Research Report

Guide to the Characteristics, Performance and Selection of Paving Asphalts

Transportation Association of Canada
Association des transports du Canada

The overall mission of the Transportation Association of Canada (TAC) is to promote the provision of safe, efficient, effective and environmentally sustainable transportation services in support of the nation's social and economic goals. To this end, TAC acts as a neutral forum for the discussion of transportation issues, serves as a technical focus in the field of roadway transportation, promotes R&D activities, and disseminates transportation related information published by TAC and others. The role of TAC's Research and Development Council is to foster innovative, efficient and effect research and technology transfer in support of Canadian transportation. This project was conducted as part of the Council's cooperative reserach program with funding provided by the federal, provincial and territorial ministries of transportation.

La mission de l'Association des transports du Canada (ATC) est de promouvoir la sécurité, l'efficience, l'efficacité et le respect de l'environnement dans la prestation de services de transport, en vue d'appuyer les objectifs sociaux et économiques du pays. À cette fin, l'ATC offre une tribune neutre pour la discussion des enjeux et des problèmes liés aux transports, sert de centre d'études techniques dans le domaine des transports routiers, encourage les activités de R-D et diffuse l'information sur le secteur les transports qu'elle-même et d'autres organismes réunissent. Le rôle du Conseil de la recherche et du développement de l'ATC est de contribuer à l'essor du secteur canadien des transports en favorisant la mise en oeuvre de projets de recherche innovateurs, efficients et efficaces ainsi que le transfert de la technologie issue de ces derniers. Le présent projet a été exécuté à la faveur de programme de recherche collaborative du Conseil, programme financé par les ministères fédéral, provinciaux et territoriaux des Transports.

2323 St. Laurent Blvd., Ottawa, Canada K1G 4K6
Tel. (613) 736-1350 Fax (613) 736-1395

ISBN 1-55187-094-0

TAC REPORT DOCUMENTATION FORM

Project No.	Report No.	Report Date	IRRD No.
1316		JAN 1996	

Project Manager

Christopher Hedges

Title and Subtitle

Guide to the Characteristics, Performance and Selection of Paving Asphalts

Author(s)

Cece Dawley
Bert Pulles

Corporate Affiliation(s)

EBA Engineering

Sponsoring/Funding Agency and Address

Research and Development Council
Transportation Association of Canada
2323 St. Laurent Blvd.
Ottawa, ON K1G 4K6

Performing Agencies Names and Addresses

EBA Engineering
14535 - 118 Avenue
Edmonton, AB T5L 2M7

Abstract

The purpose of this study was to document a set of procedures for the selection of paving asphalts based on the Canadian General Standards Board (CGSB) 1990 specification ***Asphalt cements for road purposes*** **(CAN/CGSB-16.3-M90)**. This specification was published in 1990 and was intended to serve as a national standard for asphalt cements. While the Strategic Highway Research Program has more recently produced new performance-based binder specifications, several Canadian jurisdictions have indicated their intention to continue using the CGSB penetration grades for some time into the future. A User Guide to accompany the CGSB specification was never published, although a number of draft versions were developed. It was recognized by TAC that the selection of asphalt paving cements on a project specific basis using the CGSB standard alone is not a straightforward procedure. Therefore, TAC has sponsored the development of a guide to the characteristics, performance and selection of paving asphalts based on the best information currently available on: 1) Canadian climatic conditions, 2) sources and characteristics of asphalts used in Canada; 3) Canadian pavement performance data; 4) current provincial asphalt specifications and test procedures; 5) the likely impacts of the results of the Strategic Highway Research Program asphalt research results; and 6) use of premium and polymer modified asphalts. The User Guide is intended to be used as a supplement to the CGSB standard and contains step by step procedures with worked examples for the selection of asphalts for any paving project in Canada based on climatic conditions and anticipated traffic levels. This report contains the User Guide as well as detailed documentation of the research that led to its development. The User Guide itself is also available from TAC as a separate document. To assist with the pavement selection process, a printout of minimum and maximum temperature data from Canadian weather stations has been extracted from the SHRPBIND weather database and is also available separately from TAC.

Keywords (IRRD)

Bituminous mixture
Design (overall design)
Selection
Specifications
Guidance
Traffic count
Weather
Durability
Properties
Canada

No. of Pages	No. of Figures	Language	Price
		English	

Supplementary Information

FICHE DE RAPPORT DE L'ATC

Projet n°	Rapport n°	Date du rapport	DIRR n°
1316		JAN 96	

Gestionnaire du projet

Christopher Hedges

Titre et sous-titre

Guide to the Characteristics, Performance and Selection of Paving Asphalts

Auteur(s)	Affiliation(s)
Cece Dawley Bert Pulles	EBA Engineering

Nom et adresse de l'organisme parrain	Nom et adresse de l'organisme exécutant
Conseil de la recherche et du développement Association des transports du Canada 2323, boulevard Saint-Laurent Ottawa (ON) K1G 4K6	EBA Engineering Consultants 14535 - 118 Avenue Edmonton, Alberta T5L 2M7

Résumé

La présente étude décrit les modalités de sélection d'un liant bitumineux en fonction de la norme de 1990 de l'Office des normes générales du Canada (CGSB, intitulée ***«Liants bitumineux pour les routes» (CAN/CGSB-16.3-M90).*** Cette norme, publiée en 1990, devait servir de norme nationale pour les liants bitumineux. Bien que le programme stratégique de recherche routière ait donné lieu plus récemment à une nouvelle norme sur les liants axée sur le rendement, plusieurs administrations canadiennes ont exprimé leur intention de continuer à utiliser les catégories de pénétration CGSB. Plusieurs versions provisoires d'un guide d'utilisation de la norme CGSB ont été préparées, mais aucune n'a été publiée. L'ATC a reconnu que la sélection d'un liant bitumineux en vue d'un projet particulier d'après la seule norme CGSB n'est pas une tâche simple. Par conséquent, l'ATC a fait préparer un guide des caractéristiques, du rendement et de la sélection des bitumes routiers en fonction des meilleures connaissances actuelles relatives 1) au climat canadien, 2) aux sources et aux caractéristiques des bitumes utilisés au Canada, 3) aux données canadiennes sur le rendement des revêtements, 4) aux normes et méthodes d'essai provinciales actuelles se rapportant aux bitumes, 5) à l'effet probable des résultats du programme stratégique de recherche routière se rapportant aux bitumes et 6) à l'utilisation des bitumes de qualité supérieure et des bitumes modifiés par des polymères. Le guide d'utilisation doit servir de complément de la norme CGSB et il décrit méthodiquement les étapes, avec exemples à l'appui, de la sélection des bitumes pour tout projet de revêtement exécuté au Canada, compte tenu des conditions climatiques et de la densité de la circulation. Le présent rapport contient le guide d'utilisation de même que la documentation détaillée qui en a permis la préparation. Le guide d'utilisation lui-même est un document distinct diffusé par l'ATC. Afin de faciliter la sélection des revêtements, on a extrait de la base SHRPBIND des données sur les températures minimales et maximales signalées par les stations météorologiques canadiennes, données qui sont également diffusées séparément par l'ATC.

Mots-clés

Enrobe
Dimensionnement
Choix
Recommandation
Guidage
Comptage (trafic)
Climat
Durabilité
Caracteristiques
Canada

Nombre de pages	Nombre de figures	Langue	Prix
		Anglais	

Renseignements supplémentaires

Acknowledgements

This project was conducted with funding provided by TAC's Research and Development Council.

TAC would like to express its appreciation to the members of the project steering committee who volunteered their time to provide advice, guidance and direction. TAC would also like to thank all those individuals and agencies who contributed to the success of the study by responding to survey questionnaires.

Project Steering Committee

David Haughton (Chairman)
B.C. Transportation & Highways

Imants Deme
Shell Canada Products Limited

Keith Foster
Nfld. Dept. of Works, Services & Transportation

Chuck Juergens
Husky Oil Marketing Company

Anne-Marie Leclerc
Ministère des Transports du Québec

Daryl Macleod
Saskatchewan Department of Highways and Transportation

Lyle Moran
Imperial Oil Ltd. Research Department

Dave Palsat
AGRA Earth & Environmental

Kai Tam
Ministry of Transportation, Ontario

Christopher Hedges (Project Manager)
Transportation Association of Canada

Principal Consultants

Cece Dawley
Bert Pulles

EBA Engineering Consultants Ltd.

In Memoriam

John Corkill
1930-1993

EXECUTIVE SUMMARY

In 1992, the Transportation Association of Canada (TAC) engaged EBA Engineering Consultants Ltd. (EBA) to undertake a study entitled "Characteristics, Performance and Selection of Paving Asphalts". The study focus was the existing Canadian General Standards Board specification **Asphalt Cements for Road Purposes (CAN/CGSB-16.3-M90)**.

The Study incorporated a survey of current Canadian agency practices that relate to methodology used to select asphalt binders used in the production of hot mix asphalt concrete, a literature review, a survey of asphalt suppliers and an assessment of the likely impact of the Strategic Highway Research Program (SHRP) research and specifications. The objectives of the Study were to:

i) provide a summary of available information with respect to Canadian climatic conditions, sources and characteristics of asphalts used in Canada, pavement performance in Canada and existing Canadian agencies' asphalt specifications and testing procedures.

ii) review and comment upon the SHRP asphalt binder specification and its applicability in Canada.

iii) assess the current status of modified asphalts in Canada and how their future utilization can be accommodated.

iv) produce a TAC User Guide for the existing CGSB Standard.

TAC recognized that utilization of the current CGSB Standard CAN/CGSB-16.3-M90 could be enhanced if a User Guide was available to aid in the asphalt cement selection process. The current CGSB Standard was published in 1990, after a decade of development by a committee comprised of some of Canada's leading experts in asphalt technology. A matrix of eighteen candidate asphalt

cements exists, as a function of penetration grade and temperature susceptibility Group (i.e. A, B and C).

Specified properties (penetration at 25°C, viscosity at 60°C) are expressed in the CGSB Standard on the basis that the asphalt cements are in their original (as manufactured) condition. Initial age hardening of the asphalt cement, that occurs in the asphalt mix production plant, and that continues throughout the service life of the pavement, must be recognized by the individual who selects the asphalt cement for a specific construction project. This is primarily because the age hardening phenomenon modifies the low service temperature at which thermal cracking will occur in the pavement. Thus the asphalt cement selection methodology that has been developed for utilization in the TAC User Guide includes a Safety Factor of 10°C.

The SHRP Superpave asphalt binder specification was developed to classify asphalt binders based upon the physical and rheological properties that relate to pavement performance. The Superpave binder specification addresses the three main pavement distress modes of rutting, fatigue cracking and thermal cracking and wherein specified properties are applicable to initially aged binders (either after Rolling Thin Film Oven or Pressure Aging Vessel conditioning). This is viewed to be an advantage over the CGSB Standard, which, as previously noted, provides no direct linkage to the age hardening process.

The EBA Project Team has utilized conventional asphalt cement technology, that has been previously developed by eminent technologists, to develop the methodology necessary to select CGSB asphalts to minimize low temperature thermal cracking. In addition, a concept has been developed to enable the TAC Guide user to select the most appropriate asphalt binder to mitigate instability rutting at high seasonal pavement temperatures.

The report is concluded with a modest number of recommendations for consideration by TAC regarding future direction. These are for the purpose of enhancing the process by which the CGSB asphalt cements are selected to satisfy specific project requirements.

SOMMAIRE

En 1992, l'Association des transports du Canada (ATC) a chargé EBA Engineering Consultants Ltd. (EBA) de mener une étude intitulée «Characteristics, Performance and Selection of Paving Asphalts». Cette étude portait sur la norme **«Liants bitumineux pour les routes» (CAN/CGSB-16.3-M90 de l'Office des normes générales du Canada)**.

L'étude donnait un aperçu des méthodes actuelles adoptées par des organismes canadiens pour la sélection de liants bitumineux destinés à la production d'enrobés à chaud, et elle comprenait une synthèse bibliographique, une liste de fournisseurs de bitume et une évaluation des répercussions probables de la recherche et des normes relevant du programme stratégique de recherche routière (SHRP). L'étude visait les objectifs suivants :

(i) résumer les données existantes sur le climat canadien, les sources et les caractéristiques des bitumes utilisés au Canada, le rendement des revêtements au Canada et les normes et méthodes d'essai existantes d'organismes canadiens oeuvrant dans le secteur du bitume;

(ii) évaluer la norme SHRP sur les liants bitumineux et sa mise en application au Canada;

(iii) faire un bilan des bitumes modifiés et de leurs applications éventuelles au Canada;

(iv) préparer au nom de l'ATC un guide d'utilisation de la norme CGSB.

L'ATC a compris que l'utilisation de la norme CAN/CGSB-16.3-M90 serait plus facile s'il existait un guide de sélection des liants bitumineux. La norme CGSB actuelle a été publiée en 1990, après une décennie de travaux menés par un comité constitué de spécialistes canadiens de la technologie des bitumes. Il existe une liste de 18 liants bitumineux, classés selon la pénétration et la sensibilité à la température (groupes A, B et C).

Les propriétés particulières (pénétration à 25 °C, viscosité à 60 °C) sont exprimées dans la norme CGSB en fonction de l'état original (brut de fabrication) des liants bitumineux. Le durcissement structural initial du liant, qui se produit dans l'usine de production des mélanges bitumineux et qui se poursuit pendant la durée de la vie du revêtement, doit être reconnu par

la personne qui choisit le liant bitumineux en vue d'un projet de construction particulier. Il en est ainsi parce que le durcissement structural est un phénomène qui détermine la basse température de service à laquelle le revêtement subit une fissuration thermique. La méthode de sélection d'un liant bitumineux proposée dans le guide d'utilisation préparé par l'ATC englobe un facteur de sécurité de 10 °C.

La norme SHRP Superpave sur les liants bitumineux a été mise au point en vue d'un classement des liants bitumineux d'après les propriétés physiques et rhéologiques liées au rendement des revêtements. La norme Superpave aborde les trois principaux modes de dégradation des revêtements, c.-à-d. l'orniérage, la fissuration par fatigue et la fissuration thermique, et elle décrit les propriétés particulières des liants qui ont subi un vieillissement initial (soit de type Rolling Thin Film Oven, soit de type Pressure Aging Vessel). Il s'agit là d'une amélioration relativement à la norme CGSB, qui n'établit aucun lien direct avec le phénomène du vieillissement structural.

L'équipe du projet EBA a utilisé la technologie courante des liants bitumineux, mise au point par des spécialistes réputés, afin d'élaborer une méthode de sélection des bitumes CGSB qui permet de minimiser la fissuration par temps froid. De plus, l'utilisateur du guide de l'ATC y trouve une stratégie de sélection du liant bitumineux qui se prête le mieux à une réduction de l'orniérage qui se produit aux températures saisonnières élevées du revêtement.

Le rapport présente quelques recommandations à l'ATC au sujet des perspectives d'avenir. Il s'agit d'améliorer la méthode de sélection des liants bitumineux CGSB de façon à répondre aux exigences de projets particuliers.

TABLE OF CONTENTS

ACKNOWLEDGEMENTS

EXECUTIVE SUMMARY

SOMMAIRE

1.0 INTRODUCTION 1

2.0 BACKGROUND 2
- 2.1 THE EXISTING CGSB STANDARD 2
- 2.2 DEFINITIONS OF TERMS 3

3.0 CANADIAN PAVEMENT PERFORMANCE 4
- 3.1 AGENCY QUESTIONNAIRE SYNOPSIS 4
  - 3.1.1 Specifications for Asphalt Cement 4
  - 3.1.2 Chemical Additives 6
  - 3.1.3 Design Properties 6
  - 3.1.4 Quality Control 7
  - 3.1.5 Other Studies 7
- 3.2 LOW TEMPERATURE CRACKING 8
  - 3.2.1 Current Significance in Canadian Jurisdictions 8
  - 3.2.2 Summary of Current Status 8
  - 3.2.3 Previous Research 10
  - 3.2.4 Synopsis 13
- 3.3 PERMANENT DEFORMATION 14
  - 3.3.1 Current Significance in Canadian Jurisdictions 14
  - 3.3.2 Summary of Current Status 14
  - 3.3.3 Synopsis 20
- 3.4 FATIGUE 22
  - 3.4.1 Current Significance in Canadian Jurisdictions 22
  - 3.4.2 Summary of Current Status 22
  - 3.4.3 Synopsis 25
- 3.5 MOISTURE SENSITIVITY AND STRIPPING 25
  - 3.5.1 Current Significance in Canadian Jurisdictions 25
  - 3.5.2 Summary of Current Status 25
  - 3.5.3 Synopsis 29
- 3.6 AGING OF THE ASPHALT/AGGREGATE SYSTEM 30
  - 3.6.1 Current Significance in Canadian Jurisdictions 30
  - 3.6.2 Summary of Current Status 31
  - 3.6.3 Synopsis 32
- 3.7 DURABILITY OF ASPHALT CONCRETE PAVEMENTS 33
  - 3.7.1 Summary of Current Status 33
  - 3.7.2 Synopsis 35
- 3.8 PREMIUM GRADE AND POLYMER MODIFIED ASPHALTS 35
  - 3.8.1 Current Significance in Canadian Jurisdictions 35
  - 3.8.2 Summary of Current Status 35
  - 3.8.3 Synopsis 37

Table of Contents (continued)

3.9 RELATIONSHIP OF PAVEMENT PERFORMANCE TO ASPHALT PROPERTIES 37

4.0 CANADIAN CLIMATIC CONDITIONS 39
4.1 INTRODUCTION 39
4.2 MINIMUM DESIGN TEMPERATURE 39
4.3 MAXIMUM DESIGN TEMPERATURE 49
4.4 SELECTION OF DESIGN TEMPERATURES 51
4.4.1 SHRP Database & SHRPBIND 51
4.4.2 Procedure for Selecting Minimum Pavement Design Temperature Value 54
4.4.3 Procedure for Selecting Maximum Pavement Design Temperature Value 58
4.5 SOURCES OF CANADIAN TEMPERATURE DATA 59

5.0 SOURCES OF ASPHALT USED IN CANADA 61

6.0 ASPHALT SPECIFICATIONS AND SELECTION PROCEDURES 72
6.1 SYNOPSIS OF QUESTIONNAIRE 72
6.2 MEASURING TEMPERATURE SUSCEPTIBILITY OF ASPHALT CEMENT BY PI AND PVN 72
6.3 LIMITATIONS OF CURRENT SPECIFICATIONS 77
6.3.1 Background 77
6.3.2 SHRP Binder Specification 77
6.3.3 Precision Aspects of Current Testing Protocols 80
6.3.4 Aging of CGSB Asphalt Cements 82
6.4 ASPHALT CEMENT GRADES USED IN CANADA 83
6.5 ELEVATED SERVICE TEMPERATURE CONSIDERATIONS 84
6.5.1 Introduction 84
6.5.2 Concept Development 85
6.5.3 Future Direction 95
6.5.4 Other Information 95
6.5.5 Synopsis 96
6.6 QUALITY CONTROL/QUALITY ASSURANCE 97

7.0 IMPACT OF SHRP RESEARCH 98
7.1 SHRP BINDER SPECIFICATION 98
7.1.1 Dynamic Shear Rheometer 98
7.1.2 The Bending Beam Rheometer 99
7.1.3 The Direct Tension Test 99
7.2 RELATIONSHIP OF SHRP SPECIFICATION TO CGSB 100

8.0 CGSB USER GUIDE FOR SELECTING ASPHALT CEMENTS FOR ROAD 101
8.1 DEVELOPMENT OF CAN/CGSB-16.3-M90 101
8.1.1 Introduction 101
8.1.2 Development of the Specification 102

Table of Contents (continued)

8.2 EXPLANATION OF THE TAC USER GUIDE 103
8.2.1 Background 103
8.2.2 The TAC User Guide 104

9.0 OTHER CONSIDERATIONS 107
9.1 MIX DESIGNS 107
9.2 CONSTRUCTION Q/C AND Q/A 107
9.3 PAVEMENT REHABILITATION AND RECYCLING 108

10. FUTURE DIRECTION 109

11. CLOSURE 111

12. ACKNOWLEDGEMENTS 112

REFERENCES
GLOSSARY
TABLES
FIGURES

APPENDIX A - AGENCY SURVEY OF PAVING ASPHALTS USED IN CANADA
APPENDIX B - CALCULATION OF PI POINTS
APPENDIX C - DEVELOPMENT OF CAN/CGSB-16.3-M90
APPENDIX D - TAC USER GUIDE

1.0 INTRODUCTION

EBA Engineering Consultants Ltd. (EBA) was retained by the Transportation Association of Canada (TAC) to undertake a project to produce a guide for the selection of the optimum paving asphalt cement grade for pavements constructed in Canada. The project is entitled "Characteristics, Performance and Selection of Paving Asphalts: TAC Final Report".

The primary objectives of the Project are to:

1. Produce a User Guide for the existing Canadian General Standards Board (CGSB) specification **Asphalt Cements for Road Purposes** (CAN/CGSB-16.3-M90) which will:

   i) be used as a supplement to the CGSB specifications.

   ii) incorporate and expand upon material prepared during the development of previous draft versions of the CGSB User Guide.

   iii) incorporate a summary of currently available information with respect to:
   - Canadian climatic conditions
   - Sources and characteristics of asphalts used in Canada
   - Canadian pavement performance
   - Provincial asphalt specifications and test procedures
   - likely impact of the Strategic Highway Research Program (SHRP) research and specifications on the use of the CGSB specification

   iv) contain guidelines for utilization and selection of premium asphalts and polymer-modified asphalts in the selection procedure.

2. Prepare a final report which is to include a synthesis of existing published information, ascertain aspects which influence the selection and performance of asphalt cement and provide a User Guide which contains a step-by-step asphalt selection process. Relevant sections within the TAC final report are to cite specific references that have been utilized in designing the selection procedure. These sections provide an interested party with some historical and evolutionary information and identify other sources of detailed information.

2.0 BACKGROUND

2.1 THE EXISTING CGSB STANDARD

CAN/CGSB-16.3-M90 was intended to serve as a national standard specification for asphalt cements to be used by pavement design engineers in Canada. Previous editions of the CGSB specification did not include viscosity as a selection criteria. The current specification consists of tabular data and two figures used for selection of asphalt cement. Figures 1 and 2 are taken from the CGSB Standard.

A group of leading Canadian experts in asphalt paving technology formed a Committee on Road Materials established by CGSB to develop a new national standard. The process began in 1979 and culminated with publication of the new standard in 1990. A User Guide was never published with the new standard, although a number of draft versions were produced. It was recognized by TAC that, without a User Guide, selection of asphalt cement on a project specific basis using the CGSB Standard is not a straightforward procedure. Some of the provincial agencies have developed their own standard using the concepts put forward in the CGSB Standard. Only a few provincial agencies have adopted the CGSB Standard in its entirety.

As discussed in more detail in Section 8.1, the CGSB Standard was intended to be performance based, and indeed the specifications for asphalt cements are performance based when the specification is used properly. The intent in this regard was to ensure that improved long term pavement performance was facilitated. In spite of the wealth of experience and background knowledge which has been acquired over the past several decades, there is still not clear consensus amongst users, producers and other industry experts as to the most suitable performance based specifications. SHRP and the Canadian counterpart C-SHRP, continue to expand the knowledge base in this regard. New scientific avenues have been proposed by SHRP researchers for assessing and categorizing both asphalt cements and asphalt paving mixtures. It is evident that several more years may pass before a consensus is achieved. Pavement performance, as a function of tentatively adopted new SHRP techniques, needs to be

validated. In the meantime, the CGSB specification will be utilized by many Canadian jurisdictions. It should be utilized to the optimum benefit of these user agencies.

The current CGSB specification is thorough and sophisticated, particularly in respect to the manner in which temperature susceptibility of conventional asphalt cements is accommodated. It properly recognizes the broad range of temperature conditions under which asphalt concrete pavements are required to perform in Canada.

2.2 DEFINITIONS OF TERMS

Terms which are commonly used by pavement designers appear throughout both this report and the TAC User Guide, as it is referred to hereafter. Some terms are explained at appropriate points in the text. A Glossary is provided to define terms which may be unfamiliar to some readers.

3.0 CANADIAN PAVEMENT PERFORMANCE

3.1 AGENCY QUESTIONNAIRE SYNOPSIS

A survey of Canadian agencies was undertaken to determine the nature of pavement performance problems, and to document the relative severity of the types of pavement distress which develop in asphalt concrete pavements in Canada. Included in Appendix A is a summary of the responses from the various agencies that were surveyed. Questionnaires were completed by all provincial highway agencies, one federal government department, one territorial highway agency and two cities. It should be noted that any analysis of a questionnaire may have a bias regarding the personal experience of the individual completing the questionnaire. Certain questions may be interpreted by that individual in light of this bias. Therefore, any individual answer may be questioned even by other people within the same agency. However, trends and clear consensus will likely result from valid observations from the questionnaire despite individual bias. Tables 1 and 2, discussed hereafter, have been prepared to summarize significant agency practices and current problems.

The following brief summary provides an indication of the responses received.

3.1.1 Specifications for Asphalt Cement

Currently all agencies specify asphalt cements using penetration grades. Viscosity requirements and tests on residue from distillation are usually specified for each specified penetration grade as well. Most of the agency specifications are similar to the current CGSB specification. Some agencies have adopted a specification similar to CGSB with slightly different viscosity or penetration values than those used by CGSB.

Many surveyed agencies expect to modify their asphalt cement specifications as a result of the SHRP research. The existing asphalt cement specifications have a wide range of perceived adequacy that varies from somewhat lacking to excellent. Most agencies believe that their

specification can be improved. Temperature susceptibility and modified asphalts appear to be the most likely areas requiring improvement.

It is apparent that asphalt cement grades are generally not selected on a site specific basis with respect to the maximum and minimum design temperature and in service temperatures. Clarification of some original responses confirmed that most agencies do not select the grade of asphalt cement on a project-specific basis.

Most agencies use more than one grade of asphalt cement and selection of the appropriate grade of asphalt cement is based on several factors. Climate, asphalt cement stiffness and traffic are the primary factors considered when choosing a grade of asphalt cement.

It is apparent that some agencies no longer keep records regarding the quantity of asphalt cement used. It appears that the advent of End Product Specification projects has reduced the need for keeping these records. For those agencies who did respond to this question, it is apparent that 150 - 200A is used extensively across Canada. Significant quantities of 80 - 100A (85 - 100) grade are used in Quebec, Ontario and British Columbia. 80-100C grade asphalt is used only in British Columbia.

As a general rule, asphalt cements used in Western Canada come from prairie crude oil sources. East coast provinces receive their crude from overseas. Most agencies would like to know the crude source. It is important to note that in eastern Canada asphalt comes from a host of sources. A stable source of high quality asphalts exists in western Canada. Therefore it may be expected that some agencies may have differing priorities with respect to performance specifications and implementation of SHRP protocols. Most agencies are actively reviewing the SHRP procedures and anticipate some impact on their asphalt cement specifications. The length of time anticipated to implement some or all of the SHRP specifications varies from two to ten years.

3.1.2 Chemical Additives

Most provincial agencies use some type of antistripping agents, on occasion, in their asphalt mixes to improve performance. When past experience indicates that an antistripping agent is required, the optimum dosage is determined using either the Lottman test or Marshall Immersion testing. Most agencies, with one exception, have found that the additives always improve the stripping performance. In that instance the agency suggested that some antistripping agents may be "cooked off" during mixing and that some aggregate sources need specific antistripping agents.

Other additives, used on occasion, include hydrated lime and silicone. Silicone was mentioned by three provincial agencies as an additive used occasionally to enhance the texture of the mix, to reduce asphalt sticking to equipment, to reduce slumping of mix during truck haul or to treat highly absorptive aggregate. British Columbia has reportedly achieved success in using Tall Oil as an antistripping agent.

3.1.3 Design Properties

All agencies responding to the questionnaire use the Marshall method of mix design. One agency (British Columbia) also uses the Hveem method of mix design. No agencies have adopted the SUPERPAVE™ method. Most of the Marshall mix designs are 75 blow mixes but some agencies use a 50 blow Marshall or have a 50 blow alternate as well as the 75 blow procedure.

The Marshall mix designs have a wide variety of attributes specified or evaluated, including a variety of stability values. Generally, design air voids are within the two to six percent range, with most mixes being designed for three to five percent air voids. Ten agencies specify limiting VMA. Some agencies include film thickness, voids filled and index of retained stability as specification requirements.

In all cases, mix designs are prepared using the grade of asphalt cement from the proposed refinery. The refinery supplies temperature-viscosity charts for selection of mixing and compaction temperatures.

3.1.4 Quality Control

In all but one jurisdiction, the properties of the asphalt cement delivered to a project are measured regularly by the agencies. Table 1 provides a summary of tests conducted on the asphalt cement. Pavement performance problems that may be associated with asphalt cement properties are varied by jurisdiction. Table 2 shows the responses noted on the questionnaire.

3.1.5 Other Studies

Questions were asked regarding studies undertaken which correlated asphalt cement properties with asphalt pavement performance, low temperature cracking and recycling. Five jurisdictions had published reports pertaining to performance based on asphalt cement properties. An additional three jurisdictions believed they had information that could contribute to an asphalt cement vs. low temperature cracking study.

Other questions were asked regarding recycling practices. Responses, together with subsequent clarifications, enable the following observations to be made: -

- The Maritime provinces, Quebec, and Manitoba do not use rejuvenating agents during recycling but occasionally they use a softer grade asphalt cement or emulsified asphalt in cold recycling.

- Rejuvenating agents have had differing but generally limited utilization. Ontario, Saskatchewan and British Columbia report occasional usage. Alberta has only used rejuvenators occasionally, on an experimental basis in hot-in-place recycling projects. Rejuvenation requirements are usually determined at the mix design stage to ascertain whether a recycled mix can be prepared to duplicate the properties of a "virgin" mixture.

3.2 LOW TEMPERATURE CRACKING

Low temperature cracking is that cracking that occurs when contraction forces, caused by cooling, result in a crack, usually transverse, across a pavement. Transverse cracking that reflects through an overlay from lower cracked layers is not considered to be low temperature cracking. It is considered to be reflective cracking.

3.2.1 Current Significance in Canadian Jurisdictions

Twelve of the respondents noted that low temperature cracking is a problem. Nine agencies included low temperature cracking on their list of top priorities to find appropriate solutions. The only provincial agency that does not believe low temperature cracking to be a problem is Nova Scotia.

3.2.2 Summary of Current Status

Of the several types of non-load associated pavement distress, low temperature cracking has probably been the one of most concern to highway engineers in the second half of this century. McLeod (1968) provided an explanation of historical events which led to the occurrence of low temperature cracking of asphalt concrete pavements in Canada. He noted that: *"Following passage of the Trans-Canada Highway Act in 1949, it was felt that the higher construction standards adopted justified the use of harder grades of asphalt cement. Throughout Canada, 150/200 penetration asphalt became generally employed, with considerable use of 85/100 penetration in Ontario and Quebec. Since the introduction of harder grades of asphalt cement, the transverse cracking of asphalt pavements has eventually become a matter of grave concern."* At this same time use of the prairie waxy crudes was an issue that may have contributed to the development of low temperature cracking.

Mechanisms of low-temperature cracking of asphalt concrete pavements have been studied since the early 1960's and, ultimately, reasonable consensus was reached as to how reoccurrence of this cracking could be mitigated. Research reports and documentation are extensive.

C-SHRP (1993), Deme and Palsat (1989) and Roberts et al (1991) provide excellent background information, primarily in respect to the role Canadian technologists have played in isolating causes of low-temperature cracking and evolving effective solutions to the problem.

It is important to recognize that factors, other than the asphalt cement properties, can cause pavement cracking to occur. McLeod (1969) identified some factors, including:

- subgrade influences (e.g. sand vs. clay)
- reflection of previously cracked pavement through a subsequent overlay
- Portland cement stabilized base or sub-base layers which have their own unique cracking characteristics
- thickness of the asphalt concrete
- age of the asphalt concrete

Cracking may occur as a result of these types of influences, irrespective of the properties of the asphalt cement within the asphalt concrete surface layer(s).

One of the earliest Canadian references to low temperature transverse cracking was made by Shields and Anderson (1964). Since that time, full scale test roads were designed and constructed in Canada for the purpose of understanding and resolving the problem. Roberts et al (1991) note *"the Ste. Anne Test Road in Canada has been the most comprehensive full scale project reported in the literature. Many researchers have based the critical asphalt cement stiffness at low temperatures on data from this project".* This test road was constructed in 1967 by the Manitoba Department of Highways, in co-operation with Shell Canada Limited.

Of more recent significance is the C-SHRP initiative that was for the purpose of correlating low temperature pavement performance to the quality of various Canadian asphalt binders. Full scale test roads were constructed by Provincial highway agencies in Alberta (Lamont - 1991), Ontario (Hearst - 1991) and Quebec (Sherbrooke - 1992). A final C-SHRP report (1994) contains progressive findings of that project.

3.2.3 Previous Research

It is commonly accepted that low temperature cracking occurs when asphalt concrete pavement is subjected to low temperatures which cause high tensile stresses to develop due to contraction. When the stresses exceed the fracture strength of the pavement, transverse cracking occurs. On the basis of five-year observations of the Ste. Anne Test Road pavements, Gaw et al (1974) concluded that:

1. Transverse cracking was due to thermal stresses which initiated at the pavement surface.
2. Transverse cracking occurred when the stresses which developed in the pavement at low temperatures exceeded the breaking strength.
3. The most critical influence on pavement cracking severity was the asphalt binder.
4. The observed road cracking correlated with asphalt stiffness at low temperatures, and that increasing temperature susceptibility (decreasing PI) and decreasing penetration both result in increased cracking.
5. Road design and soil type influenced transverse cracking.
6. Traffic effects influenced the severity of the low temperature induced cracking.
7. Variations in paving mixture constituents (asphalt content, aggregate type and gradation) had no obvious influence on transverse cracking.

It is therefore evident that the predominant cause of low temperature cracking of asphalt concrete pavements is high asphalt cement stiffness at low temperatures. Other factors, such as thickness of the pavement layer, subgrade soil type and traffic influence the frequency and extent of these cracks. Hajek and Haas (1971) have considered asphalt cement stiffness, as well as pavement thickness, pavement age and subgrade type, to develop a model to predict low temperature cracking frequency of asphalt pavements.

A number of researchers have evolved limiting asphalt binder stiffness criteria, based at least in part on the Ste. Anne Test Road data. Gaw et al (1974) reported that pavements cracked at an asphalt binder stiffness of 1×10^9 N/m² at 0.5 hr loading time. Readshaw (1972) based British Columbia asphalt cement specifications on a critical asphalt cement stiffness of

2×10^8 N/m² at 2.0 hr loading time. Fromm and Phang (1971) suggested that the critical asphalt cement stiffness is 1.4×10^8 N/m² at 10,000 seconds loading time.

Corkhill and Kopvillam (1981) demonstrated how the performance characteristics of an asphalt over its entire service temperature range can be obtained from its penetration at 25°C and its viscosity at 60°C, with the aid of the Bitumen Test Data Chart (BTDC). An extended version of the BTDC was used to permit estimation of the pavement cracking temperature.

Robertson (1987) has addressed the issue of critical binder stiffness as it relates to loading time. In commenting upon work done by Hills (1964), Robertson has stated that *"the calculation method underestimates thermal stresses because it uses a constant and inappropriately long stress relaxation time for the modulus values over the whole cooling period. In practice, stress increments generated at the start of the cooling process are almost completely relaxed at the end, due to the long loading time ---".*

Robertson used this logic to explain why thermal cracking temperatures correlate better with a limiting stiffness value at a relatively short loading time. Using, as the fracture temperature, that temperature (change) required to develop an asphalt tensile stress of 5×10^5 N/m², a design chart was produced that was stated to be applicable to all of the many types of asphalt cement currently available. He described the procedure used to derive the asphalt fracture temperature at which point the tensile stress is 5×10^5 N/m². His procedure introduces the effect of stress relaxation into the calculation.

SHRP researchers (Anderson et al, 1993) have adopted a limiting asphalt stiffness value of 300 MPa (3×10^8 N/m²) at a loading time of two hours. As described by Roque et al (1993), SHRP has incorporated a mixture specification test that includes a relaxation modulus curve, which tends to validate the Robertson procedure identified above.

Robertson has used Penetration Index (PI) as the temperature susceptibility parameter to characterize asphalt cements. PI was first defined by Pfeiffer and van Doormaal (1936), as a

function of the relationship of penetration and temperature. Penetration Index is calculated from:

$$PI = \frac{20 - 500B}{50B + 1}$$

and

$$B = \frac{d \log_{10} (Pen.)}{dT}$$

Readshaw (1972) used two penetration values to describe temperature susceptibility, namely pen (25°C, 100 g, 5 sec) and pen (5°C, 200 g, 60 sec). The term penetration ratio has evolved, where:

$$\text{penetration ratio} = \left\{ \frac{\text{pen } 5°C, 200 \text{ g}, 60 \text{ sec}}{\text{pen } 25°C, 100 \text{ g}, 5 \text{ sec}} \right\} 100$$

Robertson (1982) has confirmed that as a means of controlling temperature susceptibility in the low temperature range, penetration ratio is highly correlated with the Penetration Index. This point is worthy of note, since one concern with respect to accurate penetration index determination is that penetration (100 g, 5 sec) must be measured at least at two temperatures, usually 25°C and 5°C. C-SHRP (1994) has noted that ASTM multi-laboratory precision criteria, especially for values below 50 units, are onerous. When penetration (100 g, 5 sec) is measured at 5°C very small values typically are obtained, and thus the precision of such values should be considered a point of concern.

McLeod (1969, 1972, 1976) developed the concept of Pen-Vis. Number (PVN) as an alternative to Penetration Index to characterize temperature susceptibility of asphalt cements. He stated that *"because of the way they are derived, pen-vis numbers are numerically similar to, and may be numerically identical with penetration index values for many paving asphalts"*. McLeod developed equations to determine PVN using penetration (25°C, 100g, 5 sec) and either kinematic viscosity @ 135°C or absolute viscosity @ 60°C, where:

$$\text{PVN (vis } 135°C) = -1.5 \left\{ \frac{4.258 - 0.7967 \text{ Log P} - \text{Log } V_1}{0.7951 - 0.1858 \text{ Log P}} \right\}$$

$$\text{PVN (vis } 60°C) = -1.5 \left\{ \frac{6.489 - 1.590 \text{ Log P} - \text{Log } V_2}{1.050 - 0.2234 \text{ Log P}} \right\}$$

where P = penetration at 25°C, 100 g, 5 sec, dmm
V_1 = kinematic viscosity at 135°C, centistokes
V_2 = absolute viscosity at 60°C, poise

Robertson (1987) has reported that PVN was intended to be numerically equal to PI so that it could be used for estimating low temperature stiffness of asphalts using van der Poel's nomograph. He earlier pointed out (1978, 1982) that PI and PVN are not numerically equal for most asphalts. That finding was also reported by Puzinauskas (1979). There also continues to be concern due to evidence that suggests that PVN is unreliable in characterizing asphalt derived from waxy crudes. Nevertheless, characterization of temperature susceptibility of paving asphalts, using McLeod's PVN approach, has retained some acceptance in Canada and elsewhere, likely because of convenient availability of test data.

On the basis of findings from the C-SHRP Test Roads project (1994) it has been recommended that cracking temperatures for conventional asphalt cements should be estimated using criteria developed by Readshaw (1972) or Fromm and Phang (1971). Further, PI, which is used to calculate the stiffness of the asphalt cement in these predictive methods, must be determined from a minimum of three penetration tests performed at 25°C and at two other lower temperatures that yield a penetration value greater than ten units. While direct comparisons between PI and PVN are not made by C-SHRP (1994), data contained in that report indicates that PI and PVN are not numerically equal.

3.2.4 Synopsis

Specifying asphalt cements on the basis of Penetration Index to characterize temperature susceptibility is compatible with the criteria contained in Figure 1 of CAN/CGSB-16.3-M90 (Figure 1 in this report). Confirmation of this observation is provided in Section 6.2. Therefore, creation of guidelines for selection of asphalt cements in Canada should be in accordance with:

- design and/or cracking temperatures predicated on the use of the Penetration Index method, using the Bitumen Test Data Chart (BTDC), and using at least three penetration values as described in Section 3.2.3
- design temperature selection

- traffic (low, medium, heavy) as well as in consideration of other distress mechanisms discussed.

3.3 PERMANENT DEFORMATION

3.3.1 Current Significance in Canadian Jurisdictions

Permanent deformation can take several forms. However, the most apparent deformation is considered to be instability rutting. In its intermediate and advanced stages, instability rutting is a severe safety problem. Five agencies identified that they were not experiencing problems with asphalt cement as related to pavement deformation. Five provincial agencies and the two cities identified that developing solutions to permanent deformation was their highest priority.

3.3.2 Summary of Current Status

Permanent deformation of asphalt concrete pavement has come to be called "rutting". Since it is traffic induced, it is naturally predominant in the wheelpaths. Emery and MacDonald (1990) and Dawley et al (1990) have defined three types of permanent deformation which may exist, either individually or in combination, within a pavement, i.e.: -

1. Structural (consolidation) rutting, which is due to the permanent vertical deformation of the pavement structure under repeated traffic loads and which is essentially a reflection of permanent deformation within the subgrade and/or supporting unbound granular layers..
2. Wear (abrasion) rutting which is due to progressive loss of coated aggregate particles from the pavement surface, and which is caused by a combination of construction deficiencies, environmental and traffic influences.
3. Instability rutting which is due to lateral displacement of material within the pavement layer, and occurs in the wheelpaths. It occurs when the properties of the compacted asphalt concrete are inadequate to resist the stresses imposed upon it, particularly by frequent repetitions of channelized heavy axle loadings.

Discussing mitigation of structural rutting is beyond the scope of this project. Environmental distress mechanisms which may be associated with wear rutting are addressed in Section 3.5 (Moisture Sensitivity). The significance of the asphalt binder, in respect to instability rutting, is the primary focus of this Section.

AASHTO (1994) provides a rule of thumb "that about 40 percent of the resistance of a paving mix to permanent deformation is attributable to properties of the asphalt binder in contrast to 60 percent for fatigue cracking and as much as 90 percent for low temperature cracking".

Whereas asphalt cement properties are dominant in respect to satisfactory low temperature pavement performance, it should be accepted that they are only a contributing factor in respect to instability rutting. To mitigate instability rutting, the mineral aggregate component of the compacted paving mixture is of paramount importance. Physical properties (soundness, toughness and particle shape) and gradation characteristics of the aggregate must be carefully specified and controlled, to produce the aggregate "skeleton" within the asphalt concrete that is necessary to effect load transfer from imposed tire loadings to underlying support layers. This is only achieved when grain-to-grain contact and aggregate particle interlock exist.

Instability rutting of asphalt pavements has existed for many years. It was not until the early 1970's, however, that pavement designers started to discriminate between structural and instability components of rutting propagation. The timing, however, coincided closely with the time at which increased gross vehicle loads, higher tire pressures and radial tires came into being.

Meyer et al (1974) described this type of distress in pavements, noting that "*it occurs as the result of both vertical and horizontal movement of the material in the pavement layers*". They developed a technique which was for the purpose of predicting rut depths in pavements. They also noted that, while pavements can be designed to resist permanent deformation, it is important not to compromise to the extent that fatigue and low temperature cracking distress modes are accelerated.

Uzan et al (1980) presented case histories on three Alberta highways where permanent deformation had been observed in full depth pavement structures. Their primary focus was on refining an existing rutting prediction model. They observed that as the asphalt concrete modulus increased, the rate of rutting decreased.

The Western Association of State Highway and Transportation Officials (1984) released a report which addressed a serious rutting problem that had developed on several states' highways. It was reported that subgrade deformation was not a problem but that attempts to mitigate the historic low temperature cracking problem had perhaps created the more serious problem of pavement rutting. The use of softer asphalts and over-asphalted mixes were thought to be factors that had contributed significantly to the rutting problem.

Haas (1984) reported that rutting exists, to varying degrees, in all parts of Canada. However, it is really from 1986 onward that Canadian pavement designers became serious about mitigating the growing problem. RTAC (1986) held a Rutting Workshop at which Tam and Lynch reported that rutting exists on Ontario's freeways.

Huber and Heiman (1987) reported on rutting of asphalt pavements in Saskatchewan. Their significant conclusions were:

- Penetration and viscosity of the asphalt do not demonstrate a significant effect on rutting rate.
- Asphalt content and voids filled are the most basic parameters which affect rutting, while Marshall stability and flow showed little correlation with rutting performance.
- Fractured faces (i.e. angularity) and voids in the mineral aggregate (VMA) are secondary factors.

While their findings support some of the conclusions of other researchers, it is noteworthy that there is disagreement on the very significant issues of aggregate fracture and VMA. The range of aggregate properties may have been insufficient to adequately define this property.

Varady and Pijl (1987) reported that rutting of asphalt pavements had become a significant problem on highways in British Columbia. It had come to be recognized that existing Marshall design standards were not providing satisfactory pavement performance. The Hveem Design Method was investigated. Results of Marshall and Hveem designs were compared. The Hveem Method proved to be more competent, in that they determined that pavement rutting did not develop when paving mixtures possessed Hveem Stability values greater than 35. The Hveem method is less affected by binder and more by aggregate because of the rate of loading as observed by Vallerga and Lovering (1985). British Columbia did not report making any revisions to materials specifications. They did express concern for the quality of paving mixtures when the asphalt grade is an AC-5 or softer.

Emery and Johnston (1987) reported that rutting of asphalt concrete had become a serious problem in Metropolitan Toronto, Ontario, and that it existed in both deep strength and composite (asphalt concrete over Portland cement concrete) pavements. It was also noted that most of the rutting in the deep strength pavements had occurred in the binder (i.e. base) course. Factors that appeared to contribute to the problem were natural "rounded" sands and a VMA "hump" gradation. Interim recommendations that were made to mitigate this problem included:

- use of coarser mixes with 100 percent crushed natural aggregate or steel slag.
- reducing the VMA hump (by incorporating manufactured sand or steel slag).
- design for 4 percent air voids, flow of 8 to 14 and minimum compaction of 98% (of Marshall density).

For surface course rutting mitigation, as well as to accommodate fatigue and durability factors, use of engineered asphalts (latex and/or polymer modification) was under consideration.

In 1987, the City of Lethbridge, Alberta initiated a significant study to address an historic problem of pavement rutting. The first phase of the study comprised categorizing the type(s) of existing rutting and to develop strategies to mitigate its reoccurrence. MacDonald and Hogeweide (1988) reported that, on the basis of excavations through 10 sites which exhibited severe wheelpath rutting, there was no measurable structural (consolidation) rutting - i.e. rutting developed entirely within the asphalt concrete layer. Mixture disturbance extended approximately 75 mm into the asphalt concrete. They also determined, from laboratory testing,

that instability rutting occurred without a zero air voids condition or "bleeding" existing. It was concluded that improved resistance to instability rutting could be accomplished through:

- use of coarser, highly fractured aggregate.
- high proportion of manufactured fine aggregate.
- use of polymer modified asphalt.
- use of reclaimed asphalt pavement (RAP).
- adopting the 75 blow Marshall mix design procedure to control volumetric proportions of the paving mixture.

It was further recommended that a full-scale series of field test pavements be constructed on arterial streets in Lethbridge, and that performance of these test pavements be monitored over a five year service period. A series of six test mixes were designed and test pavements were constructed in 1988, after which performance monitoring commenced. Results of this project have been reported by Dawley et al (1990, 1991, 1993). Conclusions reached at the end of the five year in-service period included:

- The rate of instability rutting was substantially reduced by utilizing larger, highly fractured coarse aggregate, manufactured fine aggregate of high proportion (at least 75% of the total fine aggregate to be crushed particles), premium grade asphalt cement of the lowest penetration grade appropriate for low-temperature conditions, or polymer modified asphalt cement.
- Modest rates of incorporation of RAP (up to 30% by weight of virgin aggregate) could be utilized.
- Mixture design should be volumetrically based, using the 75 blow Marshall Standard, to produce 4% air voids, a minimum film thickness of 6.5 μm to yield controlled VMA properties, with the aggregate being densely graded.

The City of Lethbridge modified its asphalt concrete specifications in 1989 on the basis of early service performance of several of the test pavements. Instability rutting has been brought into line with acceptable performance criteria through this initiative.

McMillan and Anderson (1988) reported that deformation in the wheelpaths of asphalt concrete pavements reduces rideability and safety. A laboratory study was conducted to investigate the role of the binder in asphalt pavement rutting. They concluded that, while the grade of the

binder affected permanent deformation minimally, the rheology of the binder was of more significance than were the characteristics of the aggregate or the mix. Recycled mixes exhibited less deformation than virgin mixes. Polymer modified mixes increased resistance to permanent deformation significantly.

Anderson et al (1989) reported on a laboratory evaluation study which included indirect tensile tests to evaluate low temperature properties and repeated load triaxial tests to evaluate high temperature performance of paving mixtures. Enhanced binders were used in the study. The researchers utilized aggregates from the Lethbridge project (described above) as were some asphalt binders from the same supplier. The results of this study are most significant and include:

- The mixes containing the polymer modified binder exhibited higher failure stresses, higher failure strains and lower failure stiffness in the low temperature test range (as compared to mixes with conventional binder).
- The modified mixes are more resistant to permanent deformation under high temperature traffic conditions.

A City of Winnipeg study (1989) addressed the issue of permanent deformation of asphalt pavements. Recommendations contained in the report were aimed at improving aggregate quality, mix design criteria and quality control/quality assurance.

Gervais and Abd El Halim (1990) reported on rutting of asphalt overlays constructed in Nova Scotia and Ontario and concluded that new asphalt overlays, placed on previously rutted pavements, showed rutting distress soon after construction. Their conclusions included the need for greater attention to design and construction of such projects.

McMillan and Palsat (1990) identified a problem of pavement rutting on Alberta highways and described design criteria that were subsequently developed to optimize high and low temperature performance of pavements constructed in Alberta in the future. Their criteria include:

- selection of asphalt grade - use 'softest' grade that will provide acceptable rutting resistance.
- selection of aggregate characteristics - control of fracture in coarse aggregate and use of manufactured fine aggregate.
- Marshall mix design parameters - design for stability, air voids, VMA and number of blows, as a function of asphalt mix type selected.
- Traffic loading (ESALs over the pavement's design life).
- Consideration of regional summer and winter pavement temperatures.

The relationship of asphalt cement and paving mixture stiffness, as functions of temperature and loading time, has been mentioned previously, principally in respect to low-temperature service conditions. Stiffness properties at high service temperatures are a function of loading time which is reason for addressing loading conditions to mitigate permanent deformation. This is discussed in following sections.

3.3.3 Synopsis

Published research described in Section 3.3.2 typically confirms that asphalt concrete mixture design, to mitigate instability rutting within the asphalt concrete must consider five factors, namely:

1. selection of high quality aggregate
2. methodology for designing the paving mixture
3. influence of traffic
4. regional summer and winter pavement temperatures
5. selection of the appropriate grade of asphalt cement

Most published research suggests that the primary factor in optimizing pavement performance at high service temperatures relates to the aggregate skeleton created within a compacted paving mixture. Development of detailed mineral aggregate specifications is beyond the study terms of reference. However, the following general observations are illustrative of current practices in some jurisdictions. Aggregates used for paving mixtures within the upper portion of

pavements subjected to high volumes of truck and bus traffic should be comprised of highly fractured (crushed) particles. For such purposes, coarse aggregate should consist of 100 percent of particles with two or more fractured faces and should be of the largest practical nominal maximum particle size. The fine aggregate fraction should consist primarily of manufactured particles. Less onerous aggregate specifications are acceptable for low volume pavements. Most street and highway jurisdictions recognize their aggregate quality requirements as a function of past performance and regional requirements.

Properties of the asphalt cement are viewed by most researchers as a contributing, or secondary factor on pavement performance under heavy traffic. The temperature susceptibility properties of the asphalt binder are influential in the manner by which the rate of instability rutting is controlled at high temperatures under heavy traffic. This matter is addressed in more detail in Section 6.

The factor of mixture design methodology, and related design criteria, is of paramount importance in mitigating permanent deformation. However, this issue is beyond the scope of the present study.

In principle, asphalt binder selection criteria should give recognition to the relativity of heavy truck and bus traffic on a project specific basis, as well as to the annual temperature extremes which represent the in-service condition. Truck and bus traffic is normally represented by the number of cumulative equivalent single axle loads (ESALs) which are forecast by traffic engineering specialists during the design life of a pavement structure. (One ESAL is one 80 kN single-axle load). These selection criteria should generally be as follows: -

i) for low traffic level situations (eg. $\leq 1.0 \times 10^5$ ESALs), select a soft or intermediate grade of asphalt cement from those options that are available to meet the design low temperature requirement.

ii) for intermediate traffic level situations (eg. ESALs between 1.0×10^5 and 1.0×10^6); select the hardest grade of asphalt cement from those options that are available to meet the design low temperature requirement.

iii) for high traffic level situations (eg. ESALs > 1.0×10^6), select an enhanced binder that provides superior temperature susceptibility properties as compared to those that the most competent asphalt cement would provide under the same design conditions.

3.4 FATIGUE

3.4.1 Current Significance in Canadian Jurisdictions

Fatigue cracking was considered to be a problem in nine of the fourteen jurisdictions that completed the questionnaire. The two cities that responded to the questionnaire did not believe that fatigue of the asphalt concrete was a problem in their jurisdictions. Finding a solution to the fatigue problem was considered a priority by three jurisdictions.

3.4.2 Summary of Current Status

Two different types of fatigue distress can develop within an asphalt concrete pavement. The type that is best understood by pavement designers is structural fatigue, which results when repeated applications of vehicle axle loads cause cracking. The second type of fatigue distress is termed thermal fatigue and occurs as a result of repeated temperature cycles which can cause cracking to occur. Thermal fatigue theory is not well established or documented and is not addressed further. Aspects of structural fatigue are discussed below.

Roberts et al (1991) ascribe the likely causes of fatigue related performance problems to:

- low asphalt binder content
- asphalt cement too stiff
- water sensitivity
- inadequate support

They have presented a hypothesis for asphalt binder selection to minimize fatigue cracking:

- When the total asphalt concrete layer is less than about 125 mm, mixes of low stiffness (low viscosity asphalts) are appropriate, providing that the stiffness is not so low as to cause instability rutting within the pavement layers.

- When the total asphalt concrete layer is about 125 mm or greater, mixes of high stiffness (high viscosity asphalts) are preferable.

Fatigue cracking (often referred to as alligator cracking) usually occurs when an asphalt concrete pavement has been stressed to the limit of its fatigue life by repetitive load applications, or when loads are applied that exceed the capacity of the asphalt pavement structure. Phang and Chong (1974) have reported that on thin asphalt concrete pavements in Ontario, alligator-cracking has been initiated from other causes than structural strength inadequacy. For example, low temperature pavement cracking can permit the entry of surface water, which in turn, causes further distress due to frost action or weakening of an otherwise competent pavement support structure.

The relationship between asphalt cement type, mix stiffness and fatigue life is a relatively complex one. All other things being equal, a mix containing a softer asphalt will generally have a lower stiffness and a longer fatigue life. However, to make this statement requires more than an analysis of fatigue specimens in a laboratory. While many softer asphalts have longer fatigue lives for a given strain level, they may not have a longer life in a field situation. This is because the lower stiffness of the mix results in a overall weaker pavement structure and higher tensile strains at the base of the asphalt concrete layer, which may result in a shorter fatigue life. The structural design and the asphalt cement selection criteria should be integrated. If the structural design criteria has been established or validated by experience, the introduction of a softer asphalt cement should result in a re-evaluation of the structural capacity of the pavement. If the structural design is mechanistic, the appropriate lower mix stiffness value should be introduced in the design calculation if a softer asphalt cement is to be used. Despite the above caution the Roberts et al hypothesis stated above can be useful, and is discussed below: -

- When the total asphalt concrete layer is thin (less than about 125 mm), mixes of low stiffness, i.e. low viscosity asphalts, are appropriate. When the total asphalt concrete layer is thin some or all of the following conditions are likely: - low traffic volumes; good subgrade support; granular layers contributing to the pavement structure. Under these conditions a decrease in the stiffness of the asphalt concrete is not likely to result in a

significant increase in the tensile strain on the underside of the asphalt concrete layer. Thus use of a softer asphalt with its longer fatigue life is prudent.

- When the total asphalt concrete layer is thick (about 125 mm or greater), mixes of high stiffness, i.e. with high viscosity asphalt cements, are preferable. When the asphalt concrete layer(s) are thick the asphalt concrete contributes significantly to the structural capacity of the pavement structure. The use of low stiffness asphalt cement would likely increase the tensile strains at the bottom of the asphalt concrete layer and the improved fatigue properties would be negated by the higher strain levels. A higher stiffness asphalt concrete, however, would decrease the strain, likely offsetting any reduction in fatigue properties that a harder asphalt cement would create.

It is important to recognize that premature age hardening of asphalt cements in thin pavements will cause fatigue failure to occur prematurely. This phenomenon can be controlled by specifying appropriate asphalt cements, adopting suitable asphalt mix designs and by carrying out proper quality control and quality assurance testing programs during plant mixing and construction to ensure that the potential for premature age hardening of the asphalt binder is minimized.

Technical reports from Canadian transportation agencies, that relate to experience with fatigue distress, are minimal in number. There has also been little reported in respect to structural rutting distress. Fatigue and structural rutting (i.e. permanent deformation primarily at the subgrade interface) are both criteria associated with some of the standard structural design programs available to designers. Because most designers use some recognized structural design program, the frequency of occurrence of fatigue or structural rutting may be limited. It is recognized that many kilometres of Canadian asphalt concrete pavements have been rehabilitated long before the fatigue capacity of these pavements has been approached, due to safety and maintenance problems associated with instability rutting, ride deterioration and other forms of distress within the existing pavement.

3.4.3 Synopsis

For asphalt concrete pavements that have been systematically and structurally designed with regard for the level of anticipated traffic, criteria for selection of the grade of asphalt binder should be:

a) when the total asphalt concrete layer is thin (less than about 125 mm), select the lowest viscosity asphalt that is compatible with both low temperature and permanent deformation requirements.

b) when the total asphalt concrete layer is thick (about 125 mm or greater), select the highest viscosity asphalt that is compatible with both low temperature and permanent deformation requirements.

3.5 MOISTURE SENSITIVITY AND STRIPPING

3.5.1 Current Significance in Canadian Jurisdictions

All of the Maritime provinces as well as Quebec, Ontario and Saskatchewan indicated that, without antistripping agents, stripping was a problem within their jurisdictions. The same group of agencies (except Prince Edward Island and Ontario) identified that moisture sensitivity was also a problem. Manitoba, Alberta, British Columbia and the Yukon did not believe that stripping or moisture sensitivity was a problem in their jurisdictions. However, British Columbia knows that stripping can occur with some combination of asphalt and aggregate, but use antistripping agents to control stripping. In British Columbia, Tall Oil has reportedly been successfully used as an antistripping agent.

3.5.2 Summary of Current Status

One form of moisture sensitivity or susceptibility, as it relates to asphalt paving mixtures, is commonly termed "stripping". Deme and Palsat (1989) and Yoon and Tarrer (1988) report that stripping of asphalt films from aggregate surfaces occurs when there is a loss of adhesion

between the asphalt cement and the aggregate surface, and is due primarily to the action of water or moisture.

Primary factors that influence stripping are:

- Physical and chemical properties of aggregate.
- Chemical composition, viscosity and surface tension properties of the asphalt cement.

Other factors that may provoke or accelerate stripping are:

- construction practices
- traffic
- physical environment (site conditions)

Roberts et al (1991) suggest five different mechanisms by which stripping of asphalt cement from aggregate particles may occur, namely:

- detachment
- displacement
- spontaneous emulsification
- pore pressure
- hydraulic scouring

Brown et al (1990) have reported recent evidence regarding the involvement of microbes in the process of asphalt concrete degradation (i.e. stripping). Some liquid antistripping agents are rich in the nutrients (nitrates and phosphates) that support these organisms and these antistrip agents are therefore detrimental.

As referenced in Section 3.5.1, some Canadian agencies have found stripping to be a significant problem. Reports from those, and other North American researchers, are referenced below to highlight some factors that may contribute to the stripping problem.

Mineral aggregate comprises approximately 95 percent by weight of commonly utilized paving mixtures. Physical and chemical properties of the aggregate must, therefore, be of significance in respect to stripping potential.

Field (1977) reported on current and previous studies undertaken by the Ontario Ministry of Transportation and Communications. He noted that Ontario Regions that used aggregates which were carbonates had a very low incidence of stripping compared to those that used non-carbonates. He further identified initiatives that had successfully decreased stripping including:

- using antistripping additives
- using a sand asphalt layer between the granular base and first layer of binder course
- designing for lower void mixes
- providing improved drainage in the granular layer and shoulders

Specific aggregate related quality problems that contributed to stripping were:

- improper drying of water absorptive aggregates
- fine dust coatings clinging to aggregate particles
- stockpile age after crushing, where it was postulated that after suitable storage or prolonged heating, surface charges produced by crushing either dissipate or re-orient.

Yoon et al (1988) found that, although the physical properties of an aggregate affected stripping, there was no strong correlation between the physical properties of the aggregate, such as pore volume and surface area, and the stripping propensity of the aggregate. Chemical and electrochemical properties of the aggregate surface in the presence of water were observed to have a significant effect on stripping.

In the past, Manitoba Department of Highways addressed the electrical surface charge issue by requiring that freshly crushed aggregates for hot mix production be retained in stockpile for at least forty-eight hours prior to being used.

Field (1977) also addressed sources of asphalt as a factor related to stripping and reported findings that included:

- some aggregates strip for all sources of asphalt cement,
- some aggregates strip according to the source of asphalt,

Scherocman et al (1985) noted that the degree of moisture damage in an asphalt concrete pavement is a function of a number of variables, including:

- type of aggregate used,
- the source and refining process used to manufacture the asphalt cement,
- properties of the mix such as the air void content,
- environmental and traffic conditions

It is possible that asphalt cement related factors that might contribute to stripping mechanisms defined by Roberts et al (1991) are viscosity (detachment) and composition/processing (spontaneous emulsification).

Mohamed et al (1991) have reported research findings that indicate that asphalt concrete mixes exposed to moisture will be more susceptible to stripping under high temperatures.

It is imperative that asphalt mix designers provide mixture qualities that are competent to minimize the potential for water induced damage to occur. The essential limiting volumetric properties of compacted paving mixtures are well known to most asphalt paving technologists and include air voids, VMA and voids filled. Only after these required qualities or properties are ensured does the issue of water sensitivity become a legitimate concern requiring remedial action. Under such circumstances, use of liquid or solid antistripping additives, or selection of alternative aggregates and/or asphalt cements becomes warranted.

A variety of laboratory test procedures have evolved for evaluating performance of paving mixtures, and for ranking admixtures to eliminate or retard water effects. Test methods that have been developed include:

- boiling tests
- static immersion
- vacuum saturation and immersion

- immersion compression
- Lottman
- Modified Lottman
- Root-Tunnicliff method

Most tests involve three steps:

i) measurement of properties of hot mix specimens prior to water conditioning

ii) water conditioning of companion specimens and remeasurement of the same properties as i)

iii) comparing similar properties of hot mix specimens before and after water conditioning.

The testing matrix is expanded through incorporation of antistripping admixtures and ultimately assessing the relative improvement achieved in those measured test properties. The latter three of the above-listed tests have been developed because their predecessors were not considered severe enough, or proper to simulate in-service conditions. The Lottman procedures utilize freeze-thaw cycling of specimens while submerged in water. The Root-Tunnicliff method creates partially saturated air voids in specifically prepared samples by application of a vacuum, followed by twenty-five hours of water immersion (24 hours @ 60°C and 1 hour @ 25°C). In all cases load bearing capacity is measured in some manner, to provide the relative test data. The Root-Tunnicliff method is generally followed in ASTM D4867 (Standard Test Method for Effect of Moisture on Asphalt Concrete Paving Mixtures).

3.5.3 Synopsis

There is no "foolproof" method at this time for evaluating the physical and chemical properties of either a mineral aggregate or an asphalt cement for the purpose of determining the moisture sensitivity of a paving mixture.

Experience based judgement has historically been used to decide upon the use of antistripping agents. Problems associated with the incorporation of antistripping agents are known to exist when the asphalt cement is overheated or subjected to prolonged retention in the plant storage

tanks. Concerns do exist with respect to pavement life expectancy when occurrences such as these exist.

In respect to asphalt cement related issues, it is possible that high viscosity and premium grade asphalt cements and polymer modified asphalts may prove beneficial.

Nevertheless, it is apparent that a prudent precaution, at the mix design stage, is to include a performance - based laboratory test program. Decisions regarding use and application rates of liquid or powdered antistripping agents are best made well in advance of the start of construction. Additionally, in recognition of the influence of some antistripping agents on the properties of some asphalt cements, some testing should also be done to examine this factor.

SHRP has undertaken considerable research in respect to moisture effects on paving mixtures and pavement structures. Specifically AASHTO (1994) Standard TP6 'Standard Method for the Measurement of Initial Asphalt Adsorption and Desorption in the Presence of Moisture' and TP34 'Standard Test Method for Determining Moisture Sensitivity Characteristics of Compacted Bituminous Mixtures Subjected to Hot and Cold Climate Conditions' have been prepared. The SHRP Superpave Mix Design Manual (1994) specifies that the evaluation of moisture susceptibility of the design aggregate structure at the design asphalt content be performed in accordance with AASHTO Method of Test T283. This procedure is, in essence, the Root-Tunnicliff method described above.

3.6 AGING OF THE ASPHALT/AGGREGATE SYSTEM

3.6.1 Current Significance in Canadian Jurisdictions

Six surveyed agencies identified that aging of asphalt concrete was a concern. Six agencies did not believe that aging was a problem, one was uncertain and another suggested that perhaps aging was a problem. Only one agency stated that aging should be considered as a top priority for which a solution is required.

3.6.2 Summary of Current Status

Aging, or age hardening of asphalt cement, is that change in rheological properties that is undergone during the plant mixing and laydown process, and that continues throughout the service life of the pavement. Rheological tests most commonly used to characterize age hardening are penetration and viscosity tests, before and after artificial aging in the laboratory.

A significant proportion (40 to 60 percent) of the total change in rheological properties of asphalt cement occurs by the time mixing, placement and cooling take place. Rheological changes include viscosity increase and penetration decrease. It is for this reason that it is so important to exercise rigorous quality control throughout the construction cycle.

Age hardening of asphalt cement within a compacted pavement occurs, in part, due to exposure of the asphalt cement to air, water and light. Roberts et al (1991) and Barth (1968) have noted that age hardening occurs because of a number of factors, that include:

- oxidation - the reaction of oxygen with asphalt cement, which is dependent upon the character of the asphalt cement and upon temperature.
- volatization - evaporation of lighter constituents from asphalt cement which is temperature dependent.
- polymerization & thixotropy - inherent to the structural change within the asphalt cement, (i.e. molecular structure).
- syneresis and separation - migration of thin oily liquids to the surface of the asphalt film; selective absorption of lighter components by porous aggregates.

Age hardening of asphalt cement creates pavements that become more brittle and susceptible to both low temperature and load associated cracking as the process advances. Factors that are effective in retarding the rate at which age hardening occurs include:

- selection of quality asphalt cement

- developing the paving mixture design so as to ensure that adequate asphalt film thickness exists, and that the air voids system is proper (i.e. maintain relatively low air voids and appropriate VMA to decrease air and water permeability).
- exercising of rigorous quality control checks during hot mix production, transportation and lay down at the time of construction.

3.6.3 Synopsis

Asphalt cement specifications should include limiting values determined from analysis of residue from the Thin Film Oven Test (TFOT) or Rolling Thin Film Oven Test (RTFOT). Within CAN/CGSB-16.3-90M, ASTM D1754 is specified (Standard Test Method for Effect of Heat and Air on Asphaltic Materials - Thin Film Oven Test). Table 1 of the Standard specifies limiting values by grade of asphalt cement, for:

- % loss in mass
- penetration (as a percent of original penetration)

At this time, the Standard contains what is necessary to control the quality of asphalt cement delivered to the mixing plant. The adequacy of the limiting values and the method of age hardening simulation (i.e. TFOT vs RTFOT) should be reviewed. It is known that the British Columbia Ministry of Transportation and Highways now (1994) requires higher values of percent retained penetration after TFOT, for comparable grades of asphalt cement, than does the Standard, as shown in Table 3.

Alberta Transportation and Utilities (AT&U) specifies asphalt cement aging requirements as a function of asphalt type within a given grade. The current CGSB specification does not specify the "after" TFOT limits as a function of the type A, B, or C. Quebec (MTQ) and Ontario (MTO) either reference or restate the percent retained penetration requirement after TFOT as they are stated in the CGSB Standard.

Specifying agencies should incorporate a specification to control and limit the amount of age hardening of asphalt cement that may occur through mixing and construction operations.

Limiting loss of mass and penetration values should not exceed those values obtained on asphalt cement delivered to the mixing plant when subjected to TFOT. It is known that some provincial highway agencies (e.g., Ontario, Alberta, Saskatchewan) have investigated the asphalt cement characteristics of hot plant mixes shortly after paving. Further initiatives should be contemplated on the basis of this research.

Asphalt mix design related parameters are beyond the scope of both CAN/CGSB-16.3-M90 and this study.

3.7 DURABILITY OF ASPHALT CONCRETE PAVEMENTS

3.7.1 Summary of Current Status

Durability of asphalt concrete pavement is associated with its ability to withstand physical distress that is usually associated with:

a) stresses created during periods of freezing conditions when the pavement surface is in a saturated or partially saturated condition, and

b) traffic

Pavements that are under-asphalted or under-compacted (or both) are usually sufficiently permeable that surface water can enter pores in the surface of the pavement. Expansive forces created by the freezing of water can dislodge particles of coated aggregate, hence the term "ravelling". Traffic accelerates this distress mechanism.

Durability of compacted asphalt concrete pavements is influenced by:

i) adequacy of the plant mixing process and of compaction achieved during construction.

ii) air voids in the compacted pavement.

iii) film thickness, which is a measure of the thickness of asphalt cement which coats aggregate particles.

In a 1986 study undertaken for Alberta Transportation, four references were quoted of highway agencies that had reached conclusions after studying pavement ravelling.

i) Field (undated) reported on Ontario study findings, as follows: *"When the pavement void content is greater than 6% in asphaltic concrete mixes water can permeate quite readily to produce ravelling; consequently, we seldom set our mixes higher than 3 percent lab voids".*

ii) Saskatchewan Department of Highways (undated) concluded, on the basis of work performed in the late 1960's, that: *If the air voids (in the compacted pavement) are over 7 percent one might expect a significant amount of ravelling to occur within one year of paving. It would be desirable to have the mix design air voids in the order of 3 percent. If, then, we compact the pavement on the roads to 96 percent of density we will have approximately 7 percent air voids in the field.*

iii) Kandhal and Koehler (1982) reported that the State of Pennsylvania had determined that premature surface ravelling can be eliminated if the field air void content can be maintained below 8 percent.

iv) The Asphalt Institute (1983) suggests the air voids content in compacted pavements should not exceed 8 percent for dense graded mixes in order to prevent ravelling and disintegration.

Film thickness is a parameter specified occasionally to address both the durability and age hardening distress modes. Principles associated with film thickness were originally developed by Francis Hveem (formerly Materials and Research Engineer, California Division of Highways). A method for determining surface area of aggregate is presented by the Asphalt Institute in Manual Series No. 22 (1983). From that report, the "theoretical" film thickness can be computed. The City of Calgary specifies minimum film thickness values for base and surface course mixes of 6.5 microns and 7.0 microns respectively, using standardized calculation methodology based on the surface area determination procedure referenced above.

It is evident that both age hardening of asphalt cement and asphalt pavement durability are influenced by the same design and construction factors, namely air voids (permeability), VMA, and film thickness, but not necessarily asphalt cement grade.

3.7.2 Synopsis

Prevention of durability associated distress depends primarily on the quality of the asphalt mix design and on the adequacy of both plant production and compaction during construction. It also is dependent upon the stripping potential of the compacted paving mixture which was addressed in Section 3.5. Therefore, quality of asphalt cement and precautions to minimize stripping by methods previously discussed, are both fundamental to providing durable pavements.

3.8 PREMIUM GRADE AND POLYMER MODIFIED ASPHALTS

3.8.1 Current Significance in Canadian Jurisdictions

Although polymer modified asphalts are not yet used extensively in Canada, Quebec currently uses significant quantities. Ontario and British Columbia have reported nominal use. Other agencies have noted that PMAs are used in research applications. It has been noted by Newfoundland that SHRP binder tests are required to measure properties of PMAs. Quebec and Ontario have developed specifications for PMAs.

3.8.2 Summary of Current Status

The choice of asphalt grade and quality, on a project specific basis, is dependent upon the anticipated traffic and the climatic conditions under which the pavement is required to perform. Thus, the probability exists that, in some situations, asphalt cement that complies with Group A requirements of CAN/CGSB-16.3-M90 may require further enhancement to meet project specific demands.

In response to these demands, "premium grade" asphalts, "engineered asphalts" and asphalt cement modification (eg. polymer or latex) were conceived and developed. For purposes of this report, the term "enhanced binder" is used to identify these products of modification of conventional asphalt cement.

In simplistic terms, an enhanced binder would take the form of a group of modified asphalt cements whose temperature susceptibility properties would be superior to those conventional asphalt cements that comply with the properties of Group A asphalts in the current Standard.

The existing CGSB Standard was developed for asphalt cements whose rheological properties could be determined in accordance with established testing protocols such as penetration and viscosity. Enhanced binders, specifically polymer modified asphalts, cannot be reliably characterized by means of conventional rheological tests, as is noted hereafter. Therefore, use of a temperature susceptibility parameter such as Penetration Index (PI) is inapplicable. It is apparent that a new specification, similar in intent to the existing Standard, is necessary. However, development of such a Standard is beyond the current study scope.

Polymer modifiers have been classified generically by Terrel and Epps (1988) as shown in Table 4. Polymer modified asphalts have become increasingly popular as enhancers of pavement performance. Much of the published literature is currently devoted to SBR and SBS polymers identified in Table 4.

Some researchers including Khosla and Zahran (1989) and Nahas et al (1990) have reported properties of asphalt polymer blends in terms of conventional rheological tests, including viscosities, penetration and ring and ball softening point. However, King et al (1993) have more recently noted: *" it has proven difficult to evaluate modified asphalts using common asphalt cement test procedures. More and more experimental evidence suggests that traditional tests (penetration, viscosity, ring and ball softening point, ductility) historically accepted as indicative of asphalt cement performance are very poor surrogates for measuring binder stiffness at temperatures where pavements fail (rutting, cracking, etc). Although relationships long established for pure asphalt cement continue to hold, applying the same criteria to the broad array of asphalt modifiers may have catastrophic consequences, because these procedures do not accurately describe the mechanical behaviour of modified materials".*

At least three Canadian specifying agencies have developed specifications for polymer modified asphalts. Quebec (MTQ) specifications include conventional tests (penetration, kinematic

viscosity and retained penetration after TFOT), as well as softening point, RTFOT, elastic recovery and stability tests. Ontario Provincial Standard Specifications (OPSS 1155) specify test standards that are conventional in scope. The City of Calgary, Alberta has a specification that includes a mix of conventional tests, together with stiffness modulus requirements using the sliding plate rheometer, and toughness and tenacity tests.

3.8.3 Synopsis

There is a definite need to develop improved quality hot-mix asphalt concrete pavements for the most severe service conditions (traffic and environment). Solutions to low temperature cracking have been developed in Canada using asphalt cements whose properties can be specified by traditional tests procedures and relationships. It has also been discussed in Section 3.3 that aggregate characteristics are the primary factor which influences pavement performance at high temperature. It is recognized, at the same time, that pavement performance at high temperatures can be enhanced when asphalt binder with superior high temperature stiffness properties (eg. an enhanced binder) is incorporated with a high quality aggregate. For this reason it is desirable to have appropriate enhanced binder specifications in a CGSB Standard for this purpose.

3.9 RELATIONSHIP OF PAVEMENT PERFORMANCE TO ASPHALT PROPERTIES

In Sections 3.2 to 3.8 inclusive, the significance of rheological properties of asphalt cement upon the main pavement distress mechanisms was discussed. For each type of distress, there exists one or more factors that would influence the performance of a well designed and constructed pavement. However, mitigation of one distress mechanism using specific asphalt cement selection criteria may well compromise the pavement in resisting one or more of the other potential distress mechanisms that may well exist.

Table 5 illustrates sensitivity of the primary pavement performance variables in relation to changing asphalt cement and mix properties. Table 5 is similar in concept to an illustration

made by SHRP researchers (1990). The sensitivity tendencies shown in Table 5 should be interpreted as explained therein.

Each of the primary performance variables should be considered in relation to the anticipated traffic level to be accommodated by the pavement structure. In some instances, the most significant performance variable will change as the traffic level increases from a relative low volume to a relative high volume, as illustrated in Table 6.

It is important to remember that the process involved in selecting the most appropriate grade of asphalt cement for a specific project, using CAN/CGSB-16.3-M90, is relatively restricted in that it is improbable that more than two (or at most three) candidates would evolve from the candidate identification process. It is fundamental to the selection process that temperature susceptibility will be the most significant identifying criteria. Concepts illustrated in Tables 5 and 6 would be utilized thereafter to determine the most appropriate asphalt grade and group. The ultimate selection must still be made on a relative "qualitative basis", since, in spite of the wealth of research literature available, it is still not feasible to rank priorities in anything like a numerical ranking or weighting order. Performance testing of asphalt-aggregate blends is the logical final step to verify the quality of the paving mixture.

4.0 CANADIAN CLIMATIC CONDITIONS

4.1 INTRODUCTION

A discussion of past Canadian pavement performance is provided in Section 3 of this report. The effects of prevailing climatic (ambient temperature) conditions upon pavement performance were described. Some of the previous research, which was for the purpose of mitigating the influences of the prevailing temperature regime, has been identified. Existing methodology was described for selection of paving mixture constituents, on the basis of those temperature conditions which prevail at a specific project location.

This Section is devoted to describing methodology which should be included in selecting the minimum and maximum design service temperatures under which a specific pavement should be designed to perform satisfactorily. Discussion is included of previous related research and development initiatives, as well as the current status of relevant SHRP protocols.

Climatological data has been compiled by Environment Canada from hundreds of recording stations throughout the country. Types of data that are available are identified below, as is means by which this data may be accessed.

As part of the SHRP research program, climatological data has been amalgamated into a database. Data from Environment Canada is included in the database for Canadian users. This database contains the average of each winter's coldest day, the seven day average high temperature, standard deviations of these two average statistics, as well as location information.

4.2 MINIMUM DESIGN TEMPERATURE

It is characteristic of an asphalt concrete pavement that it may develop transverse cracks at some seasonal low temperature. The cracking temperature is primarily a function of the temperature susceptibility properties of the asphalt binder, and secondarily a function of pavement thickness, inter-facial bonding of layers and its age.

Alternative methodologies have been described to estimate pavement cracking temperature, on the basis of the properties of the asphalt cement. Irrespective of the methodology chosen to estimate pavement cracking temperature, the designer must still select a design temperature for a specific project, such that the risk of low temperature transverse cracking is maintained within acceptable performance criteria. Once this design temperature is determined, selection of the most appropriate asphalt binder can be undertaken.

The Steering Committee for TAC Project 92-4 mandated (Minutes of Meeting October 22, 1994) that the TAC User Guide be developed in the following manner, specific to pavement low temperature design: -

i) pavement temperature at surface, as estimated from air temperature, is to be used.
ii) the SHRP Superpave air temperatures are to be used as the basis for design.
iii) a formula is to be proposed for determining the relationship between air and pavement surface temperature, based on a review of existing Canadian work done by Deme, Robertson, Christison and others and an analysis of the most currently available data from the C-SHRP test roads.

Part (iii) of the mandate was predicated on advice from TAC that FHWA had adopted Robertson's recommendations. This was later determined not to be the case, and as of the date of completion of this report, is still not the case. As of May, 1995 when the SHRP Superpave Binder Selection Program (SHRPBIND, Version 2.0) was issued, the low design temperature, i.e. the minimum pavement temperature at the surface, is still stated as being equal to the minimum air temperature. The SHRPBIND program is discussed in Section 4.4.1. Inclusion of the foregoing information is necessary preamble to understanding of the narrative which follows.

Several researchers have previously developed relationships between air and pavement temperatures, using data collected at test sites, including Ste. Anne (Manitoba), Washington (D.C.) and Ponoka (Alberta). However, while a wealth of air temperature data is

available, only relatively small quantities of pavement temperature data exist. None has been collected over long term cycles.

Deme et al (1975) established the relationship between minimum daily ambient temperature and the daily minimum pavement temperature for varied depths in Ste. Anne test road pavements. They warned, however, that the relationship will vary from one climatic area to another, depending upon temperature cycling characteristics. They also noted that heat transfer computation techniques developed by Christison et al (1972) were also available.

Robertson (1987) restated the relationship of Deme et al for minimum pavement surface temperature (Ts) and minimum air temperature (T_{air}), in °C, i.e.: -

$$Ts = 0.859\ T_{air} + 1.7°C$$

He noted also that the minimum temperature below the surface of a pavement changes more with depth in a colder climate because the temperature far below the pavement is not affected by the air temperature. He used the assumption that the minimum temperature at any depth in a pavement is proportional to the minimum air temperature, to relate the minimum temperature at various depths (in a pavement) to the minimum air temperature by: -

$$T_{min} = 0.859\ T_{air} + (0.02\text{-}0.0007\ T_{air})D + 1.7°C \qquad \text{---- Equation 1}$$

where: T_{min} is the minimum temperature at depth D, °C

D is the distance below the surface, mm

T_{air} is the minimum air temperature during winter, °C

Guidelines contained currently in Superpave (1994), for low temperature considerations are:

i) the pavement surface is the critical location, to prevent cracking that initiates at the surface.
ii) pavement temperature for low temperature design is equal to air temperature.

It is this second consideration that is currently in dispute and an alternative has not been accepted by FHWA. It is recognized that the existing SHRP algorithm does not recognize the heat transfer which occurs from the relatively warmer pavement structure (i.e. the earth) to the colder air mass. Further, it has long been known that, under Canadian conditions at least, pavement surface and air temperatures are not the same under cold winter conditions.

Robertson (1994) modified Equation 1 (above) in order to enable estimation to be made of the minimum pavement surface temperature, with known reliability, as follows: -

$$T_{design} = 0.859\,(T_{air} - n\,\sigma_{air}) - n\sigma_p + 1.7 \qquad \text{---- Equation 2}$$

where:

T_{design} = winter design temperature, °C.

T_{air} = mean of minimum air temperatures at the pavement site, °C.

σ_{air} = standard deviation of minimum air temperature, °C.

σ_p = standard deviation of the estimated pavement temperature, °C.

n = multiplier associated with the desired reliability for the design temperature.

Following reliability factors (n) proposed by Robertson are: -

Reliability	**n**
90%	0.48
95%	0.75
97.5%	1.0

Young (1981) published temperature data collected at the Ste. Anne test road site between January, 1968 and December, 1969. Monthly maximum, minimum and mean temperature values for ambient (air) and pavement surface temperatures were included in the data.

Air and pavement temperature statistics for the three C-SHRP test roads (Lamont, Hearst and Sherbrooke) exist in the Final C-SHRP report (1994). It is important to note that pavement

instrumentation (thermistors) at these sites were installed at 12, 33, 66 and 100 mm depths from the pavement surface. Pavement surface temperature is, therefore, represented by values recorded at a depth of 12 mm in the pavement.

Christison (1994) presented air/pavement correlations developed from analyses of data compiled at two of the three C-SHRP sites, and from the Ste. Anne and Ponoka test roads. He summarized his analyses as follows: -

i) For the C-SHRP test road at Lamont, Alberta, minimum asphalt pavement temperatures (surface as represented by data at 12 mm depth) are, on average, 0.7 times daily minimum air temperatures.

ii) For the C-SHRP test road at Sherbrooke, Quebec, minimum asphalt pavement temperatures are, on average, 0.67 times daily minimum air temperatures.

iii) For the Ponoka, Alberta test site, regression analysis relating daily minimum pavement surface (T_{surf}) and air temperatures yielded the expression: -

$$T_{surf} = 0.53 + 0.780\ T_{air}, (°C)$$

Following additional data were provided: -

- Daily mean air temperature ($\bar{x}$) = -11.5°C; σ = -10.2°C
- Daily mean surface temperature ($\bar{x}$) = -8.6°C; σ = -11.4°C
- $r^2 = 0.94$
- Sey = 2.31°C (i.e. standard error of the mean of y, i.e. pavement surface temperature)

iv) For the Ste. Anne test road, regression analysis similar to that used for the Ponoka site yielded the following expression: -

$$T_{surf} = 0.29 + 0.765\ T_{air}\ (°C)$$

Following additional data was provided: -

- Daily mean minimum air temperature ($\bar{x}$) = -16.7°C; σ = -6.4°C
- Daily mean minimum surface temperature ($\bar{x}$) = -12.8°C; σ = -8.8°C
- $r^2 = 0.96$
- Sey = 2.42°C

Christison concluded his analysis by combining results from the above four instrumented sites. In doing so, he used a minimum temperature equal to (T_{surf} - 2 Sey) for design, and presented the following expression: -

$$(T_{surf} - 2\ Sey) = 0.72\ T_{air} - 5.0\ (°C)$$

where: T_{air} is the daily minimum air temperature

Robertson reported that he calculated winter design temperatures (T_{design}), for the three C-SHRP sites, using Equation 2. In doing so he utilized a value of 2.5°C to represent the standard deviation of the estimated minimum pavement temperature. He noted that this value is the upper limit found by Christison for sites with severe winter climates. Indeed, this is in close agreement with values of Sey reported above, as determined by Christison at Ponoka and Ste.Anne.

It was previously noted that C-SHRP pavement temperatures were measured at 12 mm from the surface, but not at the surface. Robertson has introduced another equation for T_{design}, and which introduces the variable D (depth in the pavement, mm). This equation is: -

$$T_{design} = 0.859\ (T_{air} - n\ \sigma_{air}) + (0.02 - 0.0007(T_{air} - n\ \sigma_{air}))\ D - n\ \sigma_p + 1.7\ (°C) \qquad \text{---- Equation 3}$$

Following tabulated data (T_{air}, σ_{air}, σ_p) was used by Robertson to determine T_{design} values, using Equation 2, for the three C-SHRP test sites. Comparative T_{design} values have been calculated using Equation 3 (D=12 mm) as a part of the current assessment and are tabulated as well.

C-SHRP Site	T_{air} °C	σ_{air} °C	σ_p °C	T_{design} (after Robertson, °C) (Equation 2)	(Equation 3)
Lamont	-39	4.7	2.5	-38.3	-37.8
Hearst	-41	2.8	2.5	-38.4	-37.9
Sherbrooke	-35	2.7	2.5	-33.2	-32.7

It is evident, from the above comparative T_{design} values, that the C-SHRP test site database, acquired at a depth of 12 mm in the pavement, may be used to represent pavement temperature at surface within approximately 0.5°C.

Robertson acknowledged that he utilized a relatively small database to develop the above relationships. Data actually used was from Ste. Anne and Washington, D.C. The TAC Steering Committee requested that a larger database, which incorporates C-SHRP site data, be utilized to define the relationship between air temperature and pavement surface temperature. It was further suggested that the linearity or non-linearity of the relationship between air and pavement temperatures be investigated.

An expanded database has been created within the current study, and which includes data from the three C-SHRP sites and Ste. Anne. The database was created by taking the coldest air and pavement temperatures (at D=12 mm) for each month during the period of information availability. Replication of data from similar occurrences has been avoided. That is to say, for example, that while four test sections were instrumented at Lamont, data from only one representative section was included in the database. It should be noted that Christison (1995) used all four.

First and second order polynomial relationships of air versus pavement temperatures were investigated, with the following resultant relationships being determined: -

i) First order ($y = ax + b$)

$$T_{pavement} = 0.863\ T_{air} + 5.4\ °C$$
$$r = 0.92$$
$$r^2 = 0.85$$

ii) Second order ($y = ax^2 + bx + c$)

$$T_{pavement} = 0.0057\ (T_{air})^2 + 1.279\ T_{air} + 12.7\ °C$$
$$r = 0.91$$
$$r^2 = 0.83$$

It is reasonable to suggest that the first order equation is of adequate precision for the intended purpose. For example, when T_{air} is -40 °C, T_{design} is -29.1 °C using the first order equation, and is -29.3 °C using the second order equation. If anything, the second order equation is less precise (as represented by the above r values) and is more complicated for the casual user.

Following analyses have also been performed:

1. The relationship of T_{air} vs. $T_{pavement}$, using the first order equation, has been compared to Robertson's equation 1 (for pavement temperature at 12 mm depth). It is observed that Robertson's equation 1 produces a pavement temperature approximately 3 °C colder at all temperatures than does the first order equation. This is illustrated in Figure 3. Approximately 0.5 °C of this 3 °C difference is associated with the small error that occurs when pavement temperature at D=12 mm is used to represent pavement surface temperature. The real numerical difference between temperature values derived by Equation 1 and by the first order equation is closer to 2.5 °C.

2. The database, because of the way it was created, may contain "paired" air and pavement temperatures that are significantly warmer than the coldest winter temperature. In order to determine if the warmer temperatures affected the derived equation, the SHRP database was used to identify the average minimum cold air temperature and standard deviation at each site. Air temperatures that were warmer than the average plus two standard

deviations were separated from the balance of the database. Separate regression analyses were performed on the two segmented portions of the database. The two resultant relationships are presented in Figure 4. Robertson's Equation 1 is replotted for comparison purposes. Subtle variances exist in the two resultant relationships, but the magnitude is very small. For example, for T_{air} = -30°C, the difference in calculated pavement temperature is less than 1°C when determined in accordance with the equations in Figure 4. Two conclusions may be drawn from Figures 3 and 4, ie: -

i) The data points, as segmented, do not represent the form of a second order relationship.

ii) Segmenting the data does not result in a significantly changed relationship. Thus, the relationship shown in Figure 3 is considered to be appropriate for all cold sites.

3. Christison's previously referenced observation regarding similar trends but unique relationships of air and pavement temperature, at separate climatic sites, has been verified. For each of the four sites included in the current analyses, individual first order regression analyses were performed using data specific to each site. Following relationships are presented and are illustrated in Figure 5.

Site	Equation	r
Hearst	$T_{pavement} = 0.817\ T_{air} + 5.1$	0.92
Lamont[1]	$T_{pavement} = 0.812\ T_{air} + 3.9$	0.95
Sherbrooke	$T_{pavement} = 0.679\ T_{air} + 0.1$	0.96
Ste. Anne[2]	$T_{pavement} = 1.039\ T_{air} + 9.7$	0.88
Combined	$T_{pavement} = 0.864\ T_{air} + 5.4$	0.92

(1) Data from all four instrumented test sections was used for this regression analysis.

(2) Data from the full depth structure was excluded.

The above relationships are consistent with the relationships reported by Christison and confirm that there is some uniqueness at each site.

Although the equations appear dissimilar, the variance in estimated pavement temperature, for any cold air temperature is relatively minor, as indicated in the following tabulation: -

	Calculated $T_{pavement}$ for T_{air} of		
Site	-20°C	-30°C	-40°C
Hearst	-11.3	-19.4	-27.5
Lamont	-12.3	-20.5	-28.6
Sherbrooke	-13.4	-20.2	-27.0
Ste. Anne	-11.1	-21.5	-31.9
Combined (1st order eq'n)	-11.9	-20.6	-29.2
Robertson Eq'n 1 (with D=12 mm)	-15.1	-23.6	-32.1

It may be seen that there is typically less than a 3°C difference between any individual site value and that represented by the first order equation. The combined first order regression analysis has produced an equation that can be used to estimate a design pavement temperature for any site, provided that an appropriate factor of safety is used for selection of air temperature.

It is also noted that Robertson's Equation 1

a) predicts pavement temperature approximately 3°C colder than the first order ("combined") equation, and

b) the predicted pavement temperature is colder than that of any individual site temperature, as predicted by the individual regressions.

At this point it is appropriate to draw the necessary conclusions, from the above narrative, to enable a methodology to be proposed for estimating pavement temperature as a function of any given cold air temperature. These conclusions are: -

i) A first order equation adequately represents the relationship between air and pavement temperatures. It is not necessary to use a second order equation.

ii) While each site should be expected to have its own unique air:pavement temperature relationship, the regression equation derived from the combined database can be considered to adequately represent the pavement temperature over the anticipated service air temperature range.

iii) Robertson's Equation 1 consistently predicts a colder pavement temperature than does the first order equation, over the service air temperature range. The magnitude of this difference is approximately 3°C.

iv) Robertson's Equation 1 produces a somewhat conservative estimation of pavement temperature. However, in light of the risk associated with selecting pavement surface temperature for design purposes, this equation is considered to be appropriate.

Use of Robertson's Equation 1, for design and asphalt binder selection purposes is considered appropriate and is elaborated upon in Section 4.4.2. It is superior to Equations 2 and 3, for reasons that are defined in Section 4.4.2.

4.3 MAXIMUM DESIGN TEMPERATURE

The stiffness properties of an asphalt concrete pavement decrease as the ambient (service) temperature increases. Under such service conditions a pavement has the potential to displace under heavy traffic loadings. It is, therefore, necessary to have a knowledge of those representative elevated service temperature conditions for which satisfactory pavement performance must be accommodated.

The pavement temperature is influenced by air temperature in warm seasons, just as it is similarly influenced in cold seasons. Other factors, including solar absorption, atmospheric radiation and wind speed, also influence the temperature of the pavement. While there exists an abundance of climatological data, with which to establish maximum air temperatures for design purposes, there is a scarcity of data to represent pavement temperature in warm seasons. There is a similar scarcity of temperature data that may be used to correlate pavement and air temperatures. Christison (1994) noted that high priority should be given to development of maximum summer asphalt concrete pavement temperatures.

Shell (1978) has illustrated the effect of temperature variability upon paving mixture stiffness. Figure 6 contains an illustration of this characteristic for two types of paving mixtures, each containing two types of asphalt cement, that are subjected to a loading time of 0.02 seconds (50 km/h). It may be seen that a decrease in stiffness of two orders of magnitude can occur over the temperature range from 5°C to 50°C. For example, mix S1-50 shows a stiffness decrease from 1.2×10^{10} N/m² to 1.9×10^{8} N/m² over this temperature range. This change in stiffness can be enough to initiate rutting.

Included within the terms of reference for the current study was a requirement to develop a concept for selecting the appropriate grade of conventional asphalt cement (as specified within the CGSB Standard) to ensure acceptable pavement performance under the maximum seasonal temperature condition. Such a concept has indeed been developed, and is described in Section 6.5. The concept is based on the work of previous researchers, primarily Finn et al, Shell and van der Poel. The concept is based on a limiting paving mixture stiffness value of 2.38×10^{8} N/m², at 40°C, proposed by Finn et al, relationships between paving mixture stiffness and bitumen stiffness published by Shell, and the classic relationship of bitumen stiffness to temperature, loading time and bitumen properties developed by van der Poel. For the CGSB asphalts (Groups A, B & C), relationships of asphalt stiffness and loading time (vehicle speed) at 40°C were developed and are tabulated in Table 7, and illustrated in Figures 7, 8 and 9. A limiting asphalt stiffness value of 1.5×10^{5} N/m² was evolved in the concept development process, and is utilized in the asphalt selection process depicted in the aforementioned Figures.

Since the development of the above concept was completed, SHRP binder testing protocols have been implemented. There is still only minimal test data available which provides a relationship between SHRP binder grades and some CGSB penetration grades. One available set of such data was developed by Alberta Transportation & Utilities and is presented in Table 8. Some data of a similar type has been reported by Québec Ministère des Transports (1995) and is presented in Table 9.

Use of the concept developed within the terms of reference for this study, and/or the protocol developed by SHRP, for maximum design temperature and asphalt binder selection purposes, is elaborated upon in Section 4.4.3.

4.4 SELECTION OF DESIGN TEMPERATURES

4.4.1 SHRP Database & SHRPBIND

SHRP has created a weather database which exists within the SHRP Superpave Binder Selection Program: SHRPBIND, Version 2.0, May, 1995 (The Program). SHRPBIND was developed by PCS/Law Engineering, A Division of Law Engineering Inc. for the U.S. Federal Highway Administration, Pavement Performance Division - HNR-30. The program includes a database for 5313 weather stations in the United States and 1800+ weather stations in Canada. Location information includes station names, province or state, and the longitude, latitude and elevation of each station. Four pieces of temperature data are provided for each station that are relevant to determination of pavement design temperatures, namely: -

i) DMAT which is the design maximum air temperature, and is the average of at least ten years of yearly maximum air temperature (YMAT), and
YMAT is defined as the yearly maximum 7 day floating average of the daily maximum temperatures.

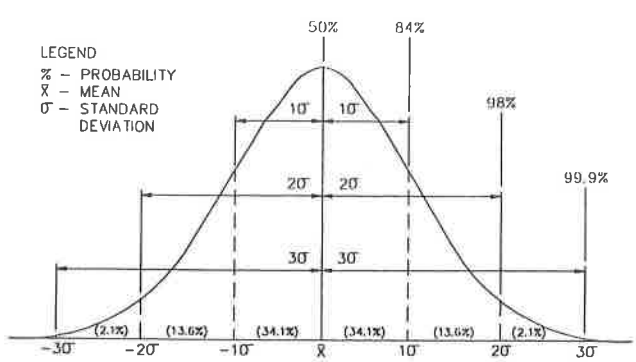
ii) σDMAT which is the standard deviation of at least ten years of YMAT.

iii) T_{air} which is the average of at least ten years of the annual coldest air temperature.

iv) σ_{air} which is the standard deviation of T_{air}

The program has been designed to allow the user to specify the desired level of reliability when selecting a SHRP PG grade of asphalt binder. Binder performance grades corresponding to at least 50 percent and 98 percent reliability levels can be identified for any project location where database information is available to calculate respective pavement design temperatures.

An understanding of statistics principles, that are associated with normal distribution theory, is necessary to appreciate how SHRP has accommodated the issue of reliability, i.e. probability. A normal distribution curve is shown below. The area under a normal distribution curve represents the total number of pieces of data in a dataset. The area under a portion of the curve represents the number of pieces of data (and hence the percent) of the dataset. In the illustration below, the 7-day average maximum pavement temperature situation is represented. For each case of $\bar{x} \pm 1\sigma$, 2σ or 3σ, the percentage of occurrences, and also the probability of temperatures occurring within those limits may be determined. In the high design temperature case, one is only concerned with probability that a temperature occurrence may exceed (i.e. fall to the right) of a definable probability level. Thus, when the mean temperature ($\bar{x}$) is considered, there exists a 50 percent probability that an annual temperature occurrence will not exceed $\bar{x}$. Similarly for $\bar{x} + 1\sigma$ and $\bar{x} + 2\sigma$, there exists 84 percent and 98 percent probabilities respectively, that annual temperature occurrences will not exceed those limits.



The same principles apply for probability considerations for the low temperature design case.

The Program contains supporting technical information to describe how high and low pavement design temperatures are derived.

For low temperature, the surface temperature was assumed to be equal to the air temperature. An equation was developed for the change in temperature with depth for low temperature as follows: -

$$T_{(d)} = T_{(air)} + 0.051d - 0.000063d^2$$

where: $T_{(d)}$ and $T_{(air)}$ are in °C and the depth, d, is in mm

This information is presented for information only and is not used in this report, for reasons that should be evident upon review of Section 4.2.

The high design pavement temperature is determined 20 mm below the layer surface, (i.e. T_{20mm}). Derivation of T_{20mm} is accomplished in two steps. Firstly, the high pavement surface temperature is based upon the principle of net heat flow at the pavement surface. Net heat flow was considered to be a function of direct solar radiation, diffuse radiation, convection, conduction and black body radiation. It is reported that the resulting energy balance is non-deterministic. A deterministic equation was developed, however, that relates temperature difference between surface and air to latitude, with the equation being: -

$$T_{(surf)} - T_{(air)} = -0.00618 \text{ lat.}^2 + 0.2289 \text{ lat.} + 24.4$$

where: $T_{(surf)}$ & $T_{(air)}$ are in °C, and latitude (lat.) is in degrees

By re-arranging the above equation, $T_{(surf)} = T_{(air)} - 0.00618 \text{ lat.}^2 + 0.2289 \text{ lat.} + 24.4$, °C.

(Equation 4.1)

A second equation was developed that expressed the change in temperature with depth, i.e.: -

$$T_{(d)} = T_{(surf)} \times (1-0.063d + 0.007d^2 - 0.0004d^3)$$

where: $T_{(d)}$ and $T_{(surf)}$ are in °F and the depth, d, is in inches

For high temperature design, i.e. T_{20mm} ($T_{0.75\ in.}$), the above expression may be reduced to the form of: -

$$T_{(0.75\ in.)} = 0.9564\ T_{(surf)},\ °F \qquad \text{(Equation 4.2)}$$

The unfortunate intermixing of temperature units in Equations 4.1 and 4.2 requires that the following steps be taken to arrive at values of T_{20mm}.

i) Convert $T_{(surf)}$ as determined in Equation 4.1 °C to °F, using: -

$$°F = 9/5\,°C + 32$$

ii) Compute $T_{(0.75\ in.)}$ using Equation 4.2 and then convert it to $T_{(20\ mm)}$ using: -

$$°C = 5/9\ (°F - 32)$$

4.4.2 Procedure for Selecting Minimum Pavement Design Temperature Value

Methodology has been described in Section 4.2 for determining the appropriate minimum pavement design temperature value on a project specific basis, using data which is contained in existing databases (SHRP or Environment Canada). Sourcing of database information is discussed in Section 4.5.

Figure No. 10 is presented to illustrate the cold air temperature regime in Canada. Values of $T_{air} - 2\sigma_{air}$, for the Canadian weather stations included in the SHRP database, have been used to develop cold air temperature isotherms at intervals of 5°C. This Figure should be used for

information purposes only. It should not be used as a substitute for determining more precise site-specific information.

The designer should be cognisant of the relevance of the data which is available from the database. Factors that should be considered in assessing whether a design temperature, obtained from the database, reliably represents the specific project site include: -

- distance of the nearest recording station from the project site.
- elevation differences between the project site and the nearest recording station.
- microclimate variations within project limits (eg. winter air temperature can vary by 5°C or more in a relatively short distance in mountainous terrain due to elevation changes).

Once the designer has assessed the relevance of the sourced database temperature data, and has applied any discretionary or local knowledge-based adjustment to that data, the critical design pavement surface temperature should be determined.

As concluded in Section 4.2, Robertson's Equation 1 can be used as the basis for selecting the minimum design pavement temperature. Robertson has proposed either Equation 2 or Equation 3 for that purpose. It is not necessary to consider Equation 3 any further, since the difference between Equations 2 and 3 is the depth term, as used in all previous comparisons. When the design pavement temperature is to be determined at the surface, i.e. D = 0, then the two equations are the same.

Equation 2 introduces three variables (n, σ_{air}, σ_p) that are not found in Equation 1. The value n is a reliability factor, and was previously defined. The relationship of n to reliability, as proposed by Robertson, is somewhat confusing to the casual reader. The value of σ_{air} is site specific. It is documented in the SHRP database or can be calculated from historic records. The value of σ_p has not been determined at most sites because of a lack of historic data. A constant value of 2.5°C has been assumed by Robertson, based on Christison's (1994) analyses. In order to use either Robertson's Equation 1, or the first order equation (derived using previously identified data sources), it is necessary to select a design air temperature. This may be simply

accomplished by using existing SHRP database data (T_{air}, σ_{air}). Reliability is addressed through addition of $1\sigma_{air}$ or $2\sigma_{air}$ to T_{air}. In Robertson's Equation 2 acceptance is required of his hypothesis that $\sigma_p = 2.5°C$. This may, or may not, be a valid assumption. The following tabulation of pavement design temperatures at selected sites is provided, as determined by: -

i) First order regression equation using

$$\text{Design } T_{air} = T_{air} - 2\sigma_{air}$$

ii) Robertson's Equation 1 using the same design T_{air} as in i).

iii) Robertson's Equation 2, with n = 1.

Site	Design Pavement Temperature, °C			
	First Order Equation	**Robertson's Eq'n 1**	**Robertson's Eq'n 2**	**Coldest Recorded Pavement Temp.**
Hearst (Kapuskasing)	-34.3	-37.2	-37.1	-28.3
Lamont (Ft. Saskatchewan)	-36.9	-40.3	-38.0	-36.1
Sherbrooke (Sherbrooke)	-29.1	-32.1	-31.2	-26.0
Ste. Anne (Winnipeg)	-36.0	-38.9	-38.0	-38.3

Note: Bracketed nameplaces are the closest weather stations which were used for T_{air}, σ_{air}.

From the above tabulation, Robertson's Equation 1 most adequately develops design pavement temperatures which protect against the coldest recorded pavement temperatures at those sites. It should be noted that the coldest reported pavement temperatures that are tabulated above are based on only two to four years of records. Therefore, design pavement temperatures that are calculated using Robertson's Equation 1 should not be viewed as ultra-conservative. Use of Robertson's Equation 1, and $T_{air} - 2\sigma_{air}$, is recommended as the basis upon which the design $T_{pavement}$ is determined. For pavement design temperature, based on pavement surface conditions, D in equation 1 becomes zero, and the following relationship applies:

$$\text{Design } T_{pavement}\ (°C) = 0.859\ (T_{air} - 2\sigma_{air}) + 1.7 \qquad \text{---- Equation 5}$$

T_{air} & σ_{air} are defined in Section 4.4.1.

The above equation comprises the methodology for determining the minimum pavement design temperature, and is supported by the four conclusions that are presented to summarize the analyses and narrative presented in Section 4.2.

The step-by-step procedure for selecting the minimum design pavement surface temperature is as follows: -

1. Determine the values of T_{air} and σ_{air} from the SHRP database at the recording station in closest proximity to the project site.

2. Validate the values of T_{air} and σ_{air}, using any local data source which may be available, and refine those values if it is considered prudent to do so.

3. Calculate the minimum design pavement surface temperature using Equation 5, and the values of T_{air} and σ_{air} that were determined in Steps 1 and 2.

The procedure for selecting the minimum pavement design temperature contains a mechanism for managing risks that are associated with selecting the most appropriate design temperature value. Within Section 6 discussion is provided of a second issue for which provision of a "safety factor" is strongly advocated. Specifications for asphalt cement that are contained in the CGSB Standard are for asphalt cements in their original (tank) condition, and do not provide for the initial aging process that occurs during asphalt plant mixing. The SHRP binder specification, on the other hand, is based on initially aged binder properties.

4.4.3 Procedure for Selecting Maximum Pavement Design Temperature Value

Methodology has been described in Section 4.3 for determining the appropriate maximum pavement design temperature on a project specific basis, using data that is contained in the SHRP database or that may be obtained from Environment Canada. Sourcing of database information is discussed in Section 4.5.

Figure No. 11 is presented to illustrate the hot air temperature regime in Canada. Values of DMAT + 2σ DMAT (defined in Section 4.4.1) for the Canadian weather stations included in the SHRP database, have been used to develop high air temperature isotherms at intervals of 5°C. This Figure should be used for information purposes only. It should not be used as a substitute for determining more precise site-specific information.

It was noted in Section 4.3 that there is a scarcity of recorded data with which to correlate pavement and air temperatures during hot seasonal service conditions. However, the equations developed by SHRP researchers provide the designer with the ability to estimate the pavement temperature (T_{20mm}) when the seven-day average high air temperature (T_{air}), and the latitude at which the project is located are both known. Both of these statistics are published in the SHRP database, as noted in Section 4.4.1.

It is proposed that the equations presented in Section 4.3 should be used to calculate T_{20mm} at any project site for which T_{air} and latitude are known, or may be reasonably estimated from existing weather stations contained in the SHRP database. The equations, in the form shown in Section 4.3, provides a value of T_{20mm} at the 50 percent reliability level. At the 98 percent reliability level (as defined in Section 4.4.1), T_{air} becomes $T_{air} + 2\sigma_{air}$.

The step-by-step procedure for selecting the maximum design pavement temperature ($T_{20\ mm}$) is as follows: -

1. Determine the values DMAT, σDMAT, and latitude from the SHRP database at the recording station in closest proximity to the project site, and ensure that those values are applicable to the site.

2. Determine the reliability level (i.e. 50%, 84% or 98%) and accordingly compute T_{air} in the following manner.

   For 50% reliability, T_{air} = DMAT

   For 84% reliability, T_{air} = DMAT + 1σDMAT

   For 98% reliability, T_{air} = DMAT + 2σDMAT

3. Calculate the maximum design pavement temperature ($T_{20\ mm}$) using Equations 4.1 and 4.2 using the values for latitude and T_{air} as determined in Steps 1 and 2. This value may also be obtained directly from the SHRPBIND program for sites that are accurately represented by weather stations in the program.

Once the appropriate design temperature value (T_{20mm}) is established, concepts described in Section 6.5.2 may be used to establish asphalt binder stiffness requirements at that elevated temperature. The asphalt binder selection procedure is described in Section 8.2.

4.5 SOURCES OF CANADIAN TEMPERATURE DATA

The SHRPBIND (Version 2.0) program includes a database of 1800+ Canadian weather stations. A printed copy of the Canadian data is maintained by TAC, at the following address:

Transportation Association of Canada
2323 St. Laurent Boulevard
Ottawa, Ontario, Canada K1G 4K6
Telephone: (613) 736-1350
Fax: (613) 736-1395

For more information about the SHRPBIND program, contact:

Mr. Monty Symons
Pavement Performance Division NHR-30
U.S. Federal Highway Administration
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, Virginia 22101-2296
FAX (703) 285-2767

Another source of information is Environment Canada. Environment Canada operates on Atmospheric Environment Services in nearly every province in Canada. Table 10 shows the addresses and current phone numbers of some regional and local Environment Canada offices.

Local sources may exist that may have reliable temperature records available (eg. local airports, agricultural offices, etc). Accessing of such information can sometimes improve the reliability of other existing data.

5.0 SOURCES OF ASPHALT USED IN CANADA

A survey questionnaire was distributed to asphalt producers in Canada. The response to the survey was generally poor. However, the basic questions posed to the asphalt producers were based on a similar survey conducted on behalf of SHRP (1990). Several of the responders to the current questionnaire requested confidentiality. For this reason none of the information from the survey has been presented in summary format. The following paragraphs are based upon responses to the TAC and to the SHRP surveys and other information provided by provincial agencies and asphalt supplier representatives. All references to asphalt are to be interpreted as referring to paving asphalts only.

Crude Oil Sources

Crude oils are commonly classified into four types on the basis of gross chemical composition:

Light paraffin
paraffin - naphthenic
naphthenic
aromatic

Naphthenic and aromatic crude oils contain significant amounts of asphalt and are favoured for asphalt production. These also do not contain high levels of paraffin waxes which are undesirable in crude oils selected for paving asphalt production. Paraffinic-type crude oils used for paving asphalt production can result in the incorporation of paraffin and microcrystalline wax into the asphalt. These latter products, if incorporated into asphalt reduce high temperature viscosity, increase temperature susceptibility and can adversely affect other properties of asphalt. The CGSB specification Group C characterizes some asphalts with higher temperature susceptibility than Group A asphalts.

The amount and molecular makeup of asphaltic materials in different crude oils varies substantially. Consequently, performance of asphalt can be variable, depending on the sources

of the crude oils in the refinery blend. This is because the high molecular weight and highly polar molecules which determine the viscosity and rheological behaviour of the asphalt, are derived directly from the parent crudes. Crude oils favoured for asphalt production are generally described as "heavy". For the purpose of this report, heavy crude oils are defined as having American Petroleum Institute (API) gravities equal or less than 27. The API gravity is an arbitrary scale expressing the gravity or density of a crude oil and is calculated from the following equation: Deg. API = (141.5/Sp. Gr. 60°F/60°F).

The sulphur, vanadium and nickel contents of each crude oil source were requested in the TAC survey. Sixty percent of the refineries responded to the question regarding sulphur and only 40% responded to the questions regarding vanadium and nickel. As was observed in the SHRP survey, the Canadian crudes generally have sulphur contents in excess of 1% although one refinery noted that the range was 0.3 to 3.5 percent. Heavy crudes often contain relatively high amounts of sulphur components. The nickel and vanadium contents noted in the survey are in agreement with the SHRP survey. Crude oils with high heavy metal concentrations are troublesome to refineries. In the refining processes the metals often act as "poisons" to the expensive catalysts used to boost the production of transportation fuel components from crude oils. However, since nickel and vanadium are tied up in asphaltene structures, generally speaking, crudes with higher nickel and vanadium are better for producing paving asphalts than fuel components.

The crude source is variable across Canada. Each refinery tends to have its own supply source. Some of the sources are very stable and have been consistently supplying the refinery for up to 15 years. Some refiners have a more unstable source and have changed crude source in the last 6 months to 2 years. Other refineries chose not to answer this question

British Columbia's only asphalt producing refinery owned by Chevron, at Burnaby, uses light crudes from BC and Alberta to produce paving asphalt. In Alberta and Saskatchewan, most refineries utilize heavy crudes from single sources within these provinces (Figure 12), thereby ensuring consistency of product.

A unique type of heavy crude, bitumen, is found in huge reserves in several areas of Northern Alberta, including Cold Lake, Lindbergh, Primrose, Athabasca/Wabasca and Peace River. Bitumen is a very heavy crude (8-12° API gravity), a viscous material which is recovered by in-situ techniques such as steam injection. At present, most in-situ bitumen is produced from the Cold Lake and the Peace River areas.

In the provinces east of Manitoba, refineries producing asphalt have access to a number of crude oils from United States and offshore sources, depending on cost and availability.

Refineries in Quebec and the maritime provinces use offshore oil supplied by tankers. These offshore oils include heavy crudes from Africa, Mexico, Venezuela and Spain and medium gravity crudes from the North Sea and the Middle East. In eastern Canada, brokers also import asphalt from these sources. In British Columbia, some asphalt is imported from sources located in Central California.

Crude Oil Pipelines

Pipelines and tankers are the prime modes of delivery of crude oil to refineries. The consistency of this supply defines the character of asphalts used by agencies throughout Canada. Generally in eastern Canada there is less certainty by the agencies with respect to the source and specific character of the asphalt produced, whereas west of Ontario, asphalt supply and properties are very stable, as a result of consistent crude sources. Figure 12 illustrates that Canadian crudes are transported by pipeline to asphalt producing refineries in Canada and the U.S.

The following table lists some Canadian crude oils which are transported by pipeline and used throughout North America for the production of asphalt.

CANADIAN CRUDE OILS USED IN ASPHALT PRODUCTION

CRUDE OIL	TYPE
Boundary Lake	Light
Bow River	Heavy
Cold Lake	Heavy
Fosterton-Dollard	Heavy
Gulf Alberta	Heavy
IPSweet	Light
Lloydminster	Heavy
Lloydminster Chauvin	Heavy
Mixed Sweet (MSW)	Light
Peace River	Heavy
Redwater	Light
South East Saskatchewan	Light

Because of the limitations on transporting heavier crudes by pipeline, few asphalts are made solely from one source of crude. For example, in Alberta, Imperial dilutes their heavy crude from Cold Lake by about 30% so that it may be transported. Lloydminster crude is also diluted by about 20% for pipelining. Bow River crude which is a mixture of heavy and medium gravity crudes is collected in the Bow River pipeline system and delivered north to the Interprovincial pipeline system at Hardisty. Fosterton-Dollard crudes are collected in southwestern Saskatchewan. The South Saskatchewan pipeline collects these and transports them to the Interprovincial pipeline at Regina, Saskatchewan.

The Interprovincial pipeline carries most of Canada's heavy crude production east across the prairies. It transports mixed blends of crude through the U.S., south of the Great Lakes to Sarnia, Ontario and Toronto. It receives Cold Lake and Peace River bitumen blends at Edmonton, Alberta; and Lloydminster and Cold Lake and Bow River heavy crudes at Hardisty, Alberta; Smiley-Coleville and Lloydminster oils at Kerrobert, Saskatchewan; Fosterton-Dollard crude at Regina, Saskatchewan; and Weyburn-Middle at Cromer, Manitoba.

The Westcoast pipeline, in British Columbia, transports light Boundary Lake crude from the Fort St. John area to Prince George and on to Kamloops where it terminates at its junction with the Trans Mountain pipeline. The Trans Mountain pipeline originates in Edmonton and carries light crude west to Kamloops. From Kamloops the pipeline carries Boundary Lake light crude and Alberta sweet crude (light) to Chevron's refinery in North Burnaby, British Columbia's only remaining asphalt producing refinery.

Asphalt Production and Refining

Asphalt in Canada is produced in two types of refineries. Integrated refineries are operated primarily to produce transportation and heating fuels in large volumes, as well as petrochemical feedstocks. For such refineries, asphalt is a small percentage of the total parent crude. Asphalt refineries are used mainly for the production of asphalt cement. Within Canada, both types of refineries are present. In the TAC survey, reported asphalt cement production varied from 1.7 to 60 percent of the crude oil processed. Crude oil refinery feed ranged from 6,500 to 160,000 barrels per day.

Atmospheric Fractionation and Vacuum Distillation are the two processes used by the refineries responding to the TAC survey. Vacuum distillation, coupled with atmospheric fractionation, is the most widely used method in petroleum refining and also in paving asphalt production. Solvent refining was not used by any of the refineries responding and air blowing was only used by refineries that produce roofing material.

Atmospheric fractionation consists of pumping crude oil through a heat exchanger where its temperature is raised to about 260°C - 320°C. It is then heated to approximately 400°C by pumping it rapidly through a coiled tube exposed to direct heat in a pipe still or furnace heater. It is continuously delivered to a flashing zone of the distillation tower where the most volatile components are stripped off, while the less volatile components are drawn off as side streams. Atmospheric fractionation produces the material termed "topped crude" which is typically delivered to a vacuum distillation tower for further processing. However, in the case of heavy

crude oils, atmospheric fractionation will directly result in a paving asphalt that is often called "straight run asphalt".

Topped crude contains high boiling, heavy petroleum fractions such as gas oils, lube oil stocks, and asphalts. These materials cannot be distilled off in the atmospheric tower without severe thermal degradation or cracking. Such materials have to be separated under vacuum or reduced pressure which, in effect, decreases their boiling temperature ranges and minimizes exposure to destructively high temperatures.

In the process of vacuum distillation, topped crude is fed through a pipe furnace where it is rapidly heated to 380 to 450°C. Superheated steam is often used to maintain temperature and increase the feed rate. Partial steam pressure also increases volatility, minimizes thermal cracking and aids separation of feedstock components. Hot feed from the pipe furnace is introduced into the flash zone of the vacuum tower where light and heavy gas oils are flashed off, and the asphalt, the highest boiling range petroleum fraction, is continuously removed from the bottom of the tower. An asphalt produced by vacuum distillation after atmospheric fractionation is still referred to as a straight run asphalt.

Asphalt is the only petroleum fraction that is not volatilized during refinery processing, and because of this, it is often referred to as the residual fraction or crude oil residue. The amount and molecular makeup of asphalt materials in different crude oils varies substantially. This means that different refineries using crude oils from different sources have to adjust their operating conditions to produce paving asphalts having the desired properties. Temperature, pressure, amount of steam and rate of crude flow all have an effect on the properties of the resulting asphalt product.

Occasionally intermediate grades of asphalt are produced by blending hard and soft grades of asphalt. Also, some refineries produce a soft grade of asphalt by blending a hard asphalt with a material that is softer than the softest asphalt cement grade.

Because crude oils from different sources are chemically and physically different, some oils yield more asphalt than do others. The following table summarizes the asphalt yields by volume for some Canadian heavy crudes.

TYPICAL ASPHALT YIELDS FROM CANADIAN HEAVY CRUDE (VOLUME %)

Crude	Penetration Grade 80/100	Penetration Grade 200/300
Lloydminster Blend	40	46
Bow River	27	32
Smiley-Coleville	28	33
Fosterton-Dollard	38	44
Cold Lake	65	72
Cold Lake Blend	45	51

This table illustrates that asphalt yield varies depending upon the crude used and that the yield is higher for higher penetration asphalts. Some asphalt is produced from light crude oil when light and heavy crudes are mixed or if "vacuum lower bottoms" are blended. Although many refineries receive heavy crude through pipelines, gravity restrictions for transport require that the crude be diluted by lighter crude or condensate. However, except for such aspects relating to blending or dilution, in Canada, heavy crude operations are generally segregated from light crude operations.

The responding refineries never use residue from other processes or recycled lube oils in the production stream. Most of the refineries do not place additives into their asphalt cements. Occasionally, antistrip additives, polymers and anti-oxidants are added at some refineries.

Questions were asked regarding dewaxing and desalting in asphalt production. All refineries responded that desalting was practised. None of the refineries used a dewaxing process. In fact, only one refinery responded that the wax content of a crude oil was determined.

Asphalt Storage and Distribution

Asphalt cement is produced continuously at some of the refineries responding to the survey. Other refineries only produce asphalt cement during the paving season which lasts from 6 to 9 months.

Refineries will typically store asphalt cement in tanks with greater than 10,000 barrel capacity. The asphalt is stored during the paving season at a temperature of 170°C to 180°C. During the paving season 10 to 20 days is the average length of time that asphalt cement is in storage at the refinery. Inert gas is not used to blanket asphalt cement in storage. Only two refineries responded to questions regarding storage temperatures during the winter season. One refinery allows the asphalt to drop to ambient temperature, while the other maintains a temperature of 120° to 150°C.

Asphalt cement is transported routinely by truck from all refineries. Most refineries routinely or occasionally transport asphalt cement by rail. Only one refinery occasionally transports asphalt cement by barge.

Table 20 indicates that most asphalt is supplied by refineries. Terminals are secondary distribution points for asphalt and oil products. In Canada, terminals are generally remote from refineries and asphalt is supplied to them by tanker or railcar. Asphalt brokers are customers of refineries. They purchase quantities of domestic and offshore asphalt and market their purchases for resale throughout Canada.

Refineries and Brokers

It was noted by most refiners in Canada that asphalt cements are sold in significant amounts to other distributors. In most cases the other distributors either further process the asphalt cement or resell the product. Further processing would include emulsification or modification. All refiners also reported that occasionally asphalt cement is exchanged between refineries when demand dictates. None of the Canadian refineries import asphalt cement.

Table 20 provides summary information on Canadian refineries, paving asphalt suppliers and sources of crude oil and asphalt. The following paragraphs present additional information, based on our present knowledge, for each area or province of Canada:

British Columbia

Until recently, west coast refineries producing asphalt included Chevron's Vancouver refinery, Shell's Shellburn refinery at Port Moody and Imperial's Ioco refinery near Vancouver. Asphalt production from these refineries has been primarily from light crudes such as British Columbia Boundary Lake crude and Alberta Redwater and Rainbow crudes. These light crudes produced Group C asphalts The Shell refinery closed in 1993 and Imperial converted the Ioco refinery to a terminal in 1995. The Imperial terminal now offers only Group A, Cold Lake asphalt.

Husky operates a terminal in Vancouver enabling distribution of a Group A asphalt produced from Lloydminster crude. In 1995, Imperial opened a small railway terminal at Kamloops. From this location Imperial distributes Group A, Cold Lake asphalts supplied from their Strathcona refinery in Edmonton. McTar Petroleum Co. Ltd. is a broker supplying both Group A and C asphalts in British Columbia. Because of the Wet-No Freeze coastal environment British Columbia is the only province which uses CGSB Group C asphalts.

Prairie Provinces

Prairie asphalts are primarily produced from refineries in Alberta and Saskatchewan using a stable number of crude sources. For example, in Edmonton, Alberta, Imperial's Strathcona refinery produces paving asphalt from Cold Lake bitumen. Husky (Lloydminster) uses Lloydminster Crude to produce asphalt and Shell's Peace River, Alberta refinery produces asphalt from the Peace River bitumen. The Moose Jaw Asphalt (Wascana) refinery is located in Moose Jaw, Saskatchewan and produces asphalt from Fosterton-Dollard and Cold Lake crude. Moose Jaw asphalt produces both Group A and B asphalts.

Ontario

Presently asphalt is only made in Ontario at Petro Canada's Clarkson refinery at Oakville. However, Quebec refineries, Petro Canada (Montreal), Shell (Montreal) and Ultramar

(St. Romuald) supply some asphalt to Ontario. Imperial reports that in 1996 their refinery at Nanticoke, Ontario will produce paving asphalt. Crudes used to produce asphalt in Ontario include prairie crudes and offshore crudes. The agency survey indicates that other suppliers include Imperial Oil, Amoco Oil, Husky Oil, Ashwarren International, Bitumar, Canadian Asphalt, IKO Industries, Michigan Marine, Norjohn and McAsphalt.

Quebec

The three refineries producing asphalt in Quebec are: Petro Canada and Shell at Montreal and Ultramar at St. Romuald. These refineries use offshore and Canadian crudes with most recent productions coming from imported crude. The Shell refinery uses Mexican and Venezuelan crudes and vacuum bottoms to produce asphalt. Tankers deliver crude oil to the Ultramar refinery. With tanker delivery such refineries can use very heavy crudes. Bitumar is a broker of asphalt in Quebec. In Ontario and Quebec there are numerous terminals, most operated by asphalt resellers, which serve as local supply points and source asphalt either from Canadian refiners or import asphalt from the United States or offshore.

Maritime Provinces

Newfoundland obtains the majority of its asphalt from Irving Oil's refinery at St. John, New Brunswick and the Ultramar Canada's refinery at St. Romuald, Quebec. Newfoundland Hardwoods Ltd. is a broker supplying asphalt. These refineries and Imperial's refinery at Dartmouth, Nova Scotia service Nova Scotia, Prince Edward Island and New Brunswick. The Petro Canada refinery, at Montreal Quebec also supplies asphalt to New Brunswick. In addition, brokers including Brunswick Asphalt, Bitumar and Barret, supplement this latter provinces asphalt supply.

Testing and Quality Control

Only three refineries chose to answer the questions regarding testing and quality control presented in the survey. Most of the refineries responding use penetration, viscosity, softening point and yield as the criteria to monitor production of asphalt cement. These are the same measures that are used for quality control of the end product. Since these properties are

generally used to specify the asphalt cement, it should not be surprising that these properties are used for quality control.

Two refineries replied to the questions regarding failure to meet specifications of purchasers. In both cases, only rarely did asphalt products not meet specifications.

6.0 ASPHALT SPECIFICATIONS AND SELECTION PROCEDURES

6.1 SYNOPSIS OF QUESTIONNAIRE

The results of the agency survey showed that all responding agencies specify their asphalts on a penetration and viscosity basis. Most use the CGSB Standard or a variation of the CGSB Standard. The variations from the CGSB Standard usually consist of differences in the slope of the lines separating the A, B and C Groups.

All agencies regularly test asphalt cement that is delivered to their projects. Most commonly specified tests are penetration, viscosity, flash point and specific gravity. Kinematic and absolute viscosity tests are measured by Manitoba, Alberta and British Columbia. All other agencies measure only one of the two viscosities. Other tests such as solubility, ash content, ductility, softening point, penetration after TFOT, penetration after RFOT, penetration after plant mixing and viscosity after TFOT are used by individual agencies. Obvious regional trends are identified in Table 1.

6.2 MEASURING TEMPERATURE SUSCEPTIBILITY OF ASPHALT CEMENT BY PI AND PVN

Temperature susceptibility is defined as the rate at which the consistency of an asphalt cement changes with a change in temperature. The two best known methods for determining or characterizing temperature susceptibility in Canada are:

1. Penetration Index (PI)
2. Pen-Vis Number (PVN)

Penetration Index (PI) was first defined by Pfeiffer and van Doormaal (1936) and is discussed in Section 3.2.2. Van der Poel developed a nomograph to determine the stiffness of asphalt cement at any temperature and any rate of loading. Stiffness (or stiffness modulus) is defined as the relationship between stress and strain as a function of time of loading and temperature. Asphalt cement properties which had to be known to use his original nomograph were the

softening point (ring and ball) and penetration index (pen/R & B). Later modifications to these original concepts included:

- Determination of PI by means of penetration values (100 g, 5 sec) at two temperatures, and also the temperature at which the penetration value is 800 ($T_{800\ pen}$) as a replacement for softening point.
- Replacement of R & B softening point with $T_{800\ pen}$ in van der Poel's nomograph.

McLeod proposed PVN to determine temperature susceptibility, at least in part, on the basis that parameters required to determine PVN were normally specification requirements (i.e. penetration and viscosity at either 135°C or 60°C). It has been previously stated that PI and PVN were intended to be numerically similar, so that PVN could be used in conjunction with van der Poel's nomograph. However, it has also been noted previously that both Robertson and Puzinauskas found that PVN and PI are not equal for most asphalts.

During the time that the current CAN/CGSB-16.3-M90 was under development, the Committee had to rationalize a performance based asphalt cement specification on the basis of PI or PVN (or both).

An historical review is provided, in Section 8.1, of the current CAN/CGSB-16.3-M90 (Asphalt Cements for Road Purposes). The task of evolving this Standard began in 1979 when Mr. F.D. Young (Manitoba Department of Highways) was charged with convening a Task Force to meet the objective of creating a more widely acceptable specification. In 1979 correspondence, Mr. Young stated two significant criteria which should desirably be achieved in the specification:

.1 Specifications for asphalt cements should meet end use or performance requirements.

.2 Stiffness of asphalt cements should be considered at both high and low temperatures.

In May, 1988, a Sixth Draft Specification was submitted to the Secretary of CGSB by Mr. I. Deme of Shell Canada. A covering report, prepared by Messrs Deme and Lenters (Shell

Canada) explained rationale which had been used in preparation of the Sixth Draft. Of significance in that report are the following points: -

a) Previous attempts to develop a performance - based asphalt specification on the basis of viscosity at 135°C vs penetration at 25°C had failed. The reason for this failure was that temperature susceptibility (ie rate of consistency change with temperature) in the pavement performance temperature range below 60°C differs from that in the asphalt mixing and compaction range above 60°C.

b) Instead, the Western Canadian (Alberta, Saskatchewan & Manitoba) specification was adapted for use in the new specification because it was a rational, performance based asphalt specification, based on viscosity at 60°C vs penetration at 25°C. That specification allowed, for each penetration grade, the selection of asphalt from three temperature susceptibility ranges, namely A, B and C, where A is the lowest and C is the highest.

c) The Committee had, in the autumn of 1987, agreed to develop an asphalt specification based on:

   i) viscosity at 60°C, which would serve as a performance criterion to focus on pavement rutting, and

   ii) used in conjunction with penetration at 25°C it (viscosity at 60°C) would serve as a means for defining temperature susceptibility to avert/minimize low temperature pavement cracking.

d) An empirical chart was prepared and presented as Figure 2 in the sixth Draft, for use by those who wished to pursue grading asphalt by viscosity at 135°C vs penetration at 25°C.

e) Heukelom's Bitumen Test Data Chart (BTDC) was utilized to graphically estimate Penetration Index (PI) values based on the slope of the viscosity at 60°C: penetration at 25°C line. This was stated to be acceptable for non-waxy asphalts. However, when greater precision is required, PI can be calculated from penetration values at two test temperatures.

It is evident that specified properties for Group A, B and C asphalts were established in the Sixth Draft on the basis of PI and the limiting value of viscosity at 60°C as a function of

penetration at 25°C. Only minor modifications were made prior to issuance of the current Standard. These are discussed in Section 8.

It is worthwhile examining the existing specification requirements, and resultant PI trends. Graphical estimations have been made of PI values, using the Shell BTDC, and which indicate:

i) the line separating Group A and B asphalts is equivalent to a PI of about -1.1

ii) the line separating Group B and C asphalts is equivalent to a PI of about -1.5(±).

iii) the line representing the lower limit of Group C asphalts is equivalent to a PI of about -2.0(±). However, for softer grades, i.e. 150/200, 200/300 and 300/400, the negative PI value increases significantly.

In recognition of the approximations associated with the graphical solution method, K.O. Anderson, P.Eng. Professor Emeritus, University of Alberta, computed PI values at each "node" point, in Figure 1 of the current specification. For this discussion, "node" points are defined as the points of intersection of the lines separating asphalt groups A, B and C with the vertical lines representing specified penetration values (25°C, 100 g, 5 sec.). Anderson's methodology is described, and resultant data presented in a report dated March, 1994, and is presented in Appendix B. It should be noted that SHELL BANDS software was used in the procedure.

The following PI statistics have been computed from tabulated data contained in Appendix B: -

- Node points separating A and B asphalts:

  $\bar{x} = -1.15, \sigma = 0.06$

- Node points separating B and C asphalts:

  $\bar{x} = -1.53, \sigma = 0.04$

- The average PI value for node points on the lower limit of Group C asphalts is approximately -2.0 for grades of 120 pen. (and lower), but increases to -2.58 at the 400 pen. node point.

Thus it is verified that lines on Figure 1 (current Standard) that represent the minimum requirements for Group A and B asphalts are precise and consistent in respect to characterization by PI.

PVN (pen 25°C/vis 135°C) values were computed using the equation contained in Section 3.2.3, and resultant data is presented in Table 11. The PVN determinations were made on the basis of viscosity at 135°C, as that was the basis upon which McLeod originally developed PVN. Professor Anderson's computed PI values are also presented in Table 11 for comparison purposes. The data confirms that:

i) PVN values are <u>not</u> equal to PI values at each node point, and

ii) PVN values representing node points along each line vary measurably from hardest grade to softest grade.

It is interesting to note that while PVN and PI values representing Groups A and B asphalts are very divergent, they actually do converge at the softer grades of Group C asphalt.

The TAC User Guide, and its method of derivation, should be consistent with the objectives which the Committee on Road Materials accomplished with adoption of the current Standard. A rational, performance based asphalt specification is presented in Figure 1 of the Standard and has, as its basis, methodology pertinent to temperature susceptibility in the pavement performance temperature range. The TAC User Guide format and content should be (and is) developed accordingly.

The current Standard does, as previously mentioned, provide for those who wish to specify asphalt cement based on viscosity at 135°C vs penetration at 25°C (i.e. by PVN). Figure 2 of the Standard exists for that purpose. However, as stated by Deme and Lenters: *"asphalt temperature susceptibility classification from Figure 2 cannot be expected to be as precise as from Figure 1 as asphalt temperature susceptibilities for some asphalts differ between the performance and handling temperature ranges".*

Guide users should be warned of this limitation. This issue is discussed further in Section 8.2. Although it is beyond the scope of this project to judge the relative merits of PI and PVN to characterize temperature susceptibility, it is apparent that several recognized researchers have disregarded the concept of PVN. On the other hand, until the advent of the recent SHRP

initiative, PI methodology has retained its credibility, and is expected to continue to do so for conventional asphalts.

6.3 LIMITATIONS OF CURRENT SPECIFICATIONS

6.3.1 Background

It is not within the study mandate to propose revisions to the current edition of CAN/CGSB-16.3-M90. However it is prudent to offer commentary on the existing specifications in respect to two specific factors that may impact upon the future relevance of this Standard, ie:

1) SHRP binder specifications
2) Precision related aspects of test procedures that are integral to the current specifications.

These issues are discussed below.

A major consideration, or limitation, associated with the current CGSB Standard is the fact that the specified properties of asphalt cements are stated in terms of the original (tank) asphalt requirements. SHRP, on the other hand, recognizes that initially aged binder properties more reliably represent the binder (and paving mixture) properties that must be considered in the product selection process. Initial aging (i.e. through the plant mixing and subsequent construction process) must be accounted for in the process of selecting asphalt cements that are specified in accordance with the CGSB Standard. This should be accommodated in the manner described in Section 6.3.4, through introduction of a safety factor that recognizes the significance of the initial aging process.

6.3.2 SHRP Binder Specification

The objectives of the SHRP Asphalt Research Program were focused to deliver two main products:

1) An asphalt binder specification

2) An asphalt-aggregate mixture specification (including an asphalt-aggregate mixture analysis system),

Development of the asphalt binder specification was declared to be the primary objective (even at the expense of the second objective) by the SHRP Executive Committee. Both specifications were to be performance based, in order to obtain satisfactory pavement performance with respect to the pavement distress modes which have been discussed in Section 3 of this Report.

SHRP Contract A-002A (Binder Characterization and Evaluation) had the objective of developing test methods and specification criteria that would be the basis of the new asphalt binder specification. It appears that this mandate evolved from the premise that current test methods and specification criteria are not sufficient to ensure good long-term pavement performance. Those methods and specifications addressed asphalt binder consistency but, in the view of SHRP researchers, did not address specific distress modes or ensure adequate long term pavement performance.

Conventional methods of characterizing the rheological behaviour (stress-strain-time-temperature) of asphalt cement were deemed to be confusing, and to some extent concentrated excessively on an incorrect temperature condition. Anderson et al (1991) have noted that van der Poel first used the term "stiffness" to define the stress-strain-time-temperature properties of asphalt cement.

SHRP researchers have developed the asphalt binder specification with surrogate test procedures which, used in conjunction with specified test data parameters, address the pavement distress modes of rutting, fatigue and low temperature cracking. This is accomplished as follows:

- Rutting - Dynamic shear tests on asphalt binder residue specimens after RFTOT or TFOT.
- Fatigue - Dynamic shear tests on asphalt binder residue specimens after aging in a pressure vessel.

- Low temperature cracking - creep stiffness and direct tension tests on asphalt binder residue specimens after aging in a pressure vessel.

Anderson et al (1993) reported that, in the SHRP methodology, stiffness properties are determined using a bending beam rheometer to measure flexural creep stiffness in the design low temperature range. The limiting stiffness value has been set at 300,000 kPa (300 MPa) at two hours loading time at the specified design temperature. Anderson et al (1993) also noted that the test is conducted 10°C warmer than the design temperature to obtain results in a shorter period of time. SHRP researchers recognized the time dependency of stiffness (creep compliance) in the development of thermal shrinkage stresses, and also that the time dependency of different asphalts varies. They concluded that the shape of the creep compliance, or stiffness, mastercurve should influence the magnitude of the thermal stresses that develop during cooling. Thus an additional specification parameter was devised, which is the absolute value of the slope of the creep compliance (stiffness) curve. This slope value is defined as "m" and is determined after 60 seconds loading time. The m-value controls the shape of the creep compliance, or stiffness, curve and effectively recognizes the spectrum of relaxation times and rheological types of candidate products.

Limiting stiffness values put forward by Canadian asphalt paving technologists (see Section 3.2) are based on van der Poel's nomograph which was developed from conventional rheological test procedures. As has been previously reported by C-SHRP (1994), satisfactory low-temperature pavement performance can be enhanced when existing selection procedures are followed (eg. Robertson, Readshaw, Fromm and Phang).

It would appear necessary or beneficial to corroborate that limiting stiffness criteria, based on Canadian researchers' work, is indeed adequately provided for in the SHRP limiting stiffness value of 300 MPa at two hours loading time (at the design temperature) and that the creep compliance curve parameter is prudent. SHRP limiting criteria are based on flexural creep stress-strain data, using the bending beam rheometer, whereas van der Poel (the basis of CGSB criteria) deduced stiffness on the basis of penetration index associated tests.

There is no doubt that SHRP philosophy, which is based on direct testing of aged specimens, is thorough and addresses the modes of pavement distress in a most thorough manner. SHRP specifications still require validation by field performance of pavements designed in accordance with SHRP protocols.

6.3.3 Precision Aspects of Current Testing Protocols

Requirements for asphalt cement grades are specified in Table 1 of CAN/CGSB-16.3-M90, and are referenced to standard ASTM procedures.

ASTM test methods usually include a "Precision and Bias" statement that defines the acceptability of test data in two forms, ie:

- repeatability - the ability of a single operator to reproduce test values on separate samples from the same source.
- reproducibility - the ability of two laboratories to produce comparable test values on separate samples from the same source.

Most significant of the test methods that are specified in Table 1 of the Standard are:

- Penetration, 25°C, 100 g, 5 sec. (ASTM D5)
- Viscosity @ 60°C (ASTM D2171)
- Viscosity at 135°C (ASTM D2170)
- Thin-Film Oven Test (ASTM D1754) with requirement for testing of residue to ASTM D5

Precision criteria applicable to each of the above standard tests are shown in Tables 12 to 15, respectively. The aforementioned precision statements are of great significance in respect to both quality control (Q/C) initiatives undertaken by paving contractors and asphalt suppliers and the quality assurance (Q/A) initiatives undertaken by specifying agencies. It is possible, and indeed probable, in accordance with good practice, that three different laboratories would regularly test the same asphalt cement. It is important to recognize that, on the basis of multi-

laboratory test data, significant differences can exist in key test properties, and which can lead, for example, to measurable differences in predicted pavement cracking temperatures.

To illustrate, a sensitivity analysis is shown in Table 16 of the influence of penetration values at two temperatures (25°C and 5°C) as determined by two laboratories, and to which the ASTM precision criteria are applied. It should be noted that ASTM D5 does not contain precision criteria for penetration tests performed at other than 25°C. However, it does contain a criteria for asphalts below 50 pen at 25°C, which is reasoned to be realistic and is used in the illustration. Following precision limits or values are referenced from Table 12. Multi-laboratory testing (permissible range of 2 test results):

@ 25°C, pen > 50 8% of their mean

@ 25°C, pen < 50 4 units

PI values were calculated using equations found in Section 3.2.3. Using the Asphalt Selection Chart developed by Robertson (1987) shown in Figure 13, design temperatures were derived for purposes of this demonstration. Cracking temperature is determined as equalling Design Temperature minus 10°C according to Robertson. Five possible combinations of pen 25°C/pen 5°C (and mean values of each) result, for which unique design and cracking temperatures also result. The design temperature ranges from -25°C (PI = -1.73) to -30°C (PI = -1.10) with respective cracking temperatures of -35°C and -40°C. All data points shown in Table 16 are plotted on Figure 13.

It is reasonable to suggest that refiners would be most comfortable in processing asphalt cements within the central portion of any specification range in order to avoid disputes, arising from second or third party quality assurance testing, that are related to precision limitations associated with the testing procedures. It may be demonstrated from the foregoing discussion, that refiner's test data could indicate product compliance, while second or third party testing could indicate noncompliance - with all sets of test data being valid according to ASTM precision criteria.

6.3.4 Aging of CGSB Asphalt Cements

Satisfactory low temperature pavement performance can be jeopardized if CGSB asphalt cements are selected without giving due consideration to the aging of those asphalt cements that initiates during asphalt plant mixing and that continues throughout the service life of the pavement. The effects of asphalt cement aging must be considered at the pavement design stage, and in particular with selection of the CGSB asphalt cement. A procedure is outlined in Section 8 of how the TAC User Guide will assist a designer to select appropriate CGSB asphalt cement(s) for a specific project environment. A safety factor is included in the selection process that is for the purpose of accounting for asphalt cement aging.

Utilization of a safety factor, as described below, has precedent. Robertson (1987) incorporated a safety factor of 10°C into his design chart (Figure 13). In this manner, he related design pavement temperature to cracking temperature, with a 10°C safety factor, in the following manner: -

Cracking temperature = Design temperature -10°C

For example a design temperature of -30°C corresponds to a fracture temperature of -40°C. One therefore selects an asphalt cement on the basis of its ability to withstand cracking at 10°C colder than the design temperature that is specific to the site.

Figure 18 has been created as described in Section 8.2.1. **The predicted cracking temperature isotherms shown in Figure 18 have been developed using original (tank) properties of the asphalt cements as they are specified in CAN/CGSB-16.3-M90. No allowance has been made for the asphalt aging that occurs during plant mixing and that continues under pavement service conditions. A safety factor needs to be applied to account for asphalt aging,** in a manner that is similar to that used by Robertson.

The CGSB Standard does not address the issue of asphalt aging, other than to specify the limiting TFOT loss. To account for the asphalt aging process, the designer must assign a safety

factor to the value of "Design $T_{pavement}$" that is determined from Equation 5 in Section 4.4.2. The resultant temperature becomes the "cracking temperature" value with which to select the candidate asphalt cement(s) in Figure 18.

The magnitude of the safety factor to be applied is logically a function of the extent to which the age hardening process of the asphalt cement is expected to advance. However, to provide a definitive guidelines or design criteria, Figure 18 may be used to predict the change in cracking temperature that might occur when an asphalt cement loses 50 percent of its original (tank) penetration value. By examining Group A asphalts, from hardest to softest grade, a consistent decrease in cracking temperature of 6°C to 7°C occurs when the original penetration value is decreased by 50 percent to account for aging. For example, a 300 pen. (Group A) asphalt cement has a predicted cracking temperature of -41°C. After aging to 150 pen., the predicted cracking temperature is -35°C.

Notwithstanding, Robertson's approach is advocated as a means to defining the "safety factor". **A value of 10°C (or more if regional experience so dictates) should be routinely used to adjust the minimum pavement design temperature (i.e. To a temperature 10°C colder) and to use that resultant temperature to select the appropriate CGSB asphalt in Figure 18.**

6.4 ASPHALT CEMENT GRADES USED IN CANADA

It is apparent from responses to the agency questionnaire that two provinces (New Brunswick and British Columbia) have recently discontinued use of viscosity graded (AC-8, AC-5, AC-20 & AC-10) products. Both of these agencies have adopted CGSB specifications. Table 17 shows the uses of asphalt by provincial/territorial agency in 1993.

It is apparent that the lower penetration grades (80 - 100 or 85-100) are used in significant quantities in Quebec, Ontario and British Columbia. These provinces are considered to have at least some areas where winter temperatures are such that these lower penetration grades would perform adequately. Throughout all of Canada the 150-200A grade appears to be the most widely used asphalt with some higher penetration grades being used in many areas.

6.5 ELEVATED SERVICE TEMPERATURE CONSIDERATIONS

6.5.1 Introduction

While extensive research has been undertaken and guidelines provided to aid in selection of asphalt cement to mitigate low temperature cracking, comparable effort does not appear to have been given to the high temperature service condition. One of the objectives of this study is to give consideration to criteria for selecting and utilizing asphalt binders which would provide superior pavement performance at the high end of the service temperature range. Enhanced binders were to be given consideration within the study. However, the study focus is related to the current CGSB Standard, and not with development of a new CGSB Standard for enhanced binders.

In developing a selection process for conventional asphalt cements specified in the current CGSB Standard, it is important to recognize two significant and related issues, namely

- acceptable low temperature stiffness properties of the asphalt cement should be retained
- the mineral aggregate constituent in the paving mixture plays the dominant role in mitigation of instability rutting, as is referenced in Section 3.2.2. In this respect the aggregate skeleton, grain size distribution and particle shape and texture are all significant.

Published information has been sourced in this study to provide a concept for developing guidelines to be used for selecting asphalt cements in designs that address permanent deformation (instability rutting) on a project specific basis. Repeated emphasis is made of the fact that resistance to permanent deformation is a direct function of the high temperature stiffness properties of compacted paving mixtures, of which asphalt cement is but one component. It remains for others to develop relationships between mixture stiffness, ESAL frequencies and rutting propagation. SHRP (1994) has undertaken this initiative in developing the Superpave process, wherein selection of performance graded asphalt binder is on the basis of climate, traffic speed and traffic level.

6.5.2 Concept Development

As noted elsewhere in this Report, the present CGSB specification was rationalized on the basis that the limiting viscosity value at 60°C would serve as a performance criteria to focus on rutting (i.e. high temperature performance), and used in conjunction with penetration at 25°C, would serve to define temperature susceptibility to minimize low temperature cracking. Researchers have developed low temperature stiffness criteria to avert low temperature cracking.

The objective of the following narrative is to use available published literature to develop a concept that would serve to guide a designer in the asphalt cement selection process for high temperature performance. It is imperative that the designer gives due consideration to aggregate quality aspects of paving mixture design when significant volumes of trucks are to be accommodated. In this regard paving mixture stiffness, at a given design temperature and loading condition, is of primary significance.

SHRP (1990) refers to a guideline proposed by Finn et al that, to mitigate rutting distress, paving mixture stiffness (S_{mix}) corresponding to a Hveem stability value (S) of 35 should be achieved at 40°C. It was further suggested that an S value of 35 corresponds approximately to S_{mix} = 34, 500 psi (2.38 x 10^8 N/m²). In following discussion, procedures are described for correlating the necessary asphalt cement stiffness (S_{bit}) to S_{mix} = 2.38 x 10^8 N/m² at elevated temperatures and under specific loading time conditions. The role played by the mineral aggregate is also demonstrated.

Shell (1978) describes a procedure for determining asphalt cement and mixture stiffness modulii. The procedure requires knowledge of four parameters, namely:

(i) $T_{800\ pen}$ of the asphalt cement
(ii) Penetration Index (PI) of the asphalt cement
(iii) pavement service temperature (T)
(iv) time of pavement loading (t)

Use is made of the van der Poel (1954) nomograph shown in Figure 14 to determine the stiffness modulus of the asphalt cement as a function of (T,t). The Shell manual suggests the typical time of loading (t) is 0.02 seconds, which approximates a road traffic speed of 50 km/h. As well, Shell methodology requires that asphalt cement properties must be either measured or estimated to recognize the initial age hardening which occurs during plant mixing.

Chart M-1 in the Shell Pavement Design Manual illustrates "characteristic relationships between mix stiffness and bitumen stiffness", for two major mix groups. (Bitumen stiffness is that stiffness determined or estimated after plant mixing and laying is completed, or is estimated). Mixture groups are:-

S1 = dense base course types of mixes with average volumetric proportions of air, binder and aggregate, and

S2 = open graded mixes with high air voids and low bitumen contents,

or

dense mixes with low aggregate content and high bitumen content.

It is reasoned that the description provided for S1 types of mixes most closely approximates those properties which characterize typical paving mixtures used in Canada. Figure 15 is a reproduction of Shell Chart M-1. The "limiting" S_{mix} value of 34,500 psi (2.38×10^8 N/m^2) proposed by Finn et al has been plotted on Figure 15 to determine a corresponding initially hardened bitumen stiffness S_{bit} value of 5.0×10^5 N/m^2 for mix group S1.

For Group A asphalts, as characterized in the CGSB Standard, van der Poel's nomograph has been used to estimate values of S_{bit} at a temperature of 40°C and loading time of 0.02 seconds. S_{bit} values have been determined for both as supplied (tank) asphalts and for the same asphalts after initial hardening through the mixing plant, in the following manner:

a) For as supplied (tank) asphalts - for each penetration value, the PI and $T_{800\,pen}$ values computed by the Anderson and Bai method (Appendix B) were used to determine S_{bit} at 40°C, 0.02 sec. loading time.

b) For initially hardened asphalts - the limits of initial hardening are represented by the minimum specified retained penetration from Table 1 of the CGSB Standard.

On the premise that the as supplied PI value does not change significantly, new $T_{800\ pen}$ values were estimated from the Bitumen Test Data Chart. Summary data is presented in Table 18.

From Table 18, it may be seen that initially hardened CGSB Group A asphalt, of 150 penetration grade, has an S_{bit} value of 5.0×10^5 N/m^2 at T = 40°C, t = 0.02 seconds loading time. This corresponds to the S_{bit} value inferred from Chart M-1 of the Shell Pavement Design Manual for Mix Group S1.

Some members of the CGSB Steering Committee have expressed concern with respect to actual initial aging (TFOT) characteristics of some CGSB asphalts. Husky Oil (Calgary, Alberta) provided typical aging data for Husky asphalts that is presented in Table 18. Values of S_{bit} for the CGSB asphalt cements, have been determined, based on initially aged conditions that are noted in Table 18. From the available Husky data, it may be interpreted that these representative asphalts do not initially age to the limits permitted in the CGSB Standard (i.e. under TFOT conditions). Nevertheless, under continuing aging, it should be anticipated that the limiting conditions permitted or specified by CGSB will be attained at some future time, and that the approach used in the present concept development is rational.

A further evaluation has been undertaken to attempt to provide an indication of the role played by the mineral aggregate, within the paving mixture, as it may influence the stiffness (S_{bit}) available from or required of the asphalt cement. A relationship of S_{mix}:S_{bit} for a typical mix has been presented by Edwards and Valkering (1974) and is shown in Figure 16. For the value of $S_{mix} = 2.38 \times 10^8$ N/m^2 proposed by Finn et al, it may be estimated, from Figure 16, that S_{bit} is approximately 8.0×10^5 N/m^2. Deme (1995) has provided further insight as to the significance of aggregate angularity upon the relationship of S_{mix}:S_{bit}. This relationship is illustrated in Figure 17 for Cv = 0.85. In the plastic zone (i.e. lower range of stiffness values) the significance of aggregate angularity should be noted. By plotting $S_{mix} = 2.38 \times 10^8$ N/m^2 (after Finn et al), it may be estimated that $S_{bit} = 1.0 \times 10^6$ should exist, at the critical performance temperature, if the aggregate exhibits inferior angularity. On the other hand, if improved aggregate angularity exists, $S_{bit} = 5.0 \times 10^5$ N/m^2 (approximately) could be accommodated. The foregoing can be

interpreted as requiring that an asphalt cement (initially aged) should possess S_{bit} = 1.0 x 10^6 N/m² if aggregate angularity was poor. If, on the other hand, aggregate of improved angularity was available, asphalt cement (initially aged) with S_{bit} = 5.0 x 10^5 N/m² could be used, i.e. a softer penetration grade can be used with superior aggregates.

From the foregoing, it is reasonable to conclude the following:

- aggregate angularity is of great significance and importance in achieving satisfactory mixture stiffness properties under high temperature conditions.

- a limiting range of asphalt cement stiffness values exists, which in conjunction with aggregate angularity, yields S_{mix} values for a given loading condition, to satisfy the criteria proposed by Finn et al.

- to demonstrate the fundamental asphalt cement selection process, for high temperature design, minimum S_{bit} values could be tentatively set at 5.0 x 10^5 N/m² (for aggregate of excellent angularity) to 1.0 x 10^6 N/m² (for aggregates of inferior angularity), based upon stiffness values of initially aged asphalts.

Van der Poel's nomograph (1954) has been used to determine the stiffness modulus of Group A, B, and C asphalts (CGSB) at 40°C, for three different loading times, i.e.:

1) 0.01 seconds (100 km/h travel speed)
2) 0.02 seconds (50 km/h travel speed)
3) 0.05 seconds (20 km/h travel speed)

Summary data is presented in Table 7. Figure No.'s 7, 8 and 9 contain plots of stiffness at 40°C as a function of loading time (0.01, 0.02 and 0.05 seconds) for Groups A, B and C asphalts respectively, in their original (tank) condition.

McLeod (1975, 1976) presented a nomograph which related S_{mix} for paving structures with specific volumetric properties. This nomograph was an adaptation of an earlier nomograph developed by van der Poel. In developing his nomograph, McLeod noted that he had extended the chart developed by van der Poel. Deme (1994) observed that in extending the chart, McLeod had extrapolated the relationship between bitumen and mix stiffness from the elastic to plastic zone, where rutting takes place. It is therefore considered inappropriate to make any further reference to, or use of, McLeod's nomograph.

The pavement designer would utilize Figures 7 to 9 inclusive to determine which asphalt cements, that will satisfy low design temperature requirements, may be selected to satisfy high design temperature requirements for the pre-determined highway or street operating speed. In the ideal situation, the designer would select the hardest (i.e. lowest penetration) grade which would satisfy both the low and high design temperature requirements. In the event that a conventional asphalt cement is not available to satisfy both service conditions, the designer would have to then consider:

a) a compromise solution, or
b) specifying an enhanced binder.

The detailed asphalt binder selection procedure is described in Section 8.2. It is important to recognize that, ultimately, the most appropriate asphalt binder is selected in recognition of the prevailing hot and cold temperature conditions that are site specific.

A prerequisite to undertaking selection of the maximum design temperature and ultimately the asphalt binder, is to characterize the traffic that will use the pavement. Two issues are significant in this respect, i.e.: -

i) operating speed
ii) design traffic, expressed in numbers of 80kN ESALs.

Three operating speeds (i.e. loading times) are recognized both in the concept developed in this study and in SUPERPAVE. Both classify fast moving traffic as being 100 km/h and slow traffic as being 50 km/h. The speed change such as occurs at controlled intersections is referred to as 20 km/h in the concept developed herein, and as "standing" in Superpave. For practical purposes, these conditions are assumed to be similar. A fundamental decision is initially required as to which of the above three operating speed characteristics is applicable to a project design.

The second traffic characteristic, that of cumulative ESALs to be anticipated in the design life of the pavement, is an input parameter that logically is generated by traffic engineering experts. Superpave recognizes design traffic for Level 1, 2 and 3 mix designs in the following manner: -

Design Level	Design Traffic (80kN ESALs)
1 (Low)	$\leq 10^6$
2 (Intermediate)	$> 10^6, \leq 10^7$
3 (High)	$> 10^7$

The above noted traffic levels are higher than those traditionally used in Canada (see Section 3.3.3), however, to be consistent with the SHRP work, the above noted traffic levels will be used from this point forward in this report and the 'User Guide'.

It is beyond the current scope of the study to address traffic engineering issues relative to determination of design ESALs. Nevertheless, it is recognized that, on occasion, an individual user of the TAC User Guide that is discussed in Section 8.0 may have to determine ESALs as part of the asphalt cement selection procedure. One methodology is provided below. It is a modified Asphalt Institute method, and will probably be referenced in the next revision of the TAC Pavement Design and Management Guide. In that method, the following equation is used:

$$ESAL = AADT * HVP * HVDF * NALV * TDY$$

where:

ESAL	=	Equivalent Single Axle Loads per Lane per Year
AADT	=	Average Annual Daily Traffic (all lanes, both directions)
HVP	=	Heavy Vehicle Percentage (divided by 100)
HVDF	=	Heavy Vehicle Distribution Factor (percent of heavy vehicles in the design lane)
NALV	=	Number of equivalent axle loads per vehicle (Truck Factor)
TDY	=	Traffic Days per Year

Since the above concept development initiative was undertaken, the high temperature binder selection methodology developed by SHRP researchers has been published. This provides an opportunity to evaluate the concept developed in this study. The following discussion is for this purpose.

High temperature binder selection methodology developed by SHRP corroborates the concept developed in this study, in principle if not at exact operating temperatures. This is demonstrated in the following manner: -

i) The SHRP (1994) binder selection procedure, for the high pavement design temperature as a function of operating speed, is straightforward. As the operating speed decreases from one level (100 km/h) to another (50 km/h), the binder grade to be selected increases one temperature range - eg. from PG58-xx to PG64-xx. As the operating speed further decreases from 50 km/h to "standing" (i.e. 20 km/h), the grade to be selected increases one additional grade - eg. from PG64-xx to PG70-xx. Thus the SHRP concept for binder selection, as required to recognize the influence of traffic speed (loading time), is similar to that concept developed independently in this study, as may be seen in Figures 7 to 9 inclusive.

ii) A relationship between penetration grade CGSB and SHRP grade may be approximated using data acquired by Alberta Transportation and Utilities (Table 8) and Québec Ministère des Transports (1995), provided in Table 9. Following relationships are used to demonstrate the principle: -

- For Alberta (from Table 8): -

Pen Grade [1]	**Predominant SHRP Grading**
80 - 100	PG64-xx
150 - 200	PG58-xx
200 - 300	PG52-xx
300 - 400	PG46-xx

[1] All products are either identified or assumed as Group A.

- For Québec (MTQ) (from Table 9): -

  MTQ recognizes three grades or categories of penetration grade, namely: -

  1. Conventionnel
  2. Intermédiaire
  3. Supérieur

  For 80 - 100 penetration grade asphalts, 87.5 percent of the "Intermédiaire" product is classified as PG64-xx, as is 100 percent of the "Supérieur" product.

  For 150 - 200 penetration grade asphalts, 50 percent of the "Intermédiaire" product is classified as PG58-xx, as is 66.6 percent of the "Supérieur" product.

  No data was presented by MTQ for softer penetration grade products.

The MTQ data corroborates that data developed, in part, by Alberta.

iii) A series of temperature isotherms has been super-imposed onto Figure 7, and is re-presented as Figure 7(a). Each isotherm represents a high design pavement temperature condition, and is established in the following manner. The 52°C isotherm (specified to mean a design pavement temperature ≤ 52°C) is used as an example of how the isotherms are developed.

When the high pavement design temperature is less than 52°C:

1. (a) At 'fast' (100 km/h) operating speed, SHRP requires a PG52-xx.

   (b) PG52-xx correlates well with 200 - 300 penetration grade asphalt reported by Alberta and Quebec.

   (c) Assuming that all penetrations between 200 and 300 would be suitable, one point on the ≤ 52°C isotherm on Figure 7(a) lies at the intersection of the

100 km/h line and the 300 penetration curve, i.e. all points to the right of the 300 penetration curve would be suitable for use at temperatures less than 52°C.

Similarly:

2. (a) At intermediate speed (50 km/h) SHRP requires a PG58-xx (i.e. one grade higher).

   (b) PG58-xx correlates to a 150 - 200 penetration grade.

   (c) A second point on the ≤ 52°C isotherm on Figure 7(a) lies at the intersection of 50 km/h line and the 200 penetration curve.

Similarly:

3. (a) At low speed (20 km/h) SHRP requires a PG64-xx.

   (b) PG64-xx correlates to an 80 to 100 penetration grade.

   (c) The third point on the ≤ 52°C isotherm on Figure 7(a) lies at the intersection of the 20 km/h line and the 100 penetration curve.

These three points are joined to create the ≤ 52°C high pavement temperature design line. Other temperature isotherms on Figure 7(a) have been created in a similar manner.

The design pavement temperature isotherms, that were created in the manner described above, can be superimposed onto Figure 8 (for Group B asphalts) and Figure 9 (for Group C asphalts). This has been done to create Figures 8(a) and 9(a) for Group B and C asphalts respectively.

iv) An enhanced binder is required when it is impossible to plot points, using the foregoing methodology, at the intersection of a loading time - penetration grade intersection. This point is demonstrated on the 58° isotherm. It is possible to correlate a PG and a

penetration graded binder at 100 km/h and at 50 km/h, but at "standing" (20 km/h) there is no penetration grade available that correlates to the required PG grade (i.e. PG70-xx) - thus the need for a modified or enhanced binder would be indicated.

v) If this concept was accepted as is, or as statistically strengthened with more test data, the designer would identify a) the design speed and b) the maximum pavement design temperature. He would enter Figure 7(a), 8(a) and/or 9(a), and select any candidate penetration grades that lie to the right side of the project specific design temperature isotherm. Interpolation between the plotted isotherms is feasible. In the event that no penetration grade product is available to satisfy the design conditions, consideration would have to be given to specifying an enhanced binder.

It should be made very clear at this point that there should be no expectation that CGSB graded asphalt cements will be correlated to any particular SHRP binder grades. The previously referenced Alberta and Quebec datasets suggest the possible existence of trends. However, in the absence of alternative proven methodology for selecting CGSB grades of asphalt cement for specified high pavement temperature conditions, SHRP selection protocols have been sourced to support the concept presented herein.

What is significant, in the foregoing, is that SHRP protocols enable somewhat softer penetration grades to be utilized than would be the case under similar conditions, if the concept evolved independently, in this study, was utilized. The two independently developed methodologies are, however, viewed to be complementary.

In summation, the procedure for selecting the asphalt binder, to satisfy high design pavement temperature requirements should be as follows: -

i) Access the SHRPBIND Weather Database or any individual location where suitable high temperature data is available to determine high pavement temperatures at the project site.

ii) Select that temperature (i.e. 50% or 98% temperature) that represents the level of risk that is prudent in consideration of the design traffic level, as defined by SHRP, and stated above. For example, for a Level 1 (Low) traffic condition, i.e. ESALs $\leq 10^6$, it may be acceptable to select the 50% high pavement temperature. At the other extreme, at Design Level 3 (High), i.e. ESALs $> 10^7$, it would be prudent to select the 98% high pavement temperature.

iii) Enter Figure 7(a), 8(a) and/or 9(a) and determine which penetration grade(s), if any, that satisfy the design temperature - operating speed requirement.

The above methodology is utilized in guidelines that are written into the TAC User Guide, and is more fully described therein as well as in Section 8.2.

6.5.3 Future Direction

It is desirable to develop criteria for design of asphalt paving mixtures and for selection of asphalt binder, as a recommended procedure for mitigation of instability rutting at high service temperatures. As a first step, it would be necessary to confirm a strategy by which design criteria would be evolved. Two approaches appear feasible:

1. Consider adoption of SHRP design and specification protocols.

2. Proceed in a manner which makes use of conventional technology, using for example, mixture and asphalt cement stiffness criteria as previously described, or as may be amended after further consideration and/or research.

6.5.4 Other Information

As has been previously noted, polymer-modified asphalts have been used in Quebec and Ontario. Each province has produced its own individual specification requirements.

"Premium" grade or "multi-grade" asphalt cements have been produced. Some relevant test data, representing multi-grade asphalts, has been reported (C-SHRP, 1994).

6.5.5 Synopsis

Conceivably, proposed SHRP protocols and specifications will provide a long awaited solution to the challenge of designing asphalt concrete paving mixtures that will demonstrate improved performance under heavy duty traffic. However, some time will probably elapse before all systems are in place in Canada to take advantages of these initiatives. In the meantime, it is feasible and desirable to provide an interim alternative procedure. Such a procedure would include the following steps:

i) Adopt the limiting paving mixture stiffness value proposed by Finn et al, i.e. 2.38×10^8 N/m^2 at a temperature of 40°C.

ii) Determine the design speed, i.e. the representative travel speed of heavy trucks and buses.

iii) Select candidate asphalt binders (conventional, premium or modified).

iv) The limiting (minimum) asphalt stiffness, at the design condition should be:

   a) for aggregates of excellent angularity - (5×10^5 N/m^2 initially aged)

   b) for aggregates of poor angularity - (1.0×10^6 N/m^2 initially aged)

v) Select aggregates to provide the quality necessary for the specific design.

vi) Prepare laboratory trial mixtures at the optimum volumetric design properties for air voids, VMA and percent voids filled with asphalt. The use of Superpave mix design technology is encouraged wherever feasible.

vii) Determine the stiffness (resilient modulus) of candidate paving mixtures in accordance with ASTM D4123 (Indirect Tension Test for Resilient Modulus of Bituminous Mixtures) and adopt the Finn et al limiting mixture stiffness criteria of 2.38×10^8 N/m^2 at 40°C for performance acceptance.

Selection and acceptance of paving mixtures which contain modified binders (e.g. polymers) could be undertaken using the limiting paving mixture stiffness parameter proposed above.

6.6 QUALITY CONTROL/QUALITY ASSURANCE

This Section is included to remind the individual who may utilize the TAC User Guide that product quality (in this case the asphalt binder) must comply with requirements that are selected by the individual who designed a paving mixture for a specific purpose. The original asphalt binder, at the time of delivery into a contractor's storage tanks, must comply with specified requirements. This task should be viewed as one of "quality control", and is the responsibility of the supplier to provide, and of the recipient (the contractor) to ensure. Historically the ultimate owner (the "agency") has monitored product quality at delivery on many occasions. This aspect of ensuring product quality has typically been well managed in the past.

Ensuring that acceptable asphalt binder properties remain after the asphalt binder has been blended with mineral aggregate, through the mixing plant, has not generally been well managed. Few agencies have written clauses into their construction specifications to ensure that excessive damage to the asphalt binder does not occur through the storage and plant mixing process. Agencies should take positive steps to address this issue. This is the "quality assurance" task. In the most simple form, a quality assurance specification could require that initial aging of the asphalt binder, in the plant mixing process, not exceed that represented by the Thin Film Oven test when performed on original (tank) asphalt specimens. Several Canadian highway agencies have become more proactive in this regard in recent years. Appendix A may be referenced for further information in this regard .

7.0 IMPACT OF SHRP RESEARCH

7.1 SHRP BINDER SPECIFICATION

One of the primary objectives of the SHRP research was to develop test procedures that would directly measure the properties of asphalt binders that would be indicative of their performance in the field. To this end three new pieces of equipment were developed. Equipment was developed to measure dynamic shear (high temperature response), creep stiffness (low temperature response) and direct tension (alternate low temperature). An entirely new specification has been created to specify asphalts based on these new measured properties. Grades within the new specification have a standard nomenclature in the form of PGXX-YY

where: PG stands for Performance Grade, and

XX is the numeric grade of the design high temperature and can be 46, 52, 58, 64, 70, 76, or 82 (expressed in °C).

YY is the numeric grade of the design low temperature and can be -46, -40, -34, -28, -22, -16 or -10 (expressed in °C).

It is beyond the scope of this report to give a detailed description of the SHRP binder tests. Interested parties should reference the latest standard specification. However, the following Sections provide a brief description of each test and its significance to binder selection.

7.1.1 Dynamic Shear Rheometer

The Dynamic Shear Rheometer is a versatile device used in many industries. It measures a fundamental property of the asphalt binder. The rheometer is controlled by a computer program. The intent of the dynamic shear rheometer is to measure the high temperature properties of the asphalt binder.

A specimen of asphalt binder is placed in the rheometer. A test temperature is determined based on in the in-service design temperatures. A dynamic shear strain is applied to the test specimen at a frequency of 10 radians per second. The torque and the angular displacement of the asphalt binder are monitored to determine the complex shear modulus and the phase angle. The

complex shear modulus is a measure of the stiffness of the specimen. The phase angle is a measure of the lag between the imposed stress and the measured strain. The asphalt sample is subjected to the test procedure at successively higher temperatures until the sample fails to pass the test. This direct shear test can be conducted on the original asphalt binder, on the asphalt binder after RTFO or on the residue obtained from the Pressure Aging Vessel. Specifications have been provided for each asphalt sample treatment (see Table 19).

7.1.2 The Bending Beam Rheometer

The bending beam rheometer is an apparatus that subjects a sample of asphalt binder to three point bending. The asphalt binder must be aged in the Pressure Aging Vessel. The apparatus allows the measurement of the flexural creep stiffness of an asphalt binder in the range of -40°C to 0°C.

The device operates by applying a constant load to the centre of a small asphalt binder beam (125 mm long x 12.5 mm wide x 6.25 mm thick) for under 4 minutes. The beam is contained in a constant temperature fluid bath. The asphalt beam is supported on two supports that are 100 mm apart. The load on the beam and deflection over the loading period are monitored by a computer. Both the stiffness and the slope of the stiffness versus time curve are important to the performance of the asphalt cement. Specifications have been set at a maximum stiffness of 300 MPa at 2 hours loading (in order to accomplish the two hour loading a -10°C offset has been established) and the slope of the stiffness versus time curve must be at least 0.30. The asphalt binder is tested at several successively colder temperatures until the sample fails.

7.1.3 The Direct Tension Test

The direct tension test is used to measure the uniaxial failure properties of an asphalt binder specimen in the temperature range of -30°C to 10°C. The asphalt binder must be aged in the pressure aging vessel. The equipment tests "dog-bone" shaped specimens in direct tension at a controlled rate of elongation in a temperature controlled cabinet. The samples are prepared in silicone rubber molds that include plastic inserts required for mounting in the equipment.

The sample is prepared, mounted in the equipment and tested to failure at a given loading rate. This test is not necessary if the bending beam rheometer shows the asphalt cement has a creep stiffness below 300 MPa. This test was developed because some of the modified asphalts that failed the Bending Beam Rheometer tests were considered to perform better than those tests indicated.

7.2 RELATIONSHIP OF SHRP SPECIFICATION TO CGSB

Because the SHRP Specification is based on entirely different tests than the CGSB Standard, there is no basic relationship between the two specifications. The SHRP specification is based on performance testing. The selection of an asphalt binder from the SHRP performance grades is based on a probability model that relates measured air temperature to pavement temperature. The procedures allow for the design for average temperatures or average plus one or two standard deviations. The use of 'average' extreme temperatures results in a probability that the temperature will be exceeded every second year.

Tables 8 and 9 contain results of testing of some asphalt cements used in Alberta and Quebec, respectively, when subjected to SHRP testing protocols and ultimately characterized by the SHRP performance grading methods.

8.0 CGSB USER GUIDE FOR SELECTING ASPHALT CEMENTS FOR ROAD PURPOSES

8.1 DEVELOPMENT OF CAN/CGSB-16.3-M90

8.1.1 Introduction

During the first half of the twentieth century, relatively soft asphalt pavements were constructed, predominantly with liquid asphalts (typically slow curing grades). Exceptions to this situation did exist, but by the 1950's, the demand for higher stability pavements led to increased utilization of harder, penetration grade asphalts.

The only rheological property commonly used during the 1950's to specify paving grades of asphalts was penetration at 25°C. By the mid 1960's, highway agencies in cold climate areas had come to experience severe problems with pavement cracking. This phenomenon was soon diagnosed as "low temperature transverse cracking" - a term which endures today.

The most severe occurrence of low temperature transverse cracking existed in Western Canada and northern Ontario. Highway agencies responded to the dilemma associated with this cracking problem. Individual agencies incorporated a second rheological property, namely viscosity (either at 135°C or 60°C), into their specifications.

Studies were initiated to identify those properties of asphalt cements which caused low temperature cracking to occur, and to formulate strategies to minimize its future occurrence. Major test roads were constructed for this purpose. Most significant of these facilities were constructed in Manitoba (Ste. Anne Test Road), Alberta, Ontario and Saskatchewan. The conclusions reached from such studies led to more stringent specifications being developed by provincial highway agencies which excluded certain asphalts.

The Canadian Government Specifications Board (as it was historically known) standard for asphalt cements evolved much more slowly than did some provincial agency specifications, in

respect to the use of multiple rheology parameters to specify paving asphalts. As late as July, 1977, when CGSB 16-GP-3M was issued in response to Canada's metric conversion program, asphalt viscosity was referenced by notation only. It was only in the context of "Working Temperatures for Petroleum Asphalt Cements" that it was even mentioned at that time.

Over a period of ten years, which culminated in 1989 with the issuance (in draft form) of CGSB-16.3-M89, a revised Canadian Standard was developed for Asphalt Cements for Road Purposes, under the auspices of the Canadian General Standards Board, as it had become known. The Standard was formally issued in 1990 as CAN/CGSB-16.3-M90. It was issued without benefit of a TAC User Guide, which is considered to limit the potential for its optimum utilization. Deme and Palsat (1989) have authored a synopsis of the development of CAN/CGSB-6.3-M.

8.1.2 Development of the Specification

The current (1990) Standard is the product of ten years of effort by leading Canadian experts in asphalt technology who comprised the Committee on Road Materials of CGSB.

A brief synopsis of significant events and developments which occurred over this time frame have been compiled from a number of sources, and is provided in Appendix C. Perusal of this synopsis is encouraged to provide the reader with an appreciation of the depth of the technical deliberations which were necessary to produce the specification in its present form.

8.2 EXPLANATION OF THE TAC USER GUIDE

8.2.1 Background

The TAC User Guide, which is presented in Appendix D, has been developed in accordance with principles that have been previously described herein. C-SHRP (1994) presented a number of recommendations that were pertinent to the asphalt binder evaluation and selection process. Two specific recommendations were made that are relevant to the TAC User Guide. These are described below, together with a comment as to the manner in which they were considered in respect to the TAC User Guide.

1. Agencies should adopt a means of selecting an asphalt cement for a particular project based on its anticipated low temperature cracking performance, using criteria such as those developed by Readshaw or Fromm and Phang. Details include the following.
   a) PI, used to calculate stiffness, must be determined from a minimum of three penetration tests performed at 25°C and at two other lower temperatures that yield a penetration value greater than 10 units.
   b) Asphalt cements that exhibit predicted cracking temperatures equal to or colder than the pavement design low temperature can thereby be selected based on low temperature consideration.

Comment: Readshaw's criteria was utilized, together with PI calculated using pen. 25°C and viscosity at 60°C values, in accordance with Figure 1 of the CGSB Standard, to estimate low temperature cracking properties of the Group A, B and C asphalt cements in their original (tank) condition. Predicted cracking temperature isotherms have been plotted on Figure 1 to create Figure 18. Figure 18 may be used to select the grade of CGSB asphalt cement required to mitigate low temperature cracking when the minimum design pavement temperature is known or estimated, subject to inclusion of a "safety factor", as discussed in Section 6.3.4. Figure 2 has similarly been used to create cracking temperature isotherms shown on

Figure 19 when PVN criteria are used to estimate pavement cracking temperatures. Figure 19 is for illustrative purposes. Its use is not recommended for design. While it is acknowledged that multiple penetration values enables more precise cracking temperature estimations to be made, in most instances, the CGSB Standard does not contain such specified requirements. Figure 20 has been developed to enable asphalt cement properties to be specified, using PI values calculated from penetration values at two or more temperatures, to prevent cracking at any calculated or estimated minimum pavement temperature.

2. Consideration should be given to incorporating two more classes of asphalt cement into the CGSB specification: 'AA' and 'AAA' asphalt cements with PI's of -0.6 and 0.0 respectively.

Comment: The TAC User Guide protocols are to be applicable to the current CGSB Standard. TAC is encouraged to take the initiative to develop new standards that are specific to premium grade and modified asphalt binders. The TAC User Guide is designed to enable a user to identify when project specific circumstances, or conditions, preclude the selection of any of the asphalt cements which are specified within the existing CGSB Standard.

8.2.2 The TAC User Guide

The ultimate objective of this study was the production of a TAC User Guide that is intended by TAC to aid an individual in selecting the most appropriate asphalt cement specified within CAN/CGSB-16.3-M90. The TAC User Guide will be published separately by TAC. A copy of the TAC User Guide is presented in Appendix D.

The TAC User Guide is formatted in a manner designed to provide the reader with an initial description of several important aspects of asphalt cement selection and paving mixture design, including: -

i) Purpose of the TAC User Guide; i.e. to provide the user with a step-by-step procedure to select the most appropriate grade of asphalt cement available within the CGSB Standard. The CGSB Standard and the TAC User Guide are to be utilized together in this process.

ii) Concepts involved in selecting the grade of asphalt cement, including: -
- service temperature range
- traffic volume and loading
- thickness and design of the pavement structure
- characteristics of available aggregates

iii) Asphalt cements available within the CGSB Standard, with an explanation of penetration grades and Groups (A, B, C) and the relevance of temperature susceptibility properties of the matrix of available products. Requirements for inclusion of a safety factor for low temperature design are emphasized.

iv) Pavement distress types and methodology for mitigation of their occurrence as a function of the asphalt cement selection process.

v) Methodology for determining design pavement temperatures using the SHRP database and the relevant equations adopted by TAC for this purpose.

vi) Traffic considerations, and specifically the importance of determining operating speed and traffic levels (expressed in 80 kN ESALs).

vi) Other considerations, including influence of aggregate properties, performance testing of candidate paving mixtures.

vii) Limitations, specifically in respect to materials that are available in the CGSB Standard.

The asphalt binder selection procedure is presented in a step-by-step format, and is enhanced through inclusion of a Flow Chart that is designed to be used also as a Worksheet. This Report provides all the background information that has been compiled to facilitate development of the TAC User Guide. A number of worked examples are provided to assist the user in becoming familiar with the binder selection procedure.

The TAC User Guide contains Figure 10 contained in this Report (Minimum Design Air Temperature Isotherms Based on $T_{air} - 2\sigma_{air}$) and Figure 11 (Maximum Design Air Temperature

Isotherms Based on DMAT & 2σDMAT). These Figures are maps of Canada, upon which the relevant isotherms are plotted at intervals of 5 °C. The user is cautioned that these Figures are illustrative only and should not be used to determine project specific design temperatures.

The TAC User Guide contains a warning or caution that in certain design situations, relating to temperature and traffic, none of the materials specified in the CGSB Standard may be appropriate. Under those conditions, the designer may have to specify an alternative asphalt binder, or consider some compromise to enable a CGSB Standard material to be specified.

The TAC User Guide presented in Appendix D may be reviewed for further information.

9.0 OTHER CONSIDERATIONS

9.1 MIX DESIGNS

The mix design usually does not affect the choice of the asphalt cement that will be used for a project. However, because the properties of the mix and the properties of the asphalt cement are so inter-related, it is difficult to discuss one without discussing the other. The mix design must be performed keeping in mind the critical components of low temperature cracking and high temperature instability rutting. If low temperature cracking is a concern and soft asphalts are to be used in the mix, the mix design must ensure that an adequate asphalt film thickness is maintained to preserve desirable pavement qualities. If a harder asphalt is to be used with high quality aggregates, for rutting prevention, the mix design must assure that the asphalt films are not excessive, resulting in an over-asphalted mix that may rut. SHRP has developed the SUPERPAVE™ system for paving mix design which examines and tests the mix in ways which differ from the traditional Marshall procedure. The SUPERPAVE™ system provides substantial promise for superior performing paving mixtures to be designed.

9.2 CONSTRUCTION Q/C AND Q/A

Construction procedures must be suitable to ensure that the mix is manufactured and placed according to design. Problems with asphalt content, overheating the mix, addition of unsuitable or undefined additives, inadequate compaction and erratic thickness control can result in low temperature cracking, stripping, ravelling, fatigue, or rutting, i.e. all of the distress types which one tries to prevent by careful selection of the asphalt binder.

As a minimum the following items should be monitored and controlled within reasonable tolerances during construction: -

- asphalt content, volumetric properties (air voids) and conformance to mix design
- aggregate gradation
- mix temperature

- mix compaction
- properties of asphalt cement

It is a prerequisite to achievement of superior performing pavements that all parties involved in a project understand and fulfil their obligations for provisions of quality control (Q/C) and quality assurance (Q/A) in a timely and effective manner.

9.3 PAVEMENT REHABILITATION AND RECYCLING

Existing asphalt concrete pavements, which exhibit one or more of the distress features previously discussed, have salvage value. Recognition of this has led to the development of pavement recycling technology. The most commonly utilized techniques in existence today are: -

i) "Cold milling", that produces reclaimed asphalt pavement (RAP), which in turn is a constituent in production of new hot mix asphalt concrete.

ii) Hot in-place recycling, which utilizes a specially designed equipment "train" to soften, remove, re-profile and re-lay the upper portion (usually 75 mm or less) of an existing pavement structure. Properties of the existing asphalt cement and pavement mixture can be enhanced through the addition of rejuvenating agents (aromatic products), asphalt binder or new hot mix.

These techniques, which have been developed to "capture" the salvage value of an existing pavement, require careful consideration as to their applicability on a project specific basis. For example, instability rutting within the upper portion of an existing structure cannot be resolved by simply performing a hot in-place recycling operation.

Provision should be made for a significant level of engineering design when pavement rehabilitation is contemplated at the project specific level. Guidelines and practices, which are presented in this report, are a fundamental component of this engineering design requirement. The design activity, of necessity, should also address matters that are beyond the scope of this study, including aggregate selection, paving mixture design and construction specifications. The designer should recognize the contribution to be made by the layer(s) that result from a rehabilitation activity, as a component of the total pavement reconstruction strategy, including, for example, multiple lift reconstruction.

10. FUTURE DIRECTION

The TAC User Guide, in the form presented herein, represents a significant enhancement to Canadian paving technologists who currently use CAN/CGSB-16.3-M90 to select asphalt cements for paving purposes. Some future refinements to the TAC User Guide could improve upon the precision and reliability of the process of selecting the optimum asphalt binder for project specific purposes. Several of the most significant enhancements are mentioned below, together with initiatives which may be considered to achieve those objectives: -

1. The procedure for selecting minimum design pavement temperature, as represented by Equation 5 (Section 4.4.2), is based on a very reliable, but limited number of datasets. The opportunity exists for TAC to continuously expand upon the existing database, through optimizing the instrumentation investment that has previously been made by transportation agencies in Alberta, Quebec and Ontario who constructed the C-SHRP test sites. Data collection should be continued with the intent being to strengthen, or refine as necessary, that air to pavement temperature relationship represented by Equation 5.

2. The procedure for selecting maximum pavement design temperature, as represented by Equations 4.1 & 4.2 (Section 4.4.1) is based on algorithms developed by SHRP researchers. There exists a scarcity of temperature data of Canadian origin, to demonstrate that the mathematical model, upon which the TAC User Guide's high temperature selection methodology is based, is valid at Canadian latitudes and prevailing climatic conditions. An initiative is desirable to resolve this matter.

3. The procedure that was developed conceptually in Section 6.5.2, for selection of asphalt binders for high temperature performance, requires further consideration and/or refinement. Since it is considered possible that the SHRP performance graded asphalt binder specification may not be widely adopted in Canada in the foreseeable future, the concept presented herein, or some alternative concept, with a similar intent, is needed. A number of technical issues warrant consideration in this regard, including: -

   a) Verification of the limiting paving mixture stiffness (such as has been proposed by Finn et al and referenced in Section 6.5.2) to mitigate permanent deformation at the high pavement design temperature.

b) Development of a definitive relationship for S_{mix}:S_{bit} as defined in Section 6.5.2.

c) Validation of, or alternatively development of, critical S_{bit} values for CGSB asphalt cements, as a function of loading time and temperature, in a manner similar in concept to that illustrated in Figures 7, 8 and 9, which were developed from van der Poel's nomograph (Figure 14).

4. A National Standard for enhanced binders should be developed to enable paving technologists to select appropriate materials when project specific situations preclude selection of asphalt cements which are specified in CAN/CGSB-16.3-M90.

11. CLOSURE

A TAC User Guide has been prepared for use in selecting asphalt cements for paving projects in accordance with specifications contained in CAN/CGSB-16.3-M90. This Report has been prepared to enable selection criteria to be developed and rationalized in accordance with technology that was utilized in developing the CGSB Standard during the 1980's.

Impacts of the more recently completed SHRP initiatives have been addressed herein, to the extent possible within the framework of the original project terms of reference. It would appear that an expedient solution to some of the issues addressed in Section 9.0 would be for Canadian agencies (including TAC) to endorse and embrace relevant SHRP protocols, including the performance graded asphalt specification. This implementation strategy may indeed occur throughout Canadian agencies, particularly when performance validation of the relevant SHRP protocols has progressed significantly.

12. ACKNOWLEDGEMENTS

EBA is indebted to many people and agencies who contributed to the Study. EBA's Project Team is particularly appreciative of the contributions made by the Members of the TAC Steering Committee. Their collective knowledge, experience and enthusiasm was invaluable. The TAC Project Manager, Mr. Chris Hedges, deserves mention for the positive attitude and spirit of co-operation that he brought to the project. The input of Professor Emeritus K.O. Anderson, P.Eng., and Dr. J.J. Emery, P.Eng., at the early stages of the project, is appreciated.

EBA's Project Team members recognize the contributions made to the project by many EBA staff members, and are particularly appreciative of the understanding shown by EBA's senior management that enabled this study to be successfully completed.

REFERENCES

REFERENCES

AASHTO Provisional Standards, January, 1994 Edition.

Anderson, K.O., Hussain, S.R., and Jardine, K.G., "Evaluation of Low Temperature and Permanent Deformation Characteristics of Some Engineered Asphalts", Proceedings of CTAA, 1989.

Anderson, D.A., and Kennedy, T.W., "Development of SHRP Binder Specification", Proceedings of AAPT, 1993.

Anderson, D.A., Christensen, D.W. and Bahia, H., "Physical Properties of Asphalt Cement and the Development of Performance - Related Specifications, Proceedings of AAPT, 1991.

Barth, E.J., "Asphalt Science and Technology", Gordon and Breach Science Publishers Inc., 1968.

Brown, L.R., Pabst, G.S. Jr., and Marcev, J.R., "The Contribution of Microorganisms to Stripping and the Ability of an Organofunctional Silane to Prevent Stripping", Proceedings of AAPT, 1990.

C-SHRP (1993) "Performance Correlation for Quality Paving Asphalts, Literature Review"; EBA Engineering Consultants Ltd.

C-SHRP "Final Report, C-SHRP Project Performance Correlation of Quality Paving Asphalts", EBA Engineering Consultants Ltd., 1994.

Christison, J.T., Murray, D.W., and Anderson, K.O. 'Stress Prediction and Low Temp Fracture Susceptibility of Asphalt Concrete Pavements', AAPT, 1972, pg. 494-523.

Christison, J.T. and Anderson, K.O., "The Response of Asphalt Pavements to Low Temperature Climatic Movements", Proc. Third International Conference on the Structural Design of Asphalt Pavements, Vol. 1, 1972.

Christison, J.T., Alberta Research Council, personal communication to National Coordinator of C-SHRP, April 13, 1994.

Christison, J.T., personal communication to EBA Engineering Consultants Ltd., February, 1995.

City of Winnipeg, "Asphalt Pavement Rutting Report", Research Report RS-89-01, 1989.

Corkhill, J. and Kopvillem, O., "Functional Asphalt Properties and Specifications", Proceedings of CTAA, 1981.

Dawley, C.B., Hogeweide, B.L., Bennett, W.J. and Johnston, A.G., "Instability Rutting; Field Performance of Asphalt Pavement Test Sections in the City of Lethbridge, Alberta"; Proceedings of CTAA, 1991.

Dawley, C.B., Hogeweide, B.L. and Johnston, A.G., "Five Year Performance Assessment of Lethbridge Test Pavements Designed for Instability Rutting Mitigation". Proceedings of CTAA, 1993.

Dawley, C.B., Hogeweide, B.L. and Anderson, K.O., "Mitigation of Instability Rutting of Asphalt Concrete Pavements in Lethbridge, Alberta, Canada"; Proceedings of AAPT, 1990.

Deme, I., Personal communication to C. Dawley, October, 1994.

Deme, I., Young, F.D., and Cully, R.W., 'Low Temperature Performance of Composite Asphalt Concrete Pavements', CTAA 1975, pg. 327-350

Deme, I., Personal communication to C. Dawley, March, 1995.

Deme, I., and Palsat, D.P., "Development of Standard CAN/CGSB-16.3-M Asphalt Cement for Road Purposes", Proceedings CTAA.

EBA Engineering Consultants Ltd., "Asphalt Concrete Pavement Density Study", Engineering Report to Alberta Transportation, 1986-87.

Edwards, J.M. and Valkering, C.P., "Structural Design of Asphalt Pavements for Road Vehicles - the Influence of High Temperatures", Highway and Road Construction, February, 1974.

Emery, J.J. and Johnston, T.H., "Rutting in the Urban Context and Its Mitigation", Proceedings of CTAA, 1987.

Emery, J.J. and MacDonald, A.B., "Asphalt Pavement Rutting Experience in Canada", TAC Seminar Series, 1990.

Field, F., "Mix Design for Bituminous Pavements Method and Interpretation", Undated Report, EBA Library.

Field, F., "Evaluation of Antistripping Additives for Asphaltic Concrete Mixes Effectiveness and Recommended Uses", Proceedings of CTAA, 1977.

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

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

Gervais, F. and Abd El Halim, A.O., "Rutting of Asphalt Overlays: Analytical Field Studies", Proceedings of CTAA, 1990.

Haas, R., "Major Issues and Research Needs in the Asphalt Pavements Area", Proceedings of CTAA, 1984.

Hajek, J.J., and Haas, R.C.G., "Some Factors Influencing Low-Temperature Cracking of Flexible Pavements and Their Measurement", Proceedings of CTAA, 1971.

Hills, J.F., "Predicting the Fracture of Asphalt Mixes by Thermal Stresses", J. Inst. Petroleum Technologists, (1964).

Huber, G.A., and Heiman, G.H., "Rutting of Asphalt Pavements in Saskatchewan: Mix Design Evaluation", Proceedings of CTAA, 1987.

Kandhal, P.S., and Koehler, W.C. "Pennsylvania's Experience in the Compaction of Asphalt Pavements", ASTM STP829, 1982.

Khosla, N.P. and Zahran, S.B., "A Mechanistic Evaluation of Mixes Containing Conventional and Polymer Modified (Styrelf) Asphalts", Proceedings of AAPT, 1989.

King, G.N., King, H.W., Harders, O., Arand, W., and Plancke, P.-P., "Influence of Asphalt Grade and Polymer Concentration on the Low Temperature Performance of Polymer Modified Asphalt", Proceedings of AAPT, 1993.

MacDonald, A.B. and Hogeweide, B.L., "Mitigation of Instability Asphalt Rutting", Proceedings of TAC, 1988.

McLeod, N.W., "The Case for Grading Asphalt Cements by Penetration", Proceedings of CTAA, 1975.

McLeod, N.W., "Reduction in Transverse Pavement Cracking by Use of Softer Asphalt Cements", Proceedings of CTAA, 1968.

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

McLeod, N.W., "Asphalt Cements: Pen-Vis Number and its Application to Moduli of Stiffness", J. Test. Eval. 4, 275 (1976).

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

McMillan, C., and Anderson, K.O., "An Evaluation of the Influence of The Binder on Permanent Deformation of Asphalt Concrete", Proceedings of CTAA, 1988.

McMillian, C. and Palsat, D.P., "Design of Asphalt Concrete Mixtures to Optimize High and Low Temperature Performance in Alberta", Proceedings of CTAA, 1990.

Meyer, F.R.P., Haas, R.C.G., and Hignell, E.T., "The Prediction of Rut Depths in Asphalt Pavements", Proceedings of CTAA, 1974.

Mohamed, E.H., Abd El Halim, A.O., and Kennepohl, G.J., "Influences of Temperature on the Stripping of Asphalt Concrete Mixtures", Proceedings of CTAA, 1991.

Nahas, N.C., Bardet, J., Eckmann, B. and Siano, D.B., "Polymer Modified Asphalts for High Performance Hot Mix Pavement Binders", Proceedings of AAPT, 1990.

Pfeiffer, J.P. and van Doormaal, P.M., "The Rheological Properties of Asphalt Bitumen", J. Inst. of Petroleum Technologists (1936).

Phang, W.A. and Chong, G., "A Performance Study of Thin Asphalt Pavements", Proceedings of CTAA, 1974.

Puzinauskas, V.P., "Properties of Asphalt Concrete". Proceedings of AAPT, 1979.

Québec Ministère des Transports, 1995; Personal Communication from A-M Leclerc,

Readshaw, E.E., "Asphalt Specifications in British Columbia for Low Temperature Performance", Proceedings of the AAPT, 1972.

Roberts, F.L., Kandhal, P.S., Brown, E.R., Lee, D.Y., and Kennedy, T.W., "Hot Mix Asphalt Materials, Mixture Design and Construction", NAPA Education Foundation, First Edition, 1991.

Robertson, W.D., "Temperature Susceptibility Characterization of Asphalts", Proceedings of CTAA, 1982.

Robertson, W.D., Discussion on Paper by N.W. McLeod on "Test Data from Three Ontario Test Roads After 15 Years of Service", Proceedings of CTAA, 1978.

Robertson, W.D., "Selection of Paving Asphalt Cements for Low Temperature Service", PICA-3 Workshop Proceedings, Vol. 1, pp 43-82, 1987.

Roque, Reynolds, Hiltunon, Dennis R., and Stoffels, Shelley M. 'Field Validation of SHRP Asphalt Binder and Mixture Specification Tests to Control Thermal Cracking Through Performance Modelling', AAPT, 1993, pg. 615-663.

RTAC, Rutting Workshop, Proceedings of RTAC, 1986.

Saskatchewan Department of Highways, "Compaction of Asphaltic Concrete", Unpublished Internal Report.

Scherocman, J.A., Proctor, J.J. and Morris, W.J., "Prevention of Moisture Damage in Asphalt Concrete Pavement", Proceedings of CTAA, 1985.

Shell International Petroleum Company Limited, "Shell Pavement Design Manual", (1978).

Shields, B.P. and Anderson, K.O., "Some Aspects of Transverse Cracking in Asphalt Pavements", Proceedings of the CTAA, 1964.

SHRP Report - SHRP-A/IR-90-015 "Asphalt Properties and Relationship to Pavement Performance: Literature Review" (Table 5.1), 1990.

SHRP Report - SHRP-A/FR-91-102 "Current Refining Practices for Paving Asphalt Production", 1990.

SHRP, "The Superpave Mix Design Material for New Construction and Overlays", SHRP-A-407, 1994.

Terrel, R.L. and Epps, J.A., "Using Additives and Modifiers in Hot Mix Asphalt", National Asphalt Pavement Association Quality Improvement Series 114, 1988.

The Asphalt Institute, Manual Series No. 22 "Principles of Construction of Hot-Mix Asphalt Pavements".

Uzan, J., Christison, J.T., Anderson, K.O., "Case Studies of a Permanent Deformation Subsystem for Full-Depth Asphalt Concrete Pavements", Proceedings of CTAA, 1980.

Vallerga, B.A., and Lovering, W.R. 'Evolution of the Hveem Stabilometer Method of Designing Asphalt Paving Mixtures' AAPT, 1985, pg. 243-265

Van der Poel, C., "A General System Describing the Viscoelastic Properties of Bitumens and Its Relation to Routine Test Data", Journal of Applied Chemistry, 1954.

Varady, B., and Pijl, W.J., "Potential Pavement Deformation by Marshall and Hveem Design Criteria", Proceedings of CTAA, 1987.

Western Association of State Highway and Transportation Officials, "Asphalt Pavement Rutting, Western States", 1984.

Yoon, H.Y. and Tarrer, A.R., "Effect of Aggregate Properties on Stripping", Transportation Research Record 1131, 1988.

Young, F.D., "Discussion: Dr. N.W. McLeod's Paper on Rational Asphalt Specifications", Proceedings of CTAA, 1981.

GLOSSARY

GLOSSARY

AAPT - Association of Asphalt Paving Technologists.

Antistripping Agent - a product that is used to improve the bond between asphalt cement and aggregate, for the purpose of improving the performance of water-susceptible paving mixtures.

Asphalt Binder - a generic term, used in this Report, to refer to the group of asphalt cements and enhanced binders that are used in production of asphalt concrete paving mixtures.

Asphalt Cement - a product refining of crude petroleum that is used in production of asphalt concrete paving mixtures. As used within this Report, asphalt cement refers to products specified in CGSB/CAN-16.3-M90.

Asphalt concrete pavement — one or more courses of asphalt - aggregate mixtures produced through thoroughly controlled hot-plant mixing processes and placed and compacted into a uniform, dense mass in accordance with recognized design and construction practices.

Asphalt pavement structure — a pavement structure with all its courses of asphalt - aggregate mixtures, together with a combination of stabilized and/or untreated aggregate base and subbase courses, placed above the subgrade.

Bitumen Test Data Chart (BTDC) - a nomograph developed by Shell Petroleum Company Limited that is used to characterize the temperature susceptibility of asphalt cement on the basis of its Penetration Index property.

C-SHRP - The Canadian adjunct to SHRP, whose activities have been co-ordinated and administered by the Transportation Association of Canada (TAC).

CGSB - Canadian General Standards Board.

CTAA - Canadian Technical Asphalt Association.

Pen-Vis Number (PVN) - a method of expressing temperature susceptibility of asphalt cements quantitatively on the basis of penetration at 25°C and viscosity at either 60°C or 135°C.

Penetration - a term used to define one rheological property of asphalt cement, and is derived from an empirical test used to measure the consistency of asphalt cement.

Penetration Index (PI) - a term used to express temperature susceptibility of asphalt cements quantitatively on the basis of penetration measurements made at two or more temperatures. PI may also be estimated for most asphalt cements using penetration value at 25°C and viscosity at 60°C and a Shell nomograph referred to as the Bitumen Test Data Chart (BTDC).

Rejuvenator - a product used to restore some of the original properties of asphalt cement that has oxidized within an asphalt concrete pavement.

Rheology - the science dealing with the deformation and flow of matter.

SHRP - The Strategic Highway Research Program, which is a unit of the National Research Council (U.S.A.) that was authorized by Section 128 of the Surface Transportation and Uniform Relocation Assistance Act of 1987.

Stiffness (Stiffness Modulus) - the relationship between stress and strain of asphalt cements, or paving mixtures, as a function of time of loading and temperature.

Stripping - the loss of adhesion between asphalt binder and aggregate in a paving mixture.

Superpave™ - an acronym for Superior Performing Asphalt Pavements, which is a product of SHRP, and is essentially a mix design and evaluation system for new construction and overlays.

Temperature susceptibility — the rate at which the consistency of an asphalt cement changes with a change in temperature.

TABLES

TABLE 1
RESULTS OF SURVEY OF TYPE OF TESTS CONDUCTED ON ASPHALT CEMENT

TEST	No. of Responses			COMMENTS
	Yes	No	N/A	
Penetration @ 25°C	13	0	1	
Penetration @ other temp.	4	9	1	
Specific gravity	10	3	1	
Kinematic Viscosity @ 135°C	11	2	1	
Kinematic Viscosity @ 60°C	9	4	1	
Absolute Viscosity @ 60°C	5	8	1	Yes answers by 4 Western Provinces & Yukon
Flash Point	12	1	1	
Solubility in CCl_4	9	4	1	
Ash	1	12	1	
Ductility	7*	6	1	
Softening Point	2	11	1	
Retained Penetration - after TFOT	6	7	1	
Retained Penetration - after RTFOT	1	12	1	
Retained Penetration after mixing	4	9	1	
Absolute Viscosity after TFOT	5	8	1	Yes answers by 4 Western Provinces

NOTE: * Ductility test performed in British Columbia when polymer or other additive is used.

TABLE 2
RESULTS OF SURVEY REGARDING TYPES OF CURRENT PROBLEMS

Problem Encountered	No. of Responses			
	Yes	No	Unknown	Perhaps
Low Temperature Cracking	12	2		
Permanent Deformation	9	5		
Fatigue Cracking	9	5		
Moisture Sensitivity	6	8		
Stripping	8	6		
Aging	6	6	1	1

TABLE 3
SUMMARY OF PERCENT RETAINED PENETRATION

GRADE	PERCENT RETAINED PENETRATION (min.)	
	B.C.	CAN/CGSB
80 - 100	55	47
150 - 200	50	40
200 - 300	45	37

TABLE 4
SUMMARY OF TYPES OF POLYMERS

TYPE	EXAMPLES
RUBBER a. Natural latex b. Synthetic latex c. Block Copolymer d. Reclaimed Rubber	 Natural rubber styrene-butadiene (SBR) styrene-butadiene-styrene (SBS) Recycled tires
PLASTIC	polyethylene polypropylene ethyl-vinyl-acetate (EVA) polyvinyl chloride (PVC)
COMBINATION	Blends of the above

TABLE 5
RELATIONSHIP OF ASPHALT AND MIX PROPERTIES TO PAVEMENT PERFORMANCE

	ASPHALT							MIX		
ASPHALT AND MIX PROPERTIES / PERFORMANCE VARIABLES	VISCOSITY	PENETRATION	DUCTILITY	TEMPERATURE SUSCEPTIBILITY	ASPHALT STIFFNESS	SOFTENING POINT	% RETAINED PEN AFTER T.F.O.T.	STIFFNESS	AIR VOIDS	ASPHALT PROPORTION (by Voids Filled or
Low Temperature Cracking (1) ↑	↑	↓	↓	↑	↑			↑		
Fatigue ↑	↑	↓	↓					↑ (2) ↓		↓
Ravelling ↑	↓	↓	↓				↓		↑	↓
Rutting ↑	↓	↑	↑	↑	↓			↓	↑	↑
Aging ↑	↑	↓	↓	↓		↑	↓		↑	↓
Moisture Sensitivity ↑	←----------(3)----------→								↑	↑

NOTES:
(1) An arrow ↑ indicates that this performance variable increases as a property value increase ↑ and vice versa.
(2) Increasing damage potential with increasing stiffness for pavements with ≤ 125 mm of pavement and decreasing damage potential for pavements > 125 mm in thickness.
(3) Performance testing is suggested to determine asphalt-aggregate compatibility.

TABLE 6
PRIORITIES OF PAVEMENT PERFORMANCE VARIABLES RELATIVE TO ROADWAY TRAFFIC VOLUMES

PERFORMANCE VARIABLE	RELATIVE PRIORITIES AS A FUNCTION OF TRAFFIC	
	LOW VOLUME	HIGH VOLUME
Low Temperature Cracking	⇐⇐⇐	
Fatigue	⇒⇒⇒	
Ravelling	⇒⇒⇒	
Rutting	⇒⇒⇒	
Aging	⇐⇐⇐	
Moisture Sensitivity	⇐ ---------- ⇒	

TABLE 7
C.G.S.B. ASPHALT CEMENT STIFFNESS[2]
AT 40°C
FOR VARIOUS LOADING TIMES
(from Van der Poel's Nomograph)

ASPHALT GROUP	GRADE (PEN 25°C)	PI [1]	T800pen °C[1]	STIFFNESS (N/m^2) FOR LOADING TIME OF		
				0.01s(100km/h)	0.02s (50 km/h)	0.05s(20km/h)
A	60	-1.15	48.6	1.0×10^6	5.0×10^5	3.0×10^5
	70	-1.13	47.2	7.0×10^5	4.5×10^5	2.0×10^5
	80	-1.11	46.1	6.0×10^5	4.0×10^5	1.8×10^5
	100	-1.10	44.1	4.5×10^5	2.5×10^5	1.2×10^5
	120	-1.10	42.4	3.5×10^5	2.0×10^5	1.0×10^5
	150	-1.12	40.3	2.8×10^5	1.5×10^5	8.0×10^4
	200	-1.14	37.6	1.6×10^5	1.0×10^5	4.0×10^4
	300	-1.19	33.9	1.0×10^5	6.0×10^4	2.0×10^4
	400	-1.30	31.2	7.0×10^4	4.0×10^4	1.5×10^4
B	60	-1.56	47.0	7.0×10^5	5.0×10^5	2.0×10^5
	70	-1.54	46.7	7.0×10^5	4.0×10^5	2.0×10^5
	80	-1.50	44.8	6.0×10^5	3.0×10^5	1.9×10^5
	100	-1.49	42.9	4.0×10^5	2.0×10^5	1.0×10^5
	120	-1.51	41.3	3.0×10^5	1.5×10^5	8.0×10^4
	150	-1.50	39.4	2.0×10^5	1.0×10^5	6.0×10^4
	200	-1.50	36.9	1.5×10^5	9.0×10^4	4.0×10^4
	300	-1.55	33.4	8.0×10^4	5.0×10^4	2.0×10^4
	400	-1.61	30.8	6.0×10^4	3.0×10^4	1.5×10^4

TABLE 7 (continued)
C.G.S.B. ASPHALT CEMENT STIFFNESS[2]
AT 40°C
FOR VARIOUS LOADING TIMES
(from Van der Poel's Nomograph)

ASPHALT GROUP	GRADE (PEN 25°C)	PI [1]	T800pen °C[1]	STIFFNESS (N/m^2) FOR LOADING TIME OF		
				0.01s(100km/h)	0.02s (50 km/h)	0.05s(20km/h)
C	60	-1.92	45.8	8.0×10^5	5.0×10^5	2.0×10^5
	70	-1.93	44.5	5.0×10^5	3.5×10^5	1.5×10
	80	-1.96	43.3	4.5×10^5	2.0×10^5	1.0×10^5
	100	-2.03	41.3	3.0×10^5	1.6×10^5	8.0×10^4
	120	-2.09	39.8	3.0×10^5	1.5×10^5	6.0×10^4
	150	-2.16	37.9	1.8×10^5	1.0×10^5	4.0×10^4
	200	-2.20	35.6	1.5×10^5	7.0×10^4	3.0×10^4
	300	-2.28	32.4	6.0×10^4	4.0×10^4	2.0×10^4
	400	-2.58	30.0	5.0×10^4	3.0×10^4	1.0×10^4

Notes: (1) Reported by K.O. Anderson (See Appendix B)
(2) As determined on original (tank) asphalts.

TABLE 8
Alberta Transportation Laboratory
SHRP Testing Program

AT&U Grade	Comments	Supplier	SHRP Grading
80 - 100	Air Blown	A	64-34
80 - 100 C		B	58-16
80 - 100	Air Blown	B	58-28
85 - 100 A		B	64-28
85 - 100 A	Rolling thin film residue	B	64-22
120 - 150		B	58-28
120 - 150	Rolling thin film residue	B	58-28
120 - 150		A	58-28
150 - 200A		A	58-28
150 - 200A	Rolling thin film residue	A	58-28
150 - 200A		C	52-22
150 - 200A		B	58-28
150 - 200A	Rolling thin film residue	B	58-28
150 - 200A		D	58-28
200 - 300A		A	52-34
200 - 300A		C	52-28
200 - 300A		B	52-34
200 - 300A	Rolling thin film residue	B	52-34
200 - 300A		D	52-34
300 - 400A		A	46-34
300 - 400A		C	46-28
300 - 400A		B	46-34
300 - 400A	Rolling thin film residue	B	46-34
300 - 400A		D	46-34
PMA-1		A	64-34
PMA-1	Rolling thin film residue	A	64-34
PMA-2		A	58-34
PMA-2	Rolling thin film residue	A	58-34
PMA-3	Rolling thin film residue	B	64-22
PMA-4	Rolling thin film residue	B	70-28

The above table is supplied for information purposes only to give an indication of the relationship between SHRP grading and AT&U grades which are close to CGSB grades.

TABLE 9

BITUMES PRODUITS AU QUÉBEC

Catégorie	Grade	Nombre	Classification SHRP										
			52-28	52-34	58-22	58-28	58-34	58-40	64-22	64-28	64-34	70-34	76-22
Conventionnel	80/100	5			40%	60%							
	150/200	3	66%	33%									
Intermédiaire	80/100	8			12,5%				25%	50%	12,5%		
	150/200	2		50%				50%					
Supérieur	50/70	1											100%
	80/100	1									100%		
	120/150	2					50%					50%	
	150/200	3		33,3%			33,3%	33,3%					

Le 10 mai 1995

TABLE 10
SELECTED ENVIRONMENT CANADA OFFICES

<u>National Office</u>
Climate Information Branch
Canadian Meteorological Centre
4905 Dufferin St.
Downsview, Ontario
M3H 5T4
Tel: (416) 739-4328
Fax: (416) 739-4446

Climate Services
Room 1000/266 Graham Ave.
Winnipeg, Manitoba
R3C 3V4
Tel: (204) 983-0586
Fax: (204) 983-4884

<u>Ontario Climate Centre</u>
Ontario Climate Centre
4905 Dufferin Street
Downsview, Ontario
M3H 5T4
Tel: (416) 739-4516
Fax: (416) 739-4521

<u>Regional and Local Offices</u>
Pacific & Yukon Region
Climate and Applications
Suite 700/1200 W 73rd Avenue
Vancouver, BC
V6P 6H9
Tel: (604) 664-9156
Fax: (604) 664-9133

Climate Services
300 Park Plaza
2365 Albert St.
Regina, Saskatchewan
S4P 4K1
Tel: (306) 780-6341
Fax: (306) 780-5311

<u>Quebec Region</u>
Climate Services
100 Alexis Nihon Blvd.
3rd Floor,
Ville Saint Laurent, Quebec
H4M 2N8
Tel: (514) 283-1296 or (514) 283-1107
Fax: (514) 283-7149

<u>Prairies & Northern Region</u>
Climate Services
Twin Atria Building
Room 200/4999 - 98 Avenue
Edmonton, Alberta
T6B 2X3
Tel: (403) 951-8881
Fax: (403) 495-3529

Climate Services
Room 301/111 Research Dr.
Saskatoon, Saskatchewan
S7N 3R2
Tel: (306) 975-6909
Fax: (306) 975-5954

<u>Atlantic Region</u>
Atmospheric Environment Branch
1496 Bedford Highway
Bedford, NS
B4A 1E5
Tel: (902) 426-9226
Fax: (902) 426-9158

TABLE 11
PVN AND PI VALUES
FOR
GROUP AND GRADE OF ASPHALT CEMENT
(from Figures 1 & 2, CAN/CGSB-16.3-M90)

PENETRATION @ 25°C, 100g, 5 sec.	LINE SEPARATING A & B GROUPS		LINE SEPARATING B & C GROUPS		LINE AT BOTTOM OF C GROUP	
	PVN	PI	PVN	PI	PVN	PI
60	-0.77	-1.15	-1.13	-1.56	-1.52	-1.92
70	-0.77	-1.13	-1.13	-1.54	-1.58	-1.93
80	-0.76	-1.11	-1.11	-1.50	-1.61	-1.96
100	-0.72	-1.10	-1.11	-1.49	-1.73	-2.03
120	-0.72	-1.10	-0.95	-1.51	-1.79	-2.09
150	-0.66	-1.12	-1.08	-1.50	-1.90	-2.16
200	-0.64	-1.14	-1.07	-1.50	-2.04	-2.20
300	-0.55	-1.19	-1.00	-1.55	-2.27	-2.28
400	-0.51	-1.30	-1.00	-1.61	-2.51	-2.58

TABLE 12
PRECISION CRITERIA
ASTM D5

MATERIAL	STANDARD DEVIATION OR COEFFICIENT OF VARIATION (IS) OR (IS%)	ACCEPTABLE RANGE OF TWO TEST RESULTS (D2S) OR (D2S%)
Single-operator precision: Asphalts at 77°F (25°C) below 50 penetration, units	0.35	1
Asphalts at 77°F (25°C) 50 penetration and above, percent of their mean	1.1	3
Tar pitches at 77°F (25°C) percent of their mean	5.2	15
Multilaboratory precision: Asphalts at 77°F (25°C) below 50 penetration, units	1.4	4
Asphalts at 77°F (25°C) 50 penetration and above, percent of their mean	2.8	8
Tar pitches at 77°F (25°C), units	1.4	4

TABLE 13
PRECISION AND BIAS
ASTM D2171

1. Repeatability - Duplicate results by the same operator using the same viscometer should not be considered suspect unless they differ by more than 7% of their mean.

2. Reproducibility - The results submitted by two laboratories should not be considered suspect unless the two test results differ by more than 10% of their mean.

TABLE 14
ACCEPTABILITY OF TEST RESULTS FOR KINEMATIC VISCOSITY
ASTM D2170

MATERIALS & TYPE INDEX	COEFFICIENT OF VARIATION (% OF MEAN)	ACCEPTABLE RANGE OF TWO RESULTS (% OF MEAN)
Single-operator precision: Asphalt cements at 275°F (135°C)	0.64	1.8
Liquid asphalts at 140°F (60°C): below 3000 cSt 3000 to 6000 cSt 6000 cSt and above	 0.53 0.71 3.2	 1.5 2.0 8.9
Multilaboratory precision: Asphalt cements at 275°F (135°)	3.1	8.8
Liquid asphalts at 140°F (60°C): below 3000 cSt 3000 to 6000 cSt above 6000 cSt	 1.06 3.11 3.6	 3.0 9.0 10.0

NOTE: The values given in column 2 are the coefficients of variation that have been found to be appropriate for the materials and conditions of test described in column 1. The values given in column 3 are the limits that should not be exceeded by the difference between the results of two properly conducted tests.

TABLE 15
PRECISION OF TEST ON RESIDUE
D1754

MATERIAL & TYPE INDEX	STANDARD DEVIATION (1S)	ACCEPTABLE RANGE OF TWO RESULTS (D2S)	COEFFICIENT OF VARIATION (percent of mean) (1S%)	ACCEPTABLE RANGE OF TWO RESULTS (percent of mean) (D2S%)
Single-operator precision: Percentage of retained penetration	1.43	4.0	...	...
Change in mass percentage: Not more than 0.4% (max) Greater than 0.4%	0.014 ...	0.04 ...	... 2.9	... 8.0
Viscosity at 140°F (60°C)	...	...	3.3	9.3
Viscosity at 275°F (135°C)	...	...	2.0	5.7
Ratio: viscosity at 140°F (60°C) after test / viscosity at 140°F (60°C) before test	...	...	5.6	16.0
Ductility at 60°F (15.6°C), cm	7	20		
Multilaboratory precision:				
Percentage of retained penetration	2.90	8.0	...	...
Change in mass percentage: Not more than 0.4% (max) Greater than 0.4%	0.055 ...	0.16 ...	... 14.0	... 40.0
Viscosity at 140°F (60°C)	...	...	11.6	33.0
Viscosity at 275°F (135°C)	...	...	6.4	18.0
Ratio: viscosity at 140°F (60°C) after test / viscosity at 140°F (60°C) before test	...	...	9.1	26.0
Ductility at 60°F (15.6°C), cm	12	34		

TABLE 16
SENSITIVITY ANALYSIS FOR MULTI-LABORATORY TESTING OF ASPHALT CEMENT TO DETERMINE PENETRATION INDEX AND CRACKING TEMPERATURE[1]

A. TEST DATA	VALUES REPORTED		MEAN VALUE	ACCEPTABLE RANGE	
PROPERTY	LAB 1	LAB 2		LOW	HIGH
Pen, 100 g, 5 sec, (dmm)					
@ 25°C	170	182	176	169	183
@ 5°C	18	16	17	15	19

B. PI, SERVICE TEMPERATURE & CRACKING TEMPERATURE

COMBINATION OF PEN 25°C/PEN 5°C	PI	DESIGN TEMP (°C)[1]	CRACKING TEMP (°C)
a) 169/15	-1.73	-25	-35
b) 169/19	-1.10	-30	-40
c) 183/15	-1.93	-25	-35
d) 183/19	-1.33	-29	-39
e) 176/19	-1.52	-28	-38

NOTES: 1) Based on Robertson's Asphalt Selection Chart (Figure 13)

TABLE 17
1993 USAGE OF ASPHALT (tonnes)
ASPHALT CEMENT GRADE
(By Provincial Highway Department)

AGENCY	80-100A	80-100C	85-100	120-150	150-200	200-300	300-400	Other
Newfoundland					25,122			
Nova Scotia					34,000	8,000		
New Brunswick				7,464	25,445	6,235	600	
Quebec	74,080				8,695			6,593*
Ontario			115,000		35,000			
Manitoba				5,146	35,485	783		
Saskatchewan					19,671	18,157	5,263	1,887**
Alberta					Quantity Not Available	Quantity Not Available	Quantity Not Available	
British Columbia	4,956	1,989			36,400	878		
Yukon					1,060	825		

* PMA or Engineered Asphalt
** 400 - 500

TABLE 18
STIFFNESS OF C.G.S.B. GROUP A ASPHALTS (S_{bit})
T = 40°C, t = 0.02 sec. loading time

GRADE	ORIGINAL PROPERTIES[1]			INITIALLY AGED PROPERTIES[3] (C.G.S.B. MAX. LOSS)				INITIALLY AGED PROPERTIES BASED ON HUSKY ASPHALTS			
	PI	$T_{800\,pen}$ °C	Sbit[2] (N/m²)	PI	Pen 25	$T_{800\,pen}$ °C	Sbit[2] (N/m²)	PI	Pen 25[4]	$T_{800\,pen}$ °C	Sbit[2] (N/m²)
60	-1.15	48.6	6.0×10^5	-1.15	31	55	1.8×10^6	--	--	--	--
70	-1.13	47.2	5.0×10^5	-1.13	36	54	1.3×10^6	--	--	--	--
80	-1.11	46.1	4.0×10^5	-1.11	38	53	1.2×10^6	-1.11	46	51	8.0×10^5
100	-1.10	44.1	2.5×10^5	-1.10	47	51	8.0×10^5	-1.10	58	48	5.0×10^5
120	-1.10	42.4	2.0×10^5	-1.10	50	50	7.0×10^5	-1.10	70	46	4.0×10^5
150	-1.12	40.3	1.5×10^5	-1.12	63	48	5.0×10^5	-1.12	86	45	3.0×10^5
200	-1.14	37.6	9.0×10^4	-1.14	80	46	4.0×10^5	-1.14	102	43	2.0×10^5
300	-1.19	33.9	5.0×10^4	-1.19	111	43	3.0×10^5	-1.19	150	40	1.5×10^5
400	-1.30	31.2	3.0×10^4	-1.30	140	40	1.5×10^5	-1.30	200	37	1.0×10^5

Assumed in the absence

NOTES:

1) Original PI and T800pen values are from Anderson and Bai
2) Sbit values were estimated from a "blown up" version of Van der Poel's nomograph.
3) Initially aged properties are on the premise that minimum specified C.G.S.B. retained penetration exists after TFOT.
4) Data was provided by Husky Oil.

eba

TABLE 19
Performance Graded Asphalt Binder Specification (AASHTO MPI)

PERFORMANCE GRADE	PG 46-			PG 52-							PG 58-					PG 64-					
	34	40	46	10	16	22	28	34	40	46	16	22	28	34	40	10	16	22	28	34	40
Average 7-day Maximum Pavement Design Temperature, °C[a]	<46			<52							<58					<64					
Minimum Pavement Design Temperature, °C[a]	>-34	>-40	>-46	>-10	>-16	>-22	>-28	>-34	>-40	>-46	>-16	>-22	>-28	>-34	>-40	>-10	>-16	>-22	>-28	>-34	>-40
ORIGINAL BINDER																					
Flash Point Temp, T48: Minimum °C	230																				
Viscosity, ASTM D4402:[b] Maximum, 3 Pa•s, Test Temp, °C	135																				
Dynamic Shear, TP5:[c] $G^*/\sin\delta$, Minimum, 1.00 kPa Test Temp @ 10 rad/s, °C	46			52							58					64					
ROLLING THIN FILM OVEN (T240) OR THIN FILM OVEN RESIDUE (T179)																					
Mass Loss, Maximum, percent	1.00																				
Dynamic Shear, TP5: $G^*/\sin\delta$, Minimum, 2.20 kPa Test Temp @ 10 rad/s, °C	46			52							58					64					
PRESSURE AGING VESSEL RESIDUE (PP1)																					
PAV Aging Temperature, °C[d]	90			90							100					100					
Dynamic Shear, TP5: $G^*\sin\delta$, Maximum, 5000 kPa Test Temp @ 10 rad/s, °C	10	7	4	25	22	19	16	13	10	7	25	22	19	16	13	31	28	25	22	19	16
Physical Hardening[e]	Report																				
Creep Stiffness, TP1:[f] S, Maximum, 300 MPa, *m* - value, Minimum, 0.300 Test Temp @ 60s, °C	-24	-30	-36	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	-30
Direct Tension, TP3:[f] Failure Strain, Minimum, 1.0% Test Temp @ 1.0 mm/min, °C	-24	-30	-36	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	-30

[a] Pavement temperatures are estimated from air temperatures using an algorithm contained in the SUPERPAVE software program, may be provided by the specifying agency, or by following the procedures as outlined in PPX.

[b] This requirement may be waived at the discretion of the specifying agency if the supplier warrants that the asphalt binder can be adequately pumped and mixed at temperatures that meet all applicable safety standards.

[c] For quality control of unmodified asphalt cement production, measurement of the viscosity of the original asphalt cement may be substituted for dynamic shear measurements of $G^*/\sin\delta$ at test temperatures where the asphalt is a Newtonian fluid. Any suitable standard means of viscosity measurement may be used, including capillary or rotational viscometry (AASHTO T201 or T202).

[d] The PAV aging temperature is based on simulated climatic conditions and is one of three temperatures 90°C, 100°C or 110°C. The PAV aging temperature is 100°C for PG 58- and above, except in desert climates, where it is 110°C.

[e] Physical Hardening — TP1 is performed on a set of asphalt beams according to Section 13.1, except the conditioning time is extended to 24 hrs ± 10 minutes at 10°C above the minimum performance temperature. The 24-hour stiffness and *m*-value are reported for information purposes only.

[f] If the creep stiffness is below 300 MPa, the direct tension test is not required. If the creep stiffness is between 300 and 600 MPa the direct tension failure strain requirement can be used in lieu of the creep stiffness requirement. The *m*-value requirement must be satisfied in both cases.

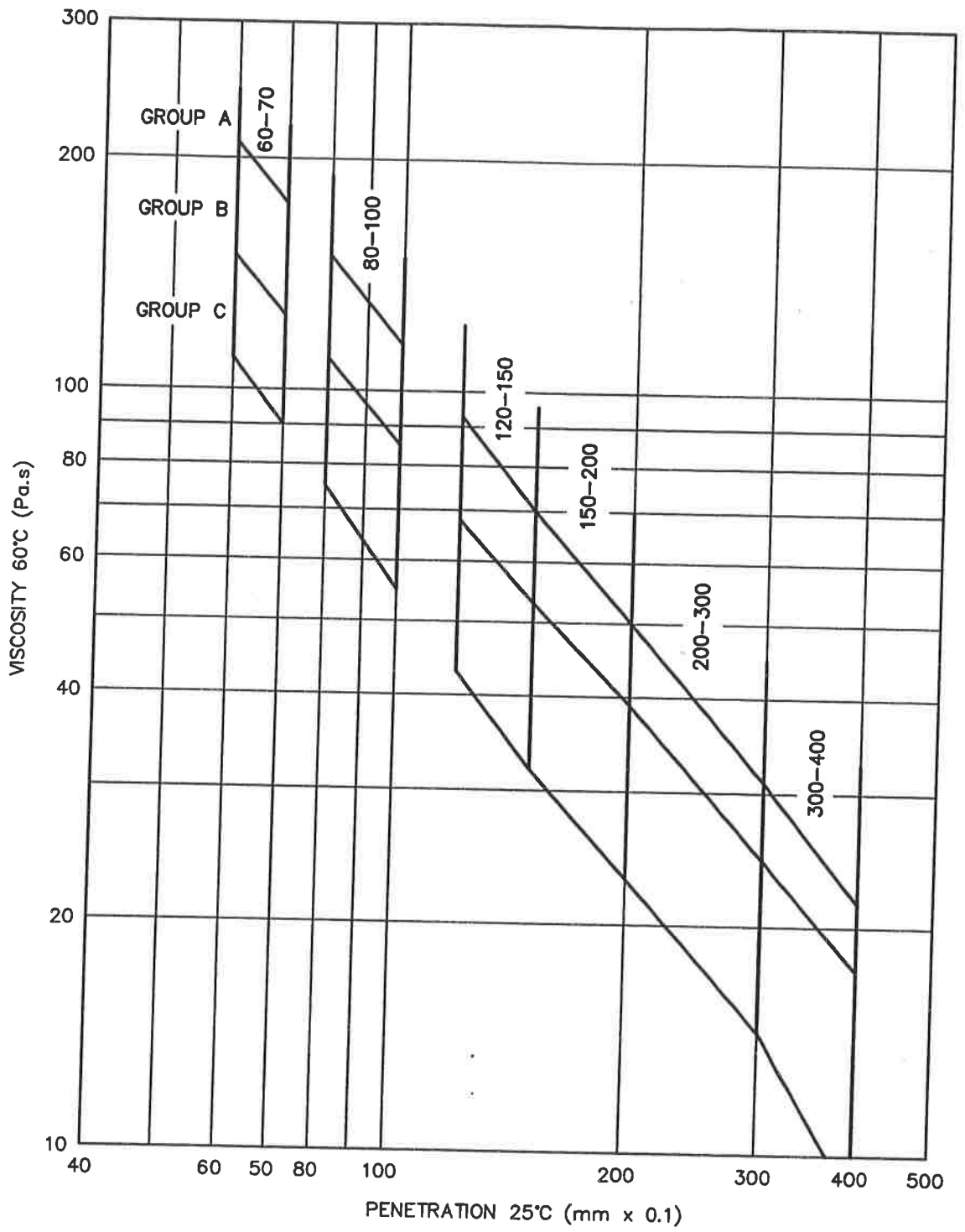
TABLE 19 (continued)
Performance Grade Asphalt Binder Specification (AASHTO MP1)

PERFORMANCE GRADE	PG 70-						PG 76-					PG 82-				
	10	16	22	28	34	40	10	16	22	28	34	10	16	22	28	34
Average 7-day Maximum Pavement Design Temp, °C[b]	<70						<76					<82				
Minimum Pavement Design Temperature, °C[b]	>-10	>-16	>-22	>-28	>-34	>-40	>-10	>-16	>-22	>-28	>-34	>-10	>-16	>-22	>-28	>-34
ORIGINAL BINDER																
Flash Point Temp, T48: Minimum °C	230															
Viscosity, ASTM D4402:[b] Maximum, 3 Pa•s, Test Temp, °C	135															
Dynamic Shear, TP5:[c] $G^*/\sin\delta$, Minimum, 1.00 kPa Test Temp @ 10 rad/s, °C	70						76					82				
ROLLING THIN FILM OVEN (T240) OR THIN FILM OVEN (T179) RESIDUE																
Mass Loss, Maximum, percent	1.00															
Dynamic Shear, TP5: $G^*/\sin\delta$, Minimum, 2.20 kPa Test Temp @ 10 rad/s, °C	70						76					82				
PRESSURE AGING VESSEL RESIDUE (PP1)																
PAV Aging Temperature, °C[d]	100(110)						100(110)					100(110)				
Dynamic Shear, TP5: $G^*\sin\delta$, Maximum, 5000 kPa Test Temp @ 10 rad/s, °C	34	31	28	25	22	19	37	34	31	28	25	40	37	34	31	28
Physical Hardening[e]	Report															
Creep Stiffness, TP1:[f] S, Maximum, 300.0 MPa, *m* - value, Minimum, 0.300 Test Temp @ 60s, °C	0	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	0	-6	-12	-18	-24
Direct Tension, TP3:[f] Failure Strain, Minimum, 1.0% Test Temp @ 1.0 mm/min, °C	0	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	0	-6	-12	-18	-24

TABLE 20
CANADIAN ASPHALT SUPPLY

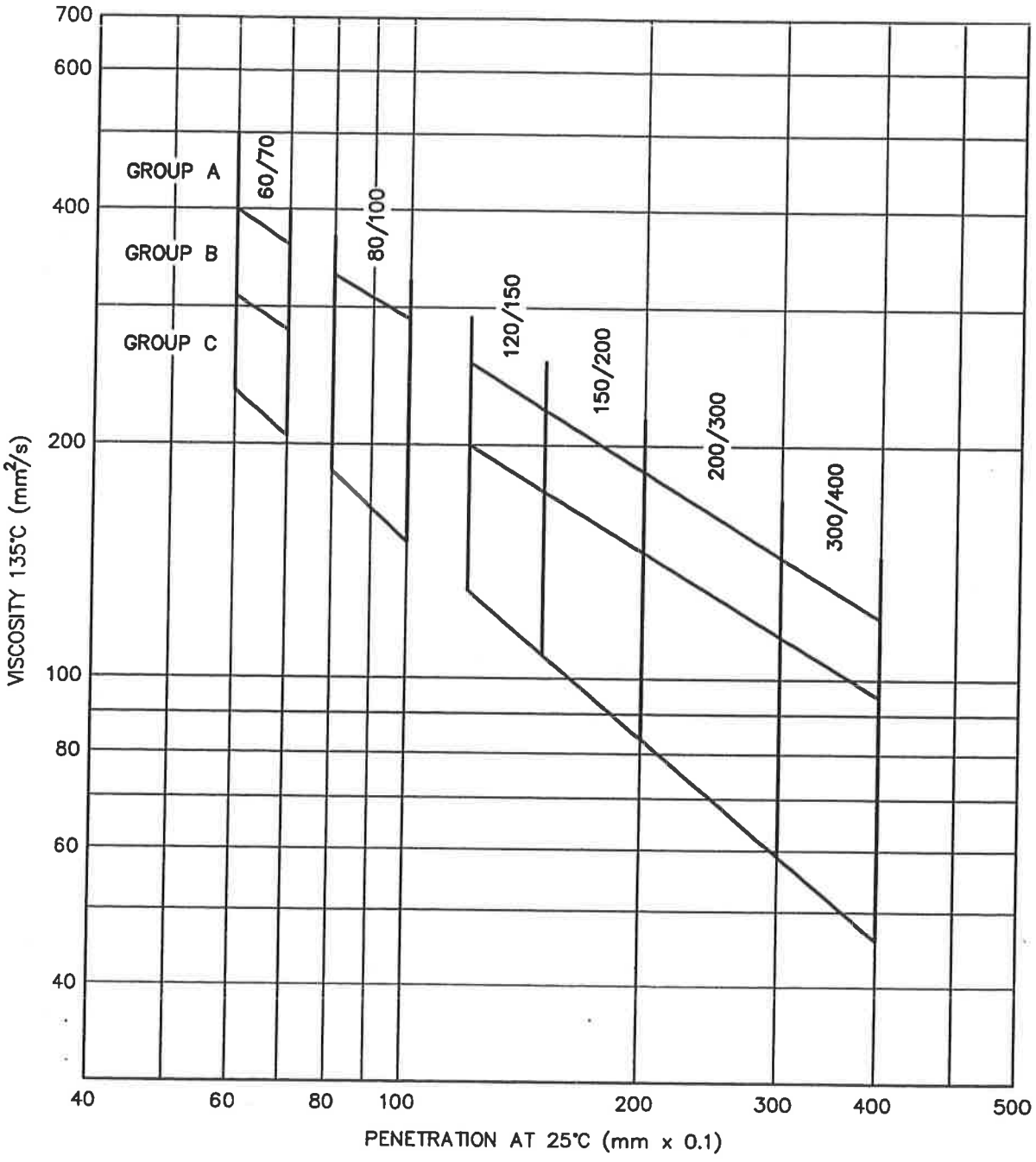
Province	Producer/Broker	Refinery	Source	
Newfoundland	Irving Oil Ultramar Canada Newfoundlands Hardwoods Ltd.	Irving Oil, St. John, New Brunswick Ultramar Canada, St. Romuald, Quebec	Middle East Mexico Africa	North Sea Venezuela Spain
Nova Scotia	Imperial Oil Irving Oil Ultramar	Imperial, Dartmouth, Nova Scotia Irving Oil, St. John, New Brunswick Ultramar Canada, St. Romuald, Quebec	Middle East Mexico Africa	North Sea Venezuela Spain
Prince Edward Island	Imperial Oil Irving Oil Ultramar Canada	Imperial, Dartmouth, Nova Scotia Irving Oil, St. John, New Brunswick Ultramar Canada, St. Romuald, Quebec	Middle East Mexico Africa	North Sea Venezuela Spain
New Brunswick	Imperial Oil Irving Oil Ultramar Canada Petro Canada Barrett Bitumar Brunswick Asphalt	Imperial, Dartmouth, Nova Scotia Irving Oil, St. John, New Brunswick Ultramar Canada, St. Romuald, Quebec Petro Canada, Montreal, Quebec	Middle East Mexico North Sea Venezuela Spain	
Quebec	Ultramar Canada Petro Canada Shell Canada Imperial Oil Bitumar	Ultramar Canada, St. Romuald, Quebec Petro Canada, Montreal, Quebec Shell Canada, Montreal, Quebec	Middle East North Sea Mexico Venezuela Spain	Prairie Crudes
Ontario	Imperial Oil Ultramar Canada Petro Canada Shell Canada Bitumar Amoco Oil Ashwarren International Canadian Asphalt Husky Oil Iko Industries Michigan Marine Norjohn McAsphalt	Imperial Oil, Nanticoke, Ontario (1996) Ultramar Canada, St. Romuald, Quebec Petro Canada, Oakville, Ontario Shell Canada, Montreal, Quebec	Middle East Mexico Venezuela Spain Prairie Crudes U.S. Crudes	Spain
Manitoba	Imperial Oil Husky Oil Moose Jaw Asphalt Koch Materials	Imperial Oil, Edmonton, Alberta Husky Oil, Lloydminster, Alberta Moose Jaw Asphalt, Moose Jaw, Alberta Koch Materials, Minneapolis, Minnesota	Imperial Oil, Cold Lake Husky Oil, Lloydminster Moose Jaw Asphalt, Cold Lake Koch Materials, Cold Lake, U.S. Crudes	
Saskatchewan	Imperial Oil Husky Oil Moose Jaw Asphalt Koch Materials	Imperial Oil, Strathcona, Alberta Husky Oil, Lloydminster, Alberta Moose Jaw Asphalt Moose Jaw , Alberta Koch Materials, Minneapolis, Minnesota	Imperial Oil, Cold Lake Husky Oil, Lloydminster Moose Jaw Asphalt, Cold Lake Koch Materials, Cold Lake, U.S. Crudes	
Alberta	Imperial Oil Husky Oil Shell Canada Moose Jaw Asphalt	Imperial Oil, Edmonton, Alberta Husky Oil, Lloydminster, Alberta Shell Canada, Peace River, Alberta Moose Jaw Asphalt, Moose Jaw, Alberta	Imperial Oil, Cold Lake Husky Oil, Lloydminster Shell Canada, Peace River Moose Jaw Asphalt, Cold Lake	
British Columbia	Imperial Oil Husky Oil Shell Canada Chevron McTar Asphalt	Imperial Oil, Strathcona, Alberta Husky Oil, Lloydminster, Alberta Shell Canada, Peace River, Alberta Chevron, Burnaby, British Columbia	Imperial Oil, Cold Lake Husky Oil, Lloydminster Shell Canada, Peace River Chevron, Boundary Lake & Alberta Light McTar, asphalt from California light crude	

FIGURES



9734–M11D

FIGURE 1 ABSOLUTE VISCOSITY VS. PENETRATION (AFTER CAN/CGSB–16.3–M90, FIGURE 1)



9734-M10D

FIGURE 2 KINEMATIC VISCOSITY VS. PENETRATION
(AFTER CAN/CGSB-16.3-M90, FIGURE 2)

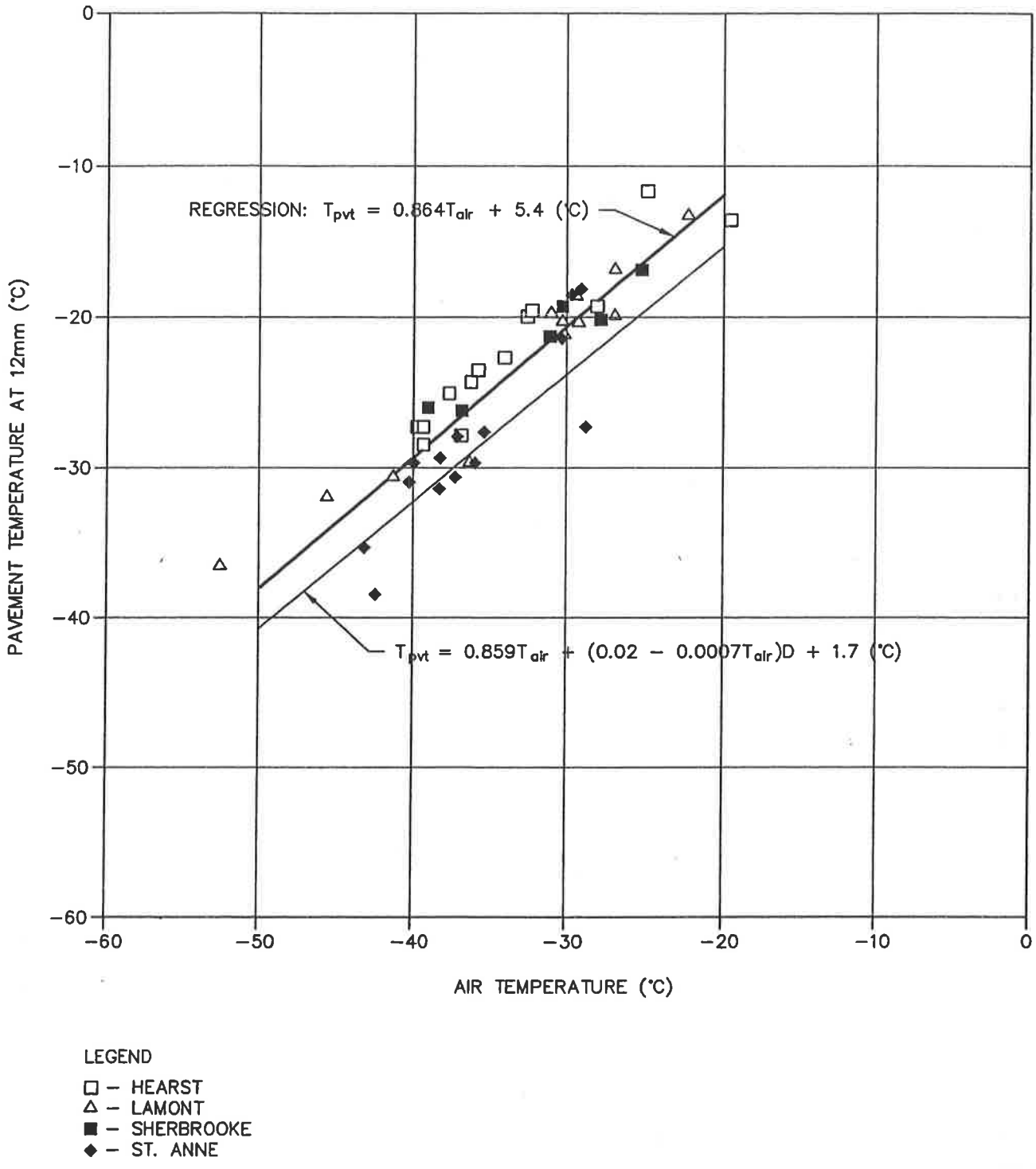


FIGURE 3 REGRESSION ANALYSIS AIR TEMPERATURE VERSUS PAVEMENT TEMPERATURE FOR COLD EVENTS

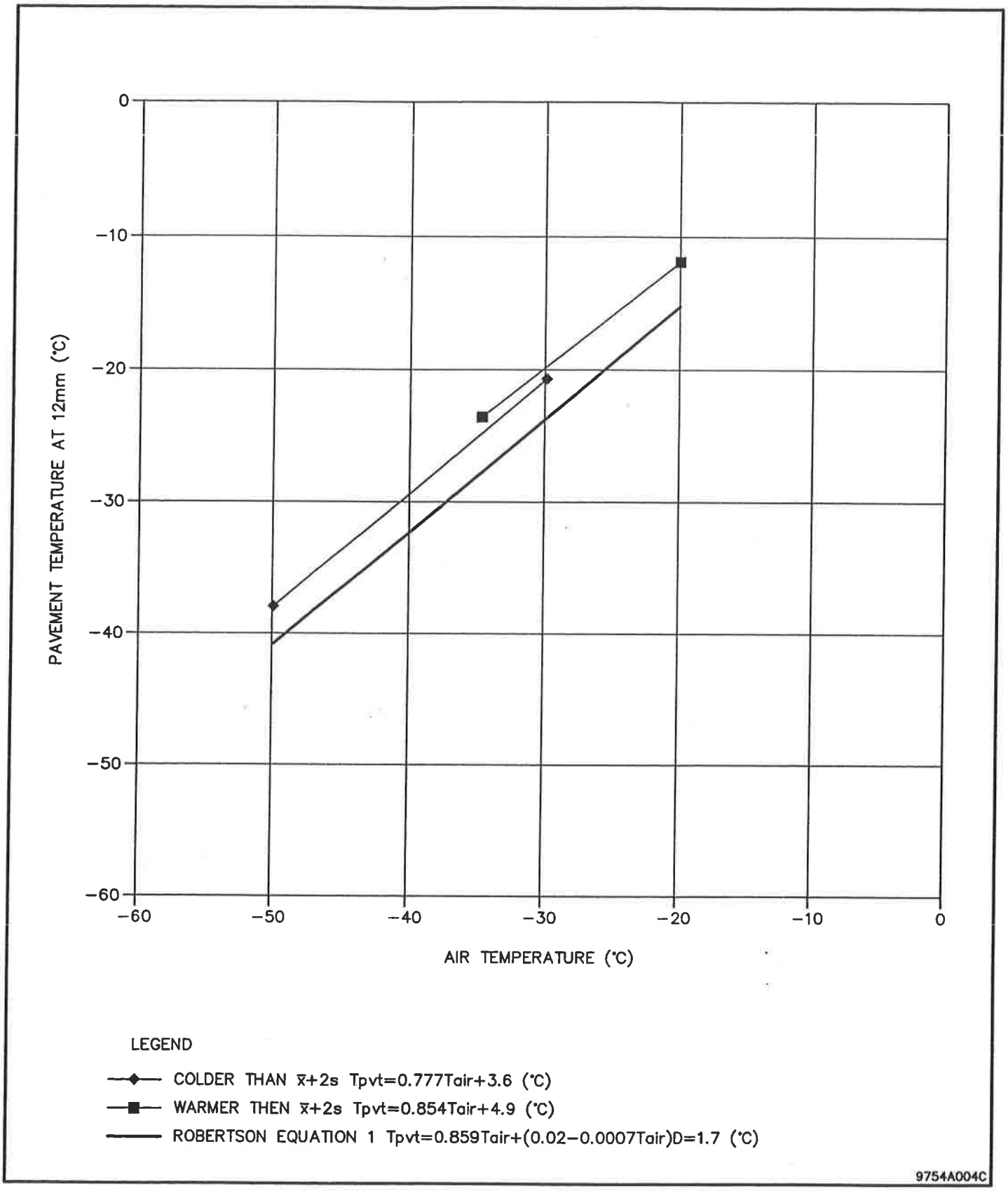
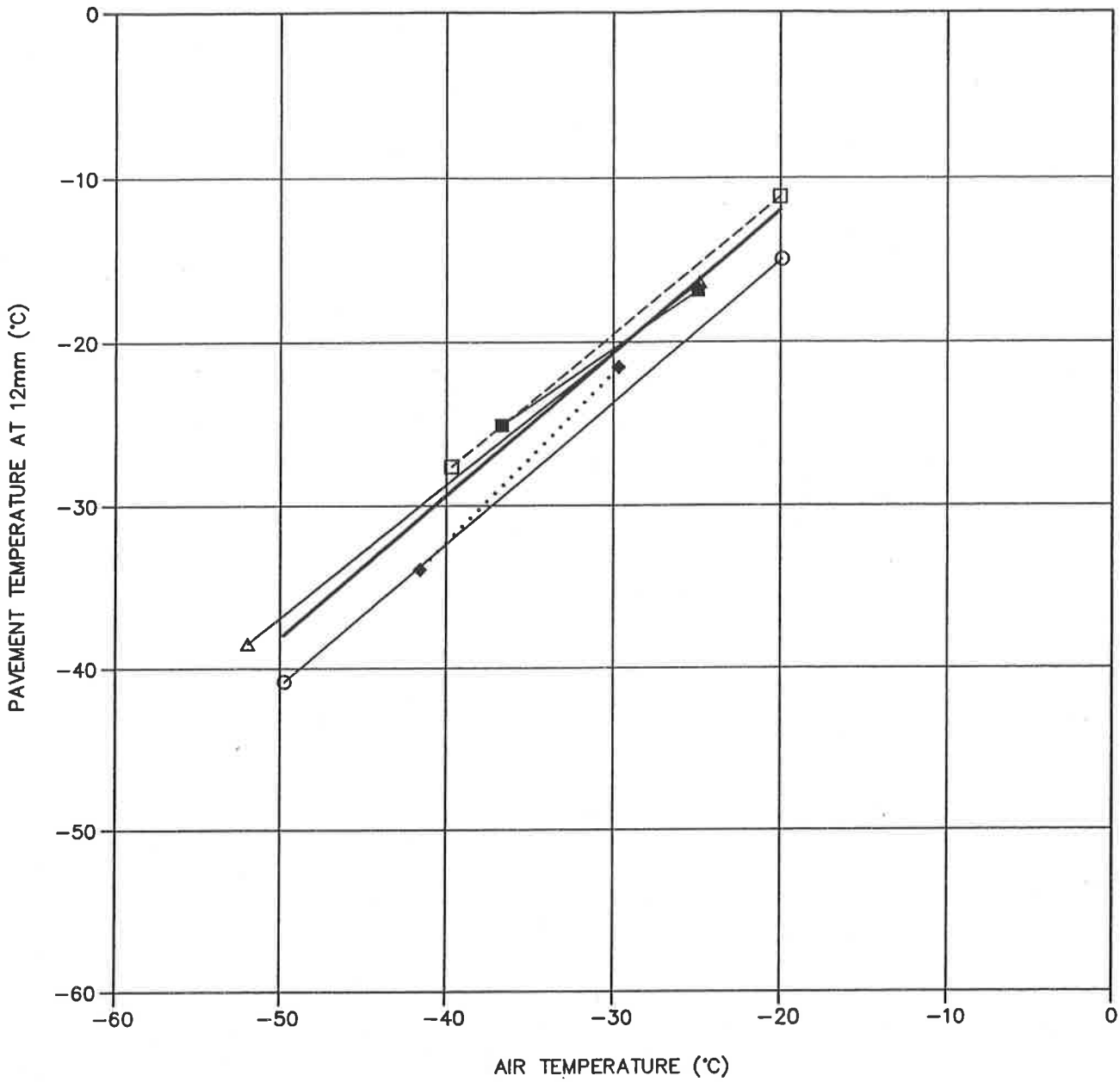


FIGURE 4 REGRESSION ANALYSIS OF TEMPERATURES WARMER AND COLDER THAN ($\bar{x}$+2σ)

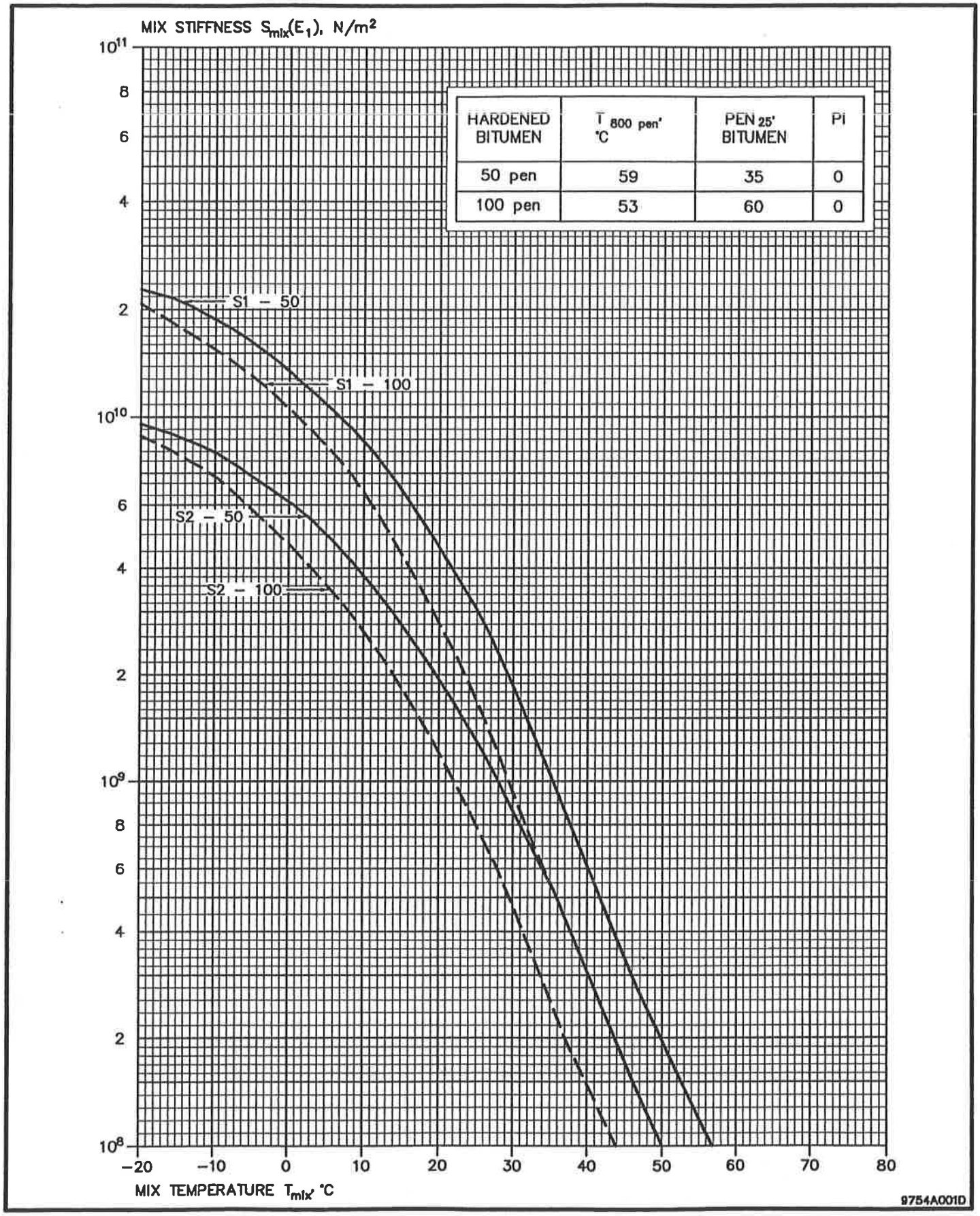


LEGEND

- ALL DATA Tpvt = 0.863Tair + 5.4 (°C)
- HEARST Tpvt = 0.817Tair + 5.1 (°C)
- LAMONT Tpvt = 0.812Tair + 3.9 (°C)
- ST. ANNE Tpvt = 1.039Tair + 9.7 (°C)
- SHERBROOKE Tpvt = 0.679Tair + 0.1 (°C)
- ROBERTSON Tpvt = 0.859Tair + (0.02 − 0.0007Tair)D=1.7 (°C)

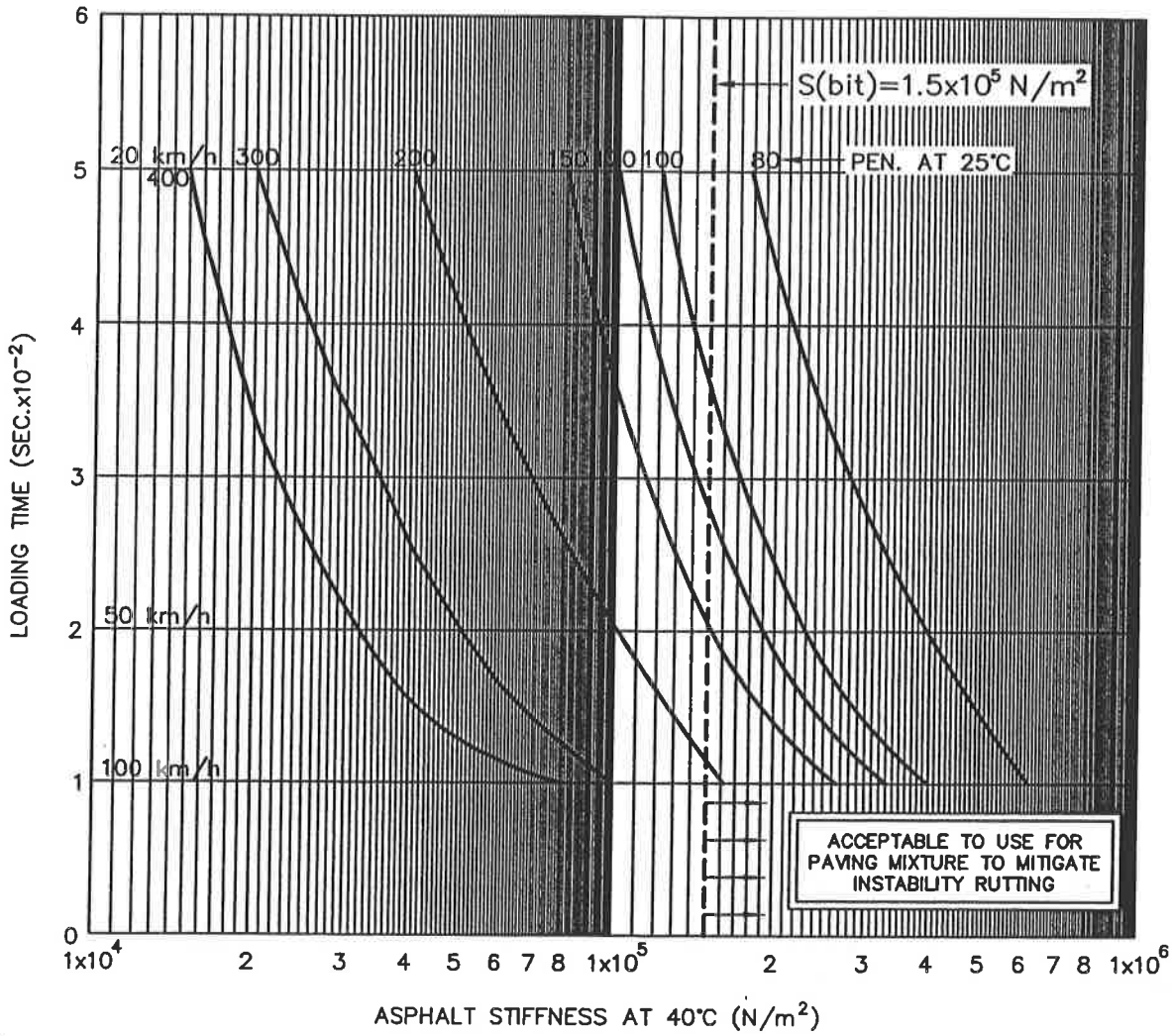
9754A005B

FIGURE 5 REGRESSION ANALYSIS FOR TEST ROADS PAVEMENT TEMPERATURE VERSUS AIR TEMPERATURE



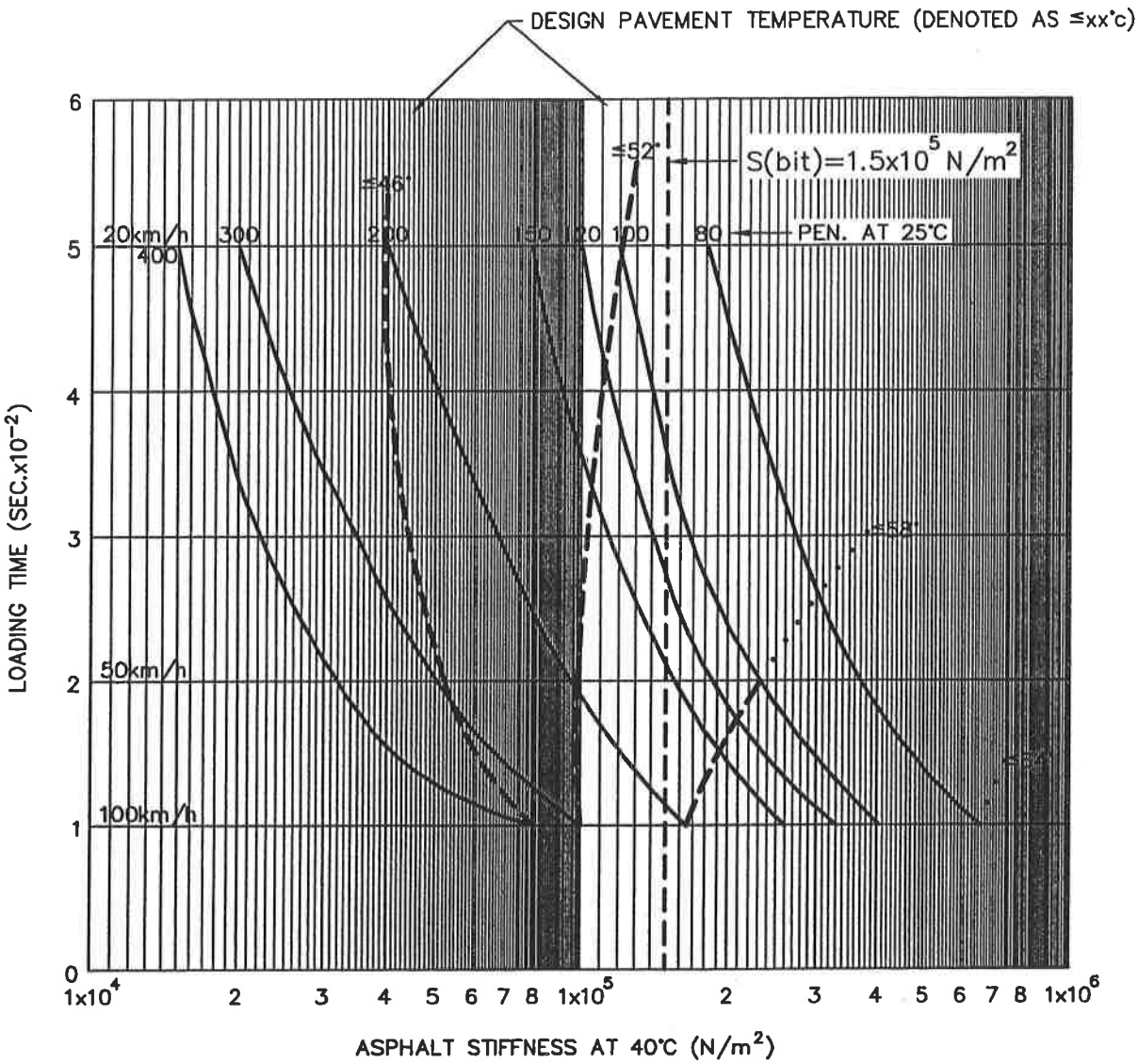
HARDENED BITUMEN	$T_{800\ pen}$, °C	$PEN_{25°}$ BITUMEN	PI
50 pen	59	35	0
100 pen	53	60	0

FIGURE 6 RELATIONSHIP BETWEEN STIFFNESS OF MIX AND TEMPERATURE (FROM SHELL PAVEMENT DESIGN MANUAL)



9734-M12D

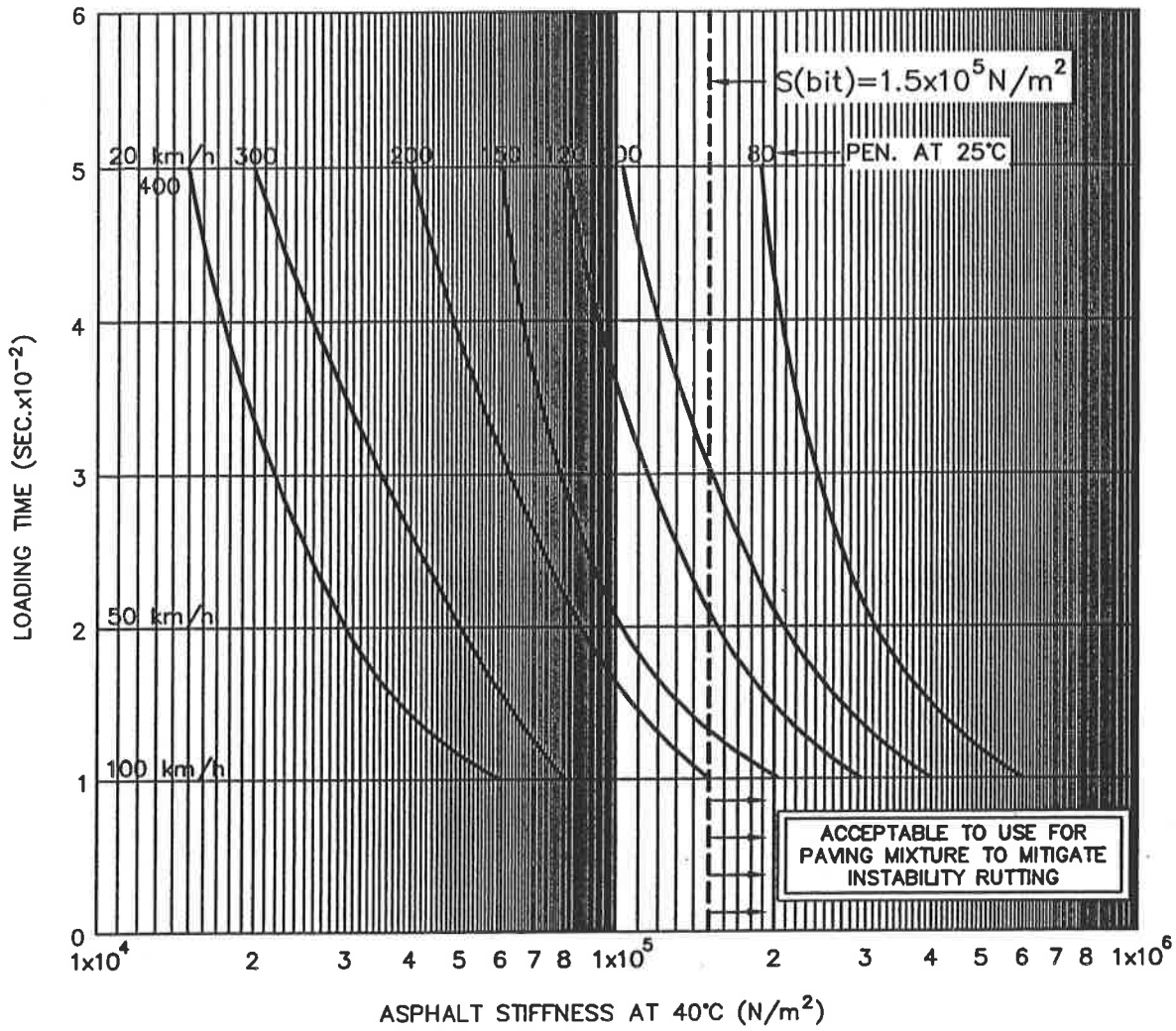
FIGURE 7 STIFFNESS OF CGSB TANK ASPHALTS (GROUP A) AT 40°C AND VARIABLE LOADING TIME



Notes: 1) Service conditions that prevail where the heavy lines are plotted (• • • • •) would have to be satisfied using a modified asphalt binder.
2) Section 6.5.2 should be read in conjunction with this Figure.

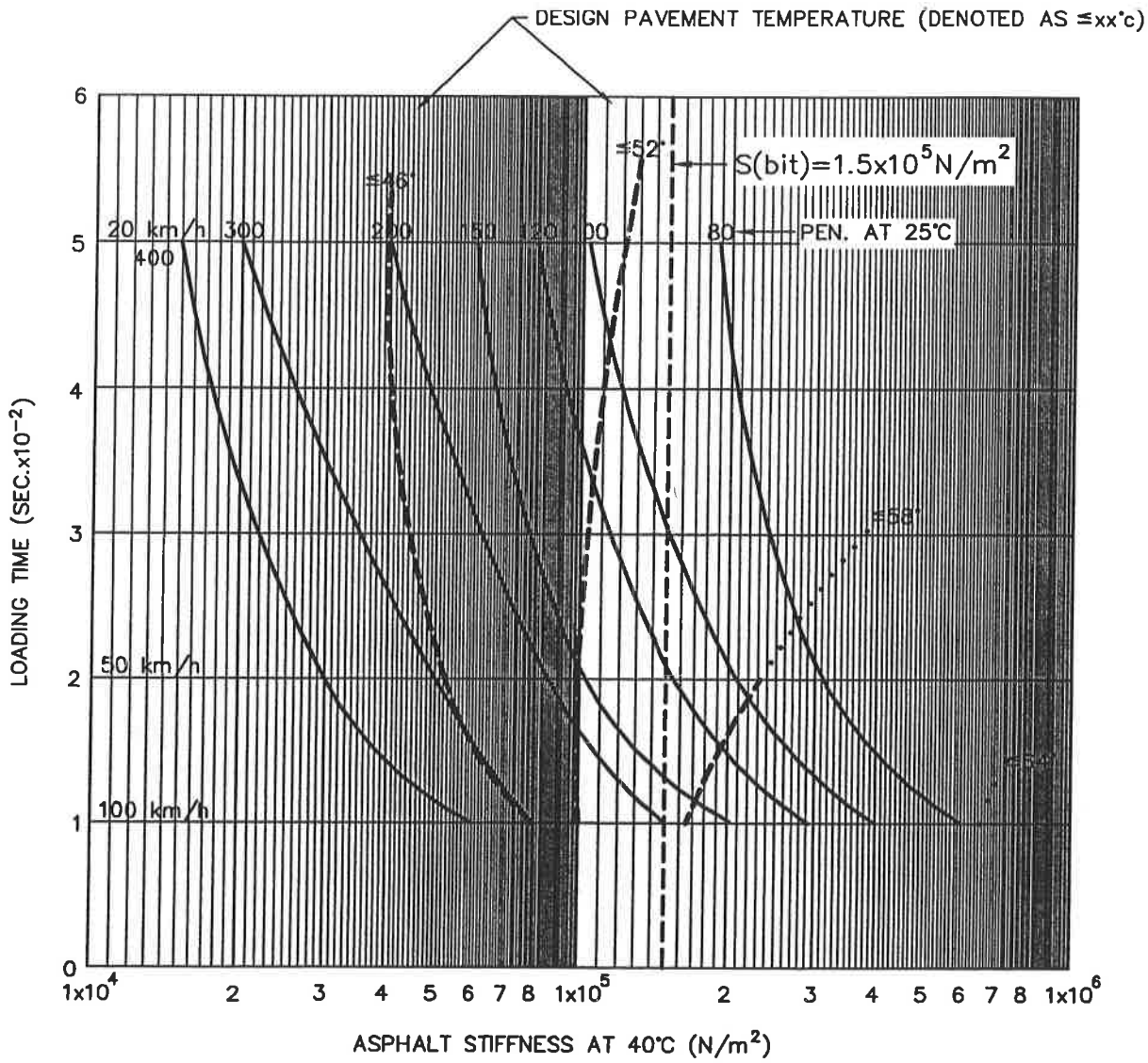
9754—A13F

FIGURE 7A CGSB GROUP A ASPHALTS WITH MAXIMUM DESIGN PAVEMENT TEMPERATURE ISOTHERMS SUPERIMPOSED



9734—M03G

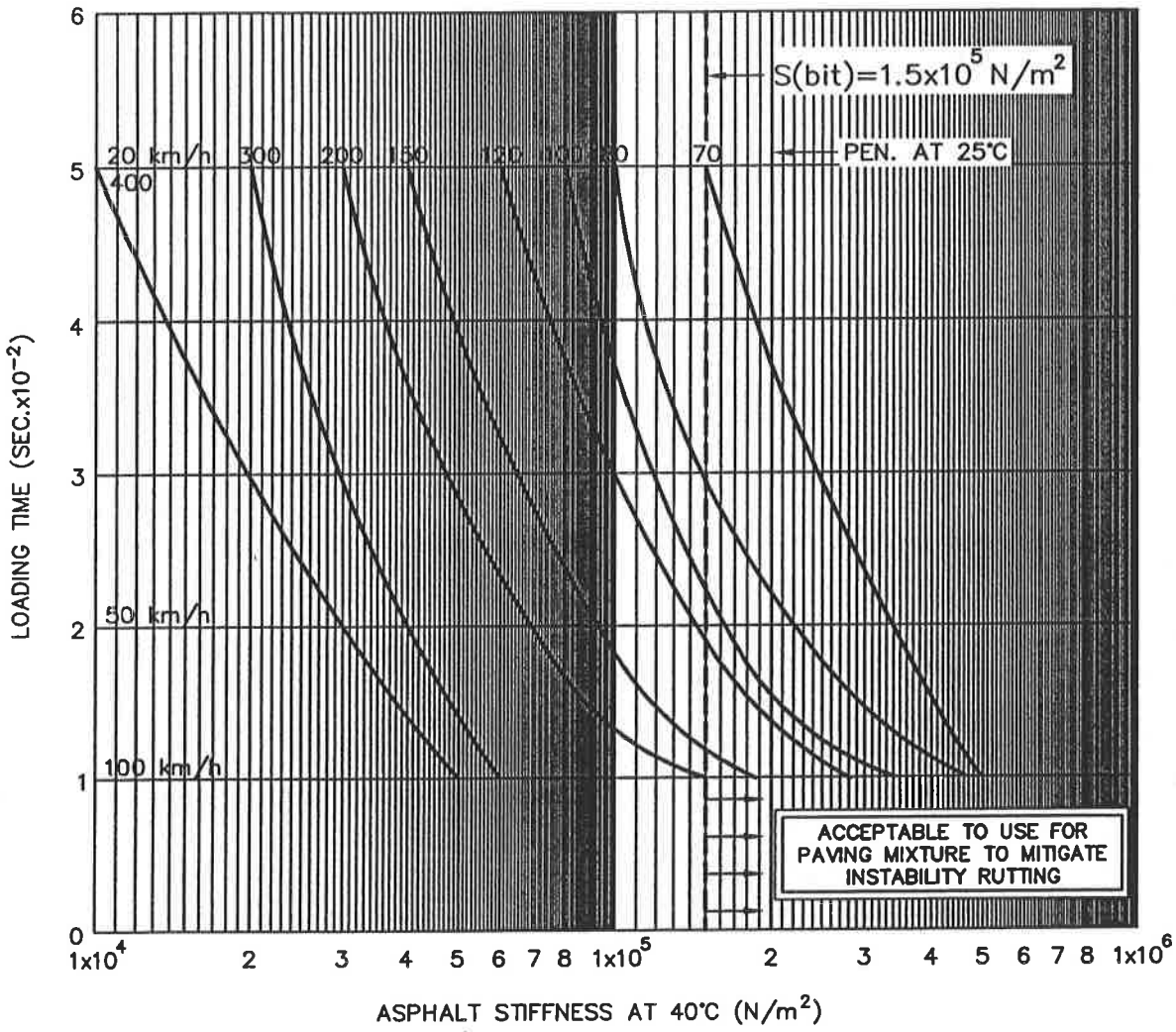
FIGURE 8 STIFFNESS OF CGSB TANK ASPHALTS (GROUP B) AT 40°C AND VARIABLE LOADING TIME



Notes: *1) Service conditions that prevail where the heavy lines are plotted (• • • • • •) would have to be satisfied using a modified asphalt binder.*
2) Section 6.5.2 should be read in conjunction with this Figure.

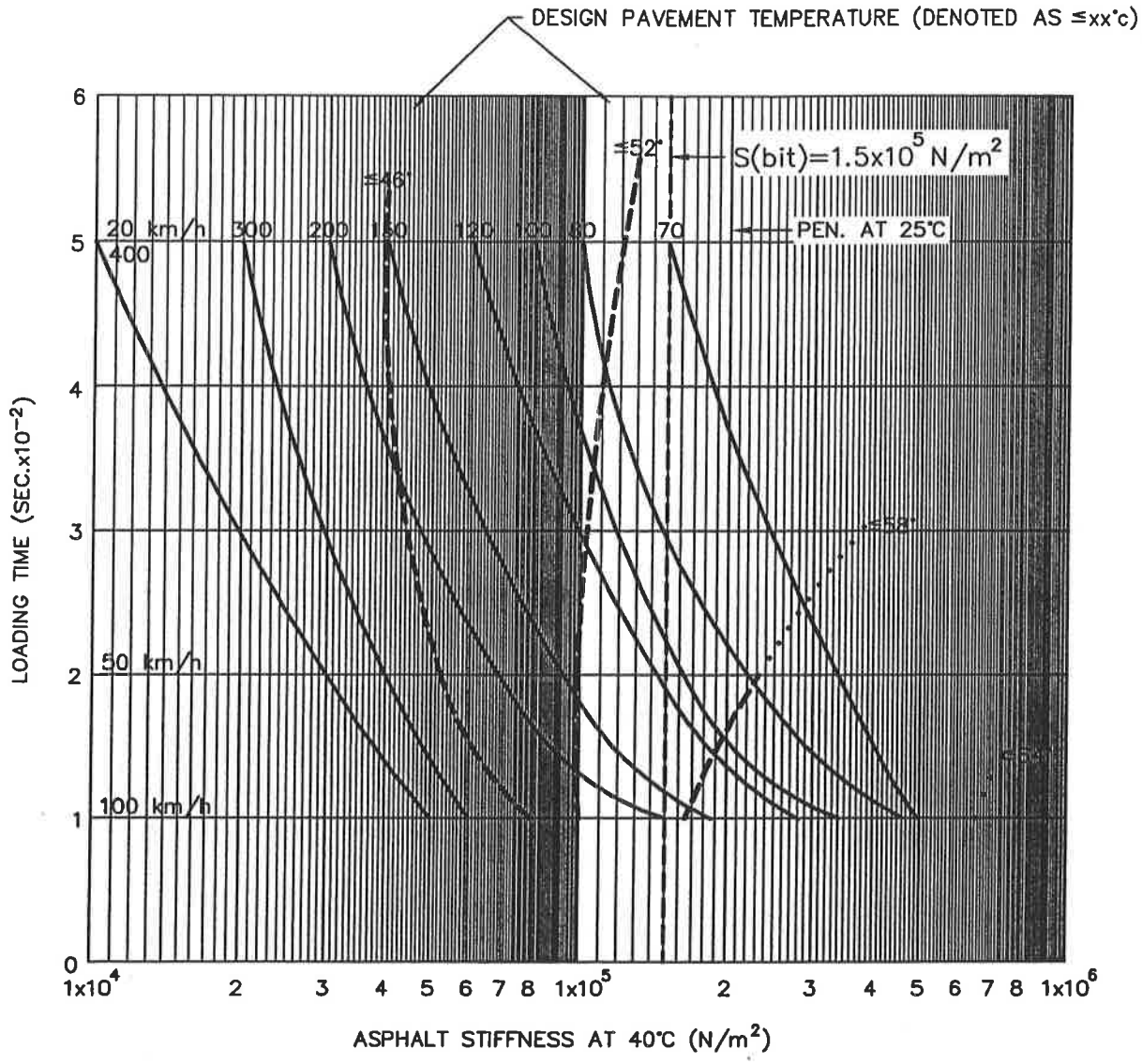
9754–M138

FIGURE 8A CGSB GROUP B ASPHALTS WITH MAXIMUM DESIGN PAVEMENT TEMPERATURE ISOTHERMS SUPERIMPOSED



9734–M23H

FIGURE 9 STIFFNESS OF CGSB TANK ASPHALTS (GROUP C) AT 40°C AND VARIABLE LOADING TIME



Notes: *1) Service conditions that prevail where the heavy lines are plotted (• • • • • •) would have to be satisfied using a modified asphalt binder.*
2) Section 6.5.2 should be read in conjunction with this Figure.

9754–M14B

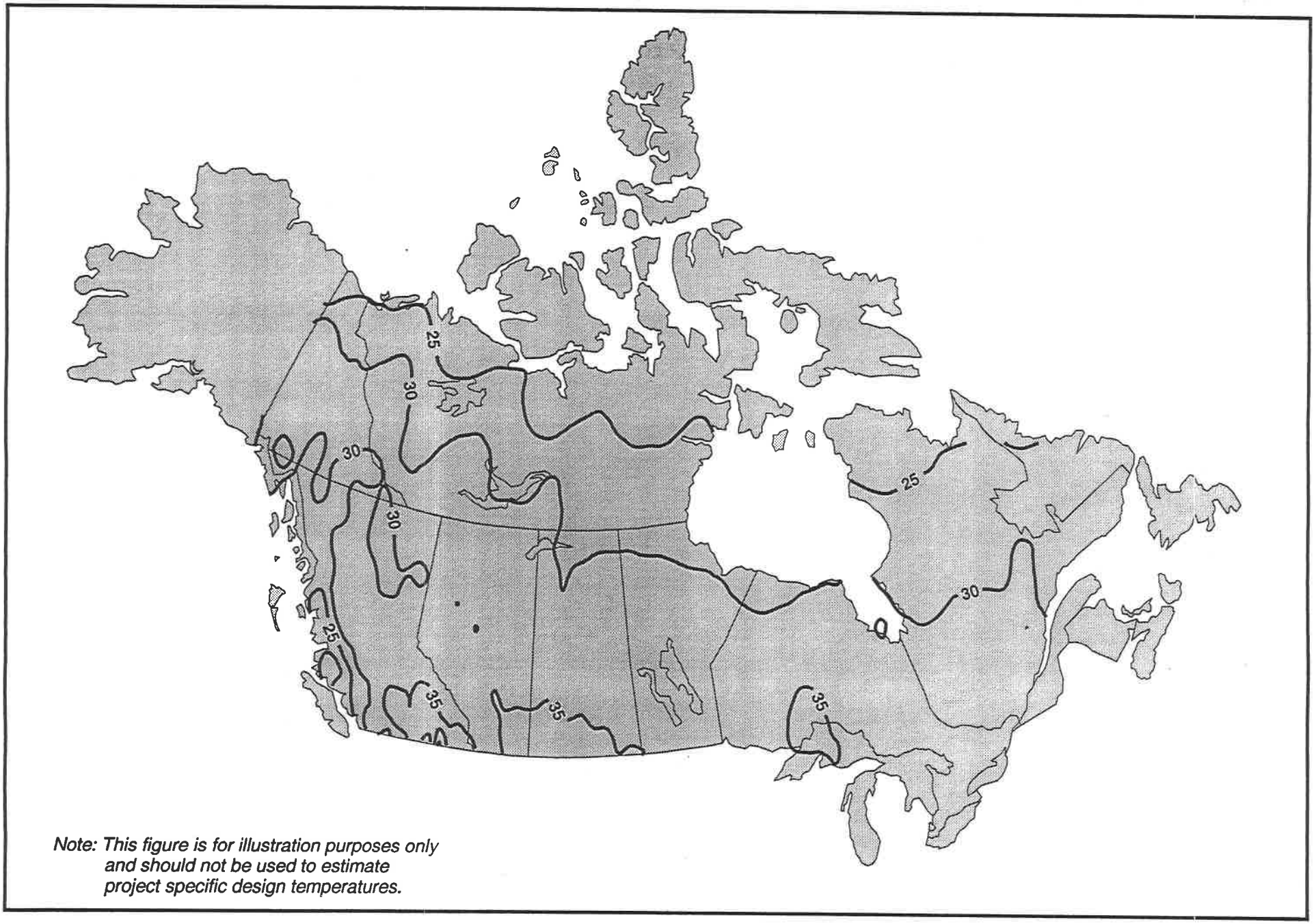
FIGURE 9A CGSB GROUP C ASPHALTS WITH MAXIMUM DESIGN TEMPERATURE ISOTHERMS SUPERIMPOSED

Note: This figure is for illustration purposes only and should not be used to estimate project specific design temperatures

R9754M06D

FIGURE 10 MINIMUM DESIGN AIR TEMPERATURE/ISOTHERMS (Tair - 2 σair), °C

Note: See Section 4.4.1 for Definitions of Tair, σair

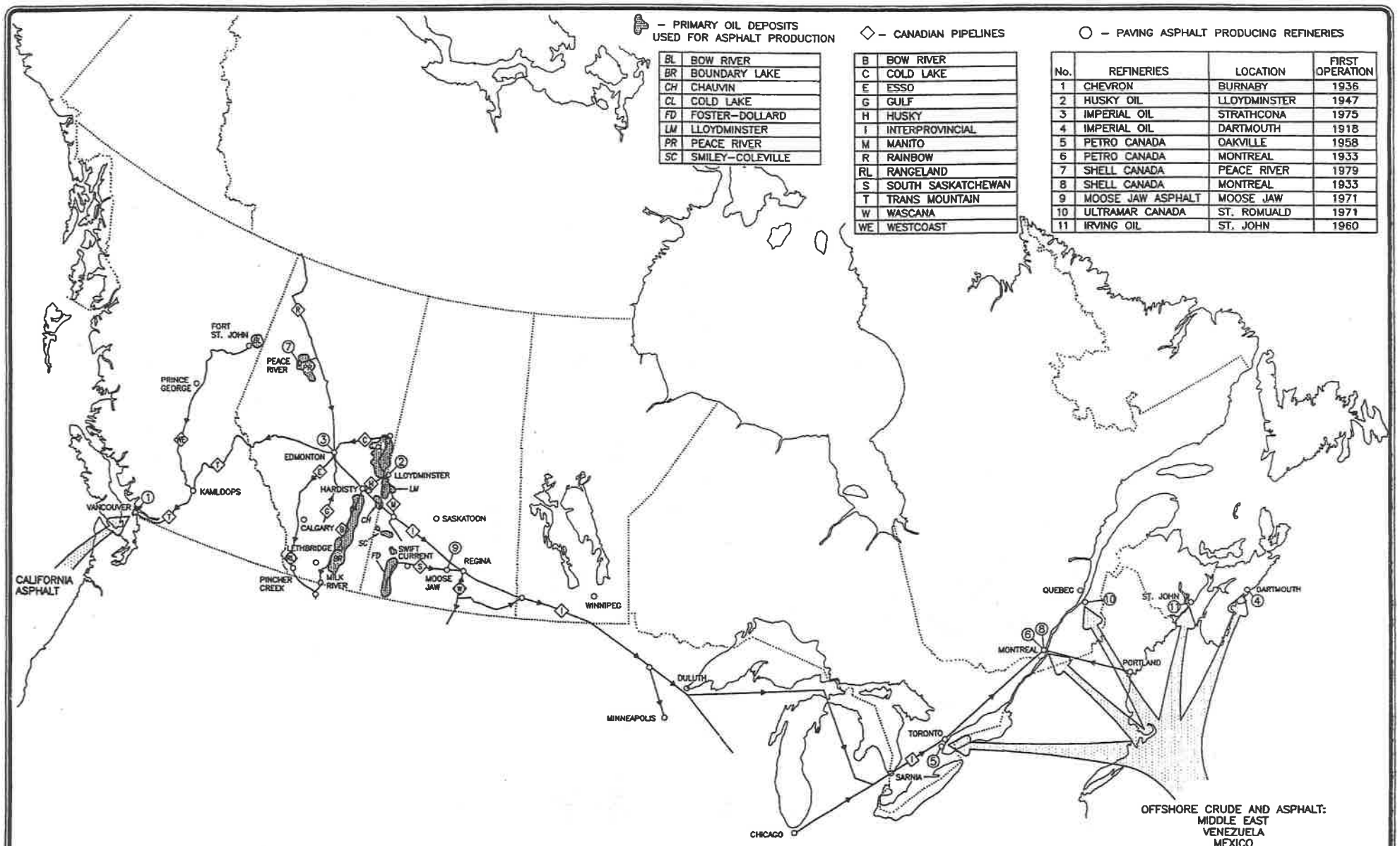


Note: This figure is for illustration purposes only and should not be used to estimate project specific design temperatures.

R9754M01C

FIGURE 11 MAXIMUM DESIGN AIR TEMPERATURE ISOTHERMS (DMAT+ 2σDMAT), °C

Note: See Section 4.4.1 for Definitions of DMAT and σDMAT



– PRIMARY OIL DEPOSITS USED FOR ASPHALT PRODUCTION

BL	BOW RIVER
BR	BOUNDARY LAKE
CH	CHAUVIN
CL	COLD LAKE
FD	FOSTER–DOLLARD
LM	LLOYDMINSTER
PR	PEACE RIVER
SC	SMILEY–COLEVILLE

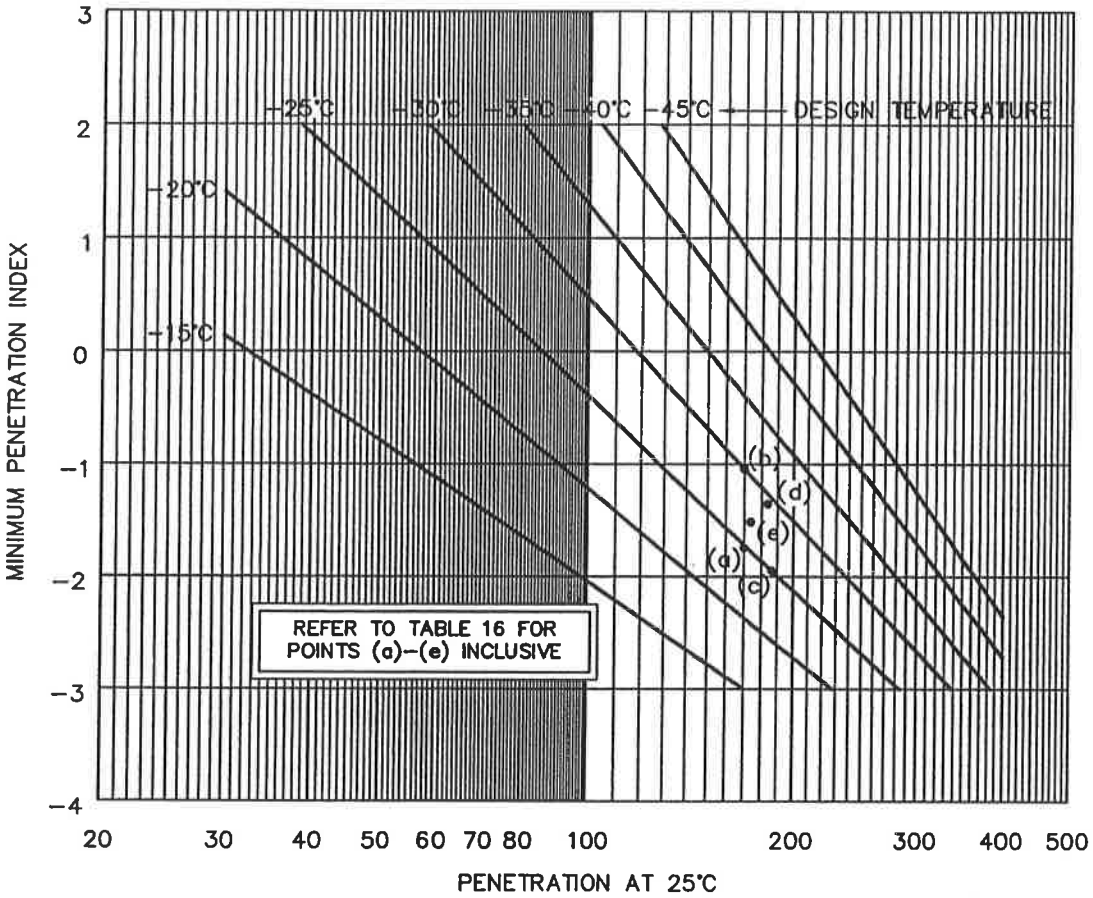
◇ – CANADIAN PIPELINES

B	BOW RIVER
C	COLD LAKE
E	ESSO
G	GULF
H	HUSKY
I	INTERPROVINCIAL
M	MANITO
R	RAINBOW
RL	RANGELAND
S	SOUTH SASKATCHEWAN
T	TRANS MOUNTAIN
W	WASCANA
WE	WESTCOAST

○ – PAVING ASPHALT PRODUCING REFINERIES

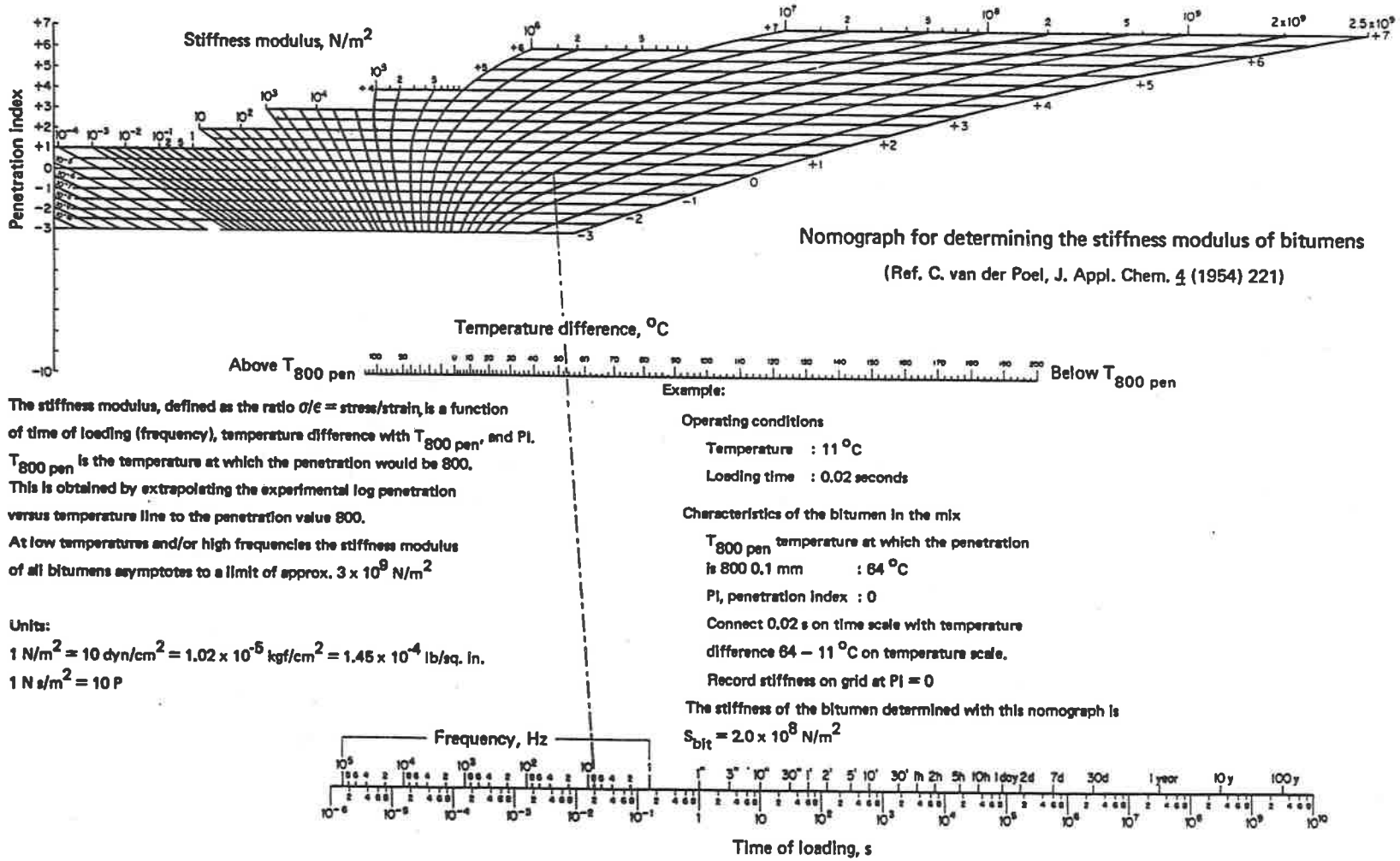
No.	REFINERIES	LOCATION	FIRST OPERATION
1	CHEVRON	BURNABY	1936
2	HUSKY OIL	LLOYDMINSTER	1947
3	IMPERIAL OIL	STRATHCONA	1975
4	IMPERIAL OIL	DARTMOUTH	1918
5	PETRO CANADA	OAKVILLE	1958
6	PETRO CANADA	MONTREAL	1933
7	SHELL CANADA	PEACE RIVER	1979
8	SHELL CANADA	MONTREAL	1933
9	MOOSE JAW ASPHALT	MOOSE JAW	1971
10	ULTRAMAR CANADA	ST. ROMUALD	1971
11	IRVING OIL	ST. JOHN	1960

FIGURE 12 – SOURCES OF CRUDE OIL FOR CANADIAN REFINERIES



9734−M08E

FIGURE 13 ASPHALT SELECTION CHART (AFTER ROBERTSON)



The stiffness modulus, defined as the ratio σ/ϵ = stress/strain, is a function of time of loading (frequency), temperature difference with $T_{800\ pen}$, and PI.

$T_{800\ pen}$ is the temperature at which the penetration would be 800.

This is obtained by extrapolating the experimental log penetration versus temperature line to the penetration value 800.

At low temperatures and/or high frequencies the stiffness modulus of all bitumens asymptotes to a limit of approx. 3×10^9 N/m²

Units:

1 N/m² = 10 dyn/cm² = 1.02×10^{-5} kgf/cm² = 1.45×10^{-4} lb/sq. in.

1 N s/m² = 10 P

Example:

Operating conditions

Temperature : 11 °C

Loading time : 0.02 seconds

Characteristics of the bitumen in the mix

$T_{800\ pen}$ temperature at which the penetration is 800 0.1 mm : 64 °C

PI, penetration index : 0

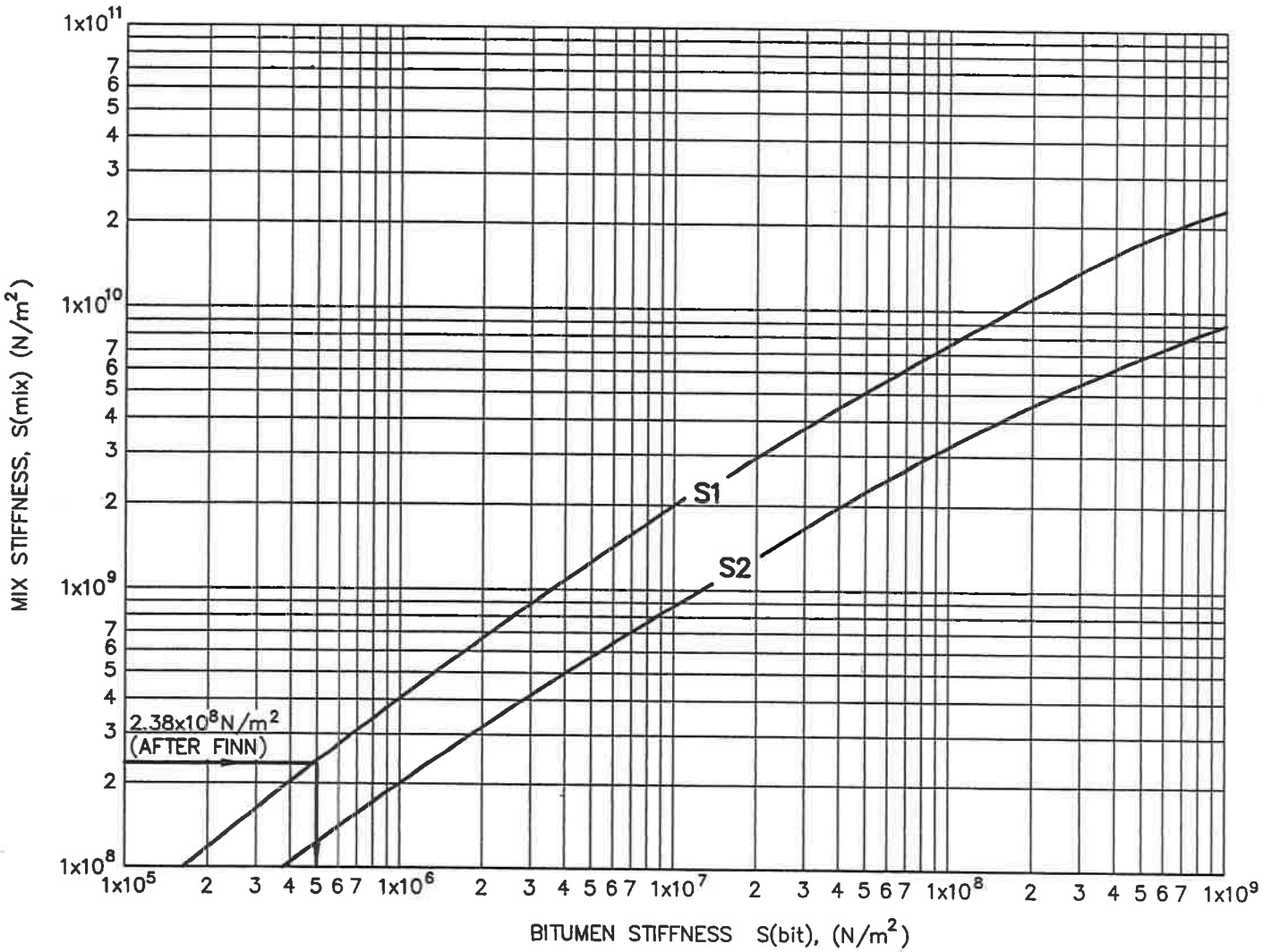
Connect 0.02 s on time scale with temperature difference 64 – 11 °C on temperature scale.

Record stiffness on grid at PI = 0

The stiffness of the bitumen determined with this nomograph is

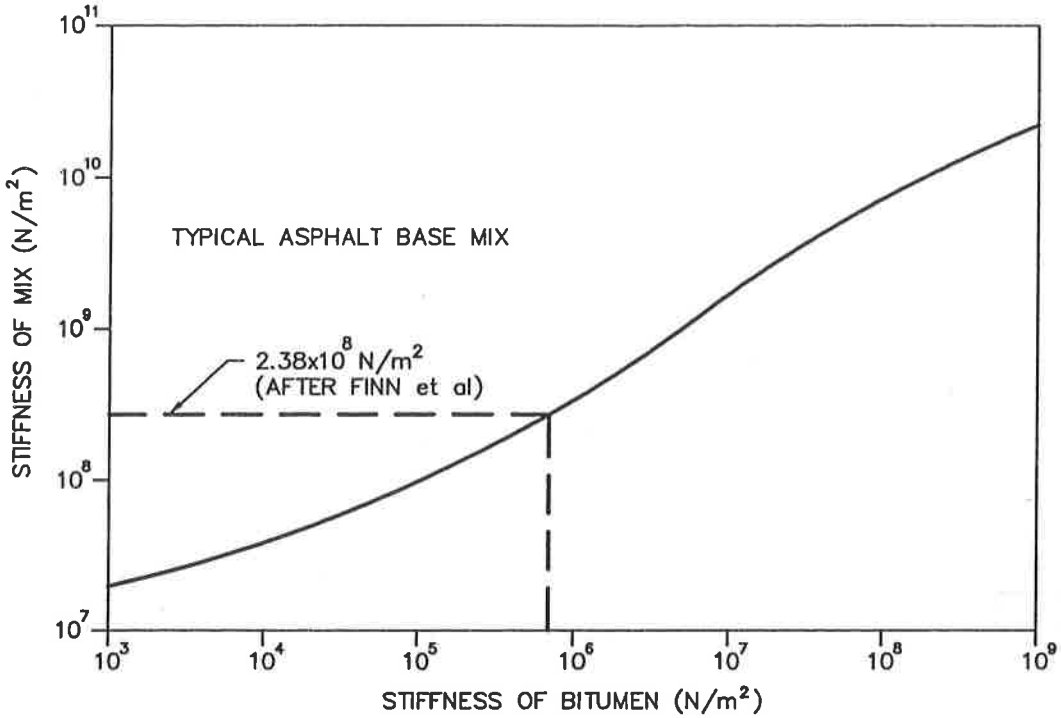
$S_{bit} = 2.0 \times 10^8$ N/m²

FIGURE 14. VAN DER POEL's NOMOGRAPH FOR DETERMINING THE STIFFNESS MODULUS OF BITUMEN



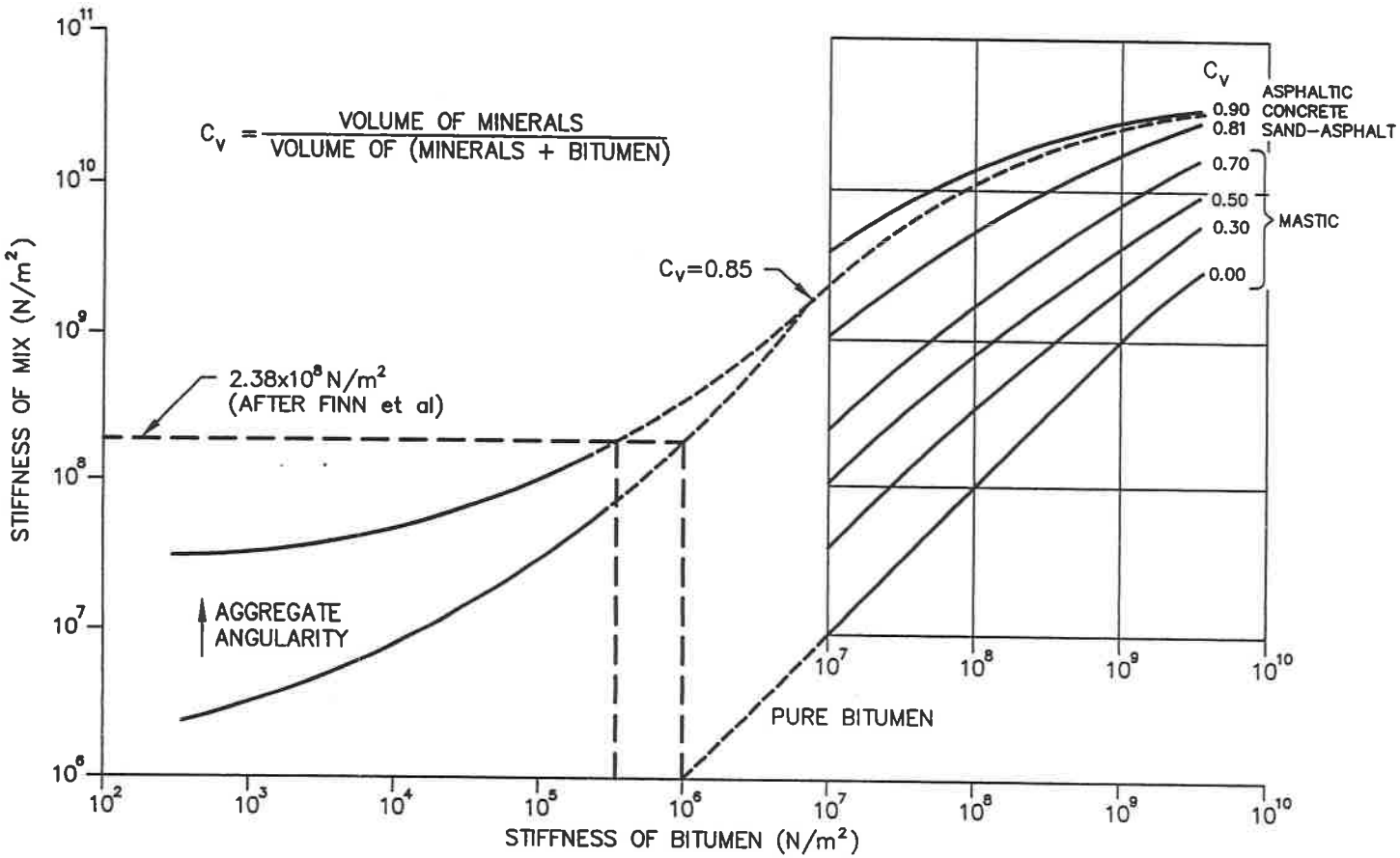
9734–M07D

FIGURE 15 CHARACTERISTIC RELATIONSHIPS BETWEEN MIX STIFFNESS AND BITUMEN STIFFNESS (FROM SHELL PAVEMENT DESIGN MANUAL)



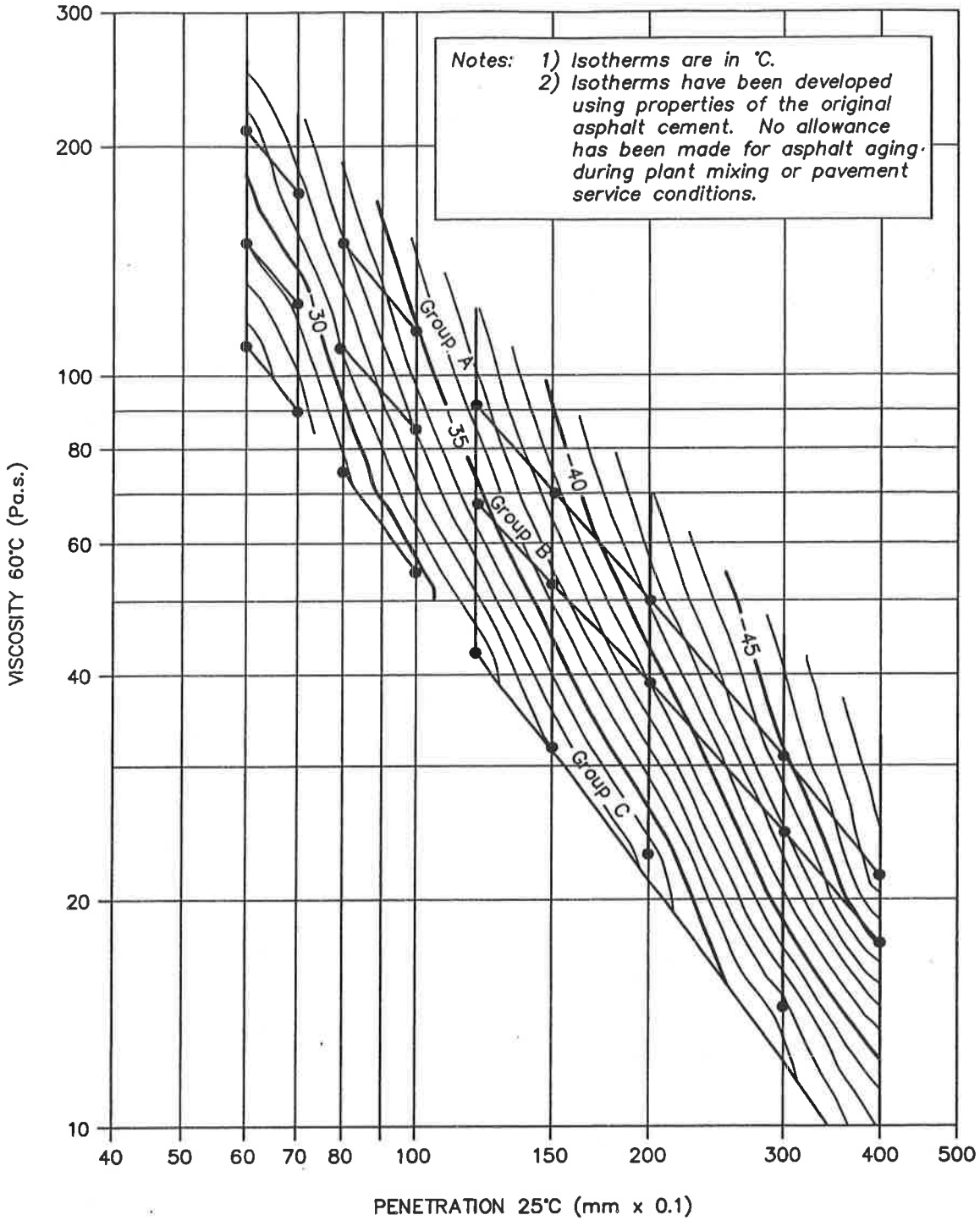
9734-M24E

FIGURE 16 STIFFNESS OF MIX/STIFFNESS OF BITUMEN (AFTER EDWARDS AND VALKERING, 1974)



9734–M22E

FIGURE 17 GENERAL RELATION BETWEEN STIFFNESS OF INITIALLY AGED BITUMEN AND STIFFNESS OF MIXES (FROM INTERNAL SHELL TEXT)



9754-C01H

FIGURE 18 CRACKING TEMPERATURES VERSUS CGSB ASPHALT GRADE

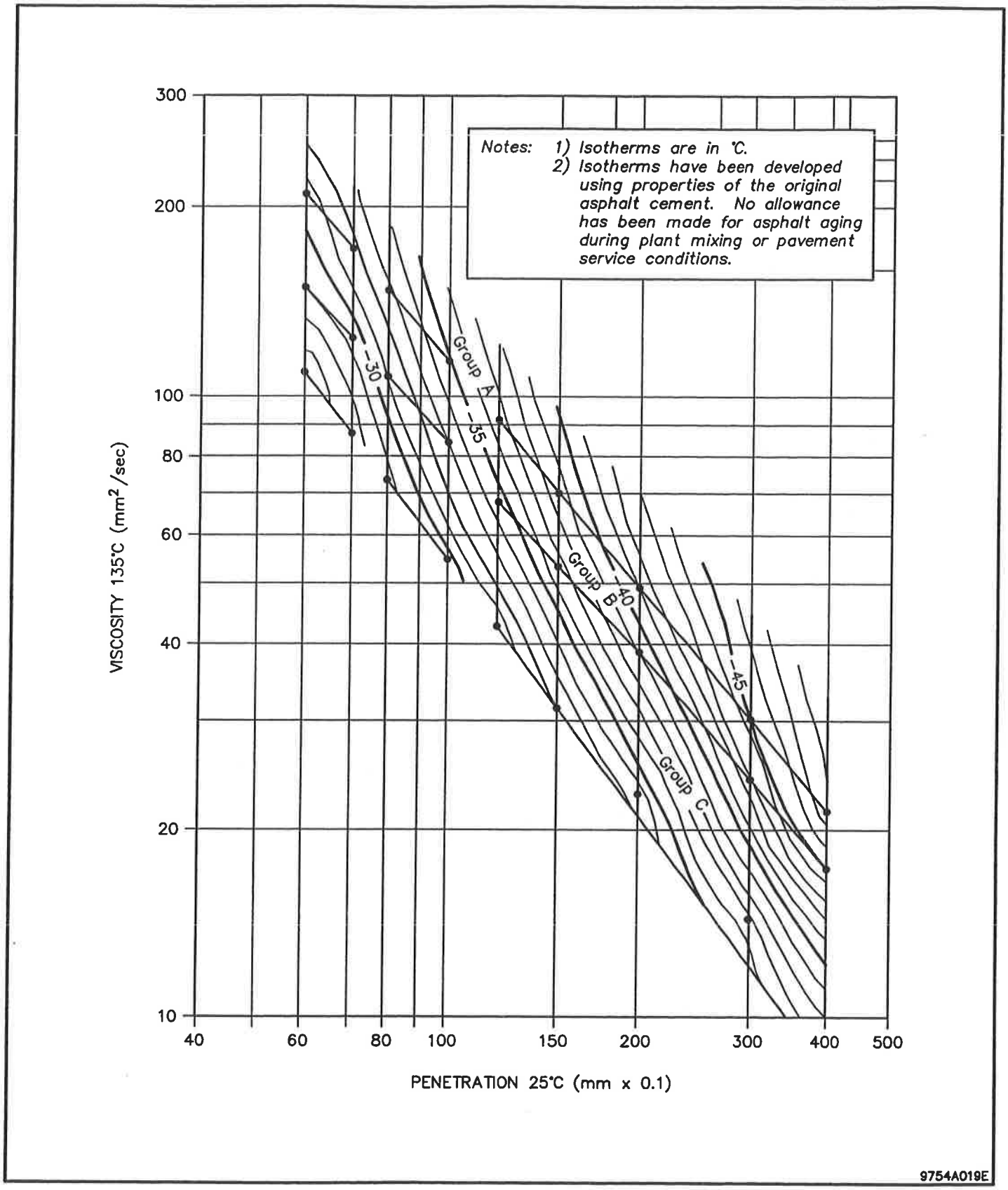
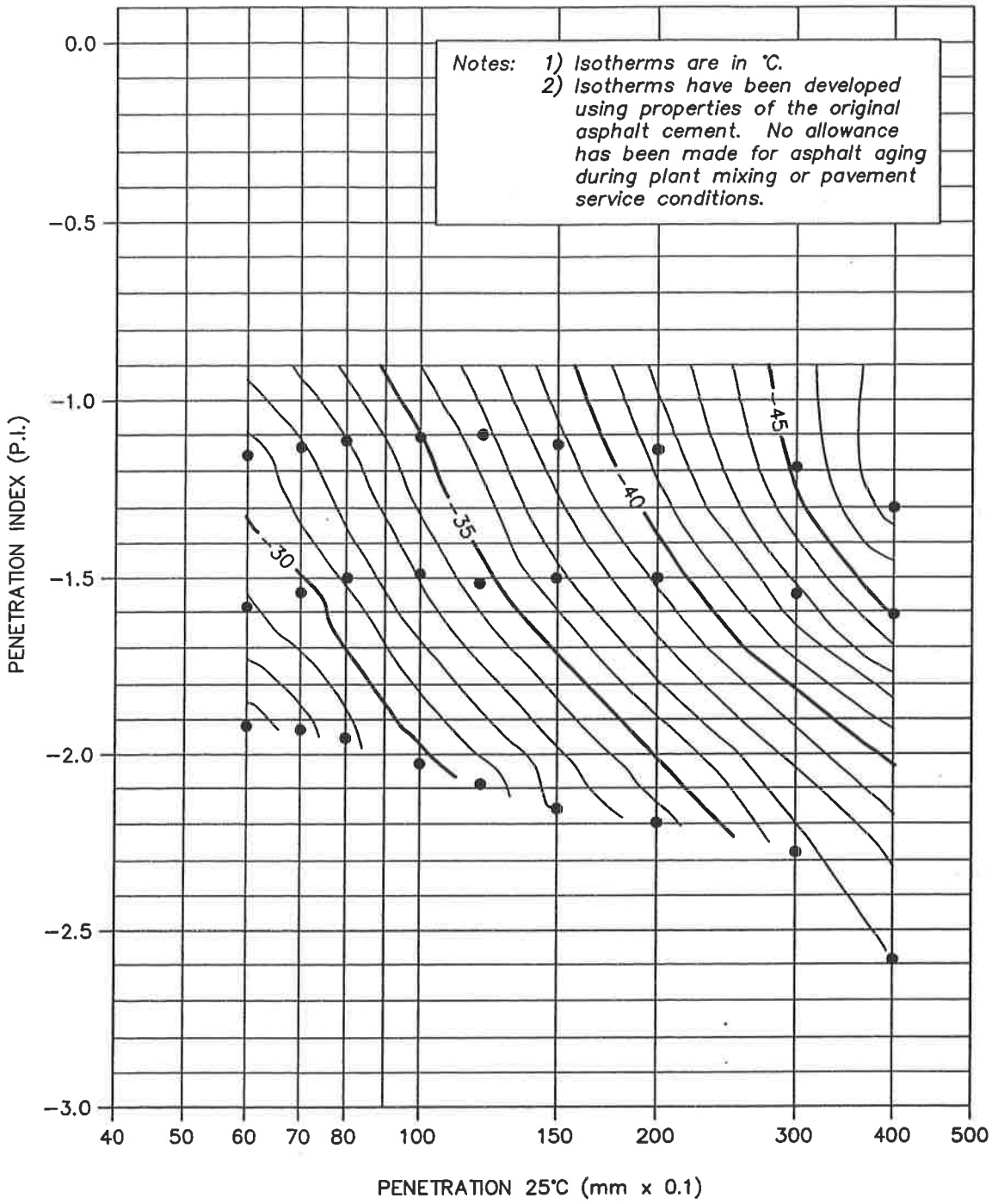


FIGURE 19 CRACKING TEMPERATURES VERSUS CGSB ASPHALT GRADE (FIGURE 2)



9754-C02E

FIGURE 20 CRACKING TEMPERATURES VS PENETRATION AND PENETRATION INDEX (P.I.)

APPENDIX A

AGENCY SURVEY OF PAVING ASPHALTS USED IN CANADA

	1.0 Specifications For Asphalt Cement.							
	1.1 Department's specifications for asphalt cement.		1.2 In use since.	1.3 Last modified.	1.4 Reason for modification.	1.5 Do you anticipate changes in your AC specs? 1.6 If so, what changes do you anticipate?	1.7 What is your opinion of the adequacy of your AC specification?	1.8 What areas of specification could be improved?
	Attached	Grades Specified.						
Newfoundland	No.	-	1990	1994	Now request only Group "A" asphalt cement.	Yes. Low temperature cracking susceptibility	Somewhat lacking.	Temperature susceptibility.
Nova Scotia	Yes.	80/100, 150/200, 200/300, CGSB	1960	1991	Changes to CGSB.	Yes. SHRP	Adequate.	Selection criteria and modified asphalt contents.
Prince Edward Island	No	-	1970's	1994	Added pen. grade, specified CGSB Group A	Yes. CSHRP	Adequate.	
New Brunswick	Yes.	80/100, 120/150, 150/200, 200/300, 300/400	1969	1993	Eliminated viscosity grades.	Yes. SHRP	Somewhat lacking.	Temperature susceptibility.
Quebec	Yes.	60/70, 80/100, 150/200, 200/300 300/400.	1988	1993	Addition of high and low service temperature tests. (Pen. @ 4°C and softening point)	Yes. Specifications of polymer or modified asphalts. Implementation of SHRP. Traditional test methods used in conjuction with SHRP methods.	Adequate.	Polymer modified asphalt.
Ontario	Yes.	60/70, 85/100, 100/200, 300/400	1960's	1989	0	Yes. SHRP	Adequate.	End result type specification and include polymer and engineered asphalt.
Manitoba	Yes.	120/150A, 150/200, 200/300, 200/400, PMA	1985	1985	Return to Pen. grading, but still classifying the various grades according to viscosity.	No.	Very appropriate.	None.

	1.0 Specifications For Asphalt Cement.							
	1.1 Department's specifications for asphalt cement.		1.2 In use since.	1.3 Last modified.	1.4 Reason for modification.	1.5 Do you anticipate changes in your AC specs? 1.6 If so, what changes do you anticipate?	1.7 What is your opinion of the adequacy of your AC specification?	1.8 What areas of specification could be improved?
	Attached	Grades Specified.						
Saskatchewan	Yes.	150/200(A), 200/300(A), 300/400(A), 400/500(A), 200/300(B), 300/400(B)	1985	1992	Payment adjust for non-spec. asphalt, Removed upper limit for A Grade AC.	No.	Very appropriate.	Adopt performance based specifications, ie. SHRP.
Alberta	No.	150/200(A), 200/300(A), 300/400(A), 200/300(B), 300/400(B)	1980	1980	To better address temperature susceptibility.	No.	Very appropriate.	Tests to define absolute and fundamental physical and chemical properties to aid design.
British Columbia	Yes.	80/100, 150/200, 200/300	1993	1993	Changed to CGSB due to temperature susceptibility.	Yes SHRP in the future when SHRP has been refined and verified.	Very appropriate.	-
Yukon	Yes.	CGSB	1989	1989	Developed in 1989.	No.	Adequate.	None
Public Works Canada	Yes.	CGSB	CGSB		N/A	N/A	N/A	N/A
City of Calgary	Yes.	150/200, 200/300, 300/400	20+ years	1992	To update specifications.	Yes. SHRP	Very appropriate.	None
City of Edmonton	Yes.	150/200, 200/300(A), 300/400(A), 200/300(B), 300/400(B)	1962	1986	To conform to AT&U.	Yes. SHRP	Excellent.	None

	1.0 Specifications For Asphalt Cement. (cont'd)				
	1.9 What changes did CAN/CGSB-16.3-M90 cause to your specifications?	1.10 Is temperature susceptibility considered in selection of asphalt cement, How?	How is it determined?	1.11 How do you determine the max-min in-service temperature?	1.12 How do you determine max-min design asphalt pavement temperatures?
Newfoundland	Adopted CGSB.	Yes. Specify Group A.	Pen -Vis Number.	Max.-average high for hottest week on record. Min.-lowest temp. on record.	Max extreme +10°C, Min extreme
Nova Scotia	Group A from Group B asphalt	No.	-	Assume.	Use Asphalt Institute Hwy Program, 7°C average.
Prince Edward Island	High volume roades must meet Group A	No.		N/A	
New Brunswick	Adopted CGSB	Yes. Specify Group A.	Specify Group A.	Not used .	Not used .
Quebec	Rational method used to choose asphalt cement with low thermal susceptibility. Only Group A now specified.	Yes. Specify Group A.	Pen -Vis Number.	There is no policy concerning this type of determination.	There is no policy concerning this type of determination.
Ontario	None, good performance of existing specification.	No. (Use different asphalt grades in problem areas)	-	Based on past performance experience, different AC grade used for different parts of the province for varing climatic conditions.	N/A
Manitoba	Grading by penetration but classifying by viscosity.	Yes. Low temperature susceptibility is not acceptable due to increased cracking.	Penetration and premium grades.	Estimate based on air temperature.	Not routinely used. High quality asphalt cements compensate for not having "pavement design temp."

	1.0 Specifications For Asphalt Cement. (cont'd)				
	1.9 What changes did CAN/CGSB-16.3-M90 cause to your specifications?	1.10 Is temperature susceptibility considered in selection of asphalt cement, How?	How is it determined?	1.11 How do you determine the max-min in-service temperature?	1.12 How do you determine max-min design asphalt pavement temperatures?
Saskatchewan	Removal of upper limit on viscosity for A Grade AC.	Yes. Include thin film oven requirements and Pen-Vis relationship is controlled.	Penetration and absolute viscosity at 60C.	N/A	Historical information.
Alberta	None, Alberta asphalts tend to have higher minimum viscosity limits	Yes. Penetration index and penetration-viscosity number	Penetration @25°C, PVN using viscosity @60C	N/A	-
British Columbia	Adopted CGSB due to clearly defined temperature susceptibility	Yes Using Group A, B and C grades	Penetration @25°C, viscosity @60C	Use different AC Groups for varied environmental conditions (see 1.13 Climate). With this conservative approach no need for precise temp.	Where min. required use min. on record. Max. temp. will be average max. as specified by SHRP.
Yukon	None, specification refers to CGSB.	No.	-	Not normally done. Historical max. and min. generally considered in AC selection.	Not applied.
Public Works Canada	N/A	Yes. Site freezing index.	Group A asphalt for site freezing index over 500.	N/A	-
City of Calgary	None, present specification covers relevant points.	Yes. Consider low temperature cracking and high temperature deformation.	Penetration index	N/A	-
City of Edmonton	None, City follows Provincial specifications.	Yes. Chart for penetration-viscosity relationship.	Penetration-viscosity number.	As specified By Alberta Transportation. City pavements assumed to be within same temperature regime for AC selected.	See 1.11

1.0 Specifications For Asphalt Cement. (cont'd)

1.13 How many grades of paving AC do you use? If more than one, what factors are considered?

	Number	Climate	Traffic	AC stiffness	Mix stiffness	Aggregate properties	Other criteria
Newfoundland	1	-	-	-	-	-	-
Nova Scotia	2	Yes.	No.	Yes.	No.	No.	No.
Prince Edward Island	4	No	Yes	Yes	No	No	-
New Brunswick	4	Yes.	Yes.	Yes.	No.	Yes.	Yes.
Quebec	3	Yes.	Yes. High volume use modified asphalt cement with high stiffness; high performance less workable.	Yes.	No.	No.	-
Ontario	5-6	Yes. Northern and NW regions softer grades preferred. Polymer /engineered asphalts so far have been considered for high traffic use.	Yes. 5000+ AADT use 85/100, 5000-AADT use 150/200 or 300-400.	-	-	-	-
Manitoba	4	150/200A or 120/150A used in all virgin mixes. Due to low temp. cracking characteristics 120/150A is used in mixes placed over PC concrete pavements subjected to high traffic volumes.	Yes.	Yes.	No.	No.	No.

1.0 Specifications For Asphalt Cement. (cont'd)

1.13 How many grades of paving AC do you use? If more than one, what factors are considered?

	Number	Climate	Traffic	AC stiffness	Mix stiffness	Aggregate properties	Other criteria
Saskatchewan	4	Yes. Use only A Grade.	Yes. Higher volumes require harder AC.	Yes.	Yes.	No.	Yes. Recycling.
Alberta	3	Yes. Refer to CTAA paper 1990	Yes.	Yes.	-	-	-
British Columbia	5 or 6	Yes. Wet or dry freeze zones and high traffic use Class A. Wet no freeze use Class A or C, depending on price and traffic.	Yes.	Yes.	No.	No.	No.
Yukon	2	Yes.	Yes.	Yes.	Yes.	Yes.	-
Public Works Canada	3	Yes.	No.	No.	Yes.	No.	Yes.
City of Calgary	4	No.	Yes.	Yes.	Yes.	Yes.	Yes.
City of Edmonton	1	-	-	-	-	-	-

1.0 Specifications For Asphalt Cement. (cont'd)

	1.14 What is your department's annual use of each grade of asphalt since 1990?					1.15 What companies supply asphalt cement to your department?
	AC grade	1990 (tonnes)	1991 (tonnes)	1992 (tonnes)	1993 (tonnes)	
Newfoundland	150/200	36116	23695	28023	25122	Ultramar Canada, Newfoundland Hardwoods Ltd.
Nova Scotia	150/200 200/300	30000 7000	37000 10000	39000 11000	34000 8000	Esso Irving
Prince Edward Island						Ultramar, Imperial, Irving Oil
New Brunswick	AC 20 AC 10 80/100 120/150 150/200 200/300 300/400	457 3161 0 0 20800 533 1533	4303 10188 0 0 18383 3296 3203	0 12469 80 2996 18721 2004 796	0 0 0 7464 25445 6235 602	Brunswick Asphalt, Bitumar, Ultramar, Barrett, Petrocan, Imperial Oil.
Quebec	80/100 150/200 PMA Engineered Asphalts These totals do not include AC provided by the contractor.	52652 9167 5045 -	55624 29250 7000 -	109647 12099 3381 -	74080 8695 3260 3333	One company provides special bitumans (Bitumar). Most common purchases come from the refineries.
Ontario	85/100 150/200 PMA (Minimal use)	100000 25000	70000 20000	100000 24000	115000 35000	Amoco Oil, Ashwarren, Bitumar, Canadian Asphalt, Esso, Husky Oil, Iko Industries, Koch Materials, McAsphalt Industries, Michigan Marine, Norjohn, Petro Canada, Polymac, Shell, Ultramar
Manitoba	120/150A 150/200 200/300 200/40 PMA PMA used on experimental basis only.	15307 17088	9565 9808	10334 15125 1750 176 22	5146 35485 783	Esso (Imperial), Husky, Koch, Moose Jaw.

	1.0 Specifications For Asphalt Cement. (cont'd)					
	1.14 What is your department's annual use of each grade of asphalt since 1990?					1.15 What companies supply asphalt cement to your department?
	AC grade	1990 (tonnes)	1991 (tonnes)	1992 (tonnes)	1993 (tonnes)	
Saskatchewan	150/200A 200/300A 300/400A 400/500A	12987 20726 19099 4720	7527 15507 - 6033	13491 9713 3940 2906	19671 18157 5263 1887	Imperial Oil, Husky, Moose Jaw Asphalt.
Alberta	Quantities not available since introduction of E.P.S.	-	-	-	-	Husky, Esso, Shell, Moose Jaw Asphalt
British Columbia	85/100 AC 8 150/200 AC 5 80/100 A* 80/100 B 80/100 C 150/200 A 200/300 A PMA * New ministry spec.	63452 13209 0 0 0 0 0 2970	72331 4908 0 0 0 0 0 0	64413 2327 0 0 0 0 0 213	0 0 4956 0 1989 36400 878 0	Esso, Husky, Shell, Chevron, Mctar Asphalt
Yukon	150/200 A 200/300 A	0 0	0 0	0 0	1060 825	-
Public Works Canada	Quantities not available.	-	-	-	-	N/A
City of Calgary	Quantities not available.	-	-	-	-	Husky Oil, Imperial Oil
City of Edmonton	AC 150/200	14520	8030	8552	13475	Husky Oil, Imperial Oil

	1.0 Specifications For Asphalt Cement. (cont'd)					
	Which refineries supply your AC?	1.16 Do you know the crude oil sources?	Would you like to know the sources?	1.17 Does the source of the refinery affect your selection of AC type?	Does the refinery affect your selection of the AC type?	1.18 Is your department actively reviewing the SHRP binder Specifications?
Newfoundland	Ultramar Canada, St. Romwald Que.; Irving Oil, St. John, NB; Imperial Oil, Dartmouth, NS; Petro Canada.	Some. Saudi Arabia, Venezuela	-	No.	No.	Yes.
Nova Scotia	Dartmouth, NS; St. John, NB	No.	Yes.	No.	Yes.	Yes.
Prince Edward Island	Dartmouth, NS; St. John, NB St. Romuald, PQ	No.	Yes.	No.	Yes. All products not available from all	Yes
New Brunswick	Irving Oil, Ultramar(Montreal), Petrocan(Montreal), Imperial Oil (Dartmouth, NS).	Some	Yes.	No.	No.	Yes.
Quebec	Ultramar, PetroCanada, Shell, Esso	Ultramar: BEE 17 and Soudieh. Shell: Arab, Maya and a mix of North Sea crude. Petro-Canada: Mexico and Venezuela.	-	No.	No.	Yes.
Ontario	-	Some	-	No.	No.	Yes.
Manitoba	Husky-Lloydminster, Moose Jaw Oil-Moose Jaw, Imperial Oil-Edmonton, Koch-Minneapolis.	Husky - Lloydminster Moose Jaw Oil - Cold Lake Imperial Oil - Cold Lake Koch - Cold Lake and various US sources blended in.	-	No.	No.	Yes.

	1.0 Specifications For Asphalt Cement. (cont'd)					
	Which refineries supply your AC?	1.16 Do you know the crude oil sources?	Would you like to know the sources?	1.17 Does the source of the refinery affect your selection of AC type?	Does the refinery affect your selection of the AC type?	1.18 Is your department actively reviewing the SHRP binder Specifications?
Saskatchewan	Esso, Strathcona AB; Husky, Lloydminster; Moose Jaw Asphalt, Moose Jaw.	Some. Cold Lake Lloydminster Southwest Saskatchewan.	Yes.	No.	No.	Yes.
Alberta	Husky, Lloydminster; Esso, Edmonton; Shell, Three Creeks; Moose Jaw Asphalt, Moose Jaw.	Yes. Three Creeks AB, Cold Lake Lloydminster	-	No.	No.	Yes.
British Columbia	Esso, Strathcona AB; IOCO, Port Moody, BC; Husky, Lloydminster; Shell, Three Creeks; Chevron, Burnaby.	Some. Esso-Cold Lake; Husky-Lloydminster; Shell-Three Creeks AB	Yes.	Yes. Light crude AC is Grade C	Yes. Some refineries produce only Grade C.	Yes.
Yukon	Not known.	No.	Yes.	No.	No.	No.
Public Works Canada	N/A	No.	No.	No.	No.	Yes.
City of Calgary	Husky	N/A	-	-	-	Yes.
City of Edmonton	Husky-Lloydminster; Imperial-Edmonton.	Some. Husky-Lloydminster; Esso-Cold Lake	Yes.	No.	No.	Yes.

	1.0 Specifications For Asphalt Cement. (cont'd)				
	1.19 Do you feel your specifications will change?	How long do you think it will take to fully implement the SHRP binder specs?	1.20 Are you aware of any changes in the petrochemical industry which have impacted on the characteristics or performance of ACs?	1.21 Have you conducted any tests which show that asphalt cement properties are changing over time?	1.22 Would you like oil companies to provide test data required for SHRP specifications?
Newfoundland	Partially.	-	No.	No.	Yes. We would like to know if their products meet SHRP specification.
Nova Scotia	Yes.	3 Years.	No.	No.	Yes.
Prince Edward Island	Unsure			No.	Yes.
New Brunswick	Yes.	Depends on testing of testing equipment.	No.	No.	Yes.
Quebec	Yes.	2 Years.	Yes, since 1974 the quality of asphalt cement has changed. No detailed follow-up was conducted here, but MTQ has abandoned the Abson method of AC recovery.	Recovery of asphalt carried out during the last 3-4 years was performed using a method similar to that proposed by SHRP	Yes. Viscosity, Brookfield, and minimum flashpoint.
Ontario	Yes.	2 Years. Full set of binder test equipment purchased, screened and parallel testing of asphalt cement planned.	No.	No. (Industry has always produced AC to meet specification)	Yes. We do have some results from the oil companies showing how their ACs fit with SHRP superpave specification.
Manitoba	Partially.	5 Years. Our department has purchased the binder equipment and is in the process of calibrating and performing trial teste.	No.	No.	Yes. This information would be used to verify test results with our SHRP binder lab.

	1.0 Specifications For Asphalt Cement. (cont'd)				
	1.19 Do you feel your specifications will change?	How long do you think it will take to fully implement the SHRP binder specs?	1.20 Are you aware of any changes in the petrochemical industry which have impacted on the characteristics or performance of ACs?	1.21 Have you conducted any tests which show that asphalt cement properties are changing over time?	1.22 Would you like oil companies to provide test data required for SHRP specifications?
Saskatchewan	Yes.	2 to 3 Years. SHRP equipment refinement and availability. Agency budget for equipment acquisition. Responce of industry and suppliers.	Emulsifiers in crude oil recovery process affecting quality and performance of AC.	Yes. Penetration @ 4C and 25C and viscosity @ 60C and 135C . This could be used to see if Pen-Vis change with time.	Yes. It would show how current asphalts conpare to the SHRP specifications.
Alberta	Unsure	5 to 10 Years. To meet SHRP specifications most Alberta asphalts would require polymers additives and for low traffic loads this may be too costly.	Yes. Polymer modified asphalts	Yes. Bituminous test charts are plotted annualy for each asphalt grade and supplier. No significant changes have been noted.	Not at this time. We have graded all of our ACs using SHRP binder test equipment. SHRP test data not required on regular basis because of very consistant product from suppliers.
British Columbia	Unsure	5 Years. BC does not have to change sources or refineries because we have three heavy crudes as our source . We depend on 92-4 to provide perspective along with other reports.	Yes. Polymer modified asphalts	No.	Yes. This would help us in evaluating our present specifications with the SHRP specifications.
Yukon	No.	-	No.	No.	Yes. If SHRP specifications are adopted.
Public Works Canada	Unsure	How and when the Canadian asphalt industry accepts the SHRP specification will have a definate impact on our decision of implementing the SHRP spec.	Polymer modified asphalts and engineered asphalt.	Yes. About 10 years ago, a coring and lab. testing program for a number of airports for a study on Asphalt Stripping and Thermal Cracking at Canadian Airports.	No. Unless we specify SHRP specification, otherwise it may confuse the issue of specifying the material intended.
City of Calgary	Unsure	Changes will occur as necessary to keep specification consistant with industrial practise.	No.	Nothing significant.	No. Does not matter.
City of Edmonton	Yes.	-	No.	Yes.	Yes.

	2.0 Chemical Additives				
	2.1 Do you use chemical additives for anti-stripping purposes?	How do you determine thet amount and type of antistrip used?	What are some typical chemical antistrip additives that you have used?	Have any of these additives failed to prevent stripping?	2.2 Are other chemical additives used by your department or by your contractors?
Newfoundland	Yes. Under special circumstances.	By performing immersion Marshall testing using addition rotes for anti-stripping agents.	Pave Bond Lite.	No.	Yes. Silicone, to reduce slumping of mix in the haul truck. As per manufacturer recommendations.
Nova Scotia	Yes. Routinely used.	Mix design, Lottman testing and Manufacturers info.	Under review.	N/A. Just started using additives.	No.
Prince Edward Island	No.	-	-	-	Investigations conducted in mix design only. Contractor has used hydrated lime as antistripping agent
New Brunswick	Yes. Routinely used where lab. testing indicates need.	0.5% blended with asphalt cement at the refinery. Based on modified Lottman laboratory tests.	Pavebond Special, Pavebond AP Special, Pavebond Lite, Redicote 82-S.	No. Different degrees of success depending on the aggregate source.	Yes. Silicone has been used for highly adsorbtive aggregates.
Quebec	No.	-	-	-	Yes. Some contractors use additives, but the choice, dosage, etc. is their sole responsibility.
Ontario	Yes. Routinely used.	Marshall immersion 70% retained stability.	Naicad RL-2A, Redicote 82-S, Redicote AP, Exxon Tomah 101-25-B, C-7742-C, SC-901, Pavebond AP, Pavebond AP special, Pavebond Lite, Miramine O, Kling-Beta 2550, Kling-Beta 2550HM, Lime.	All reduce stripping in the short term, long term is unknown.	Yes. Hydrated lime.
Manitoba	No.	-	-	-	Yes. APA 5 is a silicone additive used as a water releasing agent. Dosage 1 quart per 4500 gallons AC. As recommended by the manufacturer.

	2.0 Chemical Additives				
	2.1 Do you use chemical additives for anti-stripping purposes?	How do you determine thet amount and type of antistrip used?	What are some typical chemical antistrip additives that you have used?	Have any of these additives failed to prevent stripping?	2.2 Are other chemical additives used by your department or by your contractors?
Saskatchewan	Yes.	Stipping potential determined using Marshall specimens. Mix treated with additive and re-evaluated.	AK20 CES-446, AK20 95-S, ALCHEM RL-2A, Esso Corexit 7742C, Hydrated Lime.	No. Some stripping problems but not due to agent.	No.
Alberta	Not routinely.	Trial projects were evaluated using the shake bottle test.	Redicote 95-S, Corexit 7742C	No stripping evident in control section either. (inconclusive).	Yes. Silicone, by contractors to enhance the texture of the mix, and prevent buildup of material on the equipment.
British Columbia	Yes. Under special circumstances.	Immersion compression testing (Index of Retained Marshall Stability) and select best performance additive.	Kling, OPX-dispersal, Redicote, Hydrated lime, silicone, tall oil.	Yes. Varied results, some additives may be "cooked off", or the pit is too variable. Select additive suitable to the aggregate.	No. Silicone not used as a general practise.
Yukon	No.	-	-	-	No.
Public Works Canada	Yes.	Incorporate additive to mixes in various proportions and determine the optimum amount to be added.	Hydrated lime.	No.	No.
City of Calgary	No.	-	-	No.	-
City of Edmonton	No.	-	-	No.	-

	3.0 Design Properties					
	3.1 Which of these design procedures are used for design of AC mixtures?			3.2 For Marshall procedure, how many blows are specified? Mechanical or hand compactor?	Describe if lab. procedures and blows differ from specified blows.	Describe differences in blow counts due to different applications or mixes.
	Marshall (ASTM D1559)	Hveem	Superpave (SHRP)			
Newfoundland	Yes.	No.	No.	75. Hand compactor.	No.	N/A
Nova Scotia	Yes.	No.	No.	50 and 75. Hand compactor.	-	50- Low volume roads 75- All others
Prince Edward Island	Yes.	No.	No.	50 and 75. Hand compactor.	Mechanical compactors calibrated to hand compactor.	AADT< 1500 - 50 AADT =>1500 - 75
New Brunswick	Yes.	No.	No.	75. Hand compactor.	Lab. uses 60 blows with mechanical compactor.	N/A
Quebec	Yes.	No.	No.	60. Mechanical compactor.	N/A	N/A
Ontario	Yes.	No.	No.	75. Hand compactor.	Same	-
Manitoba	Yes.	No.	No. Preliminary binder tests only.	75. Hand compactor.	None.	N/A

	3.0 Design Properties					
	3.1 Which of these design procedures are used for design of AC mixtures?			3.2 For Marshall procedure, how many blows are specified? Mechanical or hand compactor?	Describe if lab. procedures and blows differ from specified blows.	Describe differences in blow counts due to different applications or mixes.
	Marshall (ASTM D1559)	Hveem	Superpave (SHRP)			
Saskatchewan	Yes.	No.	No.	50. Hand compactor.	Mechanical compactors calibrated to 50 blow hand compactor.	All mix designs are 97% of the 50 blow Marshall.
Alberta	Yes.	No.	No.	50 and 75. Hand compactor.	Mechanical Compactor correlated to hand, equal number of blows.	50- Community airports. - Maximum traffic load of 0.3 to 0.8 ESAL per direction over its design life.
British Columbia	Yes.	Yes.	No.	75. Mechanical compactor.	Same as specified.	N/A
Yukon	Yes.	No.	No.	50. Mechanical compactor.	-	-
Public Works Canada	Yes.	No.	No.	50 and 75. Hand compactor as per ASTM D1559.	If mechanical compactor used, it should be calibrated to 50 blows by hand.	50- Airfield 75- Roads
City of Calgary	Modified	No.	No.	50. Mechanical compactor.	N/A	N/A
City of Edmonton	Yes.	No.	No.	50 and 75. Mechanical compactors normally used.	N/A	50- Residential overlay 75- Surface, base and overlay course

	3.0 Design Properties (cont'd)								
	3.3 Design criteria.								
	Mix	Stability (kN)	Flow (mm)	Air voids (%)	VMA (%)	Voids Filled (%)	Index of retained stability	Film thickness	Other
Newfoundland	None supplied	-	- mm are specified.	-	-	-	-	-	-
Nova Scotia	Special B Type C, B	7.5 4.5	2-4 2-4mm are specified.	3-5 3-5	14 13-15	65-75 75-85	-	-	-
Prince Edward Island	Base Seal	5.8 Min. 5.8 Min.	2-4 2-4	2-4 2-4	14 15	Not specified not monitored	Not specified not monitored	Not specified not monitored	-
New Brunswick	Base Seal	6 Min. 6 Min.	2-4 2-4 0.254mm units are specified.	3-5 3-5	13-15 14-16	65-75 65-75	N/A N/A	7-10 7-10	-
Quebec	Surface Base	9 9	2-4 2-4 0.254mm units are specified.	2-5 2-5	-	8.5 8.5	-	Depends on the nature of the granular material.	-
Ontario	Medium duty binder course. Further designs enclosed.	-	2 0.254mm units are specified.	3.5-4.5	13-15	-	-	-	-
Manitoba	Conventional hot mix Sand asphalt (stabilized base course)	Not specified. Not specified.	Not specified. Not specified.	4.0-4.5 9-15	13.5-16.5 (bottom lift) 14-16 (top lift) 13.5-16.5	Not specified. Not specified.	Not specified. Not specified.	Not specified. Not specified.	-

3.0 Design Properties (cont'd)

3.3 Design criteria.

	Mix	Stability (kN)	Flow (mm)	Air voids (%)	VMA (%)	Voids Filled (%)	Index of retained stability	Film thickness	Other
Saskatchewan	150/200A 200/300A	3.3	2.0	3.5-5.5	14.0	75	75%	6	-
	300/400A 400/500A	3.3	2.0	4.0-6.0	15.0	75	75%	6	
Alberta	Class 10	3.3	2-5	2-4	15.5-17.5	-	70	-	-
	Class 12.5	4.5	2-5	3-5	15.0-17.0		70		
	Class16	6.7	2-5 mm are specified.	3-5	14.5-16.5		70		
British Columbia	Class 1 80/100	8.0	2-3.5	4-6	12	N/A	N/A	N/A	75 blows.
	Class 2 200/300 More mix designs enclosed.	6.1	2-4.1 0.254mm units are specified.	3-9	13	N/A	N/A	N/A	75 blows.
Yukon	-	4.5	2-4 mm are specified.	2-4	14.5	75-85	-	-	-
Public Works Canada	Airfield base course Airfield surface course	5-9*	2-4	3-5	13	-	75	-	-
	Roads	5-9*	2-4	3-5	15		75		
		5.5 *Depends on design aircraft.	2-4 mm are specified.	3-5	15		75		
City of Calgary	1	5.4	2.25-3.75	3-6	12	-	-	6	-
	2	7.1	2.5-4 0.254mm units are specified.	2-5	14			7	
City of Edmonton	Surface	6.7	1.5-3.0	3.8-4.2	-	65-78	75	7.0	20
	Base	6.7	1.5-3.0	3.6-4.4		67-78	75	6.0	25
	Residential	4.5	2.0-4.0	2.0-4.0		73-85	75	7.0	12.5
	Overlay	6.7	1.5-3.0 mm are specified.	3.6-4.4		68-80	75	7.0	12.5 Max. agg. size (mm)

	3.0 Design Properties (cont'd)		4.0 Quality Control				
	3.4 Are trial batches prepared using AC from the proposed source, for mix designs?	3.5 Describe how mixing and compaction temperatures are selected for trial batches.	4.1 Does your department regularly check the properties of the AC delivered to your projects?	Penetration ASTM D5	Do you conduct tests regularly at temps. other than 25°C?	What temperature, loading time, and weight?	Specific gravity ASTM D70
				Which of these tests are used?			
Newfoundland	Yes.	From temperature-viscosity charts supplied by refineries, in accordance with Asphalt Institiute MS-2 procedure.	Yes.	Yes.	Yes.	4°C 5 sec 100gm	Yes.
Nova Scotia	Yes.	ASTM D1559	Yes.	Yes.	Yes.	-	No.
Prince Edward Island	Yes.	ASTM Procedures	Yes.	Yes	No		Yes
New Brunswick	Yes.	From temperature-viscosity charts supplied by refinery.	Yes.	Yes.	No.	-	Yes.
Quebec	-	-	Yes.	Yes.	No.	-	Yes.
Ontario	Yes. Contractor design mix.	From temperature-viscosity charts supplied by refinery.	Yes.	Yes.	No.	-	No.
Manitoba	Yes.	Viscosity testing at two temperatures.	Yes.	Yes.	No.	4°C 60 or 5 sec 100 gm	Yes.

	3.0 Design Properties (cont'd)		4.0 Quality Control				
	3.4 Are trial batches prepared using AC from the proposed source, for mix designs?	3.5 Describe how mixing and compaction temperatures are selected for trial batches.	4.1 Does your department regularly check the properties of the AC delivered to your projects?	Penetration ASTM D5	Do you conduct tests regularly at temps. other than 25°C?	What temperature, loading time, and weight?	Specific gravity ASTM D70
				Which of these tests are used?			
Saskatchewan	Yes.	Specified for each AC grade. Temperature-viscosity curves are not normally used.	Yes.	Yes.	No.	-	Yes.
Alberta	Yes.	Viscosity @ 60C and 135C for all design mixes.	Yes.	Yes.	Yes. Annually.	5°C 5 sec 100 gm	Yes.
British Columbia	Yes.	Strictly by viscosity @ 60C	Yes.	Yes.	No.	25°C 5 sec 100 gm	Yes.
Yukon	Yes.	From temperature-viscosity charts supplied by refinery.	Yes.	Yes.	No.		Yes.
Public Works Canada	Yes.	From temperature-viscosity charts supplied by refinery.	No. AC is tested for acceptence and sampled regularly throughout the job for quality assurance purposes.	-	-	-	-
City of Calgary	N/A	N/A	Yes.	Yes.	Yes.	0°C 60 sec 200 gm	No.
City of Edmonton	Yes.	From temperature-viscosity charts supplied by refinery.	Yes.	Yes.	No.	-	Yes.

	4.0 Quality Control (cont'd)								
	Kinematic viscosity ASTM D2170 @ 135°C	Kinematic viscosity ASTM D2170 @ 60°C	Absolute viscosity ASTM D2171 @ 60°C	Flash point ASTM D92	Solubility in CCL4 ASTM D2042	Ash ASTM D482	Ductility ASTM D113	Softening point ASTM E28, D36	Retained penetration after thin film oven test
	Which of these tests are used?								
Newfoundland	Yes.	No.	No.	Yes.	Yes.	No.	Yes.	No.	Yes.
Nova Scotia	Yes.	No.	No.	No.	No.	No.	No.	No.	No.
Prince Edward Island	Yes	No	No	Yes	Yes	No	Yes	No	Yes
New Brunswick	Yes.	No.	No.	Yes.	No.	No.	No.	No.	No.
Quebec	Yes.	No.	No.	Yes.	Yes.	No.	Yes.	Yes. D36.	Yes.
Ontario	Yes.	No.	No.	Yes.	No.	No.	Yes.	No.	No.
Manitoba	Yes.	Yes.	Yes.	Yes.	Yes.	Yes.	No.	No.	Yes.

	4.0 Quality Control (cont'd)								
	Kinematic viscosity ASTM D2170 @ 135°C	Kinematic viscosity ASTM D2170 @ 60°C	Absolute viscosity ASTM D2171 @ 60°C	Flash point ASTM D92	Solubility in CCL4 ASTM D2042	Ash ASTM D482	Ductility ASTM D113	Softening point ASTM E28, D36	Retained penetration after thin film oven test
	Which of these tests are used?								
Saskatchewan	No.	No.	Yes.	Yes.	Yes.	No.	Yes.	No.	Yes.
Alberta	Yes.	No.	Yes.	Yes.	Yes.	No.	No.	No.	No.
British Columbia	Yes.	Yes.	Yes.	Yes.	No.	No.	Yes. Where adding polymer or other additives.	Yes. On AC with high penetration.	Yes.
Yukon	No.	Yes.	Yes.	Yes.	Yes.	No.	No.	No.	No.
Public Works Canada	-	-	-	-	-	-	-	-	-
City of Calgary	Yes.	Yes.	No.	Yes.	Yes.	No.	Yes.	No.	No.
City of Edmonton	Yes.	No.	No.	Yes.	Yes.	No.	No.	No.	No.

	4.0 Quality Control (cont'd)				
	Retained penetration after rolling thin film oven test	Retained penetration after plant mixing Using Abson).	Absolute viscosity ASTM D2171 after thin film oven test.	Do any of these tests apply to Polymer Modified Asphalts?	4.3 Has there been difficulties between you and a producer with regards to precision of testing?
	Which of these tests are used?				
Newfoundland	No.	No.	No.	PMA or engineered bituman requires testing in accordance with SHRP binder spec.	No.
Nova Scotia	No.	No.	No.	N/A	No.
Prince Edward Island	No	Yes (Using Rotavapor)	Yes	PMA's not used to date	No
New Brunswick	No.	Yes.	No.	PMA used only in research project to date.	No.
Quebec	Yes.	No.	No.	Additional testing: Elastic recovery @10°C Force ductility @4°C Stability @163°C,48 h	No. There is a contractual procedure for solving specific problems.
Ontario	No.	No.	No.	Specifications have been laid out for the testing of PMA.	No.
Manitoba	No.	No.	Yes.	PMA used only once on test section. Tests used: specific gravity, absolute viscosity @ 60°C, Kinimatic viscosity @ 135°C, penetration @ 25°C both before and after thin film oven test.	No. Not generally.

	4.0 Quality Control (cont'd)				
	Retained penetration after rolling thin film oven test	Retained penetration after plant mixing Using Abson).	Absolute viscosity ASTM D2171 after thin film oven test.	Do any of these tests apply to Polymer Modified Asphalts?	4.3 Has there been difficulties between you and a producer with regards to precision of testing?
	Which of these tests are used?				
Saskatchewan	No.	Yes.	Yes.	PMA not used. Some polymer modified sealing grade emulsions used. Tested using peneration, absolute viscosity and elastic recovery.	No. Some difficulties with emulsified asphalt and cutback asphalt products. High quality products are generally consistent.
Alberta	No.	No.	Yes.	PMA are tested with the same testing methods.	No.
British Columbia	No.	Yes. 2 of 6 labs.	Yes.	Ductility test done on some PMA, only rarely with 80/100 and 150/200.	No.
Yukon	No.	No.	No.	Not used.	No. Very little AC is used by this department.
Public Works Canada	-	-	-	N/A	No. AC is tested for acceptance prior to mix production. Marginal properties are addressed then.
City of Calgary	No.	No.	No.	PMA is currently specified by supplier (Husky). Other modified ACs are used on trial basis using specifications provided by the supplier.	No.
City of Edmonton	No.	No.	No.	No.	No.

	4.0 Quality Control (cont'd)							
	4.4 Is your department experiencing problems with asphalt cement related to the following items?						4.5 Of the problems noted in 4.4 which should be considered top priority to find solutions	4.6 Has your agency conducted any studies to correlate AC properties with pavement performance?
	Thermal cracking	Permanent deformation	Fatigue cracking.	Moisture sensitivity of ACs	Stripping	Aging of ACs		
Newfoundland	Yes.	No.	Yes.	Yes.	Yes.	No.	Thermal cracking.	No.
Nova Scotia	No.	Yes.	Yes.	Yes.	Yes.	Yes.	Stripping and permanent deformation.	No.
Prince Edward Island	Yes	Yes	No	No	Yes	No	Permanent Deformation	No
New Brunswick	Yes.	Yes.	Yes.	Yes.	Yes.	Yes.	Thermal cracking and stripping.	No.
Quebec	Yes.	Yes.	Yes.	Yes.	Yes.	No.	Thermal cracking, fatigue cracking, moisture sensitivity and stripping.	Yes.
Ontario	Yes.	Yes.	No.	No.	Yes.	Yes.	Thermal cracking and permanent deformation.	Yes.
Manitoba	Yes.	Yes.	No.	No.	No.	No.	Thermal cracking and permanent deformation.	No. Study not done, but are aware of effects of penetration and viscosity on performance.

	4.0 Quality Control (cont'd)							
	4.4 Is your department experiencing problems with asphalt cement related to the following items?						4.5 Of the problems noted in 4.4 which should be considered top priority to find solutions	4.6 Has your agency conducted any studies to correlate AC properties with pavement performance?
	Thermal cracking	Permanent deformation	Fatigue cracking.	Moisture sensitivity of ACs	Stripping	Aging of ACs		
Saskatchewan	Yes.	No.	Yes.	Yes.	Yes.	?	Stripping and moisture sensitivity. Fatigue cracking. Thermal cracking.	Yes.
Alberta	Yes.	No.	Yes Somewhat	No.	No.	Perhaps	Aging	Yes.
British Columbia	Yes.	Yes.	Yes.	No.	No.	No.	Thermal cracking and permanent deformation.	Yes.
Yukon	Yes.	No.	Yes.	No.	No.	Yes.	Thermal and fatigue cracking.	No.
Public Works Canada	Yes.	No.	Yes.	Yes.	Yes.	Yes.	Thermal cracking and stripping.	Yes.
City of Calgary	No.	Yes.	No.	No.	No.	Yes.	Rutting. The other prolems listed are inherent to the use of AC.	Yes.
City of Edmonton	Yes.	Yes.	No.	No.	No.	No.	Permanent deformation.	No.

	5.0 Other						
	5.1 Have studies been conducted to predict cracking rates due to low temperature influences?	5.2 Could you supply data for a study into low temperature cracking and AC type?	5.3 When recycling does your agency use rejuvenating agents? What additives?	How is the amount and type of agent determined?	What is the relative usage when cold-recycling? When hot-recycling?	5.4 When recycling, does your agency add asphalt?	What grade and how much is added?
Newfoundland	No.	No.	No.	-	-	-	-
Nova Scotia	No.	No.	No. Investigating at present.	-	-	Yes.	Cold recycling-emulsified asphalt.
Prince Edward Island	No	No	No			N/A	
New Brunswick	No.	No.	No.	-	-	Yes.	Depends on percentage of liquid contained in original AC, and recovered penetration of AC from cores.
Quebec	Yes.	Yes.	No.	-	100% cold recycling	Yes.	80/100 pen-grade used based on Marshall mix design, with a maximum RAP material of 20% on surface coarse and 40% on base coarse.
Ontario	Yes. Engineering Materials Report EM-88	Yes. Engineering Materials Report EM-89	Yes. Proprietary	At mix design stage (by contractor)	Minimal. cold recycling Substantial H.I.P.R.	Yes.	Grade is based on the recycling ratio and recovered pen. of the RAP to produce a pavement of equal recovered pen. as a conventional mix.
Manitoba	Yes. Ste. Anne Test Road	Yes. Includes reflective cracking.	N/A.	-	-	Yes. Department uses only Hot recycled plant mix.	Grade of AC depends on penetration and percent of the milled material to be used. Amount of AC used depends on AC content of milled material and on Marshall mix design results.

	5.0 Other						
	5.1 Have studies been conducted to predict cracking rates due to low temperature influences?	5.2 Could you supply data for a study into low temperature cracking and AC type?	5.3 When recycling does your agency use rejuvenating agents? What additives?	How is the amount and type of agent determined?	What is the relative usage when cold-recycling? When hot-recycling?	5.4 When recycling, does your agency add asphalt?	What grade and how much is added?
Saskatchewan	No.	Yes. Possibly.	Yes. High float recycling grade emulsions (HF100R, HF700R). A Grade AC.	Recycling ratio based on desired softening and penetration.		Yes.	Depends on quality, characteristics and AC content of the reclaimed. Different grades used for different amount of reclaimed AC.
Alberta	Yes.	Yes.	Yes. Cyclogen L by Witco Canada Inc. used for hot-in-place recycling.	By using a nomogragh supplied by the manufacturer. Low Marshall air voids often limits the design.	0% cold recycling less than 1% H.I.P.R.	Yes.	Depends upon the target asphalt grade and Rheology of the existing pavement.
British Columbia	No.	Yes.	Yes. Cyclogen L by Witco Canada Inc. used for hot-in-place recycling.	Recover binder, determine pen. 25°C determine Cyclogen to bring back original pen.	15% cold recycling 85% H.I.P.R.	Yes.	Depends on amount and quality of the AC recovered.
Yukon	No.	No.	Yes. As directed by consultant.	As directed by consultant.	100 % cold recycling	Yes.	As directed by consultant.
Public Works Canada	Yes.	Yes.	Yes. Various types being experimented.	As per mix design (or trial mix).		Yes.	High Pen. grade 200/300 or higher. Amount added is dependent on the RAP.
City of Calgary	No.	No.	N/A. Additives are used at the discretion of the contractor.	Additives are used at the discretion of the contractor.		-	Additional AC used at the discretion of the recycling contractor.
City of Edmonton	No.	Yes.	Yes. Cyclogen L by Witco Canada Inc. used for hot-in-place recycling.	By consultant and reviewed in field for adjustments.	60% cold recycling 40% H.I.P.R.	Yes. Used coated admixtures	150/200 about 2.5-3 % by weight.

APPENDIX B

CALCULATION OF P.I. POINTS

Final Report

Calculation of PI Values for Node Points of the Figure 2 CAN/CGSB-16.3-M90

Prepared for

EBA ENGINEERING CONSULTANTS LTD.
#2 -1420 HUNTER COURT
KELOWNA, B.C.
V1X 6E6

by

Baoqin Bai and
Kenneth O Anderson, P.Eng.

Department of Civil Engineering
University of Alberta, Edmonton T6G 2G7
Telephone: (403) 492-5112 Fax:(403) 492-0249
and
10550-26 Avenue
Edmonton, Alberta T6J 4B9
Telephone/Fax: (403) 436-1517

November 1994

K.O. ANDERSON, P.ENG.
Civil Engineer

Department of Civil Engineering
University of Alberta, Edmonton, AB T6G 2G7
Telephone: (403) 492-5112 Fax: (403) 492-0249

10550-26 Avenue (Home Office)
Edmonton, Alberta, Canada T6J 4B9
Telephone/Fax: (403) 436-1517

November 24, 1994

Your File No. 104-9754

Mr. Cece Dawley, P.Eng.
EBA ENGINEERING CONSULTANTS LTD.
#2- 1420 HUNTER COURT
KELOWNA, B.C.
V1X 6E6

Telephone: (604) 862-4832
Fax: (604) 862-2941

RE: Calculation of PI values for Node Points of the Figure 2 CAN/CGSB-16.3-M90.

As discussed in Regina on November 16th, 1994 and subsequent contact with Mr. Bert Pulles, P.Eng. in he Edmonton office, we have undertake the estimation of PI values according to the general procedures described in our earlier report entitled "Calculation of PI Values for Node Points of the Figure 1 CAN/CGSB-16.3-M90." Final Report prepared for EBA ENGINEERING CONSULTANTS LTD., March 1994.

The procedures were modified to use the viscosity at 135 C, namely:

1. In order to use SHELL BANDS to calculate PI, two penetration values or one penetration and a R&B softening point are needed. This second point was approximated by the T_{800} , i.e. temperature for a penetration of 800 dmm.

2. The viscosity at 135 C in Pa.s.was transformed to a "C" term in order to plot this value together with Pen @ 25 C on the BTDC. We then calculated a T_{800}, and PI. This PI was calculated using both BANDS. This was done for the Node Point values for the borderlines of the Group A & B, and B & C lines and the Group C lower limit.

   Separate BTDCs have been plotted for the set of points for each line and shown in Figures 1, 2 and 3.

3. These PI values were then plotted on our computerized version of Figure 2 CAN/CGSB-16.3-M90, shown as Figure 4. This has provided a series of PI values that would enable lines of equal PI values to be plotted by hand, in a manner similar to Figure 26 in Deme and Young (CTAA Proc. 1987).

The following Report contains the Figures prepared by Baoqin Bai, who performed the necessary calculations under my direction. A time sheet for his services is enclosed.

I trust these calculations and figures will be sufficient for your purposes.. If there is any further information needed please contact me at your convenience.

Yours sincerely,

K.O. Anderson, P.Eng.
Professor Emeritus

enclosure

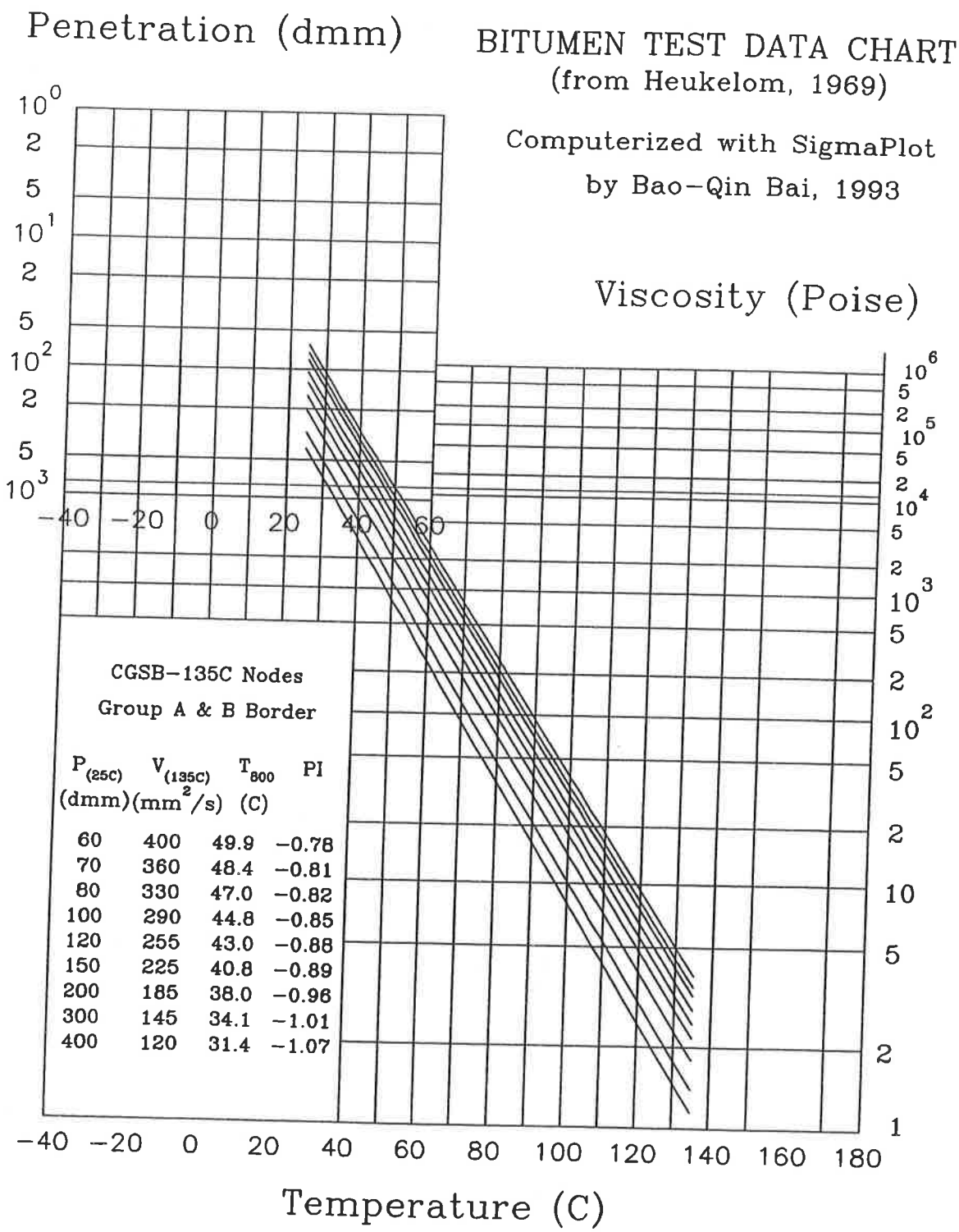


Fig.1—Group A and B Borderline Nodes from CGSB Figure 2

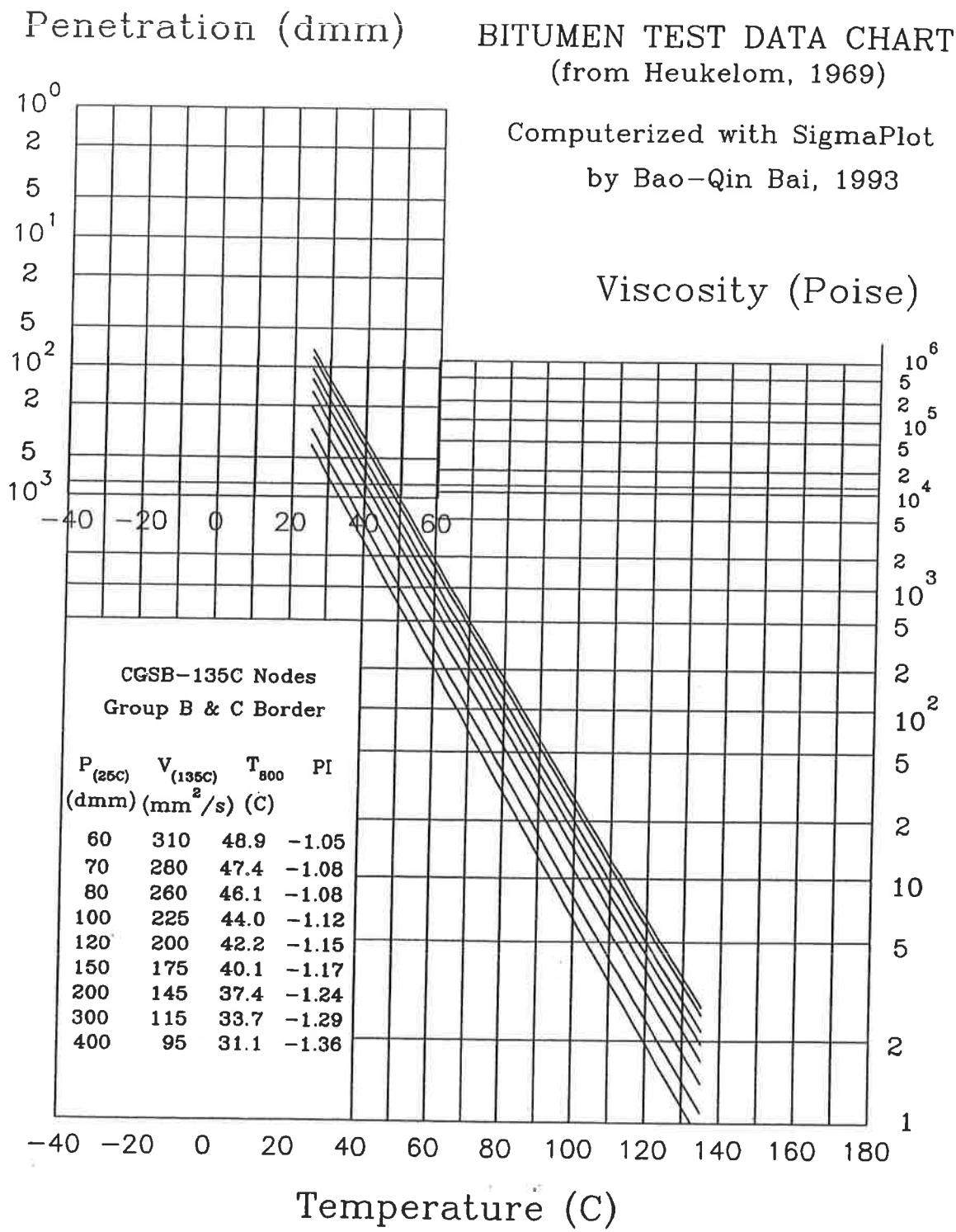


Fig.2–Group B and C Borderline Nodes from CGSB Figure 2

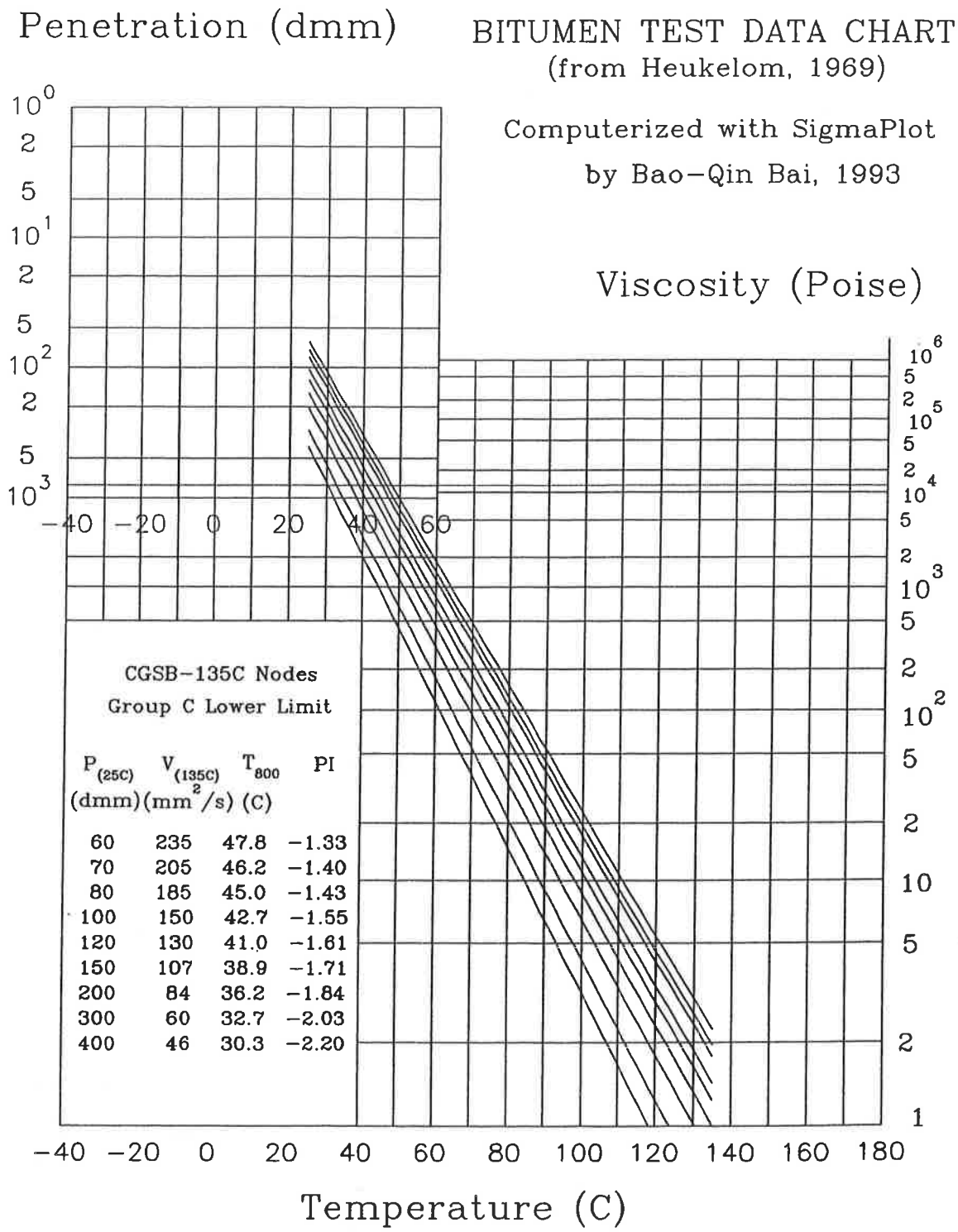


Fig.3–Group C Lower Limit Borderline Nodes from CGSB Figure 2

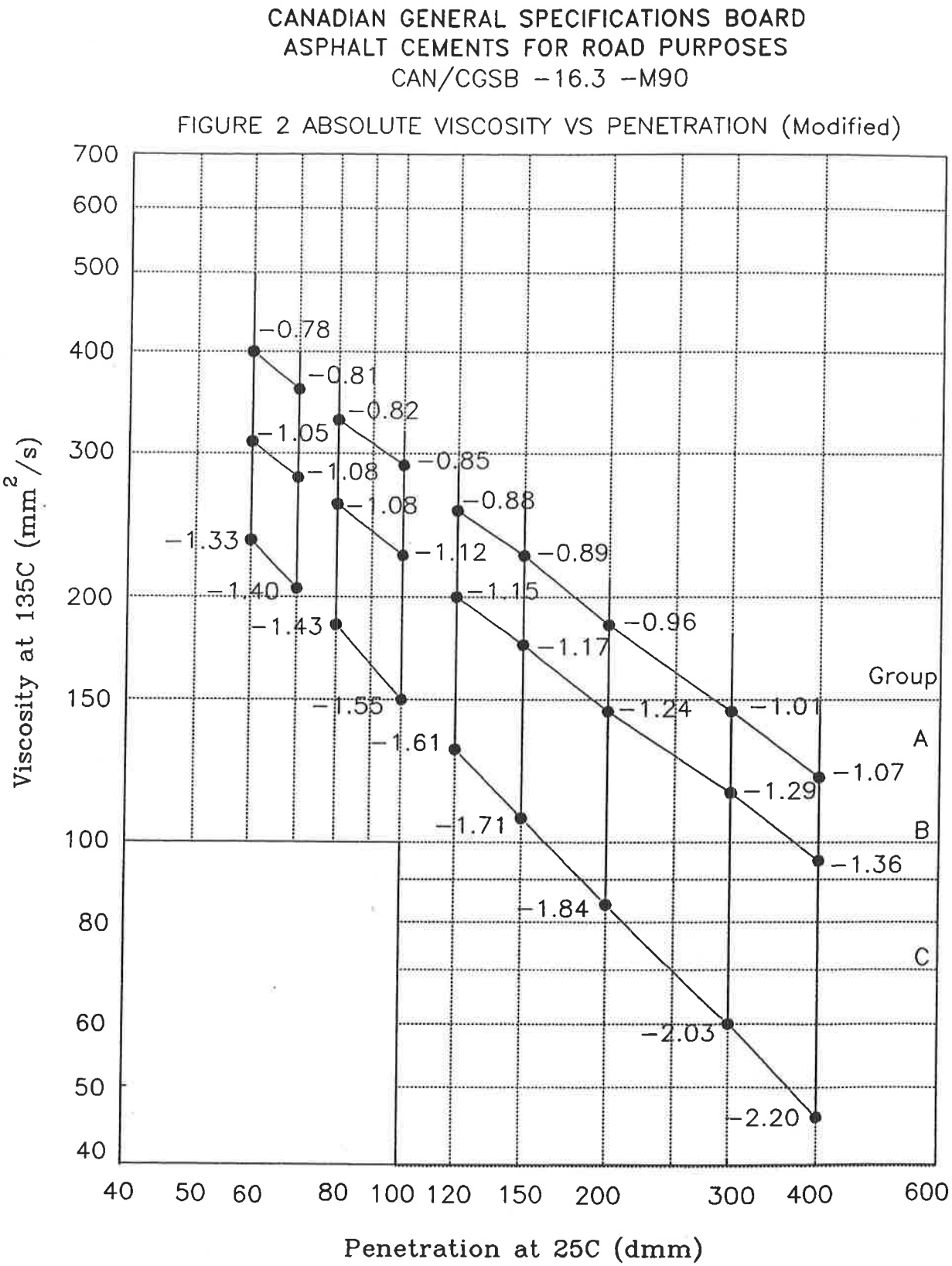


Fig.4—Penetration Indexes for the Borderline Nodes

APPENDIX C

DEVELOPMENT OF CAN/CGSB-16.3-M90

APPENDIX C

Development of CAN/CGSB-16.3-M90

Synopsis

.1 Preamble

The current (1990) Standard is the product of ten years of effort by leading Canadian experts in asphalt technology who comprised the Committee on Road Materials of CGSB. A brief synopsis of significant events which occurred over this time frame is provided below, as it relates to derivation of the contents of the current specification, the historic status of a User Guide, and the relevance of this information to the current study. Copies of Committee Meeting Minutes, personal correspondence and communications between the Secretary of CGSB and Committee members have been assembled from a number of sources. Following narrative is based on the information contained in the available records.

In January, 1979, Mr. F.D. Young, Manitoba Department of Highways communicated by letter to Mr. G.A. Berdahl (Alberta Transportation) and presumably others, for the purpose of organizing a task force "for the purpose of determining whether a specification for asphalt cements could be developed which would be more widely accepted than the present 16-GP-3M". Mr. Young had been previously requested by the CGSB Committee on Road Materials to head up this task force. In his letter to Mr. Berdahl, Mr. Young defined two objectives to be considered, ie:

1. The specification should meet "end use or performance requirements".
2. "The quality we are after seems to me to be stiffness of the asphalt cement at low and at high temperatures".

The CGSB specification in existence in 1979 was issued in July, 1977. Asphalt cements were graded by penetration at 25°C (100 g, 5 sec), with other specified requirements being ductility and penetration of residue after thin-film oven test. This edition of the specification actually contained what may be considered to be the first User Guide. Section 7.3 (Principal Uses of Asphalt Cements) was a one paragraph guide with an accompanying Table which identified appropriate grades of asphalt cement to be considered in various construction applications.

.2 Draft, October, 1979

Correspondence exists (Alberta Transportation memorandum) to indicate that a draft specification dated October, 1979 was presented by Mr. Young at a CGSB Committee meeting in November, 1981. A copy of this draft has not been located, but the above-referenced

memorandum refers to the concept of "three types of asphalt, ie. A, B and C". Conflict of opinion among Committee members, as to the acceptability of three grades of asphalt, appears to have existed at this time.

.3 Draft, August, 1982

In August, 1982 Mr. Young submitted to the CGSB Committee Secretary the "August 1982 draft". In his correspondence Mr. Young noted "one serious problem, perhaps without resolution, regarding provision of viscosity limits in both absolute and kinematic units". In the same submission Mr. Young referenced a July, 1981 draft User Guide. He also noted that sub-committee members agreed that a guide is "worthwhile if not essential". The August, 1982 draft specification referenced asphalt grade (by penetration at 25°C) and by "type" (A, B and C), for which requirements were provided in terms of viscosity at both 60°C and 135°C. This draft continued to include Section 7.3 (Principal Uses of Asphalt Cements). However it was supplemented by a new sub-section which described "grade" of asphalt and selection criteria which were dependent upon traffic, climatic conditions and pavement thickness. Preliminary minutes of the CGSB committee meeting held 25 November, 1982 state that two positions existed within the Committee in respect to the "grade classification present in the draft version of the standard" and that "discussion on this subject continued without resolution".

.4 Draft, September, 1983

In September, 1983 Mr. Young submitted to the CGSB Committee Secretary a "September, 1983 draft specification" and a "supplement on asphalt selection". In his covering letter, Mr. Young addressed the aforementioned Committee member positions ("supplier - user differences"). This draft differs from the August, 1992 draft in that direct reference to Types A, B and C asphalts (based on either viscosity at 60° or 135°C) is deleted, and replaced with detailed requirements which read "The asphalt cement shall comply with the requirements prescribed in Tables I, II or III according to penetration grade". These Tables specified viscosity requirements only at 60°C. Section 7.3 (Principal Uses of Asphalt Cements) remained intact. The Supplement (Selection of Asphalt Cement) is consistent with the concept of a User Guide.

.5 Draft, Dated August, 1984

The next draft specification, dated August, 1984, was a topic of discussion by the CGSB Committee in November, 1984. Again, significant change had occurred in the classification of asphalt cements. Section 3 (Classification) specified asphalts by penetration grade <u>and</u> by viscosity grade (at 60°C), with temperature susceptibility parameters designated within Groups I and II in each case. Requirements for penetration grade asphalts were specified in Table I, and

included kinematic viscosity properties at 135°C for Groups I and II asphalts. Requirements for viscosity graded asphalts were specified in Table II with penetration values being the variable which described Group I and II asphalts. Section 7.3, which previously addressed principal uses of asphalt cements, did not exist in the August, 1984 draft. In its place, Appendix A (Guide to Selection of Asphalt Cement) was included. The content of this Guide included narrative which addressed:

i) Temperature susceptibility
ii) Maximizing fatigue resistance
iii) Maximizing rutting resistance
iv) Maximizing resistance to low temperature cracking

Introductory paragraphs in the Guide advised the user that:

- selection of asphalt cement depends on factors which include traffic, pavement design and thickness, aggregate characteristics and service temperature range.
- selection of the asphalt cement will be a compromise between materials which perform best with respect to fatigue cracking, rutting and thermal cracking.
- factors other than the asphalt cement influence pavement performance in respect to fatigue and rutting.

In summation, this Appendix formed the basis of an excellent User Guide. However, as Mr. J.L.M. Scott (Saskatchewan Highways and Transportation) pointed out in a letter to the Committee Secretary (September 21, 1984), the Guide should contain recommendations "as to appropriate stiffness values to resist cold temperature cracking and high temperature rutting". Significant decisions were made by the Committee, in respect to this draft, at the November, 1994 meeting, including:

i) Inclusion of the User Guide as an Appendix in the standard was not appropriate and that a separate publication covering the guide should be developed.
ii) Viscosity grading of asphalt cement should be deleted from the draft.

It was ultimately agreed that the existing Task Force should develop a next draft standard and simultaneously produce a separate publication covering a User Guide.

.6 Third Draft, January, 1985

In February, 1985 the Committee Secretary circulated what was now called "Third Draft", dated January, 1985. Format and content changes included: -

- New Article 3. (Terminology);

  3.1 Temperature Susceptibility - The paving asphalt temperature susceptibility is the change in consistency (penetration or viscosity) that a paving asphalt undergoes for a given change in temperature.
- Article 4.0 (Classification);

Asphalt cements were defined firstly by penetration grade and secondly by temperature susceptibility in terms of viscosity at 135°C, i.e.:-

Group A - high viscosity
Group B - medium viscosity
Group C - Low viscosity
(all for a given penetration at 25°C)

- Requirements for each asphalt grade were specified in Table I and accompanying Figure A (which graphically presented viscosity values for each penetration grade and group - A, B, C).

The new draft standard for the User's Guide was not completed at February, 1985.

.7 Fourth Draft, May 1985

This draft followed promptly after the Third Draft. The only significant change between the two documents was in respect to Figure A, which was revised to contain tabular values for kinematic viscosity (Groups A, B, C) vs. penetration for six specified grades.

Appendix A (Guide to Selection of Asphalt Cement) was once again included and was based on the document included in the August, 1994 draft. Narrative within the original subject areas was expanded upon. Of greatest significance in this regard is introduction of PI and PVN methodologies to characterize temperature susceptibility, including two nomographs, ie:

Figure 1 - Minimum Pen-Vis Number to Avoid Low Temperature Cracking
Figure 2 - Minimum Penetration Index to Avoid Low Temperature Cracking

.8 Fifth Draft, November, 1986

Differences between the Fourth and Fifth draft specification were minimal. Figure A (Fourth Draft) had been renamed Figure 1 and was reformatted and "tidied up". Labelling of temperature susceptibilities for each Group (A, B and C) was deleted.

Appendix A, as it appeared in the Fourth Draft, was substantially abbreviated. However, the two methods for calculating temperature susceptibility, PVN and PI, were included as Appendices A1 and B1 respectively. It should be noted that the method outlined in Appendix A1 considered stiffness at both service temperature extremes (high and low). Appendix B1 addressed only low temperature service.

Minutes of the CGSB Committee meeting of 19 November 1987 addressed the status of the Fifth Draft. The Secretary reported that consensus of the committee members had been reached on the technical requirements of the standard but that most of the concerns raised were related

to the length and complexity of the Appendix for the User Guide. Following motions were passed by the Committee:

i) "that the committee adopt the present Fifth Draft with Figure 2 amended to include viscosity at 60°C and penetration at 25°C"
ii) "that the appendix as balloted in the fifth draft be deleted"
iii) "Mr. Deme to prepare the revised sixth draft"

.9 Sixth Draft, May, 1988

The sixth draft was submitted to the Committee Secretary by Mr. Deme, together with a report detailing the basis for its development, under covering letter dated May 17, 1988. Some significant changes which were made to the draft standard were:

i) Article 4.1.1 - pen grade 85-100 was revised to 80 - 100
ii) Article 4.1.1.1 (temperature Susceptibility) - All references to viscosity had been revised from 135°C to 60°C
iii) Article 6 (Detail Requirements)
   a) Table 1 - Viscosity requirements were revised to require use of either Figure 1 (Viscosity at 60°C) or Figure 2 (Viscosity at 135°C), and a notation that "user must specify either Figure 1 or Figure 2 (it is inappropriate to use both Figures simultaneously".)
   b) Figure 1 contains graphical and tabular data for Groups A, B and C asphalt cements as a function of absolute viscosity (60°C, Pa.s) versus penetration (25°C, 100 g, 5s).
   c) Figure 2 contains graphical and tabular data for Groups A, B and C asphalt cements as a function of kinematic viscosity (135°C, mm^2/s versus penetration (25°C, 100 g, 5s). Note: Most tabulated minimum kinematic viscosity values have been reduced from the Fifth Draft.

The report submitted by Mr. Deme with the Sixth Draft is pertinent. It is referenced in detail within Section 6.2 (Measuring Temperature Susceptibility by PI and PVN). It should be noted however, that Deme explained the method used to create an optional specification, based on viscosity at 135°C and penetration at 25°C. He did this by placing "most of the asphalts in Appendix 1 (of his report) in the same temperature susceptibility group as they occupy in Figure 1 (of his report).

The Sixth Draft was adopted at a Committee meeting on 17 November, 1988 after adoption of the following revisions:

i) 80/85 - 100 penetration grade in Table 2 changed to 80 - 100 en.
ii) Inclusion of a sentence in the Standard to provide the option of using either Figure 1 <u>or</u> Figure 2, but not both.

iii) Modification of Figures 1 and 2 to provide the minimum viscosity - penetration limits for Group C asphalts to accommodate inclusion of all asphalt used in British Columbia.

The Standard was published in 1990, after incorporation of the above revisions. The Standard did not include a User Guide of any form.

.10 Summary

The draft User Guide, which appeared as Appendix A in the August, 1984, forms a good starting point for developing a comprehensive, current User Guide. Reference was made earlier to Mr. Scott's recommendation to incorporate appropriate stiffness values to resist both low temperature cracking and high temperature rutting. Guidelines for selection of asphalt cement, for both low and high temperature conditions, have been included in the User Guide, which formed the primary objective of the current study.

APPENDIX D

TAC USER GUIDE

TAC USER GUIDE

submitted to:

THE TRANSPORTATION ASSOCIATION OF CANADA

Submitted by:

EBA ENGINEERING CONSULTANTS LTD.
EDMONTON, ALBERTA

December, 1995

TABLE OF CONTENTS

1.0 INTRODUCTION 2
- 1.1 THE PURPOSE OF THIS GUIDE 2
- 1.2 THE CONCEPT FOR SELECTION OF THE ASPHALT CEMENT 2
- 1.3 CONTENTS OF THE USER GUIDE 3

2.0 CAN/CGSB-16.3-M90 5

3.0 PAVEMENT DISTRESS TYPES AND THEIR MITIGATION 7
- 3.1 BACKGROUND 7
- 3.2 LOW TEMPERATURE CRACKING 7
- 3.3 PERMANENT DEFORMATION 9
- 3.4 FATIGUE 10
- 3.5 MOISTURE SENSITIVITY AND STRIPPING 11
- 3.6 AGING OF THE ASPHALT/AGGREGATE SYSTEM 11
- 3.7 DURABILITY 11

4.0 SELECTION OF DESIGN PAVEMENT TEMPERATURES 13
- 4.1 INTRODUCTION 13
- 4.2 SOURCE OF CANADIAN TEMPERATURE DATA 14
- 4.3 SELECTION OF THE DESIGN LOW PAVEMENT TEMPERATURE 15
- 4.4 SELECTION OF THE DESIGN HIGH PAVEMENT TEMPERATURE 16

5.0 TRAFFIC 18
- 5.1 CONSIDERATION OF TRAFFIC AND MAXIMUM DESIGN TEMPERATURE 18
- 5.2 CONSIDERATION OF TRAFFIC AND MINIMUM DESIGN TEMPERATURE 19

6.0 OTHER CONSIDERATIONS 20
- 6.1 SAFETY FACTORS 20
- 6.2 AGGREGATE SELECTION FOR PAVING MIXTURES 20
- 6.3 PERFORMANCE TESTING OF PAVING MIXTURES 21
- 6.4 LIMITATIONS 21

7.0 THE ASPHALT CEMENT SELECTION PROCEDURE 22
- 7.1 BACKGROUND 22
- 7.2 STEP-BY-STEP PROCEDURE 22
- 7.3 WORKED EXAMPLES 27

TABLES
FIGURES

1.0 INTRODUCTION

1.1 THE PURPOSE OF THIS GUIDE

A National Standard of Canada for Asphalt Cements for Road Purposes has been developed as CAN/CGSB-16.3-M90 by the Canadian General Standards Board (CGSB). The current standard is the product of several years of development by a group of leading Canadian experts in asphalt paving technology. The Transportation Association of Canada (TAC) recognized that utilization of the current Standard may be enhanced through development of a User Guide intended to describe how the asphalt cement selection procedure should be conducted.

Selection of the grade and group of asphalt cement for an asphalt concrete pavement, on a project-specific basis, from the alternative products that are available in the Standard, is appropriately dependent upon several project related factors. The purpose of this Guide is to provide the user with a logical step-by-step procedure to enable the most appropriate grade and group of asphalt cement to be chosen in consideration of these factors. Methodology for identifying and addressing these factors in the asphalt cement selection process is provided in following Sections. The methodology specifically addresses selection of conventional asphalt cements for new pavements. Asphalt cement selection for overlay projects should be undertaken in recognition of pre-existing pavement conditions and distress features.

1.2 THE CONCEPT FOR SELECTION OF THE ASPHALT CEMENT

Factors which are most significant in the selection of the type and grade of asphalt cement for a project include:

- traffic volume and loading
- service temperature range
- thickness and design of the pavement structure
- characteristics of the available aggregates

The asphalt cement that is ultimately selected for a project will usually be a compromise between those candidate materials which provide the best performance with respect to primary pavement distress modes, and are most commonly considered to be fatigue cracking, instability rutting and cracking due to thermal stresses. Some types of pavement distress, such as cracking due to thermal stresses, are predominantly associated with the properties of the asphalt cement.

On the other hand, other types of pavement distress, such as instability rutting, are predominantly associated with aggregate properties, and the properties of the asphalt cement are considered to be contributory.

The concept associated with use of this Guide is to enable the most appropriate asphalt cement as provided for within CAN/CGSB-16.3-M90 to be selected, in consideration of the relative significance of the factors that influence asphalt concrete pavement performance under prevailing climatic, traffic and other service conditions.

1.3 CONTENTS OF THE USER GUIDE

In following Sections of this Guide, a brief description is provided of the mechanisms that cause distress to occur in asphalt concrete pavements, and of the role played by the asphalt binder in mitigating their occurrence. An explanation is provided of the CGSB specification, and of the manner in which the rheological properties of asphalt cements have been used to create a matrix of products for use over the range of climatic conditions which prevail throughout Canada. A step-by-step procedure is presented for selecting the most appropriate asphalt cement grade and group for those anticipated project specific climatic and traffic conditions that the user must identify. Examples are provided of how the methodology serves to define an asphalt cement for specific service conditions.

This Guide is not intended to provide either direction or specifications relative to other components of an all-encompassing project of constructing an asphalt concrete pavement, including:

- pavement structure design
- paving mixture design
- aggregate selection and processing
- construction quality control and quality assurance
- construction specifications

Under certain climatic and service conditions, it is possible that asphalt cements specified in the CGSB Standard will not comply with certain requirements that are associated with prevailing traffic and/or climatic conditions. In such instances, consideration may have to be given to selection of alternative products such as polymer modified asphalt. The CGSB Standard does

not contain specifications for any types of enhanced binders, and it is beyond the scope of this Guide to provide guidelines for their selection.

TAC has published a Report "Characteristics and Performance of Asphalts Used in Canada" (The TAC Report). That Report provides substantial detail to support the development of this Guide. It references much of the research work that has been completed in Canada and elsewhere. It contains discussion of developments under the Strategic Highway Research Program (SHRP) and the Canadian counterpart (C-SHRP). The Guide user should reference the TAC Report for details which are beyond the scope of the Guide.

2.0 CAN/CGSB-16.3-M90

The Guide user should be thoroughly familiar with the contents of the CGSB Standard for Asphalt Cements for Road Purposes. Six penetration grades are provided for within the Standard:

60 - 70	150 - 200
80 - 100	200 - 300
120 - 150	300 - 400

In addition, three Groups of asphalt cements are defined in accordance with their temperature susceptibility properties. Temperature susceptibility is defined as the change in consistency (viscosity or penetration) that an asphalt cement undergoes for a given change in temperature. Thus the temperature susceptibility properties, by Group, are:

Group A - asphalt cements that have a high viscosity at 60°C (or 135°C), for a given penetration at 25°C (low temperature susceptibility).

Group B - asphalt cements that have a medium viscosity at 60°C (or 135°C) for a given penetration at 25°C (medium temperature susceptibility).

Group C - asphalt cements that have a low viscosity at 60°C (or 135°C) for a given penetration at 25° (high temperature susceptibility).

A matrix of eighteen candidate asphalt cements exists, as a function of penetration grade and temperature susceptibility Group.

Requirements and specifications for these asphalt cements are outlined in Table 1 of the Standard. Table 1 of the Standard contains reference to Figure 1 (viscosity at 60°C) or Figure 2 (viscosity at 135°C), as the means by which viscosity properties for Group A, B and C asphalt cements are specified. The Committee that developed this Standard agreed that the asphalt cement specification should be based on:

i) viscosity at 60°C, which would serve as a performance criterion to focus on rutting, and

ii) penetration at 25 °C, which when used in conjunction with viscosity at 60 °C would serve as a means for defining temperature susceptibility to avert or minimize low temperature pavement cracking.

The asphalt cement selection procedure contained in this Guide has been developed on the basis of temperature susceptibility criteria that were utilized to develop Figure 1 of the Standard. No further reference to Figure 2 is made herein.

The term Penetration Index (PI) was used by researchers to describe the temperature susceptibility parameter as a function of the relationship of penetration and temperature. Thus, with knowledge of penetration (100 g, 5 sec.) at two temperatures (eg. 25 °C and 10 °C), the PI of an asphalt could be determined, and hence its temperature susceptibility property could be defined. It was subsequently demonstrated that, by nomographical methods, use of $pen_{25°C}$ and $viscosity_{60°C}$ values could be used to approximate, very closely, the PI determined by the two-penetration method, for <u>most</u> conventional asphalts. Thus the main specified criteria in the Standard were developed on the basis of the $pen_{25°C}$ - $viscosity_{60°C}$ temperature susceptibility parameter.

3.0 PAVEMENT DISTRESS TYPES AND THEIR MITIGATION

3.1 BACKGROUND

The primary types of distress and related factors that may develop and influence long term performance of asphalt concrete pavements are:

- low temperature cracking
- permanent deformation (rutting)
- fatigue cracking
- moisture sensitivity and stripping
- aging of the asphalt/aggregate system
- durability

The role played by the asphalt cement in the paving mixture, as it influences pavement performance, is discussed below. For certain types of distress, such as low temperature cracking, the asphalt cement properties are dominant in respect to its occurrence. For other types of distress, such as permanent deformation, the quality of the aggregate in the paving mixture is dominant in preventing its occurrence, while the asphalt cement is a secondary or contributing factor. Nevertheless, the asphalt cement should be selected on the basis of the dominant and/or contributing factors that influence pavement performance.

3.2 LOW TEMPERATURE CRACKING

Low temperature cracking in newly constructed asphalt concrete pavements is predominantly controlled (up to 90 percent) by the properties of the asphalt cement. Other contributory factors include pavement thickness, pavement age, subgrade characteristics, the asphalt mix design and mixture production procedures. Low temperature cracking in newly constructed pavement overlays may be influenced by pre-existing transverse cracking in the original pavement.

Several researchers have developed predictive methods for estimation of the cracking temperature of pavements that contain conventional asphalt cements whose rheological (viscosity, penetration) properties are known. Methodology selected for incorporation into this Guide was developed by E.E. Readshaw (1972) and it, along with that of others, is discussed in the TAC Report.

Readshaw's procedure is based on a critical asphalt stiffness of 2×10^8 N/m² at two hours loading time. That is, the pavement cracking temperature is that temperature which exists for two hours, at which time the asphalt attains a stiffness value of 2×10^8 N/m². Cracking temperatures of pavements containing CGSB Groups A, B and C asphalt cements of the six previously defined penetration grades have been computed using Readshaw's methodology.

To mitigate low temperature cracking, an asphalt cement should be selected that has a stiffness value that is less than 2×10^8 N/m² at two hours loading time at the low design pavement temperature. The design pavement temperature selection procedure is explained in Section 4.0.

Figure 1 may be used to predict the cracking temperature of a pavement containing asphalt cement whose original penetration (100 g, 5 sec, 25°C) and viscosity (Pa.s, 60°C) are specified or known. Figure 2 may be used for this purpose when the original penetration (100 g, 5 sec) at 25°C is known or specified, and at least one additional penetration value (100 g, 5 sec) at a different temperature (eg. 10°C) is also specified or known, with which to determine the Penetration Index (PI) of the asphalt cement. It should be recognized that the CGSB Standard does not specify penetrations (100 g, 5 sec) at more than one temperature.

A procedure for determining the minimum pavement temperature for design purposes, as a function of coldest air temperature, is described in Section 4.0.

3.3 PERMANENT DEFORMATION

Permanent deformation, commonly referred to as rutting, can be of several types, and the cause of each type differs. Instability rutting, which occurs within the asphalt concrete layer develops when the properties of the compacted asphalt concrete pavement are inadequate to resist the stresses imposed upon it. The degree of instability rutting is enhanced when high ambient temperatures prevail and when frequent repetitions of heavy axle loads are applied. The time of loading, i.e. low vehicle operating speed, further accelerates the rate of which instability rutting develops.

Properties of the aggregate that is incorporated into the paving mixture have the predominant role in controlling instability rutting. Physical properties (particle shape, soundness and toughness) and gradation characteristics of the aggregate must be carefully specified to provide adequate shear resistance and to achieve the aggregate "skeleton" that is required within the compacted pavement to provide for load transfer from tires to the pavement support layers.

Methodology has been described in the TAC Final Report (92-4 Vol. 1) to control traffic-induced permanent deformation of asphalt pavements under high seasonal temperature conditions. The methodology is based on concepts which include:

I) The critical stiffness modulus (i.e. relationship between stress and strain as a function of time of loading and temperature) of a compacted paving mixture at 40°C is 2.38 x 10^8 N/m².

ii) A relationship exists between paving mixture stiffness at 40°C and the minimum stiffness which the asphalt cement should exhibit at that temperature, if the asphalt cement is to serve its contributory role in mitigating permanent deformation.

iii) The critical stiffness of initially aged (after plant mixing) conventional asphalt cement is 5.0 x 10^5 N/m² at 40°C, if the mineral aggregate in the paving mixture possesses excellent angularity properties, and is 1.0 x 10^6 N/m² at 40°C, if the mineral aggregate is of poor angularity. The critical stiffness value of 5.0 x 10^5 N/m² for initially aged asphalt translates to a limiting stiffness of approximately 1.5 x 10^5 N/m² for asphalt in its original (as delivered) condition.

Figures 3, 4 and 5 illustrate stiffness properties of CGSB Group A, B and C asphalts respectively, at various loading times, in their original (as delivered) condition. In principle,

to mitigate permanent deformation at the prevailing operating speed, it is necessary to select an asphalt cement (if available) that possesses the critical stiffness properties defined above, and to specify aggregates that exhibit excellent angularity properties.

To this point, methodology which has been described to determine asphalt stiffness at elevated temperatures, under a range of loading times, is based on long established, conventional technology. Some members of the TAC Steering Committee have suggested that, on the basis of Canadian experience, nomographic solutions from Figures 3, 4 and 5 are too conservative (i.e. softer asphalts are regularly used than would be specified using those Figures). The TAC Report describes a concept which has evolved recently, and which was made possible using SHRP asphalt binder specifications and testing protocols. This initiative has enabled a series of design pavement temperature isotherms to be superimposed onto Figure 3, 4 and 5 to create Figure 6, 7 and 8 respectively (for each of Group A, B and C asphalts). The method of proposed use of Figures 6, 7 and 8, for selecting the appropriate asphalt cement for the high pavement design temperature condition is described in Section 7.0 (Step 5.2).

3.4 FATIGUE

Two types of fatigue distress, thermal and structural, can develop within an asphalt concrete pavement structure. Thermal fatigue results from repetitive thermal cycles, but the process is not well understood and is not further referenced. Structural fatigue cracking usually occurs when a pavement is stressed to the limit of its fatigue capacity by repetitive loads, or when loads are applied that exceed the capacity of the pavement structure.

Predominant paving mixture properties that influence fatigue related performance include:

I) excessively stiff asphalt cement
ii) deficient asphalt cement content.

To mitigate structural fatigue cracking, following guidelines have been proposed:

- when the total asphalt concrete layer thickness is less than 125 mm, paving mixtures of low stiffness (i.e. low viscosity asphalt cements) are preferred.
- when the total asphalt concrete layer thickness is greater than 125 mm, paving mixtures of high stiffness (i.e. high viscosity asphalt cements) are preferred.

In the actual asphalt cement selection procedure described in Section 7.0, methodology is provided for considering pavement fatigue issues when more than one alternative asphalt cement is available that satisfies both low and high service temperature requirements (i.e. transverse cracking and instability rutting).

3.5 MOISTURE SENSITIVITY AND STRIPPING

Stripping of the asphalt film from the aggregate occurs when there is loss of adhesion between the aggregate surface and the asphalt cement, and is primarily due to the action of water. This problem is essentially one of aggregate-asphalt cement compatibility, and is normally resolved by incorporation of an anti-stripping agent into the asphalt mix. Performance tests are normally performed in the laboratory to determine stripping potential and the effectiveness of such admixtures in limiting its occurrence.

Stripping potential does not appear to be directly related to the penetration grade or group of asphalt cement selected, but it is related in a complex physical and/or chemical manner to the interaction between the asphalt and aggregate forming the paving mixture.

3.6 AGING OF THE ASPHALT/AGGREGATE SYSTEM

Aging, or age hardening of the asphalt cement, in an asphalt paving mixture, is the change in the rheological properties (viscosity, penetration) that occurs during the plant mixing and placement operation, and that continues during the service life of the pavement. The aging process can be controlled by ensuring that suitable volumetric properties of the paving mixture are provided during the mix design process and that proper plant mixing and compaction practices are followed during construction. However, it is important to be aware that selection of the best quality of asphalt cement that is available in the CGSB Standard will have positive effects on the pavement quality, if all other design and construction practices are properly undertaken.

3.7 DURABILITY

Durability of an asphalt pavement refers to its competence to maintain structural integrity under operating conditions, i.e. under the influence of traffic, moisture and freeze/thaw. Durability of a compacted pavement is influenced by mixture and compaction qualities achieved during

construction, by the air voids in the compacted pavement and by the thickness of asphalt film provided on the aggregate particles within the mixture. Durability properties are less significantly influenced by the grade of asphalt selected than by matters associated with asphalt-aggregate compatibility and by the paving mixture design process.

4.0 SELECTION OF DESIGN PAVEMENT TEMPERATURES

4.1 INTRODUCTION

SHRP has created a weather database that contains temperature statistics for approximately 1850 weather stations in Canada. The database is contained within the Superpave Binder Selection Program: SHRPBIND, Version 2.0, May, 1995. Location information includes station names, province or territory, longitude, latitude and elevation of each station, and the following relevant temperature information:

I) DMAT - which is the design maximum air temperature, and is the average of at least ten years of yearly maximum air temperature (YMAT), and YMAT is defined as the yearly maximum 7 day floating average of the daily maximum temperatures.

ii) σ_{DMAT} - which is the standard deviation of at least ten years of YMAT.

iii) T_{air} - which is the average of at least ten years of the annual coldest air temperature.

iv) σ_{air} - which is the standard deviation of T_{air}.

Within the TAC Report, descriptions are provided of methodology that has been used in this Guide, to enable design pavement temperatures to be predicted, for both the hot and cold service temperature conditions, using the four temperature statistics defined above.

An understanding of statistics principles, that are associated with normal distribution theory, is necessary to appreciate how SHRP has accommodated the issue of risk, i.e. probability. A normal distribution curve is shown in Figure 9. The area under a normal distribution curve represents the total number of records in a dataset. The area under a portion of the curve represents the number of pieces of data (and hence the percent) of the dataset. In the illustration in Figure 9, the 7-day average maximum pavement temperature situation is represented. For each case of $\bar{x} \pm 1\sigma$, 2σ, or 3σ, the percentage of occurrences, and also the probability of temperatures occurring outside of those limits may be determined. In the high design temperature case, one is only concerned with probability that a temperature occurrence may exceed (i.e. fall to the right) of a definable probability level. Thus, when the mean temperature ($\bar{x}$) is considered, there exists a 50 percent probability that an annual temperature occurrence

will exceed $\bar{x}$. Similarly for $\bar{x} + 1\sigma$ and for $\bar{x} + 2\sigma$, there exists 84 percent and 98 percent probabilities respectively, that annual temperature occurrences will not exceed those limits.

The same principles apply for probability consideration for the low temperature design case.

4.2 SOURCE OF CANADIAN TEMPERATURE DATA

The SHRPBIND (Version 2.0) program includes a database of 1800+ Canadian weather stations. A copy of the Canadian data is available from TAC at the following address:

Transportation Association of Canada
2323 St. Laurent Boulevard
Ottawa, Ontario, Canada K1G 4K6
Telephone: (613) 736-1350
Fax: (613) 736-1395

For information on the SHRPBIND program, contact:

Gonzalo Rada, Ph.D., P.E., Branch Manager
PCS/Law Engineering
A Division of Law Engineering, Inc.
12104 Indian Creek Court, Suite A
Beltsville, Maryland 20705-1242
Telephone: (301) 210-5105
Fax: (301) 210-5032

Climate information is available at Environment Canada offices throughout Canada. Table 1 shows the addresses and current (1995) telephone numbers of some regional and local Environment Canada offices.

Figure 10 is presented to illustrate the cold air temperature regime in Canada. Values of T_{air} - $2\sigma_{air}$ (as defined in Section 4.1) for the Canadian weather stations included in the SHRP database have been used to develop cold air isotherms at intervals at 5 °C. This Figure should be used only for information purposes and should not be used as a substitute for determining more precise site-specific information.

Figure 11 is presented to illustrate the hot air temperature regime in Canada. Values of DMAT + 2σDMAT (defined in Section 4.1) for the Canadian weather stations included in the SHRP database, have been used to develop high air temperature isotherms at intervals at 5°C. This Figure should be used for information purposes only. It should not be used as a substitute for determining more precise site-specific information.

4.3 SELECTION OF THE DESIGN LOW PAVEMENT TEMPERATURE

TAC provided following guidelines that are to apply to the methodology for selection of the design low pavement temperature:

I) pavement temperature at surface, as estimated from air temperature, is to be used.
ii) the SHRP Superpave air temperatures are to be used.
iii) the method for estimating pavement surface temperature is to be based on previous work of other researchers, and is to recognize temperature correlations that are available from existing Canadian test roads, including the three C-SHRP sites. Pavement temperatures in the winter are known to be warmer than prevailing air temperatures.

Pursuant to completion of an analysis to correlate air and pavement surface temperature data at Canadian test road sites, the following equation has been adopted to determine design low pavement temperature at the 98 percent probability level (i.e. 98 percent probability that the design low pavement temperature will not be exceeded):

$$\text{Design Low } T_{pavement}\,(°C) = 0.859\,(T_{air} - 2\sigma_{air}) + 1.7°C \qquad \text{---- Equation 1}$$

T_{air} and σ_{air} are defined in Section 4.1, and values of these statistics may be sourced as referenced on Section 4.2. The user, at his discretion may accept increased risk by deleting the $2\sigma_{air}$ value from Equation 1 (i.e. 50 percent risk factor) or by substituting 1σ (84 percent risk factor).

The TAC Report may be referenced for detailed information relative to the development of Equation 1 above.

The Guide user is reminded of the need for validating the value of T_{air}, available in the reference source, for any project specific application.

Note: **The Guide user is cautioned to read and to understand fully the contents of Chapter 6.1 (Safety Factors) and the requirement contained therein for refining design low pavement temperature values derived from Equation 1, in accordance with guidelines contained in Chapter 6.1.**

4.4 SELECTION OF THE DESIGN HIGH PAVEMENT TEMPERATURE

Protocols that were developed by SHRP have been adopted for determining the design high pavement temperature from air temperature data that is provided in the SHRPBIND program.

In the methodology that is defined below for determining the design high pavement temperature, SHRP has defined the critical location in the pavement structure as being that point located 20 mm below the surface of the pavement. The user may assign the risk of the design high pavement temperature being exceeded, in terms of probability, by defining T_{air} in Equation 2 below, as follows:

Probability (%)	Value of T_{air}
98	DMAT + $2\sigma_{DMAT}$
84	DMAT + $1\sigma_{DMAT}$
50	DMAT

DMAT and σ_{DMAT} are defined in Section 4.1 and values of these statistics may be sourced as referenced in Section 4.2

The SHRP procedure to determine the design high pavement temperature at 20 mm depth (Design High T_{20mm}) in °C is determined in the following manner:

I) Determine pavement surface temperature using the following equation:

$$T_{surf}\,(°C) = T_{air} - 0.00618\ lat.^2 + 0.2289\ lat. + 24.4 \qquad \text{---- Equation 2}$$

where: lat. = latitude of the project site

ii) Convert T_{surf} to °F using:

°F = 9/5 °C + 32

iii) Determine Design High $T_{0.75\ in.}$ using:

Design High $T_{0.75\ in.}$ (°F) = 0.9564 T_{surf} - - - - Equation 3

iv) Convert Design High $T_{0.75\ in.}$ to Design High T_{20mm} using:

°C = 5/9 (°F - 32)

5.0 TRAFFIC

5.1 CONSIDERATION OF TRAFFIC AND MAXIMUM DESIGN TEMPERATURE

A prerequisite to undertaking selection of the asphalt binder required at the design high pavement temperature is to characterize the traffic that will use the pavement. Two characteristics should be identified, i.e.:

i) operating speed
ii) design traffic loading, expressed in terms of 80 kN ESALs

Three operating speeds (i.e. loading times) have been considered in developing the asphalt binder selection methodology presented in this Guide. These speeds are 100 km/h, 50 km/h and 20 km/h, for which corresponding pavement loading times are 0.01 seconds, 0.02 seconds and 0.05 seconds. These vehicle operating conditions are similar to those that are used by SHRP in the Superpave mix design process. Figures 6, 7 and 8 have been developed on the foregoing basis.

Asphalt binder requirements for other prevailing operating speeds may be interpolated in Figures 6, 7 and 8.

Design traffic loading, expressed in cumulative ESALs over the design life of the pavement structure, influences the binder selection process in the manner presented in Section 7.0. Three traffic levels are provided in Superpave, and have been selected for utilization in this Guide, i.e.:

Design Level	Design 80 kN ESALs
1 (Low)	$\leq 10^6$
2 (Intermediate)	$> 10^6 \leq 10^7$
3 (High)	$> 10^7$

The influence of design traffic is predominantly provided for by specifying requirements of the mineral aggregate (gradation, particle shape, texture, fracture, etc.). In high and intermediate design level cases, the coarse aggregate fraction should be of the maximum practical nominal

size and should consist of 85 percent or more of particles with two or more fractured faces. The fine aggregate fraction should be of high angularity. Selection of the most appropriate asphalt binder, from those alternatives that may satisfy the maximum design pavement temperature requirements, is a function of the design traffic level, and is described in Section 7.0. In principle, for low traffic volumes (Design Level 1), the softest penetration grade that satisfies the high design pavement temperature requirement (at the prevailing operating speed) should be identified. Conversely, for high traffic levels (Design Level 3) the hardest penetration grade, of those potential candidates, would be identified.

5.2 CONSIDERATION OF TRAFFIC AND MINIMUM DESIGN TEMPERATURE

Selection criteria relative to minimum design temperature are subjective in nature and should be in accordance with the following guidelines:

I) For low traffic volumes (Design Level 1), identify one of the softer penetration grades that are available candidates.

ii) For high traffic volumes (Design Level 3) identify the hardest penetration grade from available candidates.

iii) For intermediate traffic volumes (Design Level 2) identify the most appropriate penetration grade from available candidates, using discretion and in consideration of the range of available materials.

6.0 OTHER CONSIDERATIONS

6.1 SAFETY FACTORS

.1 For Design Service Temperature

It has been discussed, in previous Sections, how the probability of extreme temperature occurrences, that exceed average annual mean temperature values, can be recognized by adding (or subtracting) 1σ or 2σ to or from mean temperature values. In this manner the temperature variable may be addressed.

.2 For Initial Age Hardening of Asphalt Cement

Asphalt cements are specified in their original (tank) condition in the CGSB specification. During asphalt plant mixing initial hardening of the asphalt cement occurs and the age hardening process continues during the service life of the pavement. As a result, rheological properties that influence low temperature pavement performance (i.e. low temperature properties) are not the same as for the original asphalt cement. This must be recognized in the asphalt cement selection process to ensure that satisfactory pavement performance results. This may be accomplished by incorporation of a "safety factor" to further reduce the design low pavement temperature (Equation 1, Chapter 4.3). Robertson (1987) used a value of 10°C to, at least in part, account for the age hardening factor. A 10°C safety factor should be used in the asphalt selection procedure outlined in Chapter 7.0, unless a Canadian agency has sufficient experience to warrant use of an alternate safety factor value.

The user should review the Final TAC Report (Section 6.3.4) for further explanation in respect to aging of CGSB asphalt cements.

6.2 AGGREGATE SELECTION FOR PAVING MIXTURES

It is beyond the scope of this Guide to identify the detailed properties required of aggregates that should be used for each traffic level and operating speed condition. The designer of paving mixtures may refer to the SHRP Superpave Mix Design Manual for guidance in this regard. As

well, previous experience with locally available aggregates may provide an indication of their suitability for similar uses in the future.

6.3 PERFORMANCE TESTING OF PAVING MIXTURES

The purpose of this Guide is to provide the user with guidelines to select the most appropriate asphalt binder for an asphalt paving mixture on a project specific basis. Beyond this point, the procedure for developing the job mix formula, on the basis of laboratory trial mixtures, require methodologies that are beyond the scope of this Guide. In this regard key properties of paving mixtures, that should be considered, are related to aggregate and mixture volumetric properties.

Laboratory testing programs exist that can assess the probable performance of candidate paving mixtures in respect to permanent deformation, fatigue, durability and moisture sensitivity and stripping. The Guide user should refer to the TAC report for further information in this regard.

6.4 LIMITATIONS

CAN/CGSB-16.3-M90 contains specified requirements for asphalt cements for road purposes that are applicable to Canadian conditions. Using the asphalt binder selection procedure that is provided in Section 7.0 of this Guide, the user may occasionally identify requirements for asphalt binder that cannot be provided by conventional asphalt cements that are available in this Standard. Hence the continuing use of the term "asphalt binder" in this Guide. The user may have to look to other sources such as polymer modified asphalts or specially "engineered" asphalts to fulfil some project specific requirements.

7.0 THE ASPHALT CEMENT SELECTION PROCEDURE

7.1 BACKGROUND

A detailed step-by-step procedure is provided in this Section, to enable the Guide user to identify, from those candidate materials which are found in CAN/CGSB-16.3-M90, that asphalt cement that is optimum for use on a project specific basis.

7.2 STEP-BY-STEP PROCEDURE

The step-by-step asphalt cement selection procedure is illustrated in Figure 12, which may also be used as a Worksheet for performing the analysis. Each step is discussed below.

Step 1: Name the Project; provide highway or street identification and the terminal (start and stop) points. Determine latitude, longitude and elevation of project.

Step 2: Define the Traffic Parameters

2.1 Design Traffic (Refer to Chapter 5). Information on heavy traffic - i.e. ESALs - is normally available from traffic engineering personnel within the highway or street agency. In the absence of available data from such sources, ESALs may be estimated using the following equation:

$$ESAL = AADT * HVP * HVDF * NALV * TDY$$

where:

ESAL	=	Equivalent Single Axle Loads per Lane per Year
AADT	=	Average Annual Daily Traffic (all lanes, both directions)
HVP	=	Heavy Vehicle Percentage (divided by 100)
HVDF	=	Heavy Vehicle Distribution Factor (percent of heavy vehicles in the design lane)
NALV	=	Number of equivalent axle loads per vehicle (Truck Factor)
TDY	=	Traffic Days per Year

The above equation provides an estimate of the number of ESALs in the design lane in one year. It is simply necessary to know the project design life and traffic growth factor to determine the anticipated cumulative ESALs in the project design life.

2.2 Operating Speed; geometric designers and/or regulatory agencies usually dictate what the operating speed will be, especially with respect to truck traffic. These sources should be requested to provide relevant information.

Step 3: Determine Low and High Air Temperatures; Refer to Chapter 4 and ascertain the four air temperature statistics that are required to determine the relevant design pavement temperatures. Sources of air temperature statistics are provided in Chapter 4.

A risk analysis should be performed, at this stage, to establish the level of probability that is acceptable in respect to those selected design temperatures being exceeded during the design period. Reference should be made to Chapter 4 in this regard. Assign either the mean temperature ($\bar{x}$) or mean temperature plus or minus 1σ or 2σ as being the high and low design air temperatures respectively.

Step 4: Calculate Low and High Design Pavement Temperatures

4.1 Design Low Pavement Temperature

Using the low air temperature statistics determined in Step 3, calculate the design low pavement temperature (Design Low $T_{pavement}$) as follows:

$$\text{Design Low } T_{pavement}\ (°C) = 0.859\ (T_{air} - 2\sigma_{air}) + 1.7° \qquad \text{---- Equation 1}$$

If the risk analysis undertaken in Step 3 determines that the 98 percent probability level (i.e. $2\sigma_{air}$) is excessive, adjust ($1\sigma_{air}$) or delete the σ_{air} factor to suit the selected risk factor (i.e. 84 percent or 50 percent).

A Safety Factor of 10°C should be applied to the design low pavement temperature determined from Equation 1. **To clarify, the value of Design Low $T_{pavement}$ (°C) should be reduced an additional 10°C, as explained in Section 6.1.**

4.2 Design High Pavement Temperature

I) Select the value of T_{air} on the basis of acceptable risk level (probability) from the following tabulation:

Probability (%)	**Value of T_{air}**
98	DMAT + $2\sigma_{DMAT}$
84	DMAT + $1\sigma_{DMAT}$
50	DMAT

Refer to the source referenced in Chapter 4 for values of DMAT and σ_{DMAT}.

ii) Determine pavement surface temperature from Equation 2:

T_{surf} (°C) = T_{air} - 0.00618 lat.^{2} + 0.2289 lat. + 24.4 ---- Equation 2
where: lat. = latitude of project in degrees

iii) Convert T_{surf} from °C to °F using:
°F = 9/5°C + 32

iv) Determine Design High $T_{0.75\ in.}$ from Equation 3:
Design High $T_{0.75\ in.}$ (°F) = 0.9564 T_{surf} ---- Equation 3

v) Convert Design High $T_{0.75\ in.}$ (°F) to Design High T_{20mm} (°C) using:
°C = 5/9 (°F - 32)

It is important to note that Equations 2 and 3, as currently issued by SHRP, require the user to convert temperatures (°C) to and from °F.

Step 5: Identify Candidate Asphalt Cements

5.1 For Design Low Pavement Temperature

Using the design low pavement temperature (adjusted colder by the 10°C Safety Factor in Step 4.1) as the pavement cracking temperature, refer to Figure 1 and record those candidate Group A, B or C asphalts by penetration grade, where the cracking temperature isotherm intersects the lines that represent the specification boundaries for each penetration grade specified in the CGSB standard.

5.2 For Design High Pavement Temperature

Using the design high pavement temperature, as determined in Step 4.2, and the design or legislated operating speed determined in Step 2.2, enter Figures 6, 7 or 8. Identify the horizontal line that represents the operating speed for the project, and proceed to the right to its intersection with the dashed line that most closely represents the design high temperature value. Interpolation between design temperature lines is a valid procedure. Identify and record those penetration grades of asphalt cement (i.e. the labelled, solid lines) that lie to the right hand side of the intersection of the operating speed/design pavement temperature intercept. If an intercept point occurs on that part of a design pavement temperature line that is shown in dot legend (i.e. not the dashed portion) then this means that there is no CGSB Group A, B or C asphalt cement that meets the required properties at the design high temperature end of the spectrum.

Step 6: Select the Optimum Asphalt Cement

Three possibilities exist at this point. These are defined below and an appropriate final step is described in each case.

I) One or more CGSB asphalt cements exist in both the lists prepared in Step 5.1 and Step 5.2 (i.e. there is more than one product that satisfies both design temperature conditions). In this situation, the Guide user should consider the significance of the selected product upon permanent deformation and fatigue as discussed in Chapter 3. This is done by reviewing preferred

paving mixture and asphalt cement stiffness requirements as a function of traffic level and pavement thickness, using the following matrix:

Traffic Level	Pavement Thickness ($T_{pavement}$)	
	≤ 125 mm	> 125 mm
1 (Low)	low viscosity	high viscosity
2 (Intermediate)	user discretion	high viscosity
3 (High)	not normally applicable	high viscosity

ii) One or more CGSB asphalt cements will satisfy each of the two design temperature conditions, but none will satisfy both.

In this situation, the Guide user should re-consider the risk analysis that was undertaken as part of Step 3. In the Canadian climate environment, it is not normally desirable to knowingly increase the risk of incurring low temperature pavement cracking by selecting a design cold air temperature than is warmer than $T_{air} - 2\sigma_{air}$ (Chapter 4). However, it may be acceptable to compute the design high pavement temperature (T_{20mm}) on the basis of DMAT rather than DMAT + 1σDMAT or 2σDMAT, especially if Traffic Levels 1 or 2 have been estimated. That is to say, the Guide user assesses whether some increased risk of permanent deformation (rutting) can be tolerated in order to accommodate selection of a CGSB asphalt cement.

iii) One (or more) CGSB asphalt cements will satisfy the low design pavement temperature requirement, but none will satisfy the high design pavement temperature requirement at the prevailing operating speed.

In this situation, the Guide user has two fundamental options, i.e. -

- Ignore the risks that are associated with either the design low or high pavement temperature condition, and select that CGSB product that is the best compromise (not recommended), or
- Specify an alternative asphalt binder that will meet the service temperature requirements. Alternative products may include polymer modified asphalt or other types of enhanced binders.

7.3 WORKED EXAMPLES

The following examples demonstrate some of the basic possibilities that the Guide user may encounter, as identified in Step 6 of the above selection procedure. Flow Charts that have been used to complete the selection process, are provided following the examples.

Example 1: -

The project is located on a provincial highway in western Canada. The highway traverses gently rolling terrain and has a design operating speed of 100 km/h. Traffic Level No. 1 conditions prevail (ESALs < 10^6). The total design pavement thickness is 100 mm.

The SHRP database contains the following statistics for a nearby weather station, which is in close proximity to the project.

Latitude - 49.3°
Longitude - 101.0°
Elevation - 442 m

Average Low Temperature	(T_{air})	=	- 39°C
	σair	=	2.9°C
Average High Temperature	(DMAT)	=	32°C
	σ_{DMAT}	=	2.4°C

Following policies exist in respect to risk in pavement design and performance for Traffic Level 1 roads: -

- Low temperature cracking to be mitigated at the 98% probability level
- Low pavement temperature safety factor = 10°C applies
- Permanent deformation (rutting) tolerable at the 50% probability level

At completion of Steps 1 to 5 on the accompanying Flow Chart, it is apparent that, to satisfy low service temperature conditions the following CGSB asphalts may be used: -

200 - 300 Group A
300 - 400 Group B

To satisfy high service temperature requirements, any Group A asphalt with a penetration of less than 300 is acceptable. Therefore the preferred choice is 200 - 300 pen. (Group A).

Example 2: -

The project involves construction of a passing lane on a rural highway that will be subjected to loaded logging and chip trucks operating at a speed of 80 km/h or less. The design traffic level is Intermediate ($> 10^6$ ESALs $\leq 10^7$). The total design pavement thickness is 150 mm.

The SHRP database contains the following relevant statistics at a nearby weather station:

Latitude - 51°
Elevation - 500 m

Average Annual Coldest Air Temperature	(T_{air})	=	-30°C
	σair	=	3°C
Average Yearly Maximum Air Temperature	(DMAT)	=	34°C
	σDMAT	=	2°C

Following jurisdiction policies exist in respect to risk for pavement design for Traffic Level 2 designs: -

- Low temperature cracking is to be mitigated at the 98% probability level.
- A low pavement temperature safety factor of 10°C applies.
- Permanent deformation is to be mitigated at the 98% probability level.

The candidate asphalt cements identified in Step 5.1 on the accompanying Flow Charts are the lowest pen./Group products that may be used. The candidate asphalt cements identified in

Step 5.2 are the highest pen./Group products that may be used. The CGSB pen. grade/Group products are then: -

For Low Temperature Design	For High Temperature Design
200 - 300, Group A	150 - 200, Group A or harder
300 - 400, Group A or softer	

A compromise solution is necessary since the two candidate products that most closely satisfy both temperature design requirements are 200 - 300 (Group A) and 150 - 200 (Group A).

Since a 10°C safety factor was designated for design at the low temperature condition, it is concluded that 150 - 200 pen. Group A asphalt cement should be specified for the project. This selection most adequately satisfies the criteria required to mitigate fatigue distress in a pavement whose thickness is greater than 125 mm.

Example 3:

The project is located on an urban freeway just outside of Toronto, Ontario. The design operating speed is 100 km/h. However, heavy traffic conditions will mean that traffic will be frequently stopped or slow moving. Traffic Level 3 conditions prevail (ESALs > 10^7). The tentative pavement design calls for in excess of 200 mm of asphalt concrete over a granular base layer.

The SHRP database contains the following statistics for Toronto Pearson International Airport (Station Code #6158733).

Latitude - 43.67°
Longitude - 79.63 m
Elevation - 173 m
Average Low Temperature (T_{air}) = -25°C
σair = 3.2°C

Average High Temperature	(DMAT)	=	31 °C
	σDMAT	=	1.8 °C

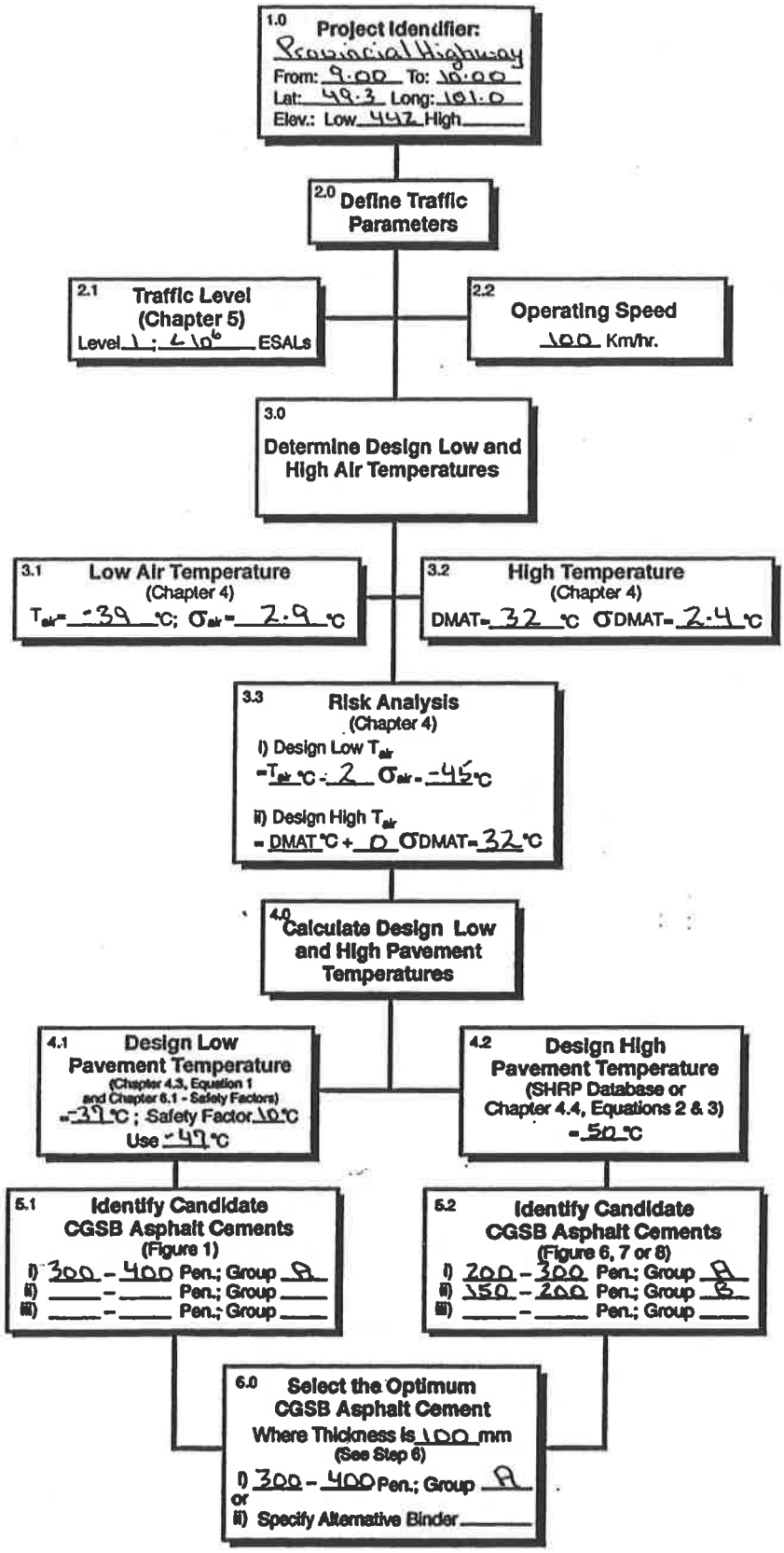
The following policies exist in respect to risk for Traffic Level 1 roads: -

- Low temperature cracking to be mitigated at the 98% probability level.
- Low pavement temperature safety factor = 10 °C.
- Permanent deformation (rutting) is to be mitigated at the 98% probability level.

To satisfy low temperature requirements, a 120 - 150 A or B Grade or softer and some C Grades are suitable.

However, for high temperature considerations, the A Grade that may be acceptable (requires extrapolation of curves on Figure 6) is harder than 80 pen. In this instance it would be preferable to select some premium asphalt.

EXAMPLE 1



R9754M12G

FIGURE 12 FLOW CHART FOR SELECTION OF ASPHALT BINDER

EXAMPLE 2

1.0 Project Identifier:
Passing Lane
From: Km 1.0 To: Km 3.0
Lat: 51 Long: ____
Elev.: Low 500 High 600

2.0 Define Traffic Parameters

2.1 Traffic Level (Chapter 5)
Level 2 ; $>10^6 \leq 10^7$ ESALs

2.2 Operating Speed
80 Km/hr.

3.0 Determine Design Low and High Air Temperatures

3.1 Low Air Temperature (Chapter 4)
T_{air} = -30 °C; σ_{air} = 3 °C

3.2 High Temperature (Chapter 4)
DMAT = 34 °C σDMAT = 2 °C

3.3 Risk Analysis (Chapter 4)
i) Design Low T_{air}
= T_{air} °C - 2 σ_{air} = -36 °C
ii) Design High T_{air}
= DMAT °C + 2 σDMAT = 38 °C

4.0 Calculate Design Low and High Pavement Temperatures

4.1 Design Low Pavement Temperature (Chapter 4.3, Equation 1 and Chapter 6.1 - Safety Factors)
= -30.1 °C ; Safety Factor 10 °C
Use -41 °C

4.2 Design High Pavement Temperature (SHRP Database or Chapter 4.4, Equations 2 & 3)
= 58 °C

5.1 Identify Candidate CGSB Asphalt Cements (Figure 1)
i) 200 - 300 Pen.; Group A
ii) 300 - 400 Pen.; Group A
iii) ____ - ____ Pen.; Group ____

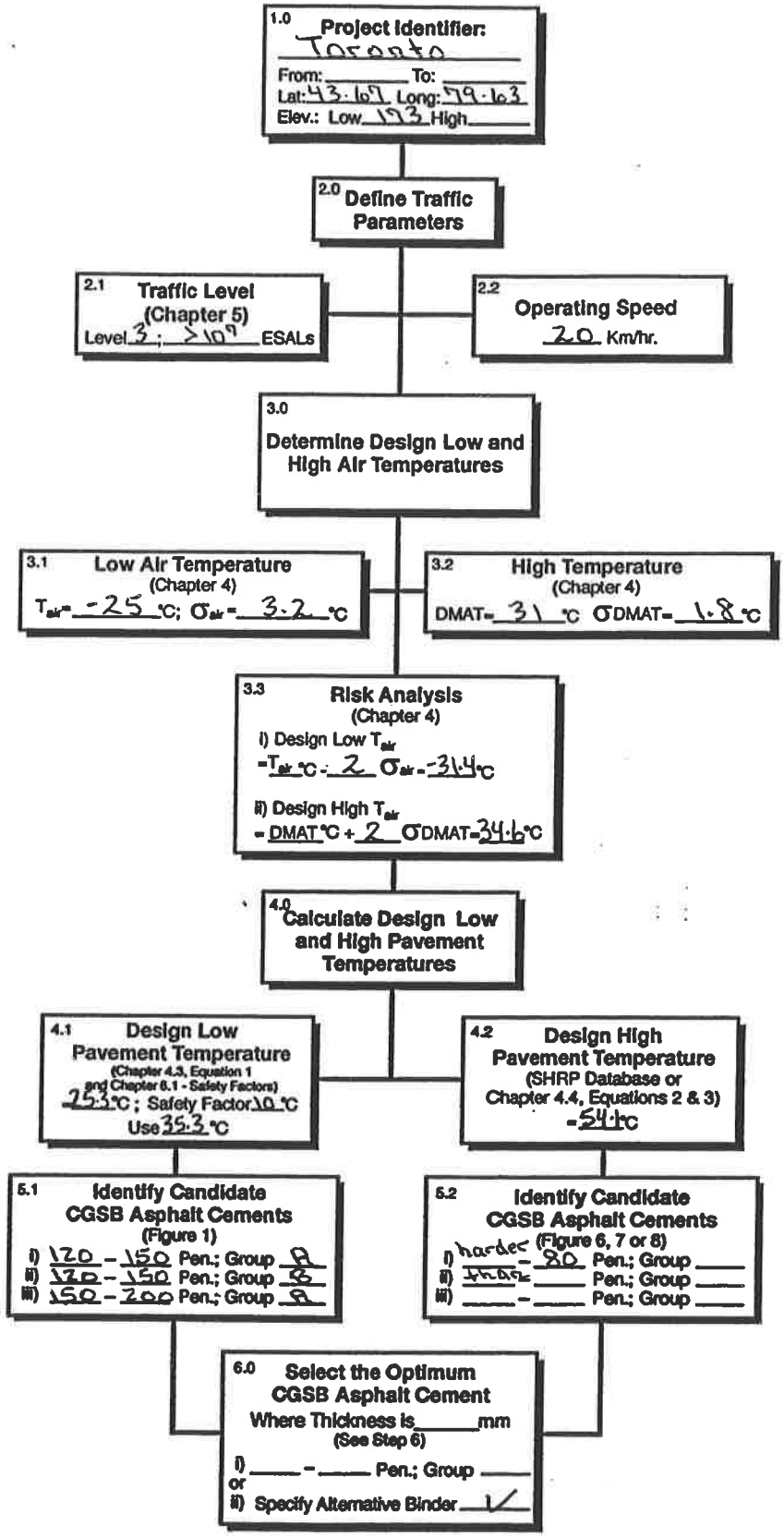
5.2 Identify Candidate CGSB Asphalt Cements (Figure 6, 7 or 8)
i) 150 - 200 Pen.; Group A
ii) ____ - ____ Pen.; Group ____
iii) ____ - ____ Pen.; Group ____

6.0 Select the Optimum CGSB Asphalt Cement
Where Thickness is 150 mm
(See Step 6)
i) ____ - ____ Pen.; Group ____
or
ii) Specify Alternative Binder ✓

R9754M12G

FIGURE 12 FLOW CHART FOR SELECTION OF ASPHALT BINDER

EXAMPLE 3



R9754M12G

FIGURE 12 FLOW CHART FOR SELECTION OF ASPHALT BINDER

TABLES

TABLE 1
SELECTED ENVIRONMENT CANADA OFFICES

National Office Climate Information Branch Canadian Meteorological Centre 4905 Dufferin St. Downsview, Ontario M3H 5T4 Tel: (416) 739-4328 Fax: (416) 739-4446	Climate Services Room 1000/266 Graham Ave. Winnipeg, Manitoba R3C 3V4 Tel: (204) 983-0586 Fax: (204) 983-4884	Ontario Climate Centre Ontario Climate Centre 4905 Dufferin Street Downsview, Ontario M3H 5T4 Tel: (416) 739-4516 Fax: (416) 739-4521
Regional and Local Offices Pacific & Yukon Region Climate and Applications Suite 700/1200 W 73rd Avenue Vancouver, BC V6P 6H9 Tel: (604) 664-9156 Fax: (604) 664-9133	Climate Services 300 Park Plaza 2365 Albert St. Regina, Saskatchewan S4P 4K1 Tel: (306) 780-6341 Fax: (306) 780-5311	Quebec Region Climate Services 100 Alexis Nihon Blvd. 3rd Floor, Ville Saint Laurent, Quebec H4M 2N8 Tel: (514) 283-1296 or (514) 283-1107 Fax: (514) 283-7149
Prairies & Northern Region Climate Services Twin Atria Building Room 200/4999 - 98 Avenue Edmonton, Alberta T6B 2X3 Tel: (403) 951-8881 Fax: (403) 495-3529	Climate Services Room 301/111 Research Dr. Saskatoon, Saskatchewan S7N 3R2 Tel: (306) 975-6909 Fax: (306) 975-5954	Atlantic Region Atmospheric Environment Branch 1496 Bedford Highway Bedford, NS B4A 1E5 Tel: (902) 426-9226 Fax: (902) 426-9158

FIGURES

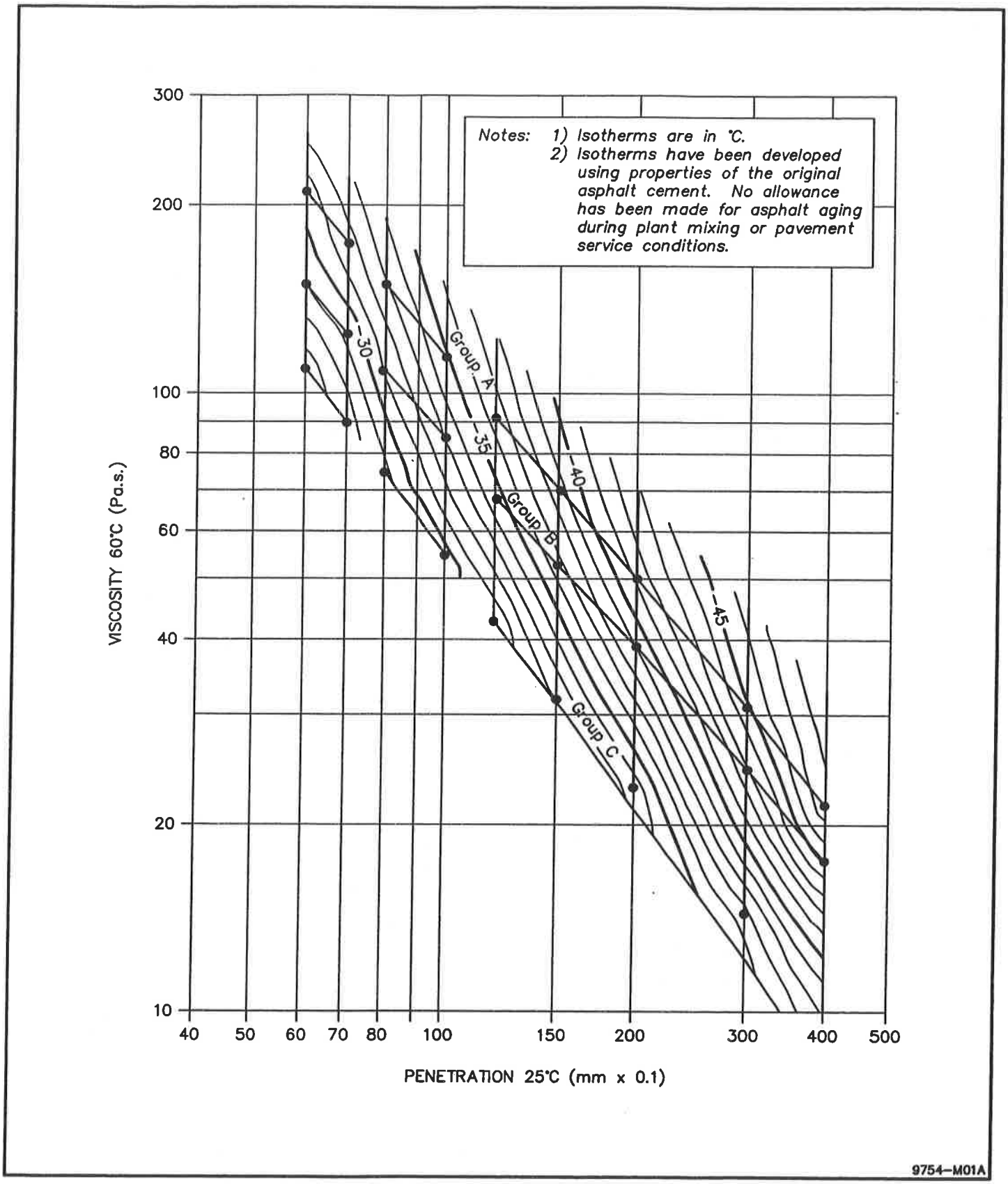


FIGURE 1 CRACKING TEMPERATURES VERSUS CGSB ASPHALT GRADE

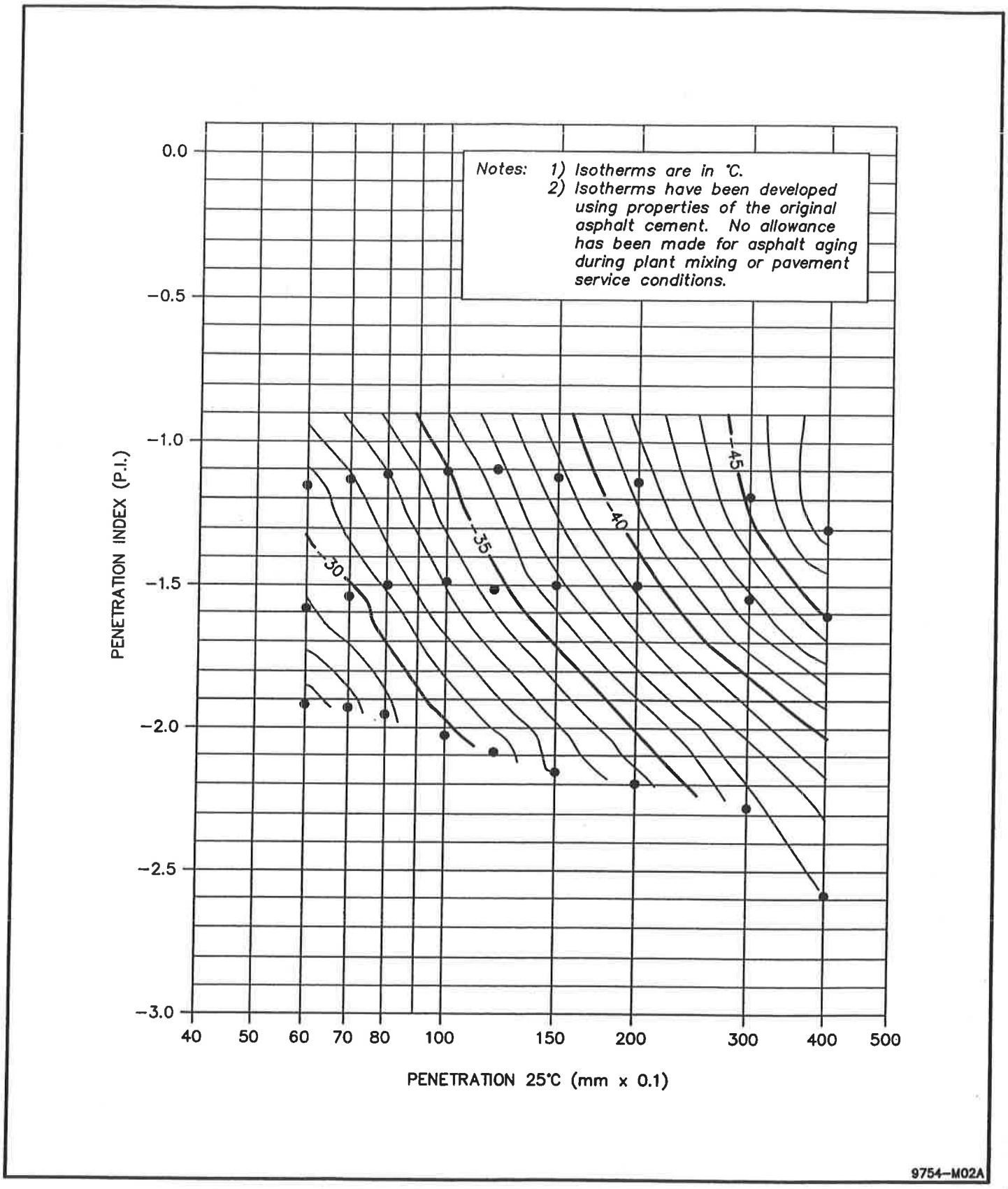
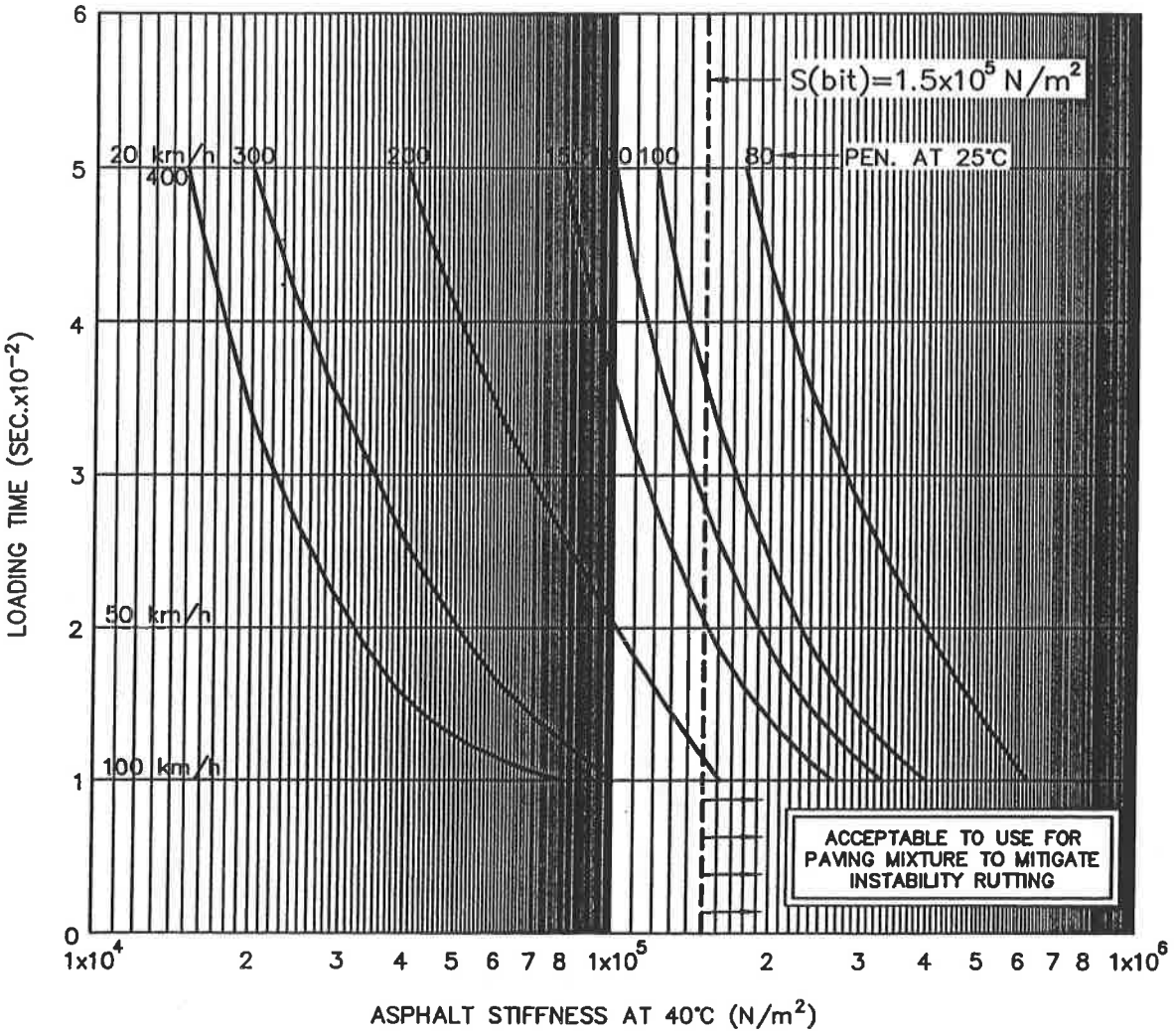
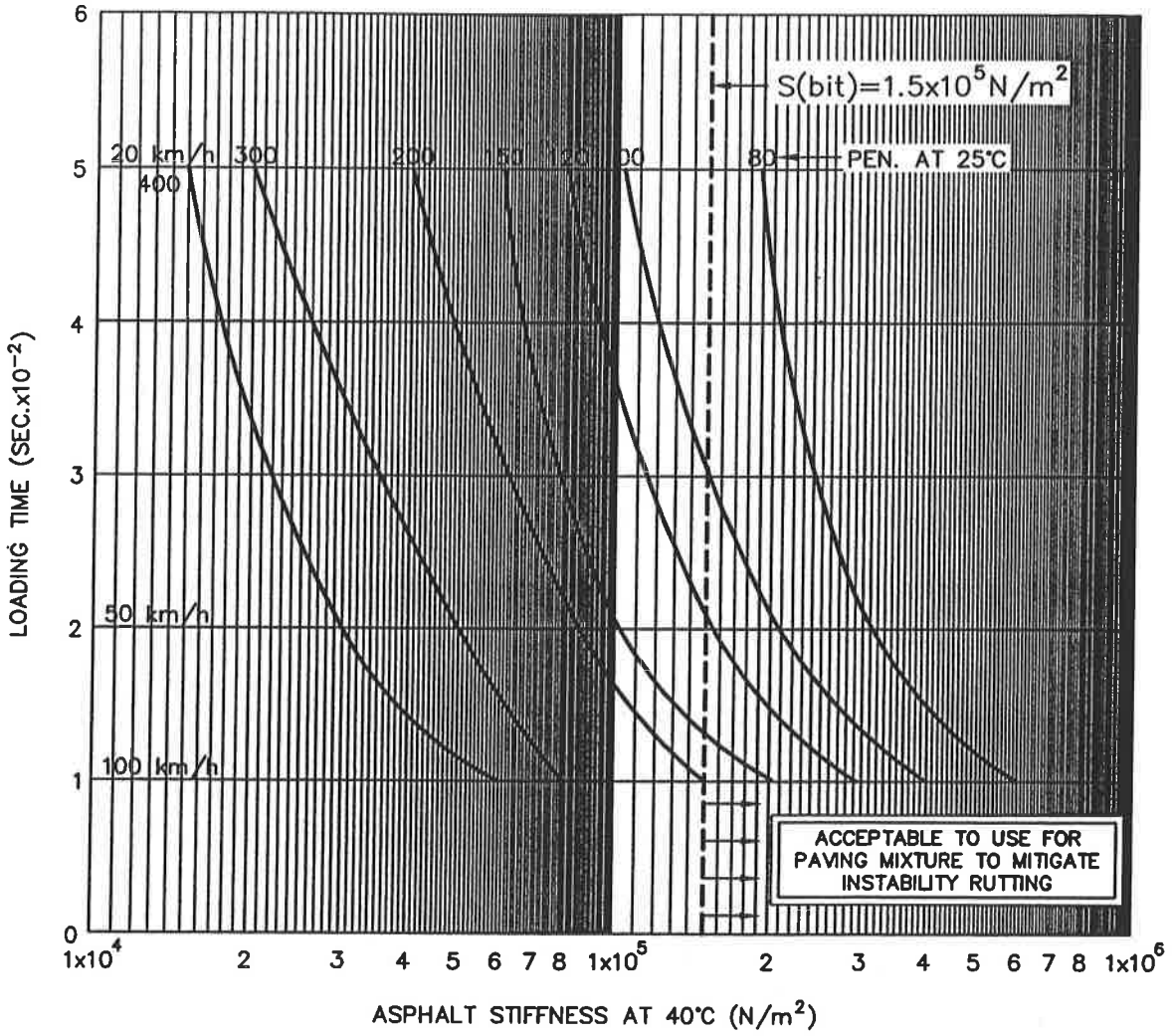


FIGURE 2 CRACKING TEMPERATURES VS PENETRATION AND PENETRATION INDEX (P.I.)



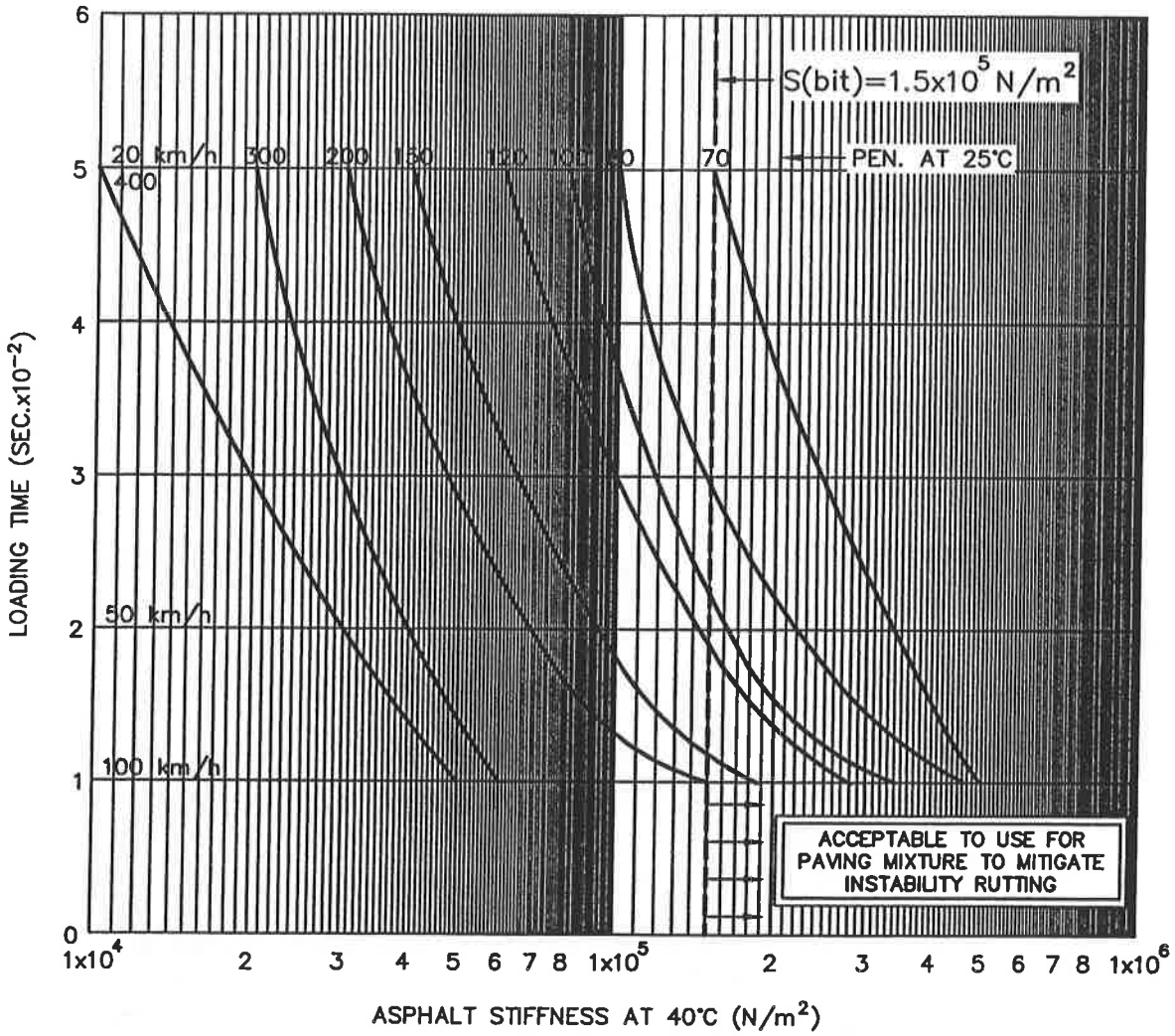
9754—M12C

FIGURE 3 STIFFNESS OF CGSB TANK ASPHALTS (GROUP A) AT 40°C AND VARIABLE LOADING TIME



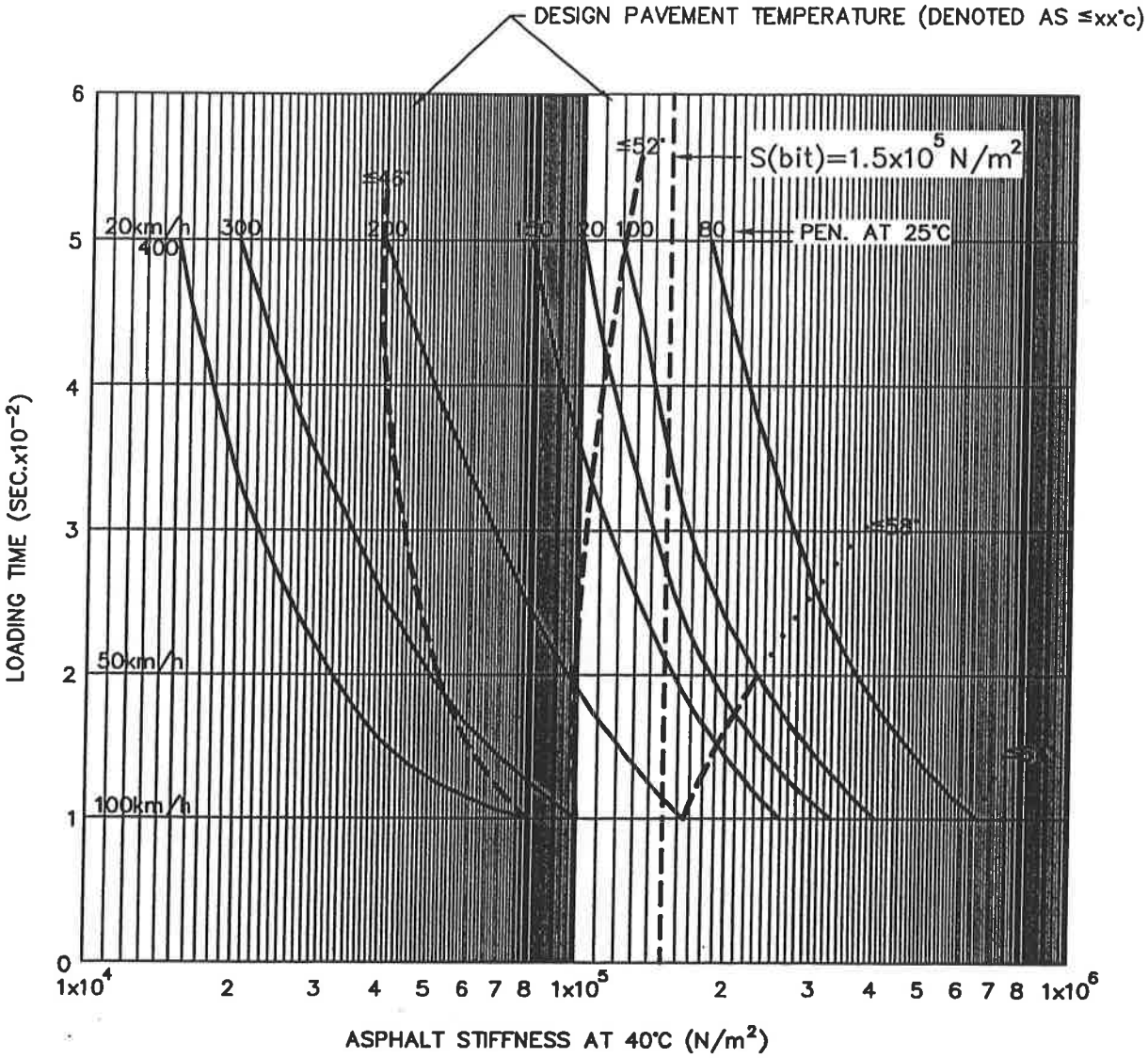
9754–M04B

FIGURE 4 STIFFNESS OF CGSB TANK ASPHALTS (GROUP B) AT 40°C AND VARIABLE LOADING TIME



9754–M05C

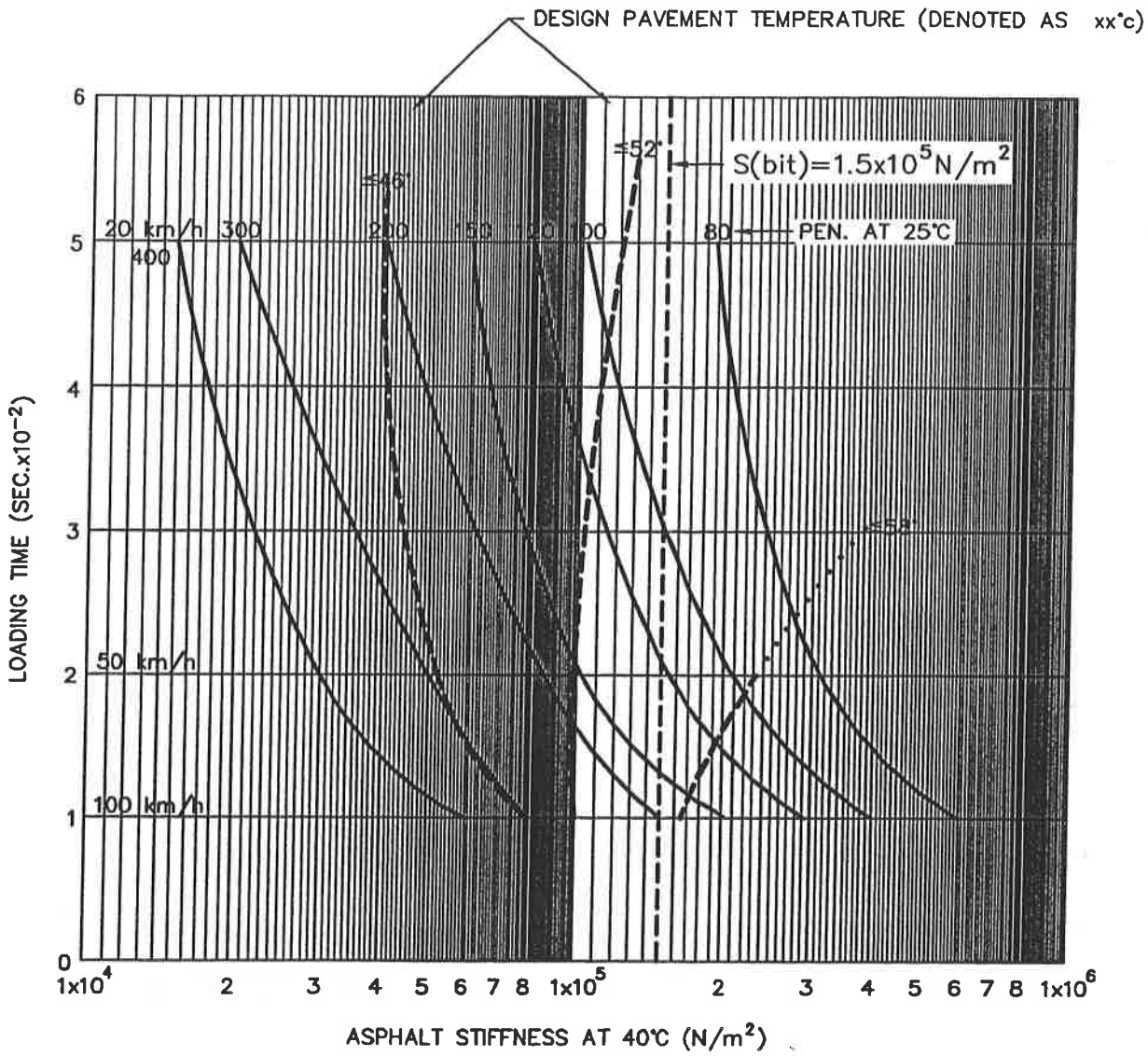
FIGURE 5 STIFFNESS OF CGSB TANK ASPHALTS (GROUP C) AT 40°C AND VARIABLE LOADING TIME



Notes: *1) Service conditions that prevail where the heavy lines are plotted (• • • • •) would have to be satisfied using a modified asphalt binder.*
2) Section 7.0 should be read in conjunction with this Figure.

9754—M06D

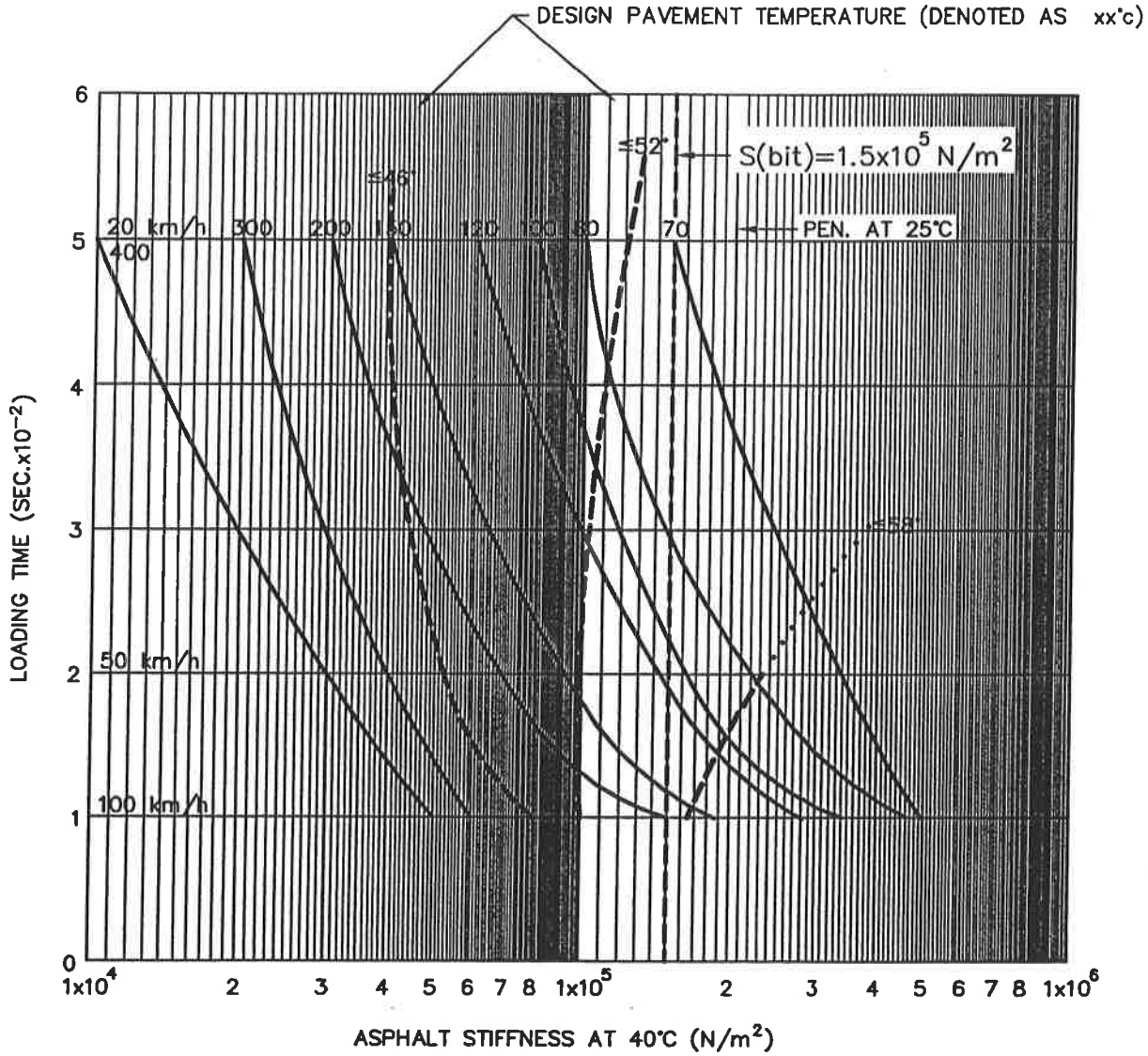
FIGURE 6 CGSB GROUP A ASPHALTS WITH MAXIMUM DESIGN PAVEMENT TEMPERATURE ISOTHERMS SUPERIMPOSED



Notes: *1) Service conditions that prevail where the heavy lines are plotted (• • • • • •) would have to be satisfied using a modified asphalt binder.*
2) Section 7.0 should be read in conjunction with this Figure.

9754—M07C

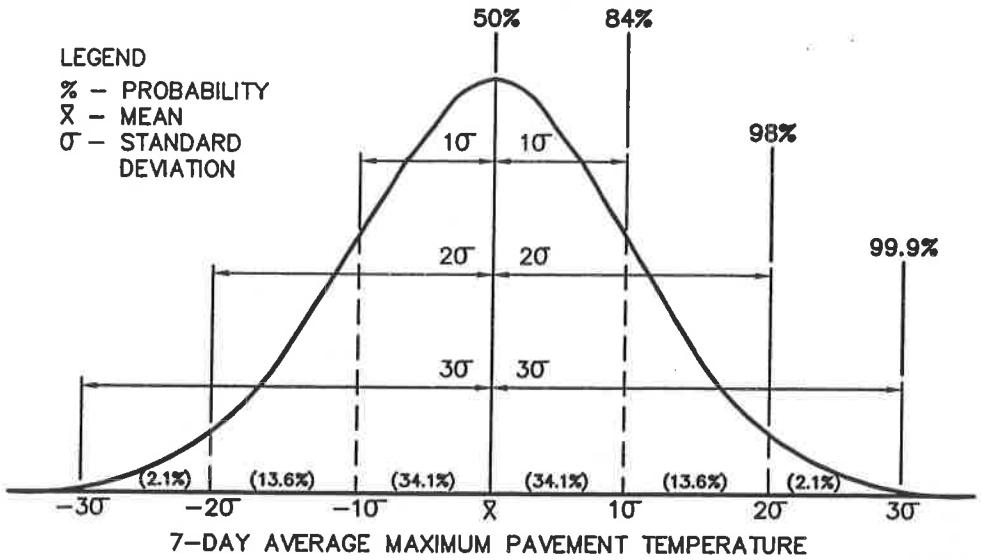
FIGURE 7 CGSB GROUP B ASPHALTS WITH MAXIMUM DESIGN TEMPERATURE ISOTHERMS SUPERIMPOSED



Notes: *1) Service conditions that prevail where the heavy lines are plotted (• • • • • •) would have to be satisfied using a modified asphalt binder.*
2) Section 7.0 should be read in conjunction with this Figure.

9754–M08C

FIGURE 8 CGSB GROUP C ASPHALTS WITH MAXIMUM DESIGN TEMPERATURE ISOTHERMS SUPERIMPOSED



9754–M09A

FIGURE 9 NORMAL DISTRIBUTION CURVE

Note: This figure is for illustration pupose only and should not be used to estimate Project Specific Design Temperatures

R9754M10A

FIGURE 10 MINIMUM DESIGN AIR TEMPERATURE/ISOTHERMS (Tair - 2 σair)

Note: See Section 4.1 for Definitions of Tair, σair

Note: This figure is for illustration pupose only and should not be used to estimate Project Specific Design Temperatures

R9754M11A

FIGURE 11 MAXIMUM DESIGN AIR TEMPERATURE ISOTHERMS (DMAT+ 2σDMAT)

Note: See Section 4.1 for Definitions of DMAT and σDMAT

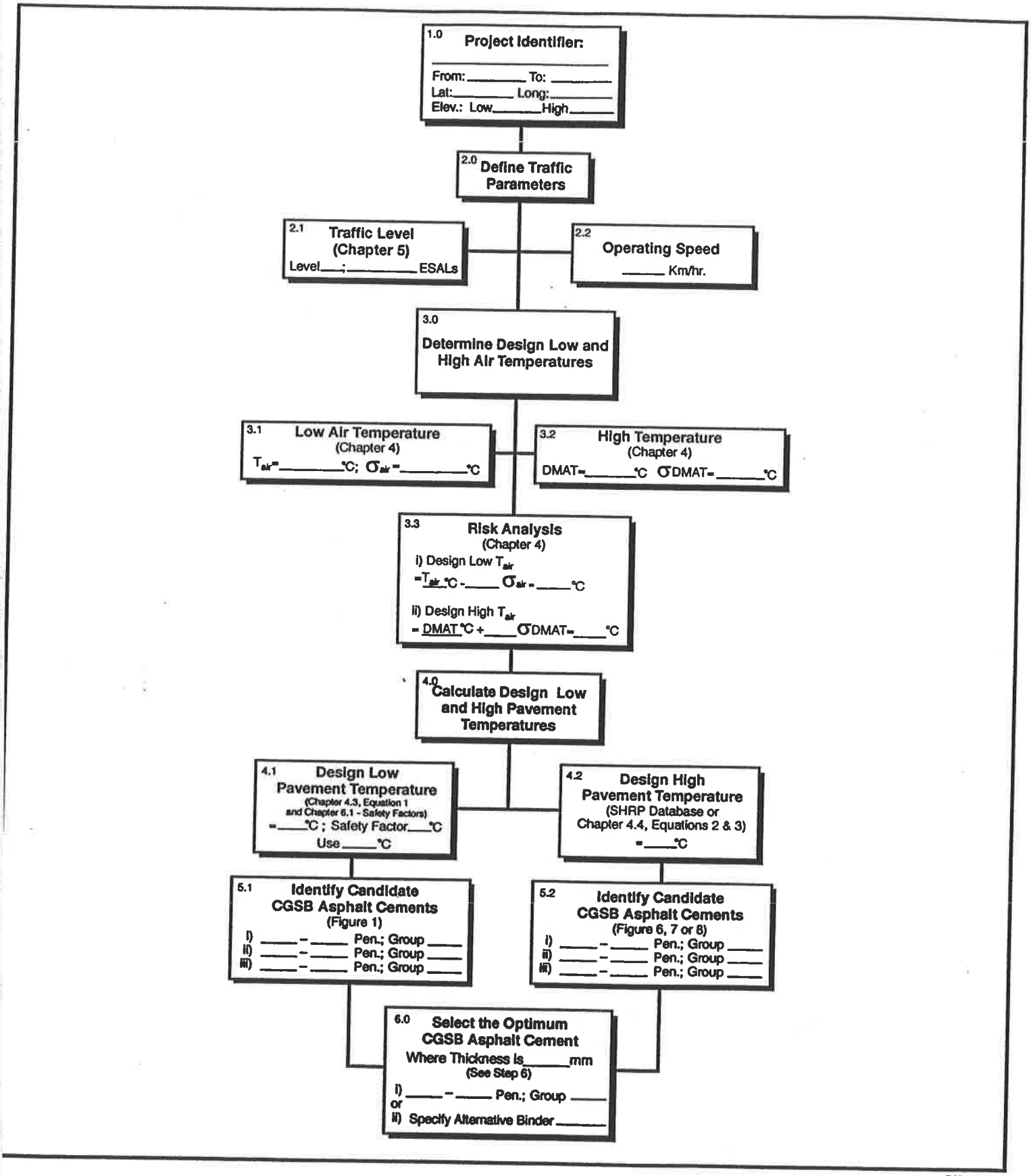


FIGURE 12 FLOW CHART FOR SELECTION OF ASPHALT BINDER