THE UNIVERSITY OF ALBERTA

MATERIALS CHARACTERIZATION OF RECYCLED ASPHALT CONCRETE PAVEMENTS

by

KHASHAYAR HADIPOUR

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Edmonton, Alberta
Canada T6H 5R7
403/438-1555 Telex 037-2147

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....

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....

....

....

....

....

External Examiner

Date ..Dec. 19/86..........

This dissertation is dedicated

to my beloved parents for

their wisdom and devotion

ABSTRACT

Asphalt concrete pavement recycling is a growing technology. The increasing interest in recycling has created a need to fully investigate the physical properties of the recycled materials and to categorize their characteristics. The purpose of this study is to develop a data bank system for recycled asphalt concrete pavements and to evaluate the detailed engineering properties of recycled materials by means of an extensive laboratory investigation.

The study firstly concentrates on developing a data bank system for recycled pavements. The data bank is intended to objectively identify and organize the past and present records, concerning the material characteristics, design, construction, cost, maintenance, and performance of recycled asphalt concrete projects. The data bank system assists in the long term study of recycled pavements performance.

The primary goal of this study is to evaluate the detailed engineering properties of the recycled asphalt concrete materials and to compare these properties with those of the conventional materials. To achieve this, a testing program is designed, necessary experimental equipment is developed, and a series of tests are conducted on various binders and mixtures. The Marshall Stability, durability, nomograph stiffness, low temperature tensile properties, permanent deformation, and resilient modulus of various

recycled and conventional mixtures are determined. There are very extensive low temperature cracking and permanent deformation in asphalt concrete pavements in Western Canada, as a result of extreme climatic conditions. In this study, more emphasis is placed on low temperature cracking and permanent deformation phenomena.

The results of the laboratory experiments are utilized to develop statistically based prediction models. These models can be employed in predicting the low temperature tensile properties, permanent deformation and resilient modulus of any mixture with similar characteristics to those tested in this study, under different loading and environmental conditions.

The principal findings of this study are that the recycled asphalt concrete materials possess higher stability, durability, and stiffness than conventional materials. They are also superior in terms of their ability to resist permanent deformation. The recycled asphalt concrete mixtures, are, however, more susceptible to low temperature cracking. Nevertheless, with proper mixture design the cracking potential can be eliminated.

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1. INTRODUCTION

1.1 Introductory Remarks

The depletion of quality aggregate, the increasing cost of asphalt cement and the growth in the cost of energy have encouraged an increase in the reuse of asphalt concrete pavement materials or recycling. Cost savings, energy savings, and the conservation of natural resources are not the only potential benefits from recycling. Other benefits may include correcting the existing mix deficiencies, minimizing or eliminating reflective cracking problems, correcting or eliminating surface deformations, increasing the structural capability of the pavement without increasing its thickness, and many more. In recent years, recycling has proven to be economically feasible and could be considered as an alternative to conventional pavement rehabilitation techniques.

The increasing number of recycling projects, the abundant number of parameters involved in each individual project, and consequently, the enormous sum of data has led to a need for a comprehensive computerized data bank system. The aim of a data bank is to collect and classify past and present information regarding the material characteristics, design, construction, maintenance, and performance of recycled asphalt concrete projects. This will assist in the long term study of recycled pavement performance.

A data bank system for recycling can be utilized for future improvements in design and construction. It may be used to develop guidelines in selecting the best alternative, and it can be useful in cost allocation and investment decisions.

With the increasing interest in the recycling of asphalt concrete pavements and the growing number of recycling projects, a need also became evident for material evaluation techniques to properly design the recycled mixtures to resist various modes of pavement distress.

In general, pavement distress can be categorized into the following three main modes: load associated fractures i.e. fatigue, non-load associated thermal cracking, and permanent deformation.

Load associated fatigue occurs when pavement is subjected to a large number of repeated loads over a period of time. It is a serious mode of distress and should be taken into consideration at design stages. Models exist that can use the measured properties of asphalt concrete mixtures to accurately predict their fatigue resistance.

Non-load associated thermal cracking or, in other words, the low temperature transverse cracking due to thermal effects is also a serious and costly pavement distress mode in many locations of relatively cold climates. Low temperature cracking, which is primarily caused by low winter temperatures that induce tensile forces in the asphalt concrete exceeding the tensile strength of the material, is a widespread problem in many parts of the world. Asphalt concrete pavement should be designed with adequate tensile strength to resist this mode of distress.

Permanent deformation or rutting is another major mode of pavement distress. It forms as longitudinal depressions in the wheel paths due to consolidation and/or lateral movement in component pavement layers as a result of repeated load applications. Excessive permanent deformation results in a lack of pavement serviceability and safety. It is a severe distress mode and should be included in any rational pavement design methodology.

With regard to conventional asphalt concrete pavements, design techniques have been developed which take into account the aforementioned three modes of distress. However, little effort has been made to develop similar design techniques for recycled asphalt concrete pavements.

The low temperature cracking and the permanent deformation modes of distress are more apparent in western Canada due to the existence of the extreme temperature regime. Hence, the development of design criteria with regard to these two modes of distress for the recycled asphalt concrete pavements is of major concern.

Recycled pavements are very complex systems, involving the interaction of many variables. Their performance is influenced by a variety of factors such as material properties, load, environmental condition and construction. In order to properly design the recycled asphalt concrete pavement to insure against various modes of distress, the detailed physical properties of the constituting materials should be evaluated under various loading and environmental conditions.

1.2 Objectives and Scope of the Investigation

The primary objectives of this study are:

1. to develop a comprehensive computerized data bank system to assist in the design, construction, cost, maintenance and performance of recycled asphalt concrete pavement projects. The data bank is to be developed in a manner such that it provides the means by which data on past, present and future recycling projects can be entered, stored, manipulated, and retrieved on a continuing basis. Various analyses and report generation are another tasks of the data bank system. Ease of use, completeness, concisiveness and cost effectiveness are to be the major characteristics of this system;

2. to evaluate various properties of the virgin and reclaimed binders individually and in several combinations. These properties include: penetration, absolute viscosity, kinematic viscosity, penetration-viscosity number, and nomograph stiffness, all to be measured before and after the Thin-Film Oven test. Durability characteristics of the binders are also to be

determined;

3. to evaluate and to compare the detailed physical properties of several recycled and conventional asphalt concrete mixtures. These properties include: stability, nomograph stiffness, durability, resilient modulus, low temperature tensile properties, and permanent deformation. Due to the extensive existence of low temperature cracking and permanent deformation in pavements in Western Canada as a result of extreme climatic conditions, more emphasis is to be placed on these two phenomena;

4. to develop analytical models, based on experimental data, for predicting resilient modulus, low temperature characteristics, and permanent deformation of recycled and conventional asphalt concrete mixtures; and

5. to present practical recommendations for the design of recycled asphalt concrete mixtures to insure against low temperature cracking and permanent deformation.

The data bank system developed in this study is designed for recycled asphalt concrete pavement projects. It may also be used for conventional pavements.

The experimental program undertaken in this investigation is limited to the testing of laboratory prepared specimens. These specimens are fabricated using virgin and reclaimed asphalt concrete materials supplied by a demonstration recycling project in the province of Alberta.

This study included the low temperature cracking and permanent deformation modes of pavement distress. Fatigue mode of distress is not, however, considered in this investigation.

1.3 Organization of the Thesis

This study is divided into 9 chapters and 4 appendices.

The introduction, along with the objectives, scope, and organization of the thesis are given in Chapter 1.

In Chapter 2 the development of the computerized recycling data bank system is discussed. Appendix A gives a detailed description of the data bank structure and documentations. An example of a recycling project data bank output is also presented in Appendix A.

Chapter 3 describes the basic properties of the asphalt blends, aggregate, and asphalt concrete mixtures used in this investigation. The detailed Marshall mixture design properties of the conventional and the five recycled mixtures are given in Appendix B.

Durability characteristics of the virgin, reclaimed, and recycled blends are described in Chapter 4. This chapter also contains the durability characteristics of the conventional and the five recycled mixtures.

Stiffness values of the asphalt blends and asphalt concrete mixtures, calculated using Van der Poel's nomograph, are presented in Chapter 5.

Chapter 6 is entirely devoted to low temperature cracking criteria. It contains a comprehensive literature review on low temperature cracking mechanisms, the existing design approaches, and the experimental program undertaken in this investigation. The tensile splitting test results and their analysis together with the development of the low temperature prediction models are also given in Chapter 6. Descriptions of the tensile splitting test apparatus and procedures along with the data acquisition system and computer programs are given in depth in Appendix C.

Pavement permanent deformation phenomenon is described in detail in Chapter 7. This chapter includes an extensive literature review on permanent deformation mechanism, the current design approaches, and the experimental program conducted in this study. The repeated loading triaxial test results and their analysis together with the development of the resilient modulus and permanent deformation prediction models are also contained in Chapter 7. The detailed description of the repeated loading triaxial test and the computerized data acquisition system is given in Appendix D. Details of the computer programs for the triaxial test data collection and analysis are also shown in this appendix.

Chapter 8 presents the summary and the major conclusions of this investigation. Recommendations drawn from the findings of this study are given in Chapter 9.

2. RECYCLING DATA BANK SYSTEM

2.1 Introduction

Recycling of asphalt concrete pavements is a developing technology, which results in cost saving, energy saving, conservation of natural resources, and preservation of environment and existing roadway geometries. Recycling technology is growing and experience has been gained through many recycling projects world wide.

Recycling operations started in the province of Alberta in 1982 and the number of recycling projects is growing every year. By the end of 1985 eleven projects had been completed with an enormous amount of data collected. Reports are available on the design and construction of some of the completed projects (1-5).

The increasing number of recycling projects, the numerous combination of variables and consequently, the vast amount of data has led to the need for a comprehensive data bank system. This phase of the study which was initiated in July 1984 addresses this need.

The development of a comprehensive data bank of information is intended to objectively identify and organize the present and past records concerning the design, construction, maintenance and performance of these recycled pavements. This will assist in the study of long term pavement performance as information is collected by future continuous monitoring of pavement deterioration due to factors attributable to traffic and environment.

The available data in such a data bank could be analyzed and used for improving implementation techniques, as a guideline for selecting the best alternative, cost allocation, investment decisions, and pavement design.

The performance of the recycling projects has to be monitored in order to validate or verify the design methods. Continuous gathering of sufficient information helps to optimize the performance of future recycling projects. The performance and condition survey data could be used along with the design information, construction details, and test results to optimize the selection of construction techniques and materials for recycling asphalt concrete pavements.

One can not benefit from one's own mistakes due to lack of a data base of information. Without having a data bank, the ability to compare design and construction strategies and cost effectiveness is limited.

The data bank should accumulate reasonable periods of in-service performance information and contain adequate and reliable data. The question of what types of data to collect is an important consideration in planning the data bank system. It is apparent that consideration should be given to reliable data on pavement design and construction procedures, cost, maintenance, and performance. Such data are needed to improve the quality of planning, research, and programming decisions for future projects.

It is necessary to organize the data bank in some logical manner so that it can be readily followed with minimal confusion and without duplication of identical items. Meaningful information is an objective of high priority. Special consideration should be given to the nature and amount of data to be collected in order to have a comprehensive and yet manageable data bank.

2.2 Development of the Alberta Transportation Data Bank

In developing the computerized data bank system an extensive literature review has been conducted, (6-9) documentation and reports on many recycling projects have been studied, and consultation has been made through arranging interviews with Alberta Transportation and Alberta Research Council personnel.

A number of interviews were also made with U.S. Federal Highway Administration (FHWA) personnel in order to benefit from their experience in the development of the data bank system. FHWA had sponsored a three year research contract to design and establish a computerized data bank system for recycled asphalt concrete projects in 1979. This task was accomplished by 1982. A detailed questionnaire was designed and information on many projects throughout United States was collected and analyzed (10-12). However, the questionnaire was rather lengthy and difficult to complete. Consequently a modified version is now being used to collect detailed information about hot recycled asphalt pavements. By 1985, the Asphalt Institute had completed a study of about 300 projects contained in the FHWA data bank for recycled asphalt concrete pavements. The Asphalt Institute has worked on a study that will use the collected information to help optimize the performance of future recycling projects (14-16).

Valuable experience was gained by consultation and reviewing publications from FHWA, which was utilized in the development of the Alberta Transportation data bank system.

An initial part of this study considered the problems in building a data bank that could be used to improve the existing design procedures, quality of construction techniques, and performance evaluation; and also accommodate Alberta Transportation needs as well. The existing pavement management system (17) does not provide adequate information on design, materials and construction in order to meet these needs.

It was decided to use a general data base management system to construct the recycling data bank. The advantage of this approach is that a data management system provides built-in capabilities for alteration of the data structure and updating and retrieval of information. A computerized management system should suit its users' needs. Ease of use, completeness, timeliness, and cost effectiveness are major characteristics that a data base system should possess.

The data base is to be developed in a manner such that it provides the means by which data on past, present and future recycling projects can be entered, stored, manipulated, and retrieved on a continuing basis. Various analysis and report generation is another task of the data base system.

A data base management system called SPIRES, an abbreviation for the Stanford Public Information Retrieval System was chosen after a review of the new existing data base systems including DMS and ADRSII.

SPIRES is a generalized on-line data base management system developed at Stanford University in the early 1970's. It is designed to handle all types of data efficiently, from compact, numerical values found in administrative and scientific data to lengthy, textual values such as bibliographic data.

SPIRES users design and maintain their own data bases, and consequently there is no centralized data base administrator. SPIRES allows one to create, own and manage one's own collection of data, known as data bases. It allows one to store, retrieve, search, manipulate, analyze, display and print data the way the user desires. For more information on SPIRES one may refer to SPIRES data development, SPIRES file definer, and SPIRES file definition manuals (18-20).

In order to meet an objective of this study a data base management program has been written in SPIRES for Recycled Asphalt Concrete Pavements (RACP).

2.3 Organization of the Data Bank

The data bank consists of 9 sections and each section is divided into several subsections with various parts as listed below:

1. PROJECT DESCRIPTION
   - 1.1 Title
   - 1.2 Location
   - 1.3 Contract quantities
   - 1.4 Rehabilitation method
   - 1.5 Agency
   - 1.6 Contract No.
   - 1.7 Contractor
   - 1.8 Date

2. EXISTING PAVEMENT PRIOR TO RECYCLING
   - 2.1 Environmental condition
   - 2.2 Soils
   - 2.3 Traffic condition
   - 2.4 Geometric design
   - 2.5 Structural design
   - 2.6 Mixture design
   - 2.7 Material properties as constructed
     - 2.7.1 Asphalt properties
     - 2.7.2 Aggregate properties

3. REHABILITATION CONSIDERATIONS
   - 3.1 Existing pavement distress condition
   - 3.2 Primary reasons for rehabilitation
   - 3.3 Probable cause of distress
   - 3.4 Rehabilitation technique chosen

4. DESIGN OF REHABILITATED PAVEMENT
   - 4.1 Geometric design
   - 4.2 Structural design
   - 4.3 Mixture design
     - 4.3.1 Reclaimed asphalt properties
     - 4.3.2 Reclaimed aggregate properties
     - 4.3.3 Virgin asphalt properties
     - 4.3.4 Recycling agent properties
     - 4.3.5 Virgin aggregate properties
     - 4.3.6 Laboratory blended binder properties
     - 4.3.7 Laboratory blended aggregate properties

5. CONSTRUCTION SUMMARY
   - 5.1 Reclaiming operation
   - 5.2 Asphalt plant operation
   - 5.3 Emission tests
   - 5.4 Spreading and compaction operation
   - 5.5 Production summary
   - 5.6 Quality control

5.6.1 Density summary
5.6.2 Binder content summary
5.6.3 Binder properties summary
5.6.4 Aggregate properties
5.6.4.1 Virgin aggregate
5.6.4.2 Reclaimed aggregate
5.6.4.3 Recycled aggregate
5.6.5 Temperature summary

6. COST
6.1 Actual cost of recycled project
6.2 Estimated cost of other rehabilitation alternatives
6.3 Comparison and savings

7. RECYCLED PAVEMENT EVALUATION
7.1 Evaluation of pavement surface condition
7.2 Evaluation of pavement serviceability
7.3 Evaluation of pavement safety
7.4 Evaluation of pavement structural capacity
7.5 Evaluation of pavement material properties using core data

8. DETAILED PHYSICAL PROPERTIES OF RECYCLED MIXTURES
8.1 Stiffness by Van der Poel normograph
8.2 Resilient modulus
8.3 Fatigue behaviour
8.4 Permanent deformation
8.5 Low temperature cracking
8.6 Tensile properties
8.7 Water susceptibility

9. MAINTENANCE AND REHABILITATION RECORDS

The following is a brief description of different sections of the data bank structure:

Section 1 - <u>Project Description</u>: This section provides a set of project identification data such as title, quantities, contract number, and contractors. It also identifies where the project is located and under which district jurisdiction it falls. A brief description of the rehabilitation method is also included in this section.

Section 2 - <u>Existing Pavement Prior to Recycling</u>: Generally, this section gives a set of historical data on the pavement under consideration, prior to recycling. This information contains subgrade soil properties, traffic conditions, geometric, structural and mixture design. The properties of the materials such as asphalt and aggregate, which has been used in the existing pavement construction is described. This section also provides information regarding the environmental condition of the region where the pavement is located.

Section 3 - Rehabilitation Considerations: A summary of the distress condition of the existing pavement and its performance is given in this section. Reference can be made to the Alberta Pavement Management System, for more details regarding available performance information.

Probable causes of pavement distress such as structural inadequacy, mixture instability, excessive traffic volume and load is considered. The factors which contribute to rehabilitation decision-making such as extensive cracking or rut depth, low skid number, and low pavement quality index are given in this section. Consequently the rehabilitation technique chosen for the project is discussed.

Section 4 - Design of Rehabilitated Pavement: This section includes a set of information on the geometric, structural, and mixture design of the new pavement. Marshall mixture design characteristics of various Reclaimed to Virgin material ratios (R/V) as well as the particular ratio chosen for the project are provided. Virgin, reclaimed, and laboratory blended asphalt and aggregate properties are also given in detail.

Section 5 - Construction Summary: A summary of the construction techniques undertaken for the project is provided in this section. It includes the reclaiming, remixing, spreading, compacting, and in general, recycling operations. Emission measurements are also given.

Construction quality control data including density, asphalt and aggregate properties, and temperature summaries are given in detail for each single unit, and consequently statistical significances are drawn from the recorded data.

Section 6 - Costs: This section is filled out essentially with information on the cost of the recycled asphalt concrete pavement. Unit prices and material costs are included as well. Cost of other rehabilitation alternatives such as conventional asphalt concrete overlay is estimated and compared to that of recycled pavement.

Section 7 - Recycled Pavement Evaluation: Information is provided in this section on performance of the recycled asphalt concrete pavement. Data on surface condition, serviceability, safety, and structural capacity of the recycled pavement are included. Material properties using core data are also identified.

Section 8 - Detailed Physical Properties of Recycled Mixtures: An extensive testing program is developed for the recycled mixtures in order to identify behavioral and physical properties. These tests include stability, water susceptibility, permanent deformation, tensile, and fatigue testing. This section covers all data obtained in the testing. Material properties such as stiffness, resilient modulus, and tensile properties are also evaluated.

Section 9 - Maintenance and Rehabilitation Records: In this section time, type and cost of the succeedings maintenance and rehabilitation activities on the recycled asphalt concrete pavement are recorded.

2.4 Current Status of the Data Bank

At this stage of development, the data bank is ready to accept information to be input (21). Detailed information required to enter into the data bank is outlined in Appendix A1. A complete set of data is desirable for all recycling projects completed to this time. However, depending on availability, partial information pertinent to any particular project may be input into the data bank.

Reports are available for the first few recycling projects in Alberta and information can be directly transferred from these reports into the data bank. For more recent and also for future recycling projects, where comprehensive reports might not be available, information should be gathered from various sources to fully or partially complete all separate sections of the data bank.

The detailed procedure for inputting data is outlined in Appendix A2. The documentation provided is basically to assist in entering the data into the SPIRES system. A set of commands and procedures has been designed for RACP data. A

MENU record is provided so that the $RUN *SPIRES is the only command needed to initiate the program, once the user has signed on to the University of Alberta MTS System.

For checking the system, data from one of the earlier recycling projects in Alberta has been entered into the RACP data bank. The output from this example project Hwy 2:18 & 20 is given in Appendix A3. It is intended that the recycling data bank be transformed from SPIRES data base management system to a system called FOCUS, by Alberta Transportation Personnel, at a later stage.

2.5 Summary

With the increasing number of recycling projects in Alberta the need for a comprehensive data bank system was the justification for this phase of the study.

A comprehensive system for collection of detailed information regarding recycled asphalt pavement projects has been developed. The organization of this data bank information is the basis for the computerized data base management system.

The Stanford Public Information Retrieval System (SPIRES) was chosen as a suitable data base management for the recycling data bank system. Detailed procedures for storage and retrieval of recycled asphalt concrete pavement data are developed.

The available data from the completed recycling projects have to be collected and stored into the data bank system on a continuing basis. The performance of the data bank system has to be monitored periodically, and the system has to be implemented as required.

3. EXPERIMENTAL PROGRAM AND MATERIALS

3.1 Introduction

The primary objective of the testing program involved in this investigation was to evaluate the physical properties of the recycled asphalt concrete mixtures and to compare these properties with those of the conventional mixtures. For this purpose a test road project was selected and the properties of the recovered asphalt and aggregate from the materials reclaimed from the test road project were evaluated. After the material evaluation, a series of mixture design was carried out and samples were prepared for the proposed testing program.

In this chapter a brief description of the tests involved in this study is presented. However, the individual tests will be fully discussed in the succeeding chapters. Also a detailed description of the project selection, mixture design, and sample preparation is accommodated in this chapter.

3.2 Testing Program

In order to determine the physical characteristics of the recycled asphalt concrete mixtures, a series of tests had to be conducted on the asphalt binders and the recycled mixtures. These tests are individually discussed in detail in the succeeding chapters. However, without going into any depth at this stage, Tables 3.1 and 3.2 summarize the testing

TABLE 3.1

TESTS CONDUCTED ON
VIRGIN ASPHALT CEMENT, RECLAIMED ASPHALT CEMENT
AND COMBINED BINDER

Before TFOT	After TFOT
Penetration At 25 C	Penetration At 25 C
Absolute Viscosity At 60 C	Absolute Viscosity At 60 C
Kinematic Viscosity At 135 C	Kinematic Viscosity At 135 C
Specific Gravity	Weight Loss

TABLE 3.2

TESTS CONDUCTED ON VARIOUS MIXTURES
AT DIFFERENT TEMPERATURES

Tests	Temperatures °C
Marshall Stability	+60
Retained Stability (After 24-hrs Water Immersion)	+60
Indirect Tensile (Tensile Splitting Test)	-10, -20, -30
Resilient Modulus (Triaxial Test)	+25, +35, +45
Permanent Deformation (Triaxial Test)	+25, +35, +45

plan undertaken on the binders and various mixes in this investigation.

3.3 Project Selection

A test road project was selected in order to provide the aged asphalt concrete materials for the purpose of mix design and sample fabrication. Another reason for choosing a suitable test road project was to evaluate the in-situ performance of the recycled asphalt concrete pavements as compared to the predictions based on the laboratory test results. The in-situ performance of the recycled pavement has to be monitored on a regular basis in order to give an indication of the behaviour of the recycled materials on the road, and also be compatible with the laboratory prediction results.

After conducting an investigation on many ongoing recycling projects in the province of Alberta in 1985, it was decided that a project on highway 41:20 located near Vermilion would be appropriate for the purpose of this study. This highway was originally constructed in 1963. The cross section prior to recycling consisted of 130 mm cement stabilized base course, 60 mm asphalt stabilized base course, and 140 mm asphalt concrete pavement. The asphalt concrete pavement was reclaimed and stockpiled in 1985, after 22 years in service.

The new construction of this project was undertaken by Alberta Transportation in the summer of 1986. It is

12 kilometers long and has a structure consisting of 150 mm recycled asphalt concrete underlain by 100 mm of granular base course.

About one tonne of reclaimed asphalt concrete and virgin aggregate was sampled and transported from the stockpile to the laboratory. After evaluating their properties, these materials were then used for conducting a series of mix designs and also for fabricating samples for the testing program.

3.4 Materials Evaluation

After collection of the reclaimed and virgin materials, their properties had to be evaluated. In order to accomplish this task, reclaimed asphalt and aggregate had first to be separated and their characteristics had then to be evaluated. The following sections describe the properties of the binder and the aggregate used in this project.

3.4.1 Asphalt Binder Properties

To evaluate the characteristics of the reclaimed asphalt cement, extraction and recovery of the asphalt from the reclaimed asphalt concrete pavement has been carried out. The Reflux method (ASTM D2172-81,B) was used for quantitative extraction of the asphalt. The extracted asphalt was then recovered from the solution by the Abson method (ASTM D1856). The consistency characteristics of the recovered asphalt such as, penetration at 25°C, absolute viscosity at

60°C, and kinematic viscosity at 135°C were then measured. These measurements were also repeated after the Thin-Film Oven test (TFOT).

In order to rejuvenate the aged asphalt to restore its original properties which had suffered under the attack of many factors such as temperature, water, and air, it had to be blended with a recycling modifier or a softer asphalt.

The recovered asphalt exhibited a penetration of 45 dmm, and an absolute viscosity of 793 Pa.s. This indicates that a softer asphalt cement is required for blending with the reclaimed asphalt to modify its properties for recycling. The virgin asphalt cement selected for this study was a 300-400A penetration grade asphalt. The recovered asphalt cement was blended with the virgin asphalt at various "reclaimed to virgin asphalt ratios" (r/v). A wide range of r/v ratios such as 0/100, 30/70, 50/50, 70/30, and 100/0 were chosen for this study. A blend was also prepared with the recovered asphalt cement and SC-3000 cut back at 50/50 ratio. All these blends were then tested for penetration at 25°C, absolute viscosity at 60°C, and kinematic viscosity at 135°C before and after the Thin-Film Oven test. Tables 3.3 and 3.4 show the reclaimed asphalt, virgin asphalt, and blends properties before and after the Thin-Film Oven test. The value of r represents the percentage of the reclaimed asphalt cement in the blend. The value of r equals zero represents the pure virgin asphalt and r value equals 100 is representative of the pure reclaimed asphalt cement. An

TABLE 3.3

BINDER PROPERTIES BEFORE THE THIN-FILM OVEN TEST

r %	Penetration At 25 C dmm	Absolute Viscosity Pa.s	Kinematic Viscosity mm^2/s	P.V.N.
0	321	34.0	180	-0.03
30	169	73.5	264	-0.24
50	108	156	355	-0.32
70	69	345	477	-0.38
100	45	793	649	-0.40
50 SC3000	272	48.2	180	-0.28

TABLE 3.4

BINDER PROPERTIES AFTER THE THIN-FILM OVEN TEST

r. %	Penetration At 25 C dmm	Absolute Viscosity Pa.s	Kinematic Viscosity mm^2/s	P.V.N.
0	154	96.1	296	-0.17
30	99	195	408	-0.20
50	69	378	543	-0.17
70	49	799	741	-0.13
100	33	1690	996	-0.15
50 SC3000	142	102	288	-0.32

approximately linear relationship was found to exist between the logarithm of the blend properties and the r value. It was discovered that penetration decreases and viscosity increases linearly as the percentage of the reclaimed asphalt cement in the blend (r) increases. This will be discussed in detail in Chapter 4. The properties of the recovered asphalt from the specimens tested for tensile splitting and triaxial test are shown in Table 3.5.

3.4.2 Aggregate Properties

After recovery of the aggregate from the reclaimed asphalt concrete, its properties had to be evaluated. A sieve analysis was performed on the aggregate to determine its gradation. Bulk specific gravity and asphalt absorption of the reclaimed aggregate were also determined and yielded values of 2.648 and 0.28 percent respectively. After the verification of its usage in the recycled asphalt concrete mixture, the reclaimed aggregate was blended with the virgin aggregate of known properties at various ratios. Table 3.6 represents the reclaimed and virgin aggregate gradation. The virgin aggregate was separated into coarse, fine, and sand.

3.5 Mixture Design

The design of recycled asphalt concrete mixes is relatively more complicated than the conventional mixes. The initial step in design of recycled mixes is to evaluate the reclaimed material properties. The next step is to select

TABLE 3.5

PROPERTIES OF RECOVERED ASPHALT FROM LABORATORY SPECIMENS AFTER TESTING

r %	Penetration At 25 C dmm	Absolute Viscosity Pa.s	Kinematic Viscosity mm^2/s	P.V.N.
0	123	181	377	-0.06
35	92	273	503	+0.03
56	46	1200	1010	+0.21
73	44	1610	1200	+0.41
92	35	3600	1800	+0.67
56 SC3000	82	393	681	+0.34

TABLE 3.6

VIRGIN AND RECLAIMED AGGREGATE GRADATION

Aggregate Gradation (sieve Size) µm	Virgin Aggregate (Percent Passing)			Reclaimed Aggregate (Percent Passing)
	Coarse	Fine	Sand	
16 000	100			99
12 500	82			96
10 000	55	100		91
5 000	9	84		74
1 250	4	34	100	50
630	4	26	99	39
315	3	19	96	29
160	2.4	13.5	51.0	21.6
80	1.7	9.4	17.3	16.3

the proper grade of virgin binder or modifier and virgin aggregate suitable for the chosen recycling ratio. The final step is to perform the Marshall stability test and to evaluate the recycled mix properties for the optimum binder content. A flowchart representing the procedures for the recycled asphalt concrete mixture design (2) is illustrated in Figure 3.1.

In this investigation several mixes were prepared in order to examine the effect of recycling ratio on the mix properties, and to optimize the usage of the reclaimed materials. Trial mix designs were carried out at various "reclaimed to virgin material ratios" (R/V) of 0/100, 30/70, 50/50, 70/30, and 100/0. The value of R represents the percentage of the reclaimed material in the mix. After performing the Marshall stability test, the optimum asphalt content and hence the percentage of the virgin binder for each individual mix was determined. Table 3.7 exhibits the identification of the different mixes, including the percentage of the reclaimed asphalt concrete in the mixture, the percentage of the reclaimed asphalt cement in the binder, the virgin asphalt grade, the asphalt content, and the aggregate proportion.

A unified target gradation was chosen for all the recycling mixtures in order to reduce the number of variables in the experimental plan. The virgin aggregate gradation was therefore adjusted for each individual mix in order to keep the gradation as close as possible for all mixes having

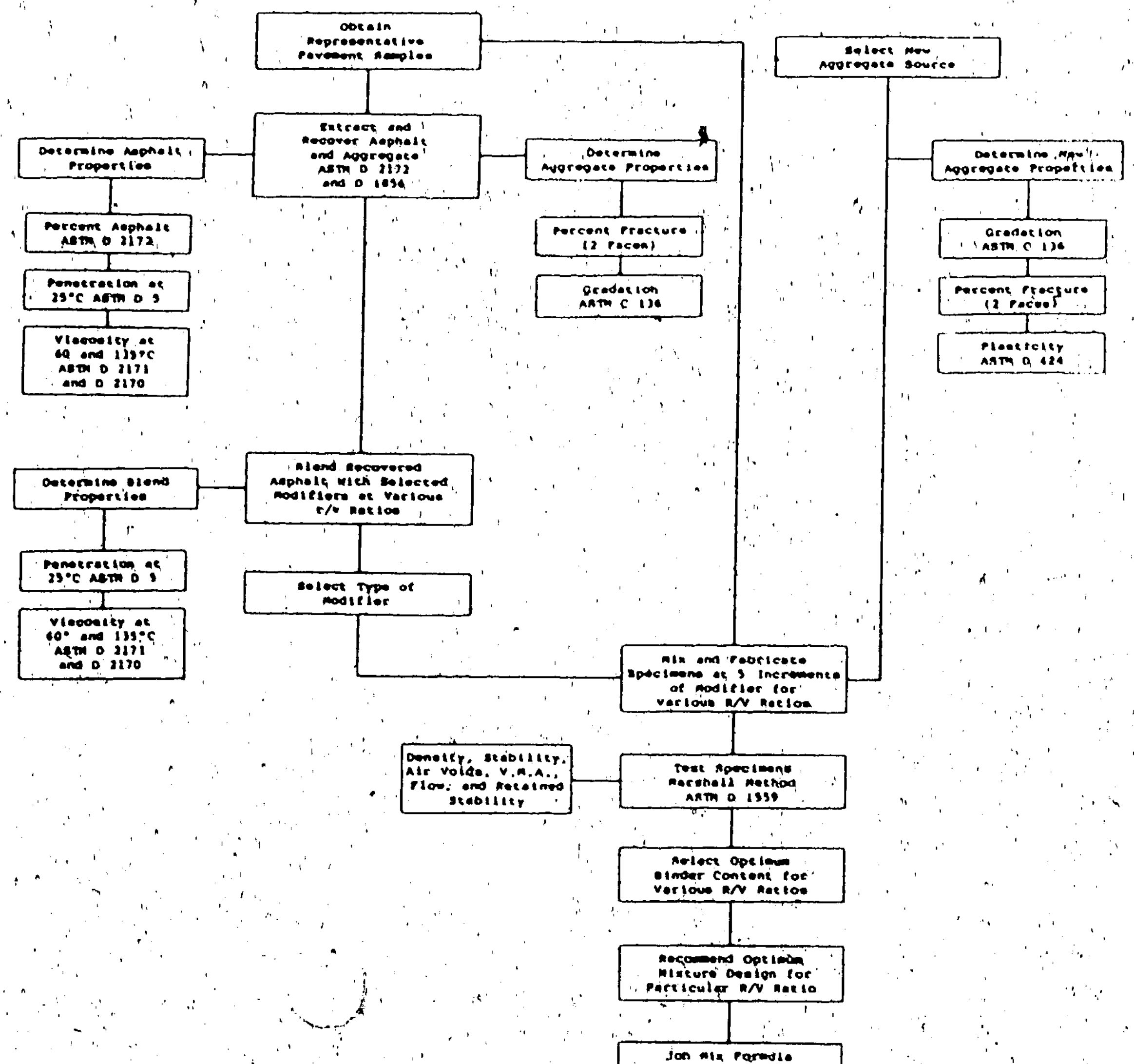


Figure 3.1 PROCEDURES FOR THE RECYCLED ASPHALT CONCRETE MIXTURE DESIGN. (After Ref. 2)

TABLE 3.7

MIXTURES IDENTIFICATION

R %	r %	Virgin Asphalt Grade	Total Asphalt Content	Aggregate Proportion r/c/f/s[1]
0	0	200-300 A	5.9	0/37/45/18
30	35	300-400 A	4.5	30/30/35/0
50	56	300-400 A	4.8	50/25/25/0
70	73	300-400 A	5.2	70/30/0/0
100	92	SC-3000	6.0	100/0/0/0
50	56	SC-3000	4.8	50/25/25/0

[1] r/c/f/s represents reclaimed/course/fine/sand

different recycling ratios. However, due to the high fines content of the reclaimed materials, some minor differences in gradation may still be observed. Table 3.8 gives the aggregate gradation of different mixes used in this study.

A 300-400A virgin asphalt was used for all the mixes with the exception of the conventional mix, for which a 200-300A virgin asphalt was used. The aim was to keep the binder consistency in all mixes as similar as possible.

A Marshall mix design was also carried out for an R/V ratio of 50/50 with SC-3000 as a recycling modifier. Using the two mixes at an R/V ratio of 50/50 with different binder characteristics will provide the means for evaluating the effectiveness of a recycling modifier.

In order to cut down the number of variables in the experimental program, it was also decided to keep the air voids in all various mixes at a 4 percent target level and to adjust the mix constituents accordingly.

The complete mix design summary for all the mixes including the Marshall stability results, design curves, and the aggregate gradation charts is represented in Appendix B.

3.6 Sample Preparation

After the completion of the mix design for all different mixes, samples had to be prepared for the proposed testing program. Seventy-two Marshall briquettes having a 64 mm (2.5 inches) thickness and a 102 mm (4 inches) diameter were fabricated for the tensile splitting test. These

TABLE 3.8

AGGREGATE GRADATION FOR VARIOUS MIXTURES

Aggregate Gradation (sieve size) µm	Reclaimed Material in the Mix, (R) %					
	0	30	50	70	100	50 SC3000
	Percent Passing					
16 000	100	100	100	99	99	99
12 500	94	94	94	96	96	94
10 000	83	84	84	80	91	84
5 000	59	59	60	55	74	60
1 250	35	33	35	36	50	35
630	31	27	27	29	39	27
315	27	21	20	21	29	20
160	16.1	14.5	14.8	15.8	21.6	14.8
80	8.0	9.6	10.9	11.9	16.3	10.9

specimens, all with various R/V ratios, were tested at three different temperatures and the failure tensile stress and strain and consequently the stiffness modulus of the material were determined.

Specimens also had to be prepared for the repeated loading triaxial test in order to obtain the resilient modulus and the permanent deformation characteristics of the various recycled materials at different temperatures. Seventy-two long samples having a 204 mm (8 inches) thickness and a 102 mm (4 inches) diameter were fabricated.

Split moulds with a 204 mm (8 inches) height, 102 mm (4 inches) diameter, and a 50 mm (2 inches) collar at the top were built specifically for fabrication of the long samples using the kneading compactor.

Trial specimens were compacted to determine the best procedure to form a specimen that would have a uniform density throughout its thickness. These samples were then cut into several lifts and the density of each lift was determined. After conducting many trials, it was found that 4080 g of mix is required to produce a 204 mm (8 inches) specimen with approximately 4 percent air voids. It was also discovered that in order to attain uniform density, it is best to compact the first two thirds of the mix with 150 blows at a compactive effort of 2.07 MPa (300 psi) and to compact the last third with 110 blows at 3.45 MPa (500 psi). It is recommended to give the sample 6 blows with a free fall hand compactor at the end for levelling.

All the specimens including Marshall briquettes and long samples were visually examined and their density, air voids, asphalt content, thickness, and diameter were determined.

3.7 Summary

The experimental plan designed for this investigation was briefly discussed in this chapter. A test road project, the performance of which would be monitored regularly, was selected. The properties of the reclaimed asphalt concrete pavement from this project and also the characteristics of the selected virgin materials were evaluated. Mixture designs were carried out for several mixes containing different recycling ratios and specimens were fabricated for the planned tests. The physical characteristics of the recycled materials will be evaluated through these tests and will be discussed in the succeeding chapters.

4. ASPHALT CEMENT AND ASPHALT CONCRETE DURABILITY

4.1 Introduction

One of the major factors influencing the service life of flexible pavements is the durability of the asphalt concrete courses. A high durability potential implies a long service life. An important component determining the life of asphalt concrete courses is the ability of the asphalt cement to resist hardening. The degree to which asphalt resists this change is an indication of its durability. Rapid hardening occurs during mixing, when both asphalt and aggregate are exposed to high temperatures. After the asphalt mix is compacted in the road, changes take place slowly over a period of years.

The following sections describe the factors involved in asphalt cement and asphalt concrete pavement durability and the test results obtained in this study.

4.2 Asphalt Cement Durability

Asphalt durability has been defined in many ways by various investigators. Hveem (22) in 1943 defined it as the ability of asphaltic materials to retain their original properties. Vallerga (23) defined asphalt durability as resistance to change in original properties, for the worse, during construction and in-service aging.

Since asphalt is used as a binder in the mixture, it is obvious that its hardening and the changes in chemical, physical and rheological properties are very important when considering pavement durability. Asphalt cement at the time of application has specified characteristics, however, destructive factors such as weathering cause degradation in the asphalt properties. It is known that changes in original properties of asphalt take place during construction and field aging. Nevertheless, suitable mix design and proper mixing and placing procedures can minimize such changes in asphalt properties.

There have been many investigations concerning the chemical and physical properties of the asphalt cement. The correlation between these properties and durability tests as well as pavement performance has been studied. Rostler et al. (24-26) have defined the individual fractional components of asphalt cement and have described the specific function of each component on durability. They developed a durability ratio based on extensive pellet abrasion testing and have found that the abrasion of aged asphalts is related to the ratio of the most reactive fractions to the least reactive fractions.

Anderson et al. (27) have reported the use of the Rostler chemical fractional component method to evaluate the durability of approximately 500 asphalts sampled in the period of 1950-1979. They used the Rostler method based on the assumption that the Rostler parameter relates the asphalt chemical composition to durability. Goodrich et al. (28), however, do not support the premise that Rostler parameters can be used as a reliable indication of durability and pavement performance. They have concluded that physical tests are more correlated to durability and performance than compositional tests. They have demonstrated that hardening of asphalt is a function of time, temperature, asphalt content, voids content, aggregate porosity, asphalt film thickness as well as type of asphalt.

4.2.1 Factors Affecting Asphalt Cement Durability

There are many factors influencing the durability of the asphalt cement. The importance of each factor depends on the individual asphalt and the environmental conditions under consideration. A detailed explanation of each element is given by Wright (29) and Traxler (30). Some major factors are very briefly discussed below:

Volatization depends on the nature of the volatile components in the asphalt. It also depends on the temperature conditions to which asphalt is exposed. Removal of the volatile components results in an alteration of the chemical composition and asphalt hardening.

Oxidation, in addition to volatization, is also responsible for hardening and loss of durability of asphalt. It proceeds slowly at low temperature and increases at higher temperatures. Griffin et al. (31) indicated that reaction with oxygen is one of the principal factors responsible for the aging of the asphalt in the road.

Light has been recognized as a degradation factor in asphalt durability. Research has shown that constituents present in asphalt are susceptible to photo-oxidation.

Heat accelerates the reactions and increases the rate of asphalt oxidation.

Water, in both liquid and gaseous phase, has the same effect as light and heat on asphalt degradation.

Some other factors which should be taken into consideration, with respect to the hardening and the loss of durability of asphalt, include the absorption of asphalt by aggregate, the existence of atmospheric oxidants, and the effect of mechanical stress due to traffic loading.

4.2.2 Evaluation of Asphalt Cement Durability

Natural weathering appears to be the most rational and practical method of evaluating the durability of asphalt. However, a long period of time is required to accomplish this task. Consequently a number of rapid methods for predicting asphalt durability, under both natural and accelerated weathering conditions, have been developed. A detailed description of these methods are given in National Cooperative Research Program Synthesis of Highway Practice 59 (32).

Among these methods the Thin-Film Oven test (TFOT) and the Rolling Thin-Film Oven test (RTFOT) are the most extensively used worldwide. For the purpose of this investigation the Thin-Film Oven test is adopted as a suitable test method for determining asphalt durability.

This method, developed by Lewis and Welborn (33), has been used in the determination of the effect of heat and air on a film of semisolid asphalt cement. The effects of this treatment are determined from the measurement of selected asphalt properties before and after the test.

The TFOT and RTFOT were adopted by AASHTO and ASTM as standard test methods to evaluate asphalt durability.

The TFOT method is primarily used to predict the relative changes in asphalt that occur during hot-plant mixing at about 150°C. A film of asphalt is heated in an oven for 5 hours at 163°C. The residue from the TFOT is tested for weight loss, penetration, viscosity, ductility, and softening point. These measurements are then compared to the original asphalt properties.

4.2.3 Asphalt Cement Durability Test Results

The TFOT was adopted in this study to evaluate the durability characteristics of the virgin and recycled asphalt cements. Extraction and recovery of the asphalt cement from the reclaimed asphalt concrete materials has been carried out. The recovered asphalt cement was blended with the virgin asphalt at various ratios. The percentage of the reclaimed asphalt cement in the blend had values of 0, 30, 50, 70, and 100. These blends were then tested for penetration at 25°C, absolute viscosity at 60°C and kinematic viscosity at 135°C before and after the Thin-Film Oven test. It should be mentioned that the before Thin-Film Oven test results illustrate the properties of the binder prior to mixing with aggregate, whereas the after Thin-Film Oven test results approximate the binder properties after being mixed and placed in the field.

Variation in penetration with recycling ratio before and after the Thin-Film Oven test is shown in Figure 4.1. As can be observed, penetration decreases as the percentage of the recycled asphalt cement in the blend (r) increases. However this happens at a slower rate for blends tested after the Thin-Film Oven test. There exists a very good correlation between the log penetration and the r value. A linear relationship can almost be assumed. An equation based upon the log penetration and the r value has been developed which is shown on the figures. This equation can be used to estimate the blend penetration at a selected r value.

Variation in absolute viscosity with recycling ratio before and after the Thin-Film Oven test can be observed in Figure 4.2. Absolute viscosity seems to be increasing as the percentage of the recycled asphalt cement in the blend increases. This occurs at a slower rate for blends tested after the Thin-Film Oven test. A similar relationship is also noted for the kinematic viscosity as shown in Figure 4.3.

The blend with r/v ratio of 50/50 and SC-3000 as virgin binder has shown similar consistency characteristics to that of 300-400A penetration grade virgin binder.

The "Pen-Vis Number" (PVN) seems to be almost constant

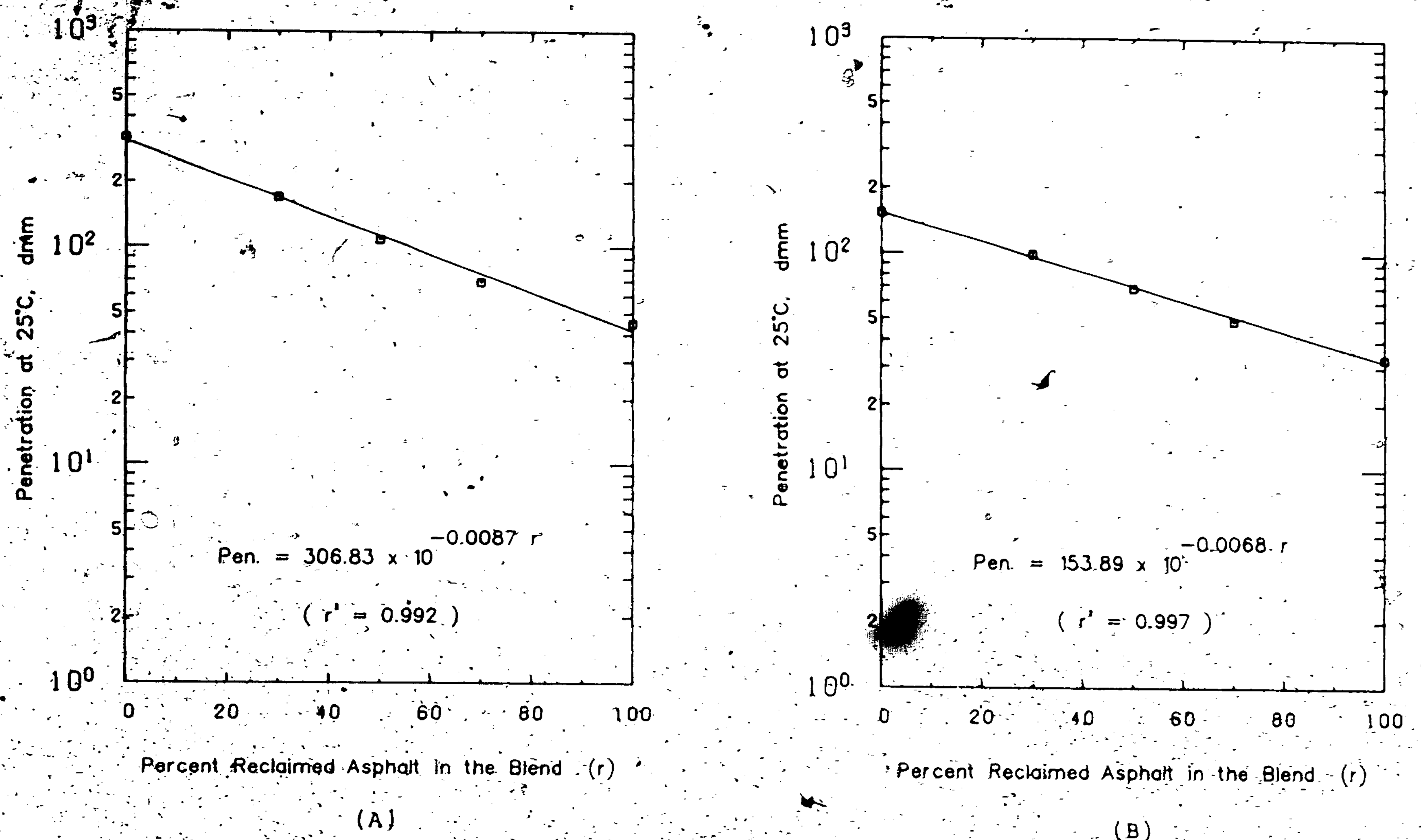


FIGURE 4.1 RELATIONSHIP BETWEEN PENETRATION AND PERCENT RECLAIMED ASPHALT IN THE BLEND (A) BEFORE TFOT, (B) AFTER TFOT.

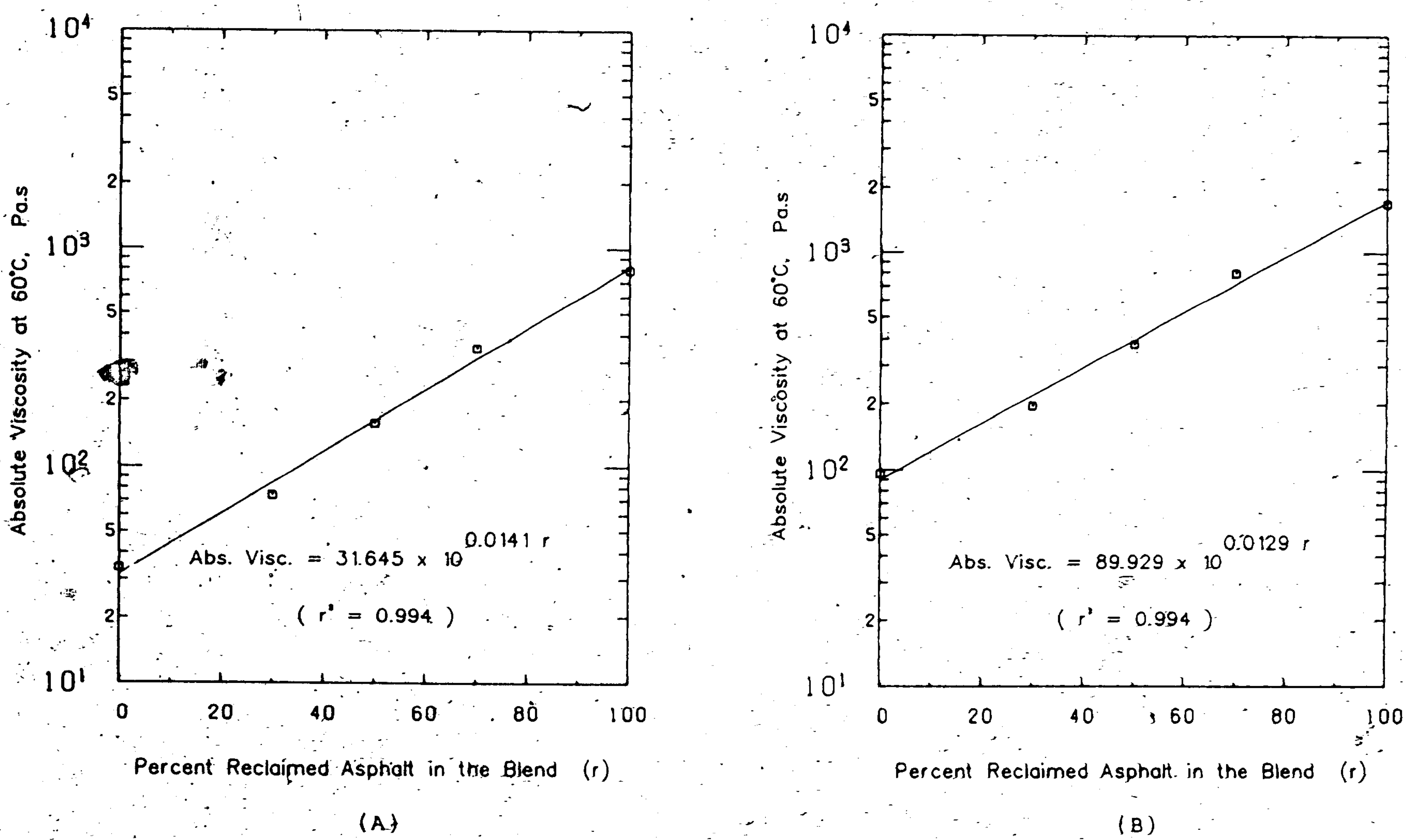


FIGURE 4.2 RELATIONSHIP BETWEEN ABSOLUTE VISCOSITY AND PERCENT RECLAIMED ASPHALT IN THE BLEND (A) BEFORE TFOT, (B) AFTER TFOT.

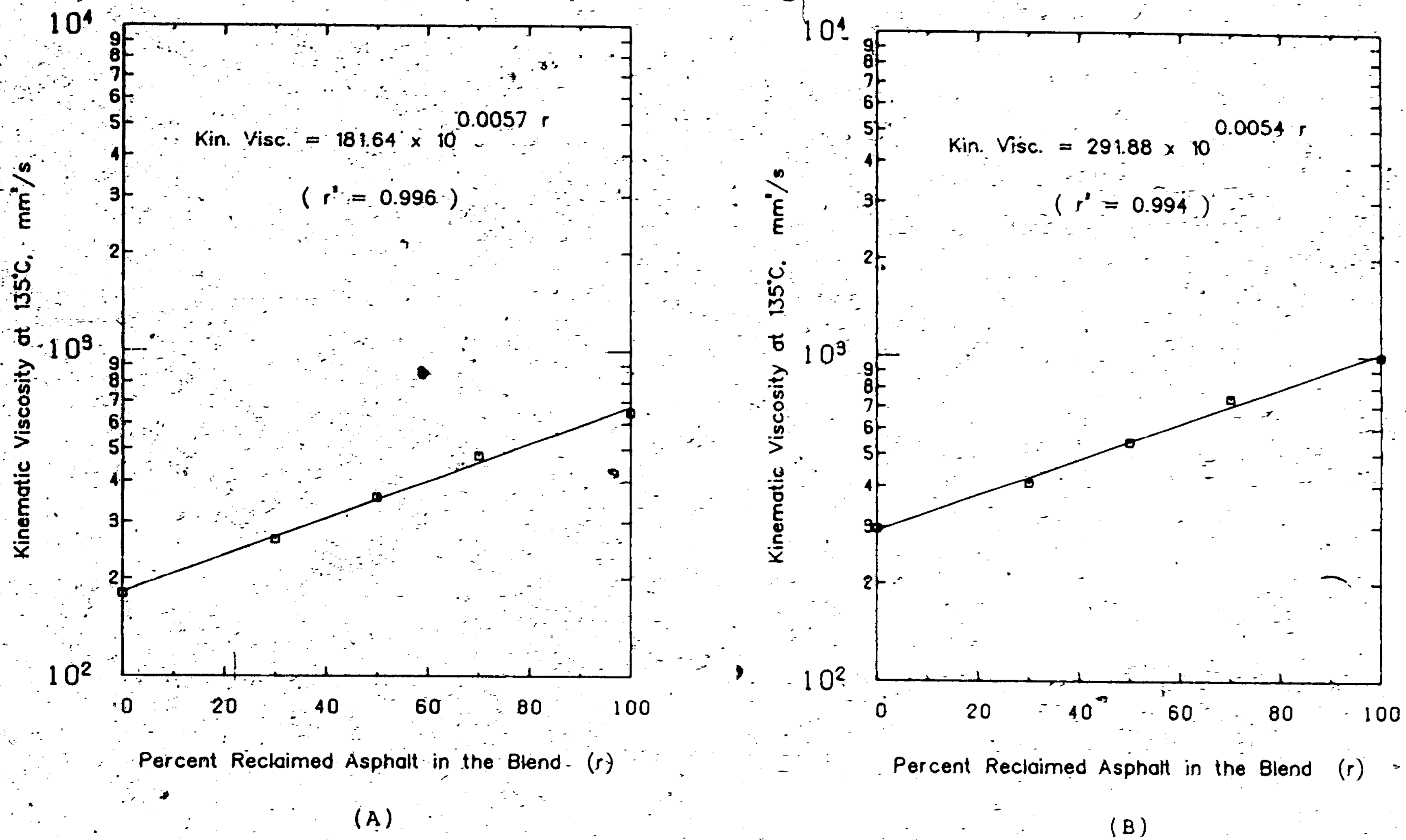


FIGURE 4.3 RELATIONSHIP BETWEEN KINEMATIC VISCOSITY AND PERCENT RECLAIMED ASPHALT IN THE BLEND (A) BEFORE TFOT, (B) AFTER TFOT.

with variation in r values after the Thin-Film Oven test. This value, which approximately equals -0.2, indicates a low temperature susceptible material. The blend with SC-3000 as virgin binder exhibits lower PVN than other blends. An ascending increasing trend of the PVN versus r value is observed before the Thin-Film Oven test. However, the PVN varies from -0.0 to -0.4 which still remains in the low temperature susceptibility category. The variation in PVN with r values before and after the Thin-Film Oven test is shown in Figure 4.4.

A summary of the binder durability characteristics, i.e. weight loss, retained penetration, and viscosity ratio, is shown in Table 4.1. The data show that the retained penetration increases in the blend as the percentage of the recycled asphalt cement increases. This is reflected by a smaller weight loss during the Thin-Film Oven test. There is a good correlation between the retained penetration and the r value. The relationship is approximately linear and is shown in Figure 4.5. The data also shows a decrease in the viscosity ratio at 60°C with an increasing r value. This indicates that less hardening occurs as pavement ages. The relationship between the viscosity ratio and r value is approximately linear and is shown in Figure 4.6.

In general, the tendency for lower weight loss, higher retained penetration, and a lower viscosity ratio indicates that the recycled asphalt cement has higher durability than the virgin asphalt. The increase in durability is

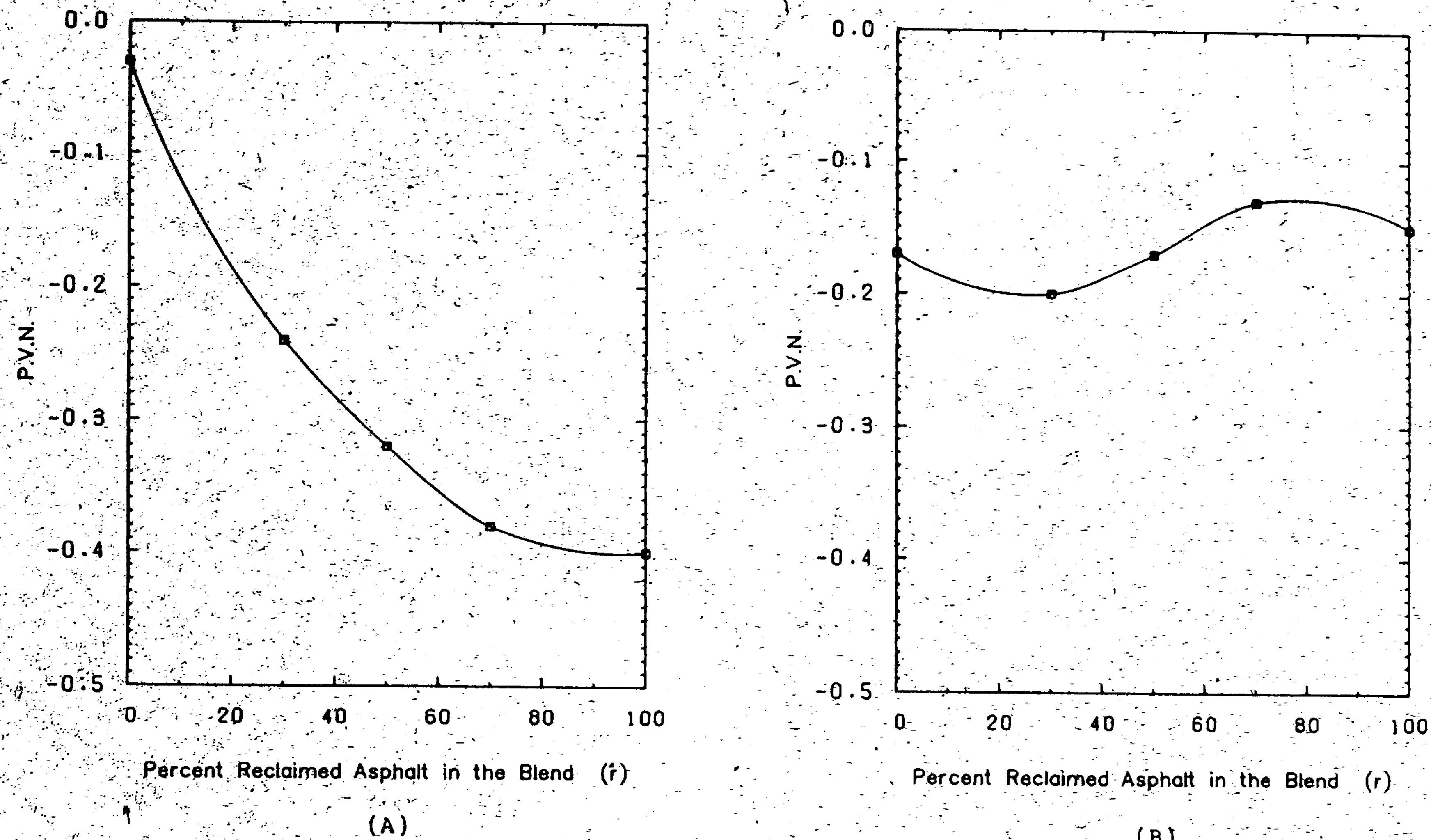


FIGURE 4.4 RELATIONSHIP BETWEEN P.V.N. AND PERCENT RECLAIMED ASPHALT IN THE BLEND (A) BEFORE TFOT, (B) AFTER TFOT.

TABLE 4.1

SUMMARY OF THE DURABILITY CHARACTERISTICS OF THE BINDERS

r %	Weight Loss %	Retained Penetration At 25 C %	Viscosity Ratio At 60 C
0	1.528	48.0	2.8
30	1.156	58.6	2.6
50	1.175	63.9	2.4
70	1.118	71.0	2.3
100	0.638	73.3	2.1
50 SC3000	2.000	52.2	2.4

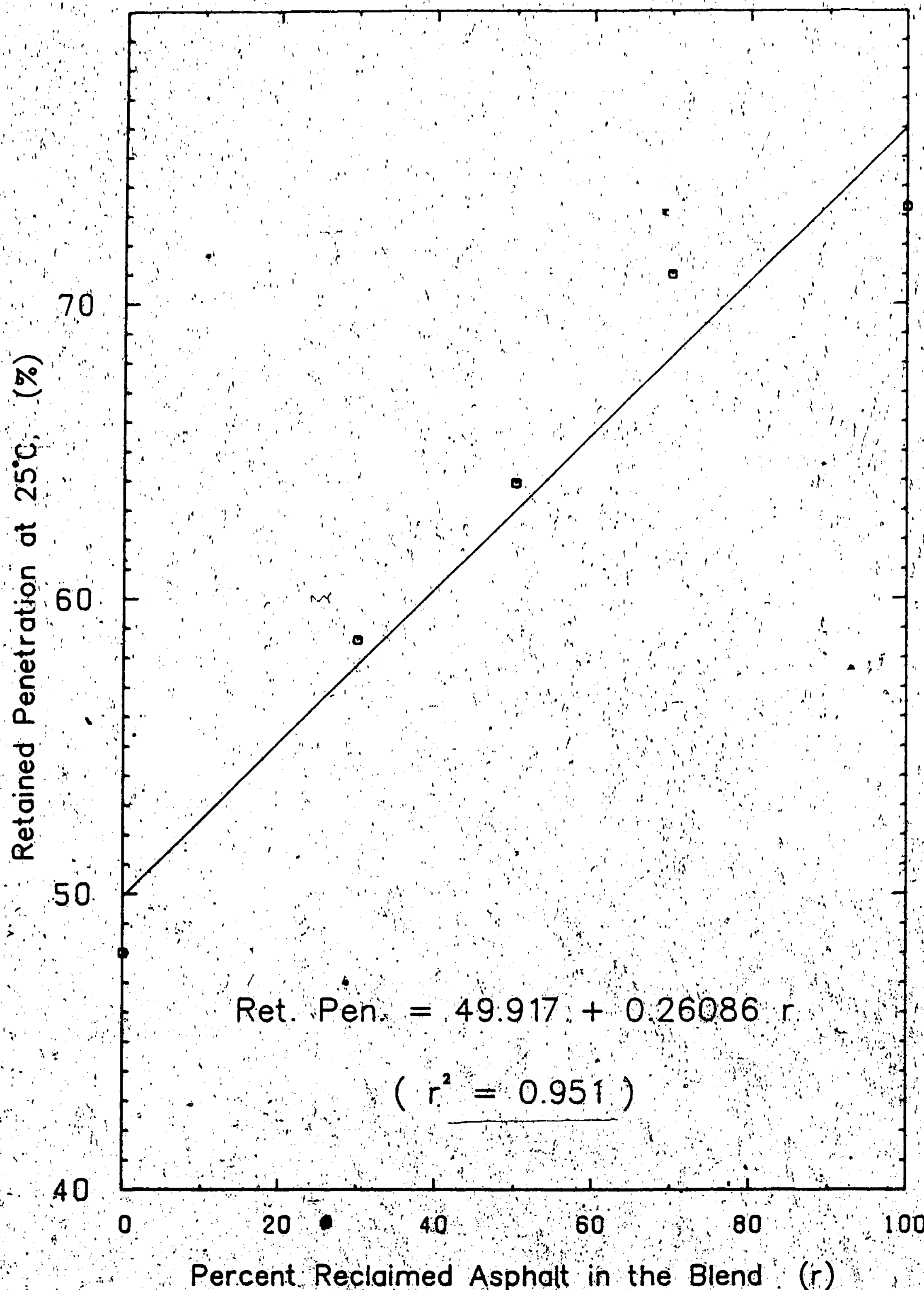


FIGURE 4.5 RELATIONSHIP BETWEEN RETAINED PENETRATION AND PERCENT RECLAIMED ASPHALT IN THE BLEND.

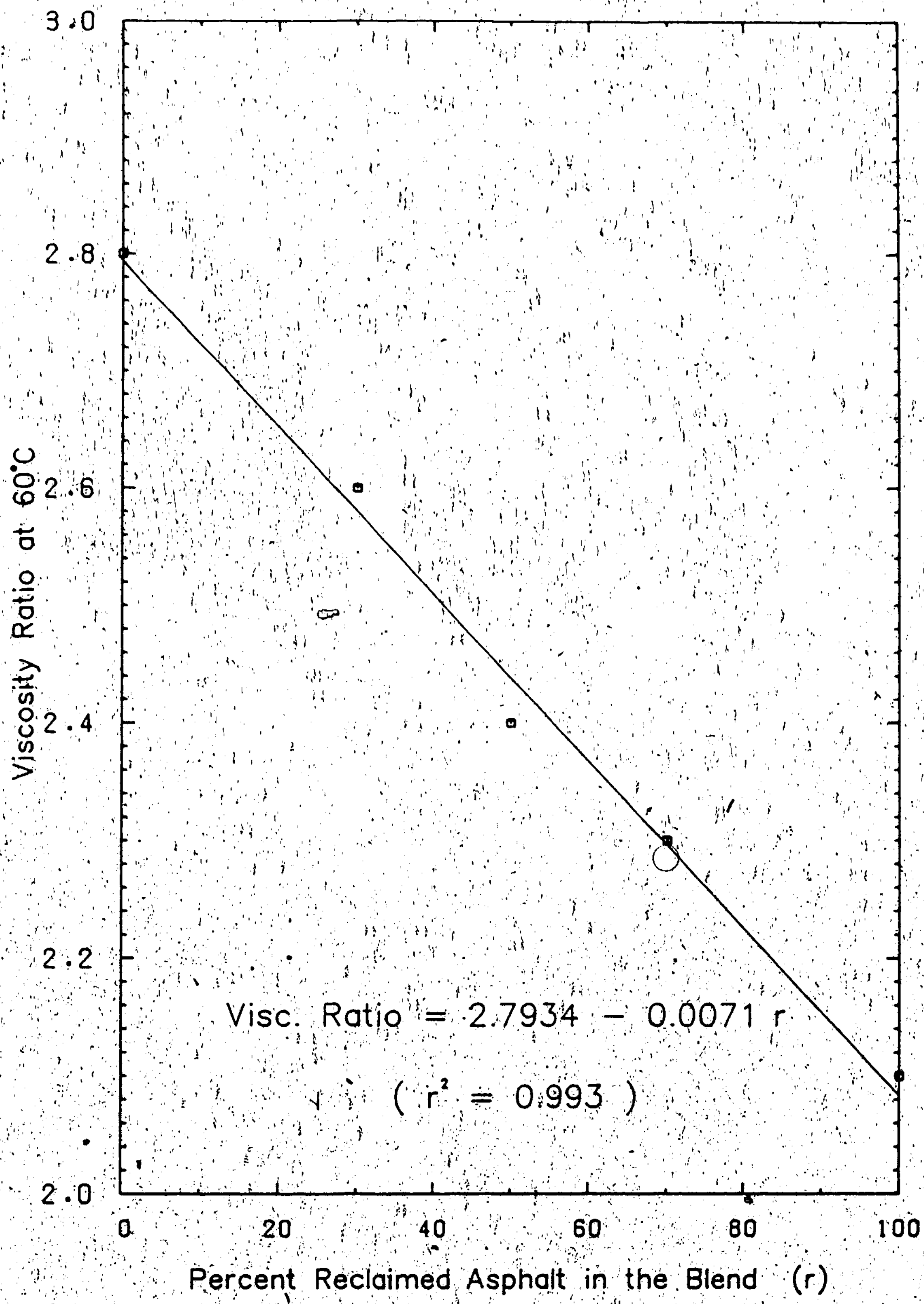


FIGURE 4.6 RELATIONSHIP BETWEEN VISCOSITY RATIO AND PERCENT RECLAIMED ASPHALT IN THE BLEND.

proportional to the increase in the percentage of the recycled asphalt cement in the blend.

4.3. Asphalt Concrete Pavement Durability

Finn (34) has defined the durability of asphalt concrete pavement as the long-term resistance to the effects of aging. Good durability can be described as the ability to yield long-term performance without an abnormal amount of pavement distress. It should be noted that durable asphalt and aggregate do not necessarily provide pavement with good durability if there is poor mixture design or construction. However, it can be stated that the performance of asphalt concrete pavement is greatly affected by the durability of the asphalt components. The hardening of asphalt, as it ages, results in the appearance of pavement deficiencies which eventually lead to pavement failure.

Other major factors influencing pavement durability are mix void content and aggregate porosity. High void content and high aggregate porosity accelerate the rate of asphalt hardening and result in a reduction of pavement durability. Kemp et al. (35) have indicated that the effect of void content and aggregate porosity become more significant as the temperature increases. However, the effect of porosity is dependent on the nature of the asphalt. They denoted that in the cooler climates, asphalt hardening rates are considerably slower and that the cause of this hardening is more equally divided among climatic factors, mix voids, and aggregate porosity.

4.3.1 Evaluation of Asphalt Concrete Pavement Durability

There is not enough emphasis placed on durability criteria in current mixture design practice. The standard test methods available to evaluate the durability potential of asphalt concrete pavements include Immersion Compression or Marshall Immersion tests. The Marshall Immersion is a common durability test method and it involves testing compacted asphalt concrete briquettes for stability after one day of immersion in a 60°C water bath. This test determines the water susceptibility of the asphalt concrete and its significant effect on pavement durability. The retained stability after one day of immersion in a 60°C water bath, has been adopted by many agencies as the criterion index for the durability potential of the mixture.

The common durability tests are limited to only short periods of hot water immersion. Craus et al. (36) have suggested assessing the durability potential of asphalt concrete mixtures during and after longer immersion periods. They have developed two quantitative durability indices based on the testing of compacted asphalt concrete briquettes, after 0, 1, 7 and 14 days of immersion, for Marshall stability and resilient modulus. However, to use the suggested method and durability indices more verification is required.

A laboratory test procedure called Cold Water Abrasion

(C.W.A.) has been developed in Minnesota (37). This test, as a measure of durability, determines if stripping occurs in asphalt concrete mixtures.

4.3.2 Asphalt Concrete Durability Test Results

The Marshall Immersion test was adopted in this investigation to assess water susceptibility and therefore a measure of durability of the recycled asphalt concrete pavements.

Four compacted asphalt concrete specimens from each group with various recycling ratios were tested for stability after 24 hours of water soaking in a 60°C water bath. The average results for each group are shown in Table 4.2. The Marshall stability test results are also represented in this table. The stability test results indicate that the recycled asphalt concrete pavements exhibit higher stability than conventional asphalt concrete pavements.

The retained stability of the specimens were calculated by dividing the values of stability after 24 hours of water soaking by the values after 30 minutes and expressed as a percentage. The retained stability values are shown in Table 4.2. It can be observed that, except for the conventional mixture, there exists an ascending trend between the retained stability and the recycling ratio. A higher recycling ratio results in higher retained stability. It should be kept in mind that the results of this type of test are subject to some variation due to the nature of the test procedure. In

TABLE 4.2

VARIATION IN STABILITY AND RETAINED STABILITY WITH PERCENT RECLAIMED MATERIAL IN THE MIX

R %	Stability after 30-min soak N	Stability after 24-hrs soak N	Retained Stability %
0	7150	6230	87
30	14750	12690	86
50	13900	12230	88
70	13400	12460	93
100	21050	20000	95
50 SC3000	12750	10970	86

general, it can be concluded that recycled asphalt concrete mixtures exhibit higher durability than conventional materials. This is expected, since, as was discussed in section 4.2.3, recycled asphalt cement, which is one of the main components of the recycled asphalt concrete pavement, have higher durability than the virgin asphalt cement.

4.4 Summary

Durability, which is one of the prime factors influencing the service life of a flexible pavement, is discussed in this Chapter. Factors influencing asphalt cement and asphalt concrete pavement durability are also described.

The retained penetration and viscosity ratio values, which are considered as durability measures for asphalt cement, were determined for various virgin and recycled blends. Retained stability values as means of measuring the durability of asphalt concrete, were also determined for various mixtures having different recycling ratios.

Test results indicated that recycled asphalt cement and recycled asphalt concrete exhibit higher durability than conventional materials. The data have also shown that as the percentage of the recycled material in the pavement increases, higher values of durability can be expected.

5. ASPHALT CEMENT AND ASPHALT CONCRETE STIFFNESS

5.1 Introduction

Stiffness modulus is one of the most important parameters, as a material characteristic of asphalt materials. The concept of stiffness modulus to represent the behaviour of asphalt materials was first introduced by Van der Poel (38) in 1954. Stiffness modulus has the same dimension as Young's modulus (E) and is defined as the ratio between stress and strain:

$$S(t,T) = \left(\frac{\sigma}{\varepsilon}\right)_{t,T} \qquad (5.1)$$

where

$S(t,T)$ = stiffness at a particular time t and temperature T,

σ = axial stress, and

ε = axial strain.

Stiffness modulus commonly shortened to stiffness is a function of the loading time (t) and the temperature (T). Heukelom (39) has indicated that the strain developing in an asphalt material after application of an external stress σ, basically consists of three components as follows:

(i) elastic strain: $\varepsilon_e = \frac{\sigma}{E}$ (5.2) where

σ = external stress, and

E = Young's modulus.

(ii) viscous strain: $\varepsilon_v = \frac{\sigma t}{3\eta}$ (5.3)

where

t = loading time, and

η = asphalt cement viscosity as a function of temperature.

(iii) delayed-elastic strain:

$$\varepsilon_d = \frac{\sigma}{D} \quad (5.4)$$

where

D = modulus of delayed-elasticity as a function of time and temperature.

The total strain is then equal to the sum of these three strains

$$\varepsilon = \varepsilon_e + \varepsilon_v + \varepsilon_d \quad (5.5)$$

By substituting ε from equation (5.5) into equation (5.1) stiffness modulus can then be determined according to deformation characteristics of the asphalt:

$$\frac{1}{S} = \frac{1}{E} + \frac{t}{3\eta} + \frac{1}{D} \tag{5.6}$$

This equation expresses the visco-elasticity of asphalt cement. At times approaching zero, which signifies instantaneous deformation, the stiffness is the same as the modulus of elasticity (E) and the behaviour is nearly elastic. However, at extended time, the material is not a pure elastic body and shows a permanent flow with time, so that the stiffness is almost directly proportional to the viscosity and its behaviour is viscous. At moderate loading times, the modulus of delayed-elasticity influences the transition from elastic to viscous behaviour.

Van der Poel's concept uses the simple elastic relationship between stress and strain but specifies the time and temperature, thereby recognizing the viscoelastic nature of the material.

The use of stiffness to characterize asphalt materials for low temperature cracking has received a great deal of attention. While there are many factors influencing the occurrence and extent of low temperature cracking, the major influential parameter is the mix stiffness. A low temperature pavement design method is available in which a limiting mix stiffness is the critical parameter. Due to the existence of a strong correlation between asphalt cement and mix stiffness, low temperature asphalt concrete pavement cracking can be directly related to the asphalt cement stiffness.

5.2 Different Approaches to Determine Stiffness

Approaches to determine the stiffness of asphalt materials can be classified into two categories: direct testing and indirect estimation. These are briefly discussed in the following sections.

5.2.1 Direct Determination of Asphalt Cement Stiffness

As discussed previously, the behaviour of asphalt concrete pavements at low temperature, which is one of the major concerns in this investigation, is related to stiffness of the asphalt cement. Accurate evaluation of low temperature asphalt stiffness requires that it would be measured directly rather than predicted. However, most of the instruments available for measuring the stiffness at low temperatures are generally too complex and costly for routine design purposes. A variety of instruments are available but most have limitations when considered in the context of low temperatures experienced in Northern climates.

Schweyer (40-41) developed a constant stress rheometer based on capillary flow principles, but its lower temperature limits of -5°C are substantially above typical pavement cracking temperatures and it therefore has limited applicability.

The Shell Sliding-plate rheometer developed by Fenign and Krooshof (42) has an upper stiffness limit of 1×10^{8} N/m^{2} which is not sufficient. Gaw (43) has described a modified sliding-plate rheometer with an extended upper stiffness limit of 1.5×10^9 N/m^2 which is close to the limiting stiffness value. The modified sliding-plate rheometer is an instrument well suited for the measurement of asphalt stiffness in the range of 1×10^3 to 1.5×10^9 N/m^2, at a corresponding temperature range of approximately -20°C to -45°C.

5.2.2 Direct Determination of Asphalt Concrete Mix Stiffness

There are a number of test methods in existence to establish the necessary stress and strain responses to loading of asphalt mixtures under various conditions.

Direct testing by creep, relaxation, constant rate of strain, dynamic, flexural, and tensile splitting tests are in use. In this study, the tensile splitting test is chosen as the means of investigating the stress-strain behaviour of recycled asphalt concrete mixtures at low temperatures. The detailed test procedures and results will be discussed in chapter 6.

5.2.3 Indirect Estimation of Asphalt Cement Stiffness

It is more desirable and accurate to evaluate the stiffness of the asphalt cement directly. However, for routine design purposes the most practical approach is to predict the stiffness by using available techniques.

The original method for estimating the stiffness of asphalt cement was first introduced by Van der Poel (38). He constructed a nomograph based on results of creep and dynamic testing. Using this nomograph, as shown in Figure 5.1, the stiffness of an asphalt cement can be easily estimated at any particular temperature and time of loading. Use of the nomograph is restricted to asphalts containing less than two percent by weight of wax.

Van der Poel's nomograph requires two basic parameters for computing asphalt cement stiffness. These parameters are the ring and ball softening point temperature ($T_{R\&B}$) and the penetration index (PI). Assuming that the penetration at the temperature of the ring and ball softening point equals 800 dmm. Heukelom (39) presented a reshaped version of van der Poel's nomograph with a slight correction at very low PI values.

The PI used by Van der Poel was initially proposed by Pfeiffer and Van Doormaal (44). It is related to the slope of the line obtained by plotting the logarithm of the penetration of asphalt cement against temperature. The penetration index can be determined by using the following equation:

$$\frac{\text{Log } 800 - \text{Log Pen}}{T_{R\&B} - T_p} = \frac{1}{50} \times \frac{(20 - PI)}{(10 + PI)} \qquad (5.7)$$

where

PI = penetration index,

Pen = penetration of asphalt cement, dmm

$T_{R\&B}$ = ring and ball softening point, °C, and

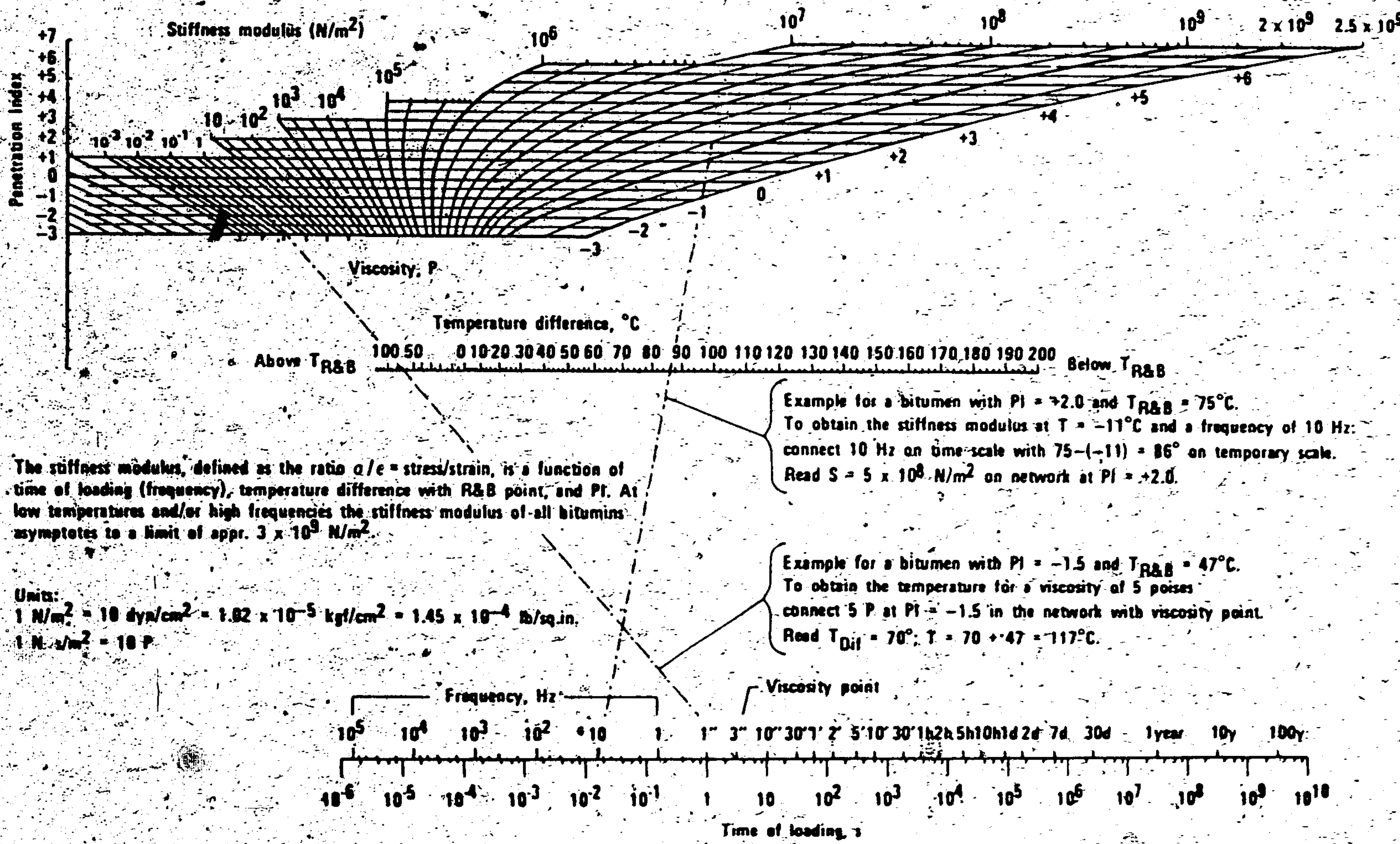


FIGURE 5.1 VAN DER PEOL'S NOMOGRAPH FOR DETERMINING STIFFNESS OF ASPHALT CEMENT.

T_p = temperature at which penetration is determined, °C.

It should be noted that the PI value calculated from a single penetration measurement and a ring and ball softening point does not truly represent the slope of the penetration-temperature line for very waxy asphalts and, therefore, equation (5.7) might not represent the correct value of PI. A more reasonable equation for computing PI values for waxy and non-waxy asphalt is given as follows:

$$\frac{\text{Log Pen}_1 - \text{Log Pen}_2}{T_2 - T_1} = \frac{1}{50} \times \frac{(20 - PI)}{(10 - PI)} \qquad (5.8)$$

where

Pen_1 & Pen_2 = penetration of asphalt cement measured at temperatures T_1 and T_2 respectively.

Using equation (5.8) for calculating PIs requires the measurement of penetration at two temperatures under similar test conditions.

Anderson et al. (45) have indicated that for air blown and very waxy asphalts, the Van der Poel's method of determining stiffness could result in large errors. Heukelom (46) developed a Bitumen Test Data Chart to relate the consistency of asphalt with temperature. He demonstrated that the ring and ball softening point of air blown asphalts and asphalts having a relatively high wax content can be modified by using the Bitumen Test Data Chart. The new derived PI from the modified ring and ball softening point can then be used in conjunction with Van der Poel's nomograph for estimating the asphalt cement stiffness. The applicability and effectiveness of this method as applied to some Canadian asphalts is described by Kopvillem et al. (47). A schematic representation, showing how the values of modified $T_{R\&B}$ and PI can be determined for a waxy asphalt, is presented by Heukelom (48) in Figure 5.2.

As an alternative to the penetration index, McLeod (49-53) has introduced the concept of Pen-vis number (PVN). This was developed due to false ring and ball softening point temperatures of Canadian waxy asphalts which resulted in false values for the penetration index and consequently incorrect values for asphalt stiffness.

McLeod's PVN is based on a correlation between penetration at 25°C, viscosity at 135°C, and Pfeiffer and Van Doormaal's PI for a few selected asphalts. In the development of PVN, McLeod intended to have asphalt cement PVN values as numerically equal as possible, to corresponding PI values. This would then allow the employment of PVN together with Van der Poel's nomograph to determine the asphalt cement stiffness.

In the PVN development process, McLeod has first selected asphalt cements with PI values of 0.0 ± 0.2 and -1.5 ± 0.2, because Canadian asphalts fall within a PI range of 0.0 to -1.5. He has determined the PI of each selected

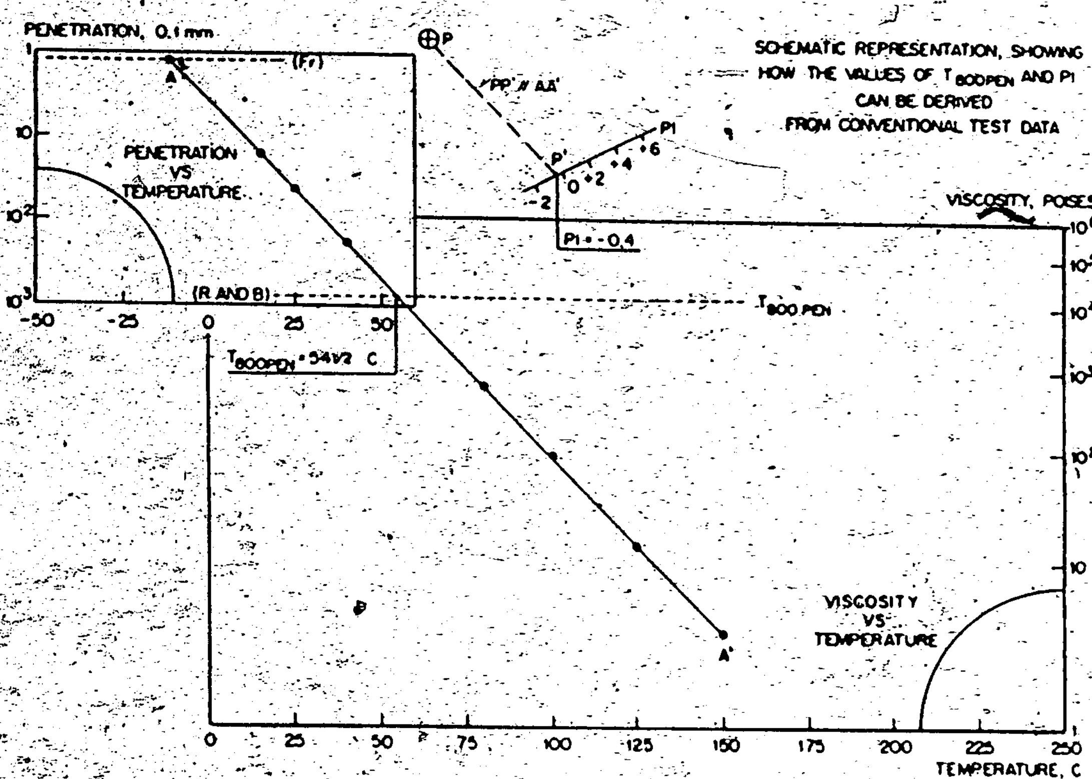


FIGURE 5.2 BITUMEN TEST DATA CHART.

asphalt from its penetration at 25°C and its ring and ball softening point. If the PI of an asphalt cement was within the range of 0.0 ± 0.2, its viscosity at 135°C was determined. A point was then plotted for this asphalt cement in a figure employing penetration at 25°C and viscosity at 135°C as its coordinates. After all the points derived in this manner for selected asphalts had been plotted, an average line was drawn through the points based on the method of least squares. This line then represented a PVN of -1.5. Other PVN lines were established by interpolation.

The equation for the least squares line representing a PVN of 0.0 is:

$$\text{Log } V = 4.25800 - 0.79674 \text{ Log } P \qquad (5.9)$$

and the equation for the least square line representing a PVN of -1.5 is:

$$\text{Log } V = 3.46289 - 0.61094 \log P \qquad (5.10)$$

where

V = viscosity at 135°C, centistoke, and

P = penetration at 25°C, dmm.

The PVN of any asphalt cement for which the penetration at 25°C and viscosity at 135°C are known can be determined by using the following equation:

$$PVN = \frac{\text{Log } L - \text{Log } X}{\text{Log } L - \text{Log } M} \times (-1.5) \qquad (5.11)$$

where

X = viscosity at 135°C associated with the penetration at 25°C of an asphalt cement, centistoke,

L = viscosity at 135°C for a PVN of 0.0 for the penetration of 25°C of the asphalt cement, centistoke, and

M = viscosity at 135°C for a PVN of -1.5 for the penetration at 25°C of the asphalt cement, centistoke.

The base temperature introduced by McLeod which is essentially the ring and ball softening point can be determined from Figure 5.3 which is a modification of Pfeiffer and Van Doormaal's chart. By substituting numerically equal base temperatures for the same corresponding ring and ball softening point on the temperature scale of Van der Poel's nomograph, and by substituting numerically equal PVN values for the same corresponding PI values on the horizontal lines through the upper portion of Van der Poel's nomograph, there results a slightly different nomograph. As shown in Figure 5.4, such a nomograph is provided by McLeod in terms of PVN ratings. Using this nomograph, the stiffness value of an asphalt cement can be determined for any specific temperature and

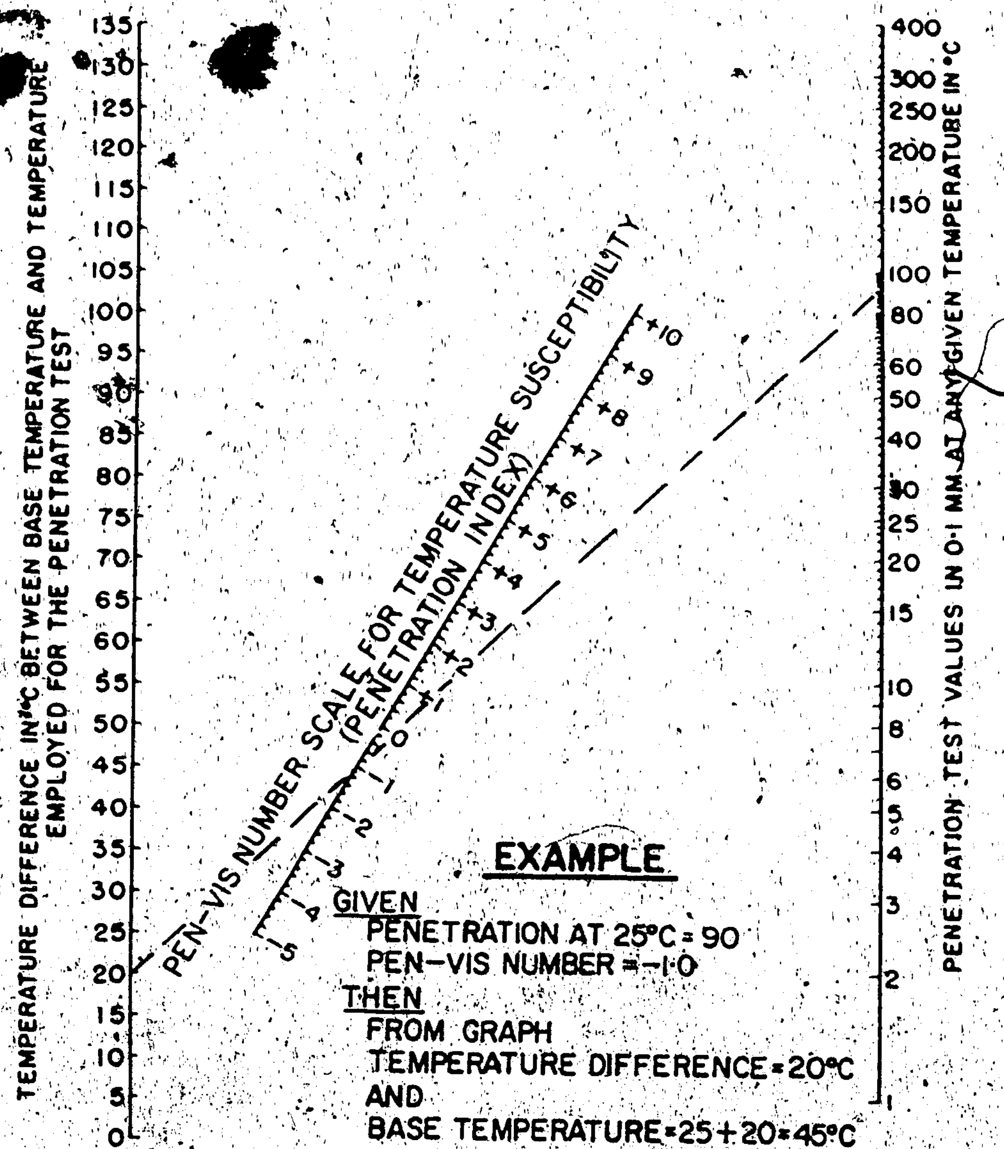


FIGURE 5.3 MODIFIED PFEIFFER'S AND VAN DOORMAAL'S NOMOGRAPH FOR DETERMINING BASE TEMPERATURE.

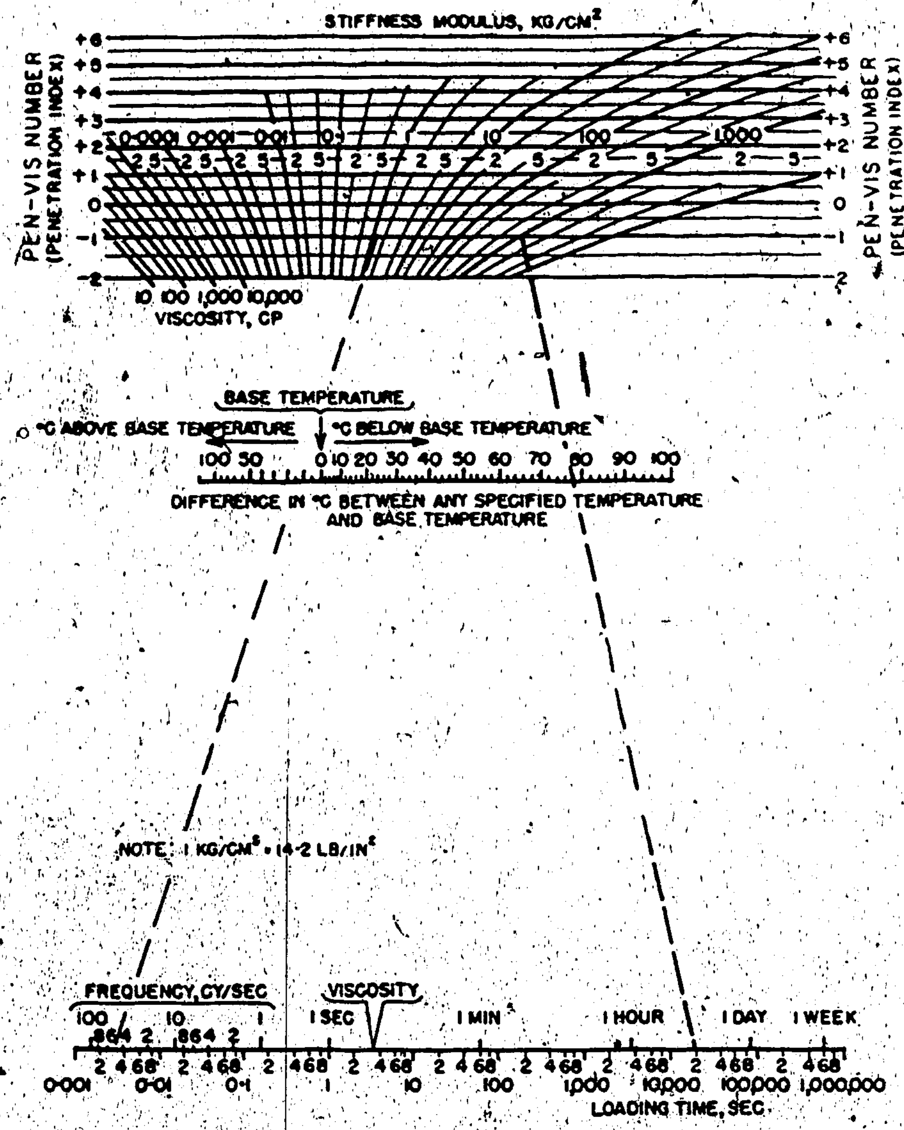


FIGURE 5.4 MODIFIED HEUKELOM'S AND KLOMP'S VERSION OF VAN DER PEOL'S NOMOGRAPH FOR DETERMINING STIFFNESS OF ASPHALT CEMENT.

rate of loading.

Robertson (54-55) has indicated that there is poor agreement between PI and PVN values for most asphalt cements. Puzinauskas (56) has also reported a poor correlation between these two parameters. Since McLeod's nomograph is based on the assumption that PIs and PVNs for various asphalt cements are equal, it must be used with care. However, PVN is a convenient method of expressing temperature susceptibility and of calculating the asphalt stiffness, because it is calculated from penetration at 25°C and viscosity at 135°C which are usually obtained from laboratory test in the course of ordinary inspection. McLeod's PVN method is used in this investigation for determining the stiffness of asphalt cements having different recycled to virgin ratios.

5.2.3.1 Asphalt Cement Stiffness Results Using An Indirect Approach

The first step to estimate the binder stiffness using the indirect approach was to evaluate the PVN for various blends having different recycling ratios. This was achieved by using equation (5.11) which was developed by McLeod, requiring penetration at 25°C and viscosity at 135°C. Having the PVN and the penetration at 25°C, the base temperatures for various blends were determined from Figure (5.3). McLeod's modified version of the Van der Poel's nomograph, as shown in Figure (5.4), was then used for the determination of blend stiffness.

Recycled and virgin asphalt cement stiffness, before and after the Thin-Film Oven test were determined for a range of temperatures varying from -30°C to +30°C at a constant loading time of 25,000 seconds.

Tables 5.1 and 5.2 show the calculated stiffness values for various blends at different temperatures before and after the Thin-Film Oven test respectively. It can be seen that, as the temperature increases, the binder stiffness decreases. The decline in stiffness is almost about one order of magnitude for every 10°C increase in temperature. This is true for both cases of before and after the Thin-Film Oven test. As an example, the magnitude of the blend stiffness after the Thin-Film Oven test may vary from 9.8×10^4 kPa for 100 percent recycled asphalt at -30°C to 2.0×10^{-3} kPa for 100 percent virgin asphalt at +30°C.

Variation in blend stiffness due to different recycling ratios can be examined from results presented in Tables 5.1 and 5.2. In general, blend stiffness increases with an increase in the recycling ratio. This is clearly illustrated in Figures 5.5 to 5.11 for a range of temperatures varying from -30°C to +30°C. The blend with SC-3000 as virgin binder, at 50 percent recycling, exhibits lower stiffness values than the blend with 300-400A penetration grade virgin asphalt at a similar recycling ratio. By performing a statistical analysis on binder stiffness results after the Thin-Film Oven test, it can be observed that the relationship

TABLE 5.1

CALCULATD BINDER STIFFNESS AT VARIOUS TEMPERATURES
BEFORE THIN-FILM OVEN TEST

r	Binder Stiffness, kPa At Temperatures of:						
%	-30 C	-20 C	-10 C	0 C	10 C	20 C	30 C
0	5.9E+02	4.9E+01	4.9E+00	3.9E-01	2.9E-02	3.9E-03	4.9E-04
30	4.9E+03	3.9E+02	2.9E+01	2.0E+00	2.0E-01	2.0E-02	2.0E-03
50	1.5E+04	9.8E+02	8.8E+01	5.9E+00	4.9E-01	3.9E-02	3.9E-03
70	4.9E+04	5.4E+03	3.9E+02	2.5E+01	2.0E+00	9.8E-02	9.8E-03
100	9.8E+04	1.5E+04	9.8E+02	8.8E+01	5.4E+00	3.9E-01	2.5E-02
50 SC3000	1.5E+03	9.8E+01	6.9E+00	5.4E-01	4.9E-02	4.9E-03	5.9E-04

TABLE 5.2

CALCULATED BINDER STIFFNESS AT VARIOUS TEMPERATURES
AFTER THIN-FILM OVEN TEST

r	Binder Stiffness, kPa At Temperatures of:						
%	-30 C	-20 C	-10 C	0 C	10 C	20 C	30 C
0	5.4E+03	4.9E+01	2.9E+01	2.5E+00	2.0E-01	1.6E-02	1.9E-03
30	1.5E+04	9.8E+02	9.8E+01	6.9E+00	5.4E-01	6.3E-02	6.6E-03
50	2.5E+04	2.9E+03	2.0E+02	1.5E+01	9.8E-01	1.4E-01	1.2E-02
70	4.9E+04	8.8E+03	5.9E+02	4.9E+01	4.9E+00	2.8E-01	2.3E-02
100	9.8E+04	2.0E+04	2.0E+03	1.5E+02	9.8E+00	6.0E-01	4.6E-02
50 SC3000	9.8E+03	9.8E+02	5.9E+01	4.9E+00	3.9E-01	2.6E-02	2.6E-03

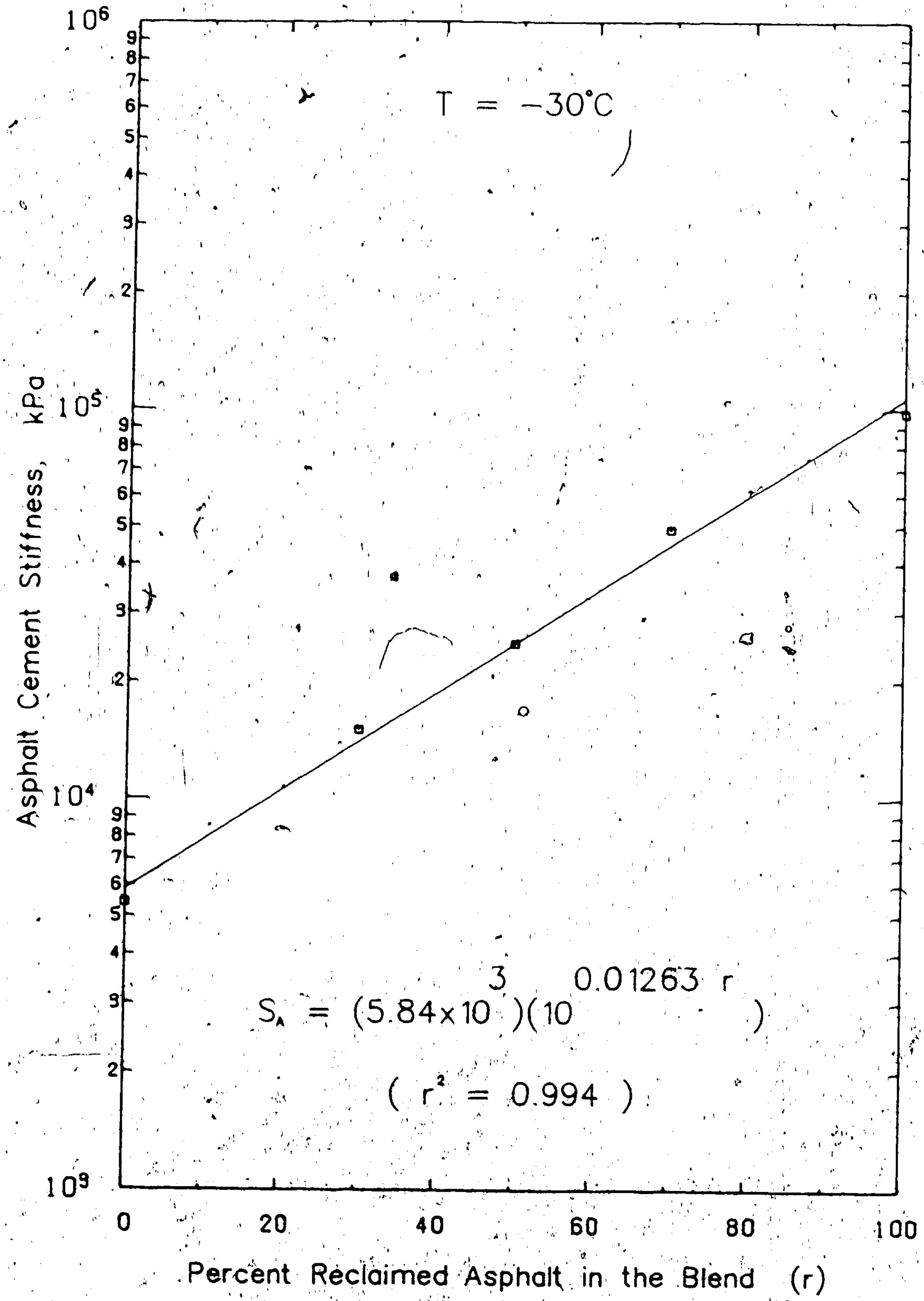


FIGURE 5.5 RELATIONSHIP BETWEEN BINDER STIFFNESS AND PERCENT RECLAIMED ASPHALT IN THE BLEND AT -30°C.

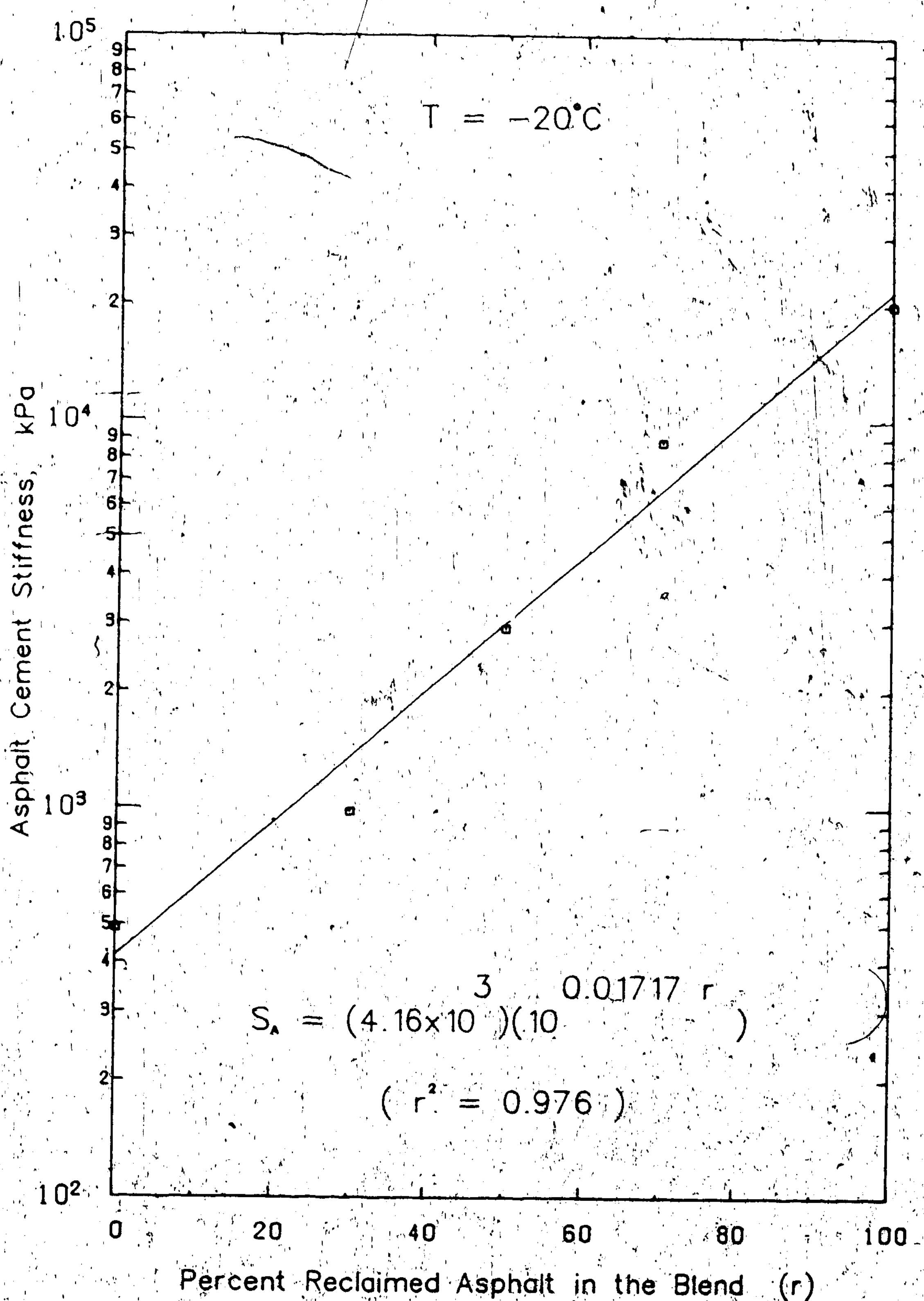


FIGURE 5.6 RELATIONSHIP BETWEEN BINDER STIFFNESS AND PERCENT RECLAIMED ASPHALT IN THE BLEND AT −20°C.

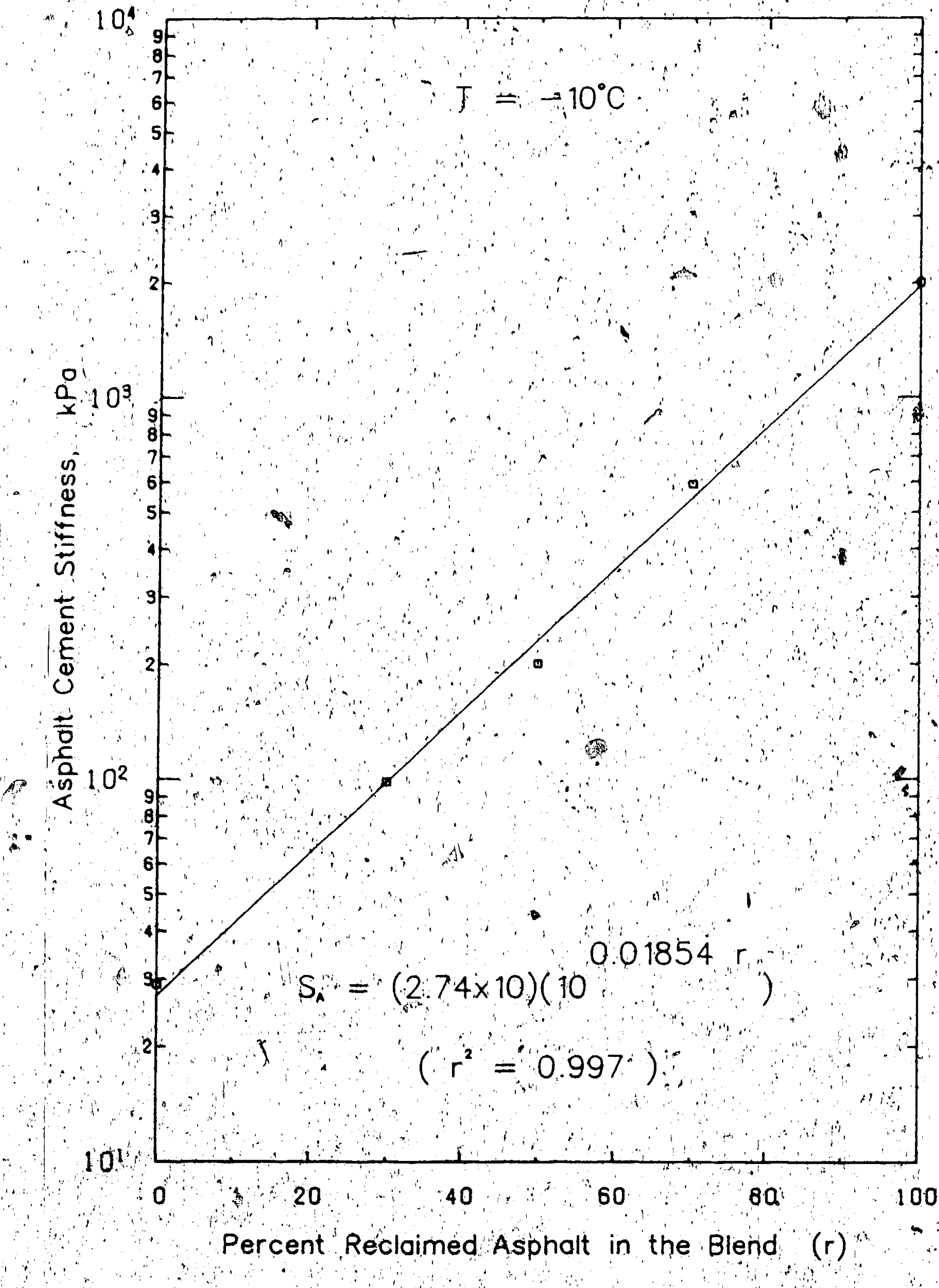


FIGURE 5.7 RELATIONSHIP BETWEEN BINDER STIFFNESS AND PERCENT RECLAIMED ASPHALT IN THE BLEND AT -10°C.

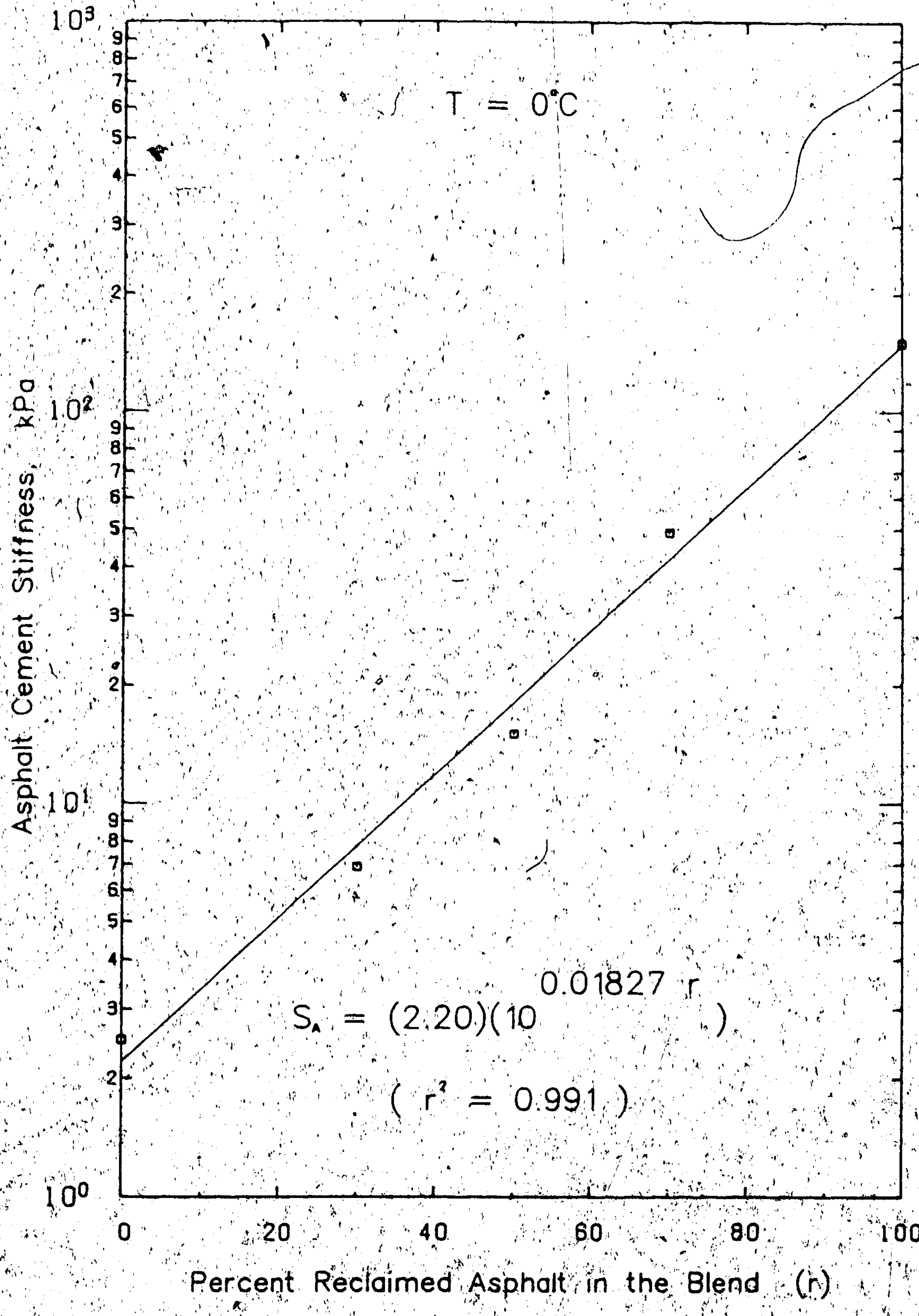


FIGURE 5.8 RELATIONSHIP BETWEEN BINDER STIFFNESS AND PERCENT RECLAIMED ASPHALT IN THE BLEND AT 0°C.

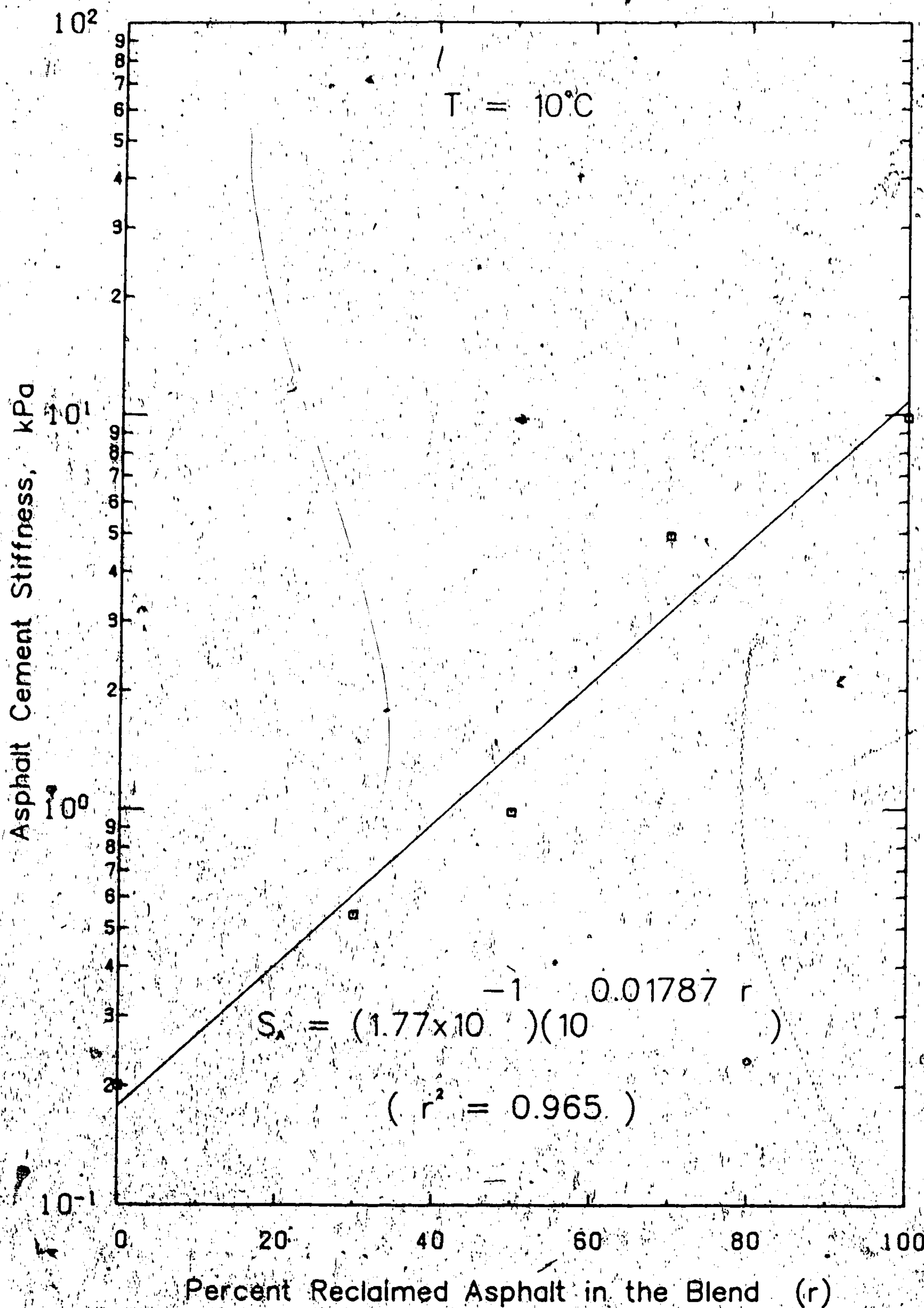


FIGURE 5.9 RELATIONSHIP BETWEEN BINDER STIFFNESS AND PERCENT RECLAIMED ASPHALT IN THE BLEND AT 10°C.

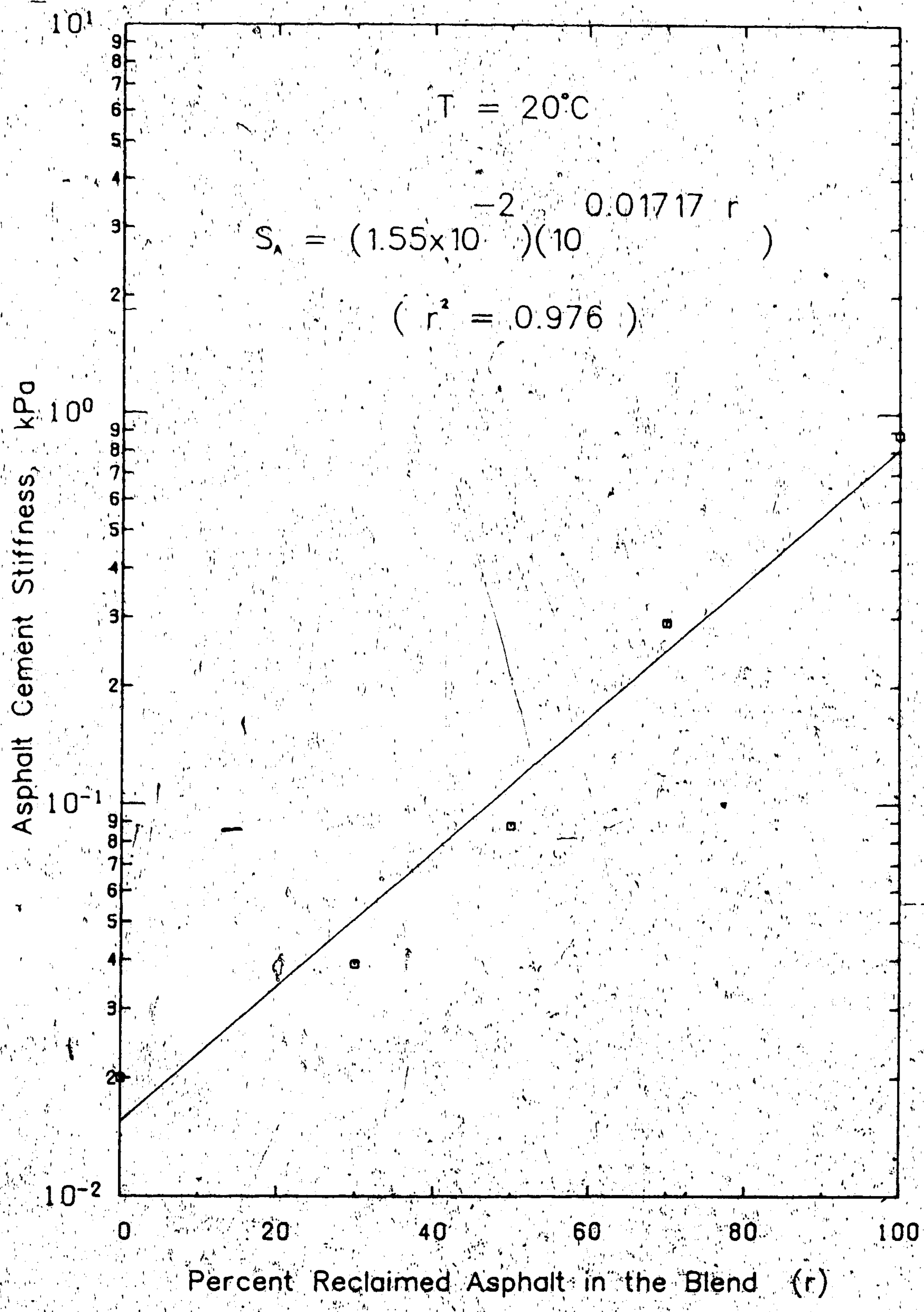


FIGURE 5.10 RELATIONSHIP BETWEEN BINDER STIFFNESS AND PERCENT RECLAIMED ASPHALT IN THE BLEND AT 20°C.

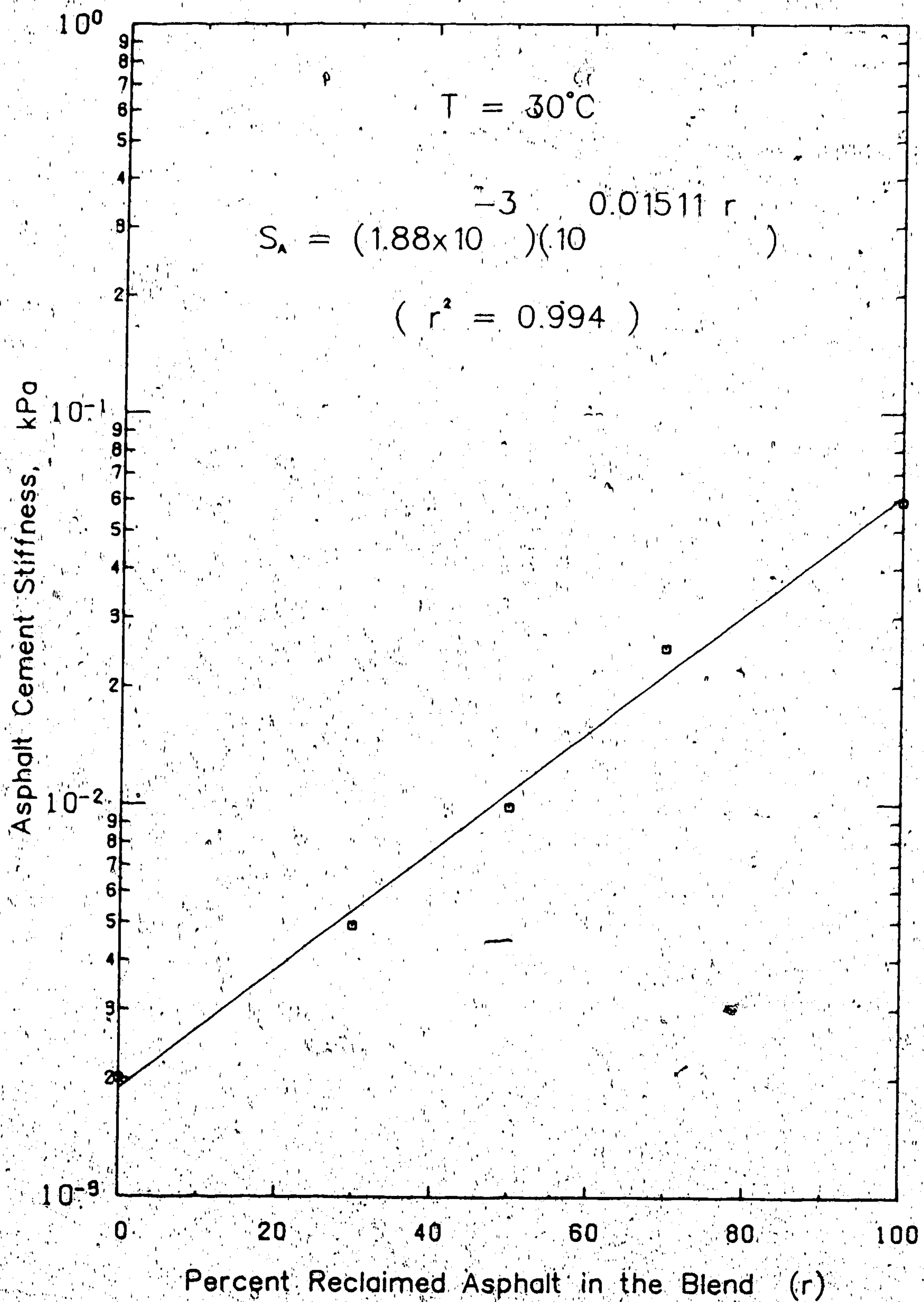


FIGURE 5.11 RELATIONSHIP BETWEEN BINDER STIFFNESS AND PERCENT RECLAIMED ASPHALT IN THE BLEND AT 30°C.

between the blend stiffness and the recycling ratio is approximately linear, with a coefficient of determination (r^2) ranging from 96.5 to 99.7 percent. The equation of the best fit line which was obtained by using the least square method for each particular case is also shown on these figures. By using these equations or the actual figures, the stiffness of the blend at any particular temperature for a desired recycling ratio may be estimated. This may then be used for calculation of the asphalt concrete mixture stiffness.

Another point which should be mentioned at this stage is that the stiffness of the blends having a ~~higher~~ percentage of recycled asphalt cement are less susceptible to the Thin-Film Oven test especially at low temperatures. This can be seen by examining the results shown in Tables 5.1 and 5.2. As an example, at -30°C the stiffness of the 100 percent virgin blend has changed from 5.9×10^2 kPa to 5.4×10^3 kPa after the Thin-Film Oven test, whereas, the stiffness of the blend having 100 percent recycled asphalt has remained unchanged.

In general, it can be concluded that the binder stiffness is proportional to the percentage of the recycled asphalt cement in the blend and it is highly affected by temperature at a particular loading time.

5.2.4 Indirect Estimation of Asphalt Concrete Mix Stiffness

Heukelom and Klomp (57) have proposed a semi-empirical formula to compute the stiffness of an asphalt concrete mixture. This formula relates the mix stiffness with stiffness of the binder at the same temperature and time of loading and is in the following form:

$$\frac{S_{mix}(t,T)}{S_{asphalt}(t,T)} = \left[1 + \frac{2.5}{n} \times \frac{C_v}{1 - C_v}\right]^n \qquad (5.12)$$

where

S_{mix} = stiffness of the asphalt concrete mixture at a particular loading time, t, and temperature, T, kg/cm²

$S_{asphalt}$ = stiffness of the asphalt cement at a particular loading time, t, and temperature, T, kg/cm²

$n = 0.83 \log \dfrac{4 \times 10^5}{S_{asphalt}}$,

C_v = volume concentration of aggregate

$$= \frac{\text{volume of aggregate}}{\text{volume of aggregate plus asphalt cement}}$$

$$= \frac{100 - \%\ \text{VMA}}{100 - \%\ \text{Air Voids}}$$

VMA = voids in mineral aggregate.

This formula was developed for asphalt concrete mixtures with C_v values ranging from 0.7 to 0.9, and air void contents in the order of three percent. A chart for solving the equation is developed and is presented in Figure 5.12.

Van Draat and Sommer (58) have developed a corrected C_v

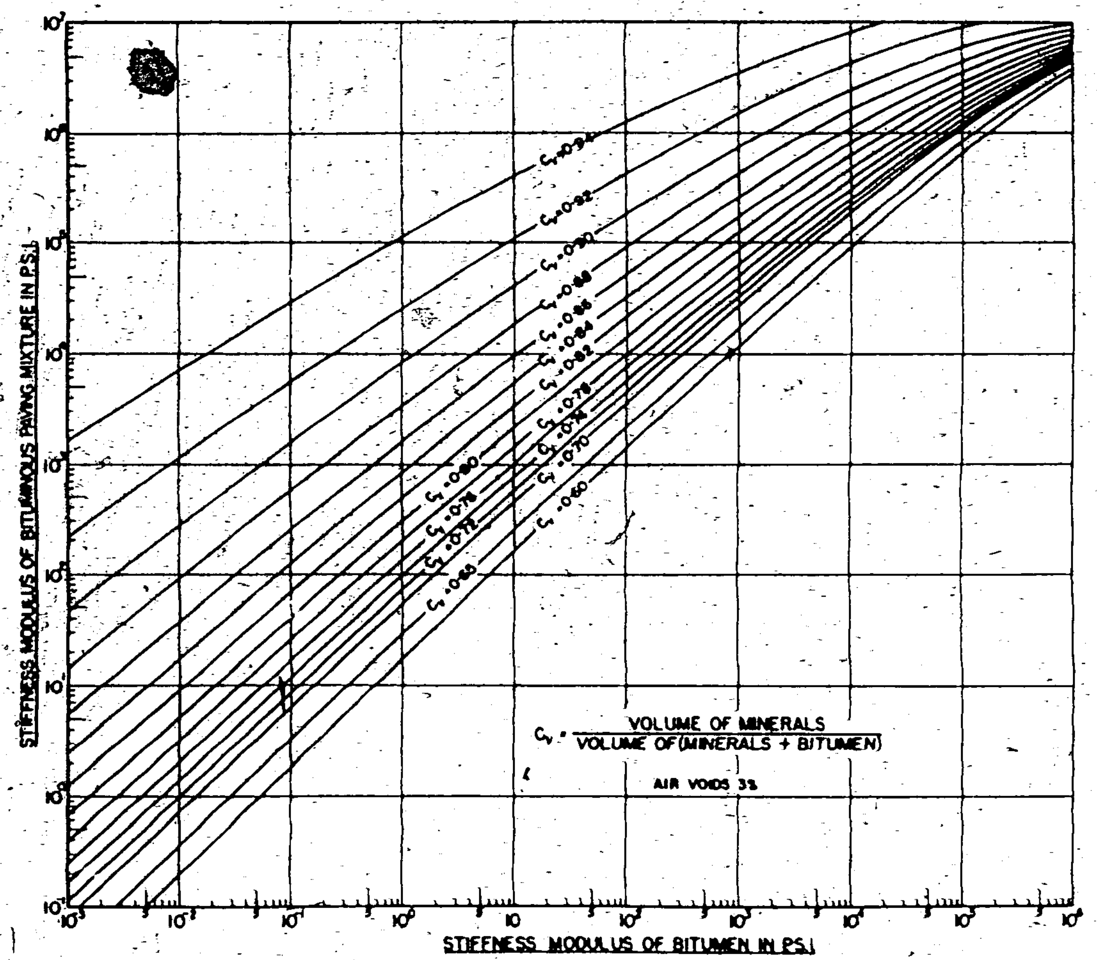


FIGURE 5.12 RELATIONSHIPS BETWEEN MODULI OF STIFFNESS OF ASPHALT CEMENTS AND OF PAVING MIXTURES CONTAINING THE SAME ASPHALT CEMENTS.

applicable to compacted asphalt concrete mixtures containing a higher percentage of air voids. The corrected C_v value can be calculated as follows:

$$C_v' = \frac{C_v}{1 + \Delta v} \qquad (5.13)$$

where

Δv = the difference, expressed as a decimal, between the percentage by volume of air voids in an asphalt concrete mixture and an air voids value of 3 percent for the same mixture.

Bonnaure et al. (59) have developed a new method of predicting the stiffness of asphalt concrete mixtures. The nomograph produced by Bonnaure is shown in Figure 5.13. By comparing stiffness predicted by this approach and experimental results, he proved the validity of this nomograph. This approach is applicable to all asphalt concrete mixtures, provided that the asphalt cement has a stiffness value higher than 5×10^6 N/m^2.

Generally the indirect methods of estimating asphalt cement and mix stiffness seem to give a sufficient degree of accuracy and are most useful. However, careful attention must be placed on the procedures in estimating the stiffness particularly when dealing with blown and waxy asphalts.

In this study, stiffness values of various types of conventional and recycled asphalt concrete mixtures are

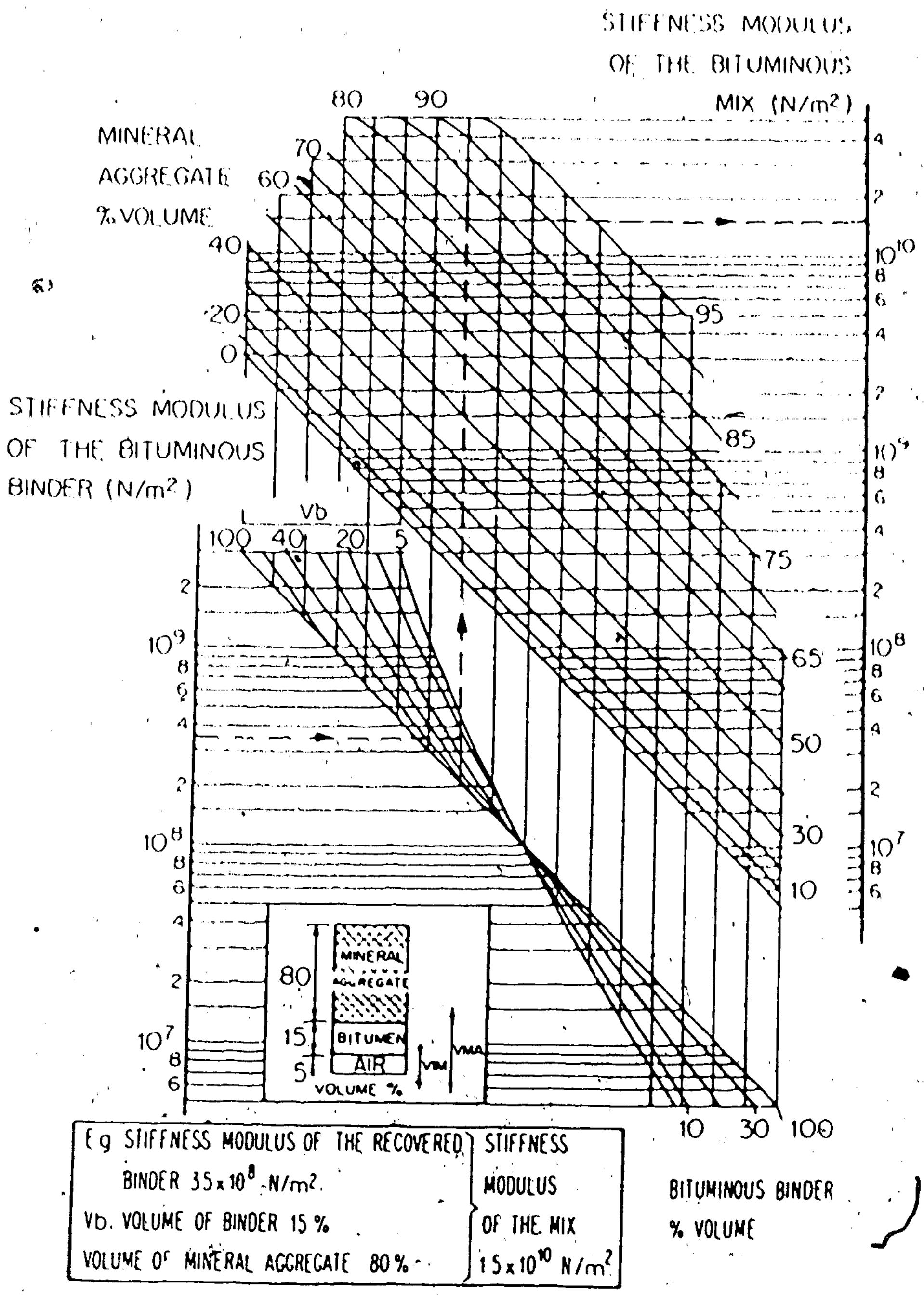


FIGURE 5.13 NOMOGRAPH FOR PREDICTING THE STIFFNESS OF ASPHALT CONCRETE MIXTURES.

predicted using McLeod's version of Van der Poel's nomograph.

5.2.4.1 Asphalt Concrete Mix Stiffness Results Using An Indirect Approach

To evaluate the asphalt concrete mix stiffness, the binder stiffness has to be determined first. For various mixes in this study with different recycling ratios (R/V), the actual percentage of the recycled asphalt cement in the blend (r) was determined. From Figures 5.5 to 5.11 the stiffness of the binder with the selected r value corresponding to the desired recycling ratio was then evaluated. This is illustrated in Table 5.3.

The next step in calculating the mix stiffness is to determine the volume concentration of the aggregate. Here, since all the various mixes contain a percentage of air voids higher than 3 percent, a corrected volume concentration of the aggregate had to be calculated using equation (5.13).

Knowing the stiffness of the binder and the corrected volume concentration of the aggregate, equation (5.12) or Figure 5.12, can then be used for estimating the stiffness of the asphalt concrete mixture. Table 5.4 shows the calculated stiffness for mixtures having different recycling ratios at various temperatures.

As can be observed from Table 5.4, temperature has a remarkable influence on the stiffness of the recycled asphalt concrete mixtures. The higher the temperature, the lower the stiffness. For example, stiffness may vary from

TABLE 5.3

CALCULATED BINDER STIFFNESS AT VARIOUS TEMPERATURES
AT THE RATIOS USED IN MIXTURE DESIGN

r	Binder Stiffness, kPa At Temperatures of:						
%	-30 C	-20 C	-10 C	0 C	10 C	20 C	30 C
0	5.8E+03	4.2E+02	2.8E+01	2.1E+00	1.8E-01	2.0E-02	2.0E-03
35	1.7E+04	1.8E+03	1.2E+02	9.7E+00	7.6E-01	3.9E-02	4.9E-03
56	3.0E+04	3.8E+03	3.0E+02	2.3E+01	1.8E+00	8.8E-02	9.8E-03
73	4.8E+04	7.6E+03	6.1E+02	4.7E+01	3.5E+00	2.9E-01	2.5E-02
92	1.1E+05	2.1E+04	1.4E+03	1.1E+02	7.6E+00	8.8E-01	5.9E-02
56 SC3000	1.3E+04	1.3E+03	6.2E+01	5.2E+00	4.1E-01	2.5E-02	2.5E-03

TABLE 5.4

CALCULATED MIX STIFFNESS AT VARIOUS TEMPERATURES

r	Mix Stiffness, KPa At Temperatures of:						
%	-30 C	-20 C	-10 C	0 C	10 C	20 C	30 C
0	1.5E+06	3.4E+05	3.4E+04	6.6E+03	6.0E+02	7.6E+01	2.1E+01
30	5.0E+06	1.1E+06	1.9E+05	3.4E+04	4.8E+03	5.7E+02	9.7E+01
50	6.9E+06	1.9E+06	3.4E+05	5.7E+04	8.3E+03	1.1E+03	1.4E+02
70	7.6E+06	2.3E+06	3.7E+05	6.6E+04	9.0E+03	2.1E+03	1.4E+02
100	8.3E+06	3.2E+06	4.3E+05	6.9E+04	9.0E+03	2.1E+03	1.2E+02
50 SC3000	1.9E+05	5.0E+04	3.5E+03	4.8E+02	6.9E+01	7.0E+00	2.1E+00

1.2×10^2 kPa at 30°C to 8.3×10^6 kPa at -30°C for 100 percent recycled asphalt concrete mixture.

Stiffness is also a function of the recycling ratio. The regression analysis has shown that the coefficient of determination (r^2) for the mix stiffness and the recycling ratio varies from 59.5 to 83.9 percent. This may not imply a very strong correlation between the two parameters. However, Figure 5.14 shows that an increase in the recycling ratio results in an increase in the mixture stiffness. It should be noted that this increase is more obvious at a lower range of temperatures. This phenomenon suggests that the recycled asphalt concrete pavements may be more susceptible to low temperature cracking. The 50 percent recycled mixtures with SC-3000 as virgin binder have shown even lower stiffness values than the conventional mixtures.

In the following chapter the stiffness of the mixtures at low temperatures will be measured directly by the use of the tensile splitting test.

5.3. Summary

The concept of stiffness, to represent the behaviour of asphalt materials, was introduced in this Chapter.

Different approaches to determine the stiffness of asphalt cements and asphalt concrete mixtures, using direct testing or indirect estimation were discussed. Indirect estimation approaches were chosen in this chapter to determine the stiffness of the asphalt cements and asphalt

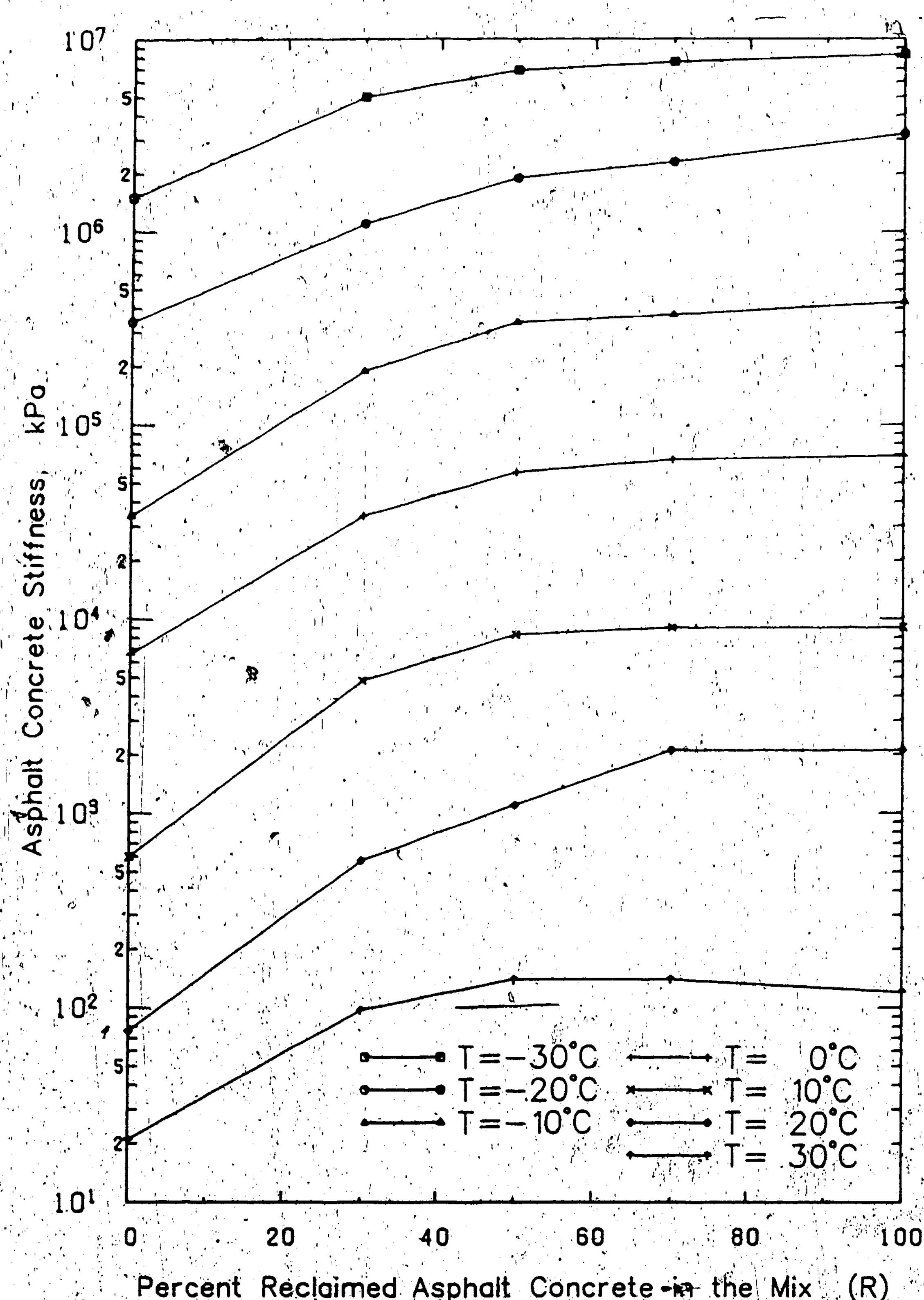


FIGURE 5.14 RELATIONSHIP BETWEEN MIX STIFFNESS AND PERCENT RECLAIMED MATERIAL IN THE MIX AT VARIOUS TEMPERATURES.

concrete mixtures.

Stiffness values were calculated for various virgin and recycled blends at different temperatures before and after the Thin-Film Oven test. It was found that the binder stiffness is directly proportional to the percentage of the recycled asphalt cement in the blend, and is significantly affected by temperature.

Stiffness values were also calculated for the asphalt concrete mixtures having different recycling ratios. A range of temperatures varying from -30°C to +30°C were used in the calculations. Results demonstrated that the value of mixture stiffness increases as the percent recycling increases. This is shown to be more pronounced at lower temperatures.

In general, it has been shown that the recycled asphalt cement and the recycled asphalt concrete mixtures exhibit higher stiffness values than conventional materials.

6. ASPHALT CONCRETE PAVEMENT LOW TEMPERATURE CRACKING

6.1 Introduction

Flexible pavement cracking due to thermal effects is a very serious and costly pavement distress mechanism in many locations with relatively cold climates.

The low temperature asphalt pavement cracking, otherwise known as non-load-associated cracking, shrinkage cracking, and thermal cracking is a widespread problem in Canada and the northern United States where low temperatures are experienced.

Low temperature cracking of flexible pavements is primarily caused by low winter temperatures that induce tensile forces in the asphalt concrete. If the induced tensile forces exceed the tensile strength of the material, cracks will be formed. Because the pavement cannot predominantly contract in the longitudinal direction, most low temperature cracks are formed in the transverse direction to the roadway route.

The cracking is not in itself detrimental. It is the loss in serviceability that is associated with cracking that is of concern. For a period after the occurrence of low temperature transverse cracking, the riding quality of the pavement may not be affected. With time however, many types of pavement failure generally develop. The intrusion of water causes loss of load-carrying capacity of the base and subgrade and reduces the service life of the pavement.

Considerable annual maintenance is required for crack filling and repairs, and because the pavement riding quality can deteriorate very rapidly, resurfacing on many sections of roadway is essential before the normal time indicated by the structural design. Hence, the presence of transverse cracking can result in a loss of pavement performance, service life, and an increase in maintenance cost.

A variety of factors such as climatic effects, subgrade type, asphalt and mix properties, pavement design, pavement age, and traffic effects, are known to influence the rate of formation and extent of low temperature pavement transverse cracking. Among these factors asphalt properties have been recognized to be the most influential and have received the most attention.

A major objective of this chapter is to characterize the behaviour of the recycled asphalt concrete pavements in low temperatures and to introduce the new concept of recycling into the design procedure of pavements exposed to low winter temperatures.

6.2 Mechanisms of Low Temperature Cracking

The occurrence of low temperatures induces tensile stresses in the pavement materials. When the tensile stresses caused by contraction exceeds the tensile strength of the material, fracture occurs.

Because of pavement geometries, the principal axis of contraction is in the longitudinal direction and, hence, the majority of low temperature cracks are in the transverse direction.

The cyclic temperature effects will result in a gradual increase in crack opening. The normal crack interval is about 6 to 10 meters, however, the spacing may vary from location to location.

In the literature (60-66) several mechanisms for low temperature cracking have been suggested:

(i) Stresses induced by pavement thermal shrinkage result in surface cracking that propagates through the asphalt concrete layer. The cracking may be initiated by sudden thermal shock or low-frequency temperature cycling at low temperatures.

(ii) Stresses in the non-asphalt treated base layer can cause transverse cracks which ultimately reflect through to the surface.

(iii) Transverse shrinkage cracks occur in the subgrade (caused by moisture and/or temperature variations) and can propagate through the pavement structure by differential movement and be reflected in the pavement surface.

(iv) Non-uniformities in the subgrade can cause differential frost-heaving of the subgrade resulting in pavement surface cracking.

There is an uncertainty about the exact mechanism of crack formation. Among all the afforementioned categories only (i) is directly associated with pavement binder and mix components and consequently is of primary concern to this study.

There are several methods that express the intensity of transverse cracking distress. One of the most widely used approaches is that recommended by the Ontario Department of Transportation (67). This method used the concept of a crack index I. Figure 6.1 illustrates the various categories of transverse cracks that might exist. The crack index is defined as in equation

$$I = N_M + N_F + \frac{1}{2} N_H \qquad (6.1)$$

In the equation N_M is the number of cracks of the "multiple" type, N_F is the "full" type, and N_H is of the "half" type. All cracks are per 152 m (500 feet) of 2-lane pavement.

An assessment of the low temperature cracking of conventional and recycled asphalt concrete requires a knowledge of the thermal regimes where the pavement is located, low temperature stiffness and strength properties of the pavement constituents, and a model by which the low temperature responses of pavement material can be evaluated. These factors will be discussed in the succeeding sections.

6.3 Background History

Low temperature cracking distress is extensive in many parts of Canada and the United States, and can cause a

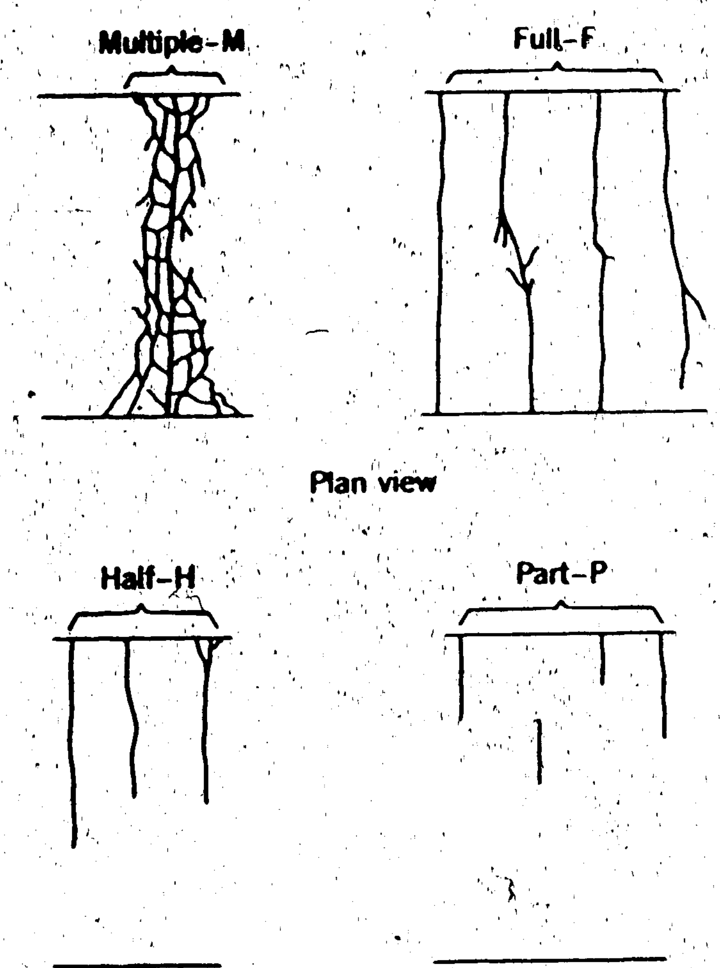


FIGURE 6.1 DIFFERENT TYPES OF PAVEMENT TRANSVERSE CRACKS.

drastic reduction in the service life of the pavements.

During the past two decades, a number of highway agencies, producers and others have worked actively toward finding a practical solution to prevent or minimize this type of distress.

Low temperature cracking distress of asphalt concrete pavement was recognized in the mid-1930's. Rader (68) published his paper on the physical properties of asphaltic materials at low temperature in 1935. Due to the limited number of pavement roadways, small traffic volumes, and lighter loads not much attention was paid to this pavement distress type prior to World War II. This problem became acute in Canada and the United States during the mid-1950's and the early 1960's when the highway network was rapidly expanding and the pavement performance requirements increased. Highway departments in Western Canada and Ontario became concerned over low temperature pavement cracking and conducted cracking surveys to determine the extent of the problem. Research investigations were also started by many agencies both in the field or laboratories, to find the cause and mechanism of this type of distress and ~~to~~ attempt to control it. Some of the first results of this early work was reported by Shields (69), Shields and Anderson (70), Domaschuk et al. (71). The Alberta Studies were summarized by Anderson et al. (72). A symposium on non-load associated cracking (73) was sponsored by the Association of Asphalt Paving Technologists in 1966.

During the mid-1960's many field test sections were designed and constructed in Canada to study the low temperature behaviour of asphalt concrete pavements. These test sections will be briefly discussed in a later section. In 1969 an ad hoc committee was established within the Canadian Good Roads Association to look into the low temperature cracking problems in Canada and to document the design approaches available to control this problem. The outcome of this comprehensive review was presented by Haas (74) in 1970.

An investigation program on minimizing premature cracking in asphalt concrete pavement was sponsored by the American Association of State Highway and Transportation in cooperation with the Federal Highway Administration. The result of this study presented by Finn et al. (75) in NCHRP Report 195 in 1978. This report contains recommendations for materials specification, mix design, structural materials selection, and construction requirements that are intended to reduce the possibilities of premature fatigue and low temperature cracking of asphalt pavements.

In 1981 the Asphalt Institute released a research report on design techniques to minimize low temperature asphalt pavement transverse cracking (76). This state-of-the-art report was prepared by the Asphalt Institute's ad hoc committee on low temperature asphalt pavement stiffness. The report's intention was to critically appraise the factors influencing pavement transverse cracking, and to re-appraise conclusions from earlier studies, toward providing pavement design engineers with clear guidelines on means of reducing low temperature pavement cracking.

Highway agencies and universities in Japan (77-82) have also conducted field and laboratory investigations in the field of low temperature pavement cracking since the late 1970's and have achieved considerable progress in design techniques and construction procedures to control this problem.

Research activities are continuing in this field by many organizations in Canada, the United States and Japan.

6.4 Description of the Canadian Test Sites

A vast amount of data on low temperature pavement cracking has been gathered and analysed from the laboratory experiments and test site observations and measurements in Canada. A brief description of the test sections and some of their overall findings are given in this section.

One of the earliest experiments in Canada was at Arkona in Southern Ontario in 1960. Three test sections were constructed on a clay subgrade. All test sections were similar in all respects, except for asphalt supplier. A 85-100 penetration grade asphalt was used. The freezing index for this area is only 650 degree days, which is exceeded in most areas of Canada. After eight years of service, sections constructed with a low viscosity asphalt showed very severe cracking whereas sections with medium and high viscosity asphalt exhibited very low amounts of cracking. Detailed descriptions and results of this test section are given by Haas et al. (83) and McLeod (84).

Another early experiment was conducted in Saskatchewan in 1963. Three test sections were constructed with different asphalt source and grade. The major finding was that the asphalt source was related to the degree of cracking, and also it was found that softer asphalt resulted in fewer cracks. This work is reported by Culley (85) in detail.

In 1966 Alberta undertook an extensive experimental program on the behaviour of asphalt concrete pavements in low temperature. Three test sections were constructed with different sources of 200-300 penetration grade asphalt cement. These three asphalt sources represented high, intermediate, and low viscosity materials as measured at 60°C and were obtained from major suppliers in Alberta. The asphalt concrete surfacing was prepared from a single aggregate source for all test sections, the sole major surfacing variable was the source of the asphalt cement. A considerable amount of data on the original and recovered material properties, structural capacity, crack initiation and crack frequency was obtained. Temperature measurements were obtained by means of a continuously recording multi-position thermograph used in conjunction with thermistors placed at various depths in the pavement structure. One of the major findings of this study was that the section with the low viscosity grade asphalt cracked earlier. This provided the basis for a revision to Alberta specification for asphalt cement. Description and preliminary results of this project is given by Shields et al. (86).

The Manitoba test project, more commonly referred to as the Ste. Anne Test Road, has been one of the most comprehensive full-scale experiments to study the low temperature cracking in asphalt concrete pavements. The test road was constructed in 1967. It consists of twenty-nine 122 m (400-feet) pavement sections constructed on clay and sand subgrade, with a variety of materials and structural variables incorporated into the experiment. The primary variables included were: three pavement structures, four different asphalt binders, two asphalt contents, and two aggregate gradation. The asphalts used were: low viscosity 150-200 penetration grade, high viscosity 150-200 penetration grade, low viscosity 300-400 penetration grade, and high viscosity SC-5 liquid asphalt. Crack surveys were conducted and continuous measurements of air and pavement temperatures were obtained by means of thermocouples positioned at various depths through each of the three pavement structures. Crack detection circuits were used in an attempt to define initial times of cracking. Records of the minimum daily temperatures within the pavement structure indicated that most of the transverse cracking resulted from the effect of prolonged low temperature during which the asphalt concrete cooled gradually throughout its thickness and reached a low temperature level. The transverse crack detection circuits revealed that some cracks initiated at the pavement surface and progressed downward through the asphalt concrete. Also in this study some structural capacity measurements such as Benkelmen Beam rebounds, plate bearing, and Shell vibratory tests were taken as well.

As also found in other experiments, the effect of asphalt type was profound, with the high viscosity 150-200 penetration grade asphalt showing superior performance. The effect of sand subgrade was also very marked. Detailed coverage of the information and analysis of this test road is provided by Young et al. (87) and Deme et al. (88). Christison (89) has developed a computer base prediction model for low temperature cracking. Christison et al. (90) have compared the observed and the predicted fracture temperatures of some of the sections of the Alberta and the Ste. Anne test roads using the prediction model.

Field inventories, the full-scale test sections, and laboratory investigations have shown that the most significant variable involved in low temperature cracking of asphalt concrete pavements is the nature of the asphalt. However this type of pavement distress is a complex phenomenon and other variables can be highly important in certain situations.

6.5 Factors Affecting Pavement Transverse Cracking

Several variables are known to significantly influence the occurrence of low temperature cracking in asphalt concrete pavements. Figure 6.2 shows a comprehensive list of potential variables that may affect low temperature cracking. Generally the situation in nature is very complex, however the mechanism and causative factors for surface cracking are better understood than cracking mechanisms occurring within the other pavement layers.

Some factors which have been found to be of great significance in low temperature pavement cracking problems are reviewed in the following sections.

6.5.1 Climatic effects

It should be noted that, severe winter climatic conditions are the major cause of pavement transverse cracking. Test road results demonstrated that most low temperature cracks were initiated when the temperature decreased to a certain level for a certain time period.

The pavement surface temperature which is directly related to pavement low temperature transverse cracking, is dependent on the ambient air temperatures. It has been shown that a correlation exists between the density and severity of pavement transverse cracking, and the lowest ambient air temperature.

The two field studies in Western Canada, the Alberta and Ste. Anne test projects, provided valuable surface and subsurface temperature records in several asphaltic concrete pavement structures exposed to climatic temperatures approaching a low of -40°C.

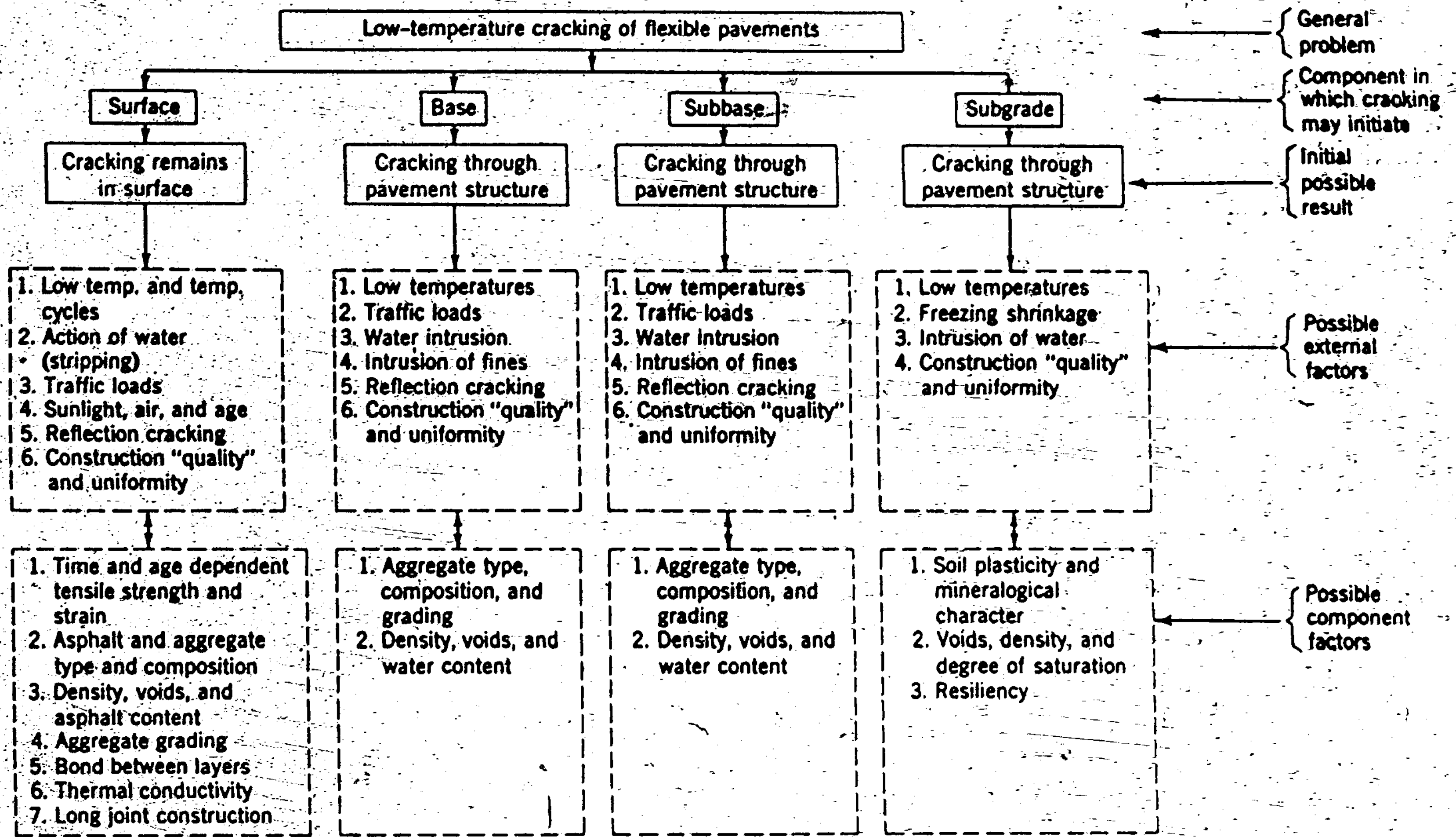


FIGURE 6.2 FACTORS OF POSSIBLE SIGNIFICANCE IN LOW TEMPERATURE CRACKING OF FLEXIBLE PAVEMENTS.

Christison (89) has shown a long-term daily mean minimum and maximum air temperatures, determined from meterological records for the Edmonton area in Figure 6.3. It can be seen that for approximately five months of the year the daily mean minimum air temperatures are below 0°C. The frequency of extreme maximum and minimum monthly air temperatures for the Edmonton area, were also presented by Christison. From his findings, it can be deduced that extreme temperatures of 35°C (95°F) and -45°C (-50°F) are expected for the Edmonton area and hence, pavements should be designed accordingly to prevent permanent deformation and low temperature cracking, which are two major modes of pavements distress.

It should be realized that asphalt concrete pavement temperatures are generally higher than air temperatures during periods of extreme cold. Christison has developed a relationship between daily air and asphalt concrete temperature variations with depth, and between daily minimum air temperatures and asphalt concrete pavement temperatures, using linear regression analysis. Figure 6.4 presents these relationships for pavement temperatures at various depth versus minimum air temperatures.

Historical weather information for a particular area then proved to be useful to estimate the minimum pavement surface temperatures. This information is available on maps delineating regions according to temperature zones, either in monthly minima or in freezing indices.

Winter climatic conditions in Canada are usually

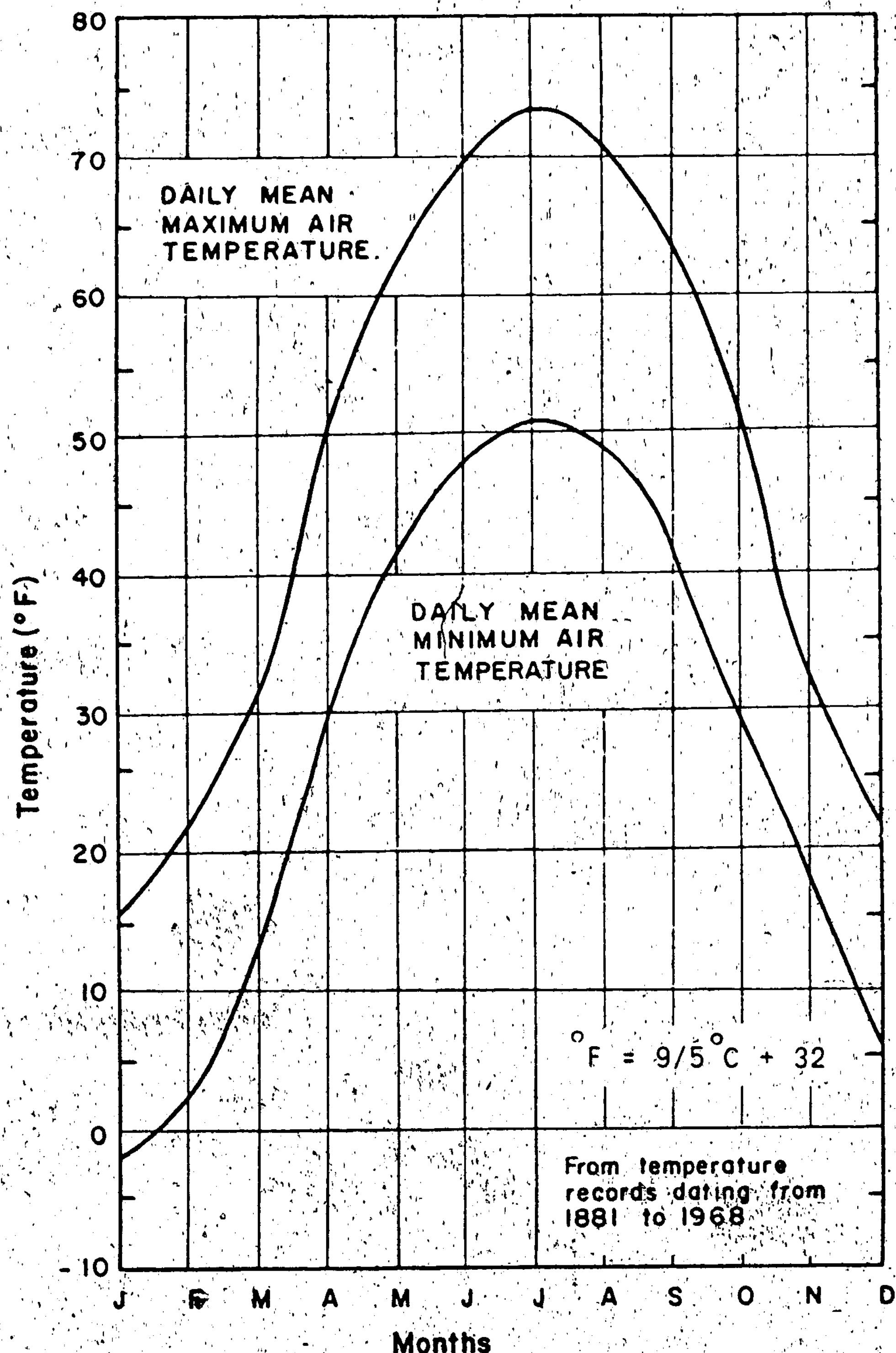


FIGURE 6.3 DAILY MEAN MAXIMUM AND MINIMUM AIR TEMPERATURE FOR EDMONTON, ALBERTA. (After Ref. 89)

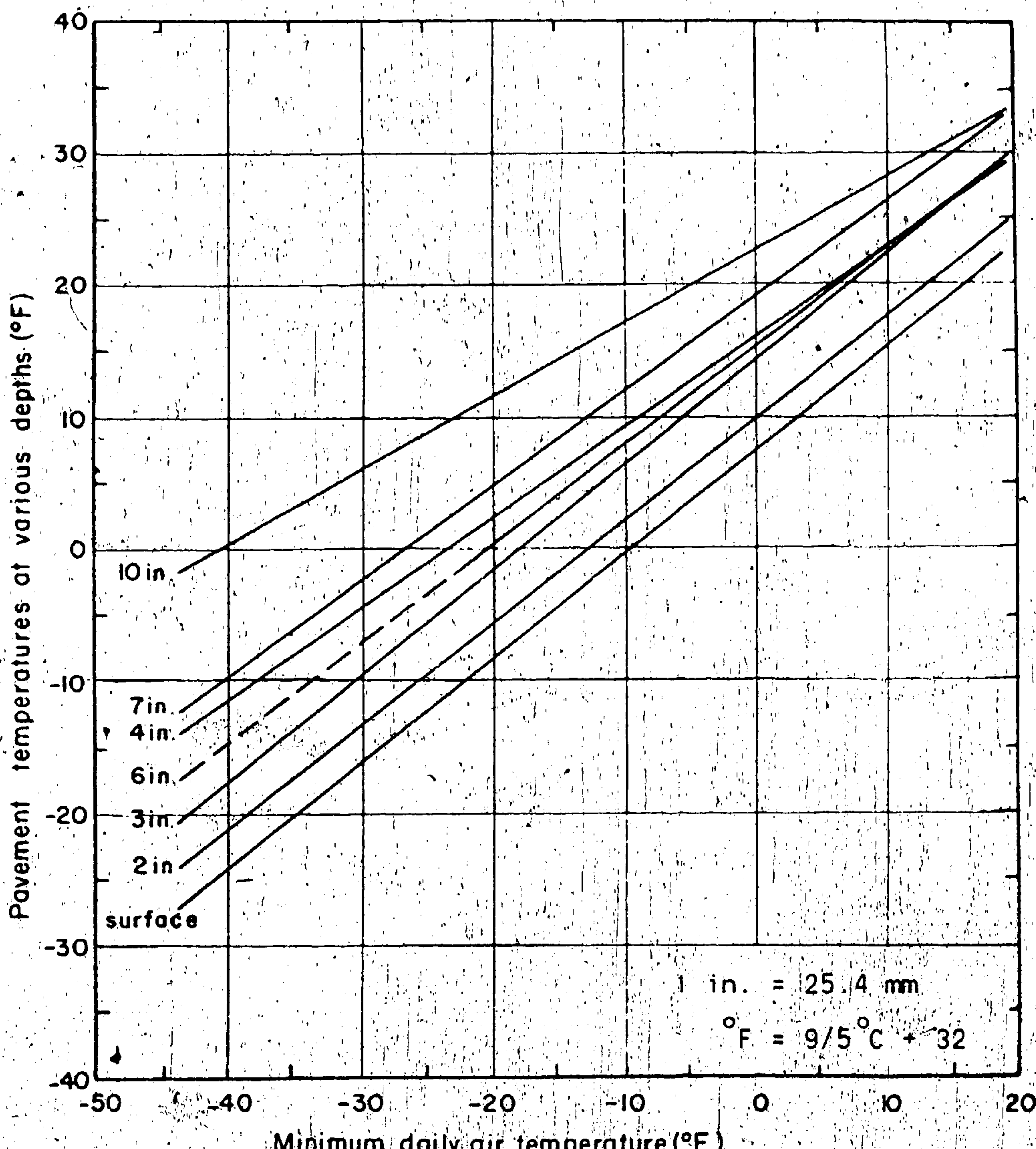


FIGURE 6.4 ASPHALT CONCRETE PAVEMENT TEMPERATURES AT VARIOUS DEPTHS VERSUS MINIMUM DAILY AIR TEMPERATURES. (After Ref. 89)

described by a freezing index. Figure 6.5 shows a map of Canada illustrating freezing indices in degree F-days.

Hajek et al. (91) have developed a relationship between the air freezing index and winter design temperatures and is presented in Figure 6.6. This covers a range of freezing indices from 500 to 3000 degree F-days. Extrapolation has been used for lower freezing indices. Figures 6.5 and 6.6 can be used together in order to obtain a good estimate of winter design temperature. Using these two figures a winter design temperature of approximately -40°C (-40°F) for Central Alberta is suggested.

6.5.2 Asphalt Properties

One of the most significant variables to affect low temperature cracking of asphalt concrete pavements is the asphalt properties used in the surface layer. Transverse cracks may be reduced or eliminated by the use of softer asphalts exhibiting low temperature susceptibility.

Asphalt characteristics such as source, grade, temperature susceptibility, ductility, and stiffness are of recognized significance in affecting pavement transverse cracking.

The crude source of the asphalt cement is an important variable associated with the asphalt concrete cracking frequency. Results of the field test projects reported by Shields et al. (86) have shown that pavement structures which have similar properties but are composed of asphalt cement

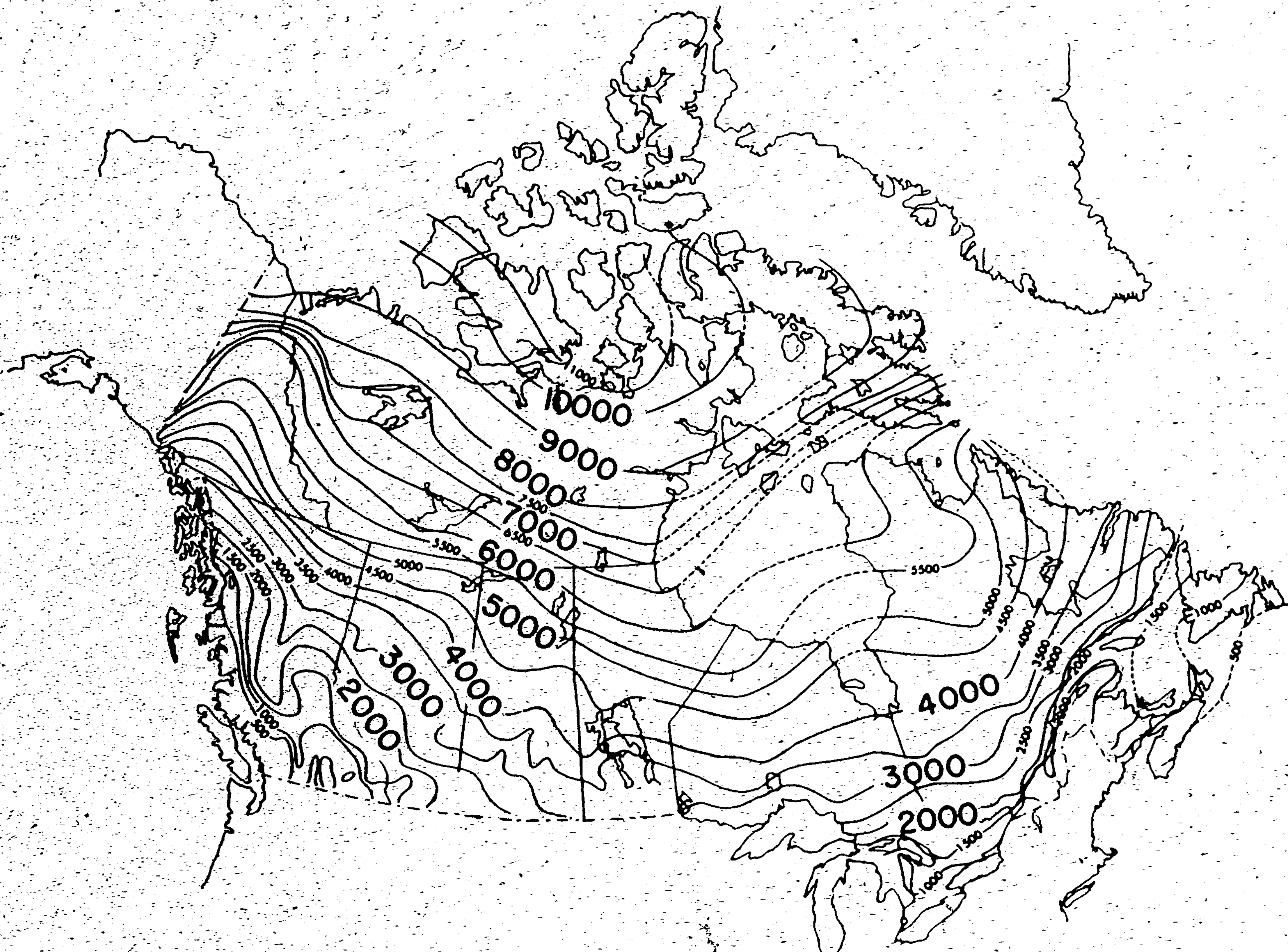


FIGURE 6.5 MAP OF CANADA ILLUSTRATING FREEZING INDICES. (After Ref. 64)

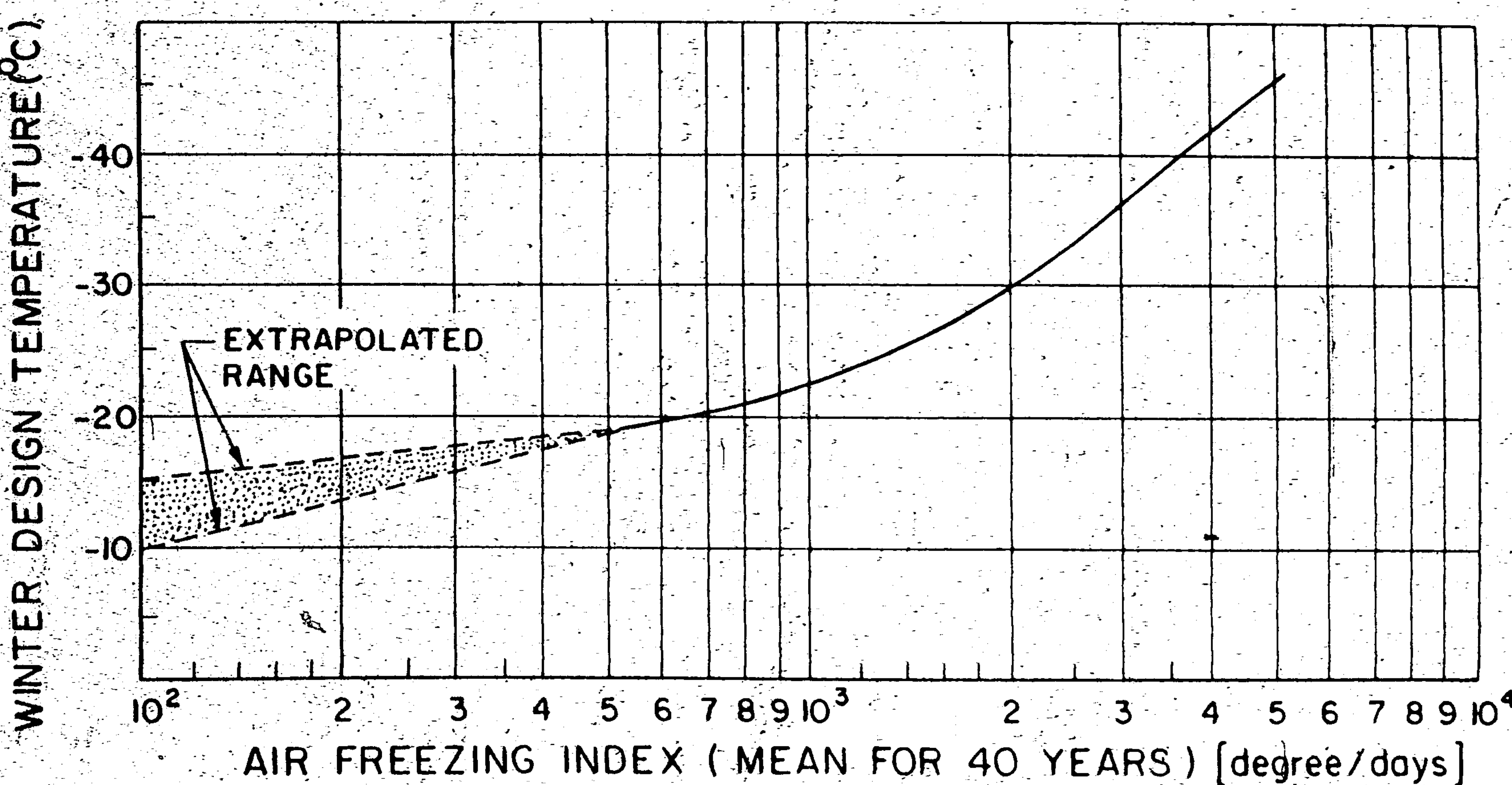


FIGURE 6.6 RELATIONSHIP BETWEEN FREEZING INDEX AND WINTER DESIGN TEMPERATURES. (After Ref. 64)

from different sources, exhibited different crack frequencies. A relationship may exist between the asphalt source and occurrance of the low temperature cracking. Asphalt cement of the same grade, either based on penetration or viscosity, but from different sources can therefore have significant different low temperature consistencies, and hence different low temperature cracking trends.

Asphalt grade is another major factor influencing the low temperature cracking susceptibility of asphalt concrete pavements. Asphalt penetration at 25°C and viscosity at 60°C are currently the main properties used to grade asphalt cements. It has been observed that, in general, softer grade asphalts have less potential for low temperature cracking. However Young et al. (87) have reported that in some cases the softer asphalt has a greater tendency to crack than a harder grade from a less temperature-susceptible source. Similar findings have also been reported by Gaw et al. (92) and Burgess et al. (93).

From the results of the field tests, it was found that asphalt concrete pavements composed of similar penetration grade asphalt but having different viscosities exhibited significantly different crack frequencies. Asphalt concretes composed of the lower viscosity asphalt were found to have a higher cracking frequency. Anderson et al. (94) have reported that, on the basis of the extensive survey of crack occurence and road test results, the Department of Highways of Alberta revised its asphalt cement specification in

1967. The new specification was restricted to a minimum penetration of 250 at 25°C with a minimum viscosity of 275 Poises at 60°C. Experience with this specification which restricted the use of low viscosity asphalt cements at a given penetration, resulted in a more uniform material supply and the reduction of abnormally high transverse crack frequencies.

Single measurements of asphalt consistency, that is, either penetration at 25°C or viscosity at 60°C, without having appropriate means for measuring variation in asphalt temperature susceptibility are not appropriate for controlling pavement transverse cracking.

<u>Temperature Susceptibility</u> of asphalt, which is the change in its consistency with temperature, is an important factor to be taken into consideration for the design of pavements exposed to low temperatures. It is dependent on both the penetration and the viscosity of the asphalt, and gives a good indication of the low temperature cracking susceptibility of the asphalt concrete pavements.

<u>Ductility</u> has also been used by some investigators as a criterion to control low temperature pavement cracking. In spite of the presence of a correlation between ductility and pavement cracking in some cases, a study by Schmidt (95) has shown that ductility is not an appropriate measure of low temperature cracking if a variety of asphalts are present.

<u>Asphalt Stiffness</u> has been related to the performance of asphalt concrete pavements at low temperature by many

investigators.

Stiffness, as defined by Van der Poel (38), is a time and temperature dependent ratio of tensile stress to strain of asphalt materials subjected to various loading conditions. As described in Chapter 5, there are several methods to be used for obtaining asphalt and mix stiffness. The methods for determining the stiffness of asphalt cements and asphalt concrete mixtures at low temperatures may be classified as: direct methods, based on direct testing of the materials; and indirect methods that estimate the stiffness modulus of the materials by using the rheological properties at higher temperatures measured by direct standard tests.

It has been observed that pavements constructed with asphalts of higher stiffness modulus exhibited higher low temperature cracking frequencies. Low temperature asphalt stiffness has been demonstrated to correlate well with pavement transverse cracking and has served as the basis for most recent low temperature pavement design procedures.

6.5.3 Mixture Properties

The influence of some mixture properties, such as asphalt content, air voids, aggregate gradation and properties, and mineral fillers have not been exhaustively investigated with respect to low temperature pavement cracking. Nevertheless, these factors have an influence on mix stiffness and fracture strength and consequently might affect low temperature cracking.

Sugawara et al. (77) have examined the intervals and crack frequencies for a number of different types of surface courses including asphalt mortar, fine-graded asphalt concrete, dense-graded gapped asphalt concrete, and fine-graded gapped asphalt concrete. They have discovered that the number of cracks with intervals of less than 10 m is proportional to the amount of asphalt cement and filler in surfacings and increases in the order of: dense-graded gapped asphalt concrete, fine-graded gapped asphalt concrete, and asphalt mortar. Figure 6.7 illustrates intervals and crack frequencies for the mentioned surfaces.

Burgess et al. (96) by using the results of the Ste. Anne Test Road, have indicated that an asphalt content in the range of one percent below Marshall optimum to one-half percent above was not significant in affecting the pavement low temperature cracking frequency. They also concluded that the incorporation of cement filler to modify the aggregate gradation was not significant in affecting cracking in the range of contents studied.

These conclusions remain to be confirmed by further field and laboratory testing.

6.5.4 Pavement Structural Design and Properties

Major factors affecting low temperature asphalt concrete cracking related to pavement design can be summarized as subsurface material properties, pavement thickness, pavement age, and traffic effects.

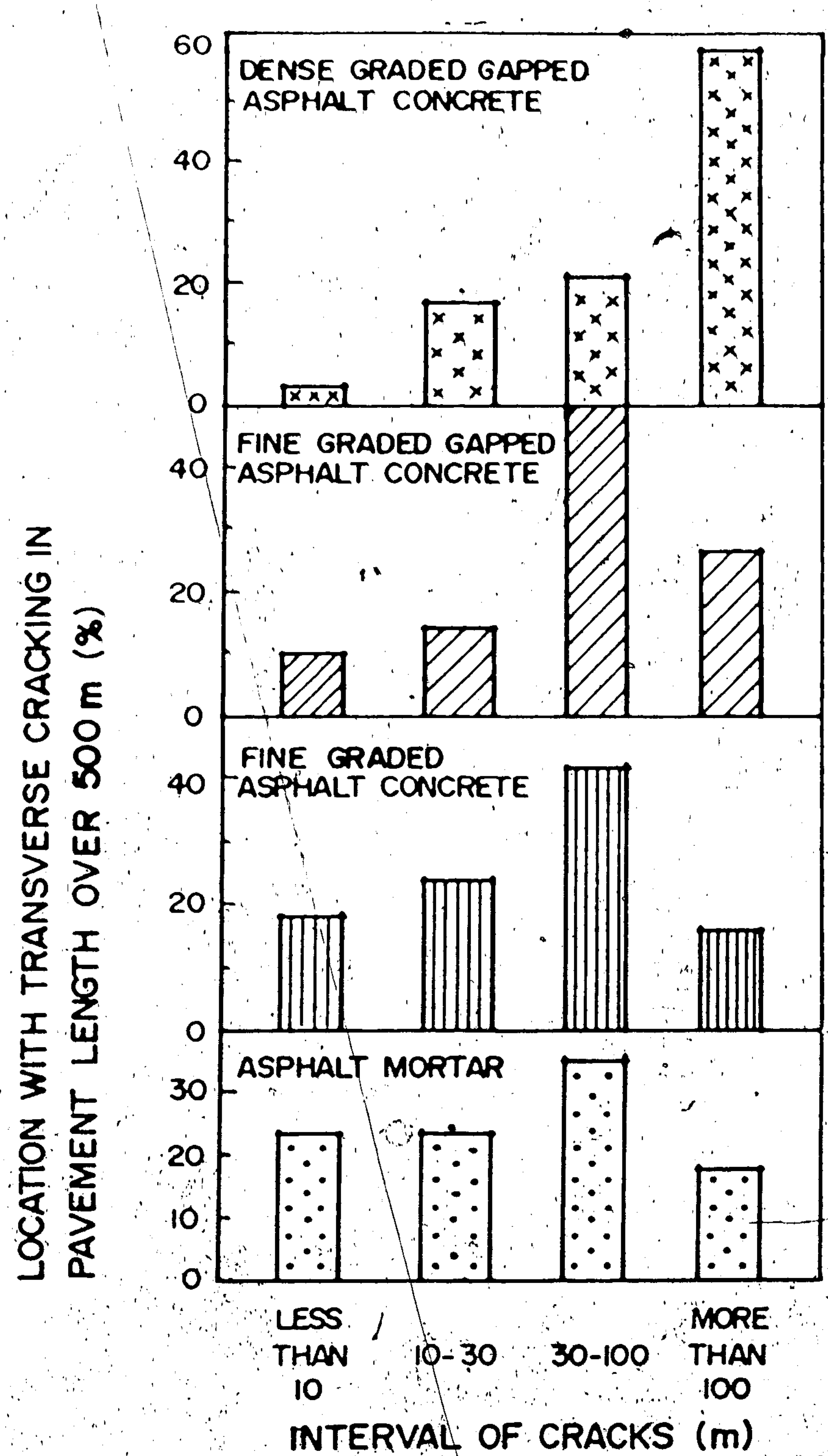


FIGURE 6.7 CRACK INTERVALS AND CRACK FREQUENCY OF VARIOUS TYPES OF PAVEMENTS IN JAPAN.

The subsurface components have received far less attention probably because of the overshadowing influence of the asphalt cement on pavement low temperature cracking. However, it has been indicated that the subgrade can have a significant effect on the severity of the pavement transverse cracking.

The Ste. Anne Test Road results demonstrated that the low temperature cracking frequency for asphalt pavements on sand subgrade is considerably higher than for pavements on clay subgrade, having identical pavement surface types.

The role of the base and subbase components in prompting or retarding transverse cracking has been taken into consideration less often, primarily, because low temperature transverse cracking usually begins in the surface of the pavement.

The effect of subsurface materials on the initiation and frequency of the low temperature cracking still need to be thoroughly investigated and quanitified.

Pavement thickness has some effect on the low temperature cracking of asphalt concrete pavements. The change in the thickness of the asphalt concrete layer may result in a change of several factors influencing thermally induced stresses in the material, such as temperature and stiffness gradients.

Young et al. (87) have presented data from the Ste. Anne Test Road, showing that low temperature pavement cracking is related to asphalt concrete layer thickness. They have shown that increase in the thickness of the asphalt concrete layer from 102 mm (4 inches) to 254 mm (10 inches), resulted in a reduction of the cracking frequency to one half, all other variables being constant.

Full-depth asphalt concrete pavements have been proven to be more resistant to transverse cracking than thin asphalt concrete pavements. However, it should be realized that if an asphalt layer is susceptible to cracking because of the environmental conditions in which it is placed, thickness has very little to do with the initiation of cracking.

Pavement age has also been recognized to have an effect on the intensity and severity of the low temperature pavement cracking. The increase of the cracking frequency can be due to an increase in the stiffness of the mix and also due to the fact that the probability of obtaining a low critical temperature increases with time. Haas (64) and Gaw et al. (97) have indicated that mix stiffness increases with pavement service due to asphalt hardening, hence making the material more susceptible to cracking.

Sugawara et al. (77), using data from a road survey in the Hokkaido area in Japan, revealed that a definite relationship exists between pavement age and cracking. They indicated that the increase in the frequency of low temperature cracks with the age of a pavement can be the result of a complex interrelationship between the thermal fatigue of pavements caused by repeated change in air temperature, hardening of asphalt cement and fatigue due to traffic loads. Extensive cracking surveys in Canada reported by Kathol (98) and Kingham (99) have also demonstrated that cracking frequency increases with pavement age.

Traffic effects on low temperature cracking have received less attention than other factors. Traffic effects are more commonly related to pavement fatigue failure, but they also may influence pavement low temperature conditions, asphalt properties, and subsurface materials are more important factors affecting initial as well as subsequent crack frequencies.

6.6 Existing Design Approaches for Low Temperature Cracking

A design procedure for asphalt concrete pavements to control low temperature cracking should take into consideration all significant variables, including asphalt properties, asphalt and mix stiffness, subsurface material properties, pavement age, pavement thickness, and traffic effects. It is desirable to establish some simple and rational pavement design approach as an interim solution until modified procedures are developed which consider all of the significant variables.

Several methods are available for design against low temperature cracking of asphalt concrete pavements. However, because of the complexity of the pavement cracking problem these methods usually over-simplify the pavement cracking mechanism, and none of them is able to take into consideration the significant variables in a completely realistic manner. As a result, no one method is preferable above all others. Comprehensive field and laboratory data are still required to correlate the pavement cracking behaviour with pavement variables.

Existing alternate ways of design against asphalt concrete pavement low temperature cracking may be grouped into the following categories:

(i) Setting a limiting specification for asphalt cement chosen as penetration at 25°C and viscosity at 60° or 135°C, based on field observations and test results.

(ii) Setting stiffness values for the asphalt cement or asphalt concrete that do not exceed limiting criteria for a particular temperature regime, and selecting the asphalt cement to be used on this basis.

(iii) Predicting pavement fracture temperature from asphalt or mix stiffness under simulated field cooling and restraint conditions, and comparing it to expected low temperatures in the field.

(iv) Estimating the cracking frequency of asphalt concrete pavements at various ages based on correlations for specific constructions compared with tolerable cracking frequencies for particular roadway functions.

The aforementioned approaches have their own advantages and disadvantages. The first two approaches have an advantage in that they can easily be incorporated into specifications and therefore, they can be used more conveniently. The last approach allows a certain amount of cracking compatible with the service requirement, whereas, the first three approaches indicate that either cracks will occur or will not occur.

A brief review of the above general approaches are given in the following sub-sections.

6.6.1 Setting Limiting Specification Approach

This method is among the initial methods of analysing low temperature pavement design. It developed after field inventories and early road test results started to show the significance of asphalt cement in low temperature cracking.

Adjustments to specifications were through the use of softer grades of asphalt, based on penetration at 25°C and/or increasing the minimum viscosity levels.

Specifications were modified by several agencies. Saskatchewan specified a 150-200 penetration grade asphalt prior to 1963, but changed to AC-4, AC-5, and AC-6 grades with minimum and maximum penetration at 25°C and viscosity at 60°C requirements. Ontario specified a fairly high minimum viscosity requirement at 135°C, and retained 60-70, 85-100, 150-200, and 300-400 penetration grades. The Alberta specification called for a minimum penetration of 250 at 25°C, with a minimum viscosity of 275 Poises at 60°C.

In general, low viscosity asphalt cement, based on viscosity at 60°C, and hard grade asphalt, based on penetration at 25°C, are the least desirable when designing against low temperature cracking. This indirect approach to design against thermal cracking has shown some degree of success. However, the long-term effect of softer asphalts on other aspects of pavement performance such as permanent deformation and fatigue have not been fully evaluated.

6.6.2 Setting Limiting Stiffness Approach

Asphalt cement stiffness is said to be a fundamental indicator of asphalt characteristics. Therefore, by placing limits on the asphalt or mix stiffness at a given temperature, low temperature cracking may be eliminated. This is a more direct design approach and is based on many field and test section observations as well as extensive laboratory work.

From the analysis of a variety of data, McLeod (100) has reached the conclusion that cracking will not occur if the mix stiffness is less than 6.9×10^6 kPa (1×10^6 psi) at 20,000 seconds loading time for the minimum encountered temperature. In this analysis a dense, well-graded mix is assumed and a slightly modified procedure is used to determine the asphalt stiffness and penetration index values. Table 6.1 summarizes McLeod's suggested mix stiffness for various temperatures to prevent low temperature cracking. This table shows the levels of stiffness at which cracking is expected and the levels at which cracking could be eliminated. The latter is a design guide which incorporates a safety factor to take into account the effect of hardening in service. McLeod has translated the Table 6.1

TABLE 6.1

MAXIMUM MIX STIFFNESS FOR SELECTING ASPHALT CEMENT GRADE

Min. Temp. at 2 Inch Depth	Stiffness Modulus, psi	
	Cracking Expected	Cracking Eliminated
-40 F	1,000,000	500,000
-25 F	700,000	300,000
-10 F	400,000	200,000
+10 F	100,000	50,000

1 in. = 25.4 mm

1 psi = 6.89476 KPa

°F = 9/5°C + 32

guide into a chart for selecting asphalt cement grade for various values of penetration indices as shown in Figure 6.8. This is based on a correlation between viscosity at 135°C (275°F) and penetration at 25°C (77°F).

Haas (101) has examined the stiffness implication of a variety of asphalt cement specifications and has compared the results with Mcleod limits. All findings indicated that Mcleod's limits are reasonable and can be used as an approximate design guide.

An asphalt cement design selection guide was also presented by Fromm et al. (102) based upon McLeod's modified nomograph approach. This guide limits asphalt cement stiffness to 1.4 x 10^5 kPa (20,000 psi) for 10,000 seconds loading time. Fromm's design selection guide is shown in Figure 6.9 for Thin-Film Oven test residues, using a minimum expected temperature on a one percent probability basis.

6.6.3 Predicting Pavement Fracture Temperature Approach

This approach is based on the postulated mechanism of cracking, that is, fracture occurs as induced thermal stresses exceed the tensile strength. This method predicts the thermally induced stresses within an asphalt concrete pavement subjected to a given temperature, using asphalt and/or mix stiffness values. A comparison is then made between the predicted stresses and the fracture strength-temperature relationship, and consequently the fracture temperature is predicted. In a way this method is a

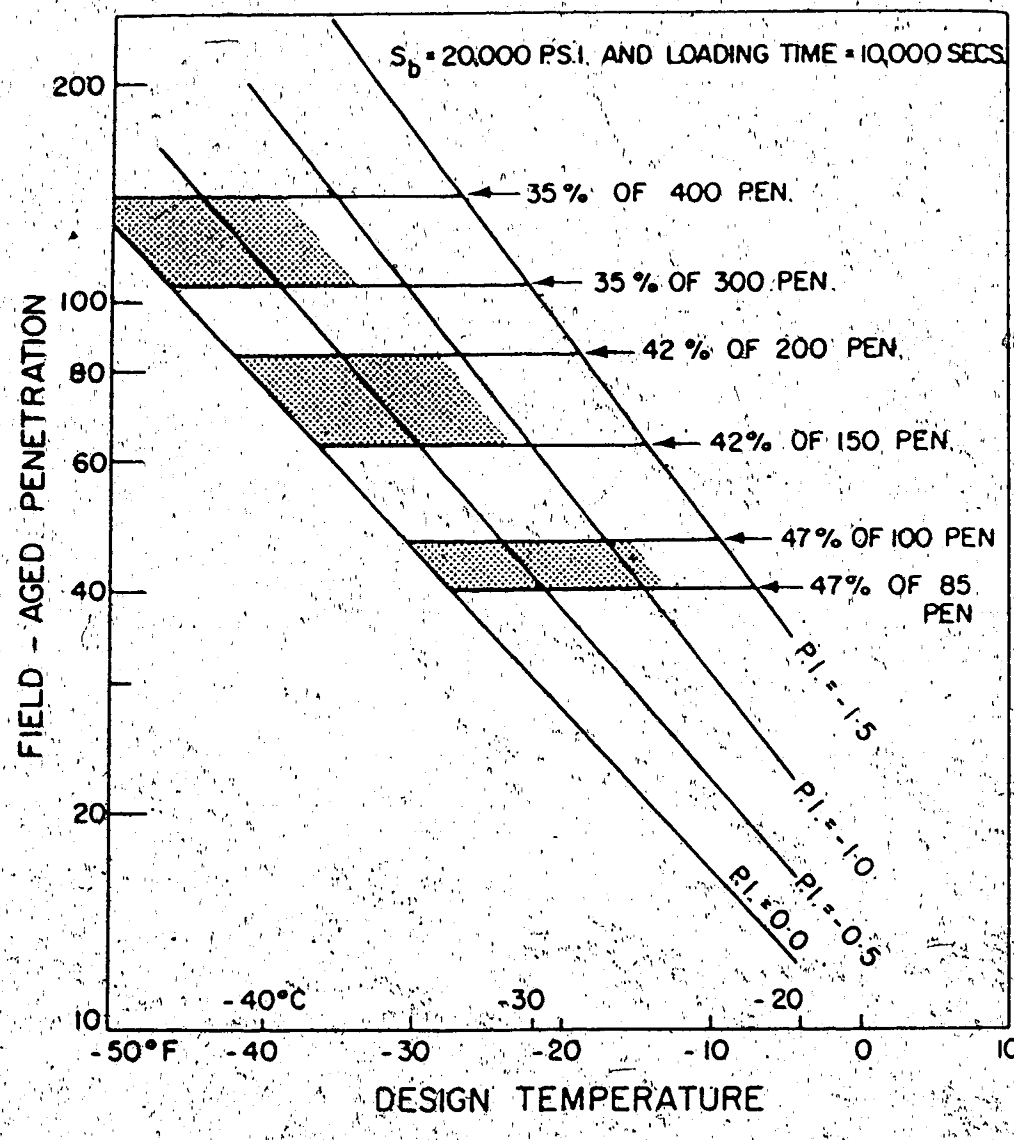


FIGURE 6.8 SELECTION OF ASPHALT CEMENT GRADE FOR VARIOUS DESIGN TEMPERATURES.

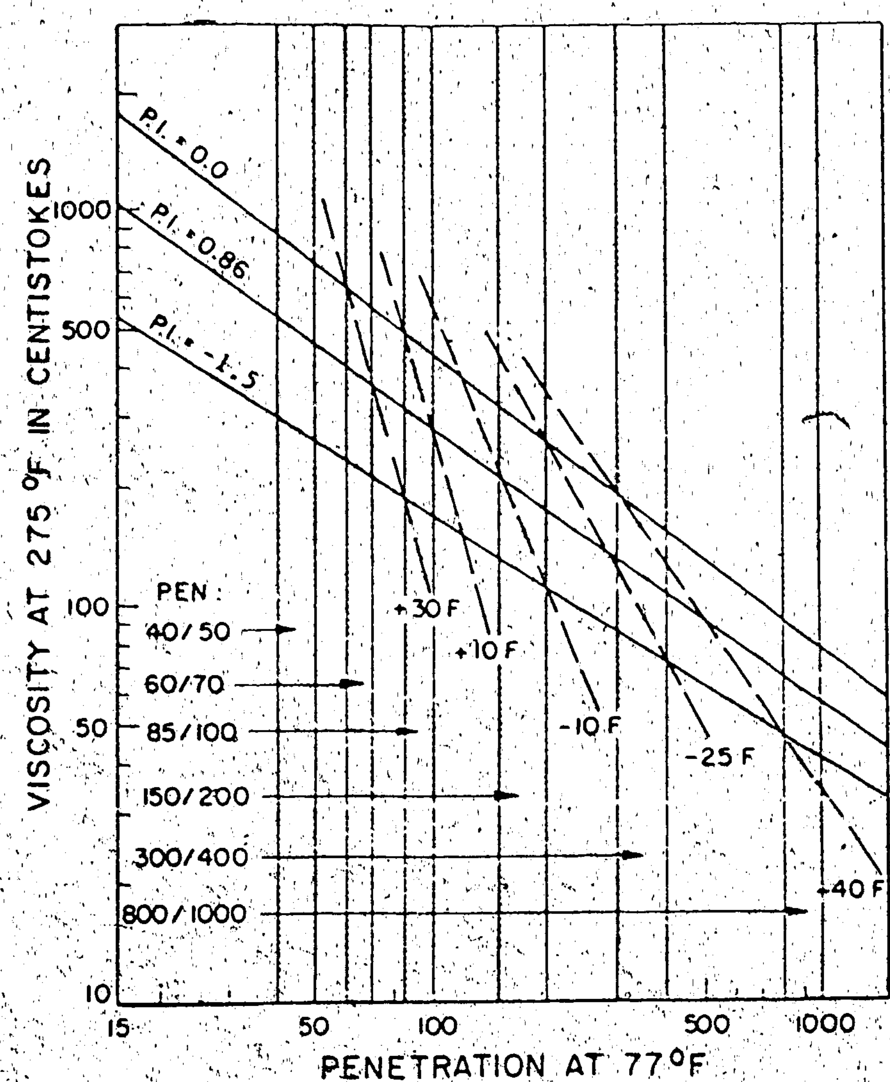


FIGURE 6.9 SELECTION OF ASPHALT CEMENT GRADE FOR VARIOUS WINTER DESIGN TEMPERATURES.

variation of the limiting stiffness approach.

An approximate expression for the thermal stress due to a drop in temperature was originally proposed by Hills et al. (45) in the following form:

$$\sigma_x(\dot{T}) = \alpha \sum_{T_o}^{T_f} S_{(\Delta T)} \; \Delta T \qquad (6.1)$$

where

$\sigma_x(\dot{T})$ = accumulated thermal stress for a particular cooling rate $\dot{T}$,

α = average thermal contraction coefficient over the temperature drop, $T_o - T_f$,

T_o = initial temperature,

T_f = final temperature,

$S_{(\Delta T)}$ = stiffness at the midpoint of discrete temperature intervals ΔT over the range of T_o to T_f, using a loading time corresponding to the time interval for the ΔT change.

A suggested value of α, typical for asphalt concrete is $0.8 \times 10^{-5}/°C$ ($1.5 \times 10^{-5}/°F$) within the temperature range of -29°C to -1°C (-20° to 30°F).

The above equation allows the prediction of the thermal induced stresses as a function of temperature. A plot of measured tensile strengths and predicted thermal stresses against temperatures, results in the prediction of a fracture temperature which can then be compared to the anticipated minimum pavement temperature.

Haas et al. (104) have extended the approach to include both temperature and stiffness gradients through the depth of the asphalt concrete layer.

Hills (105) has also developed a pavement cracking temperature criterion that permits asphalt cracking tendencies to be ranked according to cracking temperature obtained from a nomograph based on asphalt penetration at 5°C and 25°C. This nomograph is shown in Figure 6.10.

Gaw et al. (97) have demonstrated that nomographic cracking temperatures correlated well with the observed pavement cracking temperatures of the Ste. Anne Test Road. For most practical purposes, the nomographic cracking procedure is a convenient means of comparing the low temperature performance of different asphalts in road mixes, and of estimating cracking temperature, using readily obtainable asphalt test data.

A computer based prediction model, for low temperature cracking, was also developed by Christison (89) and was then modified by Finn et al. (106) under the NCHRP Project 1-10B. This program, which is called the COLD (Computation of Low Temperature Damage) program, can be used to predict the fracture temperature of asphalt concrete pavements at low temperatures.

Generally, methods of predicting asphalt concrete fracture temperatures have been proven to be reasonable. However, it should be realized that since they are primarily affected by the nature of asphalt cement and the temperature

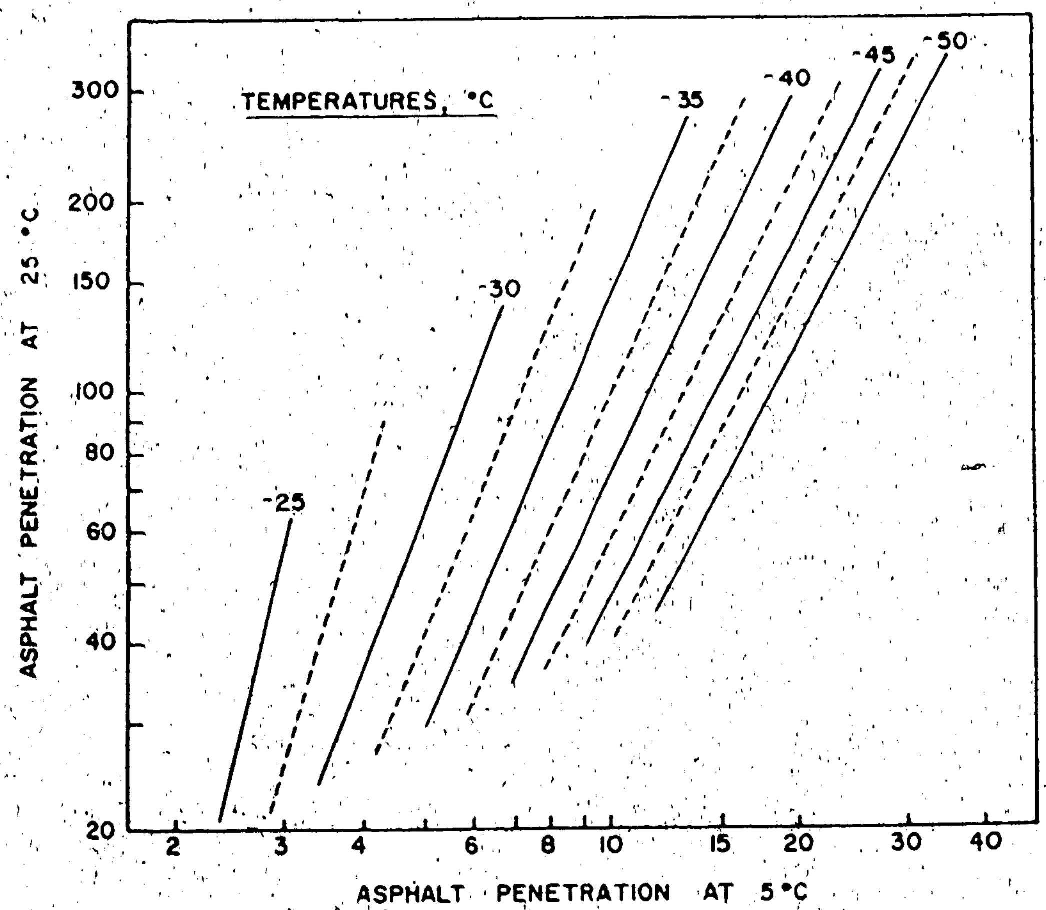


FIGURE 6.10 NOMOGRAPH FOR PREDICTING CRACKING TEMPERATURE.

conditions and are not taking into account other variables, they might be subject to some error.

6.6.4 Estimating Cracking Frequency Approach

A mathematical model based on field observations has been developed by Hajeck and Hass (107) which is capable of predicting the low temperature pavement cracking frequency at various ages in pavement life. There are several variables incorporated in the model involving stiffness of the asphalt cement, winter design temperature, subgrade soil type, thickness of the asphalt concrete, and age of the pavement.

The model is potentially a powerful design tool, since the degree of cracking is directly related to serviceability losses and maintenance costs. This model is based on field data from the province of Ontario and Manitoba.

Its construction consisted of finding a function that relates the cracking index to other variables. This function has the following form:

$$I = f(S,t,a,m,d) \tag{6.2}$$

where

I = cracking index, $I \geq 0$, defined below,

S = stiffness of asphalt cement determined for temperature m and loading time of 20,000 seconds by the modified McLeod method, $(kg/cm^2) \times 10^{-1}$,

t = thickness of all asphalt concrete layers, in.,

a = age of pavement, years,

m = winter design temperature, deg. C x -0.10, and

d = type of subgrade (dimensionless code: 2-clay, 3-loam, 5-sand).

The cracking index I, introduced by the Department of Transportation and Communication of Ontario, is defined as the number of full and half transverse cracks per 152 m (500 feet) section of a two-lane highway, and does not include transverse cracks less than one-half of the roadway width. The assumption is that such cracks usually occur subsequent to the formation of half and full cracks, and therefore they are not a primary manifestation of low temperature pavement cracking.

The form of the function chosen for the model is as follow:

$$10^{I} = C_1 S^{(C_2 + C_{3}t + C_{4}a)} C_5^{\,d} C_6^{\,m} d^{C_7 S} \qquad (6.3)$$

where I, S, t, a, m, and d are the original dependent and independent variables already defined, and d_1, c_2 ... c_7 are constants of the model. After estimating the constants the final numerical form is as shown below:

$$10^{I} = 2.497 \times 10^{30} \times 5^{(6.7966-0.8740t+1.3388a)} \times (7.054 \times 10^{-3})^{d} \times (3.193 \times 10^{-13})^{m} \times d^{0.60265s} \qquad (6.4)$$

Unlike other methods, this model predicts the cracking frequency as a function of age, rather than an either pavement cracks or does not crack result.

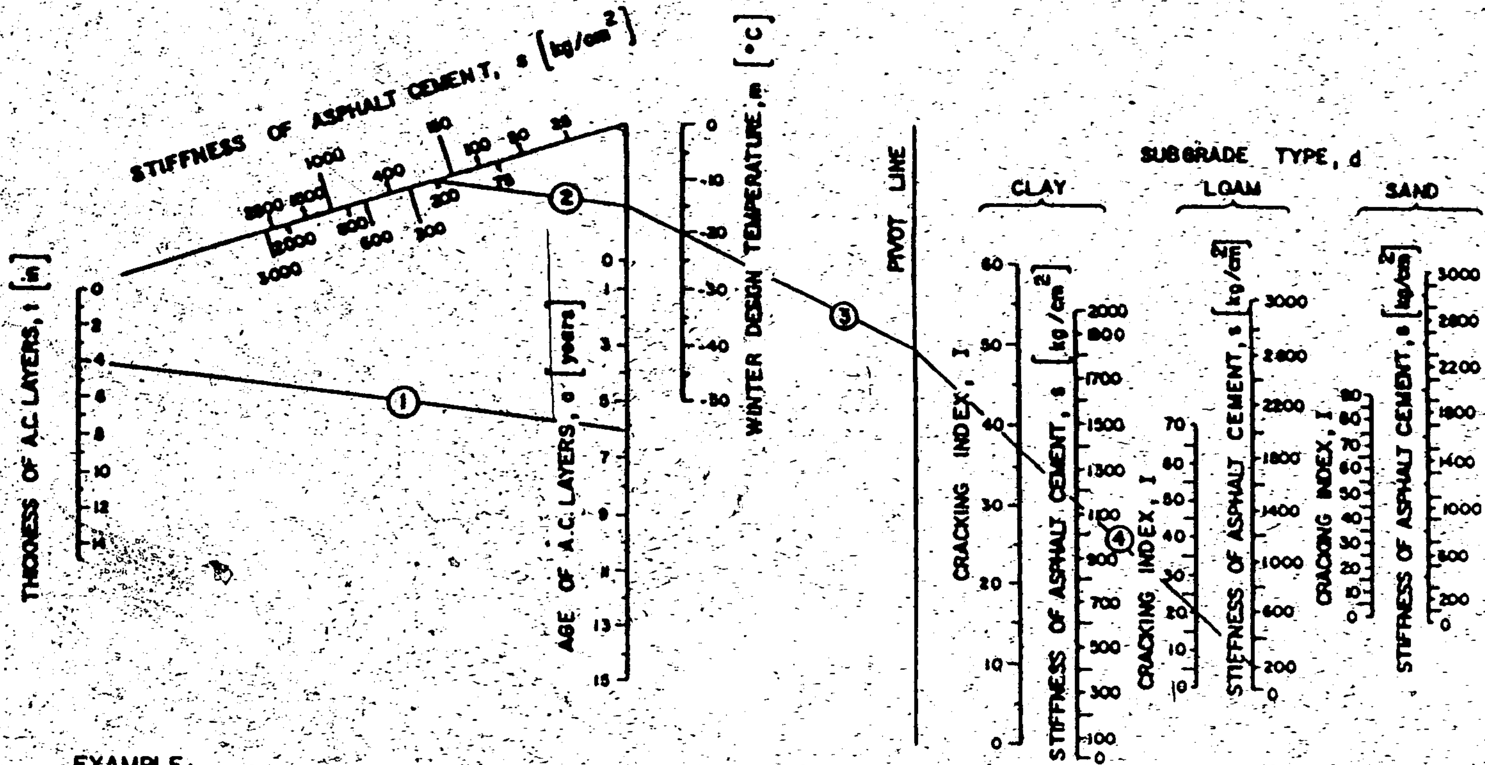
There are some limitations to this model, one of them is that, this model was constructed to fit the cracking index, but it predicts the low temperature transverse cracking frequency assuming that most of the transverse pavement cracks in the region investigated are low temperature cracks.

This model can be solved manually, or with the aid of a computer program. Alternatively, it can be quickly and easily solved by using the nomograph presented in Figure 6.11. On the nomograph the value of the winter design temperature, m, is related to the freezing index as shown in Figure 6.6.

Hajeck and Haas have concluded that the model is considered to be reasonable and to have an acceptable limit for the error involved. The use of the model in design is simple and requires easily obtainable data.

6.6.5 Other Approaches

Christison (89) has developed a design guideline for reducing the occurence of low temperature transverse cracking. This guideline was developed on the basis of the correlation between predicted and observed times and temperatures of fracture of various asphalt concrete pavements studied in Alberta and Manitoba. Using this guideline, the influence of variation in asphalt cement stiffness, asphalt concrete stiffness, and volume



EXAMPLE:

thickness = 4 inches
age = 6 years
stiffness of original asphalt cement = 200 kg/cm^2 *
winter design temperature = -20°C
subgrade type = loam
*for temp. = m and time = 20000 sec

NOTE:

A) lines (1) and (2) are parallels
B) In step (4) select scales for the appropriate subgrade type

RESULT: cracking index = 20 at 6 years

FIGURE 6.11 NOMOGRAPH FOR PREDICTING LOW TEMPERATURE CRACK FREQUENCY OF ASPHALT PAVEMENTS. (After Ref. 107) concentration of aggregate on maximum predicted thermally induced stresses computed by the pseudo-elastic beam analysis can be readily deduced.

A mechanistic model has been developed by Lytton and Shanmugham (108) to predict low temperature cracking of asphalt concrete pavement. This model is based on fracture mechanics and thermal stresses with viscoelastic theory. It requires simple input data on the properties of the mix and pavement thickness. The model generates all of the required time and temperature dependent properties internally by using a numerical form of Van der Poel's nomograph. The verification that has been done by comparing predictions made by this model, with observed thermal cracking data, has shown that this model can be used as a reasonably reliable prediction technique.

6.7. The Purpose of This Investigation and the Approach

On the basis of previous investigations, it was shown that the low temperature transverse cracking of asphalt concrete pavements is strongly associated with the tensile behaviour and properties of the pavement materials. Therefore, in order to prevent or minimize the occurrence of low temperature cracking, the tensile characteristics of the materials have to be taken into consideration in design stages.

One of the prime objectives of this investigation was to determine and compare the tensile properties of conventional and recycled asphalt concrete materials. Based on the findings of the experiments and field observations, guidelines can then be given for the designing of the recycled and the conventional asphalt concrete pavements to resist low temperature cracking. Another objective of this phase of the study was to determine the influence of the recycling ratio and also the effect of different temperatures upon the tensile properties of the pavement materials.

From the review of literature concerned with tensile testing of the material, it was deduced that the indirect tensile testing has a potential for evaluating the tensile properties of pavement materials. While the test has many advantages, the most important one is its simplicity. This test was conducted in this study to determine the tensile properties of various conventional and recycled mixes under different environmental conditions.

6.7.1 Indirect Tensile Test Method and Theory

The indirect tensile test which is also referred to as the tensile splitting test was first described independently by Carneiro and Barallos (109) in Brazil and Akazawa (110) in Japan simultaneously. The test involved loading a cylindrical specimen with a compressive load which acts parallel to and along the vertical diametral plane. This loading condition develops a relatively uniform tensile stress perpendicular to and along the diametral plane containing the applied load. The failure usually occurs by splitting along the vertical diameter.

The theoretical solution of the tensile splitting test is based on the theory of elasticity with the assumption of a state of plane stress.

Frocht (111) has developed the theoretical distribution of stresses, along the horizontal and vertical diameter, for a concentrated load. These are shown in Figure 6.12. The state of stress for the horizontal diametral plane is given by equations 6.5 to 6.7, and for the vertical diametral plane it is represented by equations 6.8 and 6.9.

Along the horizontal diametral plane:

$$\sigma_x = \frac{2P}{\pi td} \left[\frac{d^2 - 4x^2}{d^2 + 4x^2}\right]^2 \qquad (6.5)$$

$$\sigma_y = -\frac{2P}{\pi td} \left[\frac{4d^2}{(d^2 + 4n^2)} - 1\right] \qquad (6.6)$$

$$\tau_{xy} = 0 \qquad (6.7)$$

Along the vertical diametral plane:

$$\sigma_x = \frac{2P}{\pi td} = \text{constant} \qquad (6.8)$$

$$\sigma_y = -\frac{2P}{\pi t} \left[\frac{2}{d-2y} + \frac{2}{d+2y} - \frac{1}{d}\right] \qquad (6.9)$$

$$\tau_{xy} = 0$$

where

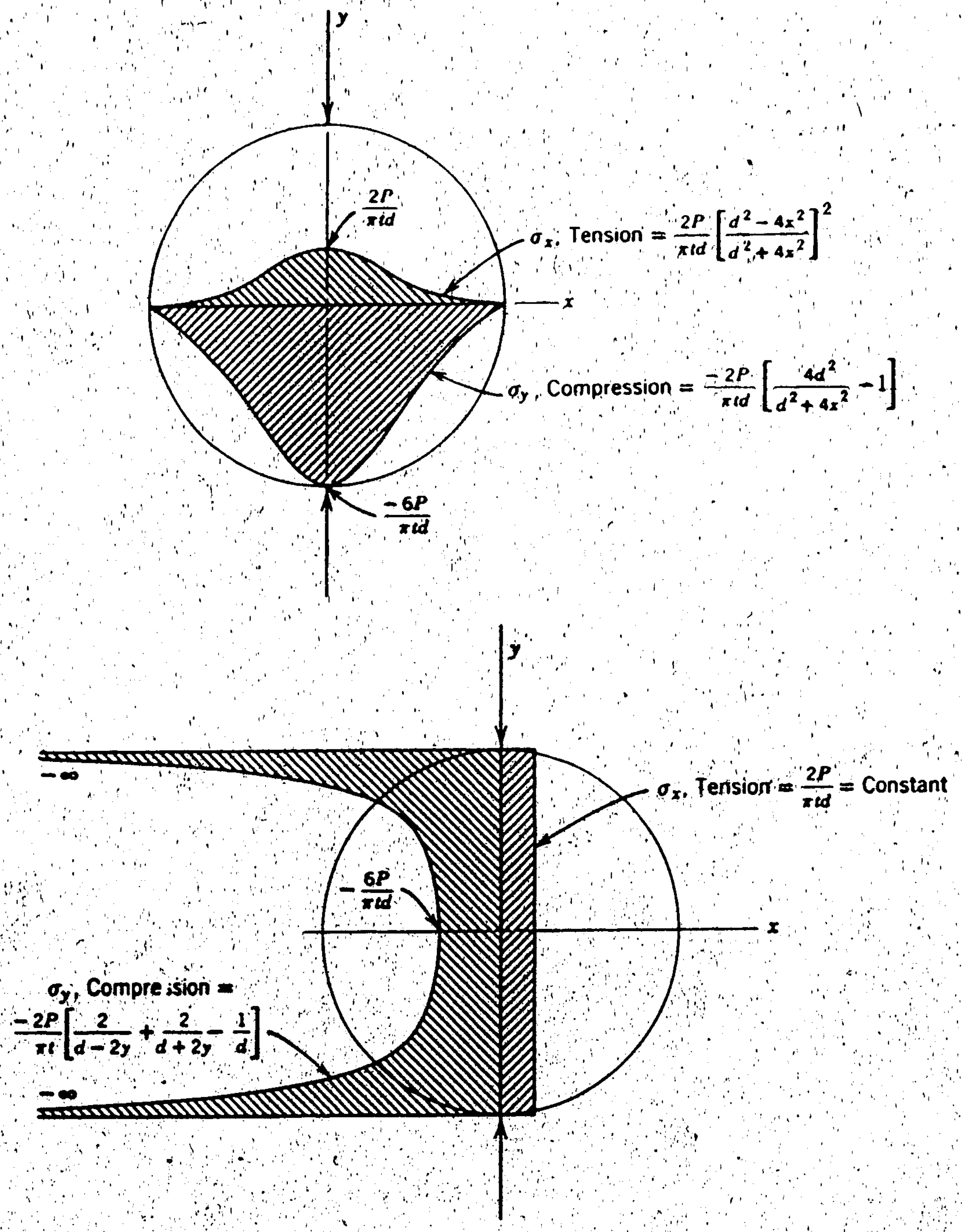


FIGURE 6.12 THEORETICAL STRESS DISTRIBUTION ON HORIZONTAL AND VERTICAL DIAMETRAL PLANES FOR INDIRECT TENSILE TEST.

P = magnitude of the concentrated load,

t = thickness of the specimen,

d = diameter of the specimen, and

x,y = coordinate values from centre of specimen.

The tensile horizontal stress along the horizontal diameter varies from a maximum of $\frac{2P}{\pi td}$ at the center to zero at the circumference, and the compressive vertical stress changes from $\frac{6P}{\pi td}$ at the centre to zero at the circumference. The horizontal stress along the vertical diameter is a constant tensile stress of magnitude $\frac{2P}{\pi td}$ while the vertical stress is compressive and varies from a minimum of $\frac{6P}{\pi td}$ at the centre to a maximum of infinity at the circumference at the load points. The theory assumes a point load on a thin disk which corresponds to line loading on the cylinder. Theoretically, the line loading causes failure to occur at the contact due to high vertical compressive stresses. However, in practice, these high stresses can be greatly reduced by applying the load through a loading strip with a width of less than d/10. Wright (112) has modified the theory and has developed approximate equations for stresses on the vertical diameter to account for the distribution of the applied load using the loading strips.

For most engineering materials, including asphalt concrete, initial failure occurs by tensile splitting along the vertical diameter. The tensile failure stress at the centre of the cylinder or, in other words, the tensile strength of the material is then given by:

$$\sigma_f = \frac{2P_{max}}{\pi t d} \tag{6.10}$$

where

σ_f = tensile failure stress (strength),

P_{max} = maximum applied load,

t = thickness of the specimen, and

d = diameter of the specimen.

The tensile splitting test has been used by many investigators to evaluate the low temperature tensile properties of asphalt concrete materials. Among the pioneers describing in detail the test method, procedures, and equipment, are Anderson and Hahn (113) and Hudson and Kennedy (114).

The tensile splitting test was selected as the desirable test method for the purpose of this investigation.

6.7.2 Experimental Program

The testing program involved the evaluation of the low temperature tensile stress-strain characteristics of the conventional and recycled asphalt concrete mixtures, subjected to various recycling ratios and environmental conditions. A comparison was to be made between the low temperature tensile properties of various mixtures and models were then to be developed.

The tensile splitting test was considered to be the most satisfactory means of determining the tensile properties of the mixtures. The cylindrical specimens were fabricated according to the standard Marshall design procedures (ASTM D1559). Five various mixes containing different percentages of reclaimed material (R) were provided. A mix with SC-3000 cut back, as a recycling modifier, was also prepared. All specimens were fabricated in duplicates of 12 for each mixture. A group of four specimens for each particular mix were tested under similar temperature and loading conditions. The average stress, strain and stiffness for each individual mix at a selected test temperature were then calculated.

6.7.2.1 Testing Conditions

The stress-strain characteristics of asphalt concrete mixtures are dependent upon time and temperature. Three test temperatures of -10°C, -20°C and -30°C were selected to examine the change in behaviour of six various mixes with temperature.

One rate of loading was adopted in this study. The compression testing machine was set at a nominal rate of loading of 1.15 mm/min. throughout the entire test.

6.7.2.2 Testing Equipment and Procedures

The tensile splitting test was carried out by loading a cylindrical asphalt concrete specimen via loading strips across a diameter in a compression testing machine within a controlled temperature chamber. As the specimen was compressed, a relatively uniform tensile stress was developed perpendicular to and along the vertical diametral plane of the specimen which ultimately caused the specimen to fail by splitting along its vertical diameter.

The applied compressive load and the vertical and horizontal deformations were monitored through a load cell and linear variable differential transducers (LVDT), respectively. The output signal from the load cell and the LVDTs were then recorded on floppy diskettes by the computerized data acquisition system developed particularly for this test, via a signal conditioner.

The raw data were then processed by the aid of the Lotus 1-2-3 spreadsheet program and the stress-strain characteristics up to failure and the stiffness modulus of the various mixes at different temperatures were computed.

Detailed descriptions of the tensile splitting test apparatus and procedures, together with the descriptions of computerized data acquisition system, are given in Appendix C.

6.8. Test Results and Analysis

The tensile splitting test was conducted on the asphalt concrete fabricated specimens under specified loading rate and environmental conditions.

This section presents the calculated results of the

TABLE 6.2

SUMMARY OF TENSILE SPLITTING TEST RESULTS AT -10°C

R %	Failure Stress (kPa x 10^3)	Failure Strain (mm/mm x 10^{-4})	Failure Stiffness (kPa x 10^6)
0	3.27 (0.13)	71.9 (18.2)	0.866 (0.222)
30	3.35 (0.08)	27.6 (16.6)	2.508 (1.521)
50	3.83 (0.48)	22.3 (11.3)	3.663 (2.599)
70	3.80 (0.16)	10.3 (0.0)	6.752 (0.283)
100	4.28 (0.66)	9.0 (0.8)	8.837 (2.147)
50 SC3000	2.93 (0.18)	43.7 (4.8)	1.240 (0.194)

Figures in brackets indicate one standard deviation.

TABLE 6.3

SUMMARY OF TENSILE SPLITTING TEST RESULTS AT -20°C

R %	Failure Stress (kPa x 10^3)	Failure Strain (mm/mm x 10^{-4})	Failure Stiffness (kPa x 10^6)
0	4.09 (0.23)	27.8 (2.9)	2.715 (0.437)
30	3.70 (0.16)	14.9 (1.9)	4.541 (0.429)
50	4.80 (0.13)	11.0 (6.6)	9.814 (4.766)
70	4.24 (0.39)	7.3 (2.3)	12.560 (4.187)
100	4.51 (0.33)	6.5 (1.0)	14.185 (3.166)
50 SC3000	4.62 (0.13)	18.0 (4.5)	4.944 (1.363)

Figures in brackets indicate one standard deviation.

TABLE 6.4

SUMMARY OF TENSILE SPLITTING TEST RESULTS AT -30°C

R %	Failure Stress (kPa x 10^3)	Failure Strain (mm/mm x 10^{-4})	Failure Stiffness (kPa x 10^6)
0	4.21 (0.31)	11.2 (0.7)	7.474 (0.159)
30	4.18 (0.52)	8.1 (2.0)	9.727 (2.092)
50	4.88 (0.01)	6.8 (2.0)	13.680 (3.546)
70	4.25 (0.67)	5.6 (0.9)	14.291 (4.192)
100	4.43 (0.40)	5.4 (0.6)	15.023 (1.910)
50 SC3000	4.92 (0.19)	7.4 (2.2)	12.775 (3.127)

Figures in brackets indicate one standard deviation.

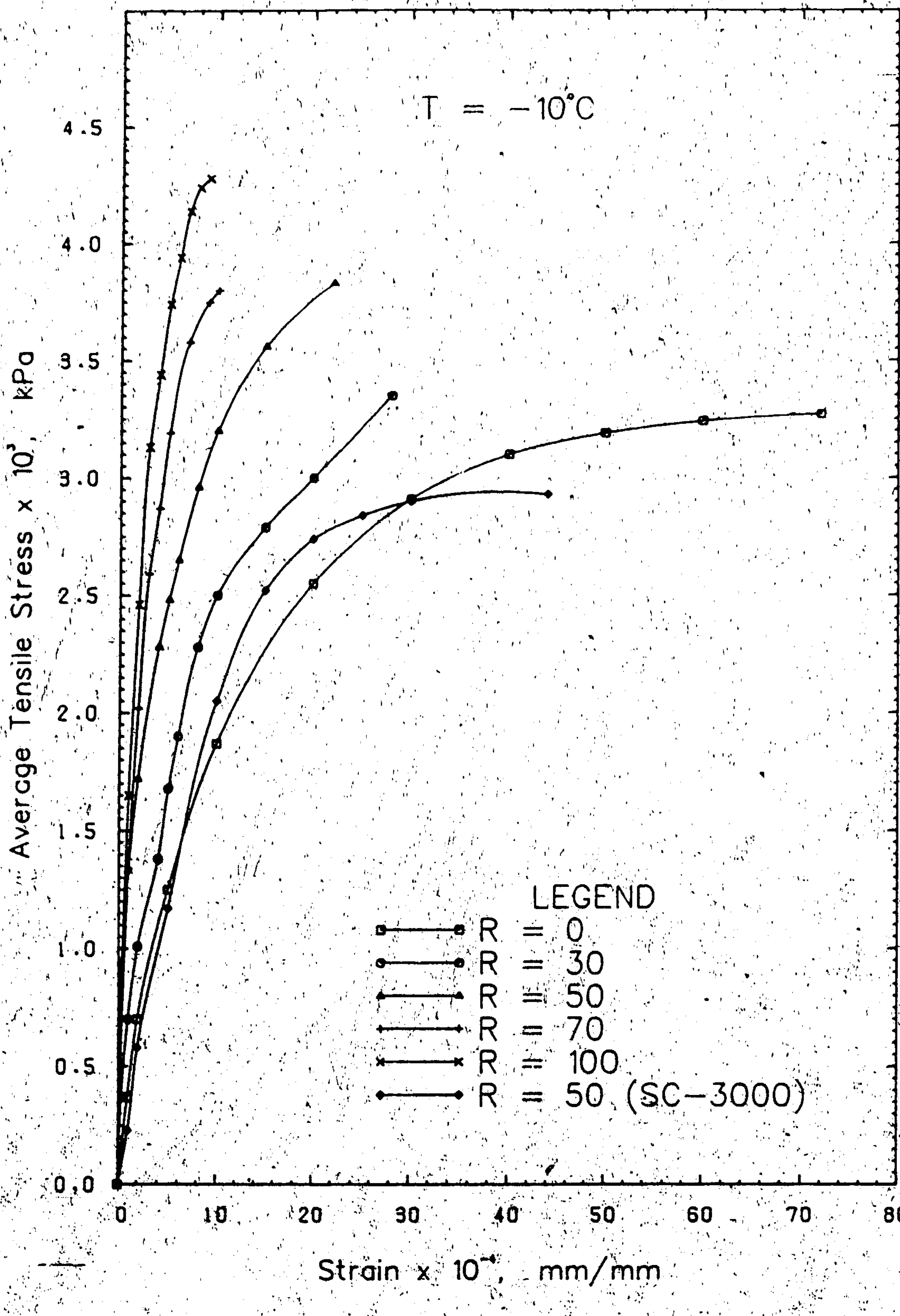


FIGURE 6.13 STRESS-STRAIN RELATIONSHIPS FOR VARIOUS MIXES AT -10°C.

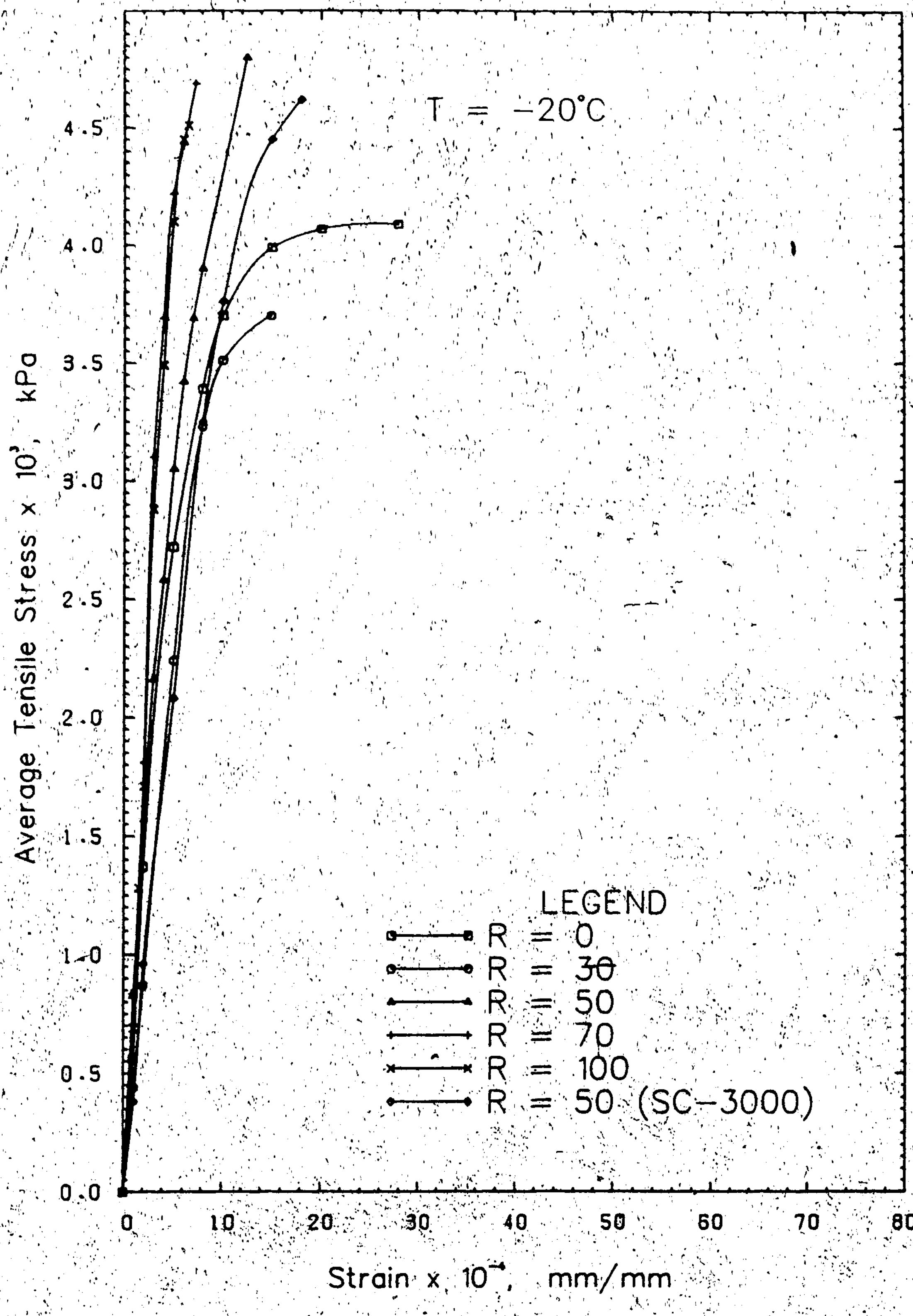


FIGURE 6.14 STRESS-STRAIN RELATIONSHIPS FOR VARIOUS MIXES AT -20°C.

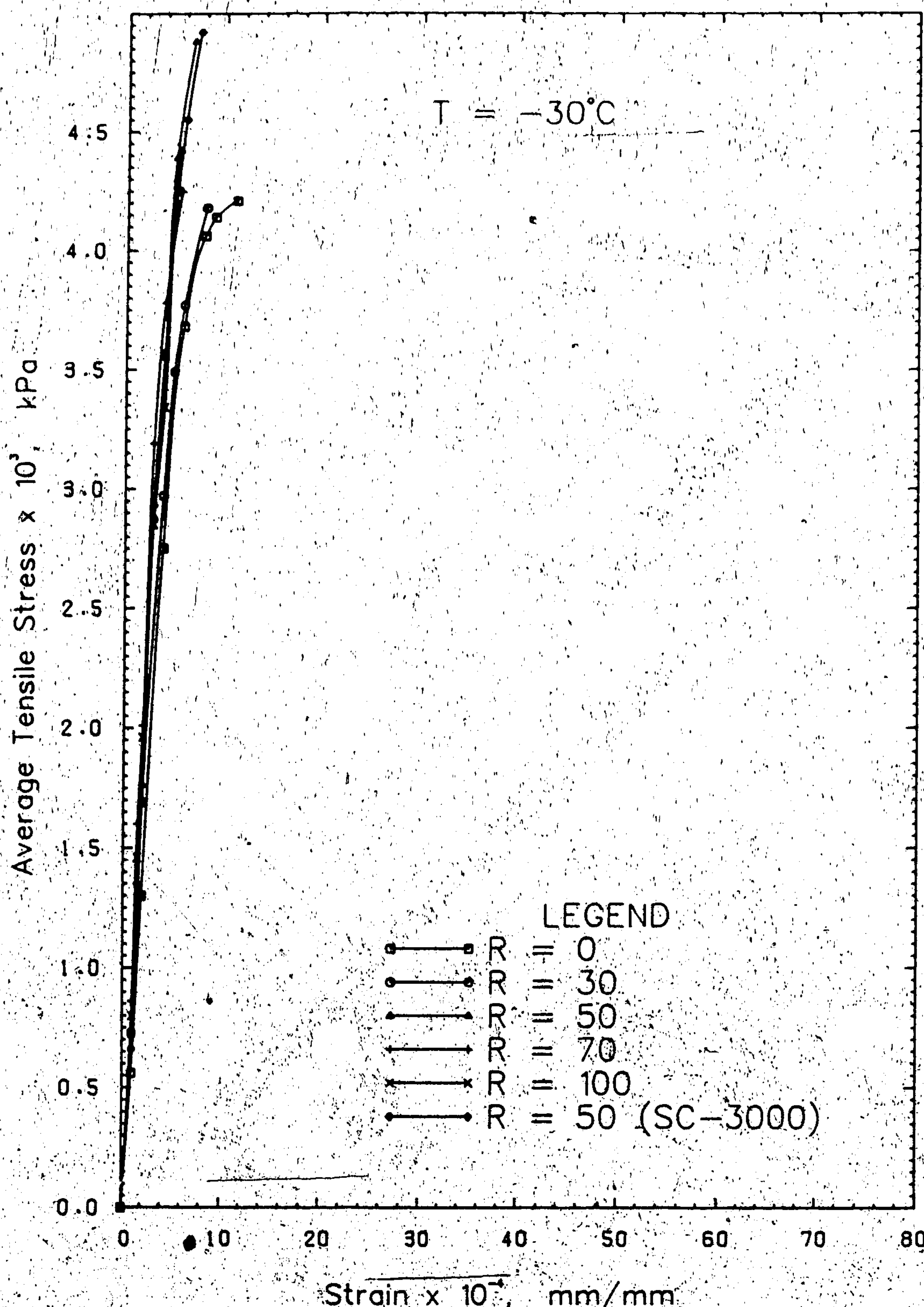


FIGURE 6.15 STRESS-STRAIN RELATIONSHIPS FOR VARIOUS MIXES AT -30°C.

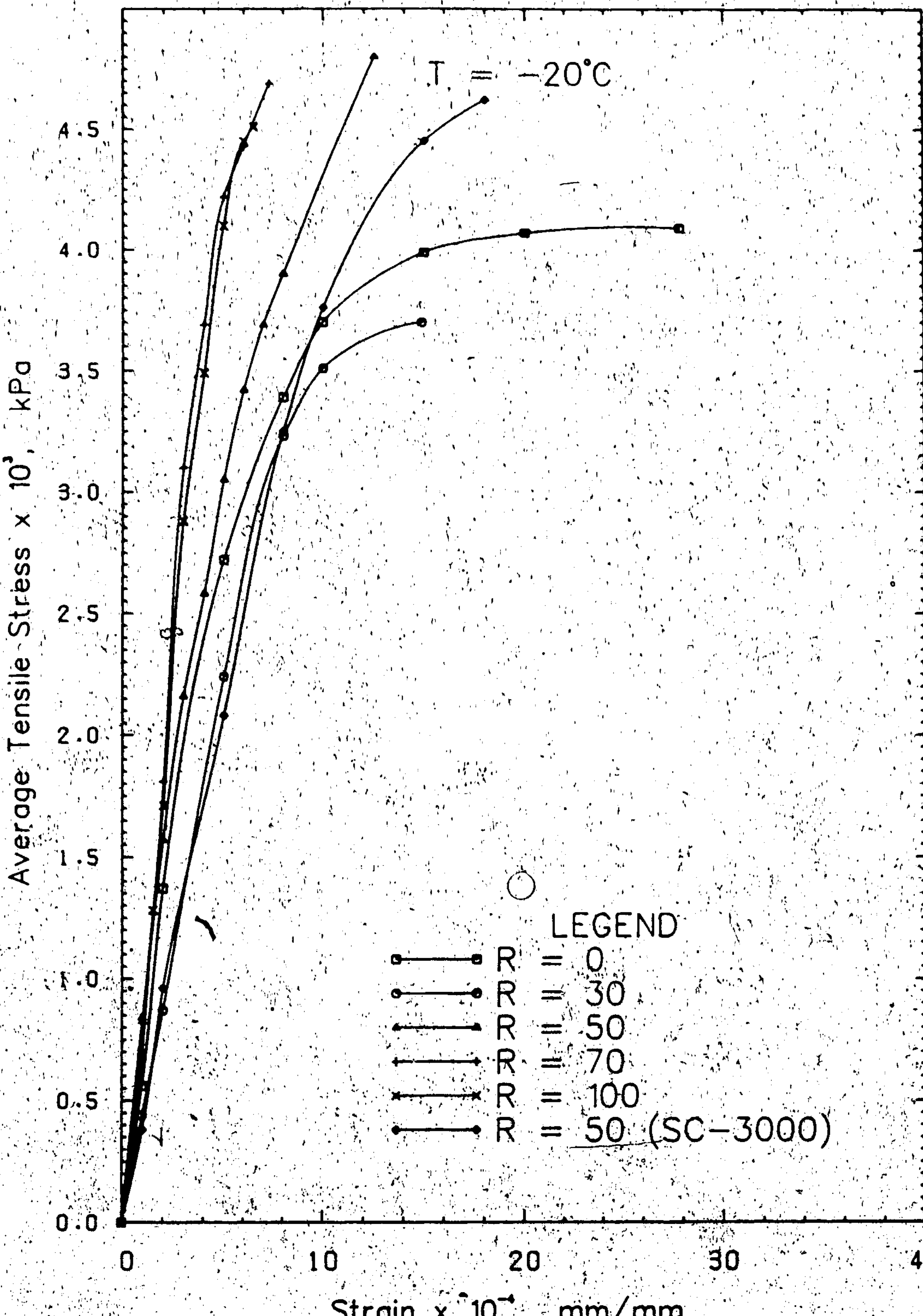


FIGURE 6.16 STRESS-STRAIN RELATIONSHIPS FOR VARIOUS MIXES AT -20°C.

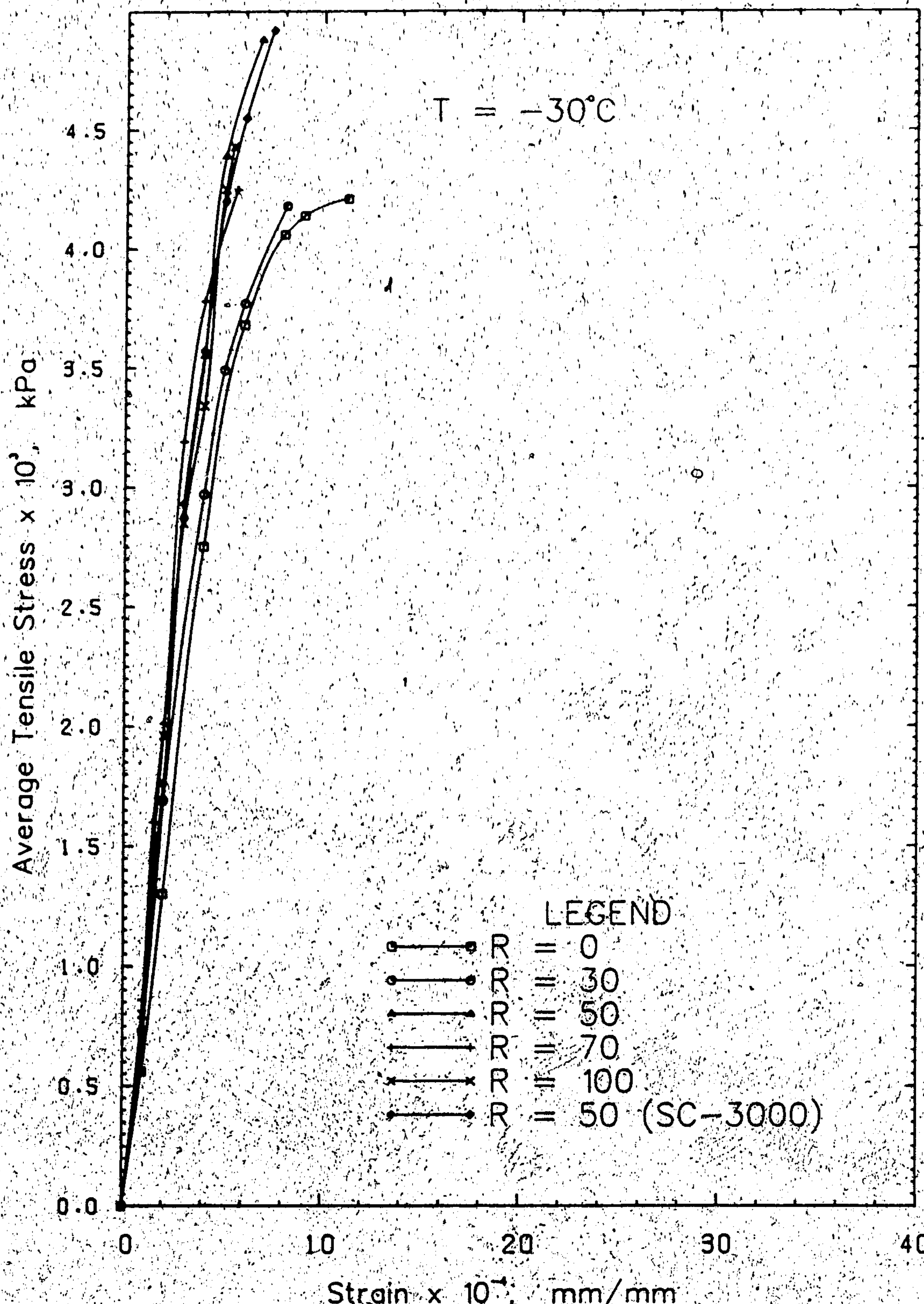


FIGURE 6.17 STRESS-STRAIN RELATIONSHIPS FOR VARIOUS MIXES AT -30°C.

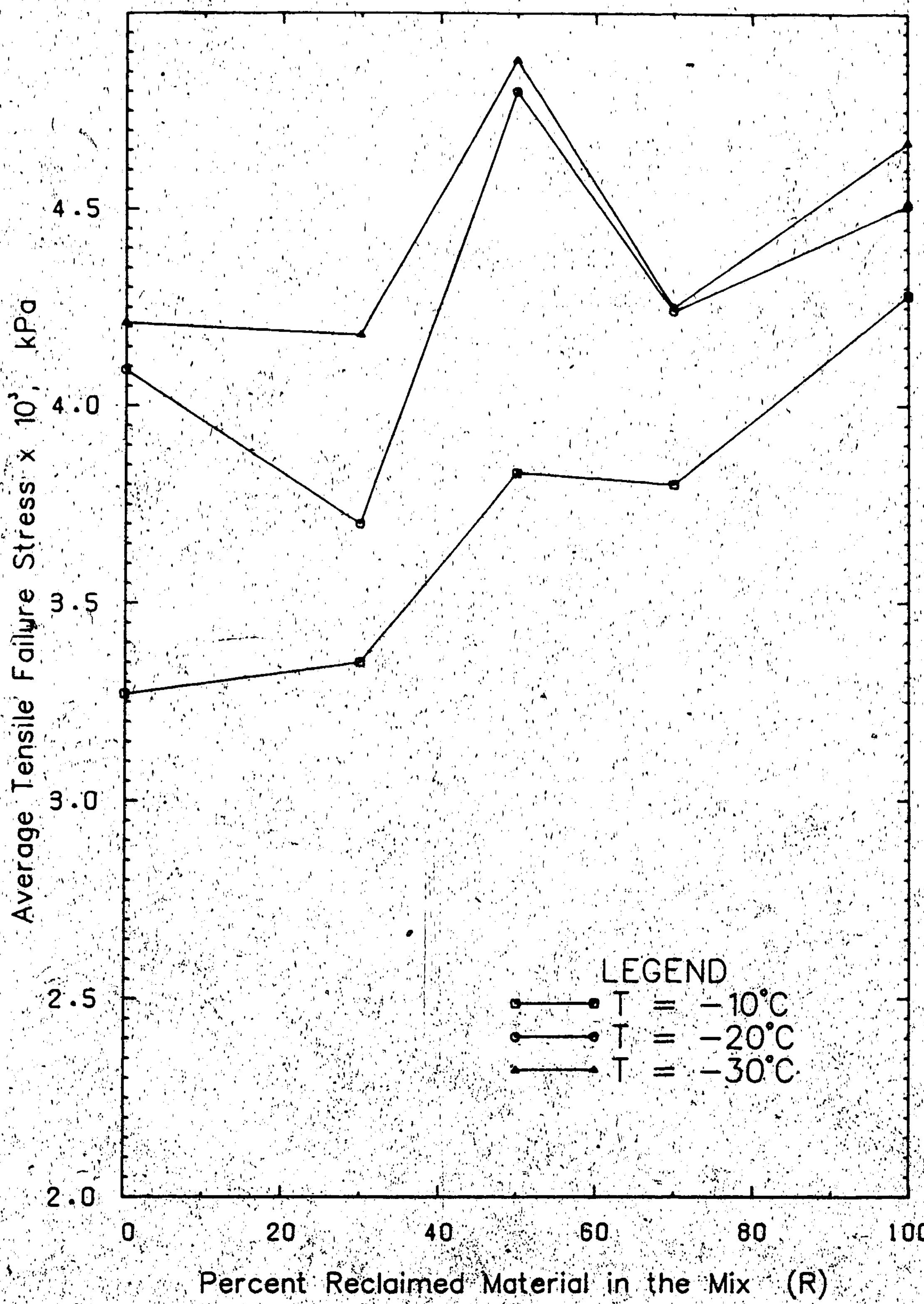


FIGURE 6.18 RELATIONSHIP BETWEEN TENSILE FAILURE STRESS AND PERCENT RECLAIMED MATERIAL IN THE MIX.

tensile splitting test. Analysis of the test results and the development of prediction models for specific properties are also discussed in this section.

6.8.1 Tensile Splitting Test Results and Analysis

The test results of each four specimens having the same mix characteristics and tested at similar temperatures, were averaged and the outcome was utilized for the analysis.

Tables 6.2 to 6.4 summarizes the averages and standard deviations of the tensile failure stress, the tensile failure strain and the stiffness modulus at failure for various mixtures at each particular test temperature. The stress-strain relationships for various mixes ~~at~~ temperatures of -10°C, -20°C and -30°C are shown in Figures 6.13 to 6.15. A similar scale was used for all the three figures for ease of comparison. Figure 6.16 and 6.17 show the stress-strain relationships at -20°C and -30°C with a larger scale.

The tensile failure stress versus the percentage of the reclaimed material in the mix (R) is shown in Figure 6.18. It is difficult to detect a specific trend common to all three test temperatures. However, it can be seen that fully recycled mixtures have higher tensile failure stresses than conventional mixtures at all three test temperatures. A rapid increase in tensile failure stress is observed when R increased from 30 percent to 50 percent, this followed by a decrease as R is raised to 70 percent. In general, a fluctuation in the value of tensile failure stress is observed as the percentage of the reclaimed material in the mix increases. Hence it can be said that the variation in R has little effect on the value of the tensile failure stress.

Figure 6.19 illustrates the tensile failure stress-temperature relationships for various mixtures. Generally, the tensile failure stress decreases as the test temperature increases. A similar trend can be seen for all six different mixtures. A rapid decrease in tensile stress is observed when the test temperature is raised from -20°C to -10°C. However, the decrease is rather small when the temperature changes from -30°C to -20°C. The mixture with SC-3000 as the virgin binder has demonstrated the sharpest decrease in tensile failure stress as temperature is raised from -20°C to -10°C. This mixture has the greatest value of tensile failure stress at -30°C and the lowest at -10°C in comparison with the five other mixtures. This shows a higher degree of temperature susceptibility. From the results of this figure, it can be concluded that the test temperature has a significant effect on the value of the tensile failure stress for various mixtures.

The variations in tensile failure strain with the percentage of the reclaimed material in the mix is presented in Figure 6.20. A decrease in tensile failure strain is observed as the percent recycling is increased. This is, however, less obvious at lower test temperatures especially at -30°C. The tensile failure strain at a test temperature of -10°C has dropped from 71.9 x 10^{-4} mm/mm for conventional

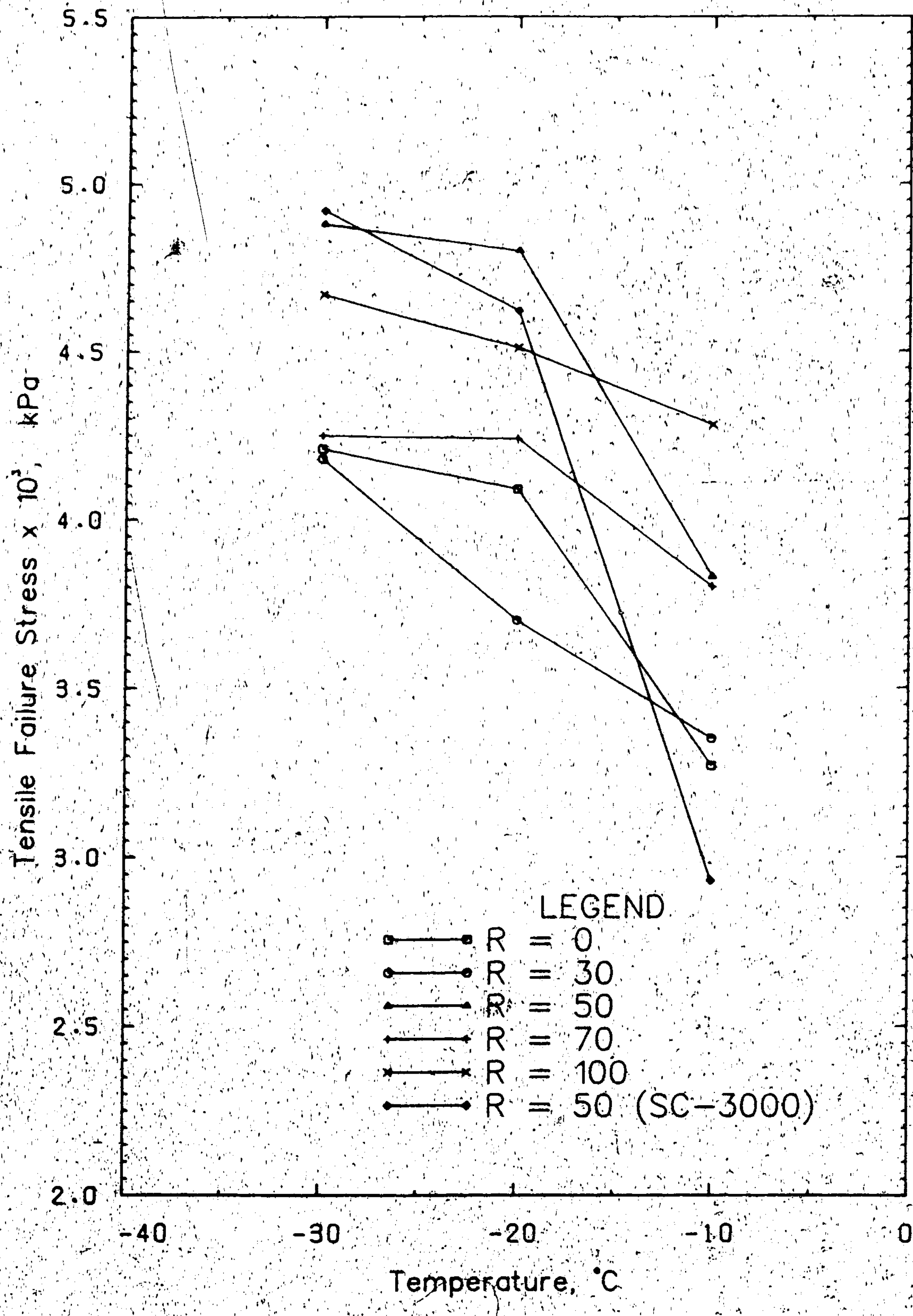


FIGURE 6.19 RELATIONSHIP BETWEEN TENSILE FAILURE STRESS AND TEMPERATURE.

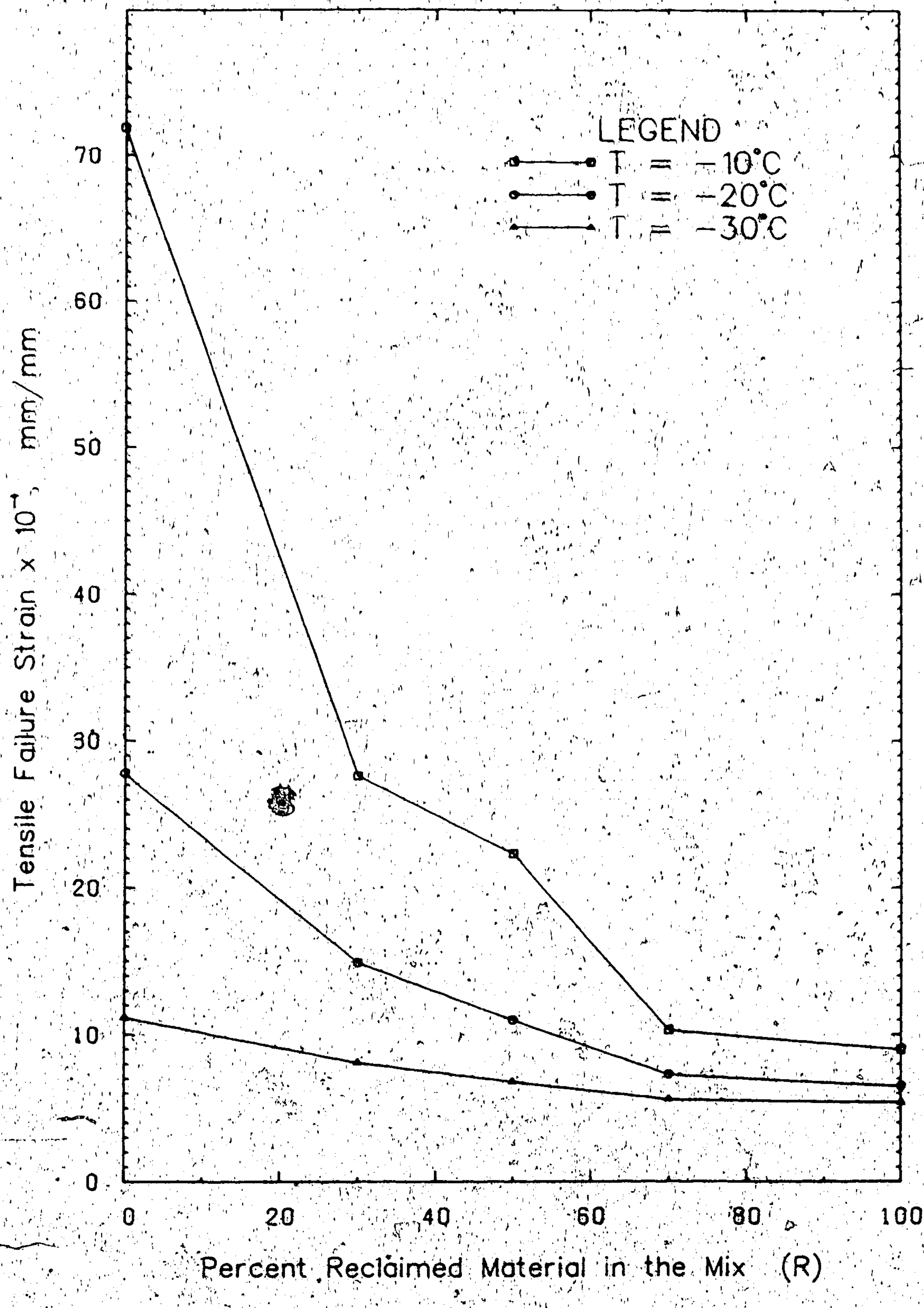


FIGURE 6.20 RELATIONSHIP BETWEEN TENSILE FAILURE STRAIN AND PERCENT RECLAIMED MATERIAL IN THE MIX.

mixtures to 9.0 x 10^{-4} mm/mm for full recycled mixtures, i.e. a 87.5 percent drop. At -20°C the reduction in tensile failure strain ranges from 27.8 x 10^{-4} mm/mm to 6.5 x 10^{-4} mm/mm, a 76.6 percent drop for conventional and fully recycled mixtures respectively. At the lowest test temperature of -30°C, the tensile failure strain has declined from 11.2 x 10^{-4} mm/mm for conventional mixtures to 5.4 x 10^{-4} mm/mm for full recycled mixtures i.e. a drop of 51.8 percent. Anderson et al. (94) have suggested that mixtures having a tensile failure strain below 10 x 10^{-4} mm/mm at temperatures of approximately -20°C have a high potential for cracking. Therefore, as can be observed from Figure 6.20, increasing the reclaimed material in the mix greater than 50 percent may not be advisable due to high cracking potential.

The tensile failure strain-temperature relationships for various mixtures are shown in Figure 6.21. It is evident that an increase in tensile failure strain is accompanied by an increase in the test temperature. The mixtures at 50 percent recycling, having SC-3000 as the virgin binder, show higher values of tensile failure strain than mixtures with 300-400A penetration grade asphalt at a similar recycling ratio. The results of Figures 6.20 and 6.21 reveal that the percent reclaimed material in the mix, the type of the virgin binder and the temperature, all have a significant effect on the tensile failure strain of the mixtures.

The variation in stiffness modulus with the percent

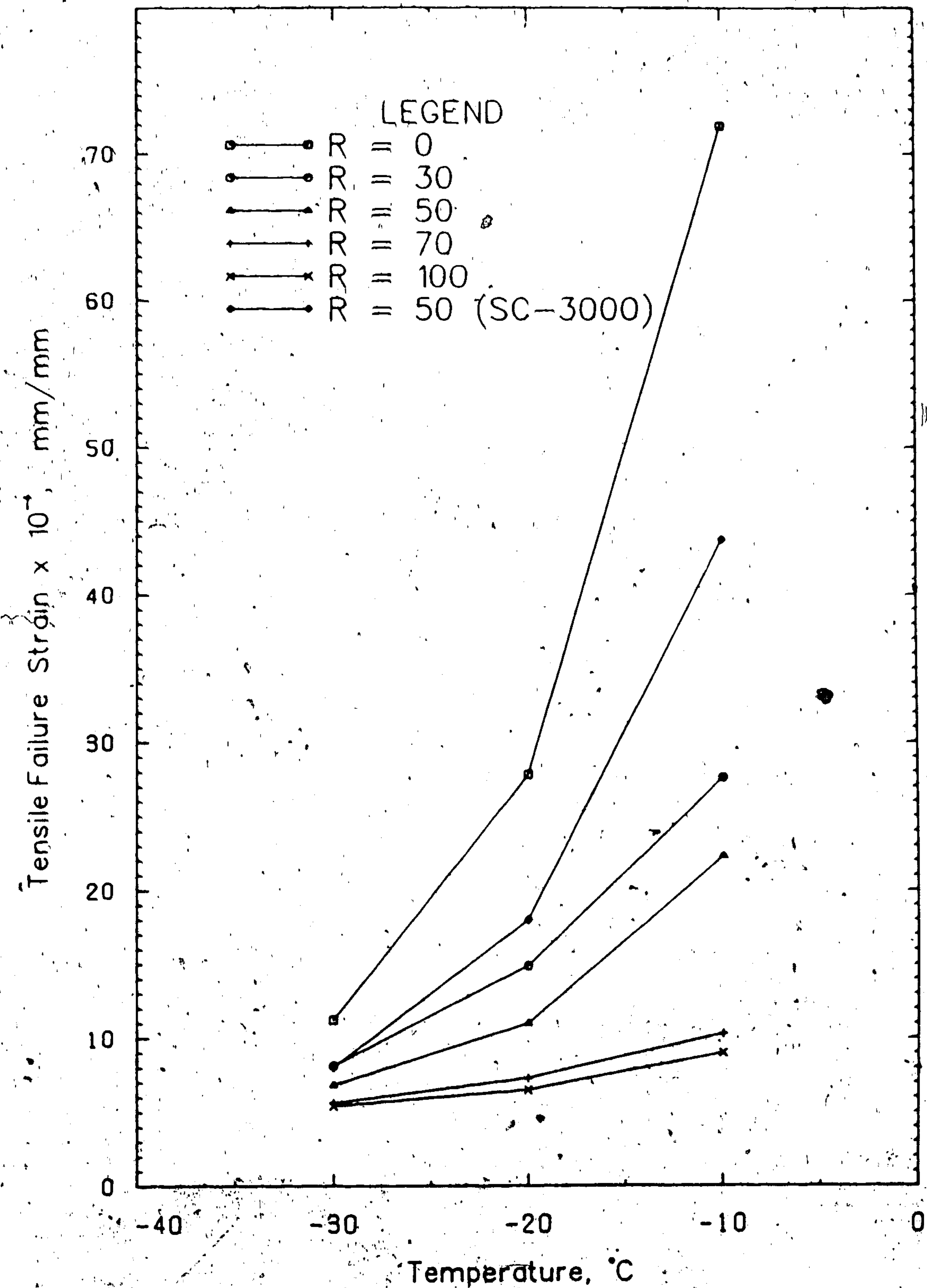


FIGURE 6.21 RELATIONSHIP BETWEEN TENSILE FAILURE STRAIN AND TEMPERATURE

reclaimed material in the mix is illustrated in Figure 6.22. Stiffness increases as the percentage of recycling increases. It ranges from 0.8 x 10^6 kPa at temperatures of -10°C for conventional mixtures, to 15.0 x 10^6 kPa at -30°C for 100 percent recycled mixtures.

The stiffness modulus temperature relationships are shown in Figure 6.23. An increase in the stiffness is observed for all the mixtures as the temperature drops. The mixture with SC-3000 as the virgin binder at 50 percent recycling illustrates lower stiffness values than mixes having the same recycling ratio but with 300-400A penetration grade asphalt as the virgin binder. Figures 6.22 and 6.23, therefore, illustrate the serious effect of the percent reclaimed material in the mix, the type of the virgin binder and the temperature, on the stiffness modulus of the mixtures.

In general, recycled mixtures exhibited higher tensile failure stress, lower tensile failure strain, and higher stiffness modulus than conventional mixtures. This makes them more susceptible to low temperature cracking. However, as discussed earlier, high cracking potential may be eliminated by not using greater than 50 percent recycling.

6.8.2 Development of Models to Predict Tensile Properties

Models were required to simply and accurately predict the tensile properties of conventional and recycled asphalt concrete mixtures as a function of several independent

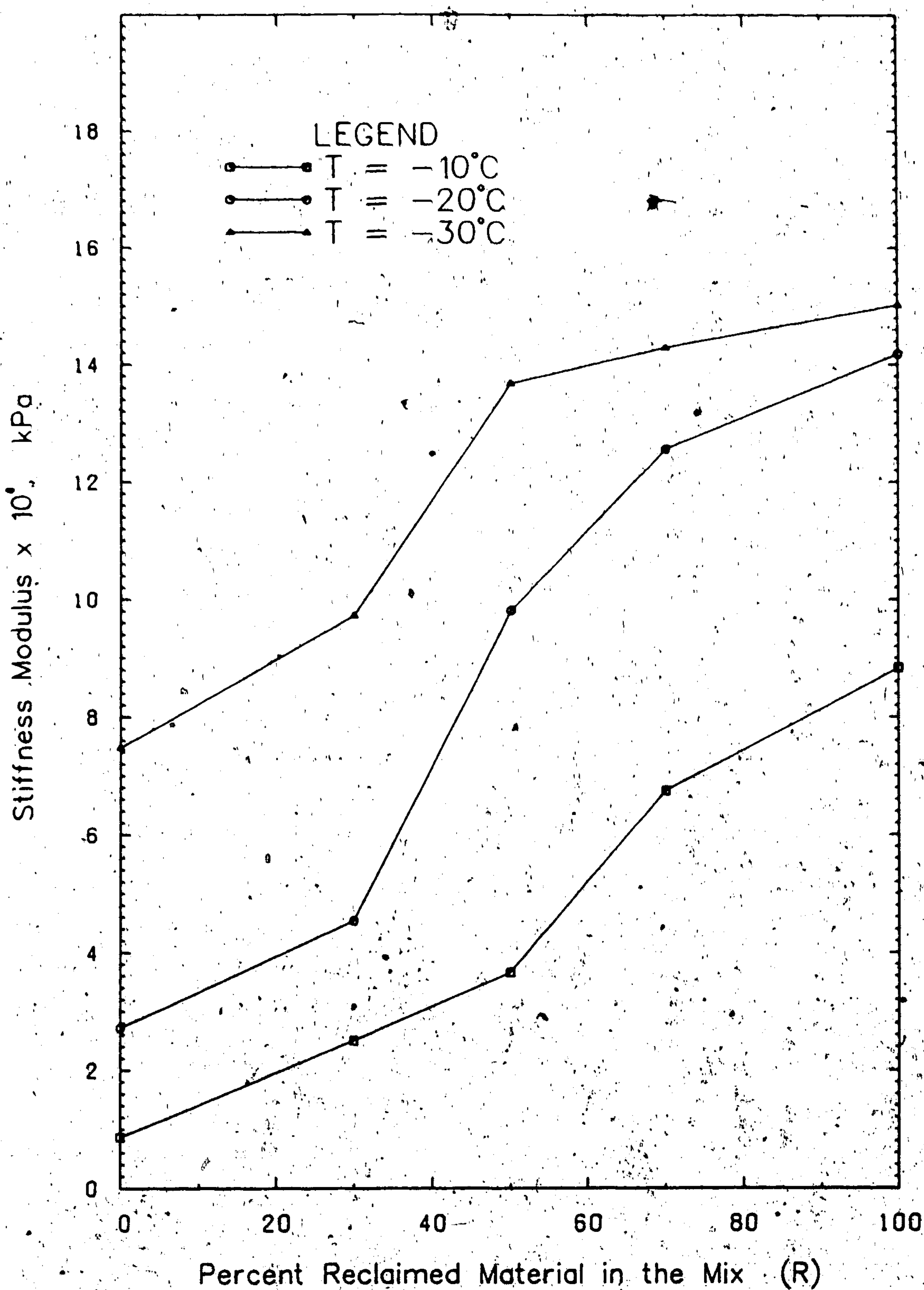


FIGURE 6.22 RELATIONSHIP BETWEEN MIX STIFFNESS AND PERCENT RECLAIMED MATERIAL IN THE MIX.

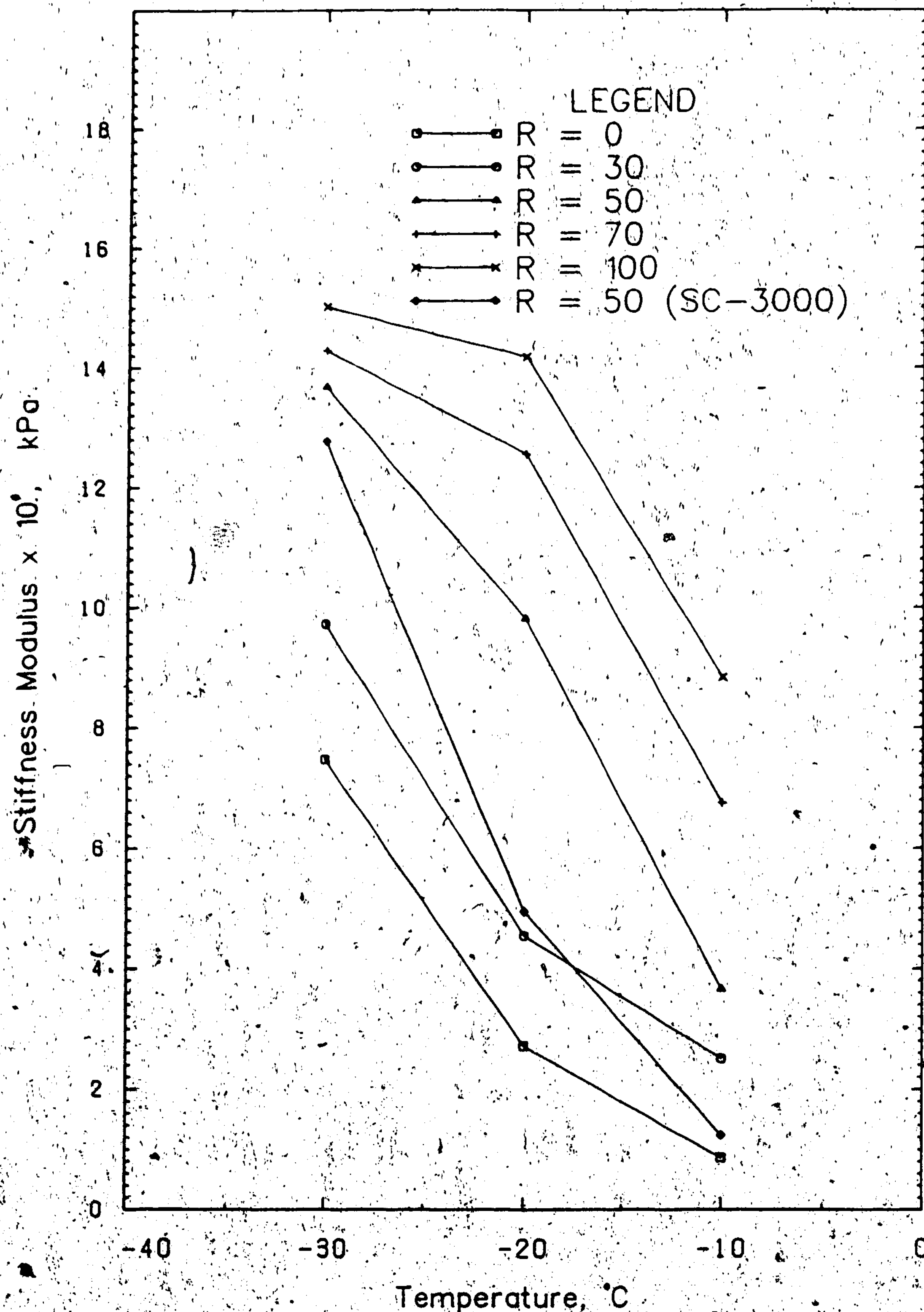


FIGURE 6.23 RELATIONSHIP BETWEEN MIX STIFFNESS AND TEMPERATURES.

variables.

The tensile properties of asphalt concrete mixtures are dependent upon many variables. The most influential independent variables considered in this study were the percent reclaimed material in the mix (R) and the temperature (T). In order to minimize the number of independent variables, mixtures having various recycling ratios were prepared with simlar aggregate properties. The asphalt content and asphalt consistency were not quite similar for various mixtures. The independent variable R was then used to reflect various mixture characteristics.

Prediction models were to be developed for tensile failure stress (σ_{tf}), tensile failure strain (ϵ_{tf}) and stiffness modulus (S_{mix}) as a function of percent reclaimed material in the mix (R) and temperature (T).

Several models were examined to select the best one to predict the tensile properties of various asphalt concrete mixtures. Models considering the two independent variables, i.e. R and T, individually and in combination, were studied. In each case, several transformations, including logarithm of dependent and/or independent variables and their inverse were used. Polynomials of up to degree four were also used to check for any non-linearity. Consequently, the following models were found to be the most appropriate for predicting the tensile properties of conventional and recycled asphalt concrete mixtures. These models were developed using temperatures ranging from -10°C to -30°C and percent recycling ranging from 0 to 100.

Tensile Failure Stress Prediction Model:

$$1/\sigma_{tf} = 0.3092 - 0.0004R + 0.0022T \qquad (6.11)$$

where

σ_{tf} = average tensile failure stress x 10^3, kPa,

R = Percent reclaimed material in the mix, and

T = Temperature.

The coefficient of determination (r^2) for this equation was 62.2 percent. This model exhibited the highest r^2 among all those examined. All the variables in the model showed to be relatively significant (probability < 0.05). The F-value for the equation exhibited a high degree of significance (probability < 0.005). The above model may therefore be considered as relatively satisfactory.

Tensile Failure Strain Prediction Model:

$$1/\varepsilon_{tf} = -0.0387 + 0.0011R - 0.0042T \qquad (6.10)$$

where

ε_{tf} = average tensile failure strain x 10^{-4}, mm/mm,

R = Percent reclaimed material in the mix, and

T = Temperature.

This model has an r^2 value of 96.3 percent which indicates a high degree of correlation. All the variables in the model were highly significant (probability < 0.001), which means that the regression coefficients were significantly different from zero. The F-value for the equation was also highly significant. Therefore, the model is very satisfactory.

This model predicts an increase in average tensile failure strain as the percent reclaimed material in the mix decreases and as the temperature increases.

Stiffness Modulus Prediction Model:

$$S_{mix} = -3.9163 + 0.0969R - 0.3757T \qquad (6.11)$$

where

S_{mix} = average Stiffness Modulus of mixtures $\times 10^6$, kPa,

R = Percent reclaimed material in the mix, and

T = Temperature.

The r^2 for this model was 93.5 percent. All the parameters in the model showed a high degree of significance (probability < 0.001). The F-value for the equation was also very significant (probability < 0.001). Hence the model can be considered as very satisfactory.

This model predicts the rise in the average value of the stiffness modulus of the asphalt concrete mixtures as the result of an increase in the percent reclaimed material in the mix and a reduction of temperature.

6.9. Summary

This study was undertaken to evaluate the low temperature tensile properties of conventional and recycled asphalt concrete mixtures by means of an indirect tensile test, and to develop prediction models and guidelines for designing conventional and recycled materials to resist low temperature cracking.

A review was conducted on the mechanism of low temperature cracking together with the factors affecting this phenomenon. The existing design approaches to resist low temperature cracking were also discussed.

The tensile splitting test was selected to determine the tensile stress-strain characteristics of various mixtures having different recycling ratios. Specimens were tested at three selected test temperatures to evaluate the effect of temperature on the material tensile properties.

The tensile failure stress, the tensile failure strain, and the stiffness modulus at failure were determined for all various mixtures at each of the three test temperatures. Models were developed to predict the aforementioned properties of asphalt concrete mixtures under various percentages of recycling and under different temperatures.

Generally, the results of this study have demonstrated that recycled asphalt concrete mixtures exhibit higher tensile failure stress, lower tensile failure strain, and higher stiffness than conventional mixtures. This indicates less resistance to low temperature cracking. However, high cracking potential may be minimized by ensuring that the reclaimed material content of the mix does not exceed 50 percent. The results have also shown that the temperature has a significant effect on the stress-strain characteristics of various mixtures.

7. ASPHALT CONCRETE PAVEMENT PERMANENT DEFORMATION

7.1 Introduction

One of the primary factors that influences the riding quality, and therefore serviceability of the pavement is permanent deformation. It reduces road serviceability and driving comfort. The hydroplaning and icing that results from accumulated water in the rutting path reduce highway safety. Therefore permanent deformation criteria should be included in any design methodology that attempts to achieve the goal of improved pavement serviceability and safety.

It has been shown by many investigators that permanent deformation is a major mode of distress in various pavement systems and should be considered as an important limiting criteria in pavement design schemes. For practical application, it is necessary to estimate the permanent deformation expected to occur in a pavement system during a certain period or set of conditions. To fulfill this task a scheme for prediction of permanent deformation is necessary.

There are structural, economic, and safety reasons for the development of the permanent deformation predictive techniques. These can respectively be summarized as follows:

(i) The introduction of new transportation means that have higher axle loads and tire pressures,

(ii) The use of softer asphalt to prevent cracking failure in asphalt concrete pavements, which consequently makes the pavement more susceptible to permanent deformation,

(iii) The high cost of rehabilitation and maintenance,

(iv) The reduction in riding comfort and serviceability,

(v) The reduction in safety due to hydroplaning and icing and when water collects in the depressions,

(vi) Introduction of recycled asphalt concrete material.

The factors mentioned show the necessity of developing a predictive model for permanent deformation estimation in asphalt concrete pavement system which, in turn, will help in formulating a new design method that controls the amount of permanent deformation.

The permanent deformation mode of distress can result from both traffic and non-traffic associated causes. This is summarized by Monismith (115) as follows:

General Cause	Specific Causative	Example of Distress
Traffic-associated	Single or comparatively few excessive loads	Plastic flow (shear distortion)
	Long-term (or static) load	Creep (time dependent) deformation
	Repetitive traffic loading (generally a large number of repetitions)	Rutting (resulting from accumulation of the small permanent deformations associated with passage of wheel loads
Non-traffic-associated	Expansive subgrade soil	Swell or shrinkage
	Compressible material underlying pavement structure	Consolidation settlement
	Frost-susceptible material	Heave (particularly differential amounts)

Every layer in the flexible pavement structure contributes to the total pavement permanent deformation. This study will focus primarily on permanent deformation in conventional and recycled asphalt concrete layers due to variations in traffic loading at different temperatures and at different recycling ratios.

7.2 Definition of Permanent Deformation

In order to develop a prediction model and consequently to formulate design criteria for controlling permanent deformation, it is necessary to precisely define this phenomenon.

Permanent deformation has been defined as any changes in the surface of the pavement including rutting, shoving, and pushing. However, this definition is too general for the purpose of structural design. Morris (116) refers to permanent deformation as the longitudinal depressions or "ruts" that form in the wheel paths due to repeated wheel loadings. Barksdale (117) similarly defines permanent deformation as the progressive accumulation of plastic strain in each layer of the pavement system that occurs under each load repetition.

The magnitude and the rate at which it occurs are governed by a combined contribution of the different pavement layers and are controlled by the layer responses under different factors.

In general, two types of permanent deformation may be considered. One form is due to a lack of proper compaction, especially in the case of heavy traffic. This form of permanent deformation will stop increasing when the pavement reaches the maximum degree of compaction. Bjorklund (118) has examined the effects of compaction on pavement permanent deformation and demonstrated that high degree of compaction results in lower air voids and consequently lower permanent deformation. The second type of permanent deformation is shear deformation which is more significant, and it reflects the pavement response under different traffic and environmental conditions. This type of permanent deformation is governed by many factors, such as stress level and its duration, temperature, and material properties. The two types of pavement deformation may be present simultaneously in the earlier part of pavement life.

7.3 Mechanism of Permanent Deformation

As described earlier, permanent deformation, in a complete and concise form, can be defined as the longitudinal channels or depressions that form in the wheel path due to consolidation and/or lateral movement in one or more of the component pavement layers due to repeated transient load application. Morris (116) stated that permanent deformation is primarily due to lateral distortion for properly compacted materials. A little densification does occur in the asphalt concrete but this is relatively constant over the full width of the pavement.

An early major investigation of the permanent deformation mechanism was conducted in 1959 at the AASHO Road Test (119). Two important points were discovered: (i) permanent deformation occurs in all layers of the pavement system, and (ii) it results primarily from lateral distortion of the pavement material. Since the AASHO Road Test, there has been a number of significant investigations related to the pavement deformation mechanism in different layers of the pavement structure, primarily subgrade soil and asphalt concrete layers. It has been observed that all layers have a considerable contribution to the pavement rutting. Based on average data from 51 sections that were trenched, it was observed that, asphalt concrete surfacing, base, subbase, and embankment soil have contributed to 32 percent, 14 percent, 45 percent, and 9 percent, respectively, of the total pavement rut depth. A recent investigation by Tam and Lynch (120) has shown that a rut depth of up to 32 mm may be expected in a 65 mm asphalt concrete overlay with low stability. This indicates that, in some cases, the contribution of asphalt concrete layer to the total pavement rut depth may be quite significant.

The permanent deformation mechanism possesses another important feature which is that the deformation consists of a continuous accumulation of incrementally small deformations from each individual load application.

7.4 Associated Problems With Permanent Deformation

Excessive permanent deformation results in loss of pavement serviceability and safety. A survey among various ~~state~~ highway departments by the AASHTO has shown the importance of permanent deformation or rutting, as a common cause of failure in flexible pavements. Majidzadeh et al. (121) have shown, in Table 7.1, a summary of the most prevalent types of pavement distresses reported by American state highway agencies. Excessive permanent deformation is one of the major factors influencing the riding quality and therefore the serviceability of the asphalt concrete pavements. It may cause longitudinal cracking at the upheaved shoulder and the crown between the wheel paths. As the pavement cracks, water can penetrate inside the pavement structure which, in turn, reduces the pavement load transfer capability. As a result of this, and other factors the pavement begins to deteriorate at an accelerating rate.

At the Brampton Road Test in Ontario, Morris (116) has shown that permanent deformation in the outer wheel path has forced the pavement edge as much as six inches over the shoulder and longitudinal cracks consequently have formed on the shoulder side of the rut as illustrated in Figure 7.1. Cracking has not occurred on the inner side probably because of the resistance offered by the slab.

The longitudinal cracking may begin at a certain level of permanent deformation. However, the level of deformation or simply rut depth depends on many variables, and there is

TABLE 7.1

MOST PREVALENT TYPES OF
PAVEMENT DISTRESS REPORTED BY STATE AGENCIES
(After Ref. 121)

Type of Distress	States Reporting Distress as Most Prevalent on							
	Interstate Highways		Primary Highways		Secondary Highways		Farm-Market Highways	
	Number	Percent	Number	Percent	Number	Percent	Number	Percent
Cracking								
Longitudinal	5	10.0	7	15.9	3	6.4	0	0.0
Alligator	1	2.0	2	4.5	5	10.6	7	25.0
Multiple	7	14.0	9	20.5	14	29.9	2	7.1
Transverse	7	14.0	7	15.9	4	8.5	2	7.1
Raveling	1	2.0	2	4.5	3	6.4	3	10.7
Rutting	14	28.0	13	29.5	6	12.8	5	17.8
Flushing	2	4.0	1	2.3	1	2.1	0	0.0
Roughness	1	2.0	1	2.3	5	10.6	0	0.0
Patching	10	20.0	1	2.3	3	6.4	5	17.9
Base Failure	0	0.0	0	0.0	2	4.2	2	7.2
Corrugations	0	0.0	0	0.0	0	0.0	1	3.6
Shrinkage	2	4.0	1	2.3	1	2.1	1	3.6
Total	50	100.0	45	100.0	47	100.0	28	100.0

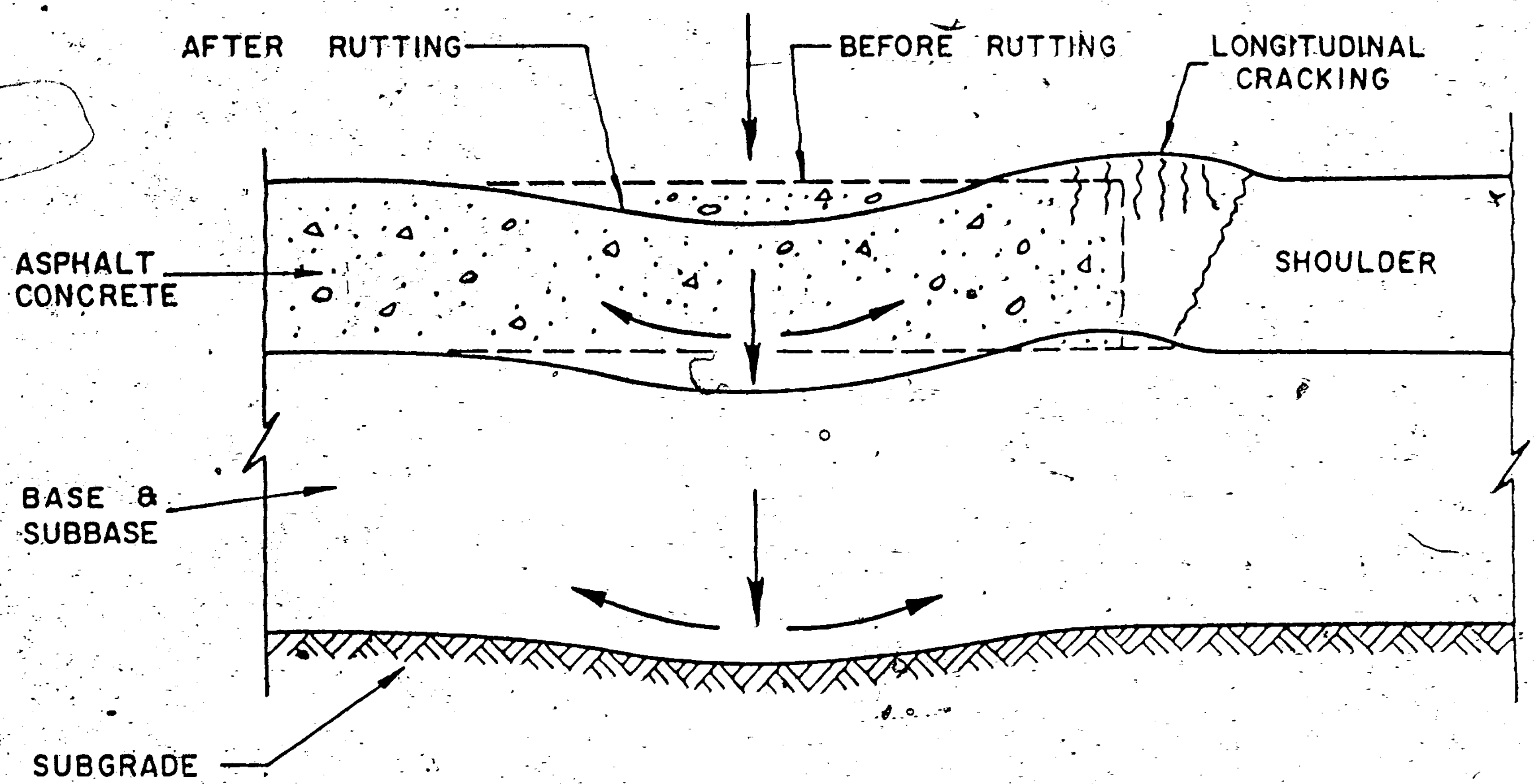


FIGURE 7.1 TYPICAL RUTTING MECHANISM IN OUTER WHEEL PATH AT BRAMPTON TEST ROAD.

really no unified limit to the level of permanent deformation. Lister (122) has considered a pavement to be a failure if cracking extends over the area of the wheel path and/or the permanent deformation exceeds 20 mm (0.8 inches).

As mentioned previously excessive permanent deformation can also result in loss of safety. The collected water inside depressions may cause either icing or hydroplaning, which both are dangerous to the driver. Another aspect of the loss of safety due to permanent deformation is that, it forces the driver to use a set of wheel paths. In such a case, changing the lane, especially at high speeds, results in a very uncomfortable and hazardous situation.

7.5 Factors Affecting Permanent Deformation

The most important factors affecting the amount and rate of permanent deformation can be, generally, summarized as: material properties, traffic load and volume, and environmental conditions. The effect of several factors on pavement permanent deformation will be examined in this chapter. Huschek (123), after conducting a number of tests, has concluded that aggregate gradation, asphalt content, degree of compaction, and softening point of the asphalt are the four most important factors influencing the resistance to permanent deformation of asphalt concrete pavements.

Material properties such as asphalt and aggregate characteristics certainly play an important role in permanent deformation phenomenon. Asphalt consistency is one of the major considerations. Softer asphalt may result in a pavement which is more susceptible to permanent deformation.

Asphalt content is another important factor, since it directly influences the stability and durability of the mix. If the asphalt content is over the optimum, it may result in a less stable mix which will be more susceptible to permanent deformation, particularly at higher temperatures.

Temperature has a great influence on the permanent deformation. Plastic and elastic strain increase as the temperature rises. Hence, the pavement is less resistant to permanent deformation at higher temperature. At temperatures below a certain limit, however, the accumulation of permanent deformation is negligible.

Another important factor that has a direct effect on permanent deformation is the stress level and its duration. It has been proven that higher vertical stress results in higher permanent strain. A recent investigation by Haas et al. (124) has shown that the tire pressure has substantial effect on the permanent deformation. They have shown that, under a wheel load of 8810 kg, an increase in tire pressure from 414 to 827 kPa, could increase the rut depth by a factor of 7 for a typical pavement in Ontario after one million load repetitions.

7.6 Permanent Deformation Prediction and Design Methods

Pavement design is a process in which the pavement structure which has been selected by a particular method is checked and modified if necessary, to ensure that various forms of distress modes are either precluded or minimized to a tolerable limit for the designed pavement life. Verstraeten et al. (125) have presented a block diagram of a rational approach to asphalt pavement structural design, shown in Figure 7.2, taking into account various forms of distress modes. In this section the concern is, basically, the estimation or minimization of the permanent deformation that occurs in the pavement structure.

Generally, there are two approaches available to design the pavement structure against the permanent deformation mode of distress. In one method, the vertical compressive strain in the subgrade surface is limited to some tolerable amount associated with a specific number of load repetitions. Controlled material characteristics, adequate mix stiffness, sufficient layer thickness, and proper construction procedures are the factors that will ensure permanent deformation to be equal or less than some specific amount. The other method is the estimation of the actual amount of rutting that might occur, from laboratory repeated load or creep tests and elastic or viscoelastic layered theories.

A summary of a number of procedures, which either limit permanent deformation to some specific amount or estimate the expected quantity from repetitive or creep loadings are presented in the following sections.

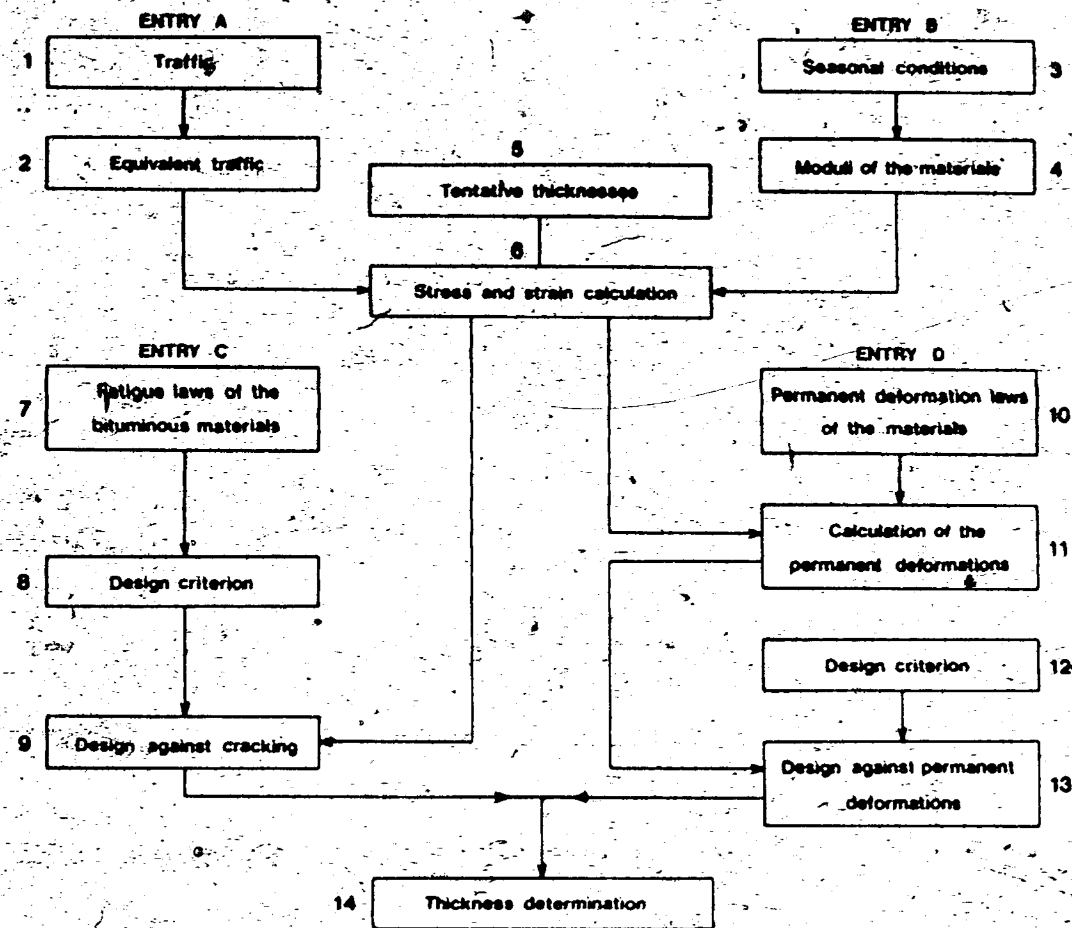


FIGURE 7.2 RATIONAL APPROACH TO ASPHALT PAVEMENT STRUCTURAL DESIGN.

7.6.1 The California Bearing Ratio (CBR) Approach

The CBR method is an empirical one and is based on experience. This method was developed in 1930, and since then, has been adopted or modified by many agencies around the world.

The main objective of the CBR method is to minimize the occurrence of excessive permanent deformation in the pavement structure. This is achieved by providing a certain layer thickness which has minimum strength based on some index test. The U.S. Corps of Engineers (126) provides thickness requirements based upon the CBR of the subgrade and specifies minimum stability values for asphalt concrete.

This approach has yielded reasonable results under conditions comparable to those from which it was developed. However, it is unable to predict the magnitude of permanent deformation.

7.6.2 Limiting Subgrade Strain Criteria

This criteria has been developed using the elastic theory approach. It is based on insuring that permanent deformation in the subgrade does not lead to excessive rutting at the pavement surface. Limiting subgrade criteria have been developed both for highway and airfield pavements.

One such criteria developed by Dorman and Metcalf (127), is termed the Shell criteria. It was developed from elastic analyses of pavements designed according to the California Bearing Ratio Procedure and for the AASHO Road Test.

Another limiting subgrade criterion was developed by Monismith and Mclean (128) from elastic analyses of pavements, the thickness of which were selected by California pavement design procedure. Monismith et al. (129) have shown that the strain values obtained by Monismith and Mclean are less than those suggested by the Shell criteria. It is possible that the smaller limiting values of permanent deformation are tolerated in pavements designed by the California procedure.

Hicks et al. (130) have analysed various sections of the San Diego Road test using the same procedure as Monismith and McLean. However, the results of their analysis, as shown in Figure 7.3, seem to be more conservative.

A strain criteria was also developed by Witczak (131) for airfield pavements based on field trials. Monismith (132) has summarized the allowable subgrade compressive strain values for highway and airfield pavements developed by various investigators in Table 7.2. It can be noted that the values for airfield pavements suggested by Witczak are substantially higher than those associated with highway pavements.

In general, limiting subgrade criteria is a useful concept, though, it has a number of shortcomings. Limiting the subgrade strain may protect the subgrade from plastic deformation but it cannot prevent surface distortion due to the deformation in the other layers of the pavement structure. This criterion is also limited to the conditions

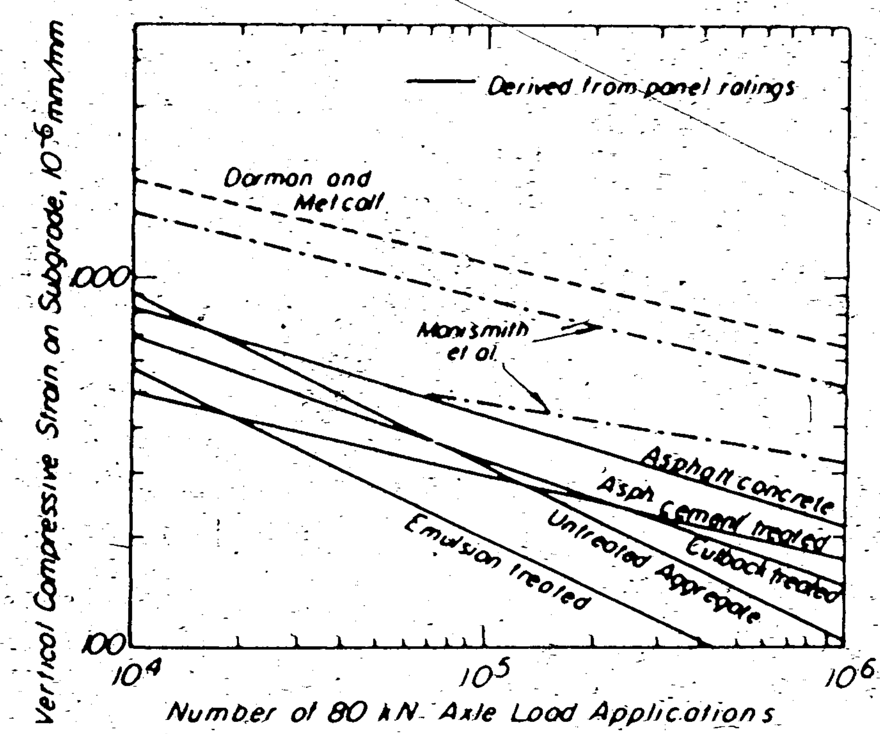


FIGURE 7.3 VERTICAL COMPRESSIVE STRAIN ON SUBGRADE VERSUS NUMBER OF 80 KN AXLE LOAD APPLICATIONS. (After Ref. 132)

TABLE 7.2

ALLOWABLE SUBGRADE COMPRESSIVE STRAIN VALUE CORRESPONDING TO DIFFERENT LOAD APPLICATIONS (After Ref. 132)

Load Applications (MPa)	Compressive Strain on Subgrade (mm/mm): Highway Pavements		Airfield Pavements
	Dorman and Metcalf	Monismith and McLean	Witczak
6.9			19.2×10^{-4}
69			16.8×10^{-4}
690	1.05×10^{-3}	8.0×10^{-4}	15.2×10^{-4}
6 900	6.5×10^{-4}	4.8×10^{-4}	14.6×10^{-4}
69 000	4.2×10^{-4}	2.9×10^{-4}	
690 000	2.6×10^{-4}	1.7×10^{-4}	

and form of its derivation.

Both the CBR and Limiting subgrade strain approaches may aim to preclude permanent deformation but cannot predict its magnitude or distribution.

Several methods are available for the prediction of permanent deformation. These approaches are briefly illustrated and discussed in the following sections.

7.6.3 Permanent Deformation Predictive Techniques

There are a number of techniques available for the estimation of the amount of permanent deformation in the pavement structure. They may be considered under two categories:

(i) the elastic approach, and

(ii) the viscoelastic approach.

The elastic layered system, which represents the pavement structure, has been used in conjunction with repeated load triaxial or creep tests. The viscoelastic layered system has been used to represent the pavement structure with material characterization by means of creep tests. A brief summary of each system, together with the predictive methods developed from them, are given in the succeeding sections.

7.6.3.1 Pavement as an Elastic Layered System

Heukelom and Klomp (133), Barksdale (134), and Romain (135) have suggested that a pavement may be represented as a layered elastic system when determining the state of stress or strain resulting from repetitive load applications. The amount of permanent deformation can then be calculated for some specific number of load repetitions with the use of an appropriate constitutive relationship.

To use this type of analysis, a relationship between plastic strain and applied stress must be available for each of the pavement components. This relationship can be shown as:

$$\epsilon_p = f(\sigma_{ij}, C_{ij}) \tag{7.1}$$

where

ϵ_p = plastic or permanent strain,

σ_{ij} = stress state, and

C_{ij} = material properties.

The permanent deformation for each particular layer can then be estimated, by computing strain at a number of points within the layer. The number of points must be sufficient in order to reasonably define the strain variation with depth. The permanent deformation is then determined by summing the products of the average permanent strains and the corresponding differences in depths among the locations at which the strains were determined. Monismith (132) has shown the schematic representation of the pavement system used to estimate permanent strain in Figure 7.4. The computed

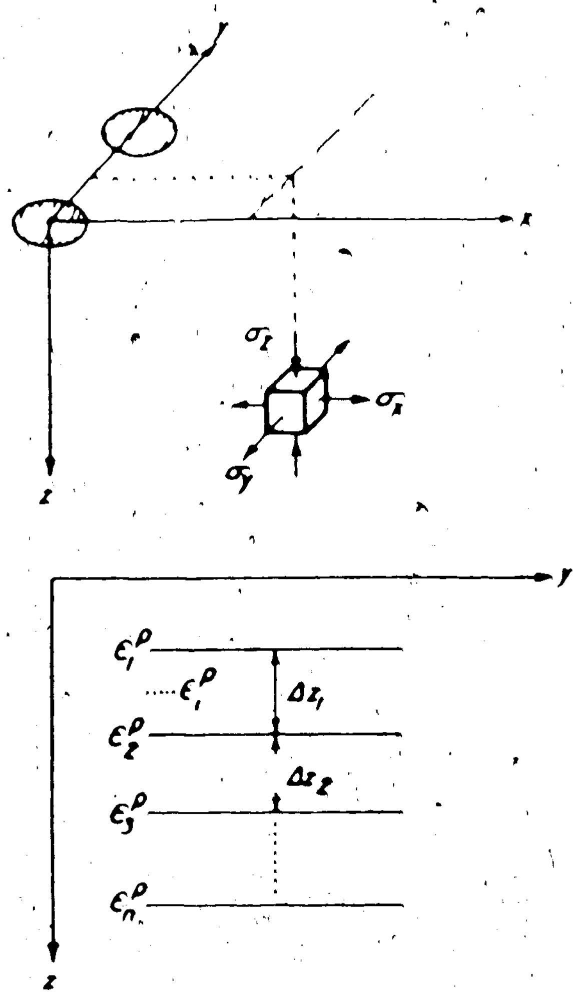


FIGURE 7.4 SCHEMATIC REPRESENTATION OF PAVEMENT SYSTEM USED TO ESTIMATE PERMANENT DEFORMATION. (After Ref. 132)

permanent deformation can be shown as:

$$\delta_{pi}(x,y) = \sum_{i=1}^{n} (\varepsilon_{pi} \Delta Z_i) \quad (7.2)$$

where

$\delta_{pi}\ (x,y)$ = permanent deformation in the ith position at point (x,y) in the horizontal plane,

ε_{pi} = average permanent strain at depth, $[Z_i + (\Delta Z/2)]$, and

ΔZ_i = difference in depth.

The total permanent deformation can then be estimated by summing the individual deformations of each layer. If the plastic strain at various numbers of load repetitions is known, the development of permanent deformation with traffic applications can be estimated.

Many investigators including Barksdale (134), McLean and Monismith (136), Morris (137), Snaith (138), Brown and Cooper (139), Majidzadeh (121), Van der Loo (140), Hills and Brien (141), and Saraf (142) have used this approach to predict permanent deformation in the pavement structure.

A brief summary of some predictive techniques developed using the elastic layered system is given in the succeeding subsections.

Barksdale Model:

Barksdale (134) has used the elastic layered system to provide an estimation of the permanent deformation potential of granular materials. He has developed data using repeated load triaxial compression tests. The data are represented by an equation of the following form:

$$\frac{\varepsilon_p}{\bar{\sigma}} = \frac{1 / K\sigma_3^{\,n}}{1 - \dfrac{\bar{\sigma} R_f (1 - \sin\phi)}{2\,(C \cos\phi + \sigma_3 \sin\phi)}} \left(\frac{N}{N_o}\right)^m \qquad (7.3)$$

where

ε_p = axial plastic strain,

$K\sigma_3^{\,n}$ = relationship defining the intial tangent modulus as a function of confining pressure,

$\bar{\sigma}$ = equivalent stress defined as

$$\frac{1}{\sqrt{2}} \left[(\sigma_1 - \sigma_2)^2 + (\sigma_3 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2 \right]^{1/2}$$

where σ_1, σ_2, and σ_3 are the principle stresses,

C = cohesion,

ϕ = angle of internal friction,

R_f = constant relating compressive strength to an asymptotic stress difference, $0.75 \leq R_f \leq 1.0$,

m = experimentally determined coefficient, and

N = number of stress repetitions.

These parameters are determined at a specific number of stress repetitions N_o.

For asphalt concrete, the suggested relationship is:

$$\varepsilon_{pz} = \delta\,(T)\, N^a\, \bar{\sigma}^{\,n-1} \left[\sigma_z - \frac{1}{2}\left(\sigma_x + \sigma_y\right)\right] \quad (7.4)$$

where

ε_{pz} = axial plastic strain,

σ_x, σ_y, and σ_z = components of stress at a point in the pavement layer, and

$\delta(T)$, a, and n = experimentally determined coefficients.

This relationship permits the estimation of permanent deformation at a point in the pavement structure.

The presented approach has the potential to be used in the prediction of the accumulation of permanent deformation in asphalt bound pavement layers. However, it requires a very long dynamic test procedure and was basically developed for granular materials.

Simulative Statistically-Based Approaches:

Morris (116) developed a statistical simulation procedure for the prediction of permanent deformation in asphalt concrete pavements. The approach was based on the premises that: (a) the permanent deformation characteristics of the materials should be obtained directly by testing under simulated field conditions, and (b) the mathematical models describing the material behaviour should be determined on the basis of statistically designed experiments.

Regression equations were developed for the triaxial testing based on factorial design, for a range of temperatures and a range of both tensile and compressive stresses. A typical representation of stress and temperature distributions in a full-depth asphalt pavement is shown in Figure 7.5.

The functional form of the expressions developed is as follows:

$$\varepsilon_p = f\left(\sigma_1/\sigma_3, T, N\right) \pm E \qquad (7.5)$$

where

ε_p = vertical permanent strain,

σ_1 = vertical stress,

σ_3 = lateral stress,

T = temperature,

N = number of load applications, and

E = error of estimate.

The total permanent deformation after N load applications, is calculated by summing the contributions of elements in each of the sublayers immediately below the centre of the wheel path. This is expressed mathematically in the following form:

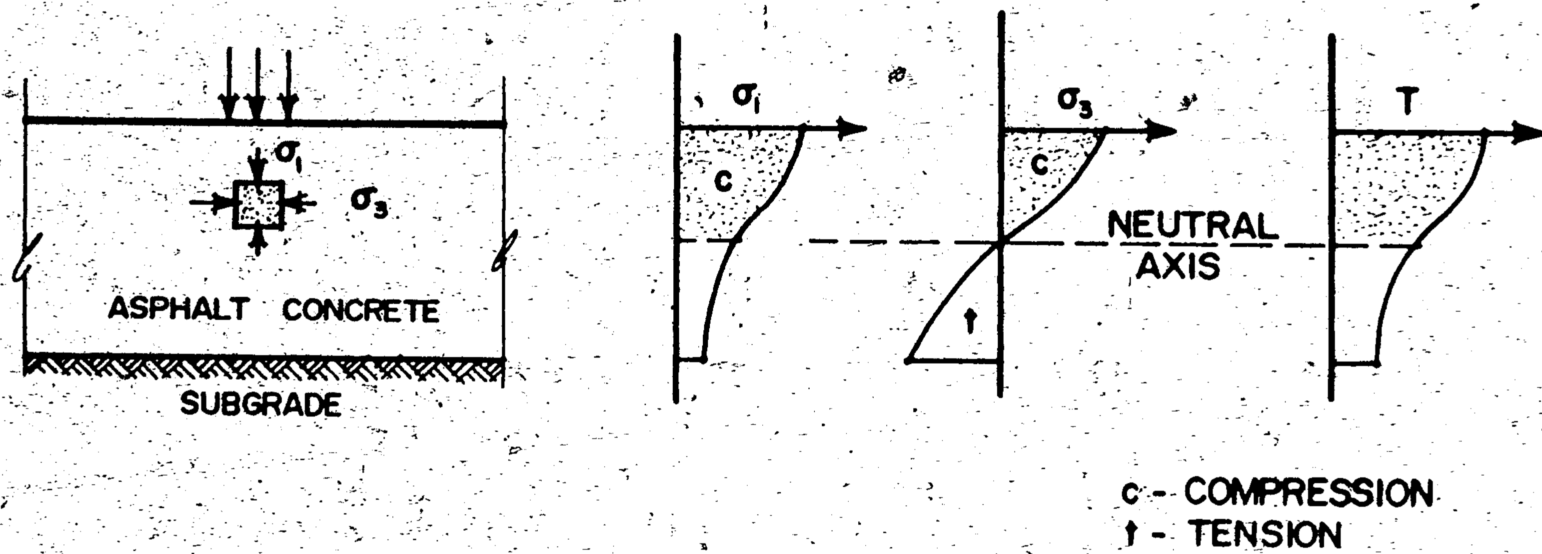


FIGURE 7.5 TYPICAL STRESS AND TEMPERATURE DISTRIBUTION IN FULL-DEPTH ASPHALT CONCRETE PAVEMENT.

$$\Delta P = \sum_{i=1}^{n} (\bar{\varepsilon}_{pi})(\Delta h_i) \qquad (7.6)$$

where

ΔP = total permanent deformation in the asphalt concrete,

$\bar{\varepsilon}_{pi}$ = average permanent strain in the ith sublayer,

Δh_i = height or thickness of the ith sublayer, and

n = total number of sublayers in the pavement structure.

Morris, in the development of the functional expression, has assumed that subgrade does not contribute to the total permanent deformation in the pavement system. Nevertheless the obtained laboratory test results were in agreement with field measurements on the Brampton Test Road in Ontario.

McLean (143) has also computed permanent deformation characteristics of asphalt bound materials under representative service conditions which include realistic stress states, time of loading, and temperatures. The data obtained from repeated load triaxial tests were fitted with a third order polynomial by using a least squares procedure. The functional expression developed is of the form:

$$\log \varepsilon_p = C_o + C_1(\log N) + C_2(\log N)^2 + C_3(\log N)^3 \qquad (7.7)$$

where

ε_p = plastic strain,

N = number of stress applications, and

C_o, C_1 and C_2 = coefficients which reflect the influence of stress state, time of loading, and temperature.

Brown and Snaith (144) have tested a dense graded macadam mix under conditions as close as possible to those likely to occur in the road. A repeated load triaxial test with various repeated vertical stresses, and static or dynamic confining pressures were used. According to the obtained data, they proposed the following expression to represent permanent deformation in asphalt concrete:

$$\log \varepsilon_t = n \log t + C \tag{7.8}$$

where

ε_t = permanent strain at time t, and

n, and C = constants which depend on the test conditions.

It was found that both constants n and C were related to the vertical stress, confining stress, and temperature, which were the major variables investigated.

Majidzadeh et al. (145) have proposed three basic steps to investigate pavement permanent deformation: (a) determination of the traffic and environmental conditions that may occur in the field, (b) finding of a reliable technique of stress-strain analysis to adequately describe the distribution of stress in the pavement system, and (c) investigation of the material characteristics under such traffic loading and the probable intensity and frequency of applied stresses, and consideration of the role of the environmental changes which are primarily those of temperature. They have performed laboratory simulation of field conditions by conducting dynamic tests under different stress levels and temperatures. A half sinusoidal compression dynamic load was applied, and a hydroelectronic material testing system was used to conduct the test program.

The equation proposed, representing the data is of the following form:

$$\varepsilon_p/N = A\ N^{-m} \qquad (7.9)$$

where

ε_p = permanent strain of each sublayer,
N = number of load applications, and
A, and m = permanent deformation parameters.

Permanent deformation parameters are dependent on mix composition, applied stress intensity and frequency, and temperature. Figure 7.6 shows the typical relation obtained between permanent strain accumulation and the number of load applications. In this study only a series of uniaxial tests were conducted and the effect of confining pressure was considered to be negligible.

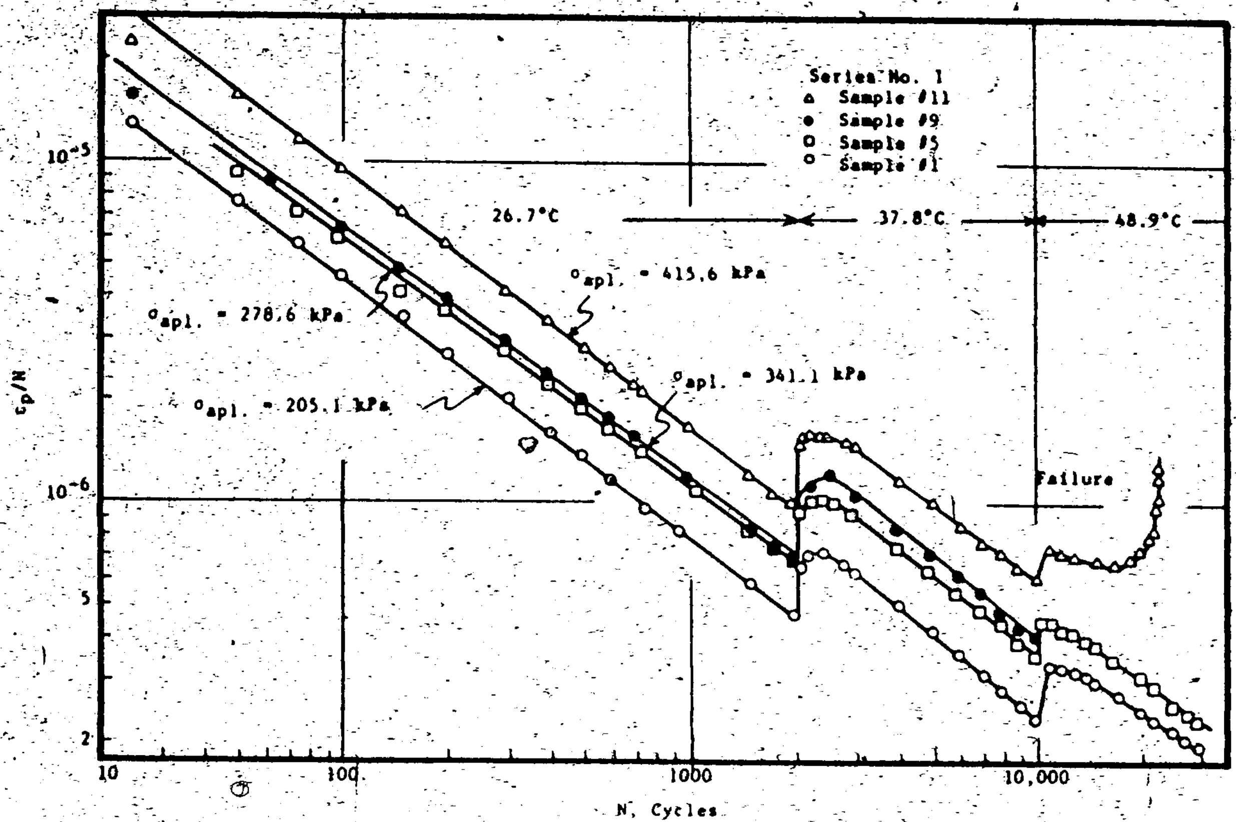


FIGURE 7.6 RELATIONSHIP BETWEEN PERMANENT STRAIN AND THE NUMBER OF LOAD CYCLES. (After Ref. 121)

Rate of Rut Depth Model:

Saraf et al. (142) have concentrated on linear elastic procedures to relate the various mechanistic responses to the rate of permanent deformation or rutting observed on 32 sections at the AASHO Road Test. The rate of rutting was influenced by the season of the year and the number of years for which traffic was applied. Figure 7.7 shows a typical pattern of rutting on AASHO Road Test. Correlations were obtained with the surface deflection, the vertical compressive stress in the asphalt concrete, the vertical strain in the subgrade, and the traffic applied to the sections. A regression model of the following form was used to obtain a correlation between the seasonal rate of rutting and the primary responses calculated for 80 kN single axle wheel load.

$$RR = f(\sigma, \varepsilon, \Delta, N_{18}) \tag{7.10}$$

where

RR = seasonal rate of rutting or permanent deformation per equivalent load application,

σ = stress in component layers,

ε = strain in component layers,

Δ = surface deflection, and

N_{18} = total equivalent number of 80 kN (18 000 lbf) single axle loads up to and including the season for which rate of rutting is to be calculated.

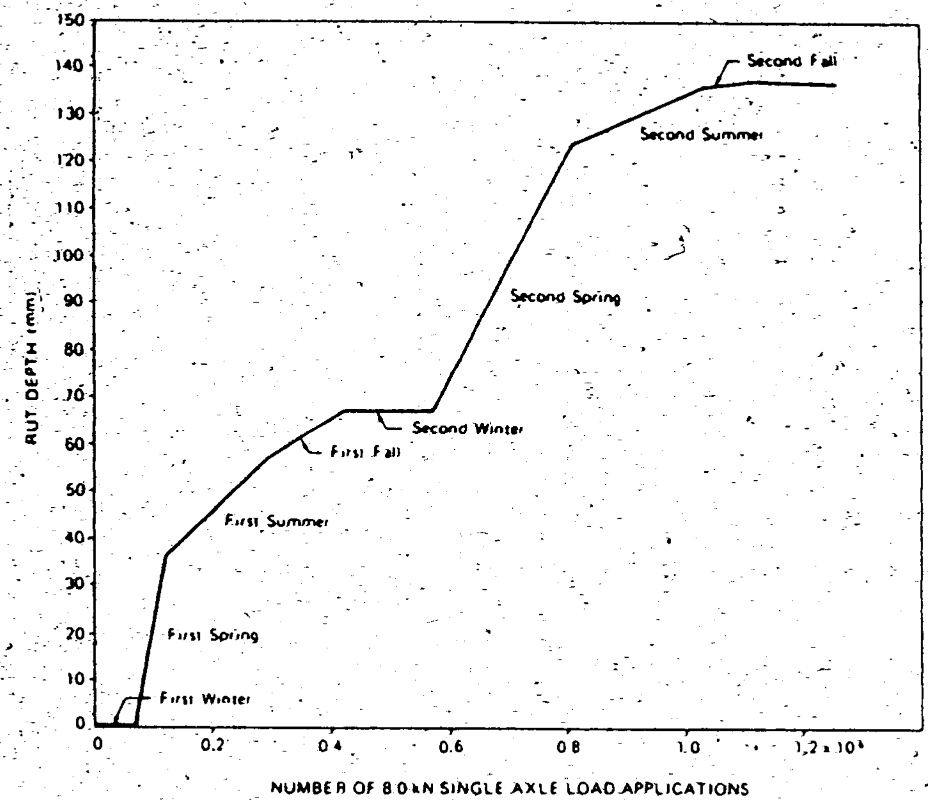


FIGURE 7.7 TYPICAL PATTERN ON RUTTING ON AASHO ROAD TEST. (After Ref. 142)

The rut depth prediction model was developed from the data obtained from the AASHO Road Test. A stepwise procedure was used involving (a) the determination of the material properties for each layer and for the subgrade, (b) the determination of the rate of rutting from the observed data, (c) the connection between the rate of rutting and various primary response factors, and (d) the selection of mechanistic models for conventional construction. Figure 7.8 shows the flow diagram for the developed rut depth prediction model. Two prediction equations were obtained.

For pavements with 152 mm or less of asphalt concrete:

$$\log RR = -5.617 + 4.343 \log d - 0.167 \log (N_{18}) - 1.118 \log \sigma_c \quad (7.11)$$

For pavements with more than 152 mm of asphalt concrete:

$$\log RR = -1.173 + 0.717 \log d - 0.657 \log (N_{18}) + 0.666 \log \sigma_c \quad (7.12)$$

where

RR = rate of permanent deformation or rutting,

d = surface deflection, and

σ_c = vertical compressive stress in asphalt concrete.

Saraf has suggested that although this model was developed for the AASHO Road Tests, it could be used for other conditions either as it is, or after the introduction

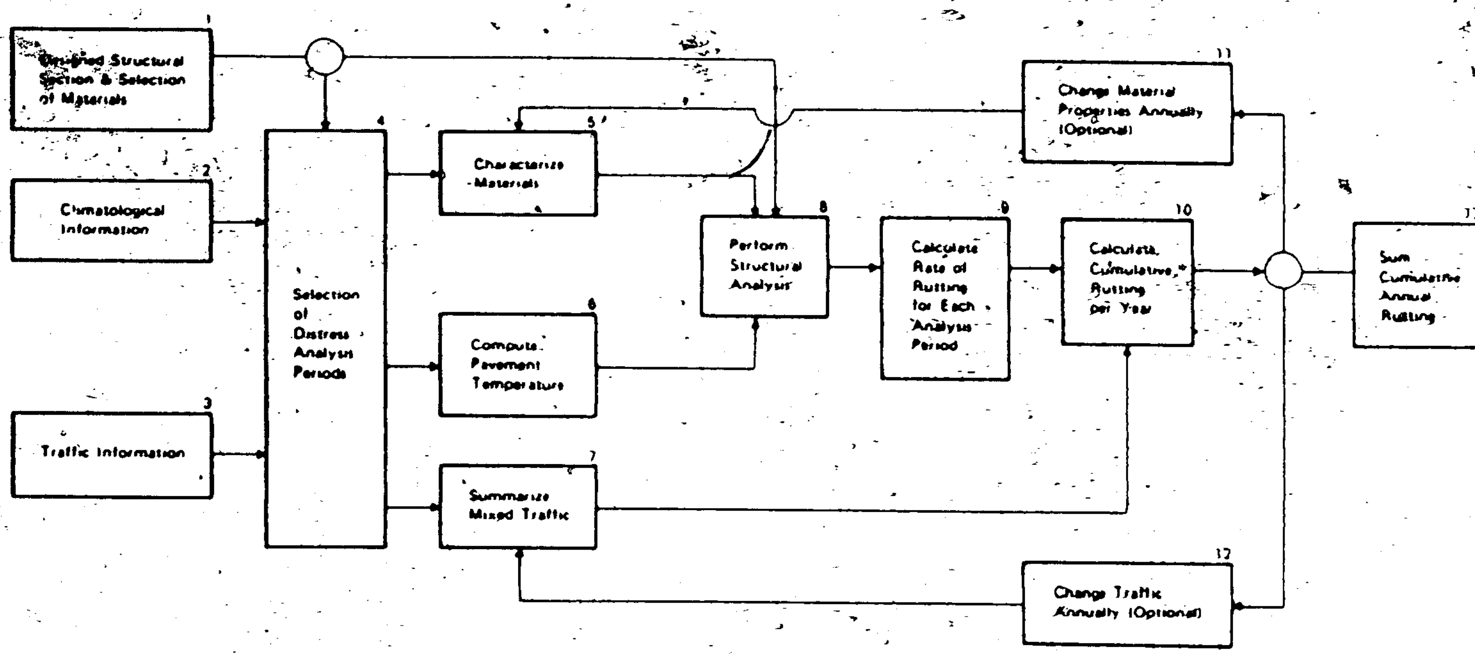


FIGURE 7.8 FLOW DIAGRAM FOR THE RUT DEPTH PREDICTION MODEL. (After Ref. 132)

of appropriate changes in the fitting coefficients.

Uzan et al. (146) have performed minor modifications to the rate of rutting prediction model. These modifications include the direct use of asphalt concrete and subgrade moduli and simplification of the computational procedure. The modified model has shown to be capable of predicting rut depths within reasonable limits.

Models on the Basis of Creep Test:

Many investigators have proposed the use of creep test on asphalt concrete, together with elastic layer theory, to represent the response of the pavement structure to load.

Bolk (147) has investigated the practical applicability and accuracy of some existing methods, such as the Shell and Huschek's method, for the prediction of pavement permanent deformation based on static creep test. Furthermore, he has introduced the development of a new prediction method based on static as well as semidynamic creep tests. He has shown, in Figure 7.9, a quantitative diagram of the deformation and stress during the static and semidynamic creep test.

The Shell method (148) for predicting permanent deformation is a subsystem of the general Shell design method (149) for asphalt pavements. It is a universal method, which in principle can be used for prediction of permanent deformation in any asphalt mixes and pavement structures under various traffic loads and climatological conditions. A flow chart of the Shell method is presented in Figure 7.10.

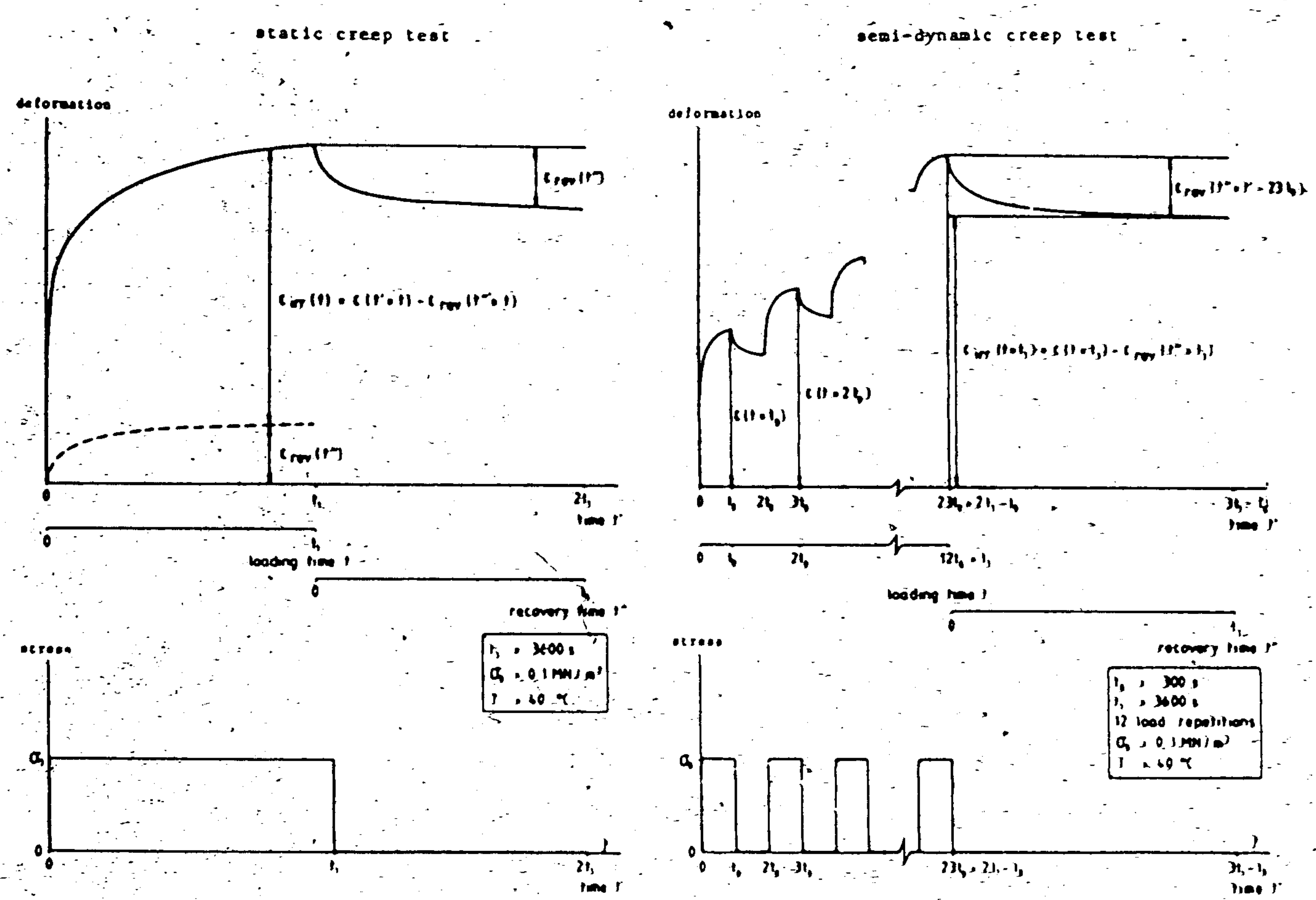


FIGURE 7.9 QUALITATIVE DIAGRAM OF THE DEFORMATION AND STRESS DURING THE STATIC AND SEMI-DYNAMIC CREEP TEST.

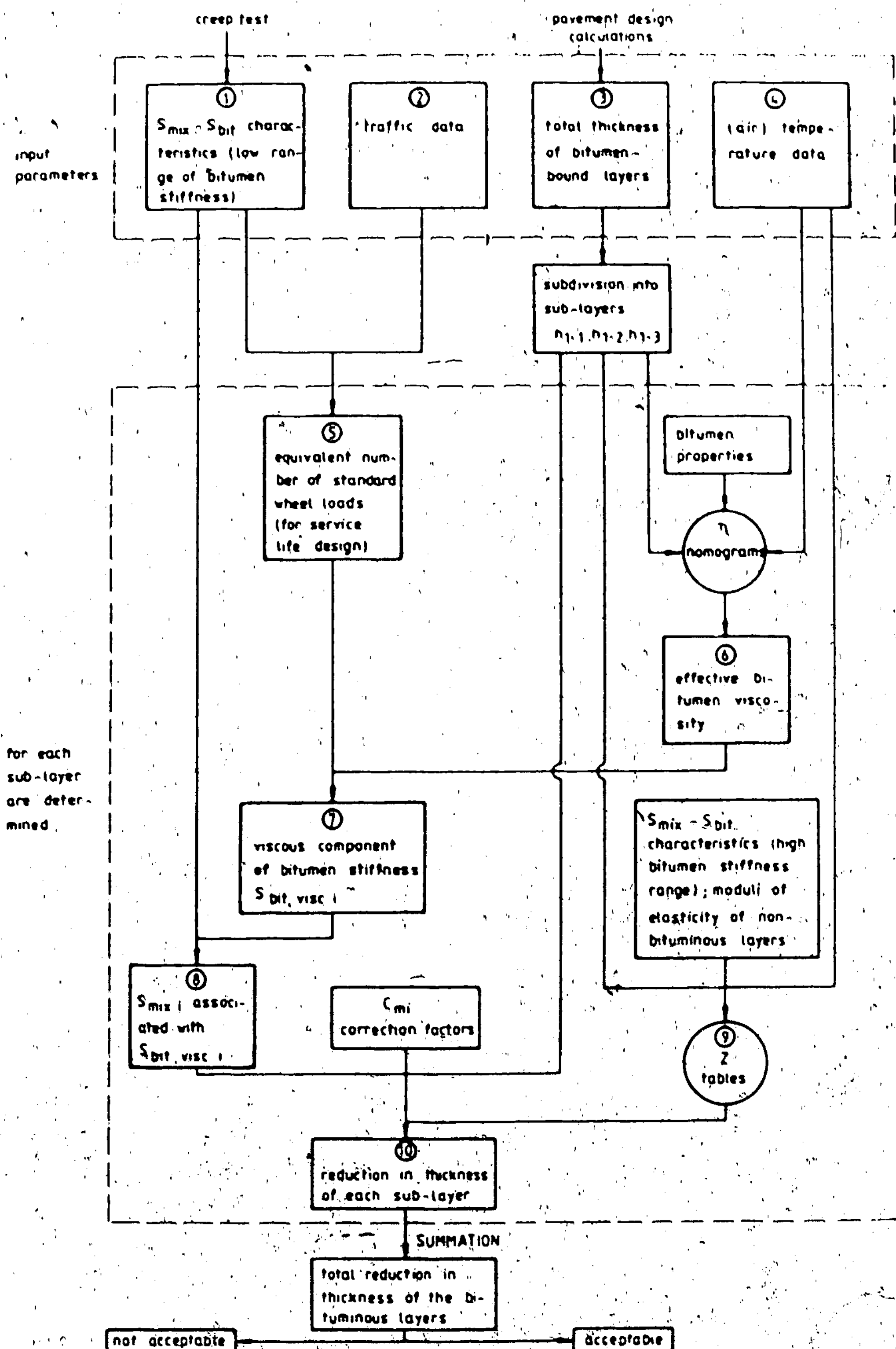


FIGURE 7.10 FLOW DIAGRAM FOR THE PREDICTION OF RUTTING IN PRACTICE ACCORDING TO THE SHELL METHOD.

The basic formula used for calculating the change in the thickness of each sublayer is given by:

$$\Delta h_i = C_{mi} h_i \frac{Z_i \sigma_o}{S_{mix,i}} \tag{7.13}$$

where

Δh_i = change in the thickness of each sub-layer,

C_{mi} = a correlation factor for the dynamic effect,

h_i = the original thickness of sublayer i,

Z_i = a correction factor for the difference in state of stress between the creep test and practice,

σ_o = the contact stress between the standard wheel and road surface, and

$S_{mix,i}$ = the stiffness modulus of the asphalt mix from sublayer i.

Another method of calculating expected permanent deformation in asphalt concrete pavements is the Huschek's method (150), which is based on creep test. This method utilizes the linear elastic multi-layer theory to calculate the stress and strain distribution in the pavement under the influence of a standard wheel load, using the BISAR computer program (151). The wheel loads spectrum is converted into the standard one and an equivalent number of wheel loads is then calculated. The static stress and strain distribution is changed into a dynamic one, which means that the stress and strain at each point as well as deflections become time dependent functions. The permanent deformation of any particular asphalt concrete layer is then calculated on the assumption that the material possesses visco-elastic properties. This calculation is based on a relationship between the permanent change in thickness of a particular layer, and the integral of the reversible (elastic) change in layer thickness according to the following equation:

$$\Delta h_{irr}(y)\,\frac{\eta V}{E} = \int \Delta h_{rev} \cdot dx \qquad (7.14)$$

where

Δh_{irr} = permanent (irreversible) change in thickness of a layer,

Δh_{rev} = elastic (reversible) change in thickness of a layer,

E = Young's modulus,

η = viscosity, and

V = speed of the standard wheel.

The total permanent deformation is finally obtained by summing the contributions from each layer.

It has been shown that among the predictive methods based on creep test, the Shell method produces relatively better results. An important advantage with respect to its practical usefulness is the fact that, this method is presented in the form of a manual which can be easily used.

7.6.3.2 Pavement as a Viscoelastic Layered System

The theory of viscoelasticity has been used for the computation of permanent deformation when considering the pavement as a viscoelastic layered system. This method offers a direct approach to the prediction of permanent deformation, in that, the employed constitutive relationships contain the time dependent factors. Therefore, the vertical deflection at the surface, due to moving load, can be calculated directly as a function of time. A part of this deflection is recoverable and the other part, which can be calculated, is non-recoverable.

Elliot and Moavenzadeh (152) have presented the viscoelastic approach. They have denoted that asphalt concrete pavement can be represented as a linear viscoelastic layered system and a creep test can be used to represent the response characteristics of the constituent materials in the pavement structure. Brademeyer (153) has revised this approach to include an exponential law that accounts for the nonlinearities of the accumulated permanent deformation.

Using the viscoelastic approach, the permanent deformation of a material specimen subjected to a single stress pulse can be expressed in terms of:

$$\varepsilon_p = f\ (\sigma,\ \xi,\ T,\ M) \tag{7.15}$$

where

ε_p = permanent deformation,

σ = stress,

ξ = time duration of loading,

T = temperature, and

M = moisture content.

The total strain response due to a single stress pulse which is given by the following expression, is illustrated by Kenis et al. (154) in Figure 7.11.

$$\varepsilon(t) = \left(\varepsilon_E + \varepsilon_{VE}\right)_{recoverable} + \left(\varepsilon_V + \varepsilon_{PL}\right)_{permanent} \quad (7.16)$$

where

$\varepsilon(t)$ = total strain response to a single stress pulse,

ε_E = elastic strain,

ε_{VE} = viscoelastic strain,

ε_V = viscous strain, and

ε_{PL} = plastic strain.

There are a number of computer programs available for calculating permanent deformation using viscoelastic theory. One of the well established programs is the VESYS II M rutting structural subsystem which takes into account the nonlinearities of the accumulated permanent deformations.

Kenis (155) and McLean et al. (136) have indicated that the pavement permanent deformations estimated using the viscoelastic approach are substantially less than those

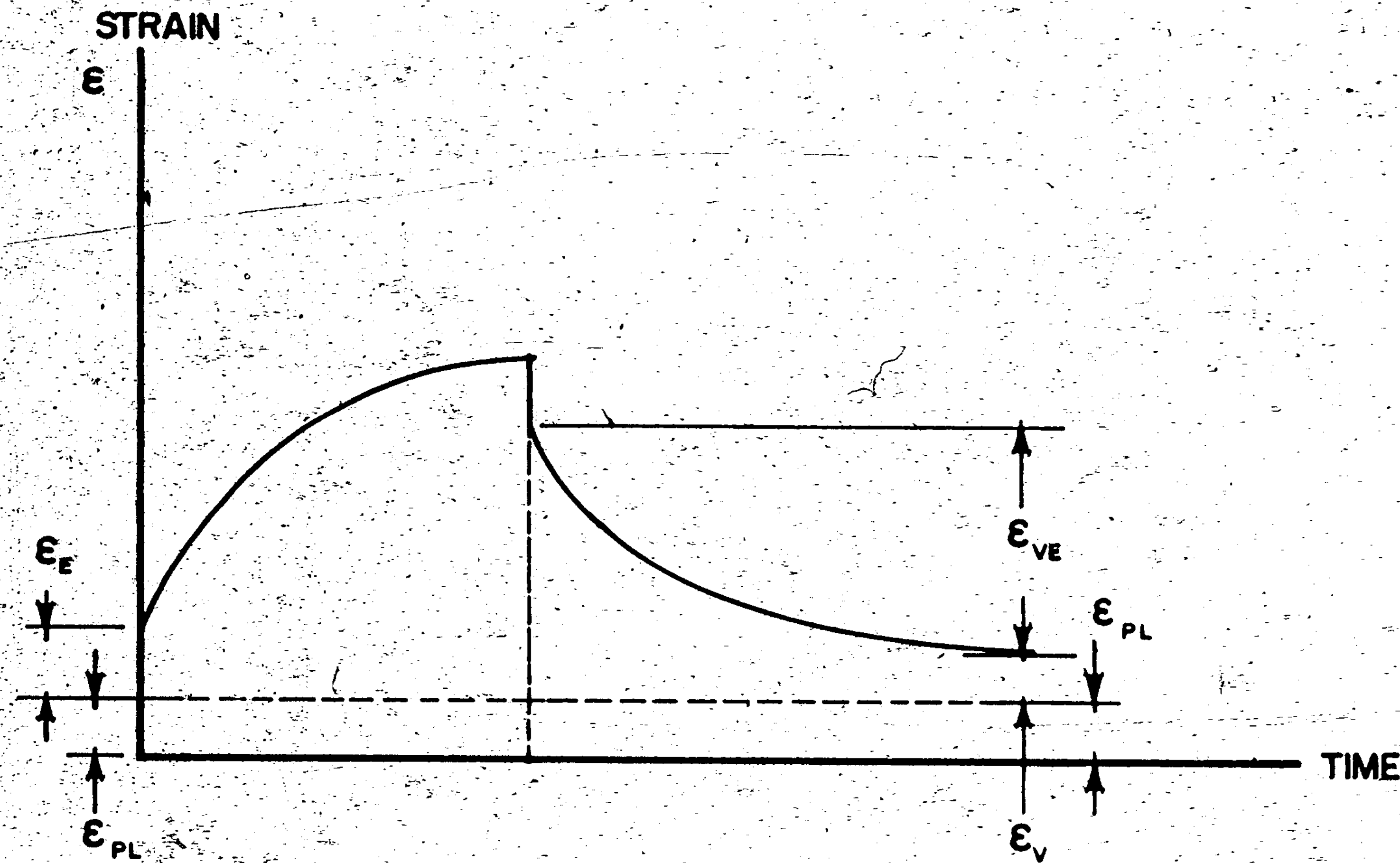


FIGURE 7.11 STRAIN RESPONSE TO STRESS PULSE. (After Ref. 154)

computed using the elastic analysis for essentially the same conditions.

7.6.4 Approach Chosen in This Investigation

The permanent deformation of asphalt concrete pavements is basically a problem of material characterization, loading and environmental conditions. It is therefore, almost impossible to establish accurate constitutive equations which take into account all variables involved in this phenomenon.

As discussed earlier, there is no unique theoretically based technique which can be used as a means of predicting permanent deformation. Existing design technology has shown some lack of confidence in predicting the permanent deformation in conventional asphalt concrete pavements. Introducing another variable such as recycling ratio would make the situation more complex and may impose restrictions on the development of a theoretical predictive technique.

In this study, an attempt is made to establish constitutive relationships from actual physical measurements. The approach chosen is based on the evaluation of the permanent deformation characteristics of the material by using laboratory tests conducted under simulated field conditions.

A repeated load triaxial type test was chosen because of its ability to duplicate the field conditions more closely than any other test. The test examines the behaviour of several mixes having different recycling ratios at various temperatures under representative compressive loading and environmental conditions. By using the triaxial test results the resilient modulus of various mixes, which is defined as the ratio of the repeated axial deviator stress to the recoverable axial strain can also be determined. Models will be developed from the observed phenomenon by using an appropriate statistical approach. The intent is to fit a relationship to data and to develop predictive equations rather than fitting the data into some postulated functions. This approach attempts to develop constitutive relationships from the actual data without presupposing any properties or deformation law.

7.7 Experimental Program

The primary objective of this particular testing program was to determine the permanent deformation characteristics and resilient modulus of the conventional and recycled asphalt concrete materials subjected to various recycling ratio, loading, and environmental conditions. A comparison was to be made between the behaviour of conventional and recycled materials and constitutive relationships were to be established to define their characteristics.

Field conditions were to be simulated in the experiment in order to give a realistic approximation of the resilient modulus and permanent deformation experienced in the field.

Pavements are exposed to mixed traffic at random intervals and different speeds. Therefore they are repeatedly loaded and unloaded by the moving traffic. This phenomenon leads to the consideration of repetitive load testing when pavement material properties are being evaluated. A repeated loading apparatus should include a system that can load and unload the specimen at a desired frequency, and also be capable of measuring and recording the applied stresses and resulting strains. The triaxial test was considered to be the most satisfactory means of determining the permanent deformation characteristics and the resilient modulus of the conventional and recycled asphalt concrete materials.

7.7.1 Testing Conditions

Permanent deformation may develop in all the different layers of asphalt concrete pavements. However, the asphalt concrete layer is of main concern in this investigation.

In order to examine the problem in a realistic context, the conditions of a full depth asphalt concrete pavement of a typical thickness of 204 mm were simulated. Thus specimens with a 102 mm diameter and a 204 mm thickness were fabricated using the kneading compactor to fulfill this purpose.

The field conditions were simulated with respect to loading, traffic, and environment. These variables were obtained from a variety of sources. The study concentrates on several variables influencing the behaviour of the conventional and recycled asphalt concrete pavements and it determines the effect of each variable on the development of permanent deformation in the pavement. A number of pilot studies were conducted to examine whether the selected levels of variables were reasonable and to ensure that outputs were measurable. In order to simplify the complexity of the problem, all variables except the recycling ratio and the temperature, which are the major concerns in this study, were maintained at the same level throughout the experiment.

The stress distribution within a typical full depth flexible pavement were estimated using a multilayered elastic computer program called CHEV5L. Figure 7.12 shows the axial and lateral stress distribution determined from the CHEV5L program for a typical 204 mm thick asphalt concrete layer. After a preliminary study of the estimated stress distributions and a consideration of the limits imposed by the testing apparatus, a compressive axial stress (σ_1) of 350 kPa (50 psi) and a lateral stress (σ_3) of 35 kPa (5 psi) were selected to be used in this testing program.

The load was to be applied to the specimen in the triaxial cell in the form of a square pulse. Barksdale (156) has found that pulse time is a function of vehicle speed and the depth of the element within the pavement structure. According to Barksdale, a loading time of 0.04 seconds would correspond to a typical vehicle speed of approximately 90 km/h. However, due to the limitations on the testing equipment, the loading time had to be increased to 0.5 seconds which corresponds to a vehicle speed of about 10 km/h for an element at a depth of 102 mm. The longer

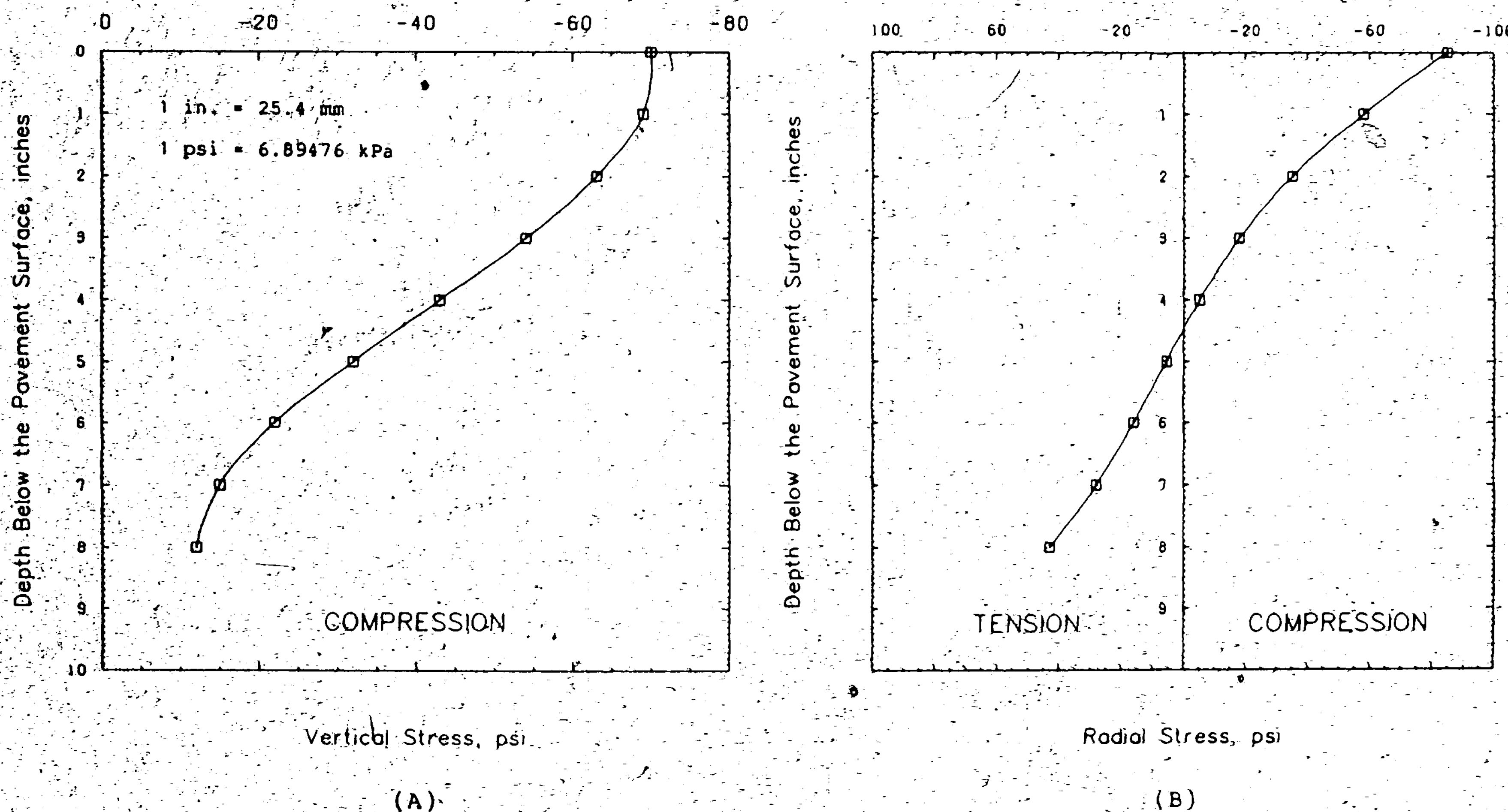


FIGURE 7.12 TYPICAL RELATIONSHIP BETWEEN (A) VERTICAL STRESS AND DEPTH, (B) RADIAL STRESS AND DEPTH.

loading time reduces the anticipated number of loading cycles corresponding to a certain level of deformation.

Morris (116) has indicated that the rest period is relatively unimportant in triaxial compression tests. After a preliminary investigation, a rest period of 0.5 seconds was chosen in this study.

After a preliminary testing, it was found that specimens reach an approximately constant rate of strain at about 10,000 load cycles. However it was decided to load the specimens at up to 100,000 cycles.

Three test temperatures of 25°C, 35°C and 45°C were chosen to examine the change in behaviour of conventional and recycled materials with temperature.

7.7.2 Testing Equipment

A repeated loading triaxial test apparatus was employed in this investigation for the purpose of measuring the permanent deformation and resilient modulus of the various mixes under investigation at different temperatures.

The triaxial test apparatus was designed in a manner such that three specimens can be tested at the same time. This system was originally constructed by Dasmohapatra (157), for testing soil samples, after the model developed by Seed and Fead (158). Major modifications had to be done on the system in order to enable the testing of fabricated asphalt concrete samples at various selected temperatures and also to be able to utilize the dynamic data acquisition system developed particularly for this test.

A steel framework was built for seating three triaxial cells along with all other operating devices. All three cells are operated in a similar manner. The specimen is loaded in a triaxial cell by a piston and loading yoke. The yoke is repeatedly loaded by means of an air-pressure cylinder located below the triaxial cell. Air has to be passed through a regulator and stored in a reservoir tank at a constant pressure. The air tank is connected to the pressure cylinder through a solonoid valve. At the same time as the valve is opened, the air pressure is transmitted through the cylinder and loading yoke to the specimen. When the valve is closed, the air in the cylinder goes through an exhaust pipe and the load is removed. By regulating the air pressure in the cylinder the applied load to the specimen can be varied. The application of pressure to the air-pressure cylinder is controlled by a solenoid-operated three-way valve. An electrical timing unit is used to control the movement of the valve to admit or exhaust air from the pressure cylinder.

The confining pressure in the triaxial cell is created when the air pressure from the reservoir tank is applied to the cell through a pressure supply valve in the base of the triaxial cell. The specimen in the triaxial cell is surrounded by water. The confining pressure is transmitted to the specimen by the confining water. This pressure is measured by a pressure transducer which is connected to a strain indicator.

A temperature control system was of primary importance in order to enable testing at various selected temperatures. The temperature of the specimen is controlled by maintaining the confining water at a constant desired temperature. This is done by circulating water through copper tubes surrounding the specimen. These tubes were installed inside the triaxial cell and were then connected to a controlled temperature water bath. Thermocouples were installed inside the cell at the base of the triaxial cell, where the sample was sitting, for precise temperature measurements of the specimen.

A linear variable differential transducer (LVDT) was mounted on each cell for measuring the deformation of the specimen. The LVDT was clamped to the piston of the triaxial cell and the core was screwed to the cell cap.

The magnitude of the dynamic load was measured by a load cell on the loading yoke which was firmly attached to the piston of the triaxial cell.

The three LVDTs and load cells were connected to a signal conditioner and then to the computerized data acquisition system to record the load and deformation for each applied repeated load. Also, the data acquisition system recorded the total number of pulses applied to the specimen and the particular number of load and deformation readings for which it was programmed.

A detailed description of the repeated load triaxial apparatus, together with the computerized data acquisition system developed particularly for this test, is given in Appendix D.

7.8. Test Results and Analysis

The experiments were conducted on the fabricated specimens under the specified loading and environmental conditions using the repeated loading triaxial test apparatus.

This section presents the results of the triaxial test experimental program for determining permanent deformation and resilient modulus. An analysis of the results and the prediction models which were developed consequently are also presented in this section.

7.8.1 Permanent Deformation Test Results and Analysis

The permanent strain, which is the summation of the plastic and viscous strains, is an indication of the pavement permanent deformation. The permanent strain (ϵ_p), was measured at each load application (N), for all the specimens having various percentages of the reclaimed material in the mix (R), at different test temperatures (T).

The results of every four specimens, belonging to the same mix group, tested at similar loading and environmental conditions were averaged and this value was used for the analysis. Results of the triaxial test including the average and standard deviation of the measured percent axial permanent strain for the selected number of load repetitions, at the test temperatures of 25°C, 35°C and 45°C are summarized in Tables 7.3 to 7.5 respectively.

A plot illustrating the general form of the output, including all the data points for two typical groups of specimens having two extreme R values of 0 and 100, i.e. conventional and 100 percent recycled asphalt concrete, is shown in Figures 7.13 and 7.14. The number of load repetitions are plotted in a logarithmic scale in order to best illustrate the large number of data points. The significant effect of the R values on permanent strain at a selected test temperature is clearly shown in these two plots which have different scales for percent permanent strain.

The percent axial permanent strain (ε_p) versus the number of load repetitions, (N), for all various mixes having a different percentage of reclaimed material, (R), at three specified temperatures are provided in Figures 7.15 to 7.17. The relationship between the percent permanent strain and the number of load repetitions shows some degree of linearity after about 10,000 load cycles. The effect of percent reclaimed material in the mix and also the temperature on percent permanent strain is clearly evident from these results. The value of ε_p increases as R decreases and T increases. Conventional materials, i.e. when R=0, have shown remarkably higher permanent strain compared to other mixes at the same number of load repetitions. They exceeded an accumulated permanent strain of 14.5 percent or a

TABLE 7.3

PERCENT PERMANENT STRAIN FOR VARIOUS MIXES AT 25°C

No. of Load Rep. (N)	percent Permanent Strain at R values of:					
	0	30	50	70	100	50 SC-3000
1	0.011 (0.002)	0.010 (0.009)	0.003 (0.002)	0.002 (0.002)	0.002 (0.002)	0.003 (0.000)
5	0.042 (0.001)	0.036 (0.023)	0.013 (0.004)	0.010 (0.003)	0.007 (0.004)	0.020 (0.004)
10	0.074 (0.004)	0.051 (0.035)	0.022 (0.005)	0.017 (0.007)	0.011 (0.006)	0.033 (0.004)
50	0.201 (0.011)	0.118 (0.051)	0.063 (0.010)	0.055 (0.039)	0.036 (0.019)	0.088 (0.012)
100	0.277 (0.023)	0.155 (0.054)	0.089 (0.011)	0.079 (0.040)	0.056 (0.025)	0.120 (0.015)
500	0.484 (0.041)	0.252 (0.057)	0.162 (0.018)	0.149 (0.073)	0.101 (0.046)	0.219 (0.028)
1000	0.586 (0.062)	0.295 (0.057)	0.200 (0.024)	0.182 (0.087)	0.124 (0.057)	0.272 (0.035)
5000	0.973 (0.081)	0.418 (0.048)	0.303 (0.035)	0.285 (0.135)	0.183 (0.074)	0.411 (0.075)
10000	1.221 (0.098)	0.480 (0.043)	0.358 (0.063)	0.328 (0.158)	0.210 (0.084)	0.481 (0.095)
20000	1.582 (0.166)	0.545 (0.030)	0.415 (0.058)	0.380 (0.487)	0.237 (0.092)	0.566 (0.124)
30000	1.937 (0.225)	0.591 (0.018)	0.452 (0.067)	0.410 (0.203)	0.251 (0.100)	0.619 (0.142)
40000	2.344 (0.275)	0.628 (0.009)	0.481 (0.079)	0.433 (0.212)	0.263 (0.107)	0.665 (0.159)
50000	2.834 (0.275)	0.655 (0.013)	0.504 (0.087)	0.453 (0.215)	0.271 (0.111)	0.700 (0.169)
60000	3.471 (0.374)	0.679 (0.529)	0.529 (0.097)	0.465 (0.225)	0.279 (0.112)	0.729 (0.180)
70000	4.291 (0.803)	0.705 (0.025)	0.547 (0.106)	0.479 (0.231)	0.287 (0.114)	0.763 (0.204)
80000	5.514 (1.650)	0.713 (0.027)	0.564 (0.113)	0.491 (0.236)	0.295 (0.112)	0.785 (0.213)
90000	7.057 (2.809)	0.737 (0.037)	0.578 (0.117)	0.50[illegible] (0.241)	0.298 (0.112)	0.807 (0.217)
100000			0.592 (0.124)	0.510 (0.241)		0.819 (0.224)

Figures in brackets indicate one standard deviation.

TABLE 7.4

PERCENT PERMANENT STRAIN FOR VARIOUS MIXES AT 35°C

No. of Load Rep. (N)	Percent Permanent Strain at R values of:					
	0	30	50	70	100	50 SC-3000
1	0.016 (0.001)	0.010 (0.003)	0.008 (0.001)	0.007 (0.002)	0.006 (0.003)	0.009 (0.001)
5	0.064 (0.013)	0.033 (0.002)	0.031 (0.011)	0.023 (0.006)	0.016 (0.002)	0.041 (0.009)
10	0.111 (0.021)	0.061 (0.010)	0.055 (0.019)	0.039 (0.010)	0.029 (0.005)	0.073 (0.013)
50	0.314 (0.064)	0.155 (0.002)	0.149 (0.061)	0.111 (0.031)	0.082 (0.018)	0.204 (0.047)
100	0.443 (0.103)	0.201 (0.012)	0.204 (0.088)	0.156 (0.042)	0.116 (0.026)	0.282 (0.074)
500	0.923 (0.212)	0.366 (0.016)	0.380 (0.161)	0.286 (0.076)	0.208 (0.048)	0.513 (0.157)
1000	1.244 (0.281)	0.452 (0.036)	0.472 (0.199)	0.353 (0.101)	0.253 (0.060)	0.637 (0.205)
5000	2.829 (0.581)	0.741 (0.073)	0.733 (0.323)	0.545 (0.173)	0.367 (0.081)	1.032 (0.368)
10000	4.438 (0.709)	0.936 (0.085)	0.872 (0.397)	0.646 (0.215)	0.421 (0.094)	1.254 (0.472)
20000	7.593 (0.734)	1.213 (0.068)	1.036 (0.480)	0.769 (0.264)	0.476 (0.107)	1.527 (0.619)
30000	10.570 (1.203)	1.422 (0.023)	1.141 (0.541)	0.850 (0.297)	0.506 (0.115)	1.723 (0.723)
40000	12.664 (1.401)	1.538 (0.037)	1.227 (0.590)	0.913 (0.321)	0.532 (0.121)	1.886 (0.799)
50000		1.735 (0.115)	1.292 (0.635)	0.964 (0.345)	0.554 (0.122)	2.022 (0.868)
60000		1.905 (0.116)	1.366 (0.673)	1.006 (0.364)	0.565 (0.132)	2.153 (0.927)
70000		2.051 (0.115)	1.381 (0.661)	1.043 (0.380)	0.580 (0.132)	2.262 (0.977)
80000		2.186 (0.133)	1.478 (0.732)	1.073 (0.399)	0.590 (0.135)	2.354 (1.024)
90000		2.301 (0.143)	1.516 (0.755)	1.094 (0.405)	0.599 (0.140)	2.428 (1.055)
100000					0.607 (0.145)	2.478 (1.076)

Figures in brackets indicate one standard deviation.

TABLE 7.5

PERCENT PERMANENT STRAIN FOR VARIOUS MIXES AT 45°C

No. of Load Rep. (N)	Percent Permanent Strain at R values of:					
	0	30	50	70	100	50 SC-3000
1	0.018 (0.001)	0.010 (0.001)	0.014 (0.005)	0.012 (0.003)	0.007 (0.004)	0.017 (0.004)
5	0.079 (0.009)	0.040 (0.002)	0.061 (0.016)	0.055 (0.003)	0.028 (0.012)	0.064 (0.013)
10	0.137 (0.012)	0.068 (0.004)	0.107 (0.027)	0.096 (0.005)	0.049 (0.023)	0.115 (0.025)
50	0.411 (0.042)	0.192 (0.013)	0.306 (0.076)	0.273 (0.013)	0.138 (0.065)	0.327 (0.078)
100	0.610 (0.064)	0.265 (0.017)	0.438 (0.116)	0.383 (0.019)	0.195 (0.093)	0.469 (0.119)
500	1.450 (0.220)	0.526 (0.036)	0.840 (0.219)	0.732 (0.050)	0.352 (0.182)	0.938 (0.271)
1000	2.228 (0.375)	0.699 (0.055)	1.058 (0.268)	0.937 (0.093)	0.428 (0.231)	1.221 (0.367)
5000	5.579 (1.444)	1.540 (0.151)	1.759 (0.389)	1.638 (0.305)	0.666 (0.403)	2.281 (0.771)
10000	8.673 (2.235)	2.280 (0.250)	2.218 (0.468)	2.065 (0.437)	0.789 (0.501)	3.048 (1.077)
20000	12.710 (2.146)	3.424 (0.467)	2.824 (0.560)	2.610 (0.619)	0.931 (0.616)	4.198 (1.567)
30000		4.423 (0.651)	3.326 (0.904)	3.022 (0.764)	1.024 (0.694)	5.165 (2.026)
40000		5.338 (0.895)	3.771 (0.760)	3.380 (0.879)	1.094 (0.752)	6.075 (2.468)
50000		6.149 (1.099)	4.198 (0.876)	3.690 (0.996)	1.153 (0.800)	6.931 (2.857)
60000		6.874 (1.289)	4.632 (1.005)	3.981 (1.115)	1.204 (0.844)	7.750 (3.176)
70000		7.499 (1.439)	5.092 (1.158)	4.255 (1.252)	1.248 (0.885)	8.524 (3.451)
80000		8.046 (1.578)	5.576 (1.319)	4.537 (1.409)	1.293 (0.930)	9.270 (3.653)
90000		8.554 (1.731)	6.067 (1.486)	4.815 (1.570)	1.333 (0.971)	9.938 (3.743)
100000						

Figures in brackets indicate one standard deviation.

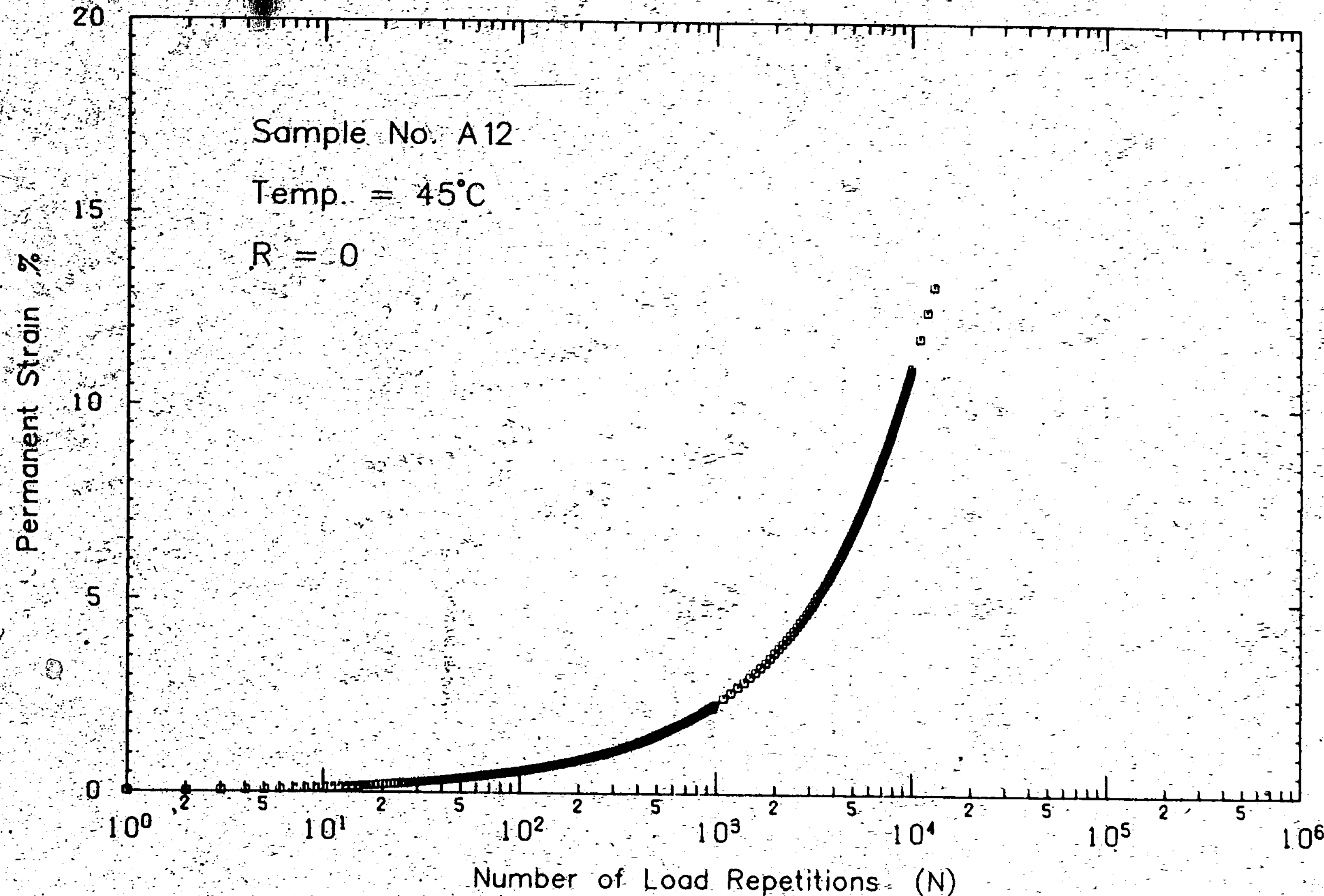


FIGURE 7.13 TYPICAL RELATIONSHIP BETWEEN PERMANENT STRAIN AND NUMBER OF LOAD REPETITIONS.

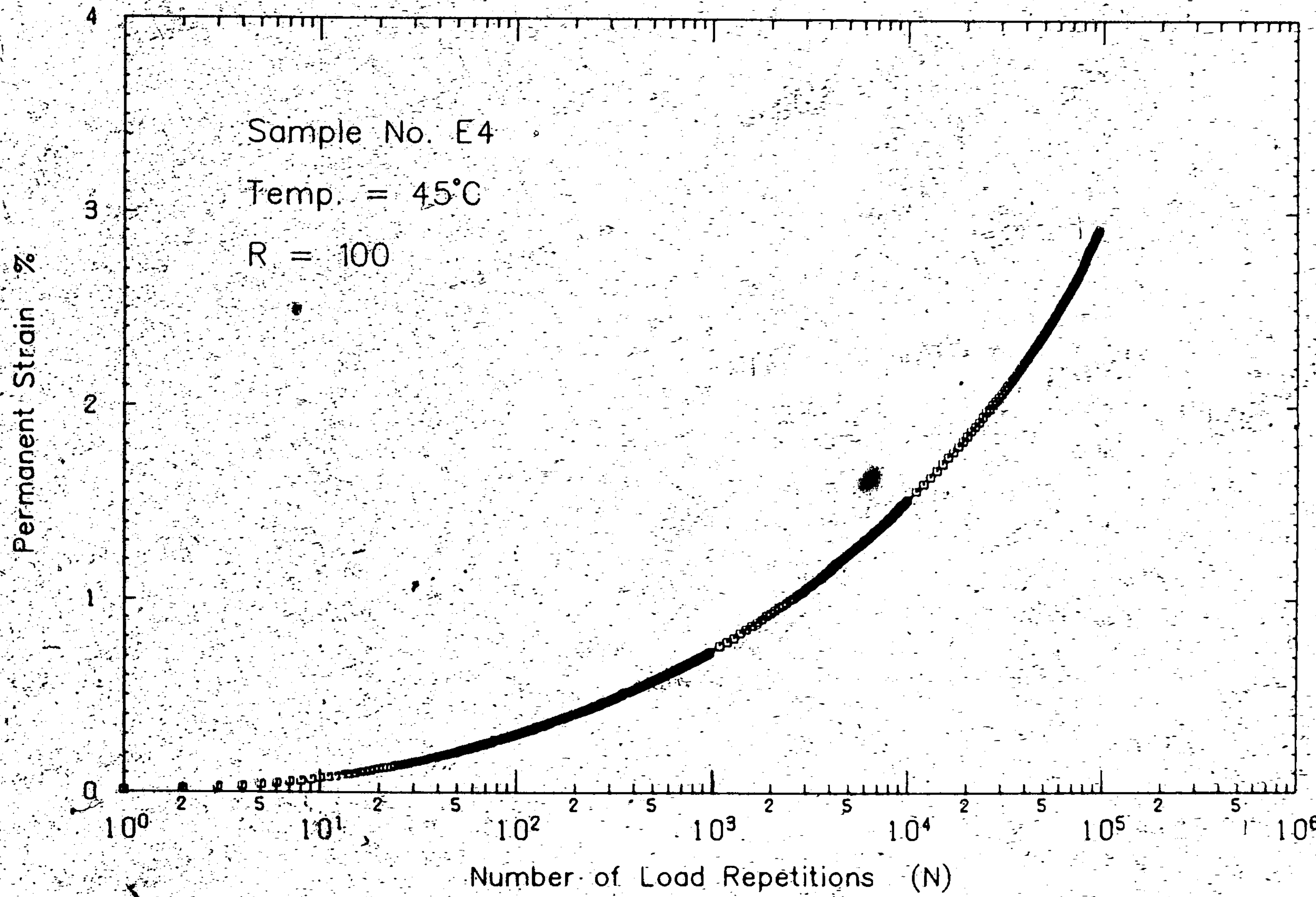


FIGURE 7.14 TYPICAL RELATIONSHIP BETWEEN PERMANENT STRAIN AND NUMBER OF LOAD REPETITIONS.

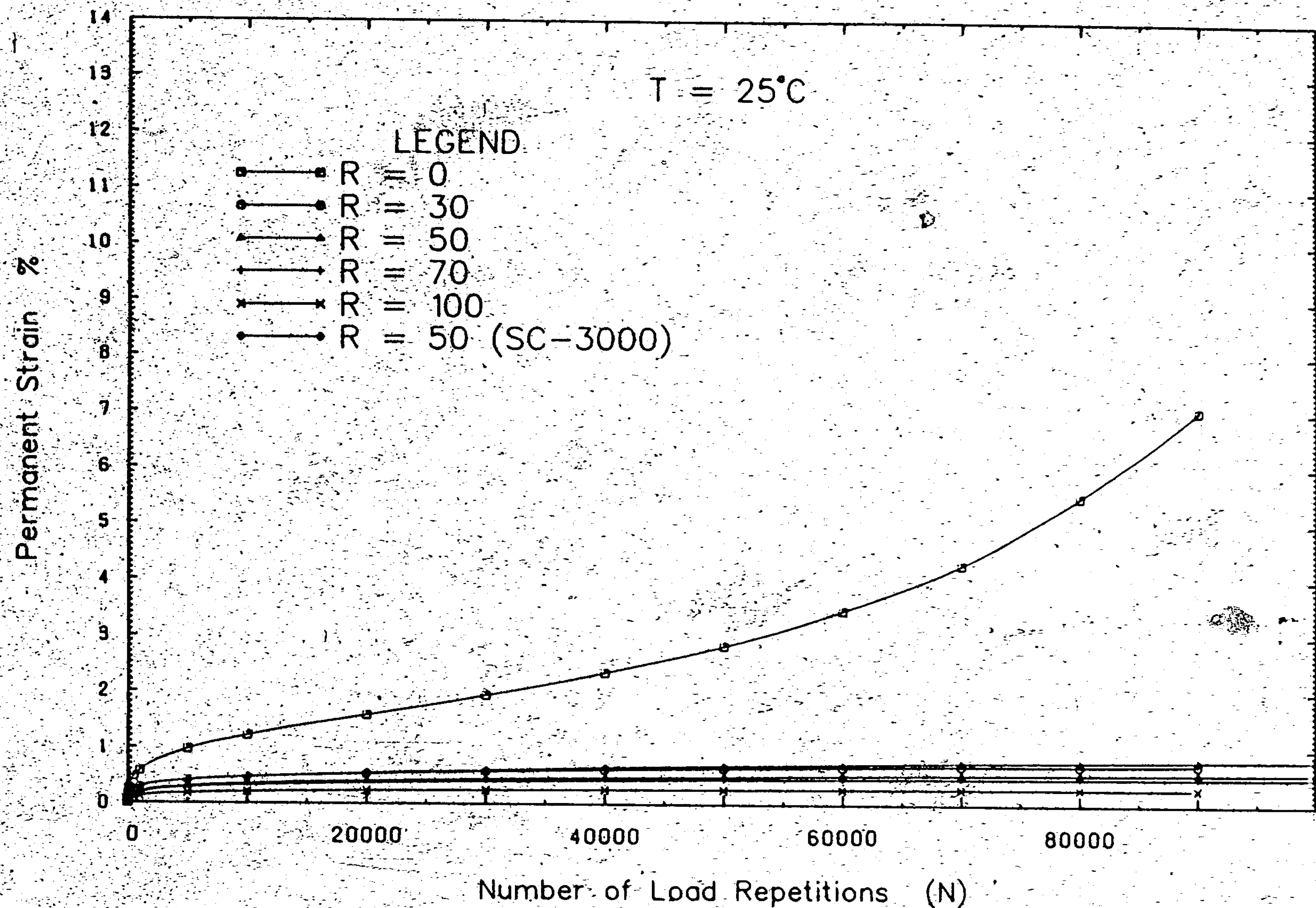


FIGURE 7.15 RELATIONSHIP BETWEEN PERMANENT STRAIN AND NUMBER OF LOAD REPETITIONS AT 25°C.

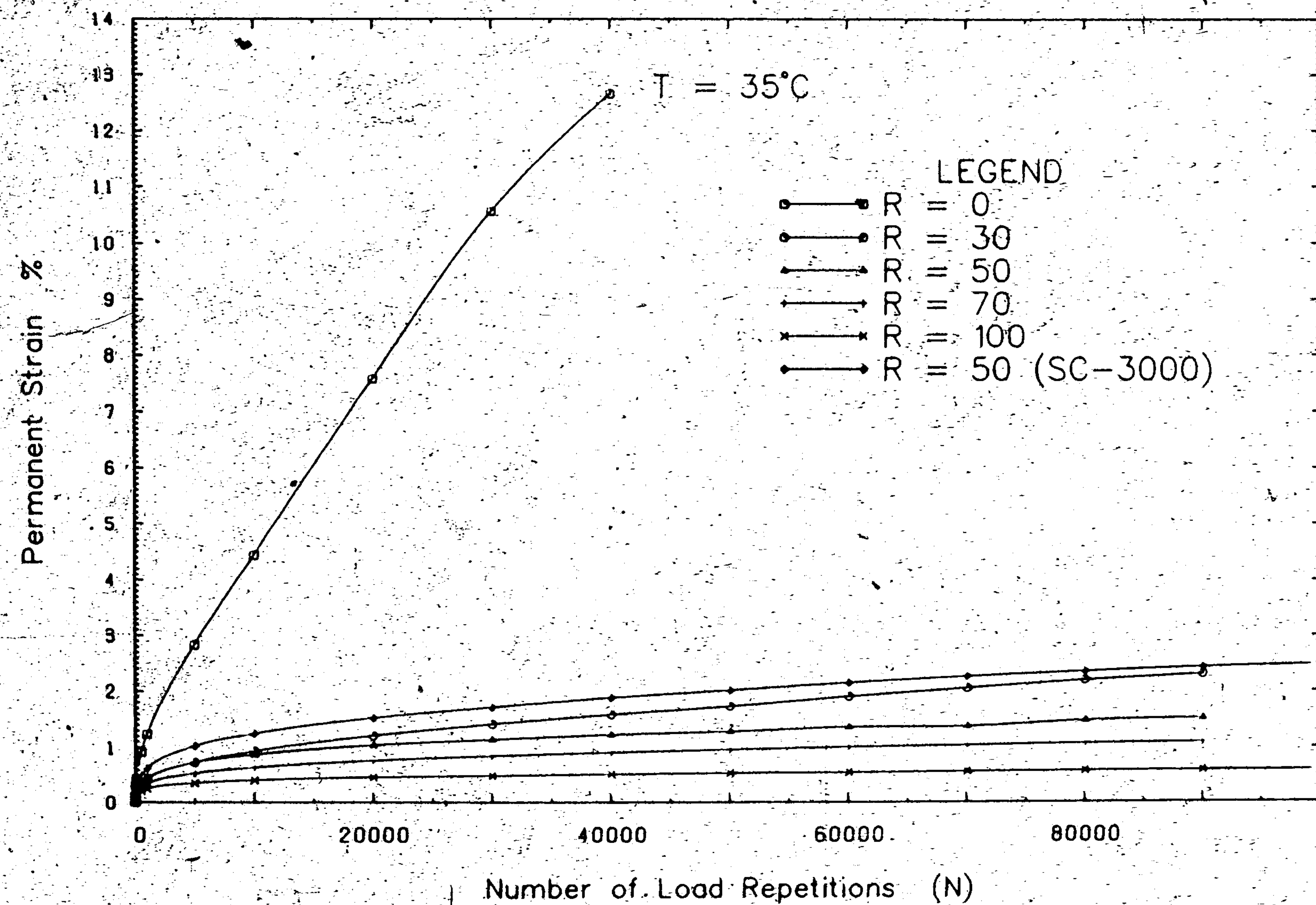


FIGURE 7.16 RELATIONSHIP BETWEEN PERMANENT STRAIN AND NUMBER OF LOAD REPETITIONS AT 35°C.

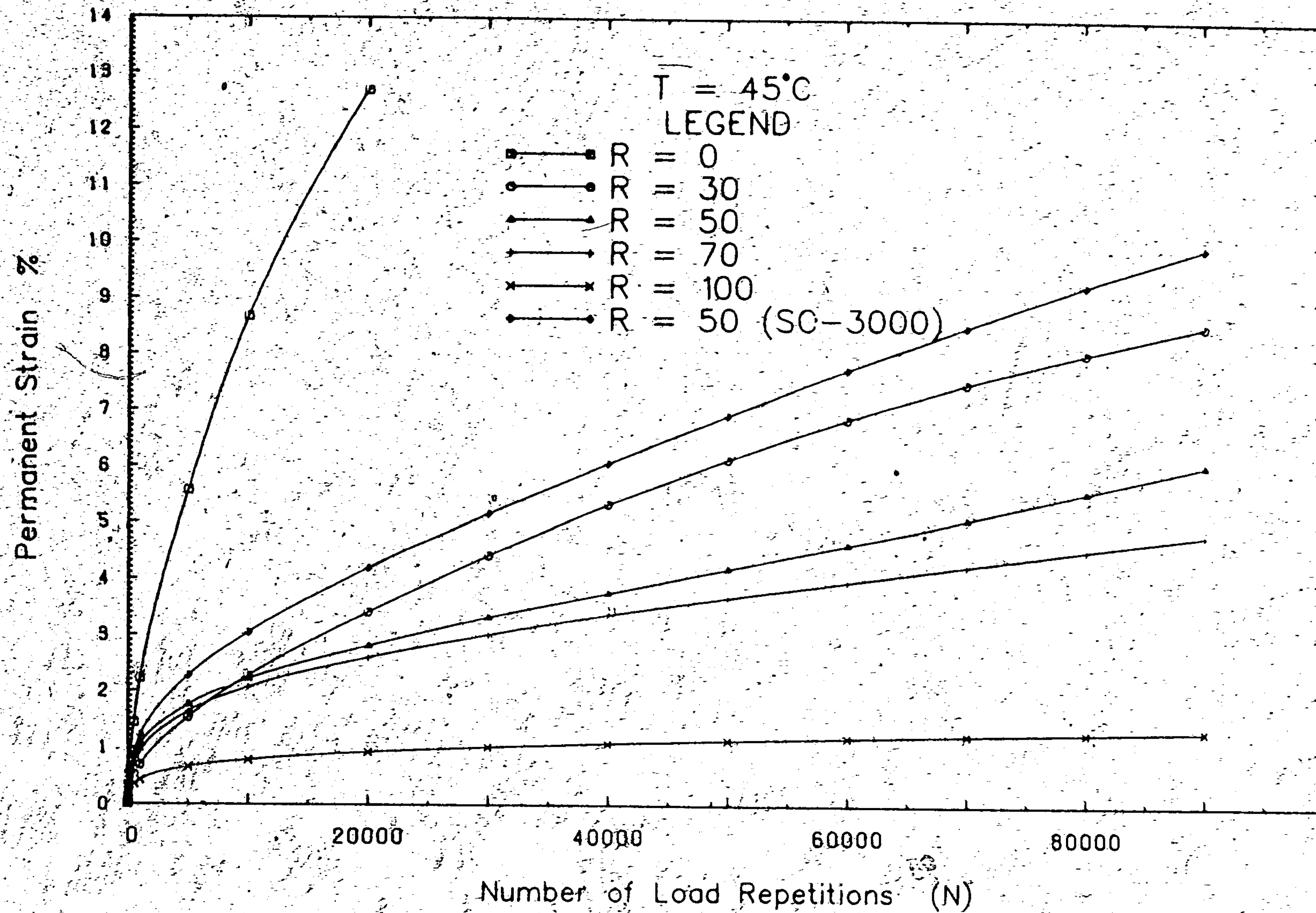


FIGURE 7.17 RELATIONSHIP BETWEEN PERMANENT STRAIN AND NUMBER OF LOAD REPETITIONS AT 45°C.

permanent deformation of about 30 mm, at relatively low numbers of load repetitions. Rut depths of this magnitude would be considered as failure by many pavement agencies. At 25°C conventional mixes reached at this permanent strain level at about 90,000 load cycles, whereas at 35°C they failed at 40,000 and at 45°C failure was at about 20,000 load cycles. Therefore the effect of temperature and mix type is very pronounced on permanent deformation.

It appears that the 50 percent recycled mixes, having SC-3000 as the virgin binder, show considerably higher deformation than those with similar R value but with a 300-400A penetration grade asphalt as virgin binder. Their characteristics are more similar to that of 30 percent recycled mixes. This is due basically to the higher degree of softness of the SC-3000 compared to a 300-400A asphalt cement.

The variation in percent permanent strain versus the number of load repetitions at different temperatures is also shown in logarithmic scales in Figures 7.18 to 7.20. These plots clearly demonstrate a trend and the high rate of increase in permanent strain at early stages of the test. It appears that the increase in ε_p is rather rapid up to approximately 10,000 load repetitions, when it then decelerates and the relationship between ε_p and N becomes relatively linear. However this may not have been quite verified for conventional mixes.

Figure 7.21 shows the variation of permanent strain with

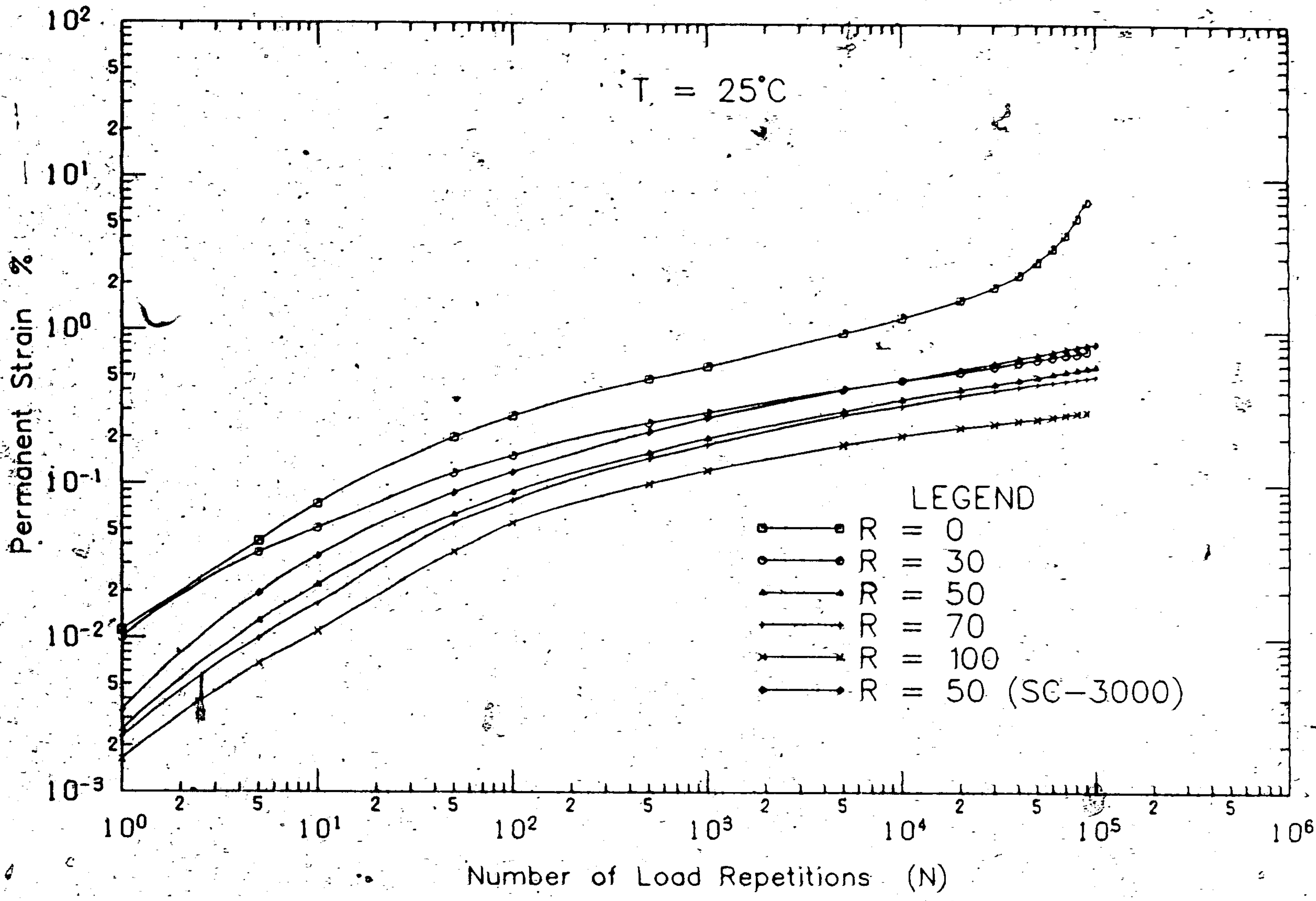


FIGURE 7.18 RELATIONSHIP BETWEEN LOG PERMANENT STRAIN AND LOG NUMBER OF LOAD REPETITIONS AT 25°C.

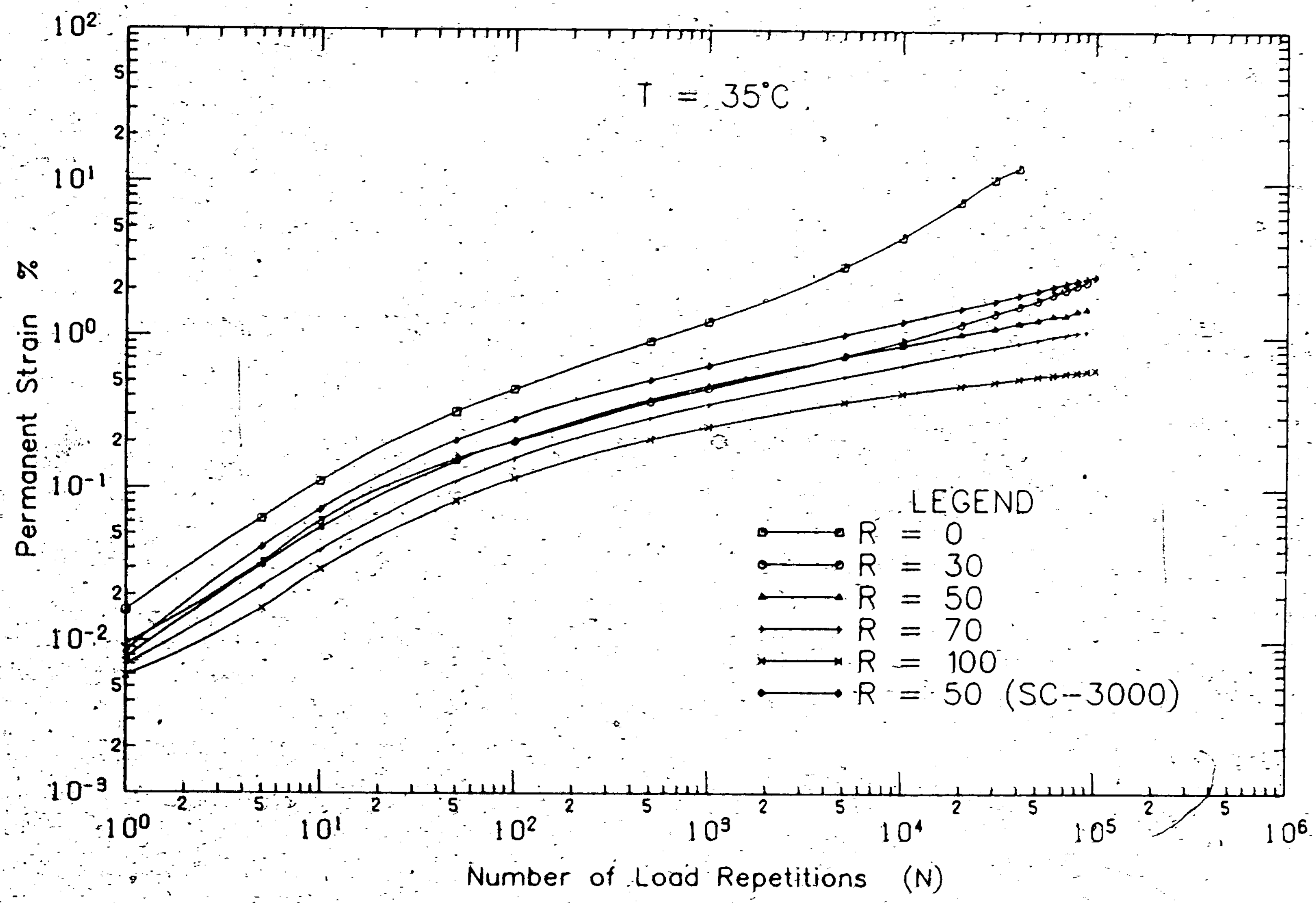


FIGURE 7.19 RELATIONSHIP BETWEEN LOG PERMANENT STRAIN AND LOG NUMBER OF LOAD REPETITIONS AT 35°C.

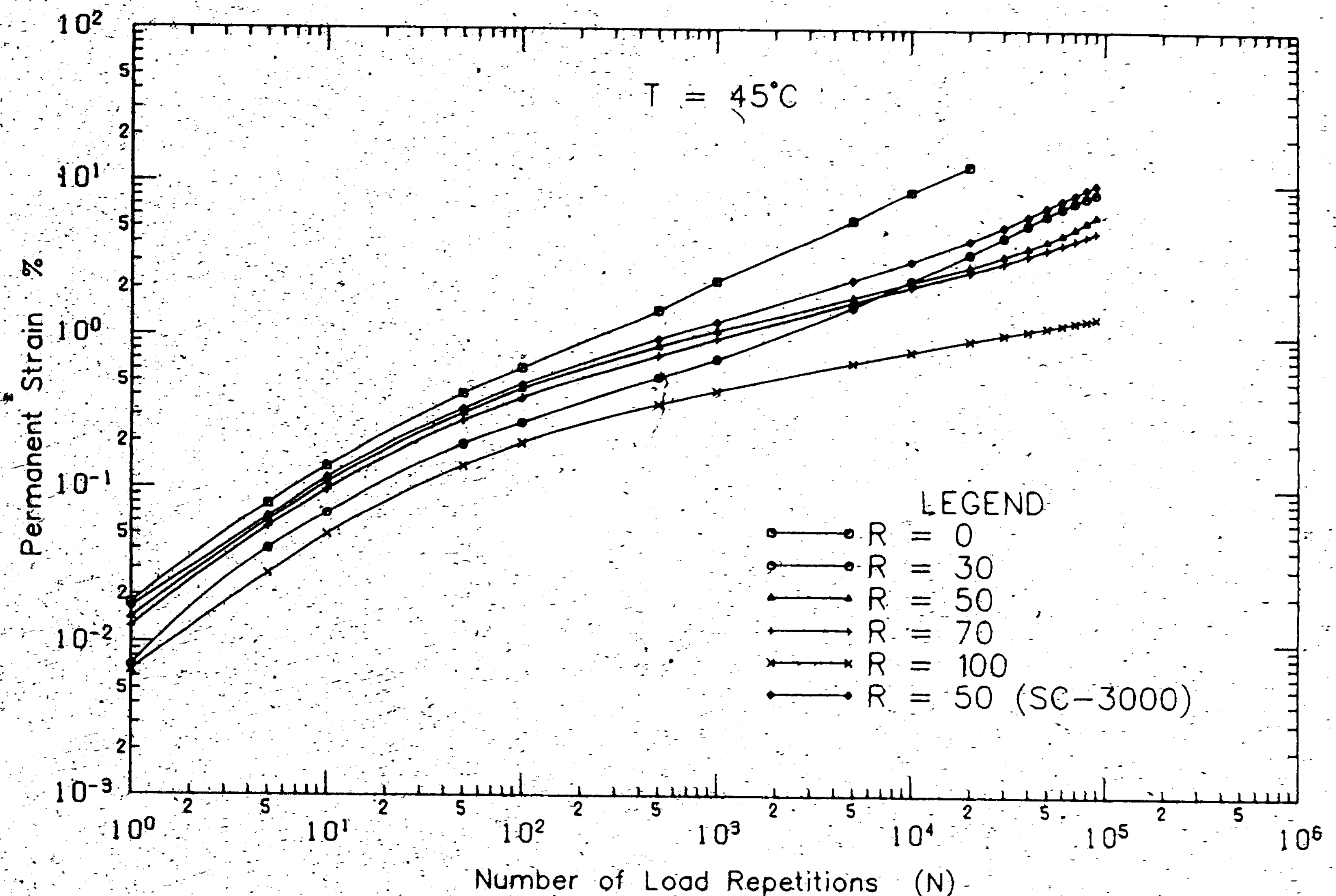


FIGURE 7.20 RELATIONSHIP BETWEEN LOG PERMANENT STRAIN AND LOG NUMBER OF LOAD REPETITIONS AT 45°C.

temperature, observed at 10,000 load cycles for various mixes. It is apparent that temperature has a significant effect on permanent strain. Higher temperatures lead to a higher degree of permanent deformation. Figure 7.21 also exhibited that the deformation of the conventional mixes are more affected by temperature than those of the recycled mixtures.

The effect of the reclaimed material contained in the mix (R) on the permanent strain at various temperatures at 10,000 load repetitions is best illustrated in Figure 7.22. An increase in the amount of reclaimed material in the mix have resulted in a decrease in permanent strain. A sharp decrease in ε_p is observed when R changes from 0 to 30, i.e. from conventional to 30 percent recycling. Hence, a relatively small quantity of reclaimed material in the mix can reduce the deformation considerably. As an example, to illustrate the effect of temperature on permanent deformation, at 45°C and after 10,000 load repetitions, full recycled mixes have shown a permanent strain of about 0.8 percent whereas conventional mixes subjected to similar loading condition and temperature have exhibited a strain which is approximately 10 times higher than that for full recycled mixtures.

In order to make a comparison between the mixtures, the t-test was performed on permanent deformation values for various R within each test temperature. The results showed that at 10,000 load repetitions, the effects of R values at

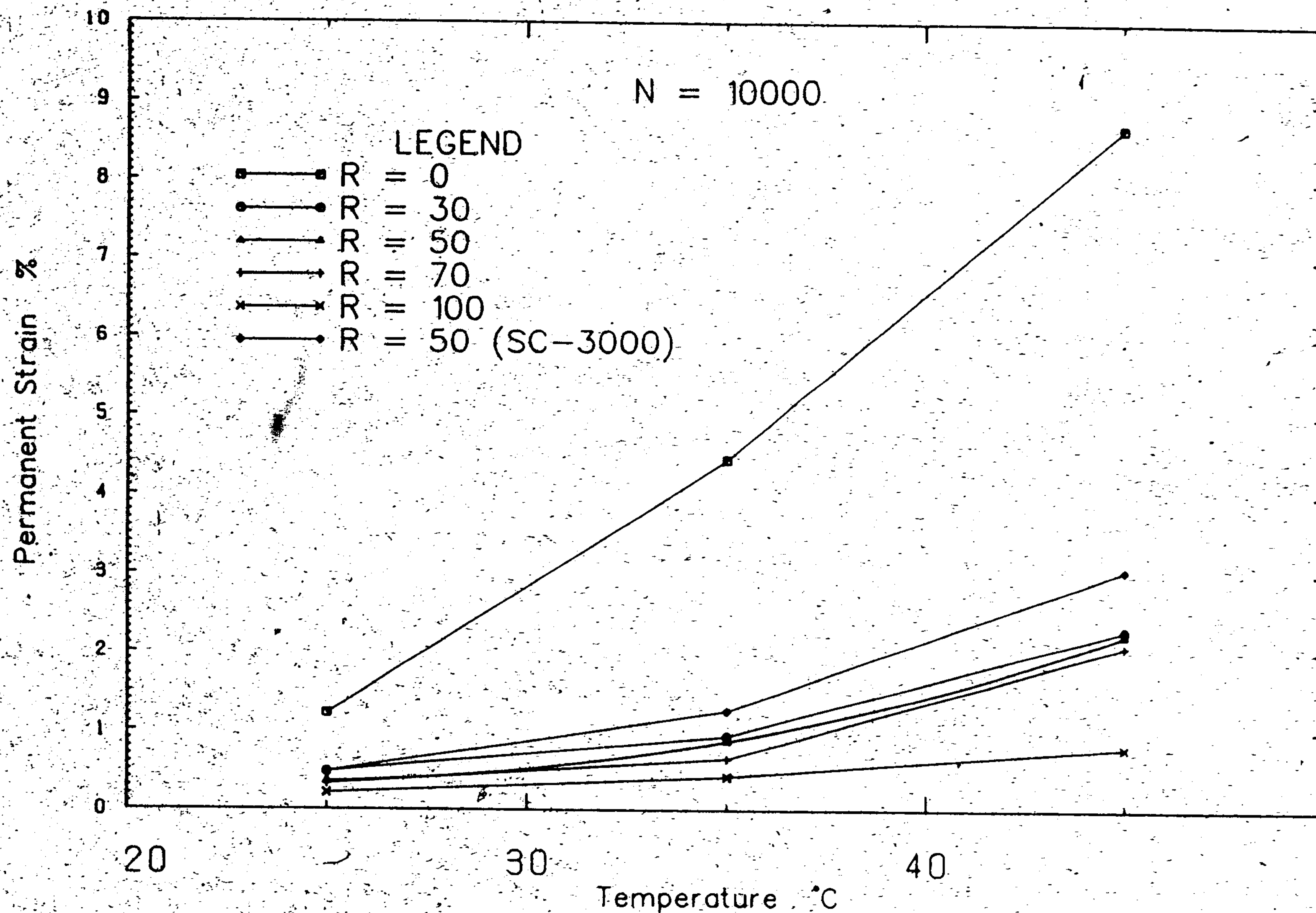


FIGURE 7.21 RELATIONSHIP BETWEEN PERMANENT STRAIN AND TEMPERATURE FOR VARIOUS MIXES.

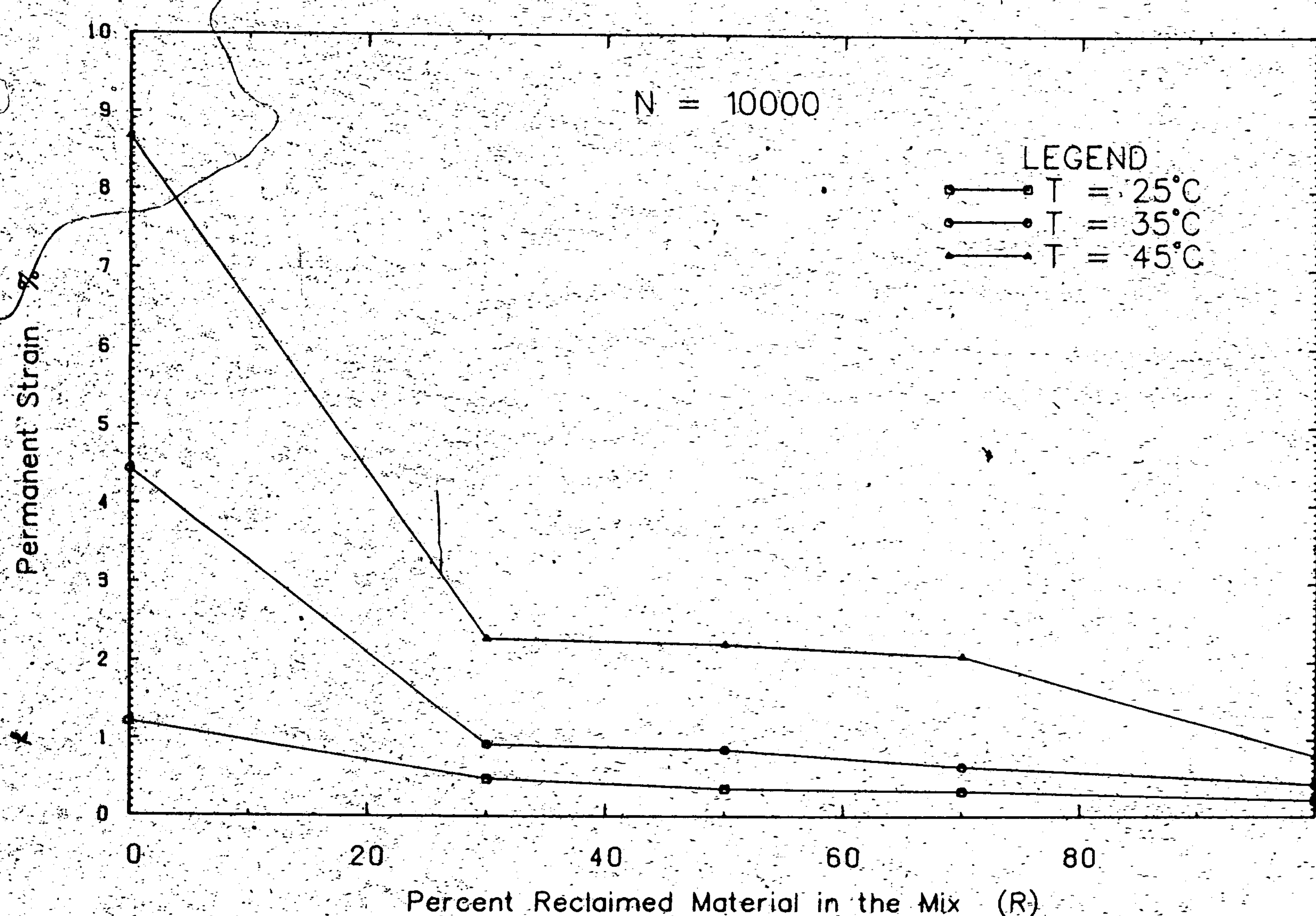


FIGURE 7.22 RELATIONSHIP BETWEEN PERMANENT STRAIN AND PERCENT RECLAIMED MATERIAL IN THE MIX.

30, 50 and 70 percent levels on the permanent deformation were not statistically significant at a 5 percent probability level for any of the three selected temperatures.

The combined effect of the percent reclaimed material in the mix and the temperature on the percent permanent strain at 10,000 and 90,000 load repetitions are shown in the form of contour lines in Figures 7.23 and 7.24. Contour lines show clearly the distinct differences between the conventional and recycled mixtures. As explained above and also by observing these contour lines, at the three chosen test temperatures, the values of permanent deformation for mixtures, containing 30, 50 and 70 percent reclaimed material, decrease slightly. However, statistically they are not significantly different.

7.8.1.1 Development of Models to Predict Permanent Deformation

It was desired to develop a model that could simply and accurately predict the permanent deformation characteristics of conventional and recycled asphalt concrete mixtures using the given independent variables.

Permanent deformation of asphaltic mixtures is a function of many variables as explained in Section 7.5. The most important variables included in this study were the number of load repetitions (N), temperature (T), and the percent reclaimed material in the mix (R) which is considered an indication of mixture characteristics. Due to the

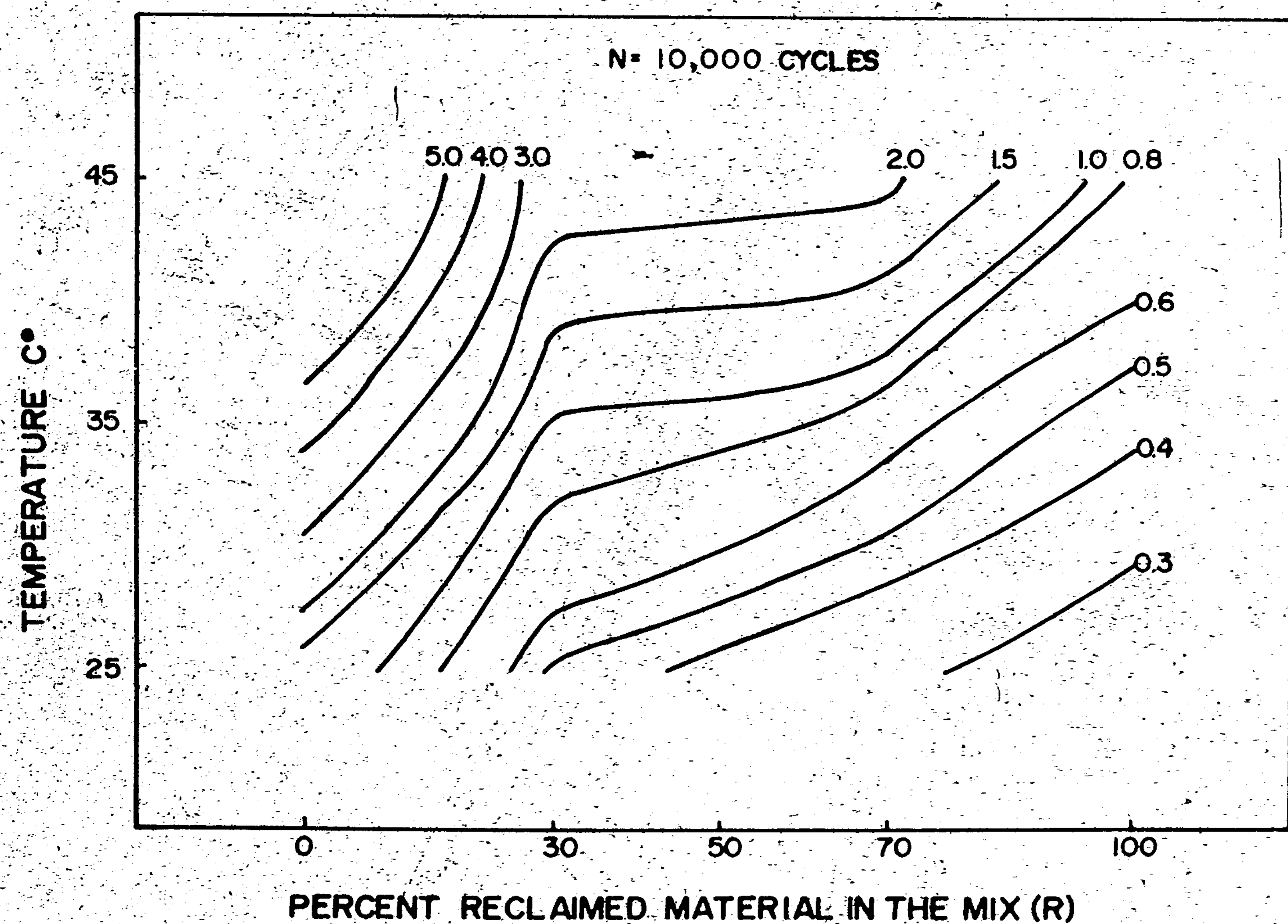


FIGURE 7.23 CONTOUR LINES FOR PERCENT PERMANENT STRAIN AT N = 10,000.

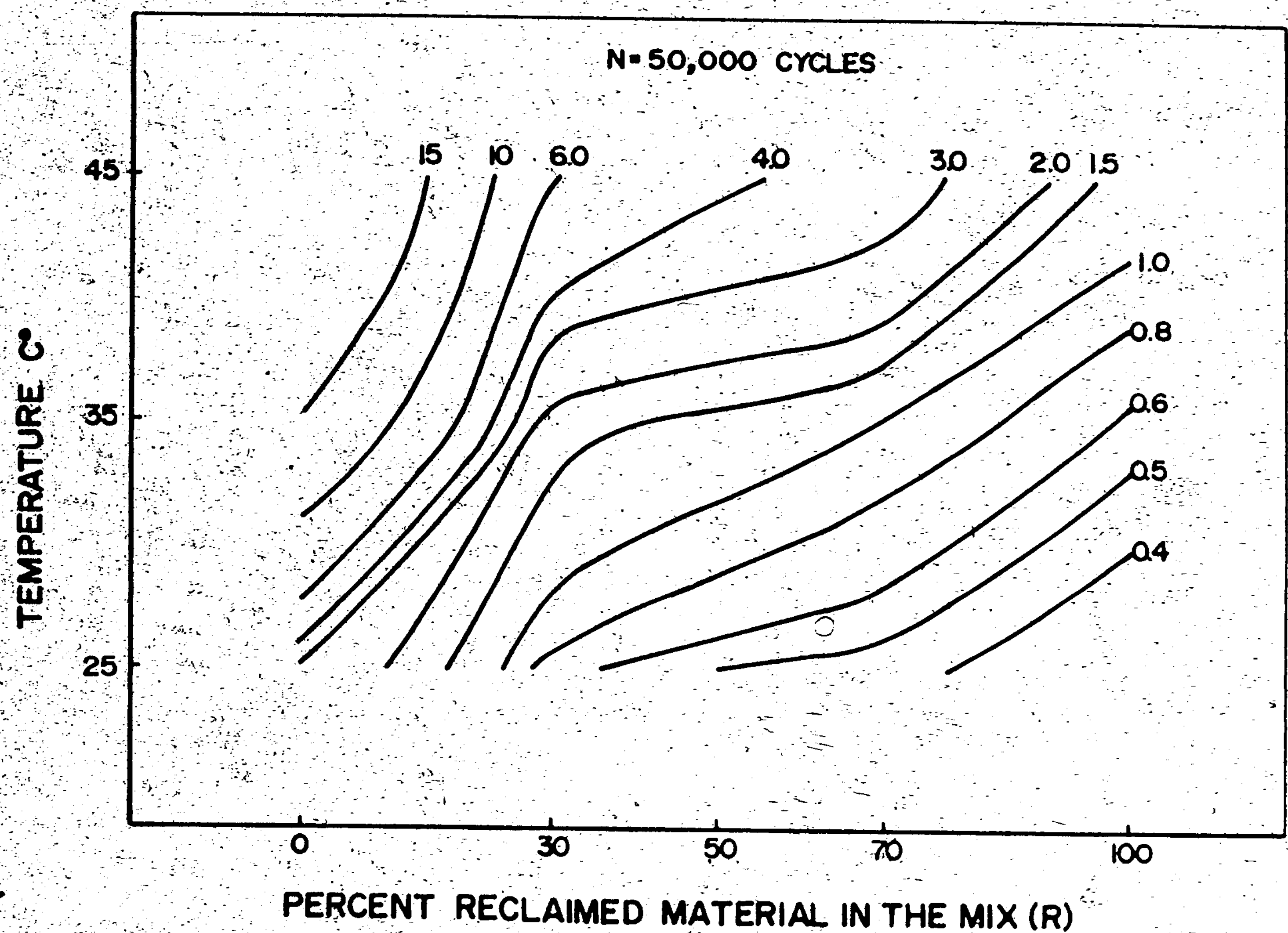


FIGURE 7.24 CONTOUR LINES FOR PERCENT PERMANENT STRAIN AT N = 50,000.

complexity of asphalt concrete mixtures and the vast number of variables involved, it was decided to use mixes having some similar characteristics. To achieve this, aggregate properties for various mixes were kept as close as possible. However the binder content and consistency i.e. penetration and viscosities were difficult to keep constant for different mixes due to the effect of the insertion of various percentages of reclaimed materials. The percent of reclaimed material in the mix was then chosen to reflect the effect of binder content and consistency and to represent the characteristics of various mixes. Table 3.7 in Section 3.5 represents the various mixture characteristics.

Several models were examined in order to establish the best model to predict the permanent deformation of various asphalt concrete mixtures. Models taking into account the three independent variables, individually and in combinations, were studied. In each case several transformations including the logarithm of dependent and/or independent variables and their inverses were used. Models in which logarithmic transformation was used proved to be the most satisfactory. Polynomials of up to degree four were also used to check for any non-linearity. Consequently a logarithmic quadratic model was found to be the most appropriate for predicting permanent deformation of various conventional and recycled mixes under different temperature and loading conditions. The general form of this model is as follows:

$$Y = b_o + b_1 X_1 + b_2 X_2 + b_3 X_3 + b_4 X_1 X_2 + b_5 X_1 X_3 + b_6 X_2 X_3 + b_7 X_1^2 + b_8 X_2^2 + b_9 X_3^2 \tag{7.17}$$

where

Y = log [average percent permanent strain (ϵ_p)],

X_1 = log [number of load repetition (N)],

X_2 = log [percent reclaimed material in the mix (R)],

X_3 = log [temperature °C (T)],

b_o = intercept, and

b_1 to b_9 = regression coefficients.

It should be noted that the values of R and T cannot be equal to zero because of logarithmic transformation. A value of one is suggested instead.

Based on a multiple regression analysis of the data, the above model was determined to be:

$$Y = 4.0968 + 0.3169X_1 + 0.3311X_2 - 9.3684X_3 - 0.0695X_1X_2 + 0.3647X_1X_3 - 0.0616X_1^2 - 0.2628X_2^2 + 3.5558X_3^2 \tag{7.18}$$

The coefficient of determination (r^2) for the above equation was computed to be 97.7 percent which indicates a very high degree of correlation for asphalt concrete mixtures which generally exhibit large variability. All the variables in the model were highly significant (probability < 0.001), which means that the regression coefficients were significantly different from zero. The F-value for the equation was also highly significant (probability < 0.001). Therefore, the above model can be considered as highly satisfactory.

For simplicity and ease of use, a linear logarithmic model was also developed as an approximation to the actual model. This model also predicts the average percent permanent strain under the influence of the number of load repetitions, the percent reclaimed material in the mix, and the temperature. This model can be expressed as follows:

$$Y = b_o + b_1x_1 + b_2x_2 + b_3x_3 \qquad (7.19)$$

where

Y = log average percent permanent strain (ε_p),

X_1 = log number of load repetition (N),

X_2 = log percent reclaimed material in the mix (R),

X_3 = log temperature °C (T),

b_o = intercept, and

b_1 to b_3 = regression coefficients.

Based on a multiple regression analysis of the data, this model was determined to be:

$$Y = -5.2823 + 0.4352X_1 - 0.3827X_2 + 2.6416X_3 \qquad (7.20)$$

This model has an r^2 value of 92.4 percent which is 5.3 percent less than that of the previous model, but it still shows a high degree of correlation. It is also a linear equation and much simpler to use. All the parameters in the model showed a very high degree of significance (probability < 0.001). The F-value for the equation was also highly significant (probability < 0.001). Thus, the model is very satisfactory. It's simplicity and absence of quadratic and interactive terms makes it more practical and easy to use.

This model predicts an increase in average percent permanent strain as the number of load repetitions increases, as the percent reclaimed material in the mix decreases, and as the temperature increases.

7.8.2 Resilient Modulus Test Results and Analysis

Resilient modulus (M_R) is a response of the material tested under dynamic condition. It is determined from the results of repeated loading triaxial test as the ratio of repeated axial deviator stress to recoverable axial strain.

The resilient modulus of all specimens having different recycling ratios was computed, at each load application, at three test temperatures under specified loading conditions. It was found that the M_R determined at each load repetition was almost constant with very little fluctuation throughout each individual test.

The results of each four specimens pertaining to a particular mix group tested at similar loading and environmental conditions were averaged. Table 7.6 summarizes the average and standard deviation of the measured resilient modulus at 10,000 load repetitions at the test temperatures of 25°C, 35°C, and 45°C.

Resilient modulus values ranged from 2.0×10^6 to 0.23×10^6 kPa for temperatures of 25°C and 45°C respectively for various mixes. Variation in M_R versus R at the specified three temperatures is shown in Figure 7.25. An increase in the percentage of the reclaimed material in the mix has caused an increase in the resilient modulus. The effect of R on M_R is rather more pronounced at values of R greater than 50 percent. A similar pattern can be observed for all three test temperatures. However, the resilient modulus exhibits lower values at higher temperatures.

Figure 7.26 shows the changes in resilient modulus with temperature. A descending trend can be noticed as the temperature rises. The reduction in resilient modulus as a result of the increase in temperature from 25°C to 35°C is quite noticeable for 100 percent and 70 percent recycled mixes. This is less apparent for mixes with lower percentages of recycling.

The 50 percent recycled mixes with SC-3000 as the virgin binder appeared to have lower resilient modulus than those with the same recycling percentage but having a 200-300A penetration grade asphalt as the virgin binder. Their M_R values were more similar to that of 30 percent recycled mixes

TABLE 7.6

RESILIENT MODULUS AT
THREE DIFFERENT TEMPERATURES FOR VARIOUS MIXES

R	Resilient Modulus (M_R) x 10^6, kPa at Temperatures of:		
%	25 C	35 C	45 C
0	0.51 (0.10)	0.33 (0.02)	0.23 (0.03)
30	0.68 (0.02)	0.46 (0.08)	0.30 (0.02)
50	0.96 (0.15)	0.59 (0.04)	0.42 (0.03)
70	1.47 (0.08)	0.71 (0.14)	0.63 (0.55)
100	2.04 (1.04)	1.03 (0.16)	0.81 (0.27)
50 SC3000	0.64 (0.10)	0.45 (0.09)	0.35 (0.06)

Figures in brackets indicate one standard deviation.

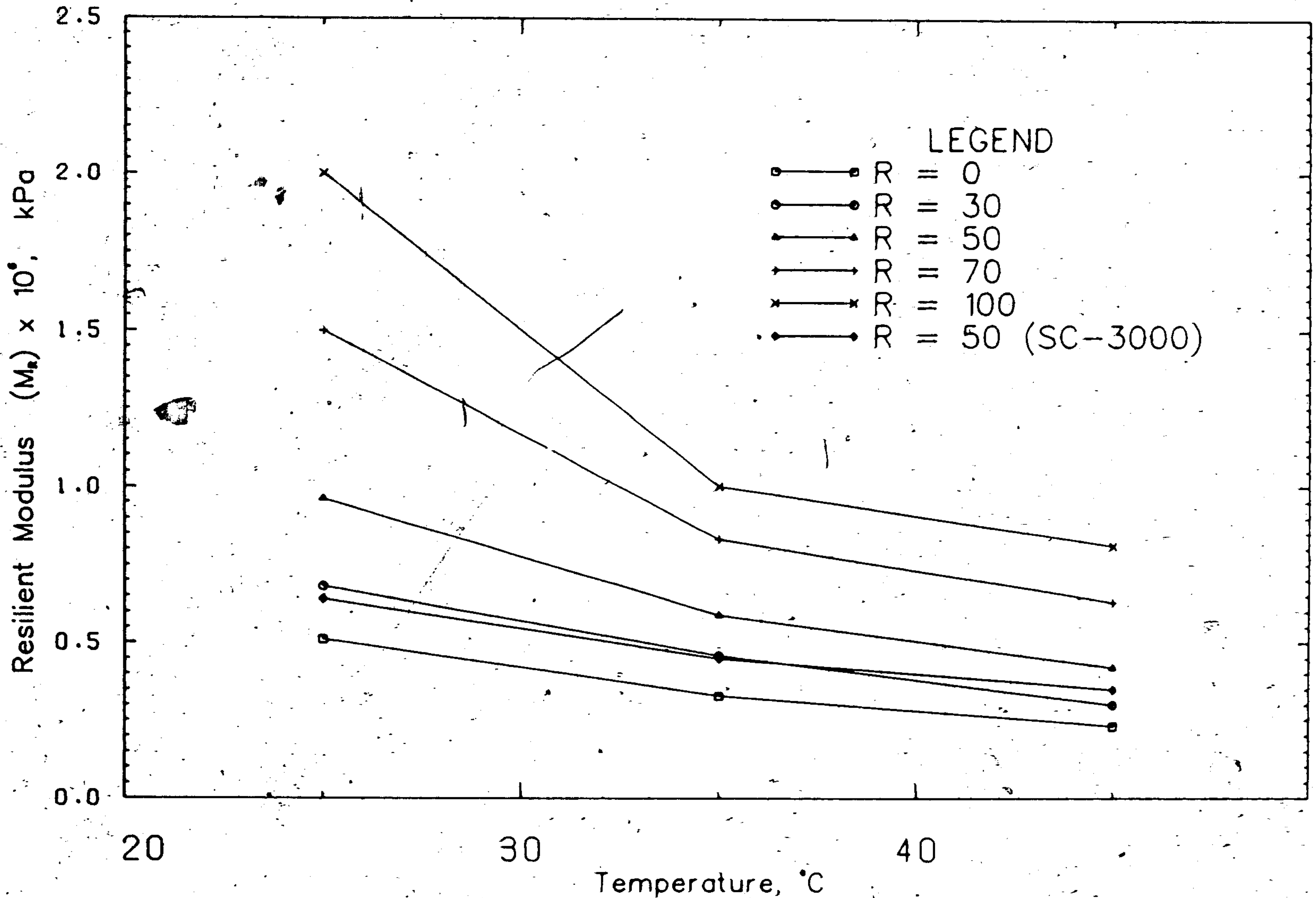


FIGURE 7.25 RELATIONSHIP BETWEEN RESILIENT MODULUS AND TEMPERATURE FOR VARIOUS MIXES.

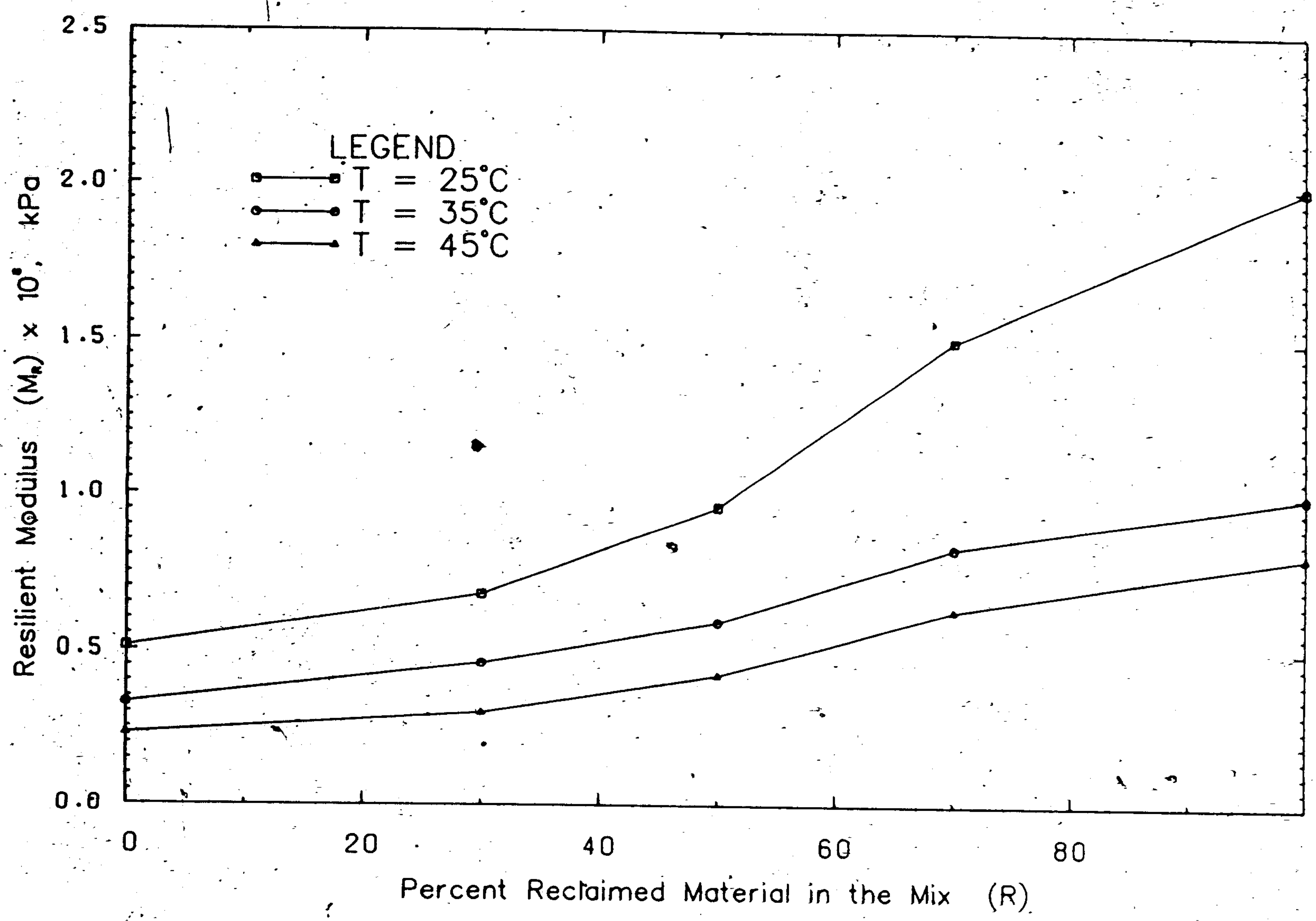


FIGURE 7.26 RELATIONSHIP BETWEEN RESILIENT MODULUS AND PERCENT RECLAIMED MATERIAL IN THE MIX.

with a 200-300A penetration grade asphalt as the virgin binder.

The joint effect of R and T on M_R is illustrated in the form of contour lines in Figure 7.27. It can be seen that an increase in the percent reclaimed material in the mix, in combination with a decrease in temperature, could raise the value of resilient modulus significantly.

7.8.2.1 Development of Model to Predict Resilient Modulus

A simple but accurate model was required to predict resilient modulus values of various mixes under different environmental conditions.

The important independent variables included in this study were percent reclaimed material in the mix (R) and temperature (T). Several models incorporating the independent variables R and T, individually and in combination were examined. Various transformations including logarithmic, inverse, square, cubic and quartic were conducted. To check for any non-linearity, polynomials of up to four degrees were also applied. As a result the following model was found to be more appropriate for predicting resilient modulus of various mixes at different temperatures:

$$Y = b_o + b_1x_1 + b_2x_2 \tag{7.21}$$

where

Y = log [average resilient modulus (M_R)],

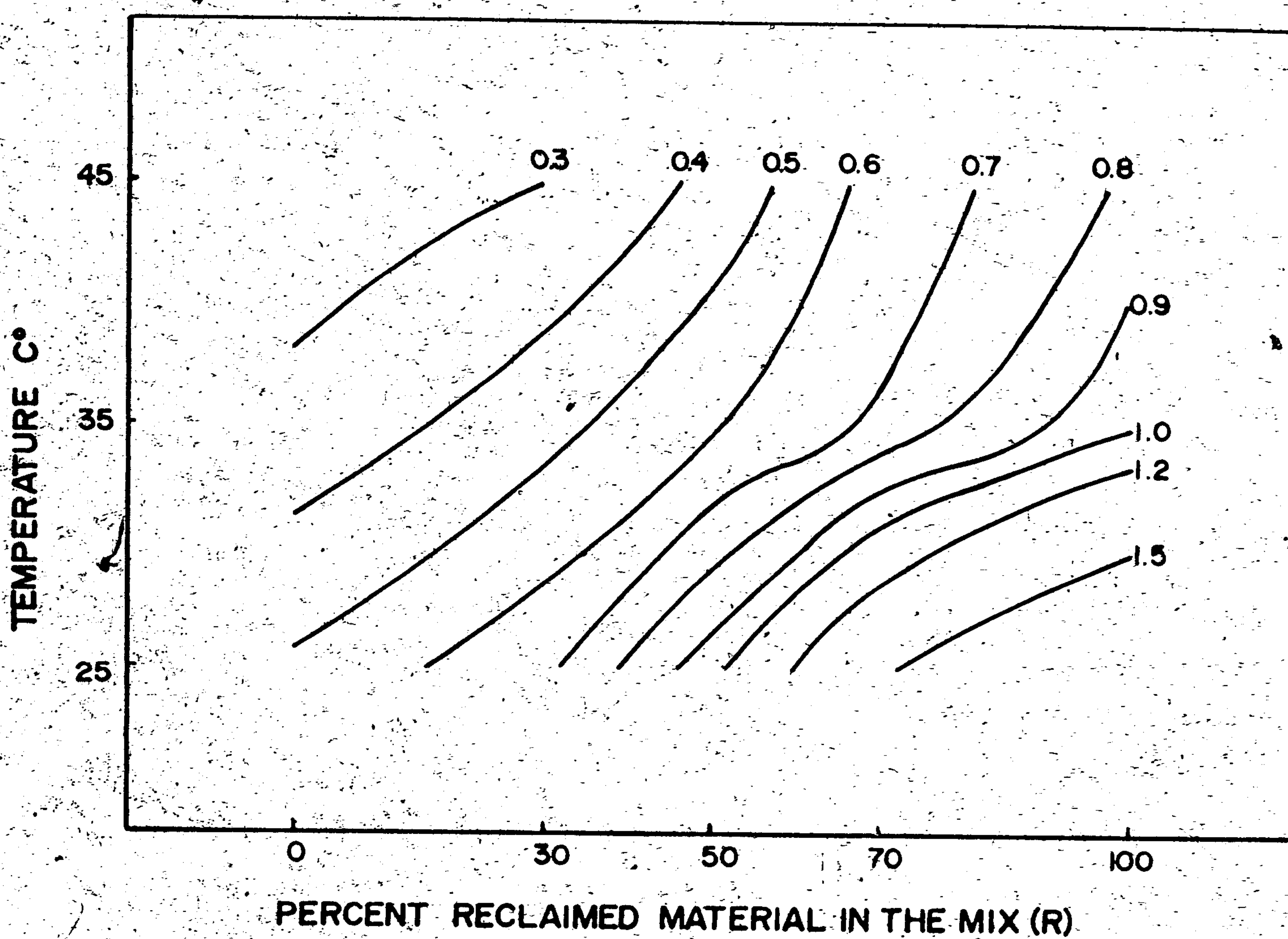


FIGURE 7.27 CONTOUR LINES FOR RESILIENT MODULUS (M_R) x 10^6, kPa.

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X_1 = percent reclaimed material in the mix (R),

X_2 = temperature °C (T),

b_o = intercept, and

b_1 and b_2 = regression coefficients.

Based on a multiple regression analysis of the data, the above model was determined to be:

$$Y = 0.1564 + 0.0056X_1 - 0.0183X_2 \tag{7.22}$$

The coefficient of determination (r^2) for this model was 97.0 percent. This shows a high degree of correlation. All the variables in the model were highly significant (probability < 0.001) and the F-value for the equation showed a very high degree of significance (probability < 0.001). Hence, the above model can be considered as highly satisfactory.

This model predicts a rise in the average value of resilient modulus as a result of an increase in percent reclaimed material in the mix and a reduction of temperature.

7.9. Summary

The main objectives of this chapter were to evaluate the permanent deformation and resilient modulus of conventional and recycled asphalt concrete mixes.

The meaning, mechanism and associated problems of permanent deformation were discussed. A review of current design and technology together with several prediction models were examined.

The design approach used in this study was based on the evaluation of the permanent deformation characteristics of materials using laboratory tests conducted under simulated field conditions. This approach attempts to develop constitutive relationships from the actual data without presuming any deformation law.

A repeated loading triaxial test was chosen for determining the permanent strain and resilient modulus of various mixes because of its ability to duplicate the field conditions. The triaxial equipment was designed to accommodate the testing of fabricated specimens at various test temperatures and stress levels. A dynamic data acquisition system was also developed for use in conjunction with the triaxial test apparatus.

The percent permanent strain and resilient modulus were determined at each load application for mixes having various recycling ratios at different test temperatures.

The triaxial test results showed that for variables considered in this analysis, the value of permanent deformation increases as:

1. number of load repetition increases,
2. percent reclaimed material in the mix decreases, and
3. temperature increases.

The relationship between percent permanent strain and the number of load repetitions has demonstrated some degree of linearity after 10,000 load cycles. A pair-wise comparison between mixes at 10,000 load repetitions has shown that the effect of recycling at 30, 50 and 70 percent levels on permanent deformation were not statistically significant at a 5 percent probability level for any of the selected temperatures.

Models were developed for predicting permanent deformation of conventional and recycled mixes under different temperatures and loading conditions.

Using the results from the triaxial test, the resilient modulus of various mixes was also determined. A model was developed for predicting the resilient modulus of various mixes under different loading and environmental conditions. This model predicts a rise in the value of the resilient modulus as a result of increasing the percent recycling and reducing the temperature. The permanent deformation and resilient modulus models were both shown to be statistically very satisfactory.

8. SUMMARY AND CONCLUSIONS

This study was primarily conduoted to evaluate the detailed physical properties of recycled asphalt concrete mixtures, in order to predict their capabilities as pavement materials. The study also included the development of a comprehensive data bank system for the recycled asphalt concrete pavements.

In general, the findings of this and other investigations indicate that properly designed recycled mixtures are expected to perform in a manner which is superior to conventional mixtures. Asphalt concrete pavement recycling has also proven to be a viable method of rehabilitation or reconstruction of flexible pavements.

The reliability of the asphalt concrete recycling has resulted in an increasing interest in this paving technology. With the growing number of recycling projects and consequently, the vast amount of data, there has emerged the need for a recycling data bank system. To fulfill one of the objectives of this study, a computerized data bank system was developed for the recycling projects. The aim of this system was to objectively recognize and categorize the present and past information on material characteristics, design, construction, maintenance and performance of the recycled pavements on a continuus basis. This will assist in the long term study of the performance of recycled pavements, which, in turn, can be used for improving implementation techniques, design and construction procedures, cost allocations and investment decisions. The data bank system developed in this study has been utilized and has proven to be successful.

In conjunction with the development of the recycling data bank system, the properties of the recycled asphalt concrete materials had also to be characterized. To determine the detailed physical properties of the recycled materials and to compare these properties with those of the conventional materials, a testing program was designed and a series of tests were conducted on various binders and mixtures.

To evaluate the properties of the reclaimed asphalt cement, extraction and recovery of the asphalt from the reclaimed asphalt concrete has been carried out. The recovered asphalt cement was then blended with the selected virgin asphalt at various ratios. These blends were then tested for penetration at 25°C, absolute viscosity at 60°C, and kinematic viscosity at 135°C before and after the Thin-Film Oven test.

Durability characteristics including the retained penetration and viscosity ratios were determined for each blend. The findings of this test are:

(i) The retained penetration increases as the percentage of the recycled asphalt cement in the blend increases, and

(ii) the viscosity ratio at 60°C decreases with an increasing percentage of the recycled asphalt cement in the blend.

Thus, recycled asphalt cements exhibit higher retained penetration and lower viscosity ratios than virgin asphalts. This is an indication of higher durability.

Stiffness values of the blends, before and after the Thin-Film Oven test, were also estimated for a range of temperatures varying from -30°C to +30°C at a constant loading time of 25,000 seconds. After calculating the Pen-Vis Number, McLeod's modified version of the Van der Poel's nomograph was used for the stiffness calculations. The results of these calculations demonstrated that:

(i) The stiffness increases with an increase in the percentage of the recycled asphalt cement in the blend. There is a linear relationship between these two variables, and

(ii) the stiffness decreases as the temperature increases. The decline in stiffness is approximately one order of magnitude for every 10°C increase in temperature.

It can, therefore, be concluded that the recycled asphalt cements exhibit higher stiffness values than virgin asphalts and the values of the stiffness are highly affected by temperature.

To evaluate the physical properties of the conventional and recycled asphalt concrete mixtures, several Marshall mix designs were conducted. Specimens were fabricated at various recycling ratios and were tested at different temperatures for stability, durability, low temperature tensile properties, permanent deformation and resilient modulus. The following summarizes the principal findings and the conclusions drawn from the literature reviews and test results.

Stability, which has been determined from the Marshall mix design, exhibited higher values for the recycled mixtures than for conventional materials.

Durability is a major factor in determining the service life of a pavement and is defined as the long-term resistance to the effects of aging. The retained stability as a measure of durability was determined for conventional and recycled mixtures. It was found that as the percentage of the reclaimed material in the mix increases, higher durability are expected.

Low temperature asphalt concrete pavement cracking is a very serious and costly pavement distress~~ mechanism~~. It is primarily caused by low winter temperatures that induce tensile stresses exceeding the tensile strength of asphalt concrete pavements. The presence of thermal cracks in the pavement can result in a loss of pavement performance, a loss of service life, and an increase in maintenance costs. The low temperature tensile properties of conventional and several recycled asphalt concrete mixtures were evaluated in this study. The tensile splitting test was used to determine the tensile properties of the mixtures having different recycling ratios. Specimens were tested at three different temperatures. The stress-strain characteristics, tensile failure stress, tensile failure strain, and the stiffness modulus at failure were determined for various mixtures. Based on the findings of the test results the following conclusions are drawn:

(i) As the percentage of the reclaimed material in the mixture increases, the tensile failure strain decreases and the stiffness modulus at failure increases. The value of the tensile failure stress fluctuates with an increase in the percentage of the reclaimed material in the mix. Nevertheless, recycled mixtures exhibit higher tensile failure stress than conventional mixtures, and

(ii) a decrease in temperature can result in an increase in tensile failure stress, a decrease in tensile failure strain, and an increase in the stiffness.

In general, recycled asphalt concrete mixtures exhibit higher tensile failure stress, lower tensile strain, and higher stiffness than conventional mixtures. This may indicate a lower resistance to low temperature cracking. However, high cracking potential may be minimized by not using greater than 50 percent reclaimed material in the mix. Models were developed that could simply predict the low temperature properties of the conventional and recycled asphalt concrete mixtures as a function of recycling ratios and temperatures. Using these models the tensile failure stress, tensile failure strain, and the stiffness at failure of conventional and recycled mixtures with similar characteristics can be estimated at different temperatures.

Permanent deformation is one of the major modes of pavement distress. It results in a loss of pavement serviceability and safety. Permanent deformation is referred to as the longitudinal depressions that form in the pavement due to the progressive accumulation of plastic strain under each load application. The permanent deformation characteristics of the conventional and several recycled mixtures were evaluated in this investigation. The design approach used in this study was based on the evaluation of the materials using laboratory tests conducted under simulated field conditions. The repeated loading triaxial test was used for determining the permanent deformation of the conventional and recycled mixtures. This particular test was used because of its ability to simulate field conditions. The percent permanent strain was determined at each load application for mixtures having various recycling ratios at three different temperatures. The principal findings of this study are as follows:

(i) The permanent deformation increases as the number of load repetitions increases. This rate of increase is rather high up to approximately 10,000 load repetitions when it then decelerates and the relationships between the permanent deformation and the number of load repetitions become relatively linear. This phenomenon is more obvious for recycled mixtures then conventional materials,

(ii) the permanent deformation of the asphalt concrete pavement is very sensitive to temperature. Permanent deformation increases as the temperature rises, and

(iii) the recycled mixtures exhibit lower permanent deformation than conventional mixtures. An increase in the percentage of the reclaimed materials in the mix results in a lower value of permanent deformation. As an example, at 10,000 load repetitions and a test temperature of 35°C, the percent permanent strain dropped from 4.4 percent for conventional mixtures to 0.4 percent for mixtures containing 30 percent reclaimed material, i.e. a 91 percent drop in permanent strain. Therefore, only a small quantity of the reclaimed material in the mix can improve the resistance of the pavement to permanent deformation remarkably.

Models were developed for predicting the permanent deformation of conventional and recycled mixtures with an incorporation of the number of load repeititions, the recycling ratios, and the temperature. These models can be employed in predicting the permanent deformation of any mixture with similar characteristics to those tested in this study, under different temperatures and loading repetitions.

Resilient modulus of various mixtures was also determined using the results of the repeated loading triaxial test. Based on the findings of this study the following conclusions are drawn:

(i) The resilient modulus decreases as the temperature increases, and

(ii) an increase in the percentage of the reclaimed material in the mixture results in an increase in the value of the resilient modulus. The effect of the recycling ratio on the resilient modulus is more pronounced for mixtures containing greater than 50 percent reclaimed materials.

A model was developed for predicting the resilient modulus of conventional and recycled mixtures as a function of the number of load repetitions, the recycling ratios, and the temperature. This model predicts a rise in the value of the resilient modulus as a result of an increase in the percent recycling and a reduction in the temperature.

To summarize succintly the findings of this entire investigation, a recapitulation of the conclusions is as follows:

1. Recycling of asphalt concrete pavement is recognized as a viable alternative for pavement

rehabilitation,

2. The recycling data bank system is a valuable tool to study the long term behaviour of the recycled asphalt concrete pavements, and can be used for improving the future design,
3. Based on this study, the reclaimed asphalt concrete materials can be considered as reliable paving materials,
4. The recycled asphalt cements possess higher durability and stiffness than the virgin asphalts,
5. The stability of the recycled asphalt mixtures is greater than that for the conventional mixtures,
6. The recycled asphalt concrete mixtures exhibit higher durability than the conventional mixtures,
7. Lower tensile failure strain, slightly higher tensile failure stress, and greater stiffness modulus at failure is expected for recycled asphalt concrete mixtures as opposed to the conventional mixtures,
8. The recycled asphalt concrete mixtures are slightly more susceptible to low temperature cracking. However, the high cracking potential can be minimized by ensuring that the reclaimed material content of the mix does not exceed 50 percent,
9. The recycled asphalt concrete mixtures display superior qualities than conventional mixtures with regard to permanent deformation,

10. The resilient modulus values of the recycled mixtures are greater than those of the conventional mixtures,

11. The conventional asphalt concrete mixtures are generally more sensitive to temperature than recycled mixtures, and

12. The models developed for predicting the physical properties of the conventional and recycled asphalt concrete mixtures are shown to be statistically highly satisfactory.

9. RECOMMENDATIONS

The following recommendations are made based on the findings of this study:

1. The data obtained in this investigation should be used as indicative of the range of the physical properties that may be expected for recycled asphalt concrete mixtures,
2. The models developed in this study could be applied to estimate the low temperature tensile properties, permanent deformation and resilient modulus of asphalt concrete mixtures when actual testing is not justified,
3. The recycled mixtures exhibited superior qualities with regard to permanent deformation and slightly inferior properties with respect to low temperature cracking. Therefore, in designing the asphalt concrete mixtures more emphasis must be placed on optimizing the tensile properties of the mixtures, to ensure against low temperature cracking.
4. It is recommended that the percentage of the reclaimed material in the mix does not exceed 50 percent. At and below this level, the high cracking potential is expected to be minimized.
5. To improve resistance to permanent deformation an insertion of even a small quantity of approximately 25 percent reclaimed asphalt concrete material in the

mix is recommended,

6. In order to study the long term performance of the recycled asphalt concrete pavements, the recycling data bank system should be updated on a continuous basis.

The recommendations for future research are summarized as follows:

1. More investigation should be conducted to examine a wide range of mix variables,
2. Further studies are recommended to determine the effect of variation in the applied axial stress and the confining pressure on the value of permanent deformation,
3. Efforts should be directed towards developing a simple routine laboratory procedure for measuring the tensile properties and permanent deformation of asphalt concrete mixtures,
4. The long term performance of the recycled asphalt concrete pavements should be evaluated. This information is required to formulate pavement design strategies for recycled materials,
5. The interrelationships between the three major modes of pavement distress, i.e. low temperature cracking, permanent deformation, and fatigue, should be evaluated for recycled asphalt concrete pavements. Consequently, a general design system should be developed.

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APPENDIX A

RECYCLED ASPHALT CONCRETE PAVEMENT

DATA BANK SYSTEM

A1. Detailed Input Information for the Recycled Asphalt Concrete Pavement (RACP) Data Bank

1. PROJECT DESCRIPTION

1.1 Title:

1.2 Location:

From: km
To: km

1.3 Contract Quantities:

Reclaimed material, tonnes

Recycled A.C.P., tonnes

Conventional A.C.P., tonnes

1.4 Rehabilitation Method:

1.5 Agency:

Region:
District:

1.6 Contract No.:

1.7 Contractor:

Main contractor:
Subcontractor:

1.8 Date:

Contract Awarded: D M Y
Project Completed: D M Y

2. EXISTING PAVEMENT PRIOR TO RECYCLING

2.1 Environmental Condition:

General climatic zone

Closest weather station

Annual precipitation

Freezing index

Maximum frost penetration

Mean annual air temperature, °C

Maximum annual air temperature, °C

Minimum annual air temperature, °C

2.2 Soils:

Type of investigation

Unified soil classification

Liquid limit

Plasticity index

Field moisture content %

Field density, kg/m^3

Estimated optimum moisture content %

Estimated maximum density, kg/m^3

Potential frost action (Neg, Low, Med, High)

2.3 Traffic Condition:

Current AADT	Y
Ave. daily ESAL	Y
Cumulative ESAL	Y
Percentage of trucks	Y
Age, year	

2.4 Geometric Design:

Type of roadway

Number of lanes (two-ways)

Lane width, m

Paved shoulder width, m

2.5 Structural Design:

	Thickness mm	Material Classification	Date of Construction
Seal coat			
Overlay			
Seal coat			
Overlay			
Surface course			
Stabilized course			
Base course			

2.6 Mixture Design:

Marshall Stability Test (MST) No.

No. of blows

Asphalt content by weight of dry aggregate %

Stability, N

Flow, mm

Density, kg/m^3

Air voids %

V.M.A. %

Voids filled with asphalt %

Retained stability (after 24 hrs soak) %

2.7 Material Properties as Constructed:

2.7.1 Asphalt properties:

Supplier

Grade

Penetration at 25°C, dmm

Viscosity at 60°C, Pa.s

Viscosity at 135°C, mm^2/s

Specific gravity

PVN

2.7.2 Aggregate Properties:

Aggregate source

Aggregate type

L.A. abrasion

Sand equivalent

Sand to filler ratio

Fractures by weight (2 faces) %

Plasticity index

Bulk specific gravity

Asphalt absorption %

Gradation:

Metric Sieve Size µm	Average Percent Passing
25 000	
20 000	
16 000	
12 500	
10 000	
5 000	
2 500	
2 000	
1 600	
1 250	
800	
630	
400	
315	
160	
80	

3. REHABILITATION CONSIDERATIONS

3.1 Existing Pavement Distress Condition:

Comment: Reference can be given to PMS

Riding Comfort Index (RCI)	Y
Visual Condition Index (VCI)	Y
Benkelman beam deflection:	Y
$\bar{x}$	
$\bar{x} + 2\sigma$	
Temp.	
Corrected fall value:	
$\bar{x}$	
$\bar{x} + 2\sigma$	
Dynaflect deflection:	Y
Structural Adequacy Index (SAI)	Y
Pavement Quality Index (PQI)	Y
Rut depth:	Y
OWP, mm	
IWP, mm	
Skid Number	Y

3.2 Primary Reasons for Rehabilitation:

The following factors contributed more to rehabilitation decision making:

Extensive cracking (type) Y/N

Excessive rut depth Y/N

Low skid number Y/N

Low PQI Y/N

Others

Comments:

3.3 Probable Cause(s) of Distress (Failure):

Structural inadequacy Y/N

Mixture instability Y/N

Poor drainage Y/N

Excessive traffic volume Y/N

Excessive traffic load Y/N

Others

Comments:

3.4 Rehabilitation Technique Chosen:

Removal technique

Reclaimed depth, mm

Reclaimed width, m ______

Recycling technique

Comments:

4. DESIGN OF REHABILITATED PAVEMENT

4.1 Geometric Design:

Type of roadway

Lane width, m

Shoulder width, m

4.2 Structural Design:

	Thickness mm	Material Classification
Surface course		
Intermediate course		
Intermediate course		
Stabilized course		
Base course		

4.3 Mixture Design:

Marshall Stability Test (MST) No.

Number of blows

Design Reclaimed to Virgin material ratio (R/V)%

General R/V ratios considered

Reclaimed aggregate %

Recovered asphalt %

Virgin aggregate %

Blend sand %

Virgin asphalt %

Recycling agent %

Optimum total asphalt content %

Stability, N

Flow, mm

Density, kg/m^3

Air Voids %

V.M.A. %

Void filled with asphalt %

Retained stability (after 24 hrs soak) %

4.3.1 Reclaimed Asphalt Properties:

Penetration at 25°C, dmm

Ductility at °C, cm

Softening point (R & B), °C

Viscosity at 60°C, Pa.s

Viscosity at 135°C, mm^2/s

Flash point, °C

Specific gravity

PVN

Residue after TFOT;

Weight loss %

Penetration at 25°C, dmm

Viscosity at 60°C, Pa.s

Viscosity at 135°C, mm^2/s

Viscosity ratio at 60°C

PVN

4.3.2 Reclaimed Aggregate Properties:

L.A. abrasion

Sand equivalent

Sand to filler ratio

Fracture by weight (2 faces) %

Plasticity index

Liquid limit

Bulk specific gravity

Asphalt absorption %

Gradation:

Metric Sieve Size µm	Average Percent Passing
25 000	
20 000	
16 000	
12 500	
10 000	
5 000	
2 500	
2 000	
1 600	
1 250	
800	
630	
400	
315	
160	
80	

4.3.3 Virgin Asphalt Properties:

Supplier

Grade

Penetration at 25°C, dmm

Ductility at °C, cm

Softening point (R & B), °C

Viscosity at 60°C, Pa.s

Viscosity at 135°C, mm^2/s

Flash point, °C

Specific gravity

PVN

PI

Residue after TFOT

Weight loss %

Penetration, dmm

Viscosity at 60°C, Pa.s

Viscosity at 135°C, mm^2/s

Viscosity ratio at 60°C

PVN

4.3.4 Recycling Agent Properties:

Product name

Supplier

Grade

Viscosity at 60°C, Pa.s

Viscosity at 135°C, mm^2/s

Flash point, °C

Residue after TFOT:

Viscosity at 60°C, Pa.s

Viscosity at 135°C, mm^2/s

Viscosity ratio at 60°C

Others:

4.3.5 Virgin Aggregate Properties:

Aggregate source

Aggregate type

L.A. abrasion

Sand equivalent

Sand to filler ratio

Fractures by weight (2 faces) %

Plasticity index

Liquid limit

Bulk specific gravity

Asphalt absorption %

Gradation:

Metric Sieve Size µm	Average Percent Passing
25 000	
20 000	
16 000	
12 500	
10 000	
5 000	
2 500	
2 000	
1 600	
1 250	
800	
630	
400	
315	
160	
80	

4.3.6 Laboratory Blended Binder Properties:

Design Reclaimed to Virgin asphalt ratio (r/v)

General Reclaimed to Virgin asphalt ratios

Reclaimed asphalt %

Virgin asphalt %

Recycling agent %

Penetration at 25°C, dmm

Ductility at °C, cm

Softening point (R & B), °C

Viscosity at 60°C, Pa.s

Viscosity at 135°C, mm^2/s

Flash point, °C

PVN

PI

Residue after TFOT:

Weight loss %

Penetration at 25°C, dmm

Viscosity at 60°C, Pa.s

Viscosity at 135°C, mm^2/s

Viscosity ratio at 60°C

PVN

4.3.7 Laboratory Blended Aggregate Properties:

Design Reclaimed to Virgin aggregate ratio

General Reclaimed to Virgin aggregate ratios

L.A. abrasion

Sand equivalent

Sand to filler ratio

Fractures by weight (2 faces) %

Plasticity index

Liquid limit

Bulk specific gravity

Asphalt absorption %

Gradation:

Metric Sieve Size µm	Average Percent Passing
25 000	
20 000	
16 000	
12 500	
10 000	
5 000	
2 500	
2 000	
1 600	
1 250	
800	
630	
400	
315	
160	
80	

5. CONSTRUCTION SUMMARY

5.1 Reclaiming Operation:

Reclaiming method

Reclaimed pavement width, m

Reclaimed pavement depth mm

Type of machinery used

Machine mandel width, m

Stockpiling technique

Stockpile height, m Max. Ave.

Percentage of virgin aggregate added

Proportioning technique

Comments:

5.2 Asphalt Plant Operation:

Plant type

Plant configuration

Plant manufacture and classification

Mix type

Average mixing temperature, °C

Total mixing time, sec

Moisture content in the virgin aggregate %

Moisture content in the reclaimed asphalt concrete %

Production rate tonne/hr

Comments:

5.3 Emission Tests:

Pollution control equipment

Plant production rate, tonne/hr at R/V ratio

Percent water added to control emission

Particulates, mg/m^3

Opacity %

Blue smoke intensity (low, average, high, very high)

Comments:

5.4 Spreading and Compaction Operation:

Paver type

Paver screed width, m

Compaction equipment used for:

Breakdown

Intermediate

Finish

Comments:

5.5 Production Summary:

Average reclaimed asphalt concrete per day, tonnes

Total reclaimed asphalt concrete, tonnes

Total reclaimed asphlat concrete used, tonnes

Total virgin aggregate used, tonnes

Total virgin asphalt cement used, tonnes

Total recycling agent used, tonnes

Average recycled asphalt concrete per day, tonnes

Total recycled asphalt concrete, tonnes

Construction duration, days

5.6 Quality Control:

5.6.1 Density Summary:

Date	Formed Specimen				Field Density - Cores					
	Unit No.	Density kg/m^3	M.C. %	Air Voids, %	Unit No.	Station	Density kg/m^3	M.C. %	Air Voids %	Compaction %
Average										
S.D.										
No.										

5.6.2 Binder Content Summary:

Unit No.	Date	Binder Content %					
		Reclaimed Asphalt Concrete			Recycled Asphalt Concrete		
		Centrifuge Ext.	Reflux Ext.	Nuclear	Centrifuge Ext.	Reflux Ext.	Nuclear
Average							
S.D.							
No.							

5.6.3 Binder Properties Summary:

		Virgin Asphalt Cement				Reclaimed Asphalt Cement				Recycled Asphalt Cement			
Unit No.	Date	Pen. at 25°C dmm	Visc. at 60°C Pa.s	Visc. at 135°C mm^2/s	PVN	Pen. at 25°C dmm	Visc. at 60°C Pa.s	Visc. at 135°C mm^2/s	PVN	Pen. at 25°C dmm	Visc. at 60°C Pa.s	Visc. at 135°C mm^2/s	PVN
Average													
S.D.													
No.													

5.6.4 Aggregate Properties Summary:

5.6.4.1 Virgin Aggregate:

Unit No.	Date	Percent Passing 25 000	20 000	18 000	12 500	10 000	5 000	2 500	2 000	1 250	630	400	315	160	80	M.C. %	Fracture %
Average																	
S.D.																	
No.																	

5.6.4.3 Recycled Aggregate:

Unit No.	Date	Percent Passing 25 000	20 000	18 000	12 500	10 000	5 000	2 500	2 000	1 250	630	400	315	160	[illegible]	M.C. %	Fracture %
Average																	
S.D.																	
No.																	

5.6.4.2 Reclaimed Aggregate

Unit No.	Date	Percent Passing															M.C.	Fracture
		25 000	20 000	18 000	12 500	10 000	5 000	2 500	2 000	1 250	630	400	315	160	80			
Average																		
S.D.																		
No.																		

5.6.5 Temperature Summary

Unit No.	Date	Temperature, °C					
		Air	Mix Discharge	Laydown	Breakdown	Intermediate	Final
Average							
S.D.							
No.							

6. COSTS

6.1 Actual Cost of Recycled Project:

Unit prices:

Reclaiming original asphalt concrete, $/tonne

Recycled asphalt concrete, $/tonne

Conventional asphalt concrete, $/tonne

Basic loading factor, $/tonne

Haul, $/tonne

Others:

Material Costs:

Asphalt cement, $/tonne

Recycling agent, $/tonne

Aggregate, $/tonne

Others:

Cost per tonne of recycled asphalt concrete pavement, $

Cost per tonne of recycled asphalt concrete pavement after salvage value deduction, $

Cost per lane-kilometer of recycled asphalt concrete pavement, $

Cost per lane-kilometer of recycled asphalt concrete pavement after salvage value deduction, $

Total cost of recycled asphalt concrete pavement, $

6.2 Estimated Cost of Other Rehabilitation Alternatives:

If all virgin material were used, having the same configuration:

Cost per tonne of conventional A.C.P., $

Cost per lane-kilometer of conventional A.C.P., $

Total cost of conventional A.C.P., $

Others:

6.3 Cost Comparison and Savings:

Comparison between recycled and conventional A.C.P.:

Asphalt saved per tonne of mix, tonnes

Aggregate saved per tonne of mix, tonnes

Total asphalt saved in the project, tonnes

Total aggregate saved in the project, tonnes

Total cost saving per tonne of mix, $

Total cost saving per lane-kilometer, $

Total cost saving in the project by recycling, $

Others:

7. RECYCLED PAVEMENT EVALUATION

7.1 Evaluation of Pavement Surface Condition:

Date of Evaluation: D M Y

Traffic characteristics at the time of evaluation:

AADT Percent trucks Ave. daily ESAL

Surface distress:

Ravelling:

Severity (Nil, slight, moderate, major, severe)

Density (Percent area)

Flushing:

Severity Density (percent area)

Others:

Surface deformation:

Rutting:

Ave. rut depth, mm Density (percent area)

Shoving:

Severity Density (percent area)

Others:

Cracking:

Type Severity Density (percent area)

Type Severity Density (percent area)

Patching:

Quality (Good, Fair, Poor) Density (percent area)

Visual condition index (VCI)

Others:

Comments:

7.2 Evaluation of Pavement Serviceability:

Date of evaluation: D M Y

Traffic characteristics at the time of evaluation:

AADT Percent trucks Ave. daily ESAL

PCA Roadmeter mm/km at the speed of kph

Riding Comfort Index (RCI)

Present Serviceability Index (PSI)

Others:

7.3 Evaluation of Pavement Safety:

Date of evaluation: D M Y

Traffic characteristics at the time of evaluation:

AADT Percent trucks Ave. daily ESAL

Skid number at the speed of kph

Others:

7.4 Evaluation of Pavement Structural Capacity:

Date of evaluation: D M Y

Traffic characteristics at the time of evaluation:

AADT Percent trucks Ave. daily ESAL

Maximum deflection:

Benkelman Beam: Mean S.D.

Dynaflect: Mean S.D.

Others:

7.5 Evaluation of Pavement Material Properties Using Core Data:

Date of evaluation: D M Y

Traffic characteristics at the time of evaluation:

AADT Percent trucks Ave. daily ESAL

Stability, N

Density, kg/m^3

Air voids, %

Physical properties of the extracted asphalt:

Penetration at 25°C, dmm

Ductility at °C, cm

Softening point (R & B), °C

Viscosity at 60°C, Pa.s

Viscosity at 135°C, mm^2/s

Others:

8. DETAILED PHYSICAL PROPERTIES OF RECYCLED MIXTURE

8.1 Stiffness by Van der Poel normograph:

Stiffness, MN/m^2 Loading time, Hz Temp, °C

8.2 Resilient Modulus:

Method used:

Modulus, MN/m^2 Loading time, Hz Temp, °C

8.3 Fatigue Behaviour:

Method used:

Conditions:

(1) Frequency, Hz Temp, °C

$N_f = A\left(\frac{1}{\sigma}\right)^B$ A = B =

$N_f = C\left(\frac{1}{\varepsilon}\right)^D$ C = D =

(2) Frequency, Hz Temp, °C

A = B = C = D =

8.4 Permanent Deformation:

Method used:

Conditions:

(1) Frequency, Hz Temp, °C

$f(N) = A(N)^{-m}$ A = m =

(2) Frequency, Hz Temp, °C

A = m =

8.5 Low Temperature Pavement Cracking:

Binder Properties:

Penetration at 25°C, dmm

Penetration at °C, dmm

Viscosity at 135°C, mm^2/s

Penetration index (PI)

Pen-Vis Number (PVN)

Stiffness, MN/mm^2 Time, sec Temp, °C

Method used:

Recycled mixture:

Stiffness, MN/mm^2 Time, sec Temp, °C

Method used:

8.6 Tensile Properties:

Method used:

Tensile strength, kN/m2 Temp, °C

Tensile strain at failure, mm/mm

Strain rate, mm/min Temp, °C

8.7 Water Susceptibility:

Alberta Immersion Compression Test:

Temp, °C Soaking period, hr.

Stability prior to treatment, N

Stability after treatment, N

Retained stability %

9. MAINTENANCE AND REHABILITATION RECORDS

Date: D M Y

Time after recycling: M Y

Type of maintenance;

Cost per tonne, $

Cost per lane-kilometer, $

Total cost, $

A2. DOCUMENTATION OF THE RECYCLED ASPHALT CONCRETE PAVEMENT DATA BANK

The RECYCLED ASPHALT CONCRETE PAVEMENTS (RACP) database is written in SPIRES (Stanford Public Information REtrieval System) and allows for flexible, quick input and retrieval of data records. The system which is currently in place has been designed for RACP data and has an order of it's own. Basically, there are a number of records, each of which is a summary in and of itself. The larger database has been modularized, with each component bearing the name of the type of data it keeps track of. If you $RUN *SPIRES you will be shown a menu of things to do. By selecting menu choice number 1, you will see a listing of the databases into which we can enter data. Here is what will happen when you $RUN *SPIRES under this account (301Q).'

```
#$run *spires
 #02:01:15
 -Welcome to SPIRES 85.01

(This is a new screen)

*
*   Note: Pressing BREAK/PA1 will return you to this screen
*
*   Time: 12:01:18    Date: May 1, 1985
*
*   Selected Subfile:
*
*
*    Do you wish to :
*
*       1.  Select a SUBFILE?
*       2.  Add a record in a subfile?
*       3.  Change a record in the database?
*       4.  Remove a record from a subfile?
*       5.  Display a record?
*       6.  Use the SCREEN facility to add to "Construction
            Summary"?
*       7.  Return to SPIRES command mode?
*       8.  Return to MTS?
*
: Please type a number 1 - 8 : 1
*
```

'Note that throughout this document, in the listings of terminal sessions, what YOU type is always printed in BOLD. COMMENTS are written in ITALICS, so that they are not confused with part of the terminal session.

(This is a new screen)

```
*
*  Private Subfiles:
*
*     1. PROJECT DESCRIPTION
*     2. EXISTING PAVEMENT PRIOR TO RECYCLING
*     3. REHABILITATION CONSIDERATIONS
*     4. DESIGN OF REHABILITATED PAVEMENT
*     5. CONSTRUCTION SUMMARY
*     6. COSTS
*     7. RECYCLED PAVEMENT EVALUATION
*     8. DETAILED PHYSICAL PROPERTIES OF RECYCLED MIXTURE

*     9. MAINTENANCE AND REHABILITATION RECORDS
*
*
: Please Enter Selection (1-9): 1
```

(This is a new screen)

```
*  Note: Pressing BREAK/PA1 will return you to this screen
*
*  Time: 02:01:30    Date: May 1, 1985
*
*  Selected Subfile: PROJECT DESCRIPTION
*
*
*   Do you wish to :
*
*     1.  Select a SUBFILE?
*     2.  Add a record in a subfile?
*     3.  Change a record in the database?
*     4.  Remove a record from a subfile?
*     5.  Display a record?
*     6.  Use the SCREEN facility to add to "Construction
          Summary"?
*     7.  Return to SPIRES command mode?
*     8.  Return to MTS?
*
: Please type a number 1 - 8 :
```

We can add data into any of these subfiles very simply. Let us say you wished to enter data into the PROJECT DESCRIPTION database. You would now choose menu choice 2, and the prompting would appear. Don't forget that what YOU type is in BOLD PRINT:

: Key or Number of Record 2

Note here that it asks for the NUMBER OF RECORD. This is given to you by whomever wishes to have the data entered...

: Title **HWY 2:18 & :20**

Structure: Location

: From (KM) **23.130 2:18**
: To (KM) **31.189 2:18**

Note that at this point SPIRES keeps repeating this prompt until we issue a NULL LINE (a carriage return, or <RETURN> without typing anything) to indicate that we have no more data for this portion.

: From (KM) **0.000 2:20**
: To (KM) **7.070 2:20**

: From (KM)

At this point we press <RETURN> to indicate that we have no more data and that SPIRES should 'get on with it'..

: Total Length (KM) **30.258**

Structure: Contract Quantities

: Reclaimed Material, tonnes **13340**
: Recycled A.C.P., tonnes **13340**
: Conventional A.C.P., tonnes **59380**

: Rehabilitation Method **Cold-Milling Partial Depth**

Structure: Agency

: Region **Crossfield**
: District **Calgary**

: Contract Number **<enter a null line>>**

Structure: Contractor

: Main Contractor **Peter Kiewit Sons Co.**
: Subcontractor(1) **Budd Bros. Ltd.**
: Subcontractor(2)

Again we press <RETURN> to indicate that we have no more data of this type.

```
Structure: Date

:   Contract Awarded                          /n
```

Note that a "/n" was entered. The slash tells the prompting format that this is a special entry. All special commands begin with a slash. You may use "/?" to see all possible commands. The "/n" was used here to indicate that no value was available for this element, but that we still wanted to be prompted for the other element in the structure. Had we not done this, SPIRES would think that we didn't want to enter anything for the whole structure.

```
:   Project Completed                         Nov 3/82

->
```

This being done, we could ask for a DISPLAY of what was just entered, by choosing menu choice 5:

```
(This is a new screen)

*
*  Note: Pressing BREAK/PA1 will return you to this screen
*
*  Time: 12:01:18    Date: May 1, 1985
*
*  Selected Subfile:
*
*
*    Do you wish to :
*
*     1.  Select a SUBFILE?
*     2.  Add a record in a subfile?
*     3.  Change a record in the database?
*     4.  Remove a record from a subfile?
*     5.  Display a record?
*     6.  Use the SCREEN facility to add to "Construction
          Summary"?
*     7.  Return to SPIRES command mode?
*     8.  Return to MTS?
*
: Please type a number 1 - 8 : 5
```

*
* Which Record? 2
*

Key or Number of Record	2
Title	HWY 2:18 & :20

Structure: Location

From (KM)	23.130 2:18
To (KM)	31.189 2:18
From (KM)	0.0 2:20
To (KM)	7.070 2:20

Total Length (KM)	30.258

Structure: Contract Quantities

Reclaimed Material, tonnes	13340
Recycled A.C.P., tonnes	13340
Conventional A.C.P., tonnes	59380

Rehabilitation Method	COLD-MILLING PARTIAL DEPTH

Structure: Agency

Region	CROSSFIELD
District	CALGARY

Structure: Contractor

Main Contractor	PETER KIEWIT SONS CO.
Subcontractor(1)	BUDD BROS. LTD.

Structure: Date

Project Completed	Nov 3, 1982

Special NOTES for the 5.6 (fullscreen) Entries

If you look at the Data Bank Structure which outlines the various headings for the data destined for the databases, you will notice that it is broken up into sections. For instance, Section 1 is PROJECT DESCRIPTION, Section 2 is EXISTING PAVEMENT PRIOR TO RECYCLING, and so on. An outline of the data necessary for the CONSTRUCTION SUMMARY is given in Section 5. The Quality Control subsection 5.6 consists of several tables which are labelled *Density, Binder Content, Binder Prop, Virgin Agg, Reclaimed Agg, Recycled Agg,* and *Temperature*. These tables consist of numerous lines of data which are all the same, so it was decided that this portion of the CONSTRUCTION SUMMARY could best be entered on a FULL SCREEN on your terminal, which resembles the tables in this section. Thus, when you use the menu choice 6 to enter data into the CONSTRUCTION SUMMARY, the tabular material is to be entered. This is done very easily. You'll enter the FULL SCREEN TABLE ENTRY FACILITY, which in turn prompts you for the name of the table to be entered. It happens like this:

SCREEN NAME? **Density**

When it asks for the SCREEN NAME, you have the choice of any table desired:

Density
Binder Content
Binder Prop
Virgin Agg
Reclaimed Agg
Recycled Agg
Temperature

You'll be put into FULL SCREEN mode. The first screen is a menu screen, of which you'll use the ADD command. It's argument is a Record Key Number. Remember that you are adding additional information to a record in CONSTRUCTION SUMMARY. You are not adding a new record! Therefore, the command will look like this:

ADD key (where "key" is a number) – E.g. **ADD 2**

This will let you add tabular data to the already existing record number 1. There will be an error message if the Key does not already exist.

After entering the ADD command, you enter the data on the screen which will look very similar to the tables you have to enter. You may enter a full screen of information before hitting the **RETURN** key to send it into the database.

To move around the screen, you can use the TAB key to move to a subsequent field, or the cursor movement keys found on all terminals.

It is very important to enter a value for every field on the screen. This does not apply to any bottom portion of the screen which is not going to be filled because of lack of data. This is relevant only where a portion of the data of a complete occurrence of a record is missing. At these positions on the screen there should be a slash "/" entered. This will act as a place holder so that the multiple occurrences stay logically separate.

If all the tabular data cannot fit on one screen, simply fill one screen, press RETURN, and then issue the ADD command again, with the same record key number. You will now be adding tabular data "under" the data previously entered.

Technical Information

The modules of the database are stored separately from the FILEDEF subfile in another subfile called RECDEF. This subfile contains all nine goal records (subfiles). This was done not only to make updating easier, but out of necessity, because the FILEDEF subfile couldn't handle the one large file. Each module's FILE-NAME is RACPn, where "n" is the number of the subfile in the order of the DATA BANK STRUCTURE Document. Then, the whole file definition in FILEDEF is called RACP, and contains all the subfile names, and by which file in RECDEF they are defined, along with the subfile sections which contain SELECT-COMMANDS, format settings, etc. The filedef also contains a couple index records. See the SPIRES manual "FILEDEF" for more info on FILEDEF and RECDEF subfiles, and how to update and compile both.

The menu driver and full-screen driver are in a subfile called protocols, which is the XEQ file set upon entry into SPIRES via the ENTRY COMMANDS subfile, and the MENU record is called up. Thus, the $RUN *SPIRES command is the only command a novice user needs to learn.

Formats exist for the full-screens, as well as for the general prompting for and displaying of records. Other formats for the output of the tabular data of the CONSTRUCTION SUMMARY subfile are also written which reproduce the tables and provide statistical information. The use of these tables is currently not dealt with in the MENU, so access to them is only available through native SPIRES language using the SET FORMAT commands. To moderately experienced SPIRES users, this is a common task.

A3. AN EXAMPLE OF THE RECYCLED ASPHALT CONCRETE PAVEMENT DATA BANK OUTPUT

PROJECT DESCRIPTION

Key or Number of Record	2
Title	HWY 2:18 & :20
Structure: Location	
From (KM)	23.130 , 2:18
To (KM)	31.189 , 2:18
From (KM)	00.000 , 2:20
To (KM)	07.070 , 2:20
Total Length (KM)	30.258
Structure: Contract Quantities	
Reclaimed Material, tonnes	13340
Recycled A.C.P., tonnes	13340
Conventional A.C.P., tonnes	59380
Rehabilitation Method	COLD-MILLING PARTIAL DEPTH, CENTRAL PLANT, HOT MIX RECYCLING
Structure: Agency	
Region	CROSSFIELD
District	CALGARY
Structure: Contractor	
Main Contractor	PETER KIEWIT SONS CO. LTD OF WINTERBURN
Subcontractor(1)	BUDD BROS. LTD OF CALGARY
Structure: Date	
Project Completed	05/03/82

EXISTING PAVEMENT

Key or Number of Record	2
Structure: Traffic Condition	
Structure: 2.3.1	
AADT	16000
Year	1981
Structure: 2.3.4	
Percentage of Trucks	15
Year	1981
Age (Years)	12
Structure: Geometric Design	
Type of Roadway	HIGHWAY
Number of Lanes (two-ways)	4
Lane Width (m)	3.66
Structure: Paved Shoulder Width	
Inside (m)	1.22
Outside (m)	3.05
Structure: Structural Design	
Structure: Overlay	
Thickness (mm)	75
Material Classification	ACP
Date of Construction	1970
Structure: Surface Course	
Thickness (mm)	100
Material Classification	ACP
Date of Construction	1958
Structure: Stabilized Course	
Thickness (mm)	350
Material Classification	S.G.B.C.
Date of Construction	1957
Structure: Mixture Properties As Constructed	
No. of Blows	75
Asphalt Content by Weight (%)	5.7
Stability, N	15000

Flow (mm)	2.92
Density (Kg/Cu M)	2321
Air Voids (%)	3.8
Retained Stability (24hrs.) %	50

Structure: Material Properties as Constructed

Structure: Asphalt Properties

Supplier	HUSKY - MOOSE JAW
Grade	AC 275
PVN	

Structure: Aggregate Properties

Aggregate Source	KIRK PIT
Bulk Specific Gravity	2.576
Asphalt Absorption (%)	0.65

Structure: Gradation

20,000	100
5000	51
1600	39
400	25
80	12.9

DESIGN

Key or Number of Record 2

Structure: Geometric Design

Type of Roadway	HIGHWAY
Lane Width (m)	3.66

Structure: Shoulder Width (m)

Inside	1.22
Outside	3.05

Structure: Structural Design

Structure: Surface Course

Thickness (mm)	60
Material Classification	ACP

Structure: Intermediate Course

Thickness (mm)	50
Material Classification	RACP
Thickness (mm)	125
Material Classification	ACP

Structure: Stabilized Course

Thickness (mm)	350
Material Classification	ACP

Structure: Mixture Design

Number of Blows	75
Design Recl. to Vir. Ratio (R/V)%	50/50

Structure: General R/V Ratios Considered

Ratio	100/00
Reclaimed Aggregate %	93.7
Recovered Asphalt %	6.3
Virgin Aggregate %	0
Blend Sand %	0
Virgin Asphalt %	0
Recycling Agent %	0
Optimum Total Asphalt Content %	6.3
Stability (N)	9140
Flow (mm)	4.1
Density (Kg/Cu. M)	2376
Air Voids %	1.1

Ratio	75/25
Reclaimed Aggregate %	70.3
Recovered Asphalt %	4.7
Virgin Aggregate %	24.6
Blend Sand %	0
Virgin Asphalt %	0.4
Recycling Agent %	0
Optimum Total Asphalt Content %	5.1
Stability (N)	14500
Flow (mm)	3.3
Density (Kg/Cu. M)	2396
Air Voids %	3
V.M.A. %	11.5
Retained Stability (24hrs) %	80

Ratio	50/50
Reclaimed Aggregate %	46.9
Recovered Asphalt %	3.1
Virgin Aggregate %	47.7
Blend Sand %	0
Virgin Asphalt %	2.3
Recycling Agent %	0
Optimum Total Asphalt Content %	5.4
Stability (N)	17900
Flow (mm)	2.9
Density (Kg/Cu. M)	2389
Air Voids %	3.4
V.M.A. %	12
Void Filled With Asphalt %	72
Retained Stability (24hrs) %	79

Ratio	25/75
Reclaimed Aggregate %	23.5
Recovered Asphalt %	1.5
Virgin Aggregate %	71.1
Blend Sand %	0
Virgin Asphalt %	3.9
Recycling Agent %	0
Optimum Total Asphalt Content %	5.4
Stability (N)	16400
Density (Kg/Cu. M)	2391
Air Voids %	3.2
V.M.A. %	12.1
Retained Stability (24hrs) %	80

Ratio	15/85
Reclaimed Aggregate %	14.1
Recovered Asphalt %	0.9
Virgin Aggregate %	80.7
Blend Sand %	0
Virgin Asphalt %	4.3
Recycling Agent %	0
Optimum Total Asphalt Content %	5.2
Stability (N)	15000
Density (Kg/Cu. M)	2392
Air Voids %	3

V.M.A. %.	11.9
Retained Stability (24hrs) %	78

Ratio	00/100
Reclaimed Aggregate %	0
Recovered Asphalt %	0
Virgin Aggregate %	94.6
Blend Sand %	0
Virgin Asphalt %	5.4
Recycling Agent %	0
Optimum Total Asphalt Content %	0
Stability (N)	8725
Flow (mm)	1.9
Density (Kg/Cu. M)	2362
Air Voids %	3.6
V.M.A. %	13
Retained Stability (24hrs) %	70

Structure: Reclaimed Asphalt Properties

Penetration at 25C (dmm)	175
Viscosity at 60C (Pa.s)	57.7

Structure: Residue After TFOT

Penetration at 25C (dmm)	96
Viscosity at 60C (Pa.s)	148.7
Viscosity at 135C (mm2/s)	365
Viscosity Ratio at 60C	0.39
PVN	-0.40892

Structure: Reclaimed Aggregate Properties

Structure: Gradation

16,000	96
10,000	77
5000	53
1250	36
630	31
315	23
160	17.9
80	14.3

Structure: Virgin Asphalt Properties

Supplier	HUSKY
Grade	150-200 A
Penetration at 25C (dmm)	164
Viscosity at 60C (Pa.s)	85.5
Viscosity at 135C (mm2/s)	295
Specific Gravity	1.03
PVN	-0.09199

Structure: Residue After TFOT

Penetration at 25C (dmm)	86
Viscosity at 60C (Pa.s)	241.2
Viscosity Ratio at 60C	0.35

Structure: Virgin Aggregate Properties

Aggregate Source	CARSTAIRS CREEK
Sand Equivalent	3.0
Sand to Filler Ratio	3
Fractures by Weight (2 Faces) %	82
Bulk Specific Gravity	2.576
Asphalt Absorption (%)	1.47

Structure: Gradation

16,000	100
10,000	83
5000	59
1250	34
630	24
315	13
160	8.6
80	6.4

Structure: Lab Blended Binder Properties

Des. Recl. to Vir. Asphl Ratio (r/v)	58/42

Structure: General r/v Ratios Considered

Ratio	00/100
Reclaimed Asphalt %	0
Virgin Asphalt %	100
Recycling Agent %	0
Penetration at 25C (dmm)	164
Viscosity at 60C (Pa.S)	85.8
Viscosity at 135C (mm2/S)	295

Structure: Residue after TFOT

Penetration at 25C (dmm)	86
Viscosity at 60C (Pa.s)	241.2

Ratio	15/85
Reclaimed Asphalt %	15
Virgin Asphalt %	85
Recycling Agent %	0
Penetration at 25C (dmm)	166
Viscosity at 60C (Pa.S)	92.6
Viscosity at 135C (mm2/S)	291

Structure: Residue after TFOT

Penetration at 25C (dmm)	92
Viscosity at 60C (Pa.s)	193.2

Viscosity at 135C (mm2/s)	399

Ratio	25/75
Reclaimed Asphalt %	25
Virgin Asphalt %	75
Recycling Agent %	0
Penetration at 25C (dmm)	161
Viscosity at 60C (Pa.S)	82.4
Viscosity at 135C (mm2/S)	277

Structure: Residue after TFOT

Penetration at 25C (dmm)	96
Viscosity at 60C (Pa.s)	148.4
Viscosity at 135C (mm2/s)	334

Ratio	50/50
Reclaimed Asphalt %	50
Virgin Asphalt %	50
Recycling Agent %	0
Penetration at 25C (dmm)	165
Viscosity at 60C (Pa.S)	80.6
Viscosity at 135C (mm2/S)	267

Structure: Residue after TFOT

Penetration at 25C (dmm)	95
Viscosity at 60C (Pa.s)	184.5
Viscosity at 135C (mm2/s)	391

Ratio	75/25
Reclaimed Asphalt %	75
Virgin Asphalt %	25
Recycling Agent %	0
Penetration at 25C (dmm)	168
Viscosity at 60C (Pa.S)	93.5
Viscosity at 135C (mm2/S)	294

Structure: Residue after TFOT

Penetration at 25C (dmm)	96
Viscosity at 60C (Pa.s)	159.7
Viscosity at 135C (mm2/s)	384

Ratio	100/00
Reclaimed Asphalt %	100
Virgin Asphalt %	0
Recycling Agent %	0
Penetration at 25C (dmm)	175
Viscosity at 60C (Pa.S)	57.7

Structure: Residue after TFOT

Penetration at 25C (dmm)	96
Viscosity at 60C (Pa.s)	148.7
Viscosity at 135C (mm2/s)	365

Structure: Lab Blended Aggregate Properties

Design Recl. to Virgin Agg. Ratio 50/50

Structure: General R/V Ratios Considered

Ratio 100/00

Structure: Gradation

16,000	96
10,000	77
5000	53
1250	36
630	31
315	23
160	17.9
80	14.3

Ratio 75/25

Structure: Gradation

16,000	98
10,000	79
5000	55
1250	26
630	29
315	21
160	16.3
80	12.7

Ratio	50/50
Sand to Filler Ratio	3
Fractures by Weight (2 Faces %)	82
Bulk Specific Gravity	2.580
Asphalt Absorption (%)	1.47

Structure: Gradation

16,000	99
10,000	82
5000	57
1250	35
630	28
315	21
160	16
80	14.2

CONSTRUCTION SUMMARY

Key or Number of Record 2

Structure: Reclaiming Operation

Reclaiming Method	COLD-MILLING
Reclaimed Pavement Width (m)	3.66
Reclaimed Pavement Depth (mm)	50
Type of Machinery Used	CMI PR-450 & 575 ROTO-MILL
Machine Mandel Width (m)	1.83 & 2.74
Stockpiling Technique	TRUCK UNLOADING

Structure: Stockpile Height (m)

Maximum 3

% of Virgin Agg. Added at Stockpile	0
Proportioning Technique	BELT SCALE

Structure: Asphalt Plant Operation

Plant Type	CENTRAL PLANT
Plant Configuration	DRUM MIXER
Plant Manufacture & Classification	BOEING 600
Mix Type	HOT MIX
Average Mixing Temperature (C)	148
Total Mixing Time (sec)	180
Moisture Content in Virgin Agg. (%)	4.1
Moisture Cont. in Recl Asph Conc (%)	2
Production Rate (tonne/hour)	280

Structure: Emission Tests

Structure: Plant Production Rate

tonne/hr 280
at R/V Ratio 50/50

% Water Added to Control Emission	1.7-2.5
Blue Smoke Intensity (L,A,H,V)	VERY HIGH

Structure: Spreading & Compaction Operation

Paver Type BLAW-KNOX PF-180H

Structure: Compaction Equipment Used For

Breakdown	DYNAPAC DUAL DRUM VIBRATORY ROLLER CC50
Intermediate	PNEUMATIC TIRE ROLLER
Finish	DYNAPAC DUAL DRUM VIBRATORY ROLLER CC42

Structure: Production Summary

Ave Reclaimed Asph Conc/Day (tonnes)	1180
Total Reclaimed Asph. Conc. (tonnes)	24700
Total Recl. Asph Conc. Used (tonnes)	9000
Total Recycling Agent Used (tonnes)	0
Ave Recycled Asph. Conc/Day (tonnes)	860
Total Recycled Asph. Conc. (tonnes)	18000
Construction Duration (Days)	21

COSTS

Key or Number of Record 2

Structure: Actual Cost of Recycled Project

Structure: Unit Prices

Reclaiming Asphalt Conc. ($/tonne)	$3.00
Recycled Asphalt Conc. ($/tonne)	$9.65
Conventional Asphalt Conc. ($/tonne)	$9.34
Basic loading factor ($/tonne)	$0.73
Hauling ($/tonne)	$0.11

Structure: Material Costs

Asphalt Cement ($/tonne)	$246.59
Aggregate ($/tonne)	$1.69

Cost/tonne RACP ($)	$24.49
Cost/tonne RACP (Less Sal. Val.) ($)	$16.50
Total Cost of RACP	$443020.00

Structure: Estimated Cost of Other Alternatives

Cost/tonne of Conv. A.C.P. ($)	$31.76
Total Cost of Conv. A.C.P. ($)	$574540.00

Structure: Cost Comparison and Savings

Total Asphalt Saved (tonnes)	520
Total Aggregate Saved (tonnes)	8582
Total Cost Saving by Recycling ($)	$131520.00

Density Summary

Date	Formed Specimen Unit No	Den. Kg/M3	MC %	Air Voids	Field Density -- Cores Unit No	Station	Den Kg/M3	MC %	Air Voids	Comp %
10/02/82	1	2333		4.3	1		2235		8.5	95.8
10/02/82					1		2026		17.2	86.8
10/02/82	2	2348		2.8	2		2296		4.5	97.5
10/02/82	3	2302		4.5	3		2153		11	93.5
10/02/82					3		2124		12.3	92.3
10/04/82	4	2354		2.4	4		2100		12.6	89.2
10/04/82					4		2095		13.2	89
10/04/82	5	2330		5.1	5		2247		8.3	96.4
10/04/82					5		2197		9.5	94.3
10/05/82	6	2361		3	6		2143		11.7	90.8
10/05/82					6		2155		11.7	91.3
10/05/82	7	2345		3.2	7		2241		7.4	95.6
10/05/82					7		2181		9.8	93
10/06/82	9	2368		2.2	9		2278		4.3	96.2
10/06/82					9		2045		15.1	86.4
10/07/82	12	2370		3.1	12		2189		10.6	92.4
10/07/82					12		2211		9.8	93.3
10/08/82	13	2364		3.7	13		2227		9	94.6
10/08/82					13		2199		10.1	93.4
10/13/82	19	2382		2.8	19		2223		9	93.6
10/13/82					19		2135		12.7	89.6
10/14/82	22	2347		3.9	22		2072		15.8	88.3
10/15/82	25	2346		4.1	25		2070		14.9	88.2
10/15/82					25		2104		13.7	89.7
10/19/82	28	2378		3.2	28		2212		9.8	93
10/19/82					28		2155		12	90.6
10/19/82					28		2145		12.4	90.2
10/19/82					28		2162		11.8	90.9
10/20/82	30	2383		2.4	30		2309		5.4	96.9
10/20/82					30		2335		4.4	98
10/21/82	34	2375		3	34		2301		6.8	96.9
10/21/82					34		2318		6.2	97.3
10/22/82	37	2380		3.5	37		2302		6.8	96.7
10/22/82					37		2362		4.6	99
10/23/82	39	2393		3.1	39		2182		10.4	91.2
10/23/82					39		2279		6.3	95.2
10/23/82					39		2300		5.5	96.1
10/25/82	43	2383		4.2	43		2163		11.3	90.8
10/25/82					43		2267		11.5	95.1
10/25/82					43		2284		10.2	95.8
10/25/82					43		2165		11.5	90.8
10/27/82	48	2381		5.8	48		2298		9.8	96.6
10/27/82					48		2242		8.2	94.2
10/27/82					48		2128		12.7	89.4
10/28/82	49	2386		2.5	49		2322		4.6	97.3
10/28/82					49		2178		10.6	91.3
10/28/82					49		2189		10.2	91.7
10/29/82	52	2380		2.3	52		2288		6.4	96.1
10/29/82					52		2280		6	95.7
10/29/82					52		2323		4.4	92.6
11/03/82	57	2379		3.2	57		2214		9.3	93.1
11/03/82					57		2210		9.5	92.9
11/03/82					57		2249		7.6	94.5
11/03/82	58	2388		2.9	58		2153		11.7	90.2
11/03/82					58		2150		11.9	90
11/03/82					58		2171		11	90
11/03/82	59	2380		2.9	59		2180		11.9	91.6
11/03/82					59		2167		12.5	91.1
11/03/82					59		2179		11.9	91.6
Number		25	0	25		0	59	0	59	59
Average		2365	0	3.36		0	2205	0	9.82	92.9
St. Dev		10.5	0	0.89		0	74.7	0	3.10	3.02

Binder Content Summary

Binder Content %

Unit No.	Date	Reclaimed Asphalt Concrete: Cent. Ext.	Reclaimed Asphalt Concrete: Reflux Ext.	Reclaimed Asphalt Concrete: Nuclear	Recycled Asphalt Concrete: Cent. Ext.	Recycled Asphalt Concrete: Reflux Ext.	Recycled Asphalt Concrete: Nuclear
1	10/02/82					7	
2	10/02/82	6.1					
3	10/02/82						
4	10/04/82	6.1				5.6	
5	10/04/82	5.4					
6	10/05/82						
7	10/05/82	5.8				6	
9	10/06/82	6.3					
11	10/06/82	5.8					
12	10/07/82	6.3					
13	10/08/82						
19	10/13/82						
22	10/14/82						
25	10/15/82						
28	10/19/82					6	
30	10/20/82						
34	10/21/82						
37	10/22/82						
39	10/23/82	5.9					
43	10/25/82					6.4	
49	10/28/82	5.2					
51	10/28/82	5.7					
52	10/29/82	5.8					
Number		11	0	0	0	5	0
Average		5.8	0	0	0	6.2	0
St. Dev.		0.3	0	0	0	0.5	0

Binder Properties

Unit No.	Date	Virgin Asphalt Cement: Pen 25	Virgin Asphalt Cement: Visc 60	Virgin Asphalt Cement: Visc 135	Virgin Asphalt Cement: PVN	Reclaimed Asphalt Cement: Pen 25	Reclaimed Asphalt Cement: Visc 60	Reclaimed Asphalt Cement: Visc 135	Reclaimed Asphalt Cement: PVN	Recycled Asphalt Cement: Pen 25	Recycled Asphalt Cement: Visc 60	Recycled Asphalt Cement: Visc 135	Recycled Asphalt Cement: PVN
0	12/-/82	155	89										
7						131	124	317		140	101	303	
10													
11						218.	89	325					
13													
16													
19													
22													
25						108	177	394					
28										177	74	303	
30										140	101	315	
37						230	51	245		90	193	420	
39						143	90	323					
43										147	93	316	
Number		1	1	0	0	5	5	5	0	5	5	5	0
Average		155	89	0		166	106.	320		138	112.	331	
St. Dev.		0	0	0		54.	47.2	52.		31.	46.3	49.	

Temperature Summary
Temperature in °C

Unit	Date	Air	Mix Dischge	Lay Down	Break Down	Inter Mediate	Final
1	10/02/84	11	144	132	120		
2	10/02/82	11	139	129	120		
3	10/02/82	13	139	130	118		
4	10/04/82	8	142	120	108		
5	10/04/82	9	162	128	109		
6	10/05/82	10	140	130	110		
7	10/05/82	10	142	135	120		
9	10/06/82	10	146	130	116		
11	10/07/82	9	132	125	120		
12	10/07/82	8	146	125	118		
13	10/08/82	10	146	136	131		
19	10/13/82	18	146	136	130		
22	10/14/82	17	138	128	126		
25	10/15/82	13	140	120	110		
28	10/19/82	4	134	115	112		
30	10/20/82	3	150	120	108		
34	10/21/82	6	132	130	128		
37	10/22/82	5	130	120	115		
39	10/23/82	6	152	148	142		
43	10/25/82	7	135				
48	10/27/82	4	154	148	140		
49	10/28/82	3	155	148	142		
51	10/29/82	1	150	136	132		
52	10/29/82	8	146	140	138		
53	10/30/82	8	142	138	134		
54	10/30/82	9	142	136	132		
55	11/02/82	1	152	150	144		
56	11/02/82	1	142	140	138		
57	11/03/82	0	150	146	144		
58	11/03/82	1	150	142	140		
59	11/03/82	0	152	142	142		
Number		31	31	30	30	0	0
Average		7.2	144	133	126	0	0
St. Dev.		4.7	7.5	9.6	12	0	0

VIRGIN AGGREGATE
Percent Passing

Un.	Date	25000	20000	16000	12500	10000	5000	2500	2000	1250	630	400	315	160	80	MC %	Ft %
1	10/02/82			100		87	63			41			21	15	11 9	4 8	
4	10/04/82			100		88	63			40			22	16	12 2	4 7	
5	10/04/82			100		89	66			42			22	16	12 6	5 9	
7	10/05/82			100		88	62			38			19	13	8 3	3 7	
11	10/07/82			100		86	65			42			19	14	11 3	6	
13	10/08/82			100		89	66			41			19	13	10	5 9	
19	10/13/82			99		80	57			34			17	12	10 1	3 2	
28	10/19/82			99		86	57			36			20	15	11 1	2 8	
30	10/20/82			100		81	52			31			17	13	9 6	3 3	
37	10/22/82			98		82	56			32			17	13	9 9	2 8	
43	10/25/82			100		91	65			41			24	17	13 7	4 7	
49	10/28/82			99		85	61			36			19	14	10 6	3 3	
52	10/29/82			99		83	58			34			19	13	9 6	3 4	
54	10/30/82			100		82	54			33			17	13	10	3 6	
56	11/02/82			99		81	57			35			17	12	9 8	2 5	
57	11/03/82			98		78	53			31			17	13	9 4	4 2	
Number		0	0	16	0	16	16	0	0	16	0	0	16	16	16	16	0
Average		0	0	99	0	84	59	0	0	36	0	0	19	13	10	4 0	0
St. Dev.		0	0	0.5	0	3.8	4	0	0	4	0	0	2	1	1 4	1 1	0

RECLAIMED AGGREGATE
Percent Passing

Un	Date	25000	20000	16000	12500	10000	5000	2500	2000	1250	630	400	315	160	80	MC %	Ft %
4	10/04/82			99		85	63			39			25	20	14.9	1.4	
7	10/05/82			99		90	69			45			32	26	21.	2	
11	10/07/82			100		93	71			44			26	19	15	2.2	
13	10/08/82			99		88	62			40			25	19	13.9	2.1	
22	10/14/82			99		86	61			38			27	22	17.7	1.9	
25	10/15/82			100		88	63			34			21	18	15.9	1.5	
28	10/19/82			98		86	63			42			28	22	17.6		
30	10/20/82			99		85	64			40			24	17	12.2	3	
37	10/22/82			98		90	66			42			24	18	14.2		
39	10/23/82			98		85	64			41			24	18	13.6		
43	10/25/82			98		90	69			46			26	19	15.4	2.1	
49	10/28/82			100		93	75			50			30	22	16		
Number		0	0	12	0	12	12	0	0	12	0	0	12	12	12	8	0
Average		0	0	98	0	88	65	0	0	41	0	0	26	20	15	2.0	0
St. Dev.		0	0	0.7	0	2.9	4.	0	0	4.	0	0	2.	2.	2.3	0.4	0

RECYCLED AGGREGATE
Percent Passing

Un	Date	25000	20000	16000	12500	10000	5000	2500	2000	1250	630	400	315	160	80	MC %	Ft %
1	10/02/82			100		87	62			38			25	20	15.8		
2	10/04/82			99		85	61			40			24	18	14.1		
4	10/04/82			99		89	64			40			24	18	13.4		
5	10/04/82			99		87	62			38			23	17	12.3		
6	10/05/82			99		89	68			43			23	17	13.3		
7	10/05/82			98		83	58			37			22	17	13.2		
9	10/06/82			100		88	65			42			25	18	13.1		
11	10/07/82			98		90	68			42			23	17	13		
12	10/07/82			100		88	65			41			23	17	13.1		
13	10/08/82			99		86	65			41			24	19	13.9		
19	10/13/82			98		86	64			40			25	20	15.8		
22	10/14/82			100		88	63			39			24	19	14.4		
25	10/15/82			99		88	64			39			24	19	15.1		
28	10/19/82			100		86	64			41			25	19	14.3		
30	10/20/82			99		92	73			48			31	24	18.8		
34	10/21/82			98		85	58			37			23	17	13.2		
37	10/22/82			99		85	60			39			23	17	13.3		
39	10/23/82			99		88	66			43			30	23	14.4		
43	10/25/82			99		87	63			39			23	17	12.9		
48	10/27/82			99		88	64			40			22	16	12.3		
49	10/28/82			98		81	56			36			21	15	11.2		
51	10/29/82			98		83	62			38			23	18	14.1		
53	10/30/82			99		88	66			42			25	18	13		
54	10/30/82			98		83	62			40			23	17	12.1		
57	11/03/82			95		81	60			38			24	19	14.5		
Number		0	0	25	0	25	25	0	0	25	0	0	25	25	25	0	0
Average		0	0	98	0	86	63	0	0	40	0	0	24	18	13	0	0
St. Dev.		0	0	0.6	0	2.6	3	0	0	2.	0	0	2.	1.	1.5	0	0

APPENDIX B

MARSHALL MIXTURE DESIGN SUMMARY

APPENDIX B

MARSHALL MIXTURE DESIGN SUMMARY

B.1 Introduction

This appendix contains details of the Marshall mix design test results provided by Alberta Transportation for the mixtures used in this investigation. Several mixtures having various "reclaimed to virgin material ratios" (R/V) of 0/100, 30/70, 50/50, 70/30 and 100/0 were designed.

A 300-400A penetration grade virgin asphalt cement was used for all the mixtures with the exception of the conventional mixture, for which a 200-300A virgin asphalt was used.

A mix design was also performed for a mixture having an R/V ratio of 50/50 but with SC-3000 as a recycling modifier.

The complete mix design summaries containing the Marshall stability results, design curves, and the aggregate gradation charts, for all six different mixtures are presented in this appendix.

TABLE B.1

MARSHALL MIXTURE DESIGN SUMMARY
R/V = 0/100

DESIGN DATA R/V = 0/100

AGGREGATE			ASPHALT	
DES. 1 CLASS 16	% FRACTURES (2 FACES)		TYPE & GRADE	200-300A
BULK SPECIFIC GRAVITY	2.625	TOP SIZE TO NEXT SIEVE 63 %	SUPPLIER / LOCATION	Imperial Edmonton
ASPHALT ABSORPTION	0.73	ALL + 5000 µm 68 %		
PLASTICITY INDEX	Trace(low)	SAND / FILLER RATIO 6.4		
		AUTO COMPACTION 75 blows/face	SPECIFIC GRAVITY	1.025

RESULTS FROM CURVES DRAWN

VIRGIN ASPHALT CONTENT	%						
TOTAL ASPHALT CONTENT	%	5.2	5.8	6.4	7.0		
DENSITY	kg/m³	2324	2353	2369	2366		
STABILITY	N	6250	7000	7700	8200		
AIR VOIDS	%	6.2	4.3	2.9	2.2		
VOIDS IN MINERAL AGGREGATE	%	15.8	15.3	15.2	15.8		
FLOW	mm	1.8	2.0	2.3	3.0		

DESIGN RECOMMENDATIONS

GRADATION – PERCENT PASSING

SIEVE ANALYSIS µm (CGSB-8-GP-2M)	MINIMUM	AVERAGE	MAXIMUM
16 000		100	
12 500		94	
10 000	77	83	89
5 000	53	59	65
1 250	31	35	39
630	28	31	34
315	24	27	30
160	14.1	16.1	18.1
80	6.0	8.0	10.0

VIRGIN ASPHALT CONTENT	%	
TOTAL ASPHALT CONTENT	%	5.9
DENSITY	kg/m³	2357
STABILITY	N	7150
AIR VOIDS	%	4.0
VOIDS IN MINERAL AGGREGATE	%	15.2
FLOW	mm	2.0
RETAINED STABILITY (24 h SOAK)	%	90

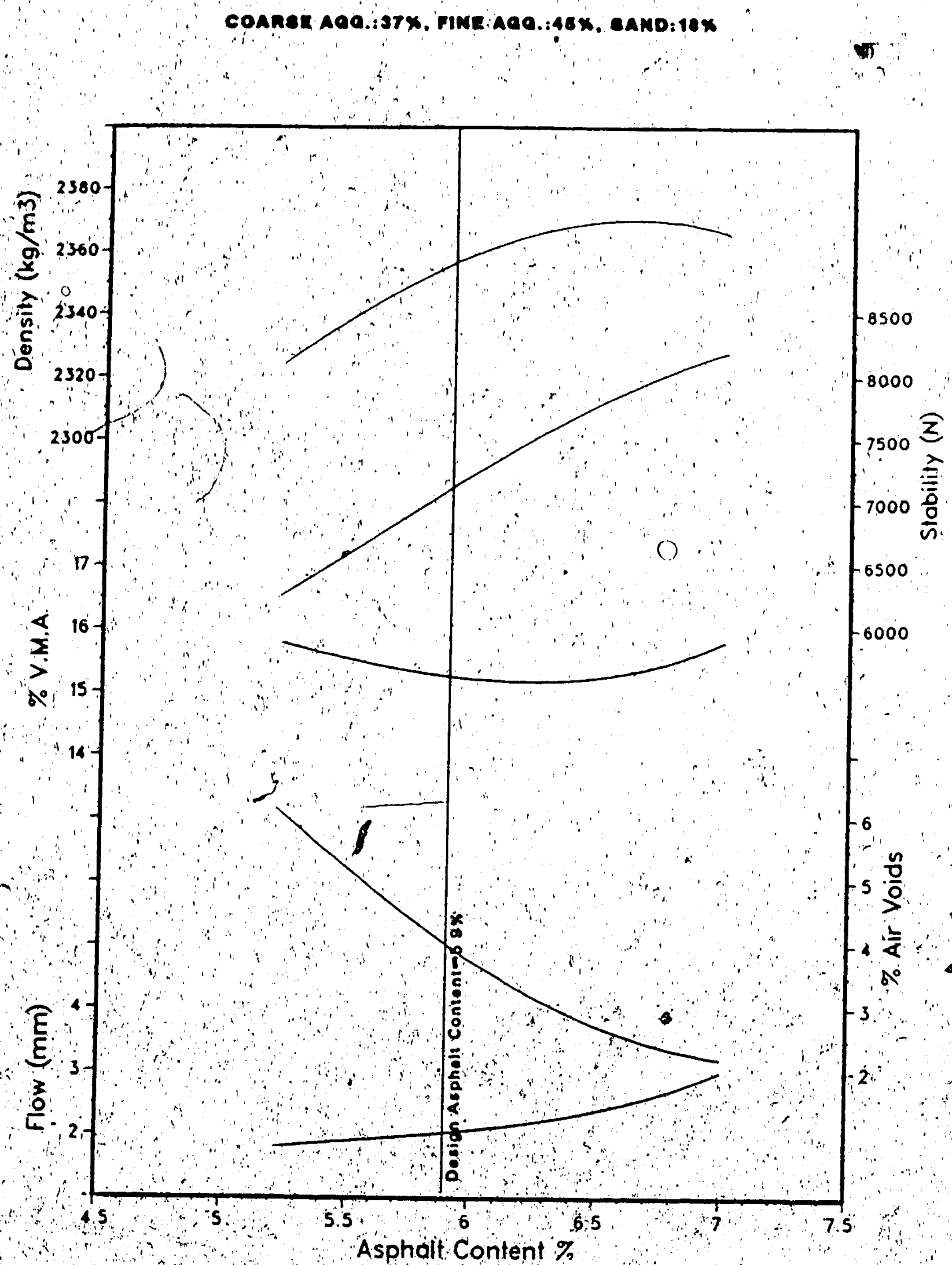

FIGURE B.1 MARSHALL MIXTURE DESIGN CURVES, R/V = 0/100.

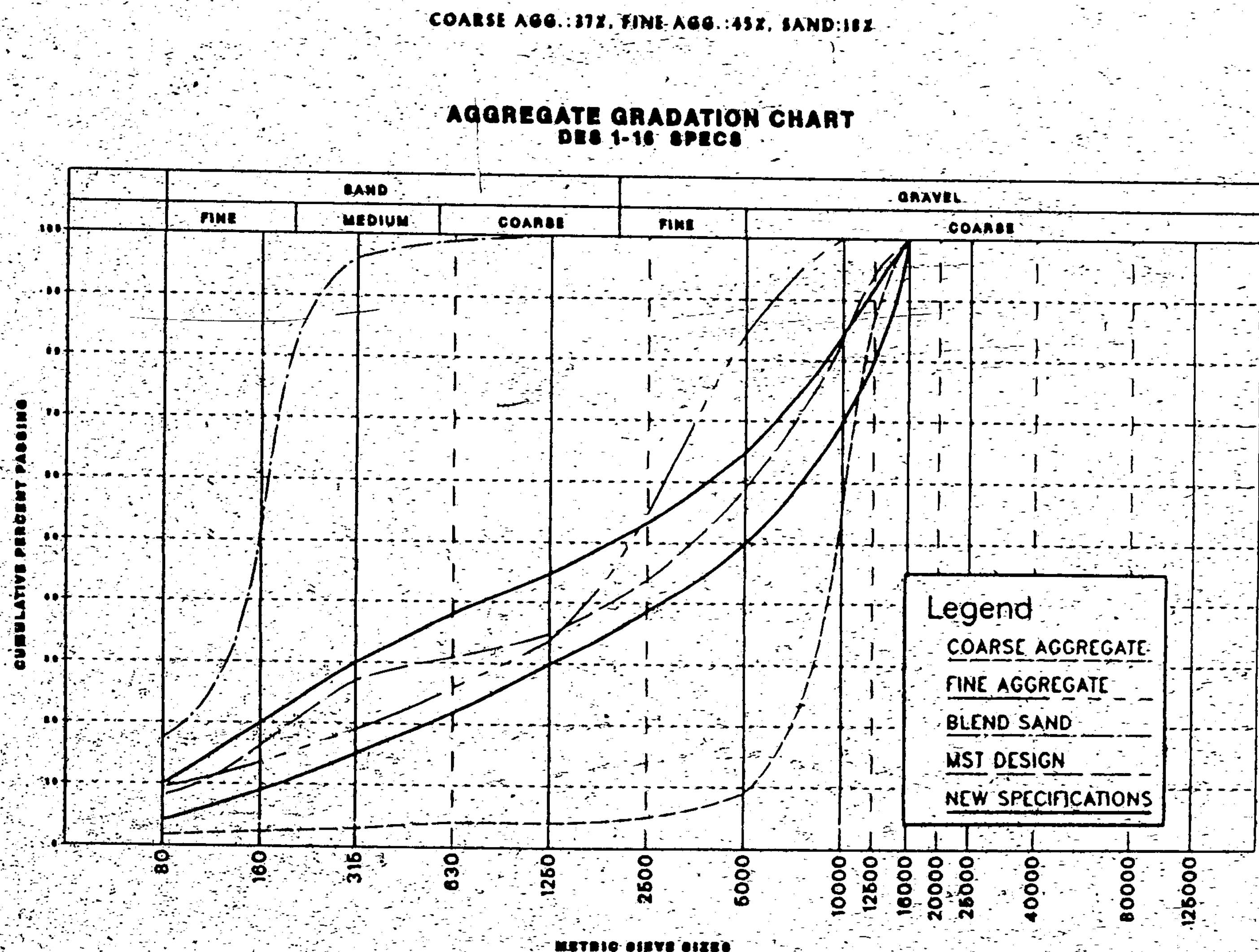

FIGURE B.2 AGGREGATE GRADATION CURVES, R/V = 0/100.

TABLE B.2

MARSHALL MIXTURE DESIGN SUMMARY
R/V = 30/70

DESIGN DATA R/V = 30/70

AGGREGATE		ASPHALT
DES. 1 CLASS 16	% FRACTURES (2 FACES)	TYPE & GRADE 300-400A
BULK SPECIFIC GRAVITY 2.632	TOP SIZE TO NEXT SIEVE ___ %	SUPPLIER/LOCATION Husky Lloydminster
ASPHALT ABSORPTION 0.18	ALL + 5000 µm ___ %	
PLASTICITY INDEX ___	SAND/FILLER RATIO 5.1	
	AUTO COMPACTION 75 blows/face	SPECIFIC GRAVITY 1.021

RESULTS FROM CURVES DRAWN

VIRGIN ASPHALT CONTENT	%	2.1	2.7	3.3	3.9		
TOTAL ASPHALT CONTENT	%	3.7	4.3	4.9	5.5		
DENSITY	kg/m³	2355	2371	2382	2382		
STABILITY	N	15 450	15 100	13 500	10 550		
AIR VOIDS	%	5.9	4.5	3.2	2.5		
VOIDS IN MINERAL AGGREGATE	%	13.7	13.6	13.7	14.2		
FLOW	mm	2.7	2.8	2.9	3.2		

DESIGN RECOMMENDATIONS

SIEVE ANALYSIS µm (CGSB-8-GP-2M)	GRADATION – PERCENT PASSING MINIMUM	AVERAGE	MAXIMUM			
16 000		100		VIRGIN ASPHALT CONTENT	%	2.9
12 500		94		TOTAL ASPHALT CONTENT	%	4.5
10 000	78	84	90	DENSITY	kg/m³	2375
5 000	53	59	65	STABILITY	N	14 750
1 250	29	33	37	AIR VOIDS	%	4.0
630	24	27	30	VOIDS IN MINERAL AGGREGATE	%	13.7
315	18	21	24	FLOW	mm	2.8
160	12.5	14.5	16.5			
80	7.6	9.6	11.6	RETAINED STABILITY (24 h SOAK)	%	86

RAP.:30%, VIRGIN AGG.:65%, SAND:5%

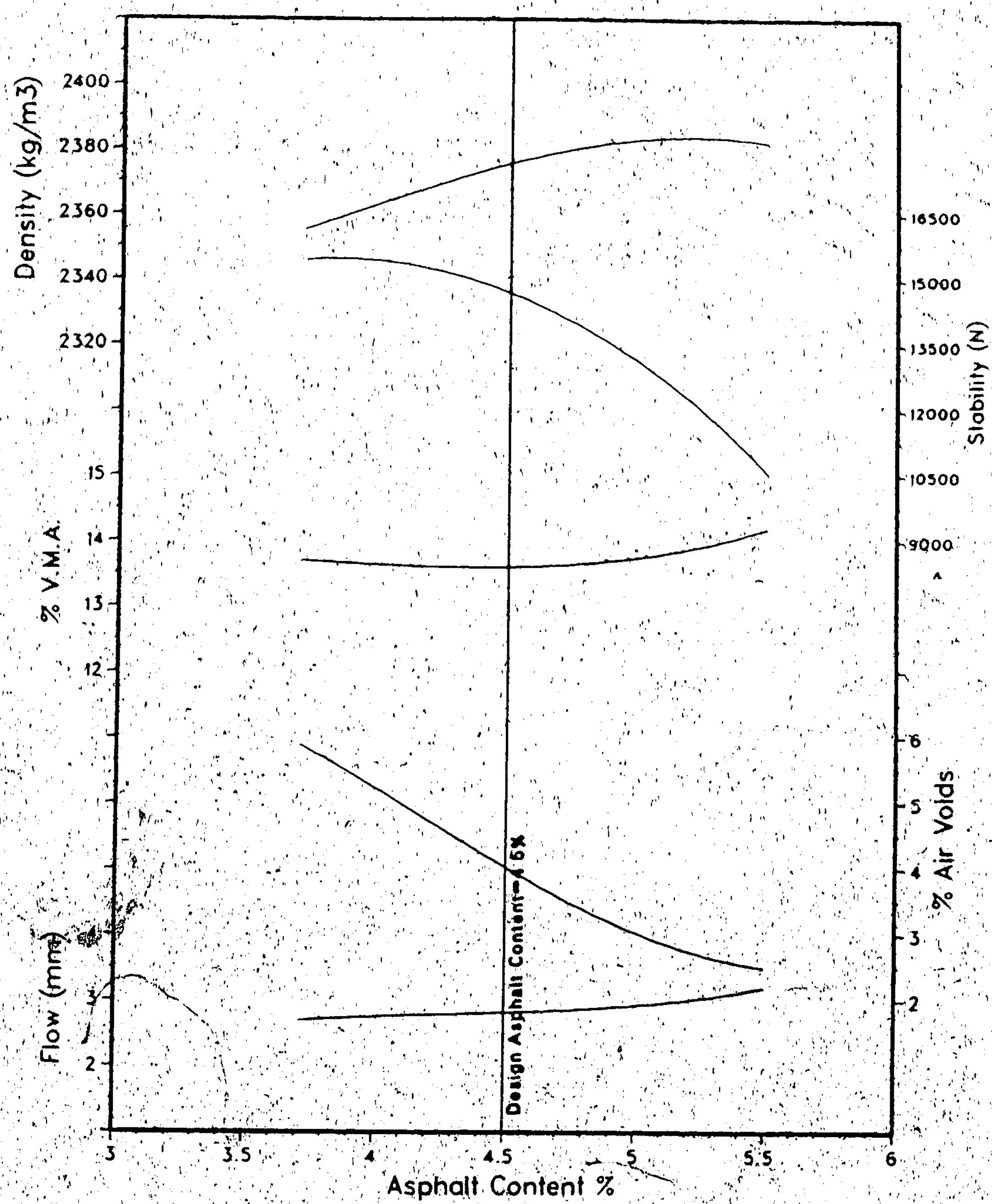

FIGURE B.3 MARSHALL MIXTURE DESIGN CURVES, R/V = 30/70.

RAP.:30%, VIRGIN AGG.:65%, SAND:5%

AGGREGATE GRADATION CHART
DES 1-16 SPECS

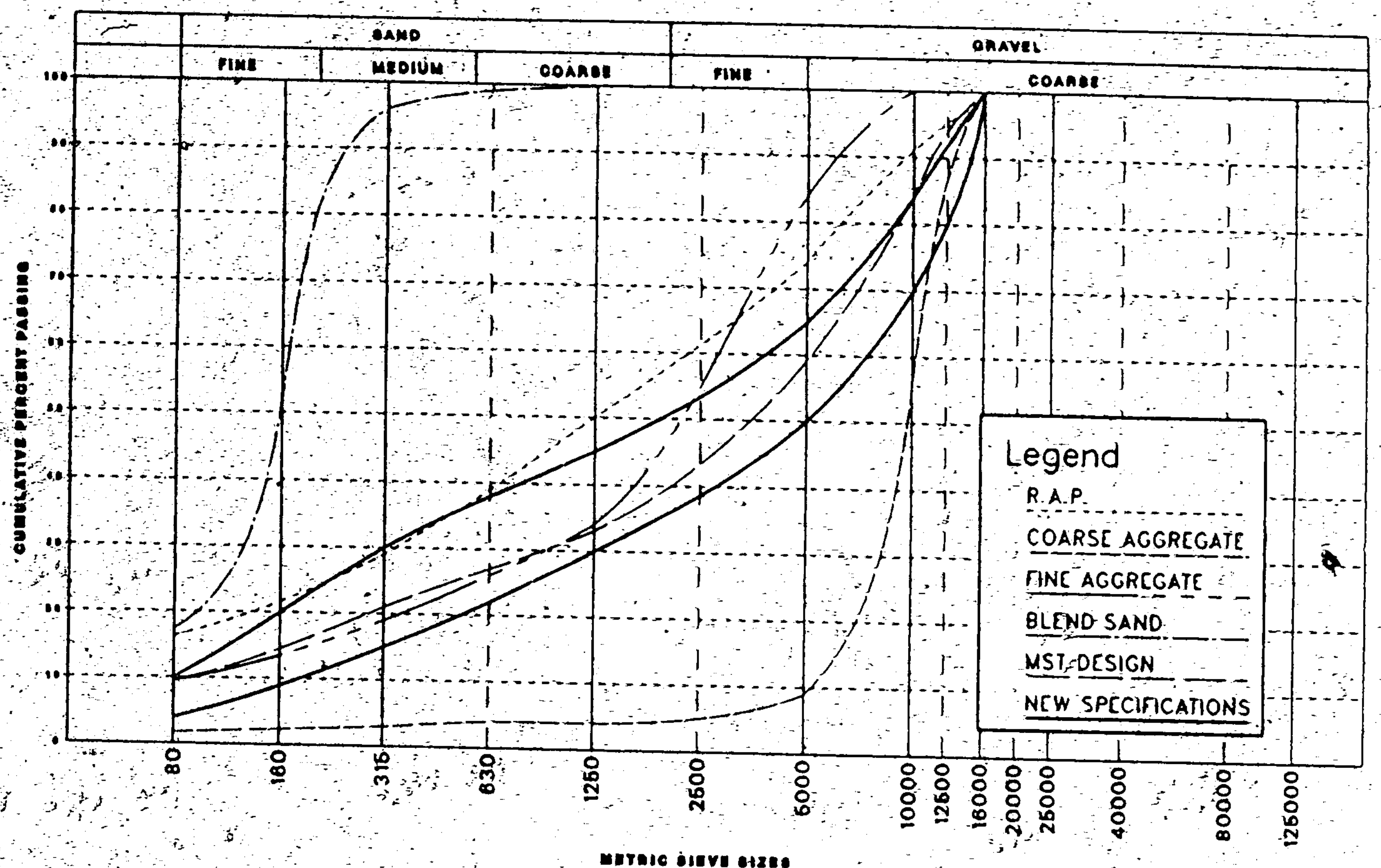

FIGURE B.4 AGGREGATE GRADATION CURVES, R/V = 30/70.

TABLE B-3

MARSHALL MIXTURE DESIGN SUMMARY
R/V = 50/50

DESIGN DATA — R/V = 50/50

AGGREGATE

DES. 1 CLASS 16

BULK SPECIFIC GRAVITY 2.636

ASPHALT ABSORPTION 0.48

PLASTICITY INDEX ____

% FRACTURES (2 FACES) ____

TOP SIZE TO NEXT SIEVE ____ %

ALL + 5000 μm ____ %

SAND/FILLER RATIO 4.5

AUTO COMPACTION 75 blows/face

ASPHALT

TYPE & GRADE 300-400A

SUPPLIER/LOCATION Husky Lloydminster

SPECIFIC GRAVITY 1.020

RESULTS FROM CURVES DRAWN

VIRGIN ASPHALT CONTENT	%	1.3	1.9	2.5	3.1		
TOTAL ASPHALT CONTENT	%	4.0	4.6	5.2	5.8		
DENSITY	kg/m³	2356	2380	2398	2405		
STABILITY	N	15 900	14 500	12 550	10 250		
AIR VOIDS	%	6.2	4.5	3.0	1.9		
VOIDS IN MINERAL AGGREGATE	%	14.1	13.7	13.5	13.8		
FLOW	mm	3.1	2.9	3.2	4.0		

DESIGN RECOMMENDATIONS

SIEVE ANALYSIS μm (CGSB-8-GP-2M)	GRADATION — PERCENT PASSING MINIMUM	AVERAGE	MAXIMUM
16 000		100	
12 500		94	
10 000	78	84	90
5 000	54	60	66
1 250	31	35	39
630	24	27	30
315	17	20	23
160	12.8	14.8	16.8
80	8.9	10.9	12.9

VIRGIN ASPHALT CONTENT	%	2.1
TOTAL ASPHALT CONTENT	%	4.8
DENSITY	kg/m³	2387
STABILITY	N	13 900
AIR VOIDS	%	4.0
VOIDS IN MINERAL AGGREGATE	%	13.6
FLOW	mm	3.0
RETAINED STABILITY (24 h SOAK)	%	88

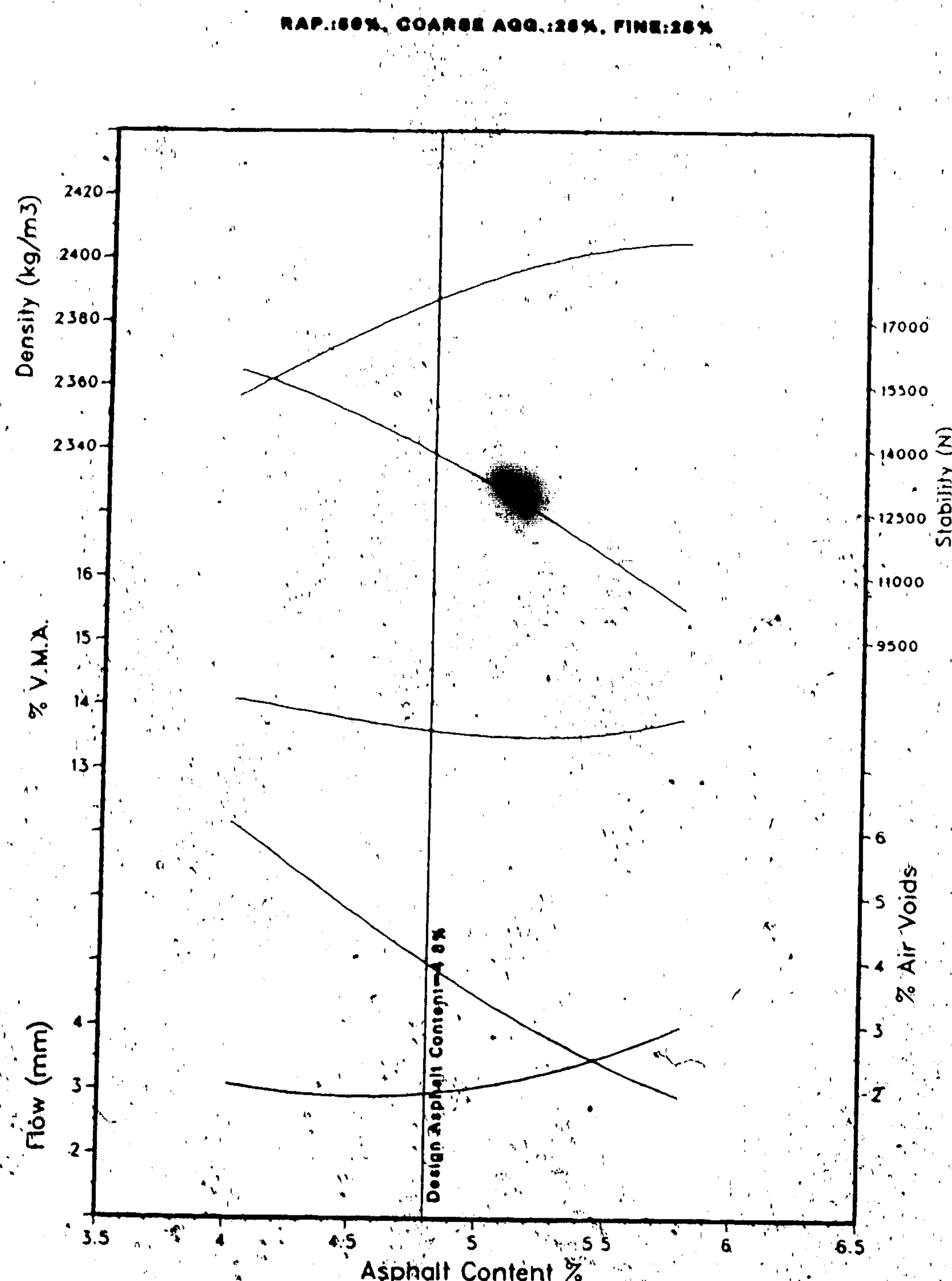

FIGURE B.5 MARSHALL MIXTURE DESIGN CURVES, R/V = 50/50.

RAP.:50%, COARSE AGG.:25%, FINE AGG.:25%

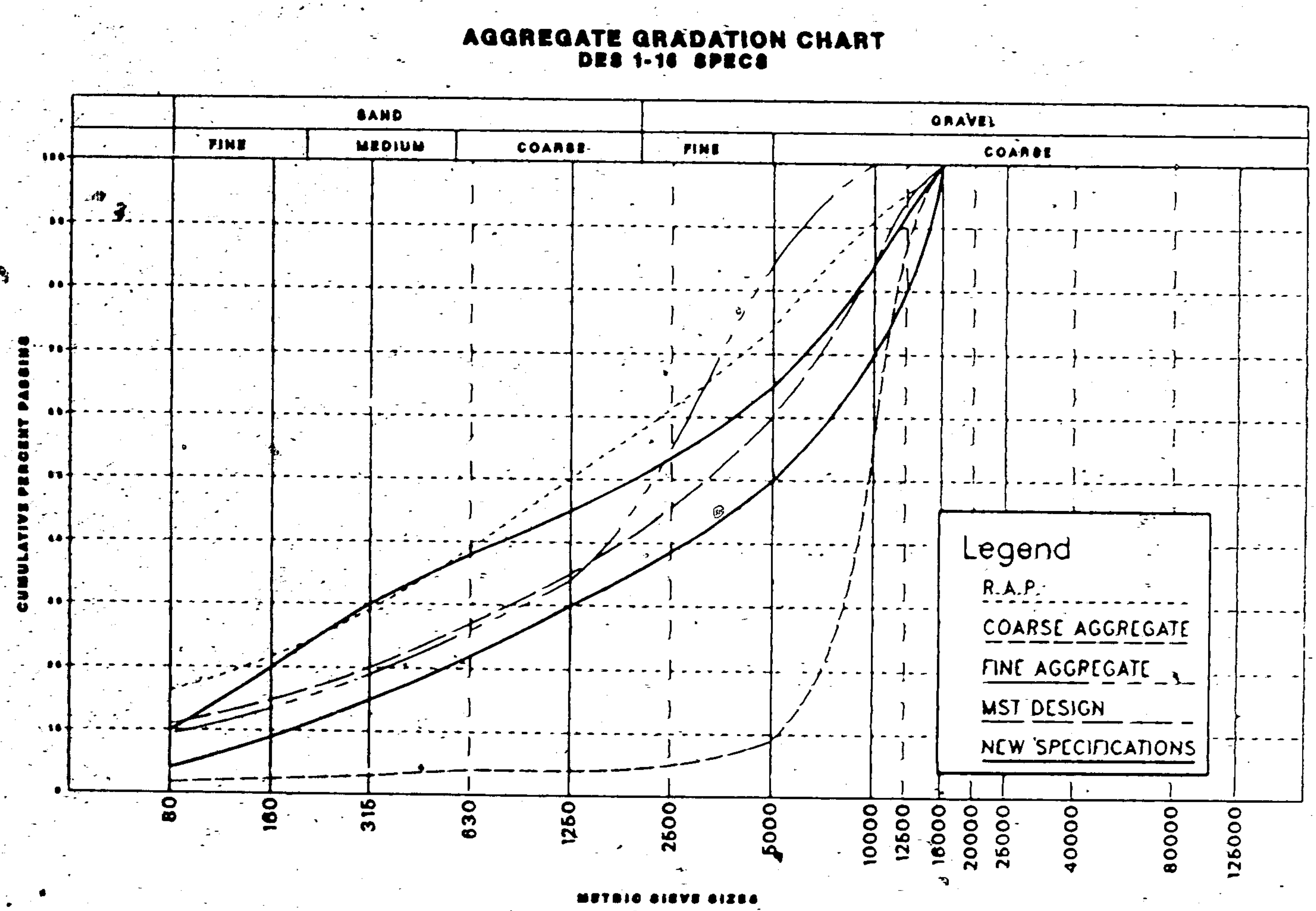

FIGURE B.6 AGGREGATE GRADATION CURVES, R/V = 50/50.

TABLE B.4

MARSHALL MIXTURE DESIGN SUMMARY
R/V = 70/30

DESIGN DATA R/V = 70/30

AGGREGATE

DES. 1 CLASS 16

BULK SPECIFIC GRAVITY 2.640

ASPHALT ABSORPTION 0.57

PLASTICITY INDEX ____

% FRACTURES (2 FACES)

TOP SIZE TO NEXT SIEVE ____ %

ALL + 5000 μm ____ %

SAND/FILLER RATIO 3.6

AUTO COMPACTION 75 blows/face

ASPHALT

TYPE & GRADE 300-400A

SUPPLIER/LOCATION Husky Lloydminster

SPECIFIC GRAVITY 1.019

RESULTS FROM CURVES DRAWN

VIRGIN ASPHALT CONTENT	%	0.2	0.8	1.4	2.0		
TOTAL ASPHALT CONTENT	%	4.0	4.6	5.2	5.8		
DENSITY	kg/m^3	2312	2360	2383	2381		
STABILITY	N	17 600	15 450	13 400	11 700		
AIR VOIDS	%	8.3	5.6	3.9	3.2		
VOIDS IN MINERAL AGGREGATE	%	15.8	14.5	14.2	14.7		
FLOW	mm	3.4	3.5	3.7	4.0		

DESIGN RECOMMENDATIONS

SIEVE ANALYSIS μm (CGSB-8-GP-2M)	GRADATION – PERCENT PASSING MINIMUM	AVERAGE	MAXIMUM
16 000		99	
12 500		96	
10 000	74	80	86
5 000	49	55	61
1 250	32	36	40
630	26	29	32
315	18	21	24
160	13.8	15.8	17.8
80	9.9	11.9	13.9

VIRGIN ASPHALT CONTENT	%	1.4
TOTAL ASPHALT CONTENT	%	5.2
DENSITY	kg/m^3	2383
STABILITY	N	13 400
AIR VOIDS	%	3.9
VOIDS IN MINERAL AGGREGATE	%	14.2
FLOW	mm	3.7
RETAINED STABILITY (24 h SOAK)	%	93

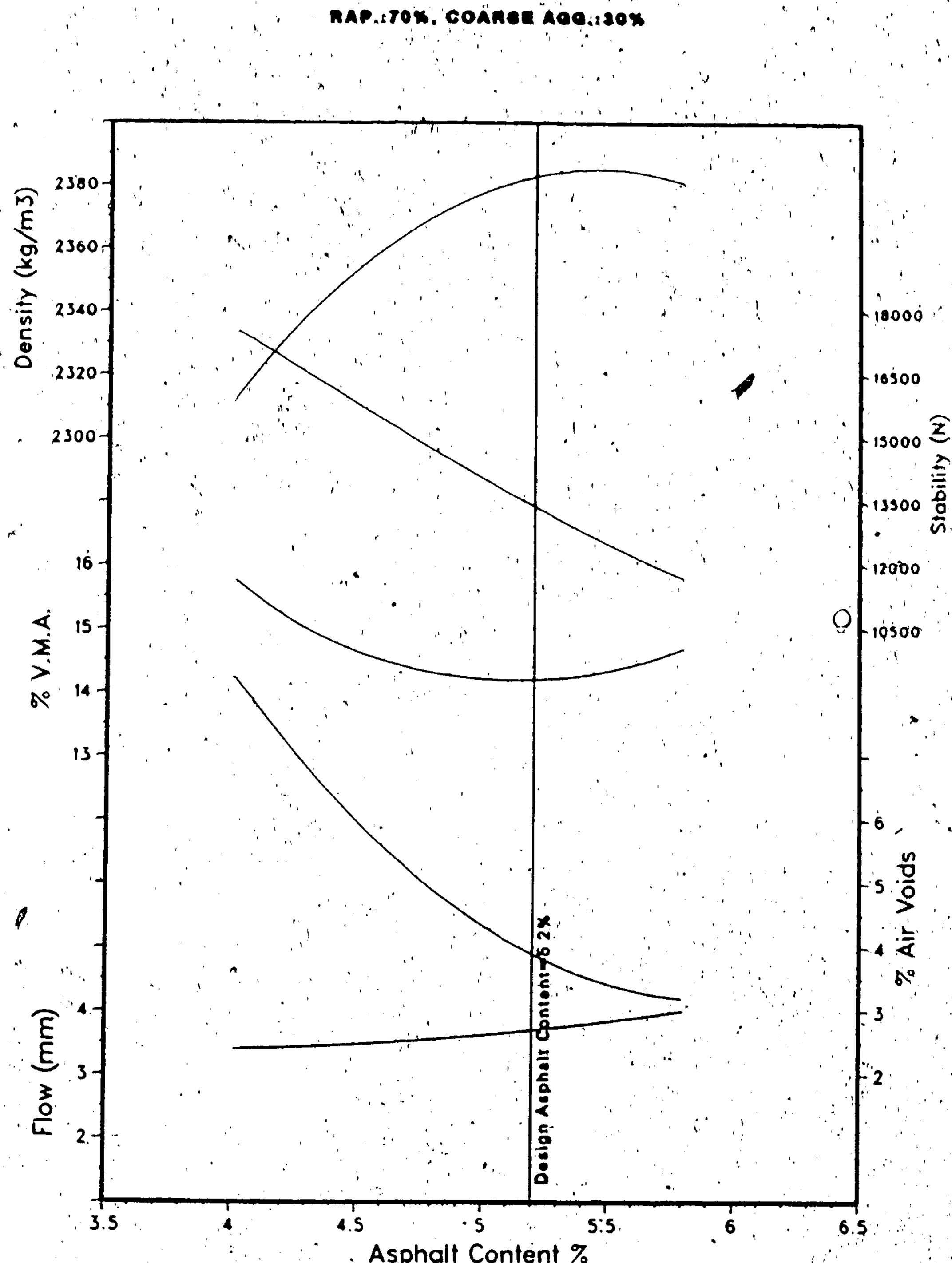

FIGURE B.7 MARSHALL MIXTURE DESIGN CURVES, R/V = 70/30.

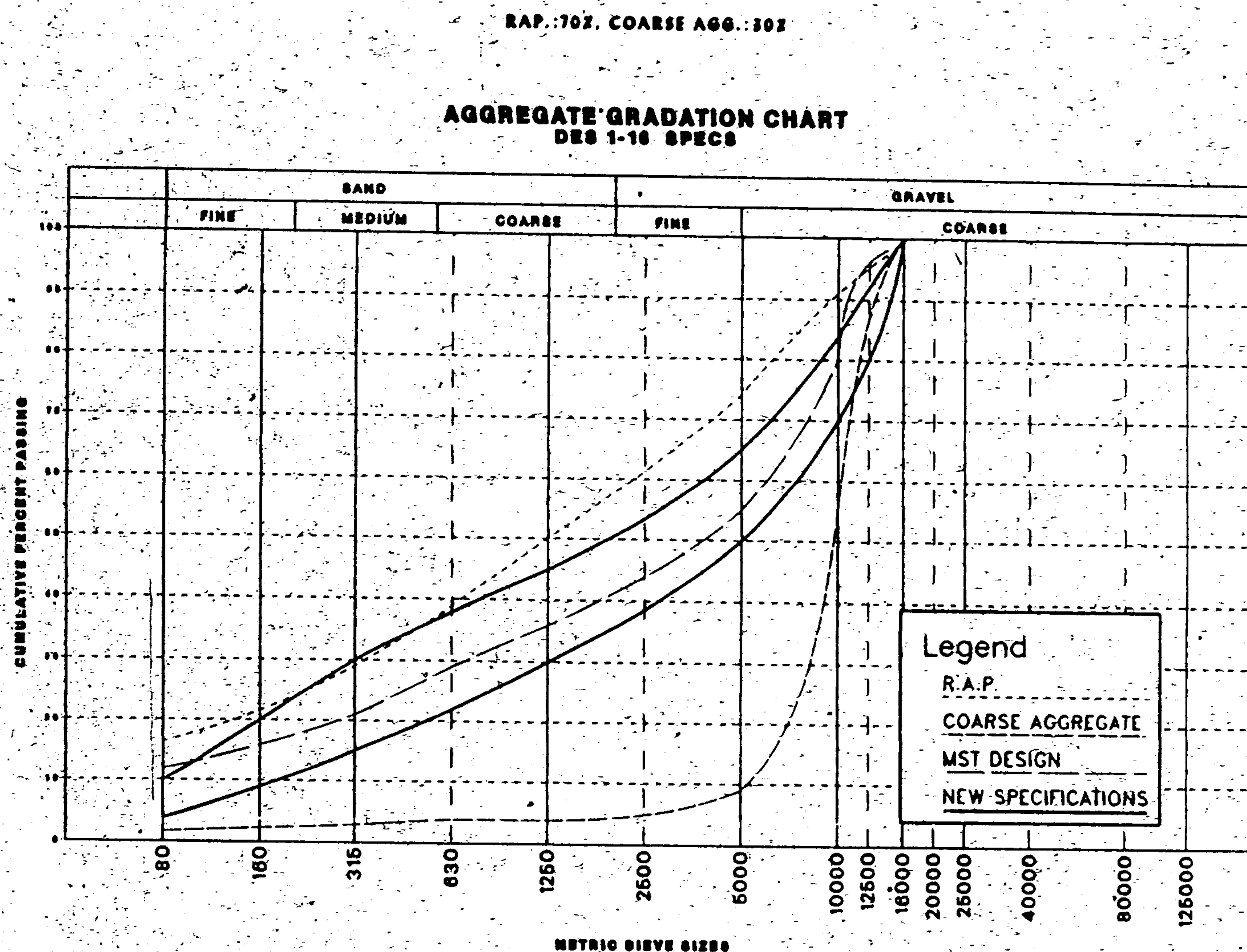

FIGURE B.8 AGGREGATE GRADATION CURVES, R/V = 70/30.

TABLE B.5

MARSHALL MIXTURE DESIGN SUMMARY
R/V = 100/0

DESIGN DATA R/V = 100/0

AGGREGATE		ASPHALT	
DES. 1 CLASS 16	% FRACTURES (2 FACES)	TYPE & GRADE	SC-3000
BULK SPECIFIC GRAVITY 2.648	TOP SIZE TO NEXT SIEVE ___ %	SUPPLIER/LOCATION	Imperial Edmonton
ASPHALT ABSORPTION 0.28	ALL + 5000 μm ___ %	SPECIFIC GRAVITY	1.018
PLASTICITY INDEX ___	SAND/FILLER RATIO 3.5		
	AUTO COMPACTION 75 blows/face		

RESULTS FROM CURVES DRAWN

VIRGIN ASPHALT CONTENT	%	0	0.3	0.8	1.4		
TOTAL ASPHALT CONTENT	%	5.5	5.8	6.3	6.9		
DENSITY	kg/m³	2304	2332	2366	2384		
STABILITY	N	24 450	22 600	18450	11 650		
AIR VOIDS	%	6.3	4.8	2.9	1.3		
VOIDS IN MINERAL AGGREGATE	%	17.5	16.8	16.0	15.8		
FLOW	mm	3.6	3.6	3.7	3.9		

DESIGN RECOMMENDATIONS

SIEVE ANALYSIS μm (CGSB-8-GP-2M)	GRADATION – PERCENT PASSING MINIMUM	AVERAGE	MAXIMUM
16 000		99	
12 500		96	
10 000	85	91	97
5 000	68	74	80
1 250	46	50	54
630	36	39	42
315	26	29	32
160	19.6	21.6	23.6
80	14.3	16.3	18.3

VIRGIN ASPHALT CONTENT	%	0.5
TOTAL ASPHALT CONTENT	%	6.0
DENSITY	kg/m³	2348
STABILITY	N	21 050
AIR VOIDS	%	3.9
VOIDS IN MINERAL AGGREGATE	%	16.3
FLOW	mm	3.6
RETAINED STABILITY (24 h SOAK)	%	95

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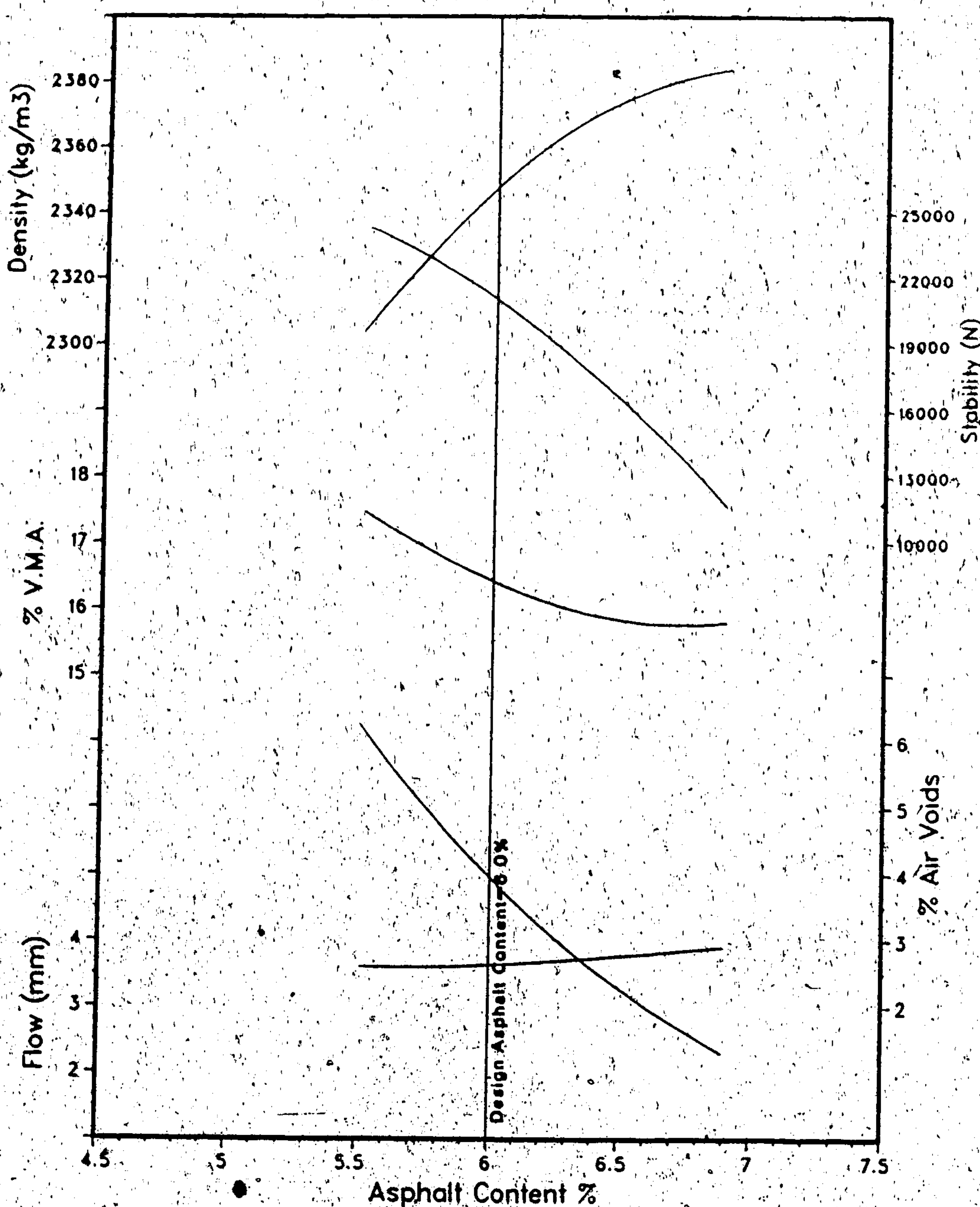

FIGURE B.9 MARSHALL MIXTURE DESIGN CURVES, R/V = 100/0.

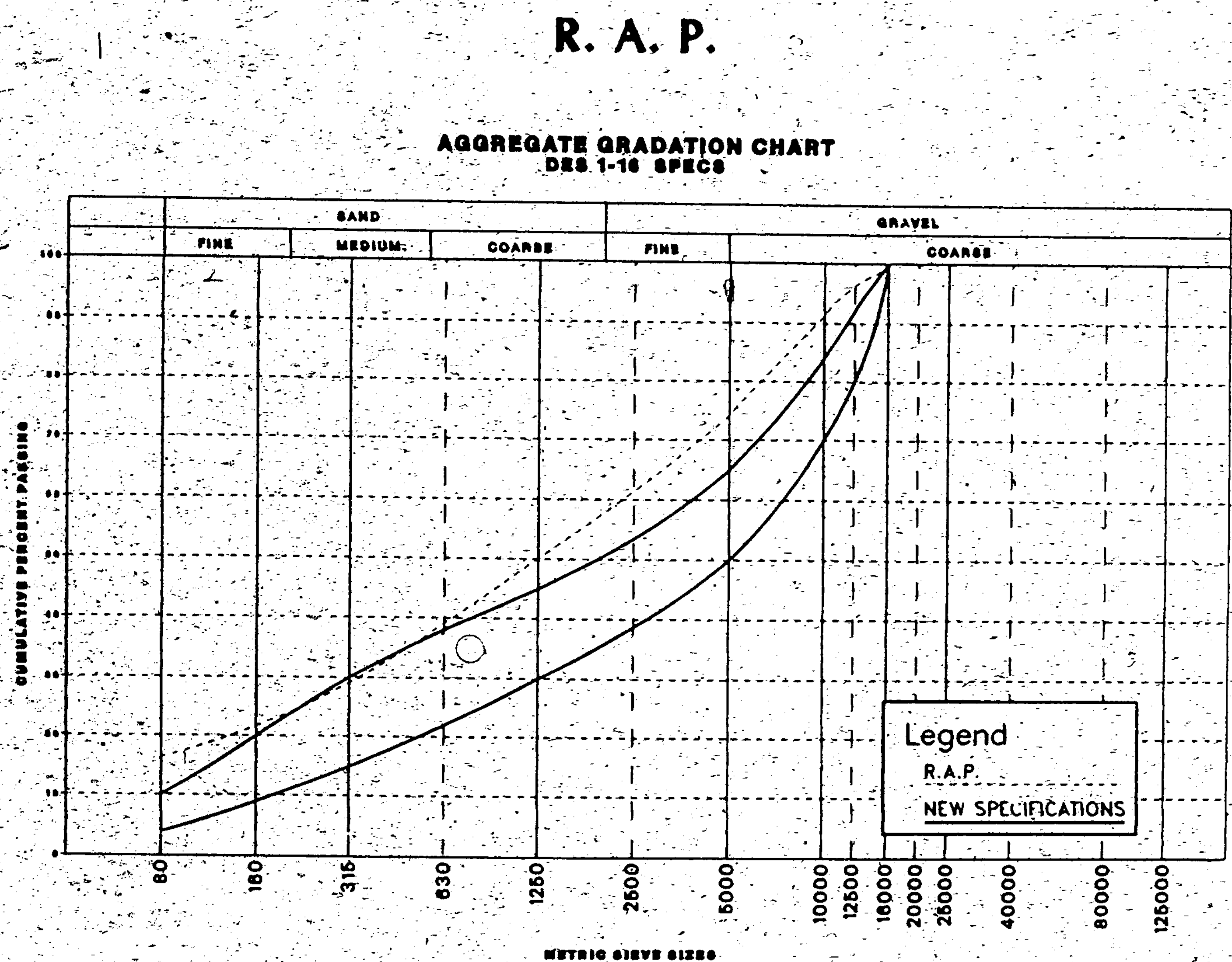

FIGURE B.10 AGGREGATE GRADATION CURVES, R/V = 100/0.

TABLE B.6

MARSHALL MIXTURE DESIGN SUMMARY
R/V = 50/50 (SC-3000)

DESIGN DATA — R/V = 50/50 (SC-3000)

AGGREGATE

- DES. 1 CLASS 16
- BULK SPECIFIC GRAVITY 2.636
- ASPHALT ABSORPTION 0.46
- PLASTICITY INDEX ____
- % FRACTURES (2 FACES)
- TOP SIZE TO NEXT SIEVE ____ %
- ALL + 5000 µm ____ %
- SAND/FILLER RATIO 4.5
- AUTO COMPACTION 75 blows/face

ASPHALT

- TYPE & GRADE SC-3000
- SUPPLIER/LOCATION Imperial Edmonton
- SPECIFIC GRAVITY 1.018

RESULTS FROM CURVES DRAWN

VIRGIN ASPHALT CONTENT	%	1.3	1.9	2.5	3.1		
TOTAL ASPHALT CONTENT	%	4.0	4.6	5.2	5.8		
DENSITY	kg/m³	2358	2380	2397	2404		
STABILITY	N	13 750	13 250	11350	9000		
AIR VOIDS	%	6.1	4.4	3.0	1.9		
VOIDS IN MINERAL AGGREGATE	%	14.0	13.7	13.6	13.8		
FLOW	mm	2.8	2.9	3.1	3.8		

DESIGN RECOMMENDATIONS

SIEVE ANALYSIS, µm (CGSB-8-GP-2M)	GRADATION – PERCENT PASSING MINIMUM	AVERAGE	MAXIMUM
16 000		100	
12 500		94	
10 000	78	84	90
5 000	54	60	66
1 250	31	35	39
630	24	27	30
315	17	20	23
160	12.8	14.8	16.8
80	8.9	10.9	12.9

VIRGIN ASPHALT CONTENT	%	2.1
TOTAL ASPHALT CONTENT	%	4.8
DENSITY	kg/m³	2386
STABILITY	N	12 750
AIR VOIDS	%	3.9
VOIDS IN MINERAL AGGREGATE	%	13.7
FLOW	mm	2.9
RETAINED STABILITY (24 h SOAK)	%	86

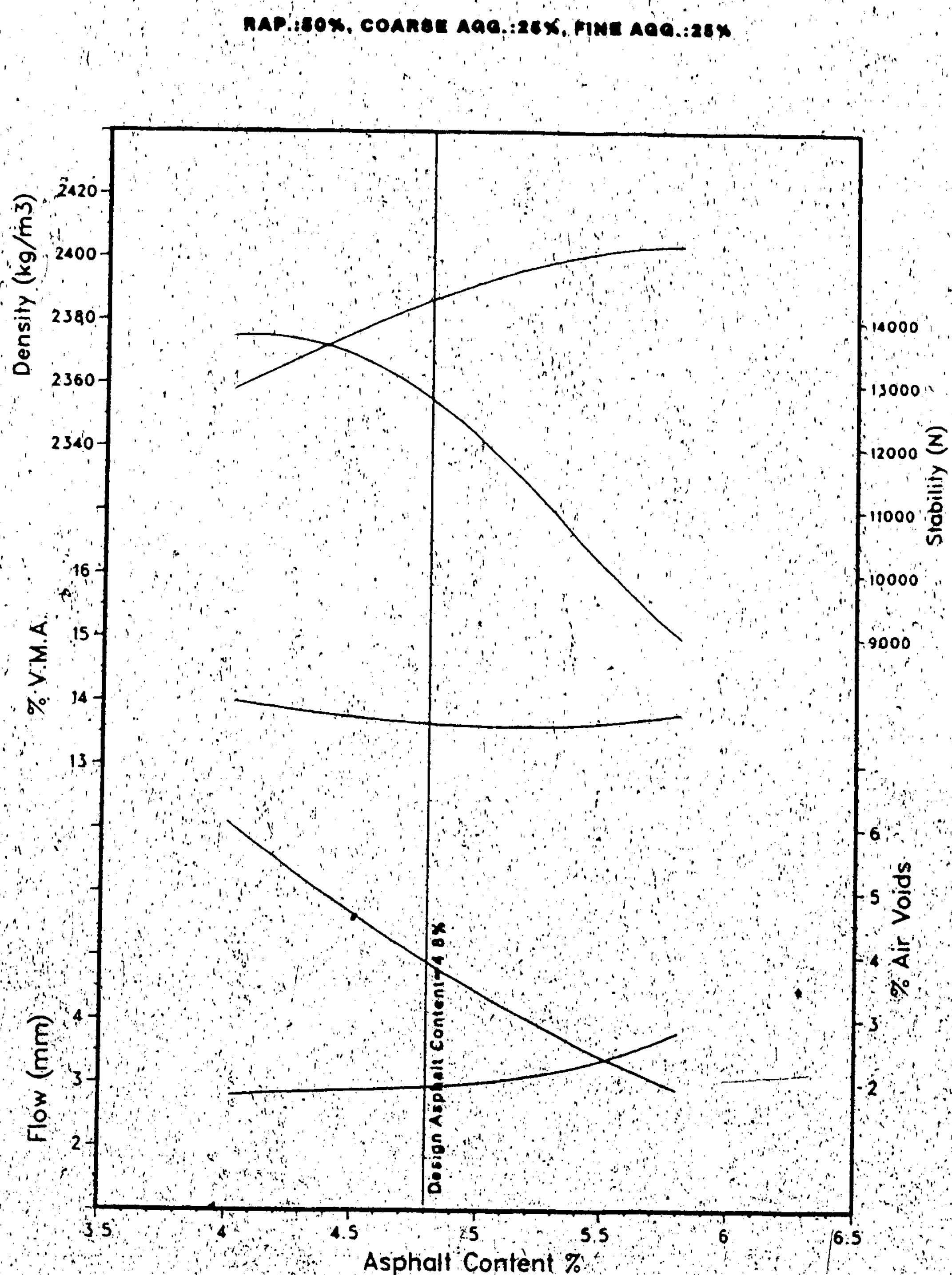

FIGURE B.11 MARSHALL MIXTURE DESIGN CURVES, R/V = 50/50 (SC-3000).

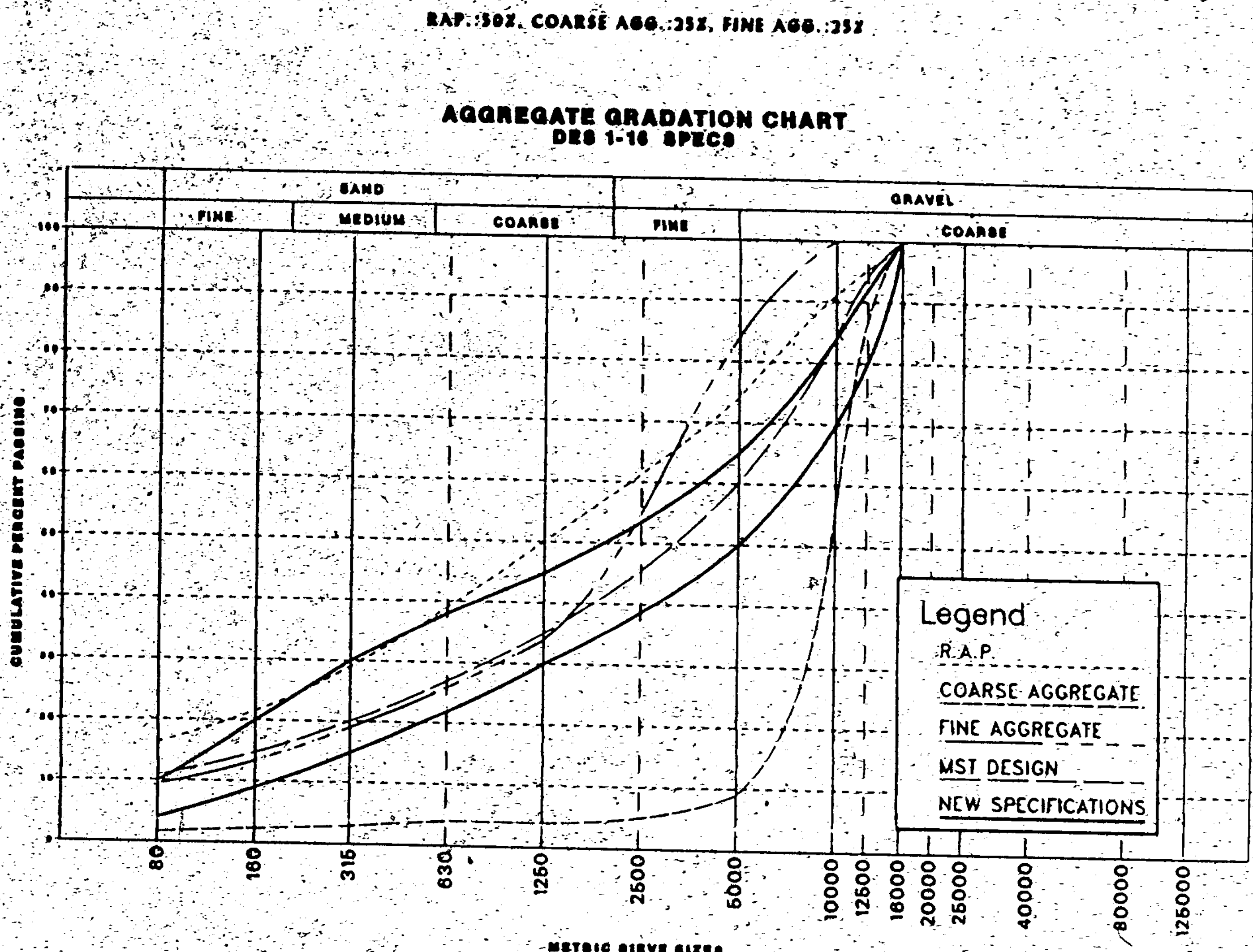

FIGURE B.12 AGGREGATE GRADATION CURVES, R/V = 50/50 (SC-3000).

APPENDIX C

METHOD OF TEST AND ANALYSIS FOR THE LOW TEMPERATURE TENSILE PROPERTIES OF ASPHALT CONCRETE CYLINDERS USING THE TENSILE SPLITTING TEST

APPENDIX C

METHOD OF TEST AND ANALYSIS FOR THE LOW TEMPERATURE TENSILE PROPERTIES OF ASPHALT CONCRETE CYLINDERS USING THE TENSILE SPLITTING TEST

C.1 Introduction

One of the objectives of this investigation was to determine the low temperature tensile properties of conventional and recycled asphalt concrete mixtures by means of the tensile splitting test.

This appendix covers the method of test and analysis developed for determining the low temperature tensile properties of asphalt concrete cylinders using the tensile splitting test. The test can be conducted on asphalt concrete laboratory specimens[1] and cored pavement specimens. One of the first detailed descriptions of the tensile splitting test method and equipment was presented by Anderson and Hahn (113) in 1968. This test has often been used since that time. Recently the test method has been improved by introducing a new computerized data acquisition and processing system. This improvement has been developed in a concurrent research program by Leung (159) and the

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1. For method of making laboratory specimens see The Standard Method of Test for Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus (ASTM Designation: D 1559). Alternate methods for preparation of laboratory specimens may be used.

author (160) at the University of Alberta.

C.2 Summary of Test Method

The tensile splitting test method consists of loading an asphalt concrete cylinder via loading strips across a diameter, in a compression testing frame and within a controlled temperature chamber maintained at a constant low temperature. Output signals from a load cell and three linear variable differential transducers[2], are recorded on floppy diskette by means of a datalog card installed on a microcomputer. Figure C.1 shows the schematic of the test equipment layout.

By the use of the Lotus 1-2-3 spreadsheet program, the raw data recorded in the diskette is processed and the tensile failure stress, failure strain, failure stiffness and the stress-strain diagram can be obtained.

C.3 Significance

This method determines the tensile stress-strain and stiffness characteristics of asphalt concrete at low temperatures and is primarily intended to assist in the design and evaluation of asphalt concrete with respect to thermal cracking.

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2. Two of the LVDTs are attached to the opposite ends of the specimen and measure the horizontal deformation of the specimen. The third LVDT is placed on the loading plate and measures the vertical deformation of the specimen.

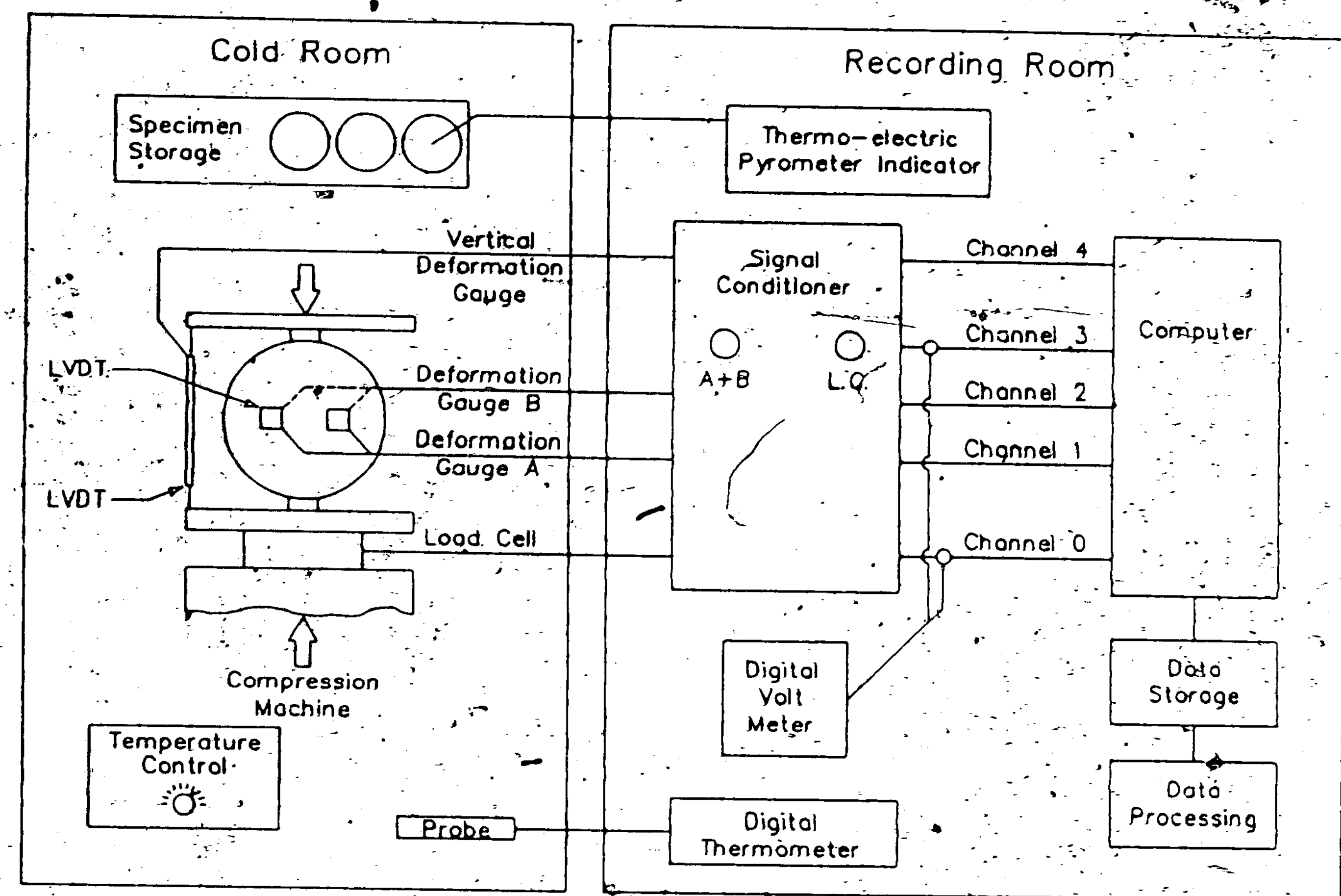

FIGURE C.1 . SCHEMATIC OF THE TENSILE SPLITTING TEST EQUIPMENT.

C.4 Apparatus

C.4.1 Controlled Temperature Chamber

The controlled temperature chamber shall be capable of maintaining test specimens at a constant temperature ± 1°C within the range of +10°C to -30°C during the course of a test. A temperature monitoring device shall have its sensor embedded in a specimen of similar size and composition to the specimen which is to be tested and shall be capable of measuring temperature to ± 0.5°C.

C.4.2 Loading Apparatus

C.4.2.1 Compression Testing Frame

The compression frame[3] shall have a minimum capacity of 5 tonnes and shall be capable of providing the rate of loading of 1.15 mm/min.

C.4.2.2 Supplementary Bearing Bar or Plate

The supplementary bearing bar or plate shall conform to the specifications for this item in the Standard Method of Test for Splitting Tensile Strength of Molded Concrete Cylinders (ASTM Designation: C 496), except that the width of the bearing bar or plate shall be not less than 33 mm.

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3. Wykeham Farrance Mod. 57, 5 ton compression tester manufactured by Wykeham Farrance Engineering Ltd., 127 Edinburgh Avenue, Slough, Bucks, U.K.

C.4.2.3 Bearing Strips

Two steel bearing strips of dimension as shown in Figure C.2 shall be placed between specimen and both the upper and lower bearing plates of the testing machine or between the specimens and supplemental bars or plates, if used.

C.4.2.4 Load Cell

The load cell[4] shall have a minimum capacity of 4.5 tonnes and shall be capable of measuring compressive loading to ± 1 percent of true at the rate of loading of 1.15 mm/min.

C.4.3 Gauge Points, and Marking and Mounting Apparatus

C.4.3.1 Gauge Points

The gauge points shall be 9.525 x 9.525 x 6.350 mm (0.025 mm from mean in any dimension) brass plates.

C.4.3.2 Gauge Point Jig

The gauge point jig shall provide slots for marking the specimen and holes for mounting the Gauge Points. Figure C.3 shows the schematic of the gauge point jigs.

C.4.4 Deformation Measurement Apparatus

---

4. Kwoya Musen Load Cell Mod. LC-5, 5 ton manufactured by Kwoya Musen Kenkyujo Co., Ltd., Tokyo, Japan.

C.4.4.1 Horizontal Displacement Gauges

The displacement gauges[5] shall be two linear variable differential transducers of matched sensitivity (within 5%) and be capable of measuring displacements to within ± 0.00125 mm and shall have a stroke of not less than ± 0.25 mm.

C.4.4.2 Displacement Gauge Core and Coil Assemblies

The two displacement gauge core and coil assemblies which hold the Horizontal Displacement Gauges shall be made of brass. The schematic of the displacement gauge core and coil assemblies is shown in Figure C.4.

C.4.4.3 Vertical Deformation Gauge

The displacement gauge[6] shall be a linear variable differential transducer capable of measuring displacement to within 0.01 mm. The gauge shall be mounted on the frame and measures the movement of the loading plate.

C.4.4.4 Displacement Gauge Calibration Jig

The displacement gauge calibration jig shall be made of brass and aluminum. The dial gauge of the displacement gauge calibration jig shall be a 0.0025 mm dial gauge. The schematic of the jig is shown in Figure C.5.

---

5. Sanborn Linear Variable Differential Transformers Mod. 595 DT 025 manufactured by Sanborn Co., 175 Wyman Street, Waltham 54, Massachusetts, USA.
6. Linear Variable Differential Transducers manufactured by Hewlett Packard.

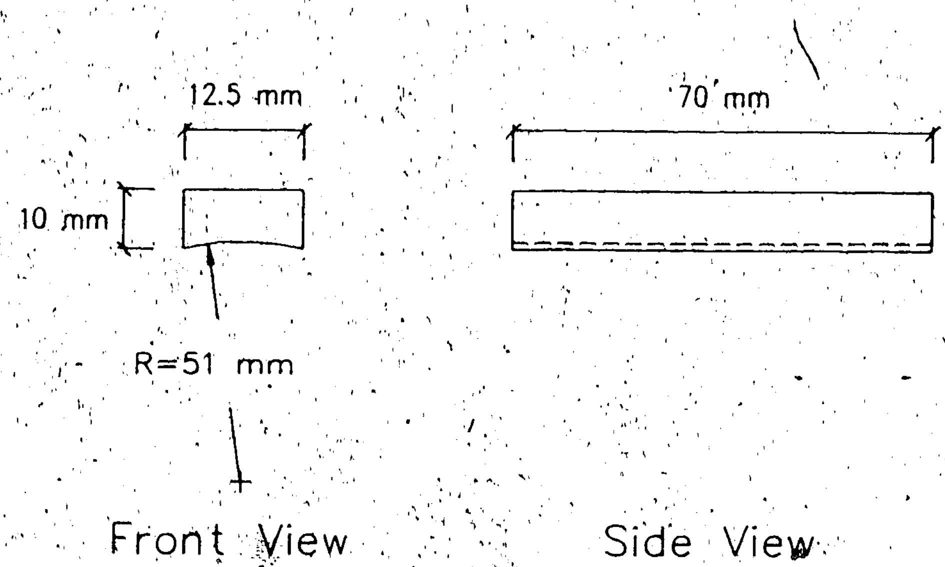

FIGURE C.2 LOAD BEARING STRIPS.

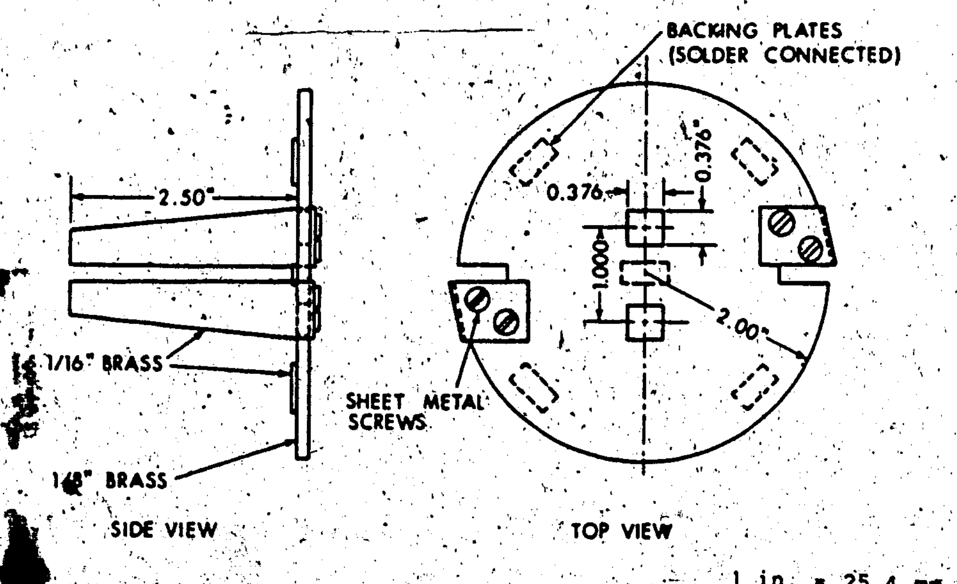

FIGURE C.3 GAUGE POINT JIG.

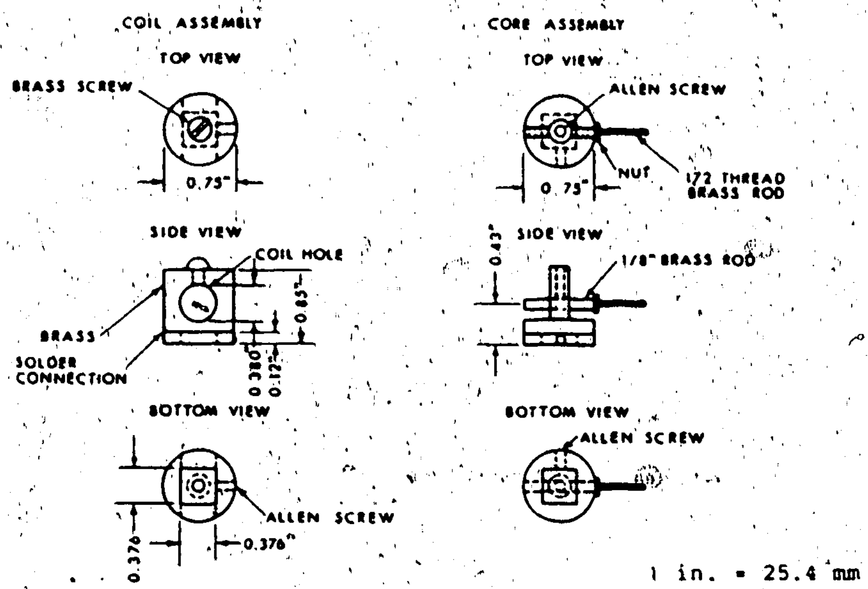

FIGURE C.4 DISPLACEMENT GAUGE CORE AND COIL ASSEMBLIES.

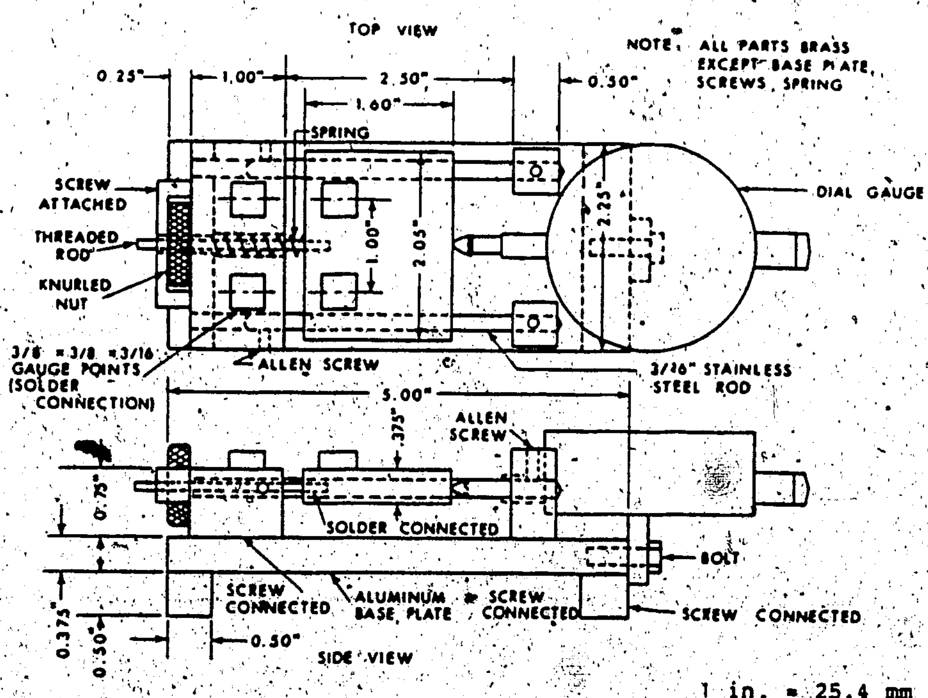

FIGURE C.5 . DISPLACEMENT GAUGE CALIBRATION JIG.

C.4.5 Data Acquisition Apparatus

C.4.5.1 Computer Hardware

The computer hardware[7] for acquiring and recording test data consists of the following:

1. A microcomputer system with minimum 512K Ram is required although 640K Ram is preferred in order to provide a margin of safety for the computer operation.
2. Two double sided, double density disk drives are required in order to run the software. The first or 'A' drive contains the operating system and the BASIC program. The second or 'B' drive is used to store test data upon completion of the test.
3. A multifunction card is used for printer communication.
4. A clock card is used to note the time.
5. A Metra Byte Dash-8 Board[8] is used to collect the test data in analog form and convert them into digital form for use by the computer. The Dash-8 board has 8 channels available for datalogging. Only five are needed. They are:
   a. Channel 0 - Load Cell
   b. Channel 1 - LVDT A
   c. Channel 2 - LVDT B
   d. Channel 3 - Average of A and B

---

7. A microcomputer, IBM PC clone, manufactured by Operand Electronics Ltd. of Edmonton.
8. The Dash-8 board is manufactured by the Metra Byte Corp.

e. Channel 4 - Vertical LVDT

The Dash-8 board has a full scale input of ± 5 volts on each channel with a resolution of 0.00244 volts.

C.4.5.2 Computer Software

Two software packages are required by the computer to acquire and record the test data. One is the IBM PC DOS version 3.1 and the other is the Dash-8 configuration package. A BASIC program written specifically for the tensile splitting test is also required.

The DOS[9] disk operating system allows the establishment of a virtual disk on the computer for temporary data storage. When the test is finished, the contents of the virtual disk is transferred to the 'B' drive.

The Dash-8 software package[10] provides the input output driver routine which can be accessed from BASIC using the Call statement.

The BASIC program defines various functions and operation in the use of the computer hardwares. The main function and operation defined in this program include the gathering of test data at designated intervals and duration; and the storing of the data in the computer and copying them to drive 'B', by using the Dash-8 software package. A

---

9. Details of the software plus technical information, are given in the DOS 3.1 manual.
10. Details of the software plus technical information, are given in the Dash-8 manual.

listing of this program is enclosed in Section C.8.1.

C.4.5.3 Signal Conditioner

The signal conditioner is used to amplify, filter and condition the input signals from the test before sending the signals to the Dash-8 board.

The conditioner also serially connects the input signals of the LVDT A and LVDT B resulting in an average value for the horizontal deformation.

In addition to the above functions, the conditioner is used to zero the signals of the load cell and the LVDTs, before the start of each test.

C.5 Test Specimens

C.5.1 Asphalt Concrete Laboratory Specimens

If Marshall specimens are to be tested they shall conform to the specifications set forth in ASTM Method C 1559-76.

C.5.2 Asphalt Concrete Cored Pavement Specimens

If cored pavement specimens are to be tested they shall be trimmed to a cylindrical shape (within ± 0.25 mm of the mean length and diameter) having a diameter of 102 mm, ± 2.5 mm and a length of less than 102 mm.

C.6 Procedures

C.6.1 Calibration

C.6.1.1. Load Cell

The Load Cell shall be calibrated at room temperature (if temperature compensating) or at the test temperature (if non-temperature compensating), on a Compression Tester whose load accuracy has been verified to ± 1 percent in accordance with the Standard Methods of Verification of Testing Machines, ASTM Designation: E4-64.

C.6.1.2 Dial Gauge

The dial gauge shall be calibrated while on the Displacement Gauge Calibration Jig described in Section C.4.4.4, using machinist's gauge blocks.

C.6.1.3 Displacement Gauges

The two horizontal displacement gauges shall be calibrated ~~when the two gauges are connected~~ in series. They shall also be calibrated separately. The calibration shall be carried out on the Displacement Calibration Jig (using a 25.4 mm gauge length at null) at the test temperature. Output signal (in terms of voltage) from the displacement gauges shall be measured by a digital voltmeter as well as by the computer data acquisition system.

C.6.2 Preparation of Specimen for Testing

C.6.2.1 Measurement

Determine the length and diameter of the test specimen to the nearest 0.25 mm by averaging four readings at each dimension.

C.6.2.2 Marking

Mark diametral loading points on each end of the specimen in the same axial plane using the Gauge Point Jig.

C.6.2.3 Gauge Point Attachment

Cool the specimens to at least -10°C for about 2 hours before attaching the gauge points. Coat one side of each of two gauge points with warm asphalt cement. (Use grade 200/300 pen asphalt cement for testing at temperature of -10°C or below. Use grade 85/100 pen asphalt cement for testing at temperature above -10°C.) Warm the gauge points and insert the two coated Gauge Points through the holes in the aligned Gauge Point Jig and press firmly onto the specimen. Leave the specimen to cool horizontally for approximately 3 minutes to firmly affix the gauge points to the specimen. Invert the specimen (and prop in a manner that will not disturb the previously attached Gauge Points) and attach the other two Gauge Points in a similar manner.

C.6.2.4 Cooling

Immediately place the specimen into the Controlled

Temperature Chamber.

C.6.3 Preparation for Loading of the Specimen

C.6.3.1 Specimen Inspection

After the specimen to be tested has reached equilibrium temperature, inspect it for Gauge Point slippage. If any slippage is evident remove the Gauge Points and repeat steps C.6.2.3 and C.6.2.4.

C.6.3.2 Positioning

Place the Load Cell on the loading ram platen of the Compression Testing Frame. Position the specimen so that the marked loading points are in a vertical plane passing through the center of thrust and so that the longitudinal axes of the Bearing Strips are in this vertical plane. Raise the loading ram of the Compression Testing Frame just enough to secure the specimen for Displacement Gauge attachment.

C.6.3.3 Displacement Gauge Attachment

Tie both of the Displacement Gauge Core and Coil Assemblies to some point on the Compression Testing Frame to obviate damage after specimen failure. Simultaneously place the rear Displacement Gauge Core and Coil Assembly onto the Gauge Points and then secure the assemblies by tightening the allen screws. Repeat the foregoing attachment procedure for the front Displacement Gauge Core and Coil Assembly.

C.6.4 Loading and Recording Procedure

C.6.4.1 Loading Rate

Set the Compression Testing Frame to a nominal loading rate of 1.15 mm/min. The actual loading rate may vary from the nominal loading rate by ± 10 percent but must be reproductible within ± 1 percent.

C.6.4.2 Loading

Engage the Compression Testing Frame and return to the recording area (Note: loading will not begin until the power supply switch for the servo motor is closed. This switch should be located in the recording area, adjacent to the recorder).

C.6.4.3 Recording

Make sure all the wirings are hooked up correctly. Adjust the signal conditioner switches such that the voltage outputs from the load cell and the displacement gauges to the datalog card are conditioned close to 0.000 V. Run the computer program for datalogging. Input information as requested from the screen. This includes the duration of recording data, the frequency of reading data and the name of channels to be used (Channel 0 to Channel 4). The name of the sample will also be requested and will be used as the file name of the dataset. Press the run key and the computer will start to record data.

Record all other pertinent data such as date of test and test temperature (air and specimen) on the laboratory log book.

The computer will stop recording data after the given period of time and copy the data set into floppy diskette.

C.6.4.4 **Termination of Test**

Upon failure of the specimen, turn off the power supply switch for the servo motor. Disengage the Compression Testing Frame and examine the fractured specimen. If the fracture surface passes under a Gauge Point the test shall be rejected.

C.7. **Calculations**

C.7.1 **Tensile Stress**

The tensile stress at any point to failure shall be calculated as follows:

$$\sigma_t = \frac{2P}{\pi t d}$$

where

σ_t = tensile stress, kPa,

t = specimen thickness, m,

d = specimen diameter, m, and

P = applied load, kN calculated as follows:

$$P = \frac{N_p k_1}{410}$$

where

N_p = values, in binary bit form as recorded in channel 0 of the data file, 1 volt = 410 bits,

k_1 = conversion factor of load cell, kN/volt, obtained from calibration.

C.7.2 Strain

The strain, at any point to failure is measured by the two horizontal deformation gauges, A and B, and calculated as follows:

$$\varepsilon_t = \frac{N_{ab} k_2}{410 \times 25.4}$$

where

ε_t = average strain of strain gauges A and B, mm/mm,

N_{ab} = value as recorded in binary bit form in channel 3 of the data file, 1 volt = 410 bits,

k_2 = conversion factor of the deformation gauges, mm/volt.

Due to the biaxial state of stress existing within the cylindrical specimen, the displacement measured between the gauge points is a result of both compressive stresses in the vertical direction and tensile stresses in the horizontal direction. The term strain is used without differentiation as to its cause. If tensile strain is desired, as for calculation of a stiffness modulus, use of equations applying the Generalized Hooke's Law is necessary.

C.7.3 Failure Strain

The failure strain shall be considered as the strain corresponding to the first maximum stress reached during the test.

C.7.4 Tensile Strength

The tensile strength shall be considered as the maximum tensile stress.

C.7.5 Stiffness Modulus

The tensile stiffness modulus at any point to failure shall be calculated as follows:

$$S_{mix} = \frac{0.912\ \sigma_t}{0.5 \times \varepsilon_t}$$

where

S_{mix} = Tensile Stiffness Modulus, MPa,

σ = tensile stress, MPa, and

ε_t = average strain of strain gauges A and B in mm/mm.

C.7.6 Data Processing

When the test is terminated, the raw test data stored in the disk is processed by an IBM XT microcomputer using the Lotus 1-2-3 spreadsheet program. A Lotus 1-2-3 Macro program is written specifically to perform the calculations which are described earlier in this chapter. A listing of the Macro program is contained in Section C.8.2.

The printout of the processed data included the stress, strain and stiffness of the specimen at each point of time during the test. The failure stress, failure strain and failure stiffness are determined by locating the maximum stress the specimen has first experienced.

By selecting the appropriate pairs of stress-strain data, the stress strain diagram of the test specimen can be drawn by means of the Lotus 1-2-3 graph software or any other plotting programs.

C.8 Listing of the Computer Programs for the Tensile Splitting Test

C.8.1 BASIC Program for Tensile Splitting Test Data Acquisition using the DASH-8 Card

```
50 '*************************************************************
60 '*                  Tensile Splitting Test                   *
70 '*             Control and Datalogging System                *
80 '*************************************************************
100 DEF SEG=&H1700 'Setup load segment
110 BLOAD "dash8.bin",0 'Loads at 1700:0000
114 '.......................
115 '   Load dash8 address
116 '.......................
120 OPEN "dash8.adr" FOR INPUT AS #1
130 INPUT #1, BASADR%
135 CLOSE #1
136 CLS
200 DIM DIO%(8),TA(8),LT%(2):AN=24 'average number.
204 '.......................
205 'Enter variables
206 '.......................
210 INPUT "Enter length of stage #1(1 to 60 min.)":S1
220 INPUT "Enter interval(1 to 60 sec)":I1
230 INPUT "Enter length of stage #2(1 to 60 min.)":S2
240 INPUT "Enter interval(1 to 60 sec)":I2
250 TCOUNT=(S1*(60/I1))+(S2*(60/I2))
260 INPUT "Enter first channel":FC%
270 INPUT "Enter last channel":LC%
272 '.......................
273 'Set dash8 first and last channel
274 '.......................
275 MD%=1:LT%(0)=FC%:LT%(1)=LC%:CALL DASH8 (MD%, LT%(0), FLAG%)
277 '.......................
278 'Set up files
279 '.......................
280 INPUT "Enter file name":F$:FF$="c:"+F$+".prn"
290 OPEN FF$ FOR OUTPUT AS 3
295 '.......................
305 'init dash8
307 '.......................
400 MD%=0:CALL DASH8 (MD%, BASADR%, FLAG%)
407 '.......................
450 'enter stage #1
455 '.......................
465 ON TIMER(I1) GOSUB 4000 'go take reading
470 ON KEY(1) GOSUB 2000:ON KEY(2) GOSUB 2500 'start and stop test
480 KEY(1) ON:KEY(2) ON ' turn function keys on
485 PRINT"Press F1 to start test"
500 IF TEST=1 THEN GOTO 3000 'start datalogging
510 GOTO 500 'wait here for test to start
1000 '.......................
2000 TEST=1:RETURN 'set flag
2400 '.......................
2500 OF=1:RETURN
2600 '.......................
3000 TIMER ON 'start datalogging
3010 IF OF=1 THEN GOTO 5000 'test finished
3020 GOTO 3010 'wait here for timer
4000 IF READNO.=TCOUNT THEN GOTO 5000 'is test finished
4001 IF READNO.=S1*(60/I1) THEN ON TIMER(I2) GOSUB 4000 'modify timer interval
4002 FOR A=0 TO AN 'an=number of averages
4003 MD%=2:CH%=FC%:CALL DASH8 (MD%, CH%, FLAG%) 'set dash8 first channel
4005 FOR I=FC% TO LC% 'a/d routine
4010 '.......................
```

```
4015 'take readings with dash8
4016 '.........................
4020 MD%=4:CALL DASH8 (MD%, DIO%(I), FLAG%):TA(I)=TA(I)+DIO%(I):NEXT I:NEXT A
4025 FOR I=FC% TO LC%:DIO%(I)=TA(I)/(AN+1):TA(I)=0:NEXT I
4026 '.........................
4027 'set increment number and get real time
4028 '.........................
4030 READNO.=READNO.+1:TC=VAL(LEFT$(TIME$,2)+MID$(TIME$,4,2)+RIGHT$(TIME$,2))
4035 '.........................
4040 PRINT TC READNO.;:FOR I=FC% TO LC%:PRINT DIO%(I);:NEXT I:PRINT
4050 PRINT #3,TC READNO.;:FOR I=FC% TO LC%:PRINT #3,DIO%(I);:NEXT I:PRINT #3,
4060 RETURN
5000 CLOSE 3:SYSTEM 'return to system
```

C.8.2 Lotus 1-2-3 Macro Program for Tensile Splitting Test Data Processing

The following is a listing of the LOTUS Macro that is used to perform the necessary calculations on the data from the tensile splitting tests. Along with the macro are included some comments to aid in future modifications of the macro. These comments DO NOT appear on the worksheet. The macro does not appear here in the same format as on the worksheet in order to facilitate the inclusion of comments.

This is the main macro which controls the selection of files to be processed. The files are to be listed under the headings of: "SAMPLE" and "TEMP" and "THICK". Under the "SAMPLE" heading input the name of the file that contains the test data. The "TEMP" requires the input of the temperature at which the sample was tested. The "THICK"ness is to be input in millimeters.

```
{goto}BEGIN¬            | this section      SAMPLE     TEMP    THICK
/rndIDA¬                | initializes
/rndTEMP¬               | the macro         RW20       -20     69.6
/rndTHIK¬               |                   H12        -30     69.7
/rncIDA¬¬
{goto}IDA¬
/rndIDA¬
{down}
/rncIDA¬¬
{right}
/rncTEMP¬¬
{right}
/rncTHIK¬¬
/xiTHIK=0¬/xq¬
/cIDA¬
SAMNO¬
/cIDA¬                   | this section copies the data
SAMNO2¬                  | to other portions of the wks for
/cIDA¬                   | subsequent use by the following macros
FRET¬                    |
/cTEMP¬                  |
SPTEMP¬                  |
/cTHIK¬                  |
THICK¬                   |
/xcSTART¬                | this transfers control to the "START-UP" macro
/rndTEMP¬
/rndTHIK¬
/xgLOOP4¬                | this loops the macro back to the first line


START-UP MACRO

/wgrm                    | set the wks recalc to manual
/reALL¬                  | erase old data
/rncTEST2¬¬
/rncTEST3¬¬
/rndTEST2¬
/rndTEST3¬
{goto}A5¬
/fInUSR\GARYV\TENSILE\               | importing the data file
H12
¬{goto}TABLE1¬
+0.0000001¬
/c¬C250..J250¬                       | initializing the first value to
{goto}TABLE2¬                        | approx. zero (0) on calc table
(@ABS(C6-$C$5)+1.0000E-11)
/410*1000¬                           | calculations to convert the imported
{right}                              | data (bit format) into SI form of data
(@ABS(D6-$D$5)+1.0000E-11)  A        | ie. deflections, loads etc.
/410*0.01¬
```

```
(right)
(@ABS(E6-$E$5)+1.0000E-11)  B
/410*0.01~
(right)
(@ABS(F6-$F$5)+1.0000E-11)  A+B
/410*0.0156~
(right)
(@ABS(G6-$G$5)+1.0000E-11)  LVDT VERT
/410*3.9267~
(goto)STRESS~                     | calculating the stress on the sample
+(TABLE2*2.204*2/
(@PI*($THICK/25.4)*4))*6.89476~
(goto)STRATE~                     | calculating the strain rate
(G251-G250)*60~
(goto)SISTIFF~                    | calculating the sample stiffness
(0.912*H251/(0.5*F251))/1000~
/cA5..B240~
A250~
(goto)TABLE2~
/cC251..J251~
C251..J490~
(calc)
/rncTEST2~~
/xiTEST2=TEST~/xcERASE~           | this is a loop which finds the maximum
/xiTEST2=TEST~/xgTIME~            | load applied.  once found the loop
/rndTEST2~                        | invokes a subroutine that erases the
(down)                            | data ten points beyond the maximum.
/rncTEST2~~                       | see the "ERASE" sub macro.
/xgLOOP~                          | after erasing control is passed to the
                                  | "TIME" macro


FAILURE

(calc)                  | this sub macro finds and copies the failure
/dqriINPUT~             | point to another location on the spreadsheet.
cCRITERION~
oOUTPUT~
eq
/xgPRINT~


ERASE

(DOWN)     | this is the macro which erases all extraneous data from
(DOWN)     | the spreadsheet.   the cell pointer moves down ten lines
(DOWN)     | beyond the failure point and then proceeds to erase.
(DOWN)     | once finished control reverts back to the calling macro
(DOWN)
(DOWN)
(DOWN)
(DOWN)
(DOWN)
(DOWN)
(DOWN)
(DOWN)
(left)
(left)
/re
.(right)
(right)
(right)
(right)
(right)
(right)
(right)
(right)
(right)
(end)
(down)~
/xr
```

TIME MACRO

```
/xiTM>99999¬/xgTIME2¬
{goto}TM¬
{edit}{home}'
{right}:
{right}
{right}:¬
{down}
/rncTEST3¬¬
/xiTEST3=0¬/xgFAIL¬
/rndTEST3¬
/xgLOOP2¬
{left}
```

this macro converts the time from a string of numbers into a readable form eg:123456 becomes 12:34:56 this macro works on numbers smaller than 99999

TIME2 MACRO

```
{goto}TM¬
{edit}{home}'
{right}
{right}:
{right}'
{right}:¬
{down}
/rncTEST3¬¬
/xiTEST3=0¬/xgFAIL¬
/rndTEST3¬
/xgLOOP3¬
```

this macro converts number strings into readable times. this macro works on times less than 9:59:59

PRINT MACRO

```
/cIDA¬IDAA¬
/pfUSR\GARYV\TENSILE\PRW
H12
¬
rA1..H244¬
gcrq
{goto}A245¬
/cIDA¬IDAB¬
/pfPSI
H12
¬
r.
{end}{down}
{end}{down}
{right}
{right}
{right}
{right}
{right}
{right}
{right}
{right}
{right}¬
gcrq
/cIDA¬IDAC¬
/cIDA¬IDAD¬
/pfUSR\GARYV\TENSILE\PSUM
H12
¬
rSUMMARY¬
gcrq
/xr
```

this macro creates the print files into which go the SI table of data as well as SUMMARY of data.

APPENDIX D

REPEATED LOADING TRIAXIAL TEST APPARATUS, PROCEDURES AND COMPUTER PROGRAMS

APPENDIX D

REPEATED LOADING TRIAXIAL TEST APPARATUS, PROCEDURES AND COMPUTER PROGRAMS

D.1 Introduction

One of the major objectives of the testing program undertaken in this investigation was to determine the permanent deformation characteristics and resilient modulus of the conventional and recycled asphalt concrete pavements. A repeated loading triaxial test apparatus together with a dynamic recording system were needed to fulfill this purpose.

This appendix describes the testing apparatus and the recording system as well as the computer programs for the recording and processing of the data. The testing procedures are included here.

D.2 Repeated Loading Triaxial Test Apparatus

D.2.1 Description of the Test Apparatus

The layout for the repeated loading triaxial apparatus is shown in Figure D1. The essential features and functions of the various components are described below. Corresponding reference letters are included on Figure D.1.

A <u>Counterbalance for the Loading Yoke</u> This unit consists of a weight and pulley system used to counterbalance the weight of the loading yoke on the piston and ram.

B <u>Load Cell</u> This device is used to measure the axial stress applied to the specimen. It has an operating range of 0 to 910 kg.

C <u>(LVDT) Linear Variable Differential Transducer</u> This is an electrical displacement transducer which measures the specimens axial deformation. The LVDTs used were Hewlett Packard displacement transducers, model 24DCDT-500 with a displacement range of ±12.7 mm in full scale.

D <u>Triaxial Cell</u> The triaxial cell used in this experiment was of a standard size for the testing of a 102 mm diameter by 204 mm high cylindrical specimen. The central area of the base of the triaxial cell is supported by the testing machine frame. This configuration allows the loading yoke to pass by the cell without obstruction. The clear removable cylinder is made of 9.5 mm thick perspex. The

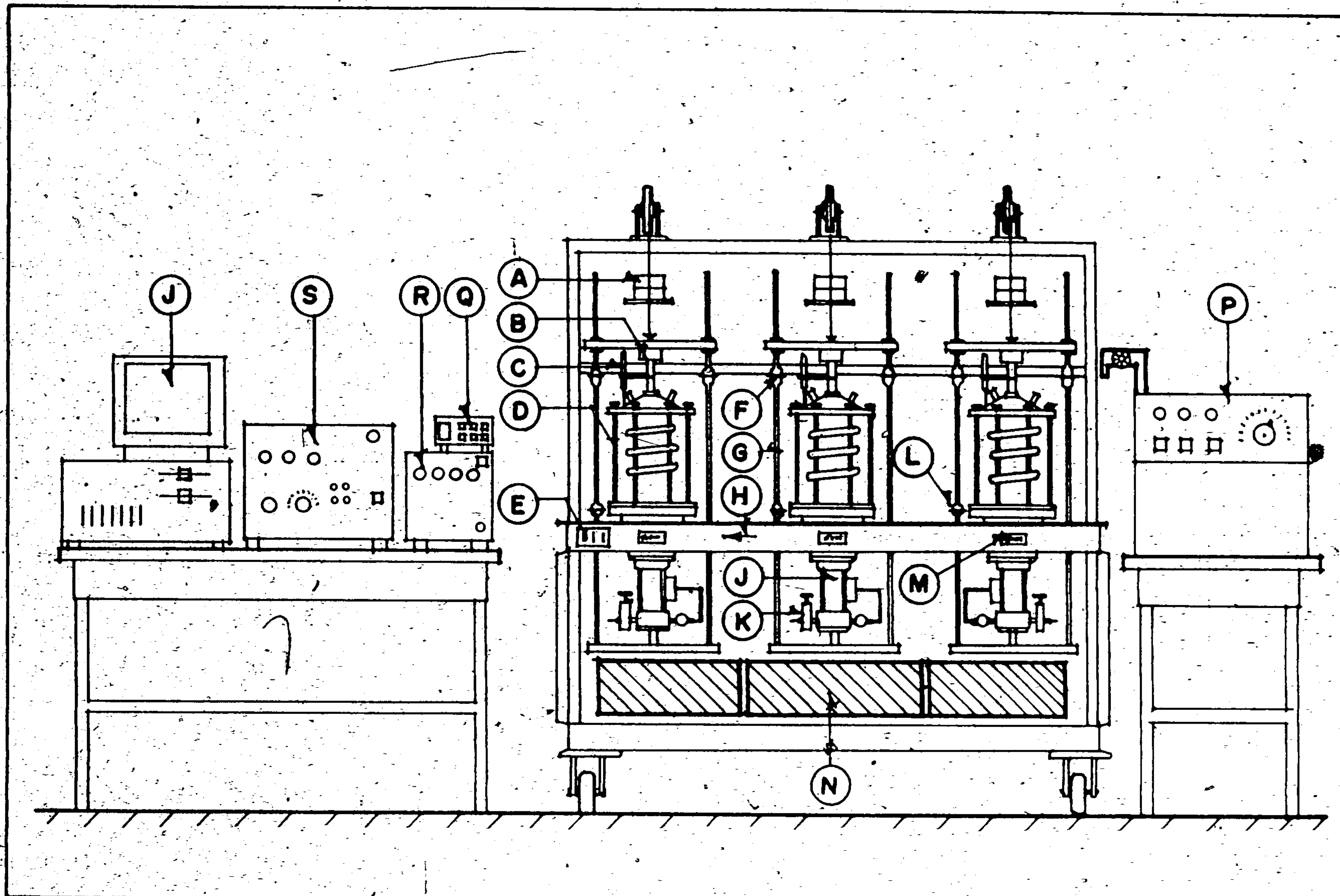

FIGURE D.1 SCHEMATIC OF THE REPEATED LOADING TRIAXIAL TEST EQUIPMENT.

loading ram is a 19 mm diameter ground stainless steel rod with a hemispherical end designed to fit into the depression in the loading cap. The top cap of the cell contains entrance and exit spouts for the temperature control lines which continue as copper coils within the cell body. The top cap also contains a threaded hole for the seating of the LVDT core. The base of the triaxial cell contains valves for the connection of the confining pressure as well as the pressure transducer line. The axial load capacity of the loading ram is 2,700 kg and the pressure capacity of the triaxial cell is 1000 kPa. The detailed description of the triaxial cell is given in Section D.2.2.

E <u>Bay Operation Switches</u> These switches supply the on-off control for the operation of their respective bays.

F <u>Guide Frame for the Loading Yoke</u> This unit is designed to restrict the horizontal movement of the loading yoke. The apparatus consists of a slotted plate bolted to the frame and a smooth sided annular cylinder which threads onto the loading yoke rods. The annular cylinder is positioned such that it fits within the slot in the plate ensuring that the load is applied vertically to the load cell.

G <u>Loading Yoke</u> This is a rectangular frame which transfers the load to the loading ram of the triaxial cell from the piston at the bottom of the frame through the load cell located at the top of the frame.

H Pressure Transducer This is a gauge used to measure the confining pressure in the three triaxial cells. In order to measure the confining pressure in each of the three triaxial cells, a four way valve is used. The transducer is connected to one of the arms while the other three arms are connected to the triaxial cells. By using a three-way valve switch, the pressure from two triaxial cells in shut off and the pressure from the third triaxial cell is measured by the transducer. The pressure transducer has an operating range of 0 to 2070 kPa.

J Air Pressure Cylinder and Piston This unit transforms air pressure from the reservoir tank into mechanical energy which is then transferred to the loading yoke. The model used was a Hannifin series 2A, style HB (NFPA style MF6) with a 102 mm diameter bone and LB + stroke equal to 123.9 mm and the rod and thread - KK, style 4 and 9 (19.1 mm-16).

K Air Pressure Regulators These units are installed at various tappings on the air pressure reservoir tank to regulate the air pressure to the triaxial cells. The operating range of the regulators is 14 to 1000 KPa.

L Safety Microswitches These units are designed such that when a pre-set maximum axial deformation is reached a steel bar will come in contact with a microswitch, shutting down the bay operation. This system avoids damage to the test apparatus in the event of sample failure. The switches are installed in each bay on the frame close to the loading yoke. A steel bar is mounted between two nuts on the threaded loading yoke. The location of the bar can be adjusted by rotating the nuts to obtain the correct gap width between the microswitch and the contact bar.

M <u>Counter</u> This is a six digit electric counter connected to the solenoid valve in order to record the number of load applications to the specimen.

N <u>Air Pressure Reservoir Tank</u> This is a cylindrical tank for the storage of compressed air. It has sufficient capacity to provide the pressure requirements for the three triaxial cells. The maximum pressure capacity of the tank is 850 kPa.

P <u>Temperature Control Bath</u> This unit is used to provide the required test temperature inside the three triaxial cells. By circulating the pre-set temperature controlled fluid through the heating coils within the triaxial cell, the temperature of the specimen can be kept at a desired constant test temperature throughout the experiment.

Q <u>Voltmeter</u> This unit reads the voltage in millivolts from the pressure transducer. It is calibrated for the pressure range of the transducer, making it possible to read the confining pressures. The model used is a Fluke 8050A digital multimeter.

R <u>Electrical Timing Unit</u> This unit consists of an electrical network of resistors, capacitors and inductance coils, which act to form a pulsating signal. By adjusting the properties of the components in the network, the frequency and duration of loading can be controlled. For this unit the minimum on-load time is 0.25 seconds.

S <u>Signal Conditioner</u> The signal conditioner is used to amplify and condition the signals from the load cells, LVDTs and pressure transducer. An Exp-16 board which acts as a multiplexer is built into the signal conditioner. This card collects the signals from the load cells and amplifies the required signals, which are then sent to the Dash-8 card installed inside the computer. The Dash-8 card transforms the signals from the analog format into digital format. The LVDTs and pressure transducer signals are also amplified inside the signal conditioner and sent to the Dash-8 card. They, however, do not pass through the Exp-16 card.

T <u>Computer and Data Logger</u> The computer collects data from the LVDTs and load cells from each of the triaxial cells. The computer used is an OPPERAND with 640 K of RAM and two disc drives. The operating system used was DOS version 3.1. The data logging card, Dash-8 is installed in the computer. This card collects and converts the output voltages from LVDTs and load cells into "bits" for storage in the computer. Computer programs were written using "BASICA" to initialize the computer and collect the data.

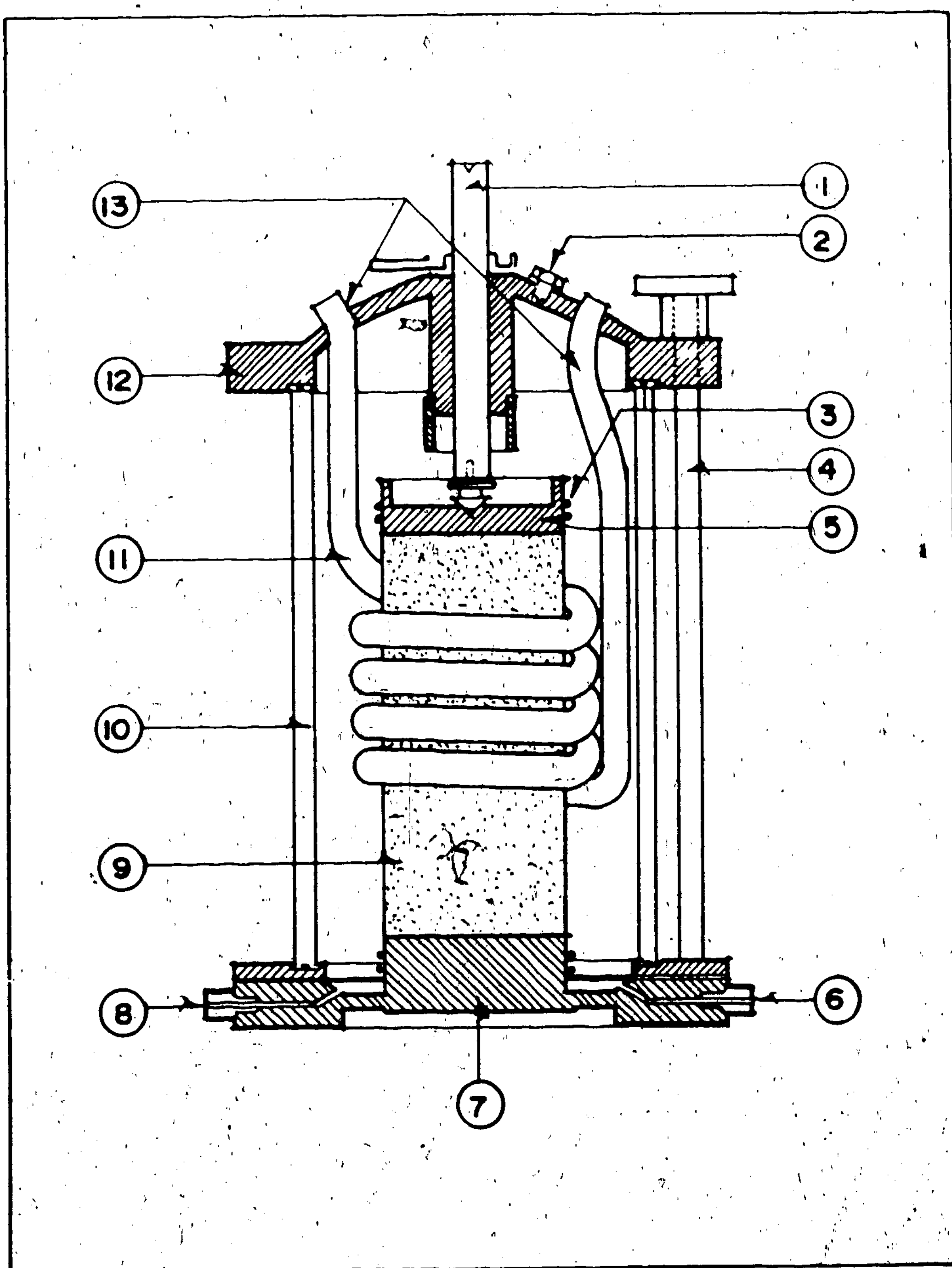

FIGURE D.2 PRINCIPAL FEATURES OF THE TRIAXIAL CELL.

Solenoid Valve (Not shown in the figure) This valve controls the flow of pressurized air from the air-pressure reservoir to the air-pressure cylinder and piston unit. The unit also provides the exhaust vent for air from the pressure cylinder and piston unit. The unit is controlled by the electric timing unit which electrically opens and closes a three-way solenoid operated valve. The units used are Hannyin - model CCJ 1-37 with a range of 0 to 830 kPa.

Air Pressure Gauges (Not shown in the figure) These gauges are installed after the air pressure regulators and indicate the air pressure being supplied by that particular line. The operating range of the gauges used are 0 to 415 kPa for the confining pressures and 0 to 1100 kPa for the axial pressure.

D.2.2 Description of the Triaxial Cell

Figure D.2 shows the principal features of the triaxial cell with a typical specimen set up within it. Refer to the figure for the corresponding numbers as described below.

1 Loading Ram This is a 19 mm diameter ground stainless steel rod.

2. Air Release Valve This valve releases the air pressure from the triaxial cell.

3 Rubber O-Rings These rings are used to seal the specimen within the rubber membrane and to seal the perspex cylinder to the top and bottom of the triaxial cell.

4 Mounting Screws Six equally spaced 12.7 mm diameter rods are used to seal the triaxial cell.

5 Loading Cap This 102 mm diameter x 16 mm thick metallic disc has a depression in its top surface which mates with the hemispherical end of the loading ram. The plate evenly distributes the load over the specimen's entire surface area.

6 Pressure Transducer Connection This valve connects a line to the pressure transducer for the measurement of the confining pressure inside the triaxial cell.

7 Triaxial Cell Base The base of the cell contains valves for the connection of the confining pressure lines and the exit line to the pressure transducer.

8 Confining Pressure Connection This valve supplies the confining pressure to the triaxial cell.

9 Specimen The cell used in this experiment will accommodate a 102 mm diameter by 204 mm high cylindrical specimen. The specimen is enclosed in a rubber membrane.

10 Perspex Cylinder This cylinder is 289 mm high with a 197 mm outer diameter and a 178 mm inner diameter.

11 Temperature Conditioning Coils These coils are used to evenly impart the preset test temperature to the water in the triaxial cell.

12 Top Cap of the Triaxial Cell The top cap contains the entrance and exit spouts for the temperature control lines, the air release valve, the entrance for the loading ram as well as a threaded hole for the seating of the LVDT core.

13 Temperature Control Lines These lines provide the entrance and exit for the controlled temperature fluid which circulates to and from the temperature control bath.

D.3. Operational Procedures

D.3.1 Sample Preparation

Level the top and bottom of the 102 mm diameter by 204 mm high specimen, ensuring that the surfaces and edges are free from protruding pieces of aggregate.

If the specimen's surface cannot be levelled it can be sawed to produce an even surface.

D.3.2 Setting up the Sample in the Triaxial Cell

1. Place the specimen on the 102 mm diameter base platten of the triaxial cell.
2. Enclose the specimen and base platten with a 102 mm diameter rubber membrane.
3. Place the loading cap on top of the sample and enclose it with the rubber membrane.
4. Install rubber O-rings over the membrane at the base platten and the loading cap, fold the excess

membrane material over the O-rings.

5. Align the specimen for verticality with the base platten and the loading cap.
6. Mount the top of the triaxial cell over the centered specimen. The loading ram should fit smoothly into the depression in the top of the loading cap. If a rotation of the loading ram causes lateral movement in the loading cap, there will be eccentricity in the positioning of the sample. In this case remove the top of the triaxial cell and realign the sample. Repeat this procedure until there is no eccentricity present.
7. Tighten the mounting screws, sealing the top of the triaxial cell to the base. Applying high vacuum grease to the surface of the loading ram was found to be helpful in stopping leaks from the rubber gaskets and the loading ram channel.

D.3.3 Preparation for Testing

1. Insert the triaxial cell into loading bay 1, ensure that the specimen is not disturbed from its aligned position. Adjust the loading yolk so that the calibrated load cell sits properly on the loading ram.
2. Close all connection valves to the test cell.
3. Fill the air-pressure reservoir tank with compressed air.

4. Connect and start the heat circulation unit which has been previously set at the test temperature.
5. Fill the cell by pumping distilled water, of the proper test temperature, through the confining pressure valve.
6. Connect the pressure transducer and confining pressure lines to the triaxial cell.
7. Switch on the (a) electric timing unit, (b) signal conditioning unit, (c) computer, and (d) volt meter.
8. Set the electrical timing unit at the desired on-load and off-load times.
9. Mount the LVDT core in the threaded hole in the top cap of the triaxial cell.
10. Insert the LVDT unit onto the core and lightly clamp the unit to the clamp bar attached to the loading ram of the triaxial cell.
11. Balance the calibrated LVDT with the aid of the computer program and then firmly clamp it into place.
12. Place a "U" shaped bar across the top of the triaxial cell, this will act as a dummy sample during the adjustment of the axial load.
13. Using the axial air pressure regulator, set the required axial pressure on the "U" shaped bar, fine tune the setting with the aid of the computer.
14. Open the pressure transducer and confining pressure valves.

15. Using the lateral air pressure regulator, set the confining pressure to the required valve, fine tune the adjustment using the voltmeter.
16. Remove the "U" shaped bar from the top of the triaxial cell.
17. Set counter to zero.
18. Adjust the guides on the loading yoke by turning them on the threaded rod so that they come within the guide frame.
19. Determine the anticipated maximum deflection the sample is likely to undergo.
20. Adjust the gap between the safety microswitch and the steel bar according to the anticipated maximum sample deflection.
21. Bay 1 is ready for testing.
22. Set up bay 2 and bay 3 by repeating the same procedure undertaken for bay 1.
23. The system is now ready for operation.
24. Initialize the computer.
25. Give proper name to the current test.
26. Start the test on bay 1 by turning on the bay 1 operation switch and F1 button on the computer keyboard simultaneously.
27. After bay 1 has completed the first 1000 cycles, start the test on bay 2 by turning on simultaneously the bay 2 operating switch and F2 button on the computer keyboard.

28. Likewise, when bay 2 has completed 1000 cycles, switch on the bay 3 operating switch and F3 button to start the test on bay 3.

D.4. Data Acquisition System

A data acquisition system was used to collect data from the triaxial test. The raw data which was first recorded by computer using several programs were then processed and percent permanent strain and resilient modulus were computed. Following is a description of the hardware and software used for the data logging system including programs for collecting and processing the data.

D.4.1 Computer Hardware Used in the Data Acquisition System

The computer that was used to gather the test data was an IBM clone that was distributed by Operand Electronics. The computer contained a number of features beyond the normal base system. The computer hardware consisted of the following:

(a) 640 K RAM The absolute minimum amount of RAM that was required by the software was 512 K but it was preferrable to use 640 K in order to provide a margin of safety for the computer's operation. 640 K is the maximum amount of RAM that can be accessed by current versions of DOS.

(b) Disk Drives Two disk drives were required in order to run the software. The first or "A" drive contained the operating system and the Basic program. The second or "B" drive was the drive to which the data would be sent upon completion of the test. Saving the data on a separate drive prevented the loss of any of the computer programs due to overwriting by test data. Before the data was written to a floppy it was stored in a RAM disk drive as explained later in the software section.

(c) Multifunction Card The multifunction card was used in place of a parallel port because the card contains both a serial and parallel port which were used for printer communications. This card also has a clock on board.

D.4.2 Data Acquisition Hardware

There were two pieces of equipment that actually formed the repeated loading triaxial test data acquisition hardware system:

(a) MetraByte EXP-16 Board The EXP-16 board was used in conjunction with the DASH-8 card in order to accommodate all of the data inputs from the test. The EXP-16 board can accommodate up to 16 channels of analog information. The board amplifies, filters and conditions the input signals from the test. The EXP-16 board multiplexs the signals into one channel that is fed into the DASH-8 board. This board was installed in the signal conditioner and was connected to the DASH-8 board in the computer by means of a ribbon cable. For further information regarding the EXP-16 board refer to the manual as published by the manufacturer, MetraByte Corp.

(b) Dash-8 Board The Dash-8 board was used to collect the data in analog form and convert it into digital form for use by the computer. The data was passed from the EXP-16 board to the DASH-8 board. The EXP-16 board collected the data because it had 16 channels available for data acquisition as opposed to the 8 channels available on the DASH-8 board. The DASH-8 board has a full scale input of +/- 5 volts on each channel with a resolution of 2.44 millivolts (0.00244 volts).

The DASH-8 board comes with a preprogrammed software package that allows the user to configure the board to his own requirements. This software plus additional technical information can be found in the DASH-8 manual that is published by the manufacturer, MetraByte Corp.

D.4.3 Data Acquisition Software

There were two prepackaged software packages used by the computer to record the test data. One was DOS version 3.1 and the other was the DASH-8 configuration package. There was also a program written in BASIC specifically for use in the repeated loading triaxial test. The detailed description of each program is given below:

(a) DOS 3.1 DOS 3.1 was used because it was the most up to date operating system available and it allowed the establishment of a "virtual disk" on the computer. When the computer "booted up" there was a command in the "config.sys" file that created a 200 K virtual disk in RAM. A virtual disk was used for temporary data storage. The virtual disk has a very quick access time so that it is possible for the computer to store the gathered information on this "disk". When the test was finished the contents of the virtual disk was automatically transferred to the diskette in drive"B".

(b) DASH-8 Software The DASH-8 board was supplied with a software package that provides the I/O driver routine. THis routine is called "DASH8.BIN" and can be accessed from BASIC using call statements. Using the DASH8.BIN software in conjunction with a BASIC program provided considerable flexibility in the use of the computer hardware. Various functions and operations were defined from within the BASIC program by using the DASH8.BIN file. The DASH-8 operations manual can be used for further reference.

(c) Triaxial Test Control and Datalogging Program A BASIC program was written to collect the test data from the repeated loading triaxial test and store it in the computer. This program made use of the DASH-8 software and therefore any attempt to analyze the program should be done with the DASH-8 manual as a reference.

This program uses the timer built into the DASH-8 board and therefore does not require any internal clock in the computer. The timer controls the data acquisition portion of the board so that there are two hundred measurements made per second. The computer takes the maximum and minimum values. The board timer also sends one pulse per second to the external timer. This external timing unit controls the operation of the air rams that load the specimens.

The limitations due to the hardware allowed only one reading per second to be recorded. Because only one reading per second could be recorded and there were three samples being tested the start-ups were staggered. Test one would commence and readings would be taken every second for the first 1000 seconds. Upon completion of the first 1000 seconds test two would begin and its first 1000 readings would be recorded and test three would start after test two completed the first one thousand cycles.

The information was recorded as follows: every second for the first 1000 seconds, every 100 seconds up until 10,000 seconds and then every 1000 seconds until the end of the test.

The BASIC program written for the repeated loading triaxial test is illustrated in Section D.4.4.1. It contains all the information and comments regarding the test which makes it self-explanatory.

(d) <u>Programs for Processing the Raw Data</u> The data which have been collected by computer were in a binary form. Computer programs were necessary to convert the collected data from the binary formats into permanent deformation and resilient modulus values by introducing the various parameters and necessary calculations.

The percent permanent strain was computed as follows:

$$\varepsilon_p = \frac{100\ N_{min} K_p}{410t}$$

where

ε_p = percent permanent strain,

N_{min} = deformation values in binary bit as recorded by computer (minimum values at each pulse), 1 volt = 410 bits,

k_p = calibration factor for LVDT, mm/volt, and

t = thickness of the specimen, mm.

The resilient modulus was computed as follows:

$$M_R = \frac{\sigma_d}{\frac{(N_{max} - N_{min})\ k_R}{410t}}$$

where

M_R = resilient modulus in kPa,

σ_d = repeated axial deviator stress, kPa,

$N_{max} - N_{min}$ = recoverable deformation values in binary bits (Maximum value - Minimum value, at each pulse), 1 volt = 410 bits;

k_R = calibration factor for LVDT, mm/volt, and

t = thickness of the specimen, mm.

A Lotus 1-2-3 Macro program was written specifically for converting the raw data from binary to real format and performing the percent permanent deformation and resilient modulus computations. This program is presented in Section D.4.4.2.

D.4.4 Listing of the Computer Programs for the Repeated Loading Triaxial Test

D.4.4.1 Triaxial Test Control and Datalogging Program

```
50 '*************************************************************
60 '*                 Repeated Loading Triaxial Test            *
70 '*                 Control and Datalogging System            *
80 '*************************************************************
100 'def seg=&h1700 'Setup load segment
110 'bload "dash8.bin",0 'Loads at 1700:0000
111 '------------------------------------------------------------
112 ' above requried for interpertive version
113 '------------------------------------------------------------
115 '
116 '     Load dash8 address
117 '............................................................
120 OPEN "dash8.adr" FOR INPUT AS #1
130 INPUT #1, BASADR%
135 CLOSE #1
136 '............................................................
137 '      Setup variables
138 'the number of reading are modified using max and diff
139 '............................................................
150 MAX1=1000: DIFF1=100: MAX2=1000: DIFF2=100: MAX3=1000: DIFF3=100
151 DIM DAH%(2),DAL%(2),LCH%(2),LCL%(2),DA%(199),DIO%(2),LT%(2),FRAN%(2)
152 '          Definations
153 '     DAH=Maxium load cell reading DAL=low load cell reading DA=readings
154 '     LCH=Maxium lvdt reading      LCL=low lvdt reading
155 '............................................................
156 '     Initialize dash8 with mode 0
157 '............................................................
160 DASH8=0 : FLAG%=0 : MD%=0
170 CALL DASH8 (MD%, BASADR%, FLAG%)
175 '............................................................
180 '............................................................
185 '     Setup of function keys
186 '     ---------------------
187 '
188 '       Function Key#             Desription
189 '
190 '            1              start test sample #1
191 '            2              stop test sample #1(cursor down at same time)
192 '            3              start test sample #2
193 '            4              stop test sample #2(cursor down at same time)
194 '            5              start test sample #3
195 '            6              stop test sample #3(cursor down at same time)
196 '            10             stops program
197 KEY(1) ON : KEY(2) ON : KEY (3) ON: KEY (4) ON : KEY (5) ON
198 KEY(6) ON : KEY(14) ON : KEY (10) ON
199 '............................................................
200 '      Setup files for data to be sent to c:drive
201 '............................................................
201 INPUT "enter file name";F$;FF0$="c:"+F$+"0.prn":FF1$="c:"+F$+"1.prn"
202 FF2$="c:"+F$+"2.prn"
203 OPEN FF0$ FOR OUTPUT AS 3 :OPEN FF1$ FOR OUTPUT AS 1
204 OPEN FF2$ FOR OUTPUT AS 2
205 '............................................................
206 '      Subroutine pointers for defined keys
207 '............................................................
210 ON KEY(1) GOSUB 400  '  function key 1
215 ON KEY(2) GOSUB 500  '               2
220 ON KEY(3) GOSUB 600  '               3
225 ON KEY(4) GOSUB 700  '               4
230 ON KEY(5) GOSUB 800  '               5
```

```
233 ON KEY(6) GOSUB 850 '         6
234 ON KEY(14) GOSUB 550 ' down cursor
235 ON KEY(11) GOSUB 360 ' up cursor
236 ON KEY(10) GOSUB 450 ' function key 10
237 '..........................................
238 '      Set dash8 time/counter configuation use mode 10
239 '..........................................
240 MD%=10 : DIO%(0)=2 : DIO%(1)=3 ' use counter 2
250 CALL DASH8 (MD%, DIO%(0), FLAG%) 'configuation 3(square wave generator)
251 '..........................................
252 '      Load the timer/counter use mode 11
253 '..........................................
260 MD%=11 : DIO%(0)=2 : DIO%(1)=23868/4 ' 23868/4=5967  one period= 4190 us
270 CALL DASH8 (MD%, DIO%(0), FLAG%) ' 5967*.4190=2500 us or 2.5 msec
271 '..........................................
272 '      Enable/disable data channel tag use mode 17
273 '..........................................
280 MD%=17 : EN%=0 '0=off , 1=on
290 CALL DASH8 (MD%, EN%, FLAG%)
291 '..........................................
300 GOSUB 1000 ' goto title page
301 '..........................................
302 '      Wait for function keys to start tests
303 '..........................................
305 IF S1=1 OR S2=1 OR S3=1 THEN GOTO 900
310 GOTO 305
315 '..........................................
316 '      Restart test
317 '..........................................
360 IF STP1=1 THEN S1=0:SR1=0:STT1=0:ST1=0:NT1=0:RETURN 'restart #1
370 IF STP2=1 THEN S2=0:SR2=0:STT2=0:ST2=0:NT2=0:RETURN 'restart #2
380 IF STP3=1 THEN S3=0:SR3=0:STT3=0:ST3=0:NT3=0:RETURN 'restart #3
385 '..........................................
386 '      Subroutine-Start test #1
400 S1=1 : SR1=0 :RETURN
450 STOP
475 '..........................................
476 '      Subroutine to stop or restart test #1
477 '..........................................
500 STP1=1 : KEY (14) ON:KEY (11) ON: KEY (14) STOP:KEY (11) STOP:STP1=0:RETURN
525 '..........................................
526 '      Subruotine-Stop tests
527 '..........................................
550 IF STP1=1 THEN S1=0 : SR1=0 : RETURN 'stop test #1
560 IF STP2=1 THEN S2=0 : SR2=0 : RETURN 'stop test #2
570 IF STP3=1 THEN S3=0 : SR3=0 : RETURN 'stop test #3
580 RETURN
585 '..........................................
586 '      Subroutine-Start test #2
587 '..........................................
600 S2=1 : SR2=0 : RETURN
605 '..........................................
606 '      Subroutine-stop or restart test #2
607 '..........................................
700 STP2=1 : KEY (14) ON:KEY (11) ON:KEY (14) STOP:KEY (11) STOP:STP2=0 :RETURN
705 '..........................................
706 '      Subroutine-Start test #3
707 '..........................................
800 S3=1 : SR3=0 : RETURN
805 '..........................................
807 '      Subroutine-stop or restart test #3
808 '..........................................
850 STP3=1 : KEY (14) ON:KEY (11) ON:KEY (14) STOP:KEY (11) STOP:STP3=0 :RETURN
852 '..........................................
875 '      Sync with control pulse supplied externally
876 '==========================================================================
900 GOSUB 1500 'input i/o
910 IF IP% AND 1 THEN GOTO 900 'is bit 1 high?
920 GOSUB 1500 'input i/o
930 IF IP% AND 1 THEN GOTO 970 'is bit 1 high
940 GOTO 920
970 GOSUB 1500:IF IP% AND 1 THEN GOTO 970 'check again
```

```
975 GOTO 3000 'goto control routine
980 '====================================================================
1000 RETURN 'reserved for title page
1100 '.....................................
1110 '      Subroutine-input dash8 i/o with mode 13
1120 '.....................................
1500 MD%=13
1510 CALL DASH8 (MD%, IP%, FLAG%):RETURN
1520 '.....................................
1900 '====================================================================
1910 '                  Dash8 datalogging routine
1920 '                  -------------------------
1930 '.....................................
1940 '     Determine which test to sample
1950 '.....................................
2000 IF SR1=1 THEN SR1=0 : STT1=STT1+1 : R=STT1 : T=0 : GOTO 2050 'do #1
2010 IF SR2=1 THEN SR2=0 : STT2=STT2+1 : R=STT2 : T=1 : GOTO 2100 'do #2
2020 IF SR3=1 THEN SR3=0 : STT3=STT3+1 : R=STT3 : T=2 : GOTO 2200 'do #3
2040 NR=1:GOTO 2060 'do none
2045 '.....................................
2046 '.....................................
2047 '     set exp16 channel no. and output
2048 '.....................................
2050 MD%=14 : OP%=0 :C1=STT1:C2=ST1:TO=3:NR=0
2060 CALL DASH8 (MD%, OP%, FLAG%)
2065 '.....................................
2066 '      Set first and last channel use mode 1
2067 '.....................................
2070 MD%=1 : LT%(0)=0 : LT%(1)=3
2080 CALL DASH8 (MD%, LT%(0), FLAG%)
2081 '.....................................
2082 '      log data use mode 5
2083 '.....................................
2085 C=0
2090 MD%=5 : TRAN%(0)=VARPTR(DA%(C)): TRAN%(1)=199
2095 CALL DASH8 (MD%, TRAN%(0), FLAG%):IF NR=1 THEN FOR NI=0 TO 100:NEXT NI:GOTO 970
2096 C=0 : STA=8 : FIN=196 : GOSUB 4500 'get max and min
2097 HTEMP1=HTEMP : LTEMP1=LTEMP 'store max and min
2098 C=T+1 : STA=T+9 : FIN=T+197 : GOSUB 4500
2099 PRINT #TO,C1 C2 HTEMP LTEMP HTEMP1 LTEMP1:GOTO 970 'output to data file
2100 MD%=14 : OP%=1 :C1=STT2:C2=ST2:TO=1:NR=0: GOTO 2060 'set exp16
2200 MD%=14 : OP%=2:C1=STT3:C2=ST3:TO=2:NR=0: GOTO 2060 'set exp16
2300 '====================================================================
2310 '                    control routines
2320 '                    ----------------
3000 PRINT ST1 STT1 "   " ST2 STT2 "   "ST3 STT3"  "HTEMP"  "LTEMP" "HTEMP1"  "LTEMP1
3001 '+++++++ turn function keys on ++++++++
3002 KEY(1) ON : KEY(2) ON : KEY (3) ON: KEY (4) ON:KEY (5) ON:KEY (6) ON
3003 IF DON1=1 AND DON2=1 AND DON3=1 GOTO 4000 'is test done
3004 KEY (1) STOP:KEY (2) STOP:KEY (3) STOP:KEY (4) STOP:KEY (5) STOP:KEY (6) STOP
3005 IF S1=1 THEN ST1=ST1+1 : NT1=NT1+1 'test 1 on?,set conditions
3006 IF NT1=DIFF1 THEN SRS1=1 : NT1=0 'diff1 reached,do reading on test 1
3010 IF S2=1 THEN ST2=ST2+1 : NT2=NT2+1 'test 2 on?,set conditions
3011 IF NT2=DIFF2 THEN SRS2=1 : NT2=0 'diff2 reached,do reading on test 2
3020 IF S3=1 THEN ST3=ST3+1 : NT3=NT3+1 'test 3 on?,set conditions
3021 IF NT3=DIFF3 THEN SRS3=1 : NT3=0 'diff reached,do reading on test 3
3025 '.....................................
3026 '      Do 100 readings on test 1 if on
3027 '.....................................
3030 IF ST1>99 THEN GOTO 3200 'goto test 2 if more then 100 readings
3040 IF S1=0 THEN GOTO 3200 '0=off,1=on,is test 1 on?
3050 SR1=1 'do a reading
3060 GOTO 2000 'goto dataloging routine
3065 '.....................................
3070 '      do 100 readins on test 2 if on
3075 '.....................................
3200 IF ST2>99 THEN GOTO 3300 'goto test 3 if more then 100 readings
3205 IF SRS1=1 THEN SR1=1:SRS1=0:GOTO 2000 'if on read test 1
3210 IF S2=0 THEN GOTO 3300 'is test 2 on?
3220 SR2=1 'do a reading on test 2
3230 GOTO 2000 'go to datalogging routine
```

```
3235 '..........................................
3236 '         Do 100 readings on test 3 if on
3237 '..........................................
3300 IF ST3>99 THEN GOTO 3400 'goto adjustments of control program
3305 IF SRS1=1 THEN SR1=1:SRS1=0:GOTO 2000 ' if on read test 1
3307 IF SRS2=1 THEN SR2=1:SRS2=0:GOTO 2000 ' if on read test 2
3310 IF S3=0 THEN GOTO 3400 ' is test 3 on
3320 SR3=1 ' do a reading on test 3
3330 GOTO 2000 ' go to datalogging routine
3335 '..........................................
3336 '     Counter adjustment routines
3337 '..........................................
3338 '              counter 1
3339 '..........................................
3400 IF S1=0 THEN GOTO 3600 'skip if off
3410 IF ST1=MAX1 THEN GOTO 3500 'check for max total readings
3420 IF SRS1=1 THEN SR1=1 : SRS1=0 : GOTO 2000 'go to datalog routine
3430 GOTO 3600 'goto test 2 counter
3500 IF ST1=100000! THEN DON1=1:CLOSE 3 :S1=0: GOTO 2000 'max reached close file
3505 NT1=0
3510 MAX1=MAX1*10 : DIFF1=DIFF1*10 : GOTO 3410 'reset max and diff
3575 '              counter 2
3576 '..........................................
3600 IF S2=0 THEN GOTO 3800 'skip if off
3610 IF ST2=MAX2 THEN GOTO 3640 'check for max readings
3620 IF SRS2=1 THEN SR2=1 : SRS2=0 : GOTO 2000 'goto datalog routine
3630 GOTO 3800 'goto test 3 counter
3640 IF ST2=100000! THEN DON2=1:CLOSE 1:S2=0:GOTO 2000 'max reached close file
3700 NT2=0 : MAX2=MAX2*10 : DIFF2=DIFF2*10 : GOTO 3610 'reset max and diff
3750 '              counter 3
3755 '..........................................
3800 IF S3=0 THEN GOTO 2000 'skip if off
3810 IF ST3=MAX3 THEN GOTO 3840 'check for max readings
3820 IF SRS3=1 THEN SR3=1 : SRS3=0 : GOTO 2000 'goto datalog routine
3830 GOTO 2000
3840 IF ST3=100000! THEN DON3=1:CLOSE 2 :S3=0: GOTO 2000
3900 NT3=0 : MAX3=MAX3*10 : DIFF3=DIFF3*10 : GOTO 3810
3950 '..........................................
3955 '      return to system
3960 '..........................................
4000 PRINT "done":SYSTEM
4100 '..........................................
4400 '..........................................
4410 '        Subroutine-max and min
4500 IF DA%(C)>DA%(C+4) THEN HTEMP=DA%(C):LTEMP=DA%(C+4): GOTO 4520
4510 HTEMP=DA%(C+4) : LTEMP=DA%(C)
4520 FOR C=STA TO FIN
4530 IF DA%(C)>HTEMP THEN HTEMP=DA%(C) : GOTO 4550
4540 IF DA%(C)<LTEMP THEN LTEMP=DA%(C)
4550 C=C+3 : NEXT C :C=0 : RETURN
10000 STOP
```

D.4.4.2 Lotus 1-2-3 Macro Program for Triaxial Test Data Processing

The following is a listing of the LOTUS macro that controls the spreadsheet for the triaxial test data calculations. The macro does not appear in the spreadsheet in the same format as it is presented here. As well the comments found here are not found in the spreadsheet they are placed here to assist in future modifications of the spreadsheet.

INITIALIZING SUBROUTINE

```
/wgrm                      | this macro initializes        INITIAL   NUMBER
/reOLDRAW¬                 | the worksheet                              1
(goto)A11¬                 |                                  4
/finUSR\GARYV\TRIAX\       | the test data is retrieved
12APBCAO                   |
¬                          |
/dq{INPUT¬                 |
cCRITERION¬                |
ddq                        | the cell pointer is moved to the
(goto)A11¬                 | correct locatin as specified
/df(esc).                  |
(end)(down)¬               |
0¬¬                        |
/xgCALC¬                   |
```

CALCULATION SUBROUTINE (performs all necessary calculations)

```
(goto)BIT1¬                                                FACTOR   THICK
+0.00000001¬                                               2.8504   8.156
/cBIT1¬H11..J11¬
(goto)BIT2¬
(@ABS(D12-$D$11))¬
(goto)PERMDEF¬
((BIT2*2.44*100*$FACTOR)/(1000*$THICK*25.4))¬       | % permanet strain
(goto)DIFFER¬                                       | claculation
@ABS(C12-D12)¬
(goto)RESMOD¬
+(310.26405/((($H12-1)*2.44*$FACTOR)/(1000*$THICK*25.4)))¬   | resilient
/cG12..L12¬                                                   | modulus
G12..L1205¬                                                   | calculation
/cB9.B1205¬
I9¬
/rncTEST¬¬                 | this section determines where the
/rndTEST¬                  | pertinent data is and then passes control
(goto)A1203¬               | to the ERASE macro.
/rncTEST¬¬                 |
/xiTEST<>0¬/xgERASE¬       |
/rndTEST¬                  |
(up)                       |
/rncTEST¬¬                 |
/xgLOOP¬                   |
```

DATA INPUT SUBROUTINE

```
(goto)BEGIN¬     | this subroutine allows the file names,
/rndIDAS¬        | sample data, and starting points to be entered
/rndIDAF¬        | entered into the worksheet.
/rndTEMP¬        |
/rndTHIK¬        |
/rndSTINT¬       |
```

```
/rndFACT1¬
/rncIDAS¬¬
{goto}IDAS¬
/rndIDAS¬
{down}
/rncIDAS¬¬
{right}
/rncIDAF¬¬
{right}
/rncTEMP¬¬
{right}
/rncTHIK¬¬
{right}
/rncSTINT¬¬
{right}                      C8
/rncFACT1¬¬
{right}
/xiTHIK=0¬/xq¬
/cIDAS¬
SAMNO¬
/cIDAS¬
PRINTID¬
/cIDAF¬
FRET¬
/cIDAF¬
FILID¬
/cTEMP¬
SPTEMP¬
/cTHIK¬
THICK¬
/cTHIK¬
THIK1¬
/cSTINT¬
INITIAL¬
/cSTINT¬
INIT2¬
/cFACT1¬
FACTOR¬
/cFACT1¬
FACT2¬
/xcSTART¬
/rndIDAF¬
/rndTEMP¬
/rndTHIK¬
/rndSTINT¬
/rndFACT1¬
/xgLOOP4¬
```

PRINT SUBROUTINE

```
{goto}I1¬
/pfUSR\GARYV\TRIAX\
B4
¬
crr
{end}{down}
{end}{down}
{end}{down}
{end}{right}
{end}{right}¬
gq
/xr
```

this is the print macro which prints out all of the calculated data

ERASE

```
{down}
/re.
{right}
{right}
{right}
```

this is the erase macro which deletes all extraneous data

```
{right}
{right}
{right}
{right}
{right}
{right}
{right}
{right}
{right}
{end}
{down}~
/xgPRINT~
```

AUTOEXEC MACRO

```
{goto}a1500~        | this macro starts the menu macro when the wks
/xg\Q~              |  is called up.
```

MAIN MENU MACRO

```
/xmMMenu~                                     | this is the menu macro
Enter    Process  Quit                        | which has overall control
Enter or Begin proLeave LOTUS 1-2-3           | of the spreadsheet
/xc\W~   /xc\Z~    /fs~r/qy
```

```
{goto}E1308~        | this sets up titles for the data input
{down}              |
{down}
{down}
{down}
{down}
{down}
/wth
```

(BELOW IS AN EXAMPLE OF DATA THAT MUST BE INPUT BEFORE MACRO EXECUTION

USE THE CURSOR KEYS TO MOVE AND PRESS "Alt" "Q" TO RETURN TO MENU

SAMPLE NAME	FILE ID	TEMP DEG C	THICK (INCHES)	START	FACTOR
B4	12APBCAO	25	8.156	4	2.8504
B1S	26MABECO	25	8.182	15	3.022
E8S	26MABEC1	25	8.143	15	3.241
C1S	26MABEC2	25	8.137	15	2.7304
A5	10APAFDO	25	8.175	11	2.8504