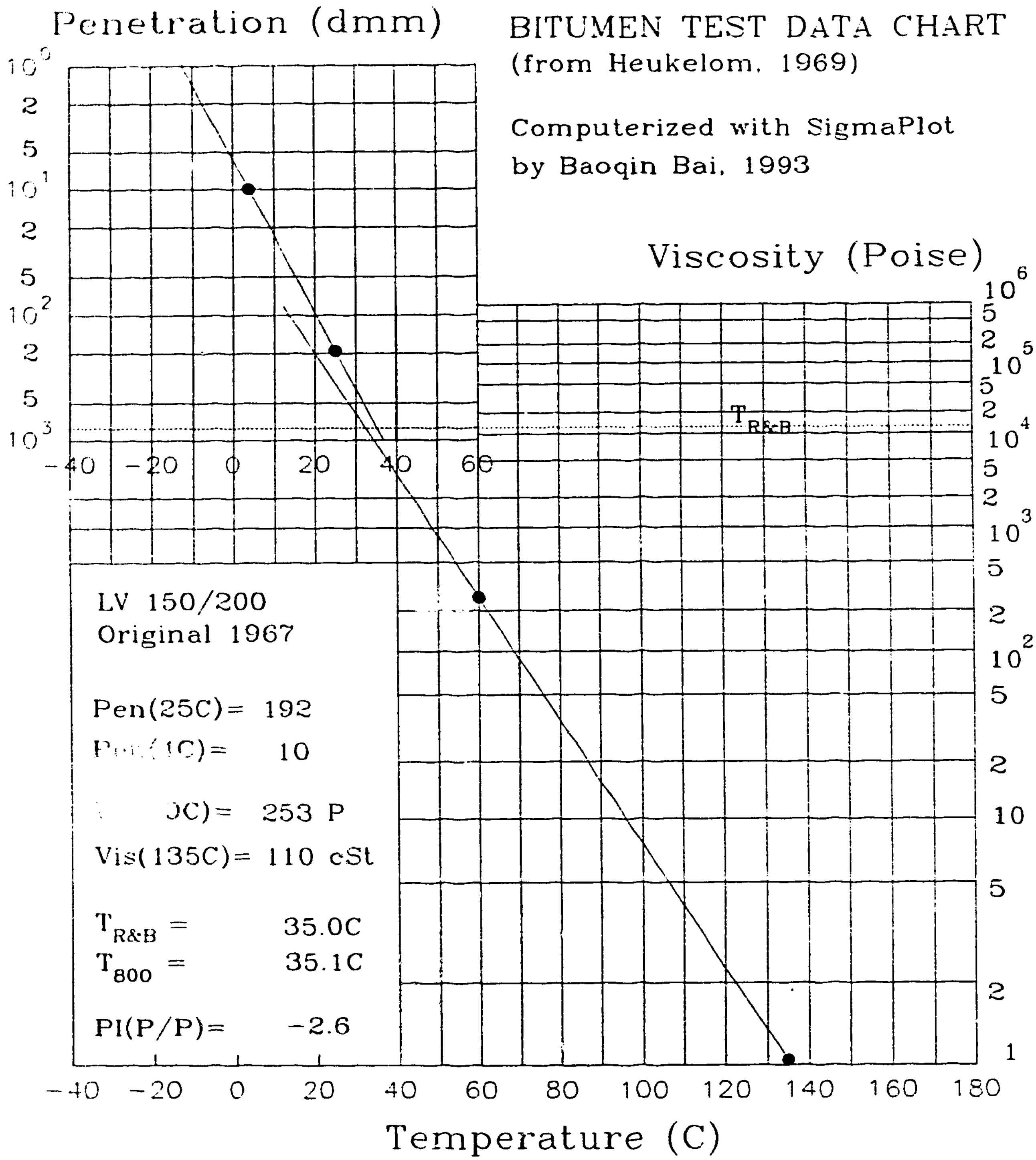

Fig.III.13-LV 150/200 Original Asphalt Used in Ste. Anne Test Road

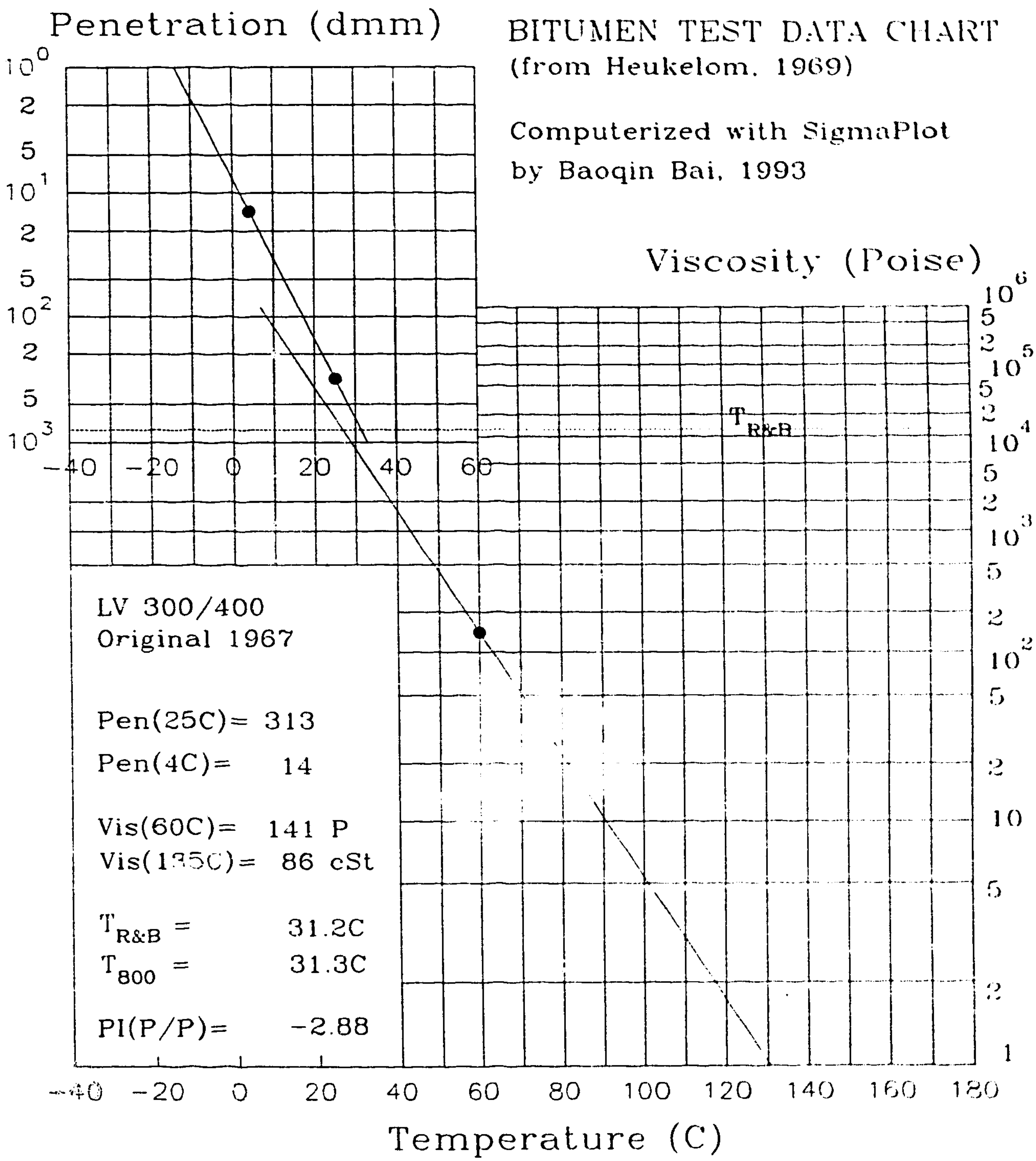

Fig.III.14-LV 300/400 Original Asphalt Used in Ste. Anne Test Road

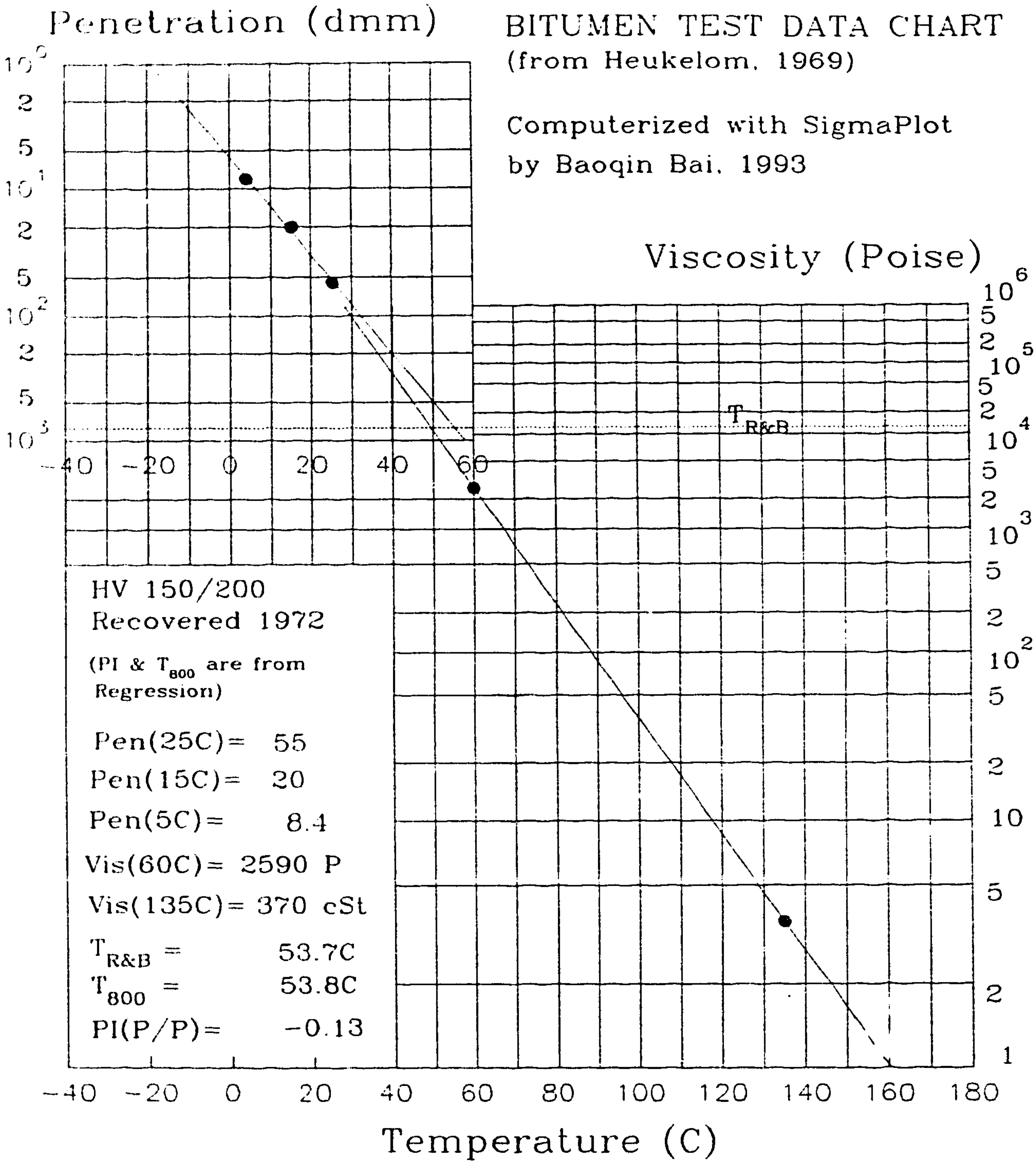

Fig.III.15-HV 150/200 Recovered Asphalt Used in Ste. Anne Test Road

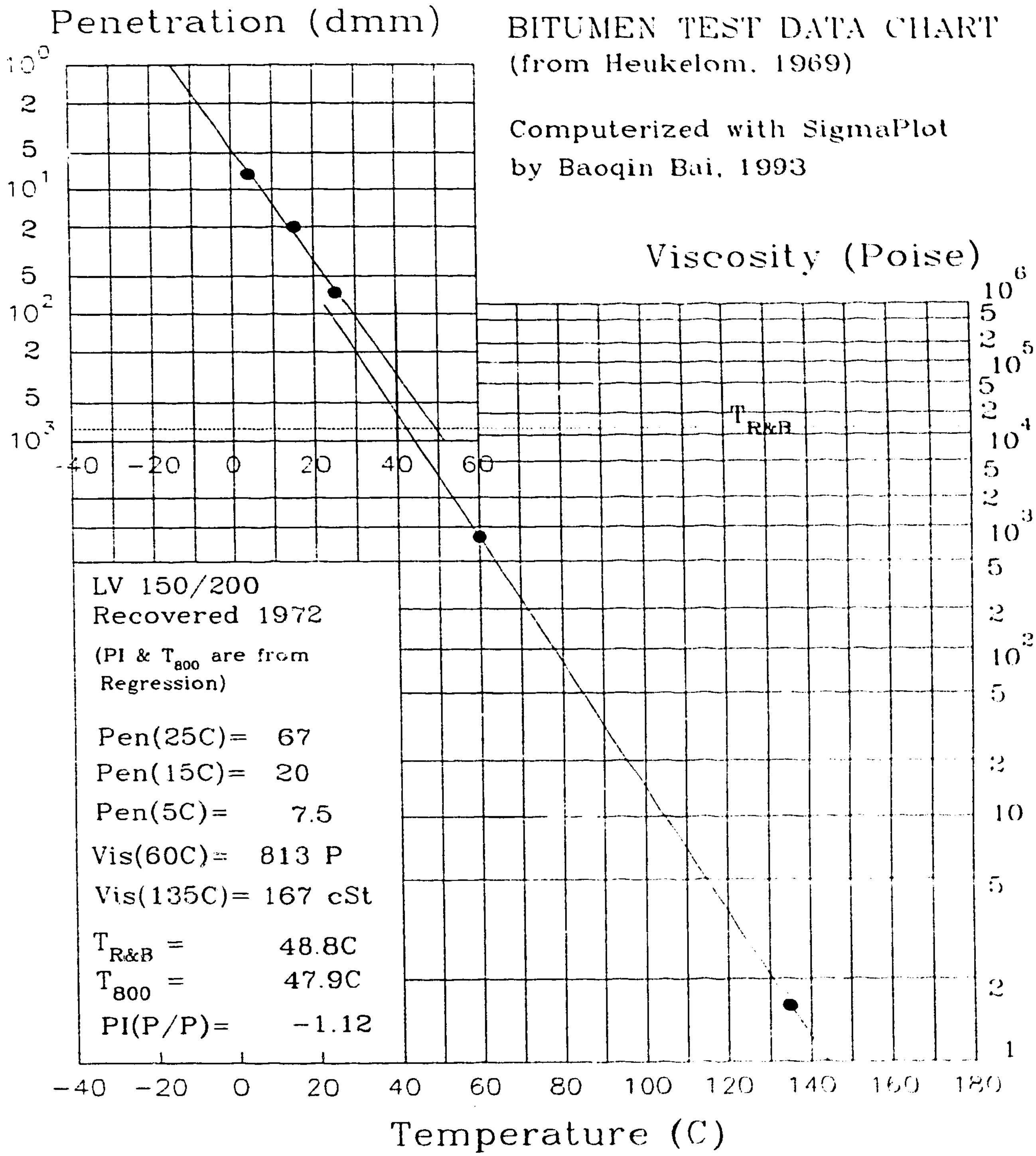

Fig.III.16-LV 150/200 Recovered Asphalt Used in Ste. Anne Test Road

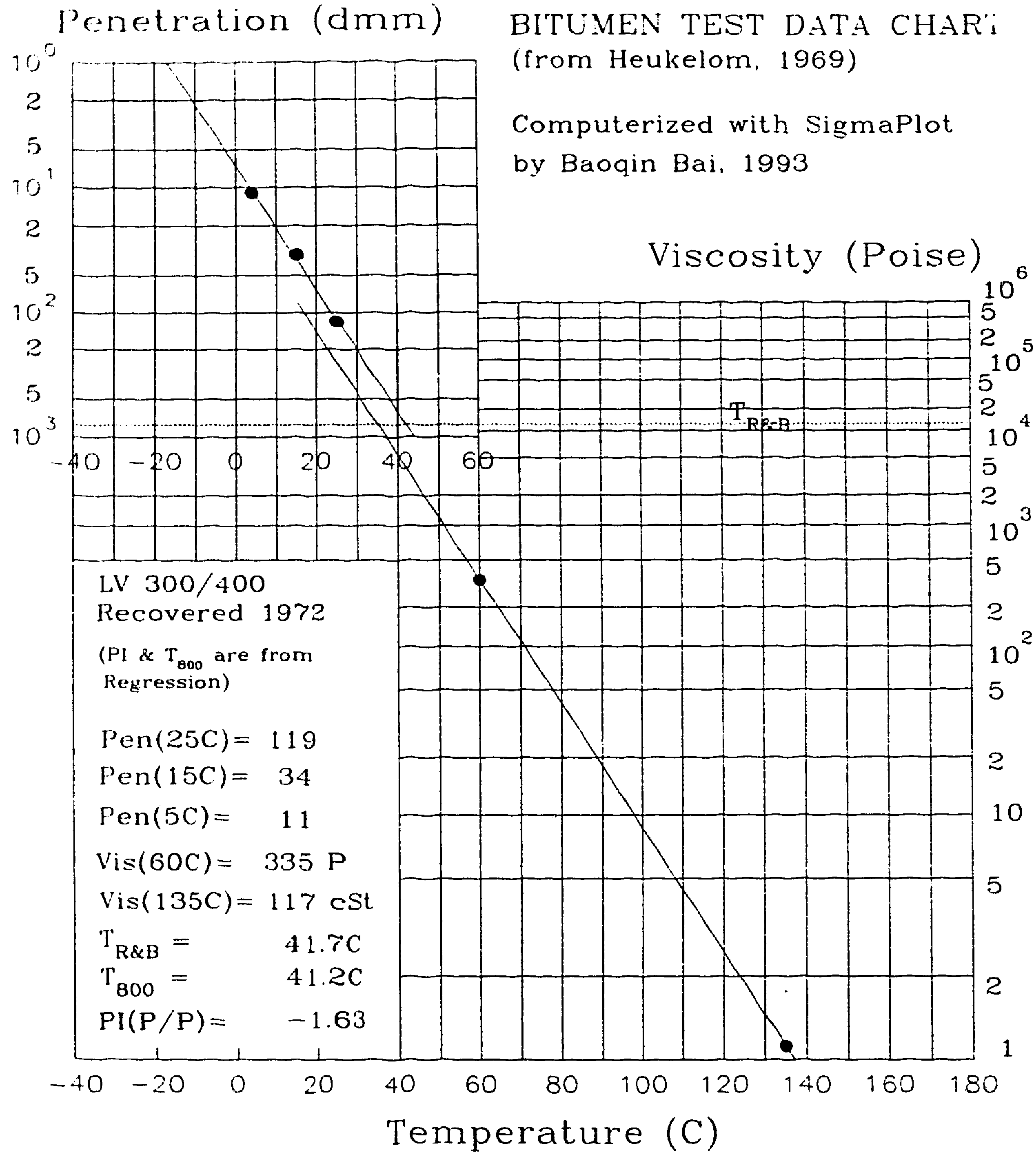

Fig.III.17-LV 300/400 Recovered Asphalt Used in Ste. Anne Test Road

Appendix IV

THE CALCULATED AND THE TESTED PROPERTIES OF THE MATERIALS USED IN LAMONT TEST ROAD

Table IV.1-Calculated and Tested Properties of the Materials Used in Lamont Test Section No.1

Asphalt Data: Esso 80/100, Air Blown, $P_{25°C}=100$, $P_{10°C}=22$, $P_{5°C}=13$ Mix Data: Vg = 84.9, Vb = 11.6, Va = 3.5 (%)								
Temp (°C)	Specim. No.	Ldg. Time (s)	Calculated Asphalt Stiffness (MPa)		Calculated Mix Stiffness (MPa)		Tested Mix Stiffness (MPa)	Tensile Strength (MPa)
			$P_{(25\&10)}$	$P_{(25\&5)}$	$P_{(25\&10)}$	$P_{(25\&5)}$		
0	TS1-4	67	1.85	1.38			4160	1.39
	TS1-5	79	1.60	1.20			696	1.90
	TS1-12	80	1.58	1.19			1090	2.12
	TS1-14	79	1.60	1.20			1160	2.03
	TS1-20	83	1.53	1.16			798	2.28
Average			1.63	1.23	687	570	1581	1.94
-10	TS1-1	82	20.8	11.3			6710	3.64
	TS1-3	82	20.8	11.3			3740	3.90
	TS1-8	77	21.7	11.7			4320	3.51
	TS1-13	97	18.4	10.1			3370	4.22
	TS1-19	87	19.9	10.8			2630	3.78
Average			20.3	11.0	3650	2430	4154	3.81
-20	TS1-7	78	172	71.0			12680	4.52
	TS1-10	83	167	68.7			29600	4.87
	TS1-11	69	183	75.6			12400	4.08
	TS1-15	74	177	72.9			9900	4.07
	TS1-17	77	173	71.4			5810	3.50
Average			174	71.9	13400	8440	14078	4.21
-30	TS1-2	86	646	267	24100	16300	34200	5.63

Note: $P_{(25\&10)}$ means penetrations at 25°C & 10°C, and the data in this column are the stiffness calculated by using the penetrations at 25°C & 10°C.

Table IV.2-Calculated and Tested Properties of the Materials Used in Lamont Test Section No.2

Asphalt Data: $P_{25^\circ C} = 150$, $P_{10^\circ C} = 20$, $P_{5^\circ C} = 11$ (dmm)
Mix Data: $V_g = 85.0$, $V_b = 11.6$, $V_a = 3.4$ (%)

Temp (°C)	Specim. No.	Ldg. Time (s)	Calculated Asphalt Stiffness (MPa)		Calculated Mix Stiffness (MPa)		Tested Mix Stiffness (MPa)	Tensile Strength (MPa)
			$P_{(25\&10)}$	$P_{(25\&5)}$	$P_{(25\&10)}$	$P_{(25\&5)}$		
	TS2-1	104	0.565	0.447			865	1.61
0	TS2-5	90	0.649	0.513			1400	2.17
	TS2-14	85	0.688	0.541			1610	2.95
	TS2-17	70	0.841	0.647			1320	2.42
	TS2-20	93	0.628	0.497			835	2.40
Average			0.674	0.529	386	329	1206	2.31
	TS2-2	74	20.3	13.1			14000	4.39
-10	TS2-3	142	11.4	7.16			4744	1.68
	TS2-6	84	18.1	11.8			4230	4.14
	TS2-7	87	17.5	11.4			12500	4.43
	TS2-19	80	18.9	12.3			6720	3.92
Average			17.2	11.2	3300	2490	8439	3.71
	TS2-11	76	312	161			6330	4.51
-20	TS2-13	75	315	162			21900	5.28
	TS2-18	86	288	148			4040	4.55
Average			305	157	17400	13000	10757	4.78
	TS2-8	85	1270	932			11800	5.68
-30	TS2-15	77	1300	965			26100	4.87
	TS2-16	77	1300	965			10200	3.90
Average			1290	954	31800	28900	16033	4.82

Table IV.3-Calculated and Tested Properties of the Materials Used in Lamont Test Section No.3

Asphalt Data: $P_{25°C}$ = 333, $P_{10°C}$ = 58, $P_{5°C}$ = 36 (dmm)
Mix Data: V_g = 84.5, V_b = 12.0, V_a = 3.5 (%)

Temp (°C)	Specim. No.	Ldg. Time (s)	Calculated Asphalt Stiffness (MPa)		Calculated Mix Stiffness (MPa)		Tested Mix Stiffness (MPa)	Tensile Strength (MPa)
			$P_{(25\&10)}$	$P_{(25\&5)}$	$P_{(25\&10)}$	$P_{(25\&5)}$		
0	TS3-6	99	0.0816	0.0750			336	1.06
	TS3-8	98	0.0824	0.0757			229	0.88
	TS3-10	109	0.0746	0.0686			118	0.87
	TS3-13	95	0.0848	0.0779			410	0.95
	TS3-18	93	0.0865	0.0794			262	0.96
Average			0.0820	0.0753	80.0	75.5	271	0.94
-10	TS3-2	101	1.24	0.991			1070	3.04
	TS3-12	105	1.20	0.959			955	3.52
	TS3-14	99	1.27	1.01			1240	2.99
	TS3-16	95	1.31	1.04			825	3.23
Average			1.26	1.00	514	439	1023	3.20
-20	TS3-7	87	19.8	14.5			5180	5.11
	TS3-11	92	19.0	13.9			3715	4.68
	TS3-17	77	21.7	15.9			8220	4.50
Average			20.2	14.8	3400	2750	5705	4.76
-30	TS3-4	102	158	104			20500	5.56
	TS3-15	88	170	112			17500	5.86
Average			164	108	12700	10500	19000	5.71

Table IV.4-Calculated and Tested Properties of the Materials Used in Lamont Test Section No.4

Asphalt Data: $P_{25°C} = 93$, $P_{10°C} = 12$, $P_{5°C} = 6$ (dmm)
Mix Data: Vg = 84.4, Vb = 12.1, Va = 3.5 (%)

Temp (°C)	Specim. No.	Ldg. Time (s)	Calculated Asphalt Stiffness (MPa)		Calculated Mix Stiffness (MPa)		Tested Mix Stiffness (MPa)	Tensile Strength (MPa)
			$P_{(25\&10)}$	$P_{(25\&5)}$	$P_{(25\&10)}$	$P_{(25\&5)}$		
0	TS4-4	93	2.30	2.69			746	2.27
	TS4-16	91	2.35	2.75			697	2.22
	TS4-17	93	2.30	2.69			4690	3.52
	TS4-20	106	2.02	2.73			1340	3.18
Average			2.24	2.72	777	885	1868	2.80
-10	TS4-10	72	47.8	63.0			7870	3.94
	TS4-11	85	42.6	55.6			8520	4.44
	TS4-14	77	45.6	59.9			12200	4.34
	TS4-15	73	47.3	62.4			15900	3.66
Average			45.8	60.2	5920	7110	11123	4.10
-20	TS4-7	101	379	491			6850	5.63
	TS4-8	78	432	545			25400	4.74
Average			406	518	19000	21200	16125	5.19
-30	TS4-3	66	1350	1540			24900	5.19
	TS4-12	74	1330	1520			6770	4.52
	TS4-13	64	1360	1540			49300	4.05
Average			1350	1530	31400	32600	26990	4.59

Table IV.5-Calculated and Tested Properties of the Materials Used in Lamont Test Section No.5

Asphalt Data: $P_{25°C}$ = 88, $P_{10°C}$ = 21, $P_{5°C}$ = 14 (dmm)
Mix Data: Vg = 84.5, Vb = 12.0, Va = 3.5 (%)

Temp (°C)	Specim. No.	Ldg. Time (s)	Calculated Asphalt Stiffness (MPa)		Calculated Mix Stiffness (MPa)		Tested Mix Stiffness (MPa)	Tensile Strength (MPa)
			$P_{(25\&10)}$	$P_{(25\&5)}$	$P_{(25\&10)}$	$P_{(25\&5)}$		
0	TS5-18	72	1.35	1.37			1460	2.01
	TS5-20	76	1.30	1.32			2200	2.43
Average			1.33	1.35	557	563	1830	2.22
	TS5-1	72	9.88	10.3			2900	3.57
-10	TS5-5	81	9.19	9.59			2490	3.95
	TS5-9	86	8.86	9.24			5130	4.07
	TS5-11	81	9.19	9.59			3860	4.10
	TS5-16	84	8.99	9.38			6350	3.58
Average			9.22	9.62	2040	2100	4146	3.85
-20	TS5-6	87	48.3	51.1			7810	4.28
	TS5-12	84	49.0	51.9			24600	4.61
Average			48.7	51.5	6230	6470	16205	4.45
	TS5-2	79	171	182			24900	5.46
-30	TS5-4	82	169	179			20900	4.36
	TS5-8	95	160	171			15800	5.20
Average			167	177	12800	13100	20533	5.01

Table IV.6-Calculated and Tested Properties of the Materials Used in Lamont Test Section No.6

Asphalt Data: $P_{25°C}$ = 176, $P_{10°C}$ = 28, $P_{5°C}$ = 17 (dmm)
Mix Data: Vg = 84.9, Vb = 11.6, Va = 3.5 (%)

Temp (°C)	Specim. No.	Ldg. Time (s)	Calculated Asphalt Stiffness (MPa)		Calculated Mix Stiffness (MPa)		Tested Mix Stiffness (MPa)	Tensile Strength (MPa)
			$P_{(25\&10)}$	$P_{(25\&5)}$	$P_{(25\&10)}$	$P_{(25\&5)}$		
	TS6-6	90	0.432	0.395			678	2.12
0	TS6-11	87	0.445	0.407			1330	1.75
	TS6-12	93	0.420	0.384			1360	1.88
	TS6-16	84	0.459	0.420			543	1.71
Average			0.439	0.402	288	272	978	1.87
	TS6-1	101	7.28	5.72			1730	4.16
-10	TS6-4	102	7.21	5.67			3670	4.05
	TS6-10	86	8.38	6.55			3000	4.13
	TS6-17	87	8.29	6.49			3000	4.12
	TS6-18	83	8.63	6.75			4960	4.45
Average			7.96	6.24	1960	1670	3272	4.18
-30	TS6-2	87	653	455			9530	4.39
	TS6-15	90	644	448			22700	5.20
Average			649	452	24200	20600	16115	4.80

Table IV.7-Calculated and Tested Properties of the Materials Used in Lamont Test Section No.7

Asphalt Data: $P_{25°C} = 241$, $P_{10°C} = 45$, $P_{5°C} = 25$ (dmm)
Mix Data: Vg = 84.5, Vb = 12.0, Va = 3.5 (%)

Temp (°C)	Specim. No.	Ldg. Time (s)	Calculated Asphalt Stiffness (MPa)		Calculated Mix Stiffness (MPa)		Tested Mix Stiffness (MPa)	Tensile Strength (MPa)
			$P_{(25\&10)}$	$P_{(25\&5)}$	$P_{(25\&10)}$	$P_{(25\&5)}$		
0	TS7-3	94	0.200	0.159			374	1.72
	TS7-9	94	0.200	0.159			755	1.51
	TS7-18	86	0.217	0.171			389	1.48
	TS7-19	69	0.265	0.206			296	1.37
Average			0.221	0.174	167	142	454	1.52
-10	TS7-2	101	3.48	1.93			1330	3.24
	TS7-6	99	3.54	1.96			1846	3.04
	TS7-8	98	3.58	1.98			806	2.74
	TS7-16	88	3.96	2.17			1780	3.72
	TS7-20	89	3.91	2.15			1630	3.53
Average			3.69	2.04	1100	742	1478	3.25
-30	TS7-1	88	468	167			12400	5.44
	TS7-5	94	455	161			9600	5.79
	TS7-7	81	484	174			18100	5.28
	TS7-11	77	494	179			16400	4.50
	TS7-13	92	459	163			2170	4.80
Average			472	169	20500	12800	11734	5.16

Appendix V

INPUT AND OUTPUT FOR THE FOUR DIFFERENT CRACKING TEMPERATURE PREDICTION METHODS

V.1 The Improved Theoretical Method

A computer program Asphalt Property Evaluation at Low Temperatures (ASPELT) has been developed for this method to calculate the cracking temperatures of asphalt pavements. Following are the input and the output used in the study.

INPUT:

The loading times and temperatures used for asphalt stiffness in van der Poel nomograph:

1) Loading times: 10^{-3}, 10^{-2}, 10^{-1}, 10^{0}, 10^{1}, 10^{2}, 10^{3}, and 10^{4} seconds,
2) Temperatures: 0, -10, -20, -30, -40, and -50°C,

For Ste. Anne Test Road:

3) Asphalt properties: penetration at 25°C and penetration at 4 or 5°C,
4) Mix properties. mix composition as shown in Table 5.3;
   Tensile strength as shown in Fig.V.1 from Deme and Young (1987);
   Coefficient of contraction = 2.04×10^{-5}/°C, average value of the contraction coefficients of the three asphalt mixes used in Ste. Anne Test Road (Burgess et al. 1971).
5) Cooling rate: 1.5°C/hour

For Lamont Test Road:

3) Asphalt properties: PI (penetration index) and T_{800} (temperature at which penetration is 800 dmm) based on the regression of the penetrations at 25, 10, and 5°C.

4) Mix properties: mix composition as shown in Table 5.4;
Tensile strength: obtained from indirect tensile tests (Fig.4.10) and from Ste. Anne Test Road direct tensile test shown in Fig.V.1 (Deme and Young, 1987);
Coefficient of contraction = 2.04×10^{-5}/°C, average value of the contraction coefficients of the asphalt mixes used in Ste. Anne Test Road and
the contraction coefficient calculated by the equation developed by Jones et al. (1968)

$$\alpha_{mix} = (V_{ac}*B_{ac} + V_{agg}*B_{agg}) / (3*V_{mix})$$

where
α_{mix} = theoretical linear thermal contraction coefficient of asphalt mix,
V_{ac} = volume percentage of asphalt in mix,
B_{ac} = cubic thermal coefficient of asphalt, input value $=6.0\times10^{-4}$/°C
V_{agg} = volume percentage of aggregates in mix,
B_{agg} = cubic thermal coefficient of aggregates, input value $= 3.0\times10^{-5}$/°C
V_{mix} = total volume of mix.

5) Cooling rate: 1, 1.5, 3, 5, and 10°C/hour

BANDS-PC (Bitumen and Asphalt Nomographs Developed by Shell for use on Personal Computers) is used to calculate the asphalt stiffness.

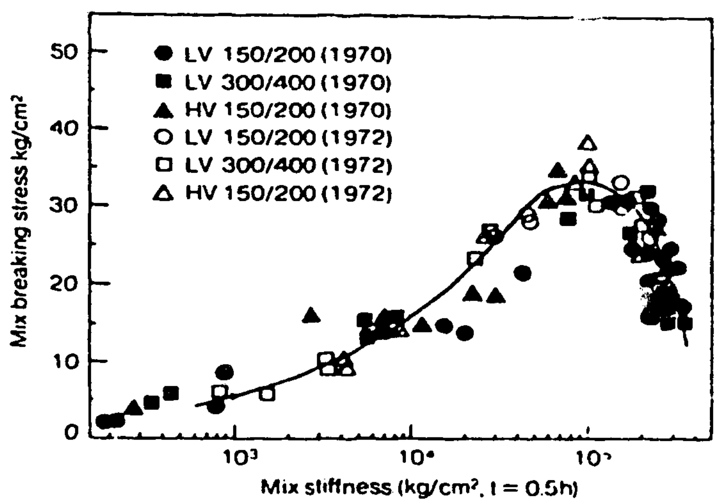

Fig.V.1-Tensile Strength versus Stiffness of Asphalt Mix

OUTPUT:

Table V.1 to Table V.5 are the results of the cracking temperature prediction for Lamont Test Road using the program ASPELT.

Table V.1-Cracking Temperature Prediction with Improved Theoretical Method for Asphalt Mixes Used in Lamont Test Road

The cooling rate: **1°C/hour**.

Test Section	Asphalt Source	Predicted Cracking Temperature (°C)				Observed Cracking Frequency
		Failure Stress from Deme & Young		Failure Stress from Indirect Tensile Test		
		α1	α2	α1	α2	(Cracks/km)
TS 1	Esso 80/100B	-39.7	-33.4	-44.1	-38.1	28
TS 2	Montana 150/200B	-35.1	-30.1	-38.9	-33.9	92
TS 3	Esso 300/400A	-47.2	-40.6	< -50	-45.9	0
TS 4	Esso 80/100C	-31.2	-26.3	-34.6	-29.7	110
TS 5	Husky 80/100A	-41.2	-34.2	-45.9	-39.2	2
TS 6	Husky 150/200A	-39.7	-34.4	-43.9	-38.4	0
TS 7	Esso 200/300A	-44.1	-38.1	-48.8	-42.8	0

Note: $\alpha 1 = 2.04\times10^{-5}$/°C which is the average value of the contraction coefficients of the three asphalt mixes used in Ste. Anne Test Road.

$\alpha 2 = (Vb^* Cb + Vg * Cg) / 300$, where

Vb is volume percentage of asphalt in asphalt mixture.

Vg is volume percentage of aggregate in asphalt mixture.

$Cb = 6.0\times10^{-4}$ /°C is the volume contraction coefficient of asphalt.

$Cg = 3.0\times10^{-5}$ /°C is the volume contraction coefficient of aggregate.

Table V.2-Cracking Temperature Prediction with Improved Theoretical Method for Asphalt Mixes Used in Lamont Test Road

The cooling rate: **1.5°C/hour**.

Test Section	Asphalt Source	Predicted Cracking Temperature (°C)				Observed Cracking Frequency
		Failure Stress from Deme & Young		Failure Stress from Indirect Tensile Test		
		α1	α2	α1	α2	(Cracks/km)
TS 1	Esso 80/100B	-38.6	-32.5	-43.0	-37.1	28
TS 2	Montana 150/200B	-34.1	-29.1	-37.9	-33.0	92
TS 3	Esso 300/400A	-46.0	-39.5	< -50.0	-44.7	0
TS 4	Esso 80/100C	-30.3	-25.5	-33.6	-28.7	110
TS 5	Husky 80/100A	-40.1	-33.3	-44.7	-38.1	2
TS 6	Husky 150/200A	-38.7	-33.1	-42.8	-37.4	0
TS 7	Esso 200/300A	-43.1	-37.0	-47.8	-41.7	0

Table V.3-Cracking Temperature Prediction with Improved Theoretical Method for Asphalt Mixes from Lamont Test Road

The cooling rate: **3°C/hour.**

Test Section	Asphalt Source	Predicted Cracking Temperature (°C)				Observed Cracking Frequency
		Failure Stress from Deme & Young		Failure Stress from Indirect Tensile Test		
		α1	α2	α1	α2	(Cracks/km)
TS 1	Esso 80/100B	-36.9	-30.8	-41.1	-35.4	28
TS 2	Montana 150/200B	-32.6	-27.6	-36.1	-31.1	92
TS 3	Esso 300/400A	-44.2	-37.7	-48.9	-42.5	0
TS 4	Esso 80/100C	-28.7	-23.8	-32.0	-27.0	110
TS 5	Husky 80/100A	-38.3	-31.6	-42.7	-36.4	2
TS 6	Husky 150/200A	-36.9	-31.4	-41.0	-35.7	0
TS 7	Esso 200/300A	-41.2	-35.3	-45.7	-39.6	0

Table V.4-Cracking Temperature Prediction with Improved Theoretical Method for Asphalt Mixes Used in Lamont Test Road

The cooling rate: **5°C/hour.**

Test Section	Asphalt Source	Predicted Cracking Temperature (°C)				Observed Cracking Frequency
		Failure Stress from Deme & Young		Failure Stress from Indirect Tensile Test		
		α1	α2	α1	α2	(Cracks/km)
TS 1	Esso 80/100B	-35.5	-29.5	-40.0	-33.8	28
TS 2	Montana 150/200B	-31.4	-26.4	-34.8	-29.8	92
TS 3	Esso 300/400A	-42.8	-36.4	-47.6	-41.0	0
TS 4	Esso 80/100C	-27.6	-22.7	-30.7	-25.8	110
TS 5	Husky 80/100A	-36.9	-30.3	-41.7	-34.8	2
TS 6	Husky 150/200A	-35.8	-30.3	-39.7	-34.5	0
TS 7	Esso 200/300A	-40.0	-33.9	-44.2	-38.4	0

Table V.5-Cracking Temperature Prediction with Improved Theoretical Method for Asphalt Mixes Used in Lamont Test Road

The cooling rate: **10°C/hour.**

Test Section	Asphalt Source	Predicted Cracking Temperature (°C)				Observed Cracking Frequency
		Failure Stress from Deme & Young		Failure Stress from Indirect Tensile Test		
		α1	α2	α1	α2	(Cracks/km)
TS 1	Esso 80/100B	-33.7	-27.8	-38.0	-31.9	28
TS 2	Montana 150/200B	-29.8	-24.8	-33.0	-28.1	92
TS 3	Esso 300/400A	-40.9	-34.8	-45.5	-39.0	0
TS 4	Esso 80/100C	-25.9	-21.0	-29.0	-24.1	110
TS 5	Husky 80/100A	-35.2	-28.4	-39.8	-32.8	2
TS 6	Husky 150/200A	-34.1	-28.5	-38.0	-32.4	0
TS 7	Esso 200/300A	-38.1	-32.0	-42.3	-36.5	0

Fig.V.2 is an example output of the program ASPELT for the asphalt mix used in the test section No.1 of Lamont Test Road. The cooling rate is 3°C/hour, the contraction coefficient is 2.04×10^{-5}/°C, and the failure stress is obtained from Fig.4.10 which is from the indirect tensile test.

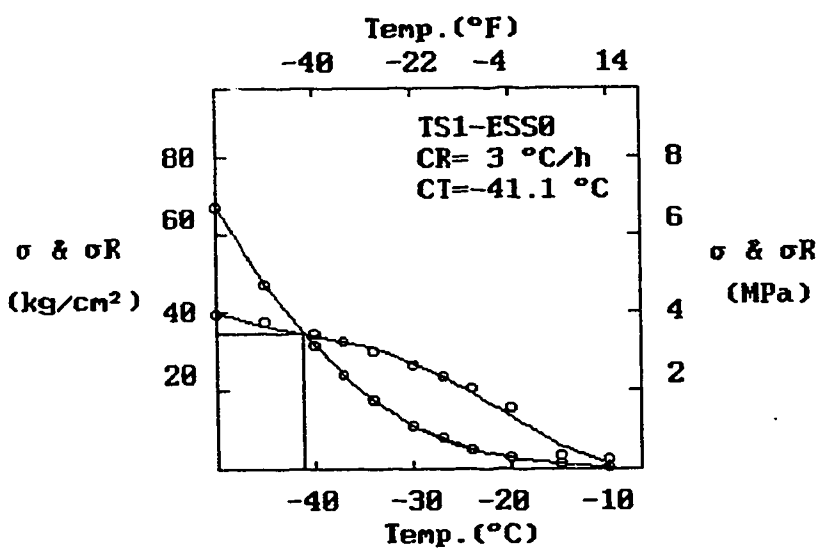

FIG. -CALCULATION OF CRACKING TEMPERATURE

Fig.V.2-An Example Output of the Program ASPELT

V.2 Critical Stiffness Method Developed by Deme and Young

BANDS-PC is used to calculate asphalt stiffness.

INPUT:

1) Loading time: 1800 seconds
2) Temperature: from 0 to -60°C
3) Asphalt properties:

   For Ste. Anne test road: penetration at 25°C and penetration at 4 or 5°C.

   For Lamont test road: PI (penetration index) and T_{800} (temperature at which penetration is 800 dmm) based on the regression of penetrations at 25, 10, and 5°C.

OUTPUT:

Following table shows the prediction process.

Table V.6-Cracking Temperature Prediction Process with Deme and Young's Method

Test Section No.	Time of Loading		Temp. Bitumen	Temp. 800 pen	Pen. Index	Bitumen Stiffness
	Seconds	Other Time Unit	°C	°C	-	MPa
TS 1	1800.000	30.00 Minutes	-40.0	45.4	-.65	382.000
	---"----	---"--- ---"---	-50.0	---"---	---"---	943.000
	---"----	---"--- ---"---	-60.0	---"---	---"---	1680.000
	1800.000	30.00 Minutes	-50.0	45.4	-.65	943.000
	---"----	---"--- ---"---	***-51.0***	—"—	—"—	***1000.000***
	---"----	---"--- ---"---	-52.0	---"---	---"---	1060.000
TS 2	1800.000	30.00 Minutes	-30.0	37.8	-2.2	327.000
	---"----	---"--- ---"---	-40.0	---"---	---"---	1270.000
	---"----	---"--- ---"---	-50.0	---"---	---"---	Glassy..
	1800.000	30.00 Minutes	-36.0	37.8	-2.2	887.000
	---"----	---"--- ---"---	***-37.0***	—"—	—"—	***1010.000***
	---"----	---"--- ---"---	-38.0	---"---	---"---	1150.000
TS 3	1800.000	30.00 Minutes	-50.0	32.9	-1.3	669.000
	---"----	---"--- ---"---	-60.0	---"---	---"---	1400.000
	1800.000	30.00 Minutes	-54.0	32.9	-1.3	955.000
	---"----	---"--- ---"---	***-55.0***	—"—	—"—	***1040.000***
	---"----	---"--- ---"---	-56.0	---"---	---"---	1130.000
TS 4	1800.000	30.00 Minutes	-30.0	40.7	-2.5	818.000
	---"----	---"--- ---"---	-40.0	---"---	---"---	1730.000
	1800.000	30.00 Minutes	-31.0	40.7	-2.5	937.000
	---"----	---"--- ---"---	***-32.0***	—"—	—"—	***1050.000***
TS 5	1800.000	30.00 Minutes	-50.0	48.9	-.1	727.000
	---"----	---"--- ---"---	-60.0	---"---	---"---	1430.000
	1800.000	30.00 Minutes	-53.0	48.9	-.1	927.000
	---"----	---"--- ---"---	***-54.0***	—"—	—"—	***1010.000***
	---"----	---"--- ---"---	-55.0	---"---	---"---	1090.000
TS 6	1800.000	30.00 Minutes	-40.0	37.9	-1.6	603.000
	---"----	---"--- ---"---	-50.0	---"---	---"---	1360.000
	1800.000	30.00 Minutes	-45.0	37.9	-1.6	935.000
	---"----	---"--- ---"---	***-46.0***	—"—	—"—	***1010.000***
	---"----	---"--- ---"---	-47.0	---"---	---"---	1090.000
TS 7	1800.000	30.00 Minutes	-50.0	35.6	-1.3	853.000
	---"----	---"--- ---"---	-60.0	---"---	---"---	1570.000
	1800.000	30.00 Minutes	-51.0	35.6	-1.3	931.000
	---"----	---"--- ---"---	***-52.0***	—"—	—"—	***1020.000***
	---"----	---"--- ---"---	-53.0	---"---	---"---	1110.000

V.3 The Method Used in CAMA

BANDS-PC is used to calculate asphalt stiffness.

INPUT:

1) Loading time: 7200 seconds,
2) Temperature: 0, -10, -20, -30, and -40°C,

For Ste. Anne test road:

3) Asphalt properties: penetration at 25°C and penetration at 4 or 5°C,
4) Mix properties: mix composition as shown in Table 5.3,
   Tensile strength as shown in Fig.V.1.
   Contraction coefficient $\alpha = 2.04\times10^{-5}/°C = 1.13 \times10^{-5}/°F$.

   Assume cubic thermal coefficient of asphalt $Cb = 4.07\times10^{-4}/°C = 2.26\times10^{-4}/°F$
   The cubic thermal coefficient of aggregates $Cg = (300 * \alpha - Vb * Cb) / Vg$
   (Jones et al. 1968). The values of Vb and Vg are given in Table 5.3.

For Lamont test road:

3) Asphalt properties: PI (penetration index) and T_{800} (temperature at which penetration is 800 dmm) based on the regression of penetrations at 25, 10, and 5°C.
4) Mix properties: mix composition as shown in Table 5.4
   Tensile strength (plots of failure stress vs. temperature) obtained from indirect tensile tests given in (Wang et al., 1992).
   Contraction coefficient: Default values suggested in CAMA:

   Cubic thermal coefficient of asphalt, input value $= 2.2\times10^{-4}$ /°C $= 1.2\times10^{-4}$ /°F
   Cubic thermal coefficient of aggregates, input value $= 2.7\times10^{-5}$ /°C $= 1.5\times10^{-5}$ /°F.

OUTPUT:

Fig.V.3 gives an example output of the program CAMA for the asphalt mix used in test section No.1 of Lamont Test Road.

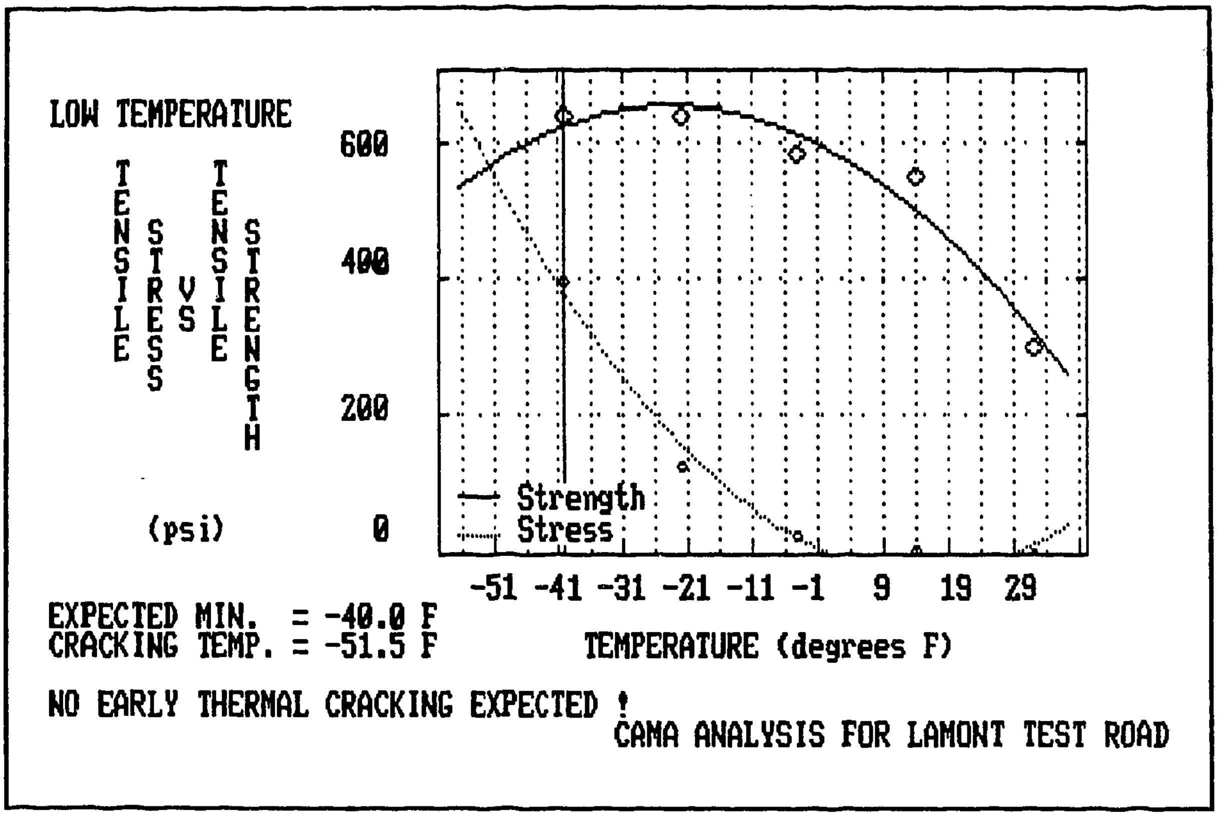

Fig.V.3-An Example Output of the Program CAMA

V.4 Robertson's Method

INPUT:

For Ste. Anne test road:

1) Asphalt properties: penetration at 25°C and PI (penetration index) based on the penetration at 25°C and penetration at 4 or 5°C,

For Lamont test road:

1) Asphalt properties: penetration at 25°C and PI (penetration index) based on the regression of penetration at 25, 10, and 5°C.

The rational design Chart is shown in Fig.V.4.

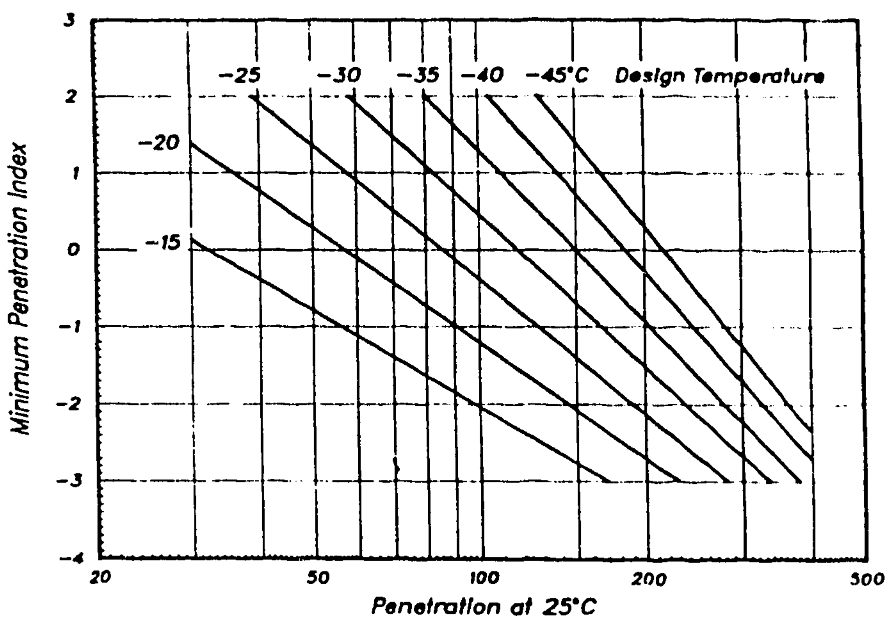

Fig.V.4-Rational Design Chart Developed by Robertson