Public Works and Government Services Canada
Travaux publics et Services Gouvernementaux Canada

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December 8, 1997

Subject: **Consultant Report: "SHRP PG Binder Specification for Airports"**

The attached report is the result of research conducted in 1996/97 by EBA Engineering Consultants Ltd. on behalf of the Airport Engineering Division of Public Works and Government Services Canada (PWGSC). This research was part of a PWGSC R&D project titled, "SHRP PG Binder and Superpave Level 1 Mix Design Investigation, Laboratory Testing and Analysis".

The goal of this research was to evaluate the new SUPERPAVE Performance Graded (PG) asphalt binder specifications and associated test methods, that were developed specifically for highways by the U.S. Strategic Highway Research Program, to verify whether they are applicable for the unique technical requirements of Canadian airport pavements.

Your feedback, questions or comments related to this report would be appreciated and should be sent directly to me at PWGSC.

Also, to allow us to maintain a record of the distribution of this document, please DO NOT DISTRIBUTE COPIES of this report. Please direct those requesting copies of this report to contact PWGSC for a copy.

Sincerely,

Original Signed by

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attach.

Public Works and Government Services Canada
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AIRPORT ENGINEERING

ATR-027

SHRP PG Binder Specification for Airports

R&D PROJECT
SHRP PG Binder and Superpave Level 1
Mix Design Investigation, Laboratory
Testing and Analysis
Part 1

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Ottawa

March 1997

Canada

SHRP PG Binder and Superpave Level I Mix Design Investigation, Laboratory Testing and Analysis

PART I
SHRP PG Binder Specification For Airports

FINAL REPORT

Public Works
and
Government Services Canada

Submitted by

EBA Engineering Consultants Ltd.

March 1997

104-97-21155.01

Executive Summary

Public Works and Government Services Canada (PWGSC) commissioned EBA Engineering Consultants Ltd. (EBA) to undertake the project entitled "SHRP PG Binder and SUPERPAVE Level 1 Mix Design Investigation, Laboratory Testing and Analysis". This project is organized in two parts. The first part, contained in this report, is a review of the SHRP PG binder specification specifically as it relates to airport pavements in Canada. The second part of the project is a laboratory study comparing asphalt mixes designed using the Marshall method of design and the Superpave gyratory method. The second part of the project is documented in a separate report.

This report provides an overview of the CGSB asphalt specifications and the SHRP PG binder specifications. Airport specific issues are discussed with respect to the asphalt grades/specifications currently used for airport construction in comparison with the SHRP PG binder specification.

The review identified several major issues which affect how PWGSC might implement the SHRP PG binder specification in the foreseeable future, as well as issues that warrant further research to better address airport specific conditions. The following highlights the main conclusions of the review:

1. The SHRP PG binder specification can be adopted for airport construction to provide a similar level of service as is currently achieved with CGSB graded asphalts by assuring that SHRP grades are specified that have similar high and low temperature characteristics as the CGSB grades currently used.

2. The difference between airport pavements and highway pavements influences the application of the SHRP PG binder specification to airport

pavements. However, further research and evaluation is required to establish the significance of the influence.

3. Test parameters in the SHRP PG binder specification which address the pavement performance at high temperatures were not developed for aircraft loads. There is a need to evaluate and define appropriate G*/Sin(δ) limits for airport pavements.

4. Low pavement temperatures may be affected by thick granular layers and therefore low temperature algorithms should be evaluated for airport pavements to assure accurate estimates of pavement temperature are established. (Current low temperature algorithms are based on the work of Canadian researchers and are not a part of the current SHRP PG binder specification).

Overall, the review highlights that the development of the SHRP PG binder specification did not specifically address aircraft loads and therefore the limiting criteria for high in-service temperatures may not be appropriate for airport pavements.

In addition, there are more minor issues such as loss on heating and solubility in trichloroethylene which need to be addressed as agencies begin to implement SHRP PG binder specifications.

However, while there are a number of parameters which need to be better defined for various loading conditions, the governing philosophy of the SHRP PG binder specifications is considered a valid approach for selecting asphalts to achieve performance at both high and low in-service temperatures for airport pavements. Enhancements and refinements to the SHRP PG binder specifications are expected to evolve well into the next decade.

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1. Introduction

Public Works and Government Services Canada (PWGSC) commissioned EBA Engineering Consultants Ltd. (EBA) to undertake the project entitled "SHRP PG Binder and SUPERPAVE Level 1 Mix Design Investigation, Laboratory Testing and Analysis". This project is organized in two parts. The first part is a review of the SHRP PG binder specification specifically as it relates to airport pavements in Canada. The second part of the project is a laboratory study comparing asphalt mixes designed using the Marshall method of design and the Superpave gyratory method.

This report documents the first part of the project dealing with the SHRP PG binder specification as it relates to airport pavements constructed in Canada.

1.1 Background

SHRP is an acronym for the **S**trategic **H**ighway **R**esearch **P**rogram which was an American initiative to examine all aspects of highway construction and maintenance over a 5 year period with a budget of $150 million U.S. ($50 million of which was devoted to asphalt products). As a part of the SHRP, research specifically related to asphalt binders and asphalt paving mixtures was undertaken with the objective to establish performance related asphalt binder specifications, a volumetric mix design method and models for the performance related design and evaluation of candidate paving mixtures.

In conjunction with the U.S. research, other countries participated by carrying out complementary research and constructing test pavements to be included in the SHRP Long Term Pavement Performance (LTPP) studies. Canada was significantly involved in SHRP through the various provincial highway agencies and provided considerable resources directly to SHRP and as a part of the complementary Canadian-SHRP (C-SHRP).

The products of the asphalt research were asphalt binder specifications that are based on fundamental engineering properties of the asphalt binder, and a mix design methodology that was expected to better mimic the actual in-place pavement characteristics. The combined output was termed Superpave as an acronym for **Su**perior **Per**forming Asphalt **Pave**ments.

Superpave is considered a system which is both the binder specification and mix design. The binder specification is based on environmental inputs in the form of high and low temperatures, while the mix design methodology is based on traffic loadings in the form of cumulative equivalent single axle loads (ESALs) and high temperature considerations.

1.2 Objectives

PWGSC identified a need to examine the SHRP PG binder specifications relative to airport pavements. This need was identified recognizing that the SHRP asphalt specifications were developed with consideration of highway conditions and the potential that the adoption of the SHRP PG specification by highway agencies across Canada may result in the SHRP specification being the new Canadian standard and thus asphalts complying with these specifications may become the only readily available asphalt products.

The objective of this study/review, is to examine the SHRP PG binder specifications and assess their suitability for the construction of pavements for Canadian airport sites and to provide recommendations for any deviations from the specifications that would be appropriate in consideration of the unique conditions of airport pavements.

The project Terms of Reference stated the following objectives:

1. Identification of the parameters within the SHRP Binder specification and associated test methods which may be influenced by the differences between airport and highway pavements and determine the degree of influence.
2. Recommendation of possible adaptations to the SHRP binder specifications for Canadian airport pavements.
3. Recommendation of possible adaptations to the SHRP binder test methods for Canadian airport pavements.
4. Recommendation for a method of selecting PG Binder to take into account the requirements of airport pavements.

This report presents the background to the development of the SHRP PG binder specifications and the CGSB Asphalt Cement for Road Purposes specifications to provide a background for a discussion on issues relative to airport pavements and the potential significance of using the SHRP PG binder specification for airport construction in Canada. The Discussion section then examines the differences between airport pavements in-service conditions and those of a highway pavement and attempts to provide insight as to the potential impact such differences may have on the selection of a SHRP PG binder grade for airport construction in Canada.

2. Complementary Study

It is worth noting that the Federal Aviation Administration (FAA) of the U.S. Department of Transportation has commissioned the U.S. Army Corp of Engineers to evaluate the SHRP specifications relative to their application to airport pavements. Dr. Kent Newman of the U.S. Waterways Experimental Station in Vicksburg Mississippi is involved in that study. Discussions held with Dr. Newman indicate that the study scope involved the review of the SHRP binder specification and recommendations as to the adoption of the SHRP specification relative to FAA guidelines.

While there are differences between US and Canadian practice, most notably the CGSB penetration-viscosity based asphalt specification and the U.S. tendency to design leaner mixtures, there are concerns which would be common. Common concerns would include the structural design on the basis of gross aircraft weight and wheel gear configuration compared to Equivalent Single Axle Loads (ESALs) that are utilized in highway design applications. (Although it is recognized that there are also significant differences between the PWGSC and FAA design procedures).

The Corp's report has been completed and provided to the FAA but is not currently available. According to Dr. Newman, the report recommends adopting the current SHRP PG binder specification with the provision to allow for field performance evaluation of binders and the potential for the adjustment of some of the specification requirements relative to airport use.

3. Pavement Performance

Prior to discussing the current asphalt specifications and those produced as a part of SHRP, it is prudent to discuss pavement performance in general terms relative to the influence of the asphalt binder on performance. Pavement performance is influenced by various distresses which are in-turn influenced to some degree by the asphalt binder properties. These distresses can be divided into the following groups:

- thermal cracking
- fatigue cracking
- rutting

Other pavement performance issues such as raveling, bleeding, and skid resistance are not considered a function of the asphalt binder per se but are affected more by the mix design, aggregate characteristics and construction practice.

The three distress groups noted are all impacted by stiffness of the asphalt concrete mix, which is affected by the binder stiffness.

Asphalt stiffness, simply stated, is analogous to the elastic modulus of other engineering materials. However, because asphalt is a viscoelastic material, the stiffness is affected by both the temperature and the total time for which the load is applied. At high temperatures and/or long loading times, the binder stiffness is lower than at low temperatures and/or short loading times.

Because of the role that the asphalt binder plays in pavement performance, it is important that an asphalt specification address the stiffness to provide for:

- Binder which is sufficiently stiff at high service temperatures to resist rutting
- Binder with sufficiently low stiffness at low service temperatures to resist thermal cracking, and
- Binder that is resistant to fatigue cracking. The relationship between asphalt fatigue and the presently measured parameters of the asphalt binder is not fully understood.

Sections 4 and 5 review the principles in the SHRP PG binder specifications and the CGSB Specification and discuss how each specification addresses the three distress groups, namely thermal cracking, fatigue cracking and rutting.

4. SHRP PG Binder Specifications

This section is intended to provide a brief background to the development of the SHRP PG binder specification and to provide some insight on the issues surrounding its development. It is recognized that at the present time the SHRP PG binder specification is being reviewed by various researchers and it is anticipated that some changes will occur in the future. LTPP sites are also expected to identify necessary changes to the specification well into the next decade. As the significance of such changes is not known at this time this project does not attempt to address future changes; however, where issues with specific parameters are known, this is noted in the following sections.

The U.S. Federal highway Administration (FHWA) is responsible for the future development and implementation of SHRP (in the U.S.) Future changes are being coordinated through the use of expert task groups (ETG) made up of experts in the various aspects of SHRP. Changes and reviews are ongoing and it is expect that refinements will be introduced as they become proven.

4.1 An Overview Of The SHRP PG Binder Specifications

Simply stated, the SHRP PG binder specification attempts to use parameters which are performance related and relates these parameters to service temperatures to which binders will be exposed during their service life. The binder specification takes the form of PGXX-YY were XX is the highest seven day average pavement temperature and -YY is the lowest pavement temperature to which the pavement will be exposed. For example, a PG58-28 would meet the SHRP specification for a design high temperature up to 58°C and design low temperature of -28°C.

The selection of the design temperatures, which govern the grade selection, and the SHRP testing protocols which are utilized to define an asphalt's SHRP PG grading are discussed in Section *4.4 SHRP PG Binder Specification Temperatures*.

4.2 SHRP "Conventional" Testing - Viscosity, Flash Point, & RTFOT Loss

The SHRP PG binder specifications contain a number of requirements which were adopted from conventional asphalt specifications. These include the viscosity at 135°C, loss on heating using the Rolling Thin Film Oven Test (RTFOT), and Flash Point. RTFOT is also used to simulate property changes during pavement construction.

4.2.1 Viscosity

A maximum viscosity specification is included in the SHRP PG binder specification to address handling (pumpability). Although the SHRP PG binder specifications require the viscosity to be determined using a rotational viscometer, the intent with respect to material handling is similar to conventional requirements. The SHRP specification (AASHTO specification MP-1) allows this specification requirement to be waived at the discretion of the user on the basis of the supplier warranting that the asphalt can be pumped and mixed at the appropriate temperatures.

The AASHTO MP-1 specification also allows for viscosity determination in lieu of dynamic shear measurements ($G^*/\mathrm{Sin}\delta$) for unmodified asphalts at temperatures where they behave as Newtonian fluids. This is discussed further in Section *4.7 High Temperature Testing - Complex Shear Modulus*.

4.2.2 Rolling Thin Film Oven Test

This specification requirement, that limits the mass loss during the RTFOT simulated aging, is used to control the use of fluxes or volatile materials that can degrade the quality of an asphalt cement. The SHRP researchers adopted the

Rolling Thin Film Oven Test (RTFOT) as the standard test for simulating initial aging during the asphalt mixing process [1]. It was recognized that both the Thin Film Oven Test (TFOT) and RTFOT were potential candidates. However, in recognizing the level of effort that would have been required to correlate the two procedures, the decision was made to utilize only the RTFOT. This decision considered that the RTFOT was a faster test than the TFOT, that it is preferable for polymer modified asphalts and that there is less between-laboratory variability than for the TFOT. In Canada, the TFOT has been more widely used than the RTFOT and therefore many laboratories are not equipped to perform the RTFOT. Industry wide resolution of this situation needs to be undertaken.

The RTFOT is considered a more severe test than the TFOT and Western Canadian heavy crude sources are impacted by this specification as they tend to be refined at lower temperatures than lighter crudes. As a result the mass loss may become an issue, especially on the softer grades. Alberta Transportation and Utilities undertook some comparative testing utilizing both RTFOT and TFOT for grading Alberta asphalts to the SHRP specifications. The use of the RTFOT made a difference in the resulting SHRP grading in only one of the five grades tested.

Conventional penetration testing (@25°C, 100g,5s) indicated that the RTFOT had varying affects when conducted on varying asphalt grades from the same supplier. Table 1 illustrates the significance of the RTFOT relative to the TFOT in terms of penetration.

Table 1 Penetration Properties After RTFOT and TFOT

	Penetration (25°C, 100g, 5s)		
Grade	Original	After RTFOT	After TFOT
85-100A	86	46	55
120-150A	128	71	74
150-200A	164	81	91
200-300A	229	115	120
300-400A	319	147	156

* Penetration in 0.1mm units, at 25°C, 100g, 5s.

4.2.3 Flash Point

The standard AASHTO T48 Method of Flash and Fire Points by Cleveland Open Cup has been retained for safety purposes although all grades have the same 230°C requirement. For comparison, the AT&U specification requirements vary from 205°C for a 150-200A grade asphalt to 175°C for softer grades. Requirements of CGSB CAN/CGSB-16.3-M90 are discussed in Section *5 Conventional (CGSB) Specifications*.

4.2.4 Other Considerations

During the 1993 AAPT meeting, Dr. T. Kennedy presented the SHRP PG binder specifications to the delegates. Discussion from the floor raised the issue of the check for “purity” of an asphalt. The current SHRP PG binder specification has no requirement for solubility in trichloroethelyne which may potentially allow for a supplier to use fillers to adjust consistency. This is currently being addressed by ASTM. Retention of a solubility requirement, for Canadian applications, is considered desirable.

4.3 Laboratory Long Term Aging of Asphalt Binders

As discussed in Section *4.2.2 Rolling Thin Film Oven Test,* the SHRP PG Specification adopted the RTFOT to simulate initial plant aging. SHRP research was targeted at developing a laboratory test procedure to simulate long term aging. The procedure adopted is simulated long term aging of binder in the Pressure Aging Vessel (PAV) which built on work of previous researchers(2). Samples aged in the PAV are first aged in the RTFOT to simulate the construction related aging. The PAV utilizes standard TFOT pans to hold a 50 gram asphalt sample, the PAV places the sample under 2.1 MPa pressure (in industrial air) for 20 hours. The sample is maintained at a constant temperature of 90°, 100°, or 110°C, depending on the high temperature regime for the asphalt. As the asphalt high temperature selection is based on the prevailing ambient temperatures at the proposed construction site, the PAV temperature is set based on the high temperature grade of asphalt binder.

The SHRP research project A-002A(3) identified and evaluated the PAV and conducted some validation testing on in-place field samples. The validation testing indicated that PAV aging may represent aging of 4-8 years, in a highway pavement. Their validation process highlighted the significance of the temperature at which the aging process was conducted at. The originally proposed procedure by SHRP used lower temperatures and longer test times. Now, because of the practicality of conducting long term tests, temperatures have been raised and testing times shortened. This, unfortunately, may lead to further deviation of the binder's chemical changes during the test, relative to those occurring during the service life. This validation utilized asphalts significantly harder than those typically used for airport construction in Canada.

In-place test results from recycling work conducted by Alberta Transportation and Utilities(4) can be used to provide some insight into the relative aging of Alberta asphalts using the PAV. These comparisons do not follow a given asphalt through its in-place aging. However, there is sufficient historical data to make some important observations. As well, it should be noted that aging during the mixing stage may, in fact, govern the overall long term aging of the asphalt.

Data from abson recovered asphalts tested for the design of some of the Alberta[(4)] recycling projects is presented in Figure 4-1.

Table 1 can be expanded to include PAV aging results for asphalts historically used by Alberta Transportation and Utilities as shown in Table 2.

Table 2 PAV Aging of Asphalt Binders

	Penetration (25°C, 100g, 5s)		
Grade	Original	After RTFOT	PAV Aged
85-100A	86	46	23
120-150A	128	71	36
150-200A	164	81	38
200-300A	229	115	65
300-400A	319	147	81

On the basis of the limited Alberta data[(4)], the PAV aging would appear to represent over 10 years aging. This is shown graphically in Figure 4-1. The aging can be seen to occur at a significantly diminishing rate after the plant aging as simulated by the RTFOT. In other words, the consistency of the binder does not appear to change significantly with time as compared to the initial aging experienced during mixing. This also suggests that the 4-8 years originally noted by SHRP researchers may be consistent with Alberta experience.

The SHRP researchers concluded that the amount of aging in the PAV was sensitive to pressure and temperature. However, it must be recognized that the aging which occurs in a mixing plant is also a significant variable; as a result it is concluded that at the present time the PAV aging is a reasonable simulation of long term aging.

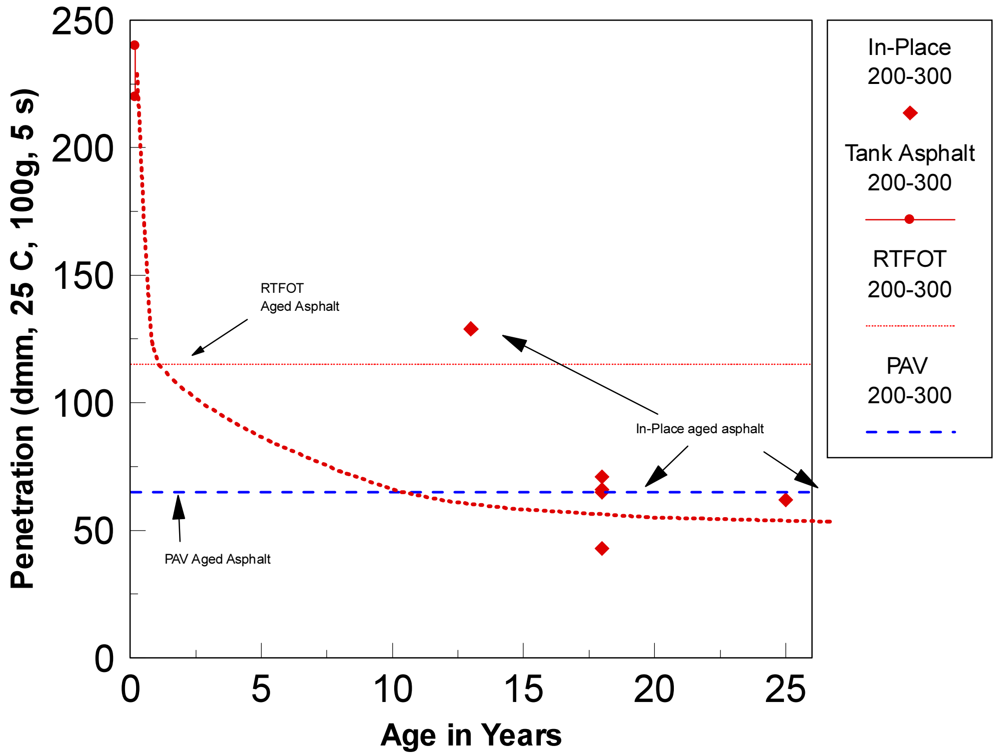

Figure 4-1 Comparison of PAV to In-Place Aging

4.4 SHRP PG Binder Specification Temperatures

The first step in selecting a SHRP PG binder grade is the determination of the pavement design temperatures for the proposed construction site. The procedure is well documented in the Asphalt Institutes SP-1[5]. The following highlights the process and some of the issues related to pavement design temperature determination.

- SHRP has developed a temperature database which is based on over 7500 weather stations in Canada and the United States. (over 1800 in Canada).
- The SHRP weather database will calculate the high pavement design temperature at a depth 20mm below the pavement surface based on the following relationship:

$$T_{20mm}=(T_{air} - 0.00618\ Lat^2 + 0.2289Lat + 42.2)(0.9545)-17.78$$

where

T_{20mm} = high pavement design temperature at a depth of 20mm

T_{air} = seven-day average high air temperature

Lat = the projects location in degrees latitude

- The design low temperature is currently a matter of debate.

SHRP originally identified that the pavement low temperature should be equal to the low air temperature. Canadian researchers[6,7] identified to SHRP that pavement temperatures are generally warmer than the air temperatures and therefore the original SHRP protocol is overly conservative in the selection of the low temperature for a SHRP PG binder. As recently as January 1997 the FHWA Expert Task Group (ETG) elected to not adopt the Canadian model for low temperatures and instead propose that the binder low temperature selection be based on the lower 50% reliability for air temperatures below -28°C. That is, use the average low air temperature as the pavement temperature for the selection of the binder grade. In February of 1997 the ETG reportedly accepted a proposed model developed from LTPP sections which is supposedly intended to be valid for high and low temperatures.

Ironically, the Asphalt Institute's SP-1 manual included a low temperature algorithm with the expectation that this would be adopted by SHRP. The algorithm was originally developed by Deme[7] based on data from the St. Anne test road and was restated by Robertson. The algorithm, as included in the SP-1 manual is:

$T_{surf} = 0.859T_{air} + 1.7$ where T_{air} = 1-day minimum air temperature

- Recent work by Robertson and Christison[8,9] has given a number of equations and incorporated standard deviation and reliability within the equation

Recent work by EBA[8] evaluated the available Canadian models and recommended that models utilizing reliability be used. This is accomplished by replacing the T_{air} term in the T_{surf} equation with the term (T_{air} - $n\sigma$) where σ is the standard deviation and n is dependent on the desired reliability as shown in Figure 4-2.

Substituting the reliability term provides the following form of the equation:
$T_{surf} = 0.859(T_{air}-n\sigma_{air})+ 1.7$ where T_{air} = 1-day minimum air temperature

Robertson's most recent work[9] introduces an additional term intended to account for the standard deviation of the actual *pavement* temperature. Robertson suggests that the probability or reliability then becomes a function of the two reliabilities associated with the two temperatures (ie: the air temperature and the pavement temperature). The most recent algorithm predicts pavement temperatures several degrees warmer than when only $n\sigma_{air}$ is used as in the above equation.

The most recent equation is as follows:

$$T_{surf} = 0.749(T_{air} - n\sigma_{air}) -1.5n$$

This equation form uses the assumed value of 1.5 as σ_{surf}. The n multiplier is related to the reliability based on the product of the 'risk' (ie: 1 - reliability) of the event happening. For example, 1σ corresponds to a 84.1% reliability that the specified temperature will not be exceeded which is then a 15.9% risk that it will. The combined risk is the product of the two (0.159 * 0.159 = 2.5%), and therefore the new reliability associated with 1σ is 97.5%.

This equation is not considered to have been sufficiently debated to allow any conclusions regarding its use for binder selection at this time; as for the other low temperature equations discussed, the basis of this most recent work is measured temperatures in highway pavements.

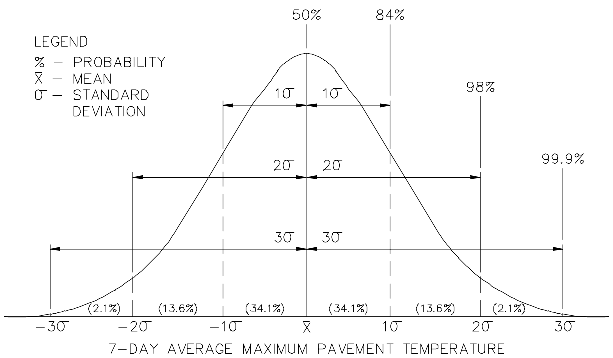

Figure 4-2 Reliability Concept For Design Temperature

Regardless of how the pavement low temperature is calculated the Superpave system incorporates this concept of reliability in the selection of design temperatures. The higher the reliability chosen the less likely the design temperature will be exceeded. It is considered more significant to maintain a high degree of reliability for low temperatures in Canada as pavement cracking may occur with a single occurrence of the low temperature whereas pavement deformation is affected both by the occurrence of the high temperatures and the number and magnitude of loads it is exposed to during that high temperature period. The concept of reliability is illustrated in Figure 4-2.

4.5 Low temperature Testing - Creep Stiffness

In order to characterize the stiffness of an asphalt binder at freezing temperatures, SHRP researchers developed the Bending Beam Rheometer (BBR) based on flexural beam theory to determine the binder stiffness at low temperatures. The BBR is shown schematically in Figure 4-3.

As asphalt is a viscoelastic material, its stiffness is a function of temperature and time of loading. At a given temperature and time of loading the stiffness is calculated as for a simple beam:

$$S(t) = \frac{PL^3}{4bh^3\delta(t)}$$ where: S(t)=Stiffness at time t

P = load in Newtons
L = distance between supports (=102mm)
b = width of beam sample (=12.5mm)
h = thickness of beam sample (=6.25 mm)
δ(t) = deflection in mm at time = t

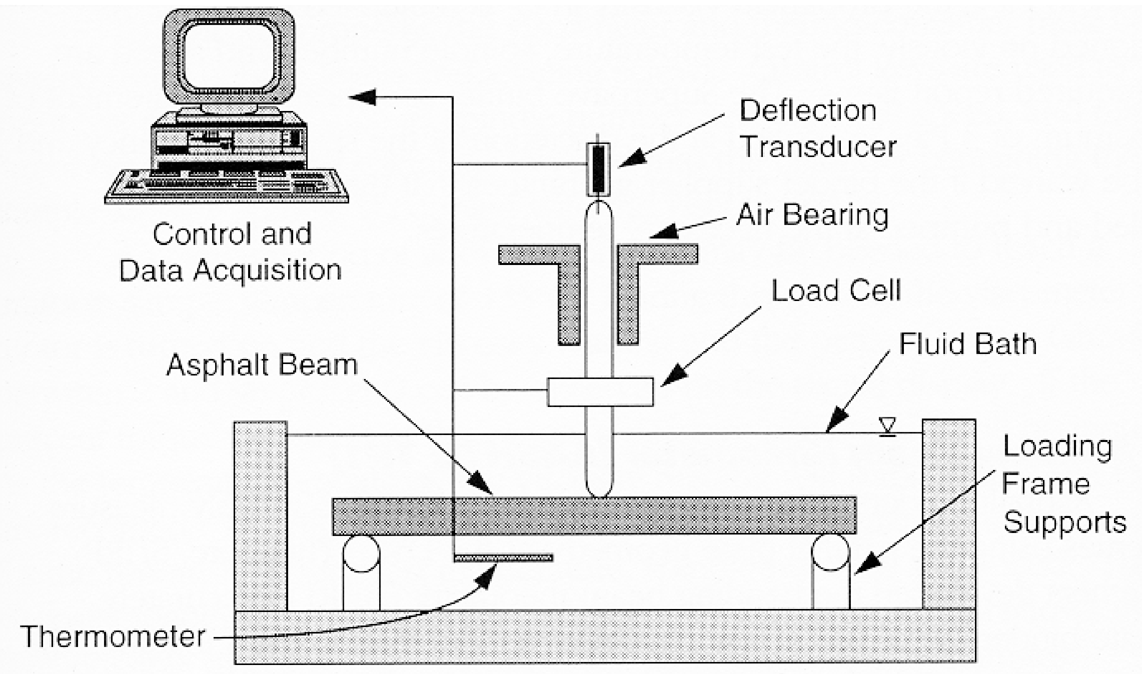

Figure 4-3 Schematic of BBR Test Apparatus

The concept of low temperature binder stiffness as a controlling factor in low temperature cracking has been studied by researchers for many decades. Many Canadian researchers identified critical stiffness values at which an asphalt pavement would crack if the stiffness was exceeded.

Previous researchers who studied low temperature cracking, identified that the loading time is related to the load induced due to thermal contraction as the asphalt pavement cooled. Loading times common in the literature varied from about 1 to 6 hours. Examples of critical stiffness values identified in the literature[(8)] include Readshaw[(10)] who identified a stiffness of 200 MPa at a loading time of 2 hours, Fromm and Phang who identified 140 MPa at a 10,000 second loading time (≈2.8 hours) and Palsat[(11)] with a value of 2.9 MPa at a loading time of 20,000 seconds (≈5.5 hours) and at a *pavement depth of 50mm.* Virtually all of this early work was based on calculated stiffness using Van Der Poel's nomograph with either Penetration Index or Pen-Vis numbers. The SHRP BBR development indicated that the correlation between calculated values and directly measured values was not good. BBR stiffness values are generally higher than those determined using the nomograph method.

Based on the SHRP researchers' review of such prior work, a critical stiffness value of 300MPa at a loading time of 2 hours was identified. Using simple curve shifting techniques, the testing time was reduced to 60 seconds at a temperature 10^0 C warmer than the actual temperature. The curve shifting process is illustrated in Figure 4-4. This can be expressed as:

$S_{(T=60s,t=X^0)} = S_{(T=7200\,\mathrm{sec},t=(X-10^0))}$. Recognizing that the stiffness of an aged asphalt is greater than an unaged asphalt, the SHRP PG binder specification requires that the BBR testing be conducted on samples aged using the PAV procedure.

The shifting factor used for binders was determined as an average value from factors determined for asphalts contained in the SHRP reference library. Unfortunately, there are indications that this factor does not apply to modified asphalts and that by transferring the 60s loading time to a 2 hour loading time (by shifting the temperature 10°C) an error is introduced in the results.

The concept of the critical stiffness prevalent in early literature and adopted by SHRP recognizes that strain to failure has been related to the binder stiffness. However, SHRP recognized that certain asphalts, especially modified asphalts, may exhibit strain tolerances (an ability to undergo increased strains) at values which exceed the critical stiffness value.

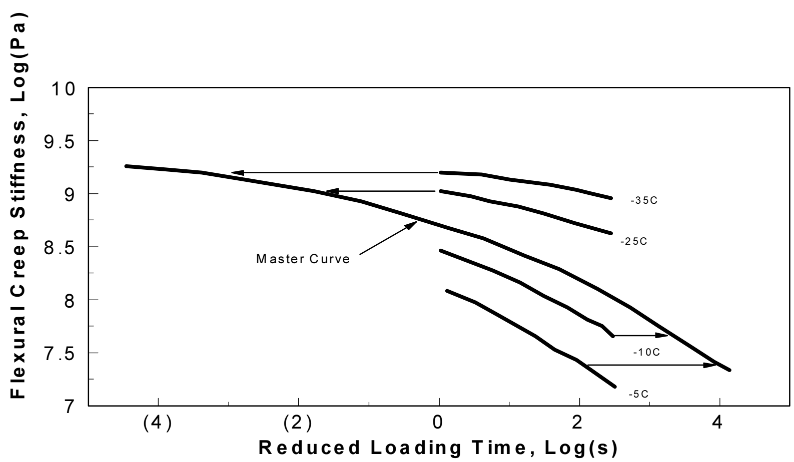

Figure 4-4 Example of Curve Shifting To Develop Master Curve

For asphalts that exceed the 300MPa allowable stiffness, but not 600 MPa, at the desired grading temperature (low design temperature - 10^0C) an 'optional' test for measuring the strain to failure was adopted. The Direct Tension Test measures the strain to failure and is discussed further in Section *4.6 Low Temperature Testing - Direct Tension Test*.

4.5.1 Low Temperature Testing - ‘m’ value

As discussed, the creep stiffness decreases with an increase in loading time. This time dependency is an important consideration in the development of thermal shrinkage stresses; this time dependency has been noted to vary widely for different asphalts. Because the ‘m’ value controls the shape of the stiffness mastercurve it is critical to assure that the slope does not exceed expected values.

Although it is an over-simplification, the m value can be thought of as a check to assure the assumptions related to the temperature shift, which allows the BBR test to completed in 60 seconds instead of two hours, hold true for a given asphalt.

It was reported by both Anderson(12) and Monismith(13) at the 1997 AAPT meeting that since the ‘m’ value controls the slope of the creep compliance curve the ‘m’ value may also include some control on the fatigue characteristics of the binder.

The ‘m’ value is a requirement for all asphalts, including those which exhibit improved strain tolerance in the Direct Tension Test. The minimum value in the current specification is 0.3; there are rumored discussions which suggest that SHRP community is debating this value and that a slightly smaller value may be more appropriate.

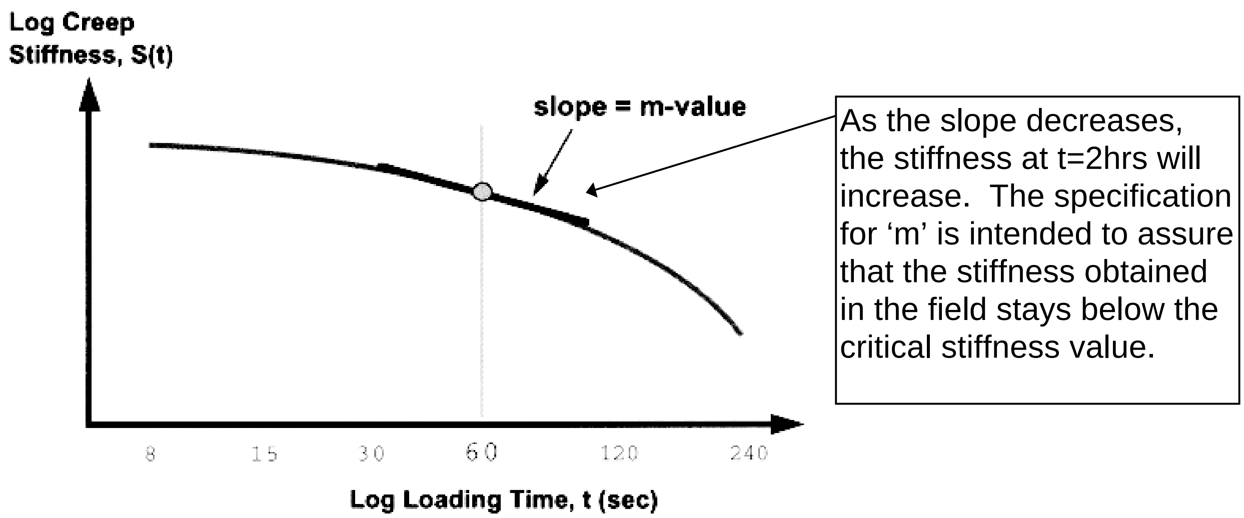

Figure 4-5 Slope 'm' Determined at t=60 seconds

4.6 Low Temperature Testing - Direct Tension Test

The Direct Tension Test (DTT) was introduced because the relationship between stiffness and tensile properties of asphalt binders is not well defined. The DTT is intended to address the issue of improved strain tolerance that might be characteristic of modified asphalts. Such strain improvement is shown in Figure 4-6.

The test method utilizes a formed binder specimen which is placed in tension and loaded at a constant rate until fracture. (The new version of the test works with a constant strain rate). The specification requires a minimum of 1% strain for the sample to ‘pass’. The asphalt stiffness as determined using the BBR must still in the range of 300 to 600 MPa for the binder to be considered acceptable at the desired temperature. The direct tension testing is conducted at the same test temperature as the BBR test.

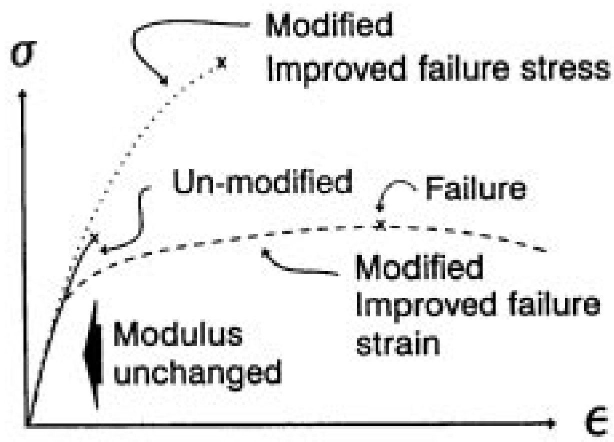

Figure 4-6 Potential Differences In Binder Behavior With Modification

At the present time, direct tension testing equipment is undergoing ruggedness testing by the FHWA. Initial DTT equipment experienced difficulty in accurately measuring the strain. Until such time as the ruggedness testing is completed it is not known how the DTT will be specified in the SHRP specification (although it is noted that the DTT is currently commercially available). It is known however, that the ETG is considering changing the loading rate from the current 1mm/min to 0.1mm/min. It is reported that a final specification for the Direct Tension Test is at least a year away.

4.7 High Temperature Testing - Complex Shear Modulus and Phase Angle

SHRP adapted an existing testing methodology to measure binder characteristics at medium to high temperatures. The Dynamic Shear Rheometer (DSR) is used to apply a rotational torque to a small asphalt binder sample. The DSR is shown schematically in Figure 4-7. The rate of loading selected by SHRP is 10 rad/s at a constant stress. A review of the literature indicates that a loading frequency of 10 radians/second ($t=1/\omega = 0.1$ sec) would be in the range of 5km/hour [14]. This rate is apparently being reviewed as it is considered too slow for highway traffic[12]. It is significant to note that a binder's characteristics at a given loading frequency does not allow the prediction to other frequencies, especially in the case of modified asphalts. Figure 4-8 illustrates the change in the complex shear modulus (|G*|) with loading frequency. It can be seen from the figure that G* might change by a factor of two if the higher loading frequency is used. The SHRP DSR does not allow for alternate testing frequencies.

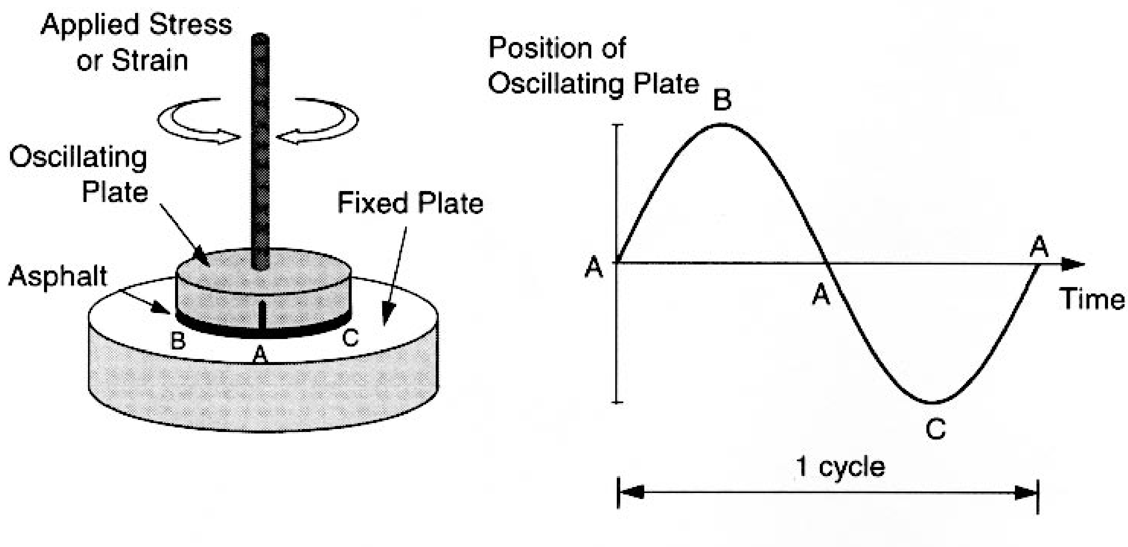

Figure 4-7 Schematic Representation of DSR Test Apparatus

In the DSR test, the sample response lags the applied load as a function of the material. This time lag is referred to as the phase angle (δ) which defines the viscous and elastic components of the overall complex shear modulus, G^*. The time lag concept is illustrated in Figure 4-9.

The specification currently is concerned with two parameters derived from the DSR testing: $G^*/\mathrm{Sin}(\delta)$ and $G^*\,\mathrm{Sin}(\delta)$.

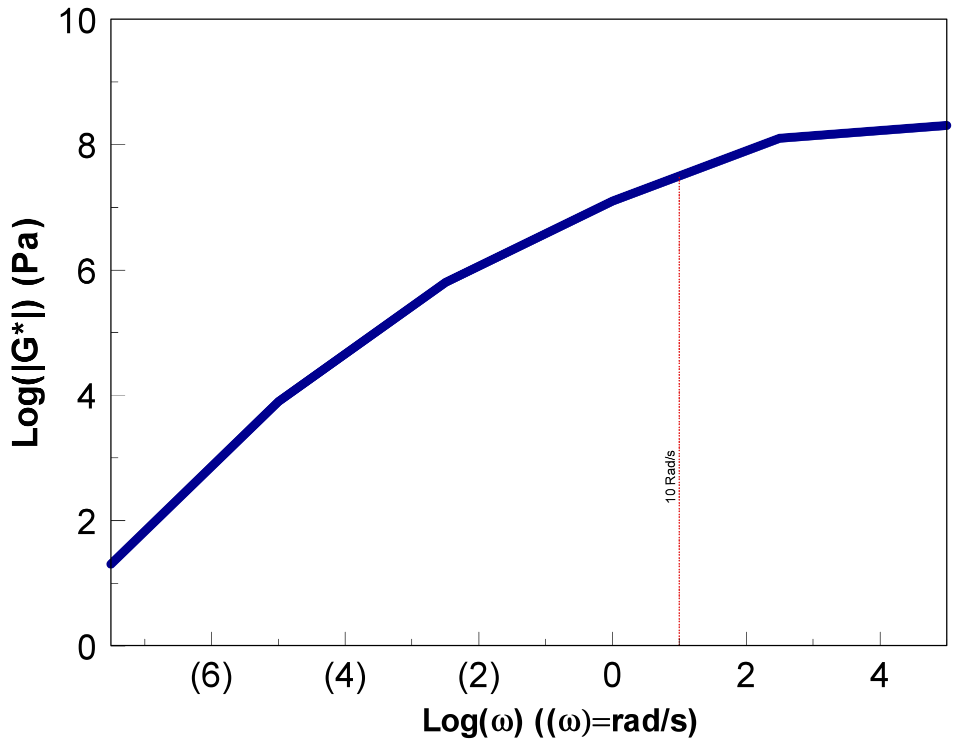

Figure 4-8 G* as a Function of Frequency

$G^*/\mathrm{Sin}(\delta)$ is determined on original (tank) asphalt and asphalt aged in the RTFOT as the critical parameter for the binder's influence on rutting. $G^*/\mathrm{Sin}(\delta)$ on the tank asphalt is more significant to provide a check on the potential change of binder properties during construction. As such, this parameter plays a similar role as the aging index in present specifications. One of the validation exercises used to confirm the appropriateness of this parameter was testing of mixes using the Hamburg Wheel Tracking Device.

G^* $\mathrm{Sin}(\delta)$ represents the viscous portion of the Complex Shear Modulus and was intended to be a significant factor in controlling load associated fatigue cracking. This parameter is currently being reviewed as recent work suggests that there is not a correlation between this parameter and fatigue performance. However, as the testing is conducted on PAV aged asphalt it also serves the purpose of identifying excessive age hardening of an asphalt.

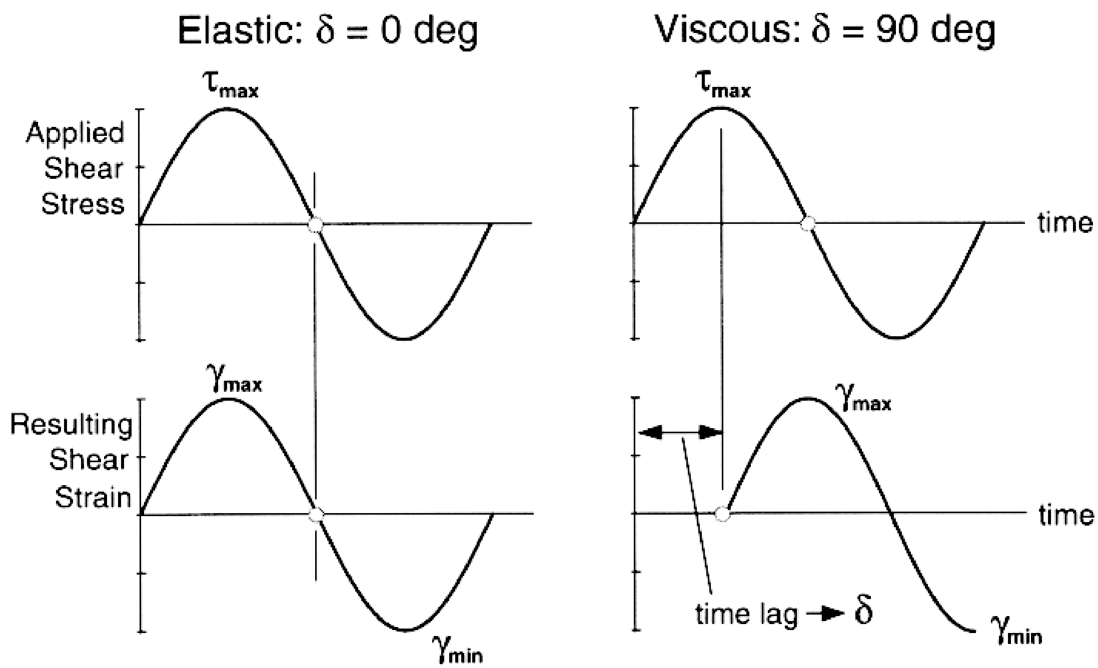

Figure 4-9 Time Lag (=δ) for Elastic and Viscous Behavior

4.7.1 Relating Complex Shear Modulus to Creep Stiffness

It is considered worthwhile to note that the complex shear modulus can be related to the creep stiffness measurements based on approximate relationships using linear viscoelastic theory.

In the linear viscoelastic region the relationship between corresponding components of stress tensor and deformation is a linear one and can be expressed as $G = \frac{\tau}{\gamma}$ where G is the modulus, τ the stress tensor and γ the infinitesimal strain tensor. When large loads are applied it is likely that the linear model is no longer valid and a much more complex non-linear model is required. This may be especially true for large aircraft at long loading times. However, the

SHRP researchers have concluded that the linear model for viscoelasticity is 'close enough' to provide meaningful results while recognizing that it is an approximation.

The complex shear modulus can be related to creep stiffness by the approximate formula: $S(t) = 2(1+\mu)G^{*}(\omega)$ where μ is Poissons ratio and is assumed to be ≈ 0.5 The expression then becomes - $S(t) = 3G^{*}(\omega)$, where ω is the test frequency ($t \rightarrow 1/\omega$). This expression is useful to examine the relation between the high and low temperature measurements in a manner that illustrates the temperature susceptibility of the binder. However, in order to compare the Complex Shear Modulus to Creep Stiffness at 60 seconds loading, a further shift is required. The time of loading can be adjusted for in terms of temperature, (in a manner similar to the BBR testing at 60 s to represent 2 hour loading at a 10°C lower temperature), by the following approximate relationship[3]:

$$T_2 = \left\{(2.303 * R * [Log(\frac{t_2}{t_1})]) / 250{,}000 + 1/(T_1 - 273)\right\}^{-1} - 273$$

where: T_2 = the adjusted temperature
R = ideal gas constant, 8.31
t_1 = actual loading time
t_2 = desired loading time
T_1 = actual test temperature

This shifting is utilized in later discussion to help present comparisons of asphalts graded to the SHRP PG binder specifications. It can be noted that while the shifting discussed herein specifically addressed G*, it is acceptably accurate to use for the parameters G*Sin(δ) and G*/Sin(δ) when $\delta \rightarrow 90°$ which is generally true for conventional asphalts[15].

5. Conventional (CGSB) Specifications

The CGSB asphalt specification (CAN/CGSB-16.3-M90 Asphalt Cement For Road Purposes) is a Canadian developed asphalt specification which was formulated over a ten year period by the CGSB Committee on Road Materials.

The CGSB specification is based on the concept of temperature susceptibility and as such is intended to be performance based. The asphalt grades are defined by their penetration range and an A, B, or C, depending in which group they are in. The groups or levels of the CGSB specification relate to the asphalt's viscosity and reflect different temperature susceptibilities and therefore lead to differing performance characteristics. Figure 5-1 and Figure 5-2 show the penetration viscosity relation for each grade for absolute and kinematic viscosities respectively. In addition to the penetration and viscosity requirements, the flash point, penetration of residue as a percentage of the original penetration, solubility in trichloroethylene, and mass loss by Thin Film Oven Test are also CGSB asphalt specifications. These specifications are shown in Table 3 for asphalt grades used for airport construction in Canada(16).

It can be noted that the Flash Point for the two softer grades used by PWGSC are lower than the 230°C required under the SHRP PG binder specification.

Table 3 CGSB Specifications for Asphalt Grades Used For Airport Pavements

Grades	80-100		120-150		150-200	
Requirements	**Min.**	**Max.**	**Min.**	**Max.**	**Min.**	**Max.**
Penetration 0.1mm at 25°C, 100g, 5s	80	100	120	150	150	200
Viscosity@60°C,Pa.s or Viscosity@135°C,mm^2/s						
*Group A *Group B *Group C	User must specify either Absolute or Kinematic Viscosity for all Asphalt Grades (Such as Figure 5-1) (Both Viscosities shall not be used simultaneously)					
Flash Point (Cleveland Open Cup), °C	230	---	220	---	220	---
Thin Film Oven Test, % loss in mass	---	0.85	---	1.3	---	1.3
Penetration of Residue at 25°C,100g, 5s, 0.1mm, % of original penetration	47	----	42	---	40	---
Solubility in trichloro ethylene, % by mass	99.0	----	99.0	----	99.0	---

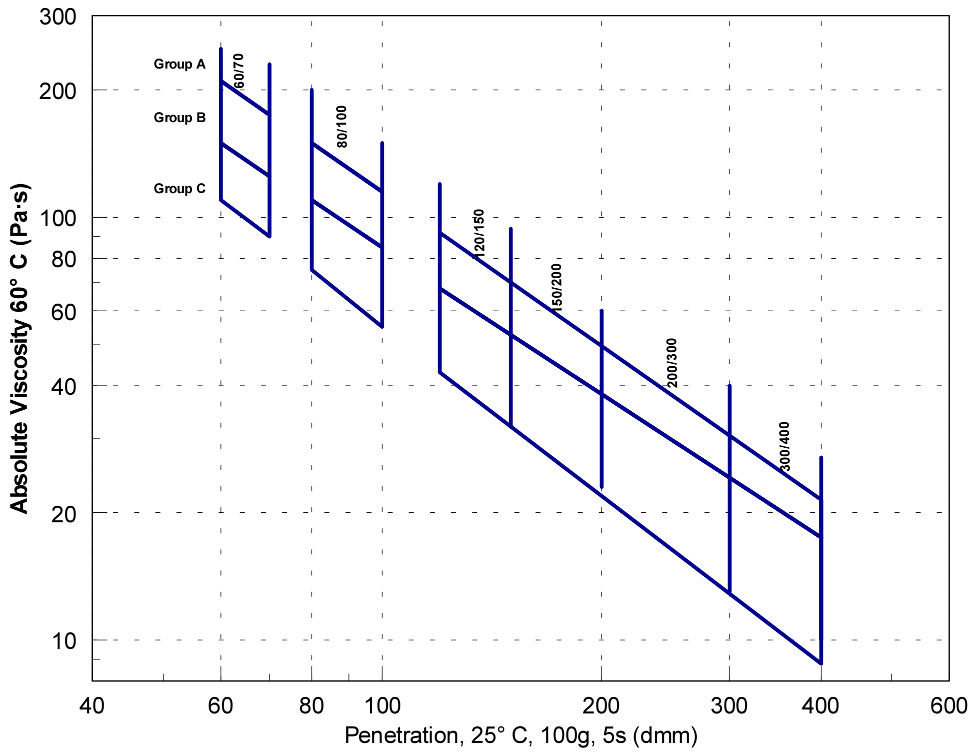

Figure 5-1 CGSB Specification for Absolute Viscosity

The concept of temperature susceptibility is addressed within the CGSB specification by the concept of Penetration Index (PI). The basic concept relates to the change in an asphalt binder's consistency with temperature. This concept is illustrated in Figure 5-3 where two asphalts of different temperature susceptibilities are shown graphically. The PI determined has been related directly to calculated binder stiffness and thus the penetration and viscosity of an asphalt can be used directly to estimate its stiffness for a given temperature and time of loading[8]. This approach allows for the selection of CGSB asphalts on the basis of their calculated stiffness. The limiting stiffnesses used for the CGSB asphalts are based on the work of Canadian researchers, some of whom were used as references in the development of the SHRP specifications.

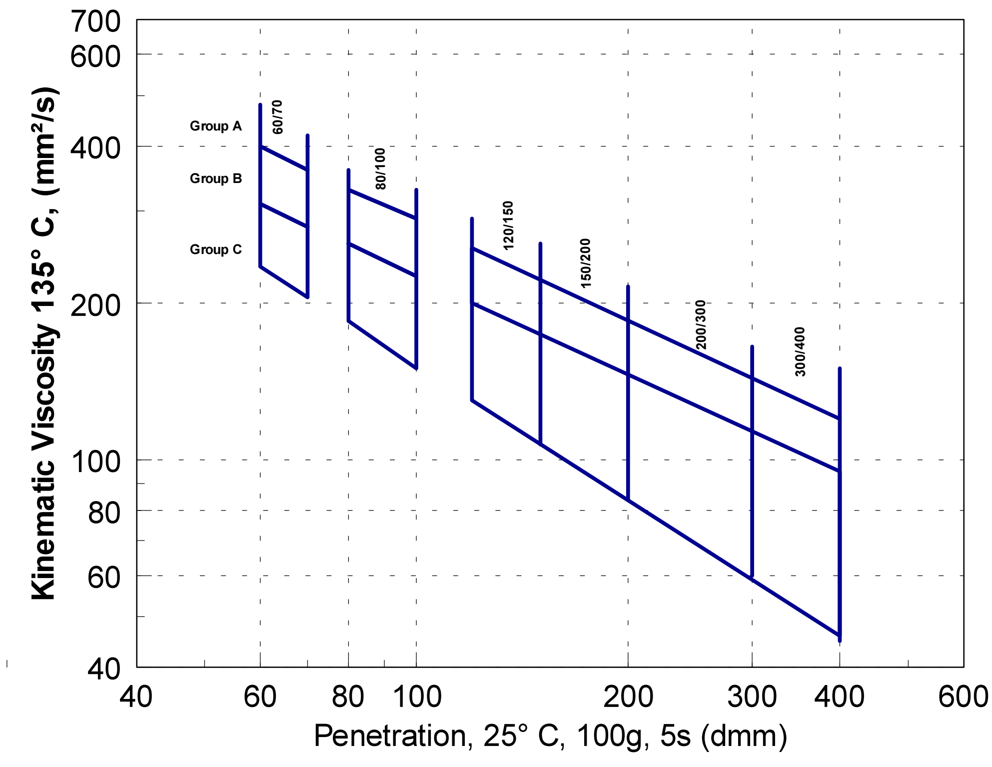

Figure 5-2 CGSB Specification for Kinematic Viscosity

The application of the CGSB specification is similar in concept to the SHRP PG binder specification (if not as intuitively obvious) in that performance of the binder must be considered for both high and low temperatures. The high temperature testing is to predict the performance of the binder relative to potential permanent deformation (rutting) while the low temperature regime is to guard against transverse cracking.

EBA has recently completed a User Guide for CGSB graded asphalts which was commissioned by the Transportation Association of Canada (TAC). The steps for selecting an asphalt grade for roadway construction are detailed in the TAC User Guide for the purpose of assisting the user in the selection of the appropriate CGSB grade.

The CGSB User Guide makes use of several input factors including high and low design temperatures, design ESALs and limiting stiffnesses for the high and low design temperature.

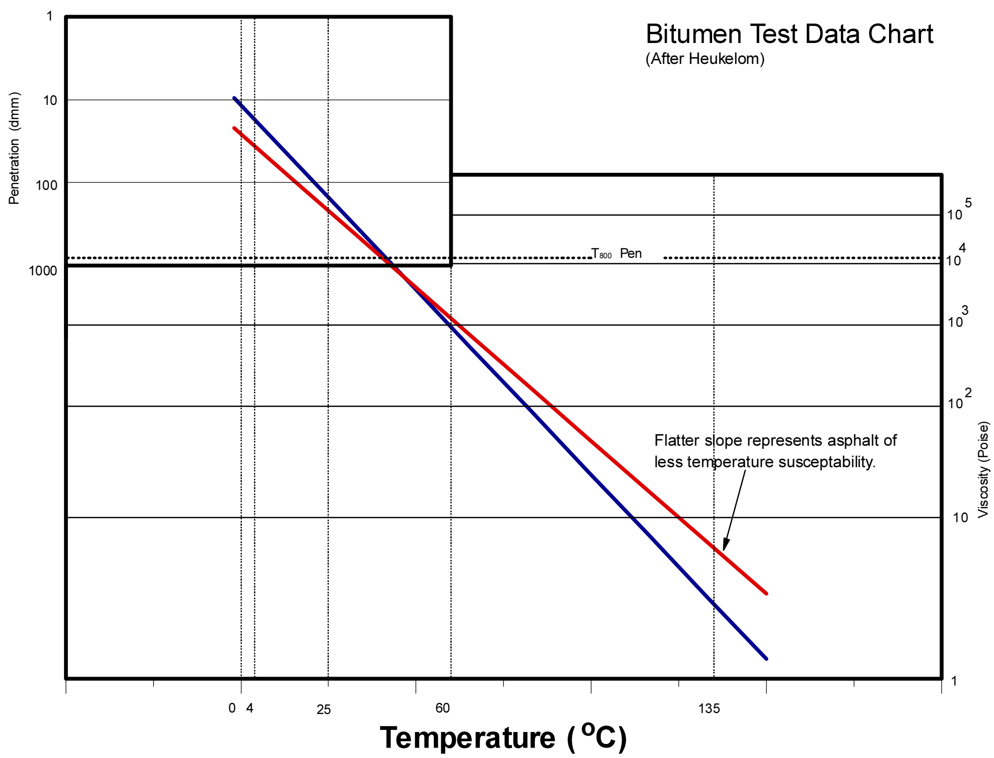

Figure 5-3 Temperature Susceptibility Concept

The significant difference between the CGSB specification and the SHRP specification is that the CGSB specification relies on calculating binder stiffness from penetration and viscosity test results rather than measuring stiffness directly. The calculated stiffnesses are considered questionable by practitioners involved in the direct measurement of these properties. Further, modified binders cannot be reliably characterized using conventionally measured penetration and viscosity parameters.

The stiffness values for the selection of a CGSB asphalt have been determined based on the PI of the asphalt based on its virgin characteristics. Because it is recognized that low temperature performance will be most critical as an asphalt binder ages, a 'safety factor' of 10°C is subtracted from the low design temperature.

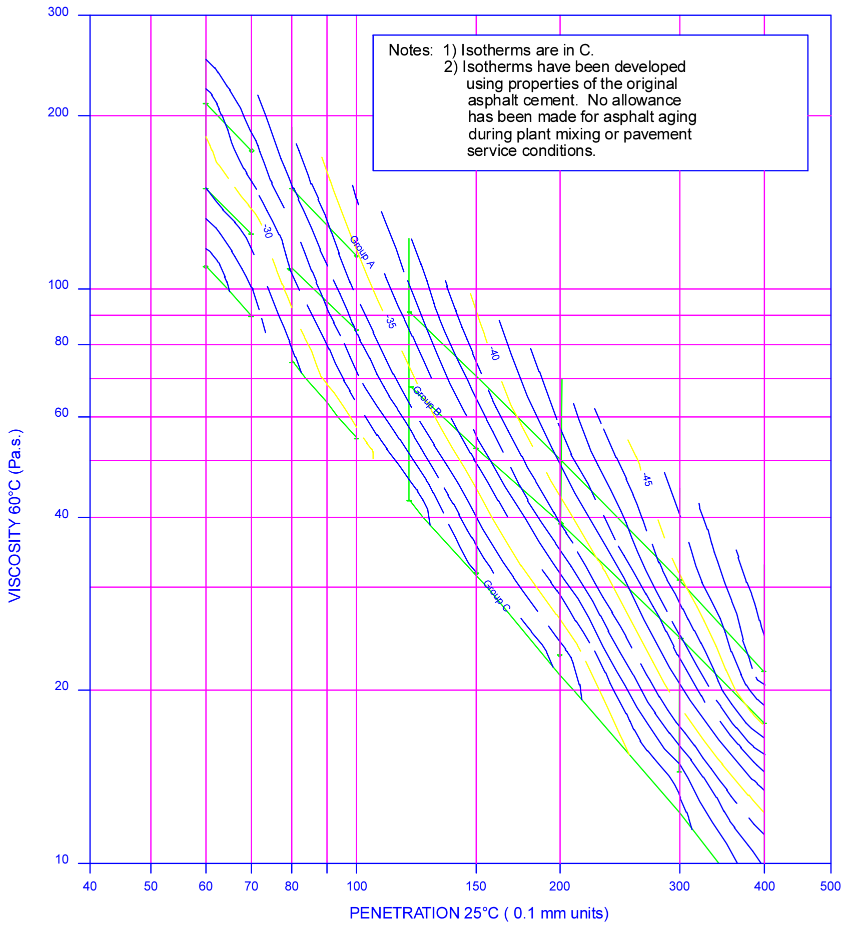

Figure 5-4 Low Temperature Selection of CGSB Asphalts

Figure 5-4 and Figure 5-5 illustrate the methods developed for the selection of CGSB asphalts for low temperature cracking and instability rutting.

Figure 5-4 shows the cracking temperature for various asphalts based on their absolute viscosity and penetration at 25°C. Figure 5-5 provides a concept for selecting a CGSB asphalt grade based on high temperature stiffness (@ 40°C).

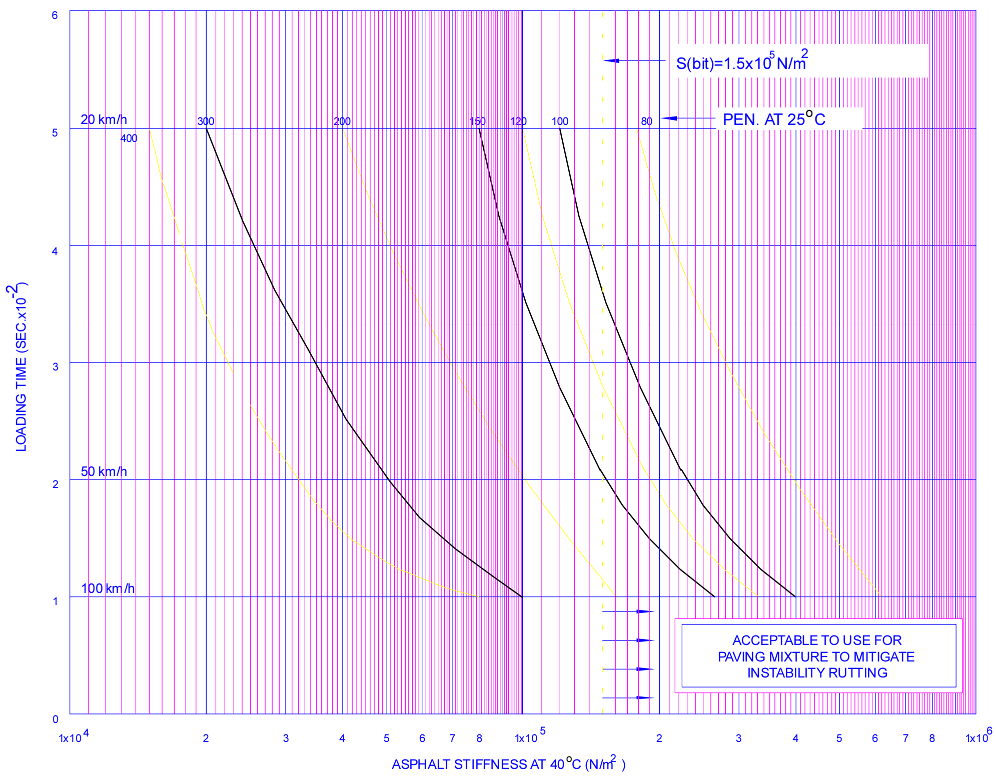

Figure 5-5 High Temperature Selection Criteria For CGSB Group 'A' Asphalts

6. Airport Specific Issues

It is recognized that airport pavement design is based on different considerations than highway pavements. In order to identify and quantify differences and similarities, Table 4 was prepared to examine the process.

Table 4 Comparison of Airport and Highway Pavements

Highway Pavements	Airport Pavements
Structural Design Based on cumulative Equivalent Single Axle Loads, and subgrade support. Typically based on limiting the horizontal strain at the base of the pavement and the top of the subgrade. Limiting strains requires the pavement modulus as an input. The pavement modulus is affected by the binder stiffness.	**Structural Design (PWGSC)** Based on aircraft loading, equivalent single wheel loads, tire pressure, subgrade strength and site freezing index. Based on plate load theory and climatic conditions, subgrade stresses and frost penetration are minimized. The Canadian design method provides for unrestricted number of applications of the design load.
Mix Designs Generally designed using the Marshall method with number of blows matched to traffic volumes.	**Mix Designs** Use 50 blow Marshall design.
Aggregates Aggregate requirements often vary for class of pavement and from agency to agency. Aggregates can be lower quality in some cases in terms of soundness, deleterious material and angularity.	**Aggregates** High quality, relatively clean aggregates required.
Base Layers Base layers designed on the basis of traffic loadings.	**Base Layers** Base layer thickness designed on the basis of providing sufficient structural capacity for subgrade soil bearing strength or sufficient depth for frost protection of frost susceptible soils, whichever is greater.
Asphalt Selection Varies by agency - CGSB User Guidelines consider temperature and traffic.	**Asphalt Selection** CGSB asphalt grade selected on the basis of freezing index (°C·days)

6.1 Structural Design

A review of the structural design process for airports[17,18] indicates that the design is intended to eliminate concerns for fatigue distress. Shear stresses, governed by tire contact pressures, dictate the thickness requirements for the asphalt concrete layers.

Using a hypothetical structure design in accordance with ASG-19[17], elastic layer theory has been used to illustrate the influence of asphalt mix stiffness on potential fatigue distress under a design load having characteristics reflecting a Standard Gear Load Rating (SGLR) of 12. The pavement structure and design loading are shown in Figure 6-1.

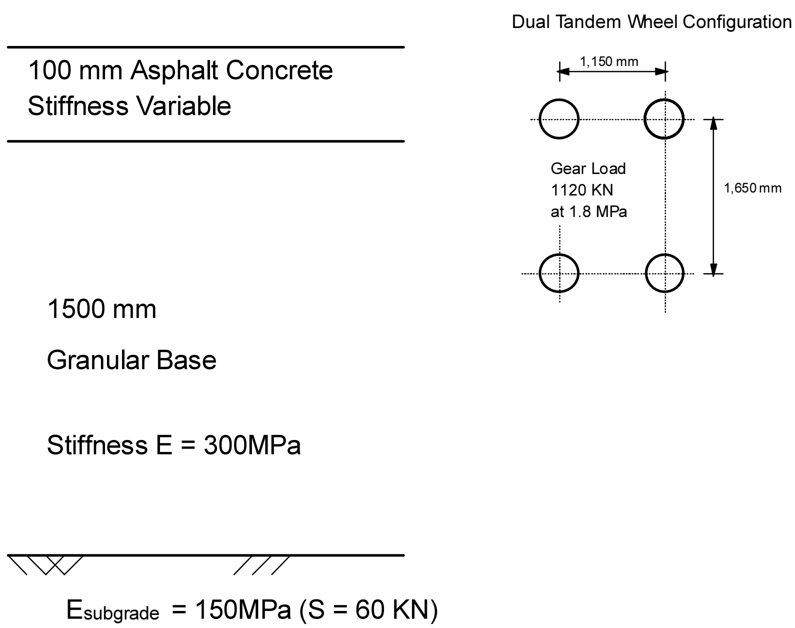

Figure 6-1 Design Load Example For Dual Tandem Landing Gear

This evaluation of fatigue response is intended to provide some insight as to the significance of the SHRP fatigue parameter (G*(Sin(δ))) for airport pavements.

Although it is recognized that fatigue is not generally a design parameter for airports it is important to evaluate the pavement response to assure that any change (to SHRP PG graded asphalts) does not introduce fatigue problems that were not previously an issue.

The structure, consisting of a 100mm asphalt concrete surface and a 1500mm granular base is representative of that required for a subgrade having a bearing strength (S) equal to 60kN and a SGLR equal to 12[18].

Using Chevron 5L based load/strain response algorithms, principal tensile strains at the bottom of the asphalt concrete layer were predicted for asphalt concrete stiffnesses equal to 2,000, 4,000, and 10,000 MPa. The 2,000MPa is representative of reported minimum flexural stiffness values of mixes at 20°C and 0.1 second loading time used in the validation of SHRP specifications[19].

The two higher stiffnesses were selected solely to reflect lower pavement temperatures common to Canada. The predicted tensile strain distributions at the bottom of the asphalt concrete layer are shown in Figure 6-2. Using the strain distributions and fatigue criteria developed by Finn et al[20], the number of applications of the dual tandem design load to 10% and 45% fatigue cracking for each of the three stiffnesses were predicted. Variations in strain levels at a particular point due to aircraft wander were accounted for by assuming a standard deviation of 900mm; a normal distribution has been reported to be true for aircraft movements on a runway[21]. Considering the large variations in gear load configurations represented by the dual tandem assembly, this lateral distribution of applications may be considered conservative and likely under-estimates the number of load applications to failure.

Table 5 Predicted Number of Applications

Mix Stiffness (MPa)	Fatigue Load Applications	
	10% Cracking	45% Cracking
2,000	≈95,000	≈130,000
4,000	≈85,000	≈115,000
10,000	≈120,000	≈165,000

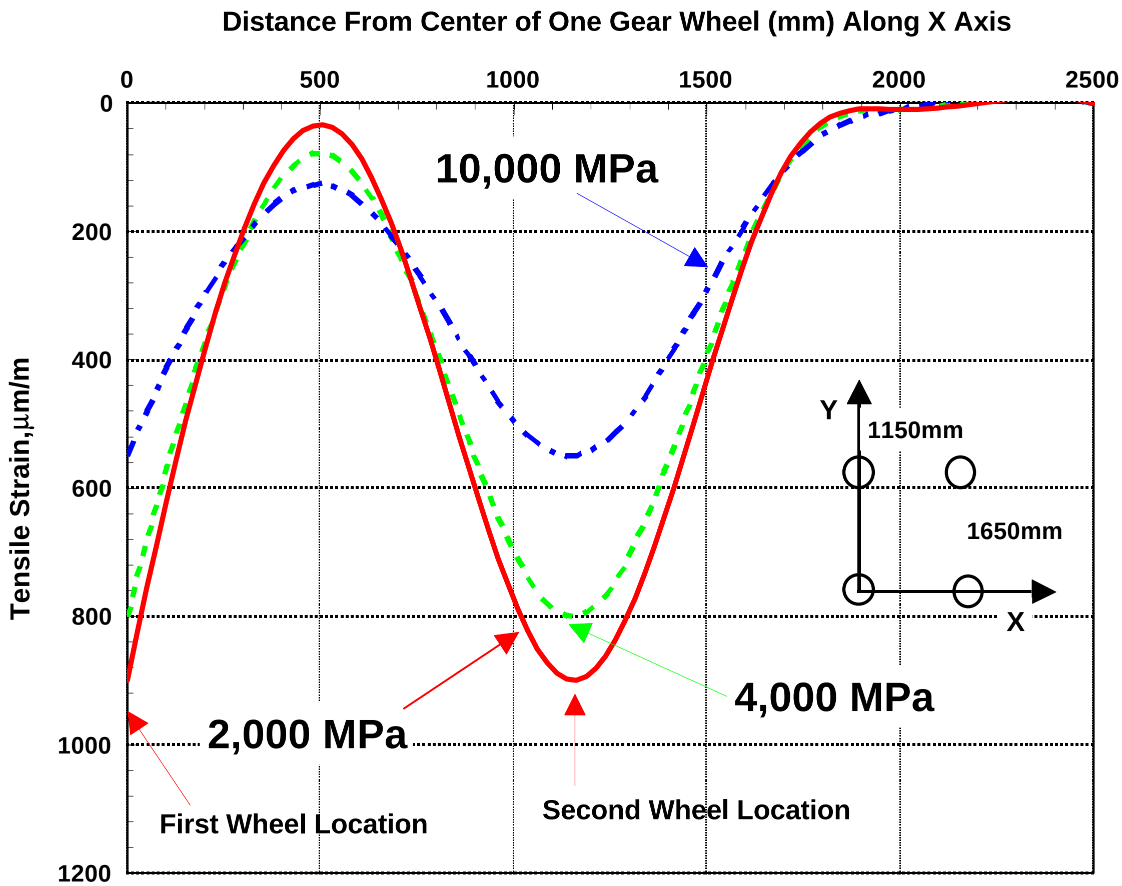

Figure 6-2 Predicted Strain at the Bottom of the Asphalt Wearing Surface

The influence of mix stiffness on predicted fatigue life does not show a conclusive trend for the pavement structure used in this example. While the analysis did not include specific stiffness/temperature relationships or address the impact of seasonal temperature variations on fatigue life, relative to 2,000

MPa at 20°C the higher stiffnesses are considered more representative of the asphalt concrete at lower temperatures experienced the majority of time in Canada.

This example is intended to examine the influence of stiffness of the asphalt concrete on fatigue life and should not be considered an estimation of the total number of allowable loads for an airport pavement. The purpose of this analysis is to examine the influence of mix stiffness on fatigue characteristics of an airport pavement in order to draw some conclusion regarding the significance of the SHRP fatigue parameter to airport pavements. The analysis shows that increasing the mix stiffness (which is related to the binder stiffness) did not result in a specific trend in the calculated fatigue life for the example structure. The fatigue life is greatest for the mixture with the highest stiffness. The fatigue life is a function of both the mix stiffness and the strain in the pavement. As the mix stiffness increases for a given strain, the fatigue life decreases. However, as the pavement stiffness increases the strain decreases. The strain decrease will be sufficient to increase the fatigue life for some structures.

In order to make specific conclusions regarding the impact of the mix stiffness it would be necessary to develop the pavements fatigue life for varying mix stiffnesses and pavement structures. However, this example analysis does suggest that the current fatigue parameter (G*Sin(δ), which specifies a maximum value) is not directly appropriate for airport pavements.

As discussed in Section 4.7, the G*Sin(δ) parameter has recently been found to be a poor indicator of fatigue resistance and its removal from the SHRP specification is being advocated by some researchers(13), this is supported by the above observations for the one example structure and load evaluated.

6.2 Mix design

While this report is to review the SHRP PG binder specification, and not the Superpave mix design method, the following discusses mix design issues which relate to the influence of the asphalt binder.

Conventionally, mix designs in Canada are based on the Marshall method for both highway and airport pavements. It has long been recognized that the Marshall method is empirical in nature and does not necessarily optimize the aggregate skeleton in a mixture.

SHRP Superpave includes a new gyratory method of mix design which is intended to develop a superior aggregate skeleton for the mix.

Both the SHRP PG binder specification and the TAC User Guide for the selection of CGSB Asphalt grades specifically select a binder based on its stiffness at the high design temperatures. The CGSB method considers traffic loadings in the form of cumulative ESALs in the selection of the high temperature grade while the SHRP PG binder specification only considers ESAL loadings for very high trafficked roadways and uses temperature alone for the initial grade selection.

While it would appear that the TAC User Guide places more significance on the traffic loadings than SHRP, both specifications recognize that the asphalt binder plays a small role in the shear resistance of a mixture relative to the role of the aggregate. (The role of the binder was quantified somewhat for Alberta highway conditions and forms part of the Alberta mix type selection criteria[(21,22)]).

The use of 50 blow Marshall designs will generally result in 0.3-0.5 % increase in asphalt binder compared to a 75 blow Marshall design. This increase in asphalt binder increases asphalt film thickness and pavement durability. As permanent deformation is not a typical distress on airfield pavements the 50 blow design has been found to produce a superior pavement for airfields.

6.3 Aggregates

The aggregate component of an asphalt concrete pavement generally represents approximately 85% of the mixture by volume. As such, the aggregate plays the major role in the ability of the mix to resist permanent deformation under aircraft or traffic loadings.

The role of the aggregate in a pavement's ability to carry heavy loads can not be overstated. An aggregate particle provides many times the load bearing capacity of a viscoelastic asphalt binder, and the selection of aggregates and gradations that provide superior aggregate interlock will result in a superior performing asphalt pavement. Aggregate characteristics of significance include angularity of both coarse and fine particles, flat and elongated particles and clay content. Superpave considers these characteristics in the aggregate requirements. As well, Superpave utilizes trial mixes to allow for the selection of an optimum aggregate gradation which provides maximum resistance to deformation.

The PWGSC standards for HMAC aggregates attempts to assure a relatively high quality aggregate. However, it is likely that the Superpave mix design requirements would optimize the aggregate gradation selected to assure an aggregate structure capable of resisting large loads. The gyratory procedure includes a protocol for selecting optimal aggregate blends by selecting aggregate proportions/gradations which optimize the resistance to compaction which are therefore expected to have better shear resistance.

6.4 Selection of Asphalt Grade

The airport design requirements for asphalt grade selection[16] provide the following:

Table 6 Airport Asphalt Material Selection Criteria

Freezing Index (°C - Days)	Runways	Taxiways and Aprons
<500	80-100	80-100
500 - 1400	120-150	80-100
>1400	150-200	120-150

For Freezing Indices greater than 500 a high viscosity asphalt is to be used.

The parameters identified in the above table can be seen to ignore high design temperatures per se (although areas with a high freezing index would generally be expected to experience lower maximum temperatures as well), and deal with low design temperatures in an indirect manner as defined by the Freezing index.

Figure 6-3 shows the asphalts required for some select locations based on the Freezing Index contours for Canada and the currently recommended CGSB asphalt grade for the Freezing Index of the locale. The SHRP binder grade is also shown for those locations. From the Freezing Index data provided in the Pavement Structural Design Training Manual (ATR-21) it is apparent that the majority of Canada is located in areas with a Freezing Index greater than 1400 °C-days. The East and West Coast regions, and the southern most region of Ontario are generally less than 500 °C-days with the intermediate areas (500> <1400 °C-days) including North-Central B.C., Southern Alberta, and the areas immediate north of the Great Lakes and along the St. Lawrence River.

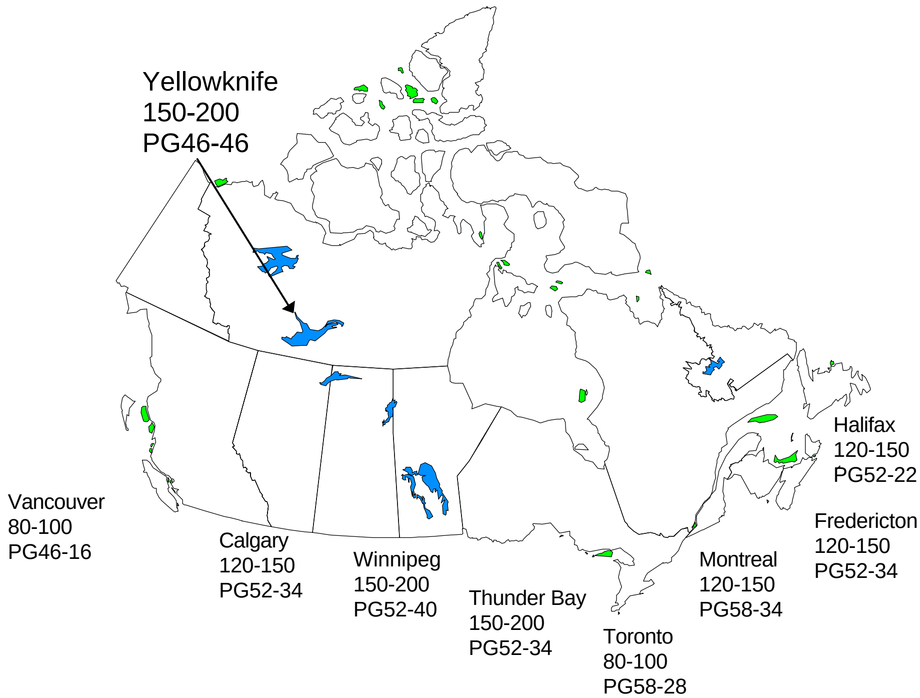

Figure 6-3 Asphalt Selection For Select Locations

The SHRP PG binder grade was selected on the basis of the 98% reliability for both high and low temperatures. Historical weather information was used from the SHRPBIND computer program and the low temperature was adjusted using the equation included in the Asphalt Institutes SP-1 as discussed in Section 4.4. The CGSB grades were selected on the basis of the Freezing Indices, no group (ie: A, B, or C) was identified although for the areas with more than 500 °C-days a 'high viscosity' binder is specified to be used.

6.4.1 Comparison of Asphalt Grades

The CGSB asphalt grades are established on the basis of their penetration and viscosity of original samples. The SHRP PG binder specification is based on the asphalt's stiffness (low temperature) and complex shear modulus and phase angle (high service temperatures) of original, RTFOT aged, and PAV aged samples.

Comparisons of the two asphalt grading systems relative to actual test results and the specific asphalt grades as would be used for airport construction are described below.

Figure 6-4 illustrates test results for two asphalts graded to the SHRP PG binder specification. Figure 6-4 shows the results of all three test parameters (ie: stiffness, G*(Sin(δ), and G*/(Sin(δ))) for the various temperature ranges and binder conditioning. The test results for the individual parameters at varying test temperatures relative to their limiting or minimum values are shown. For example, the DSR results for the original binder (far right of the figure) show that the 150-200A asphalt did not meet the minimum G*/Sin(δ) at 64°C but did at 58°C. Likewise, the 150-200B asphalt did not meet the G*/Sin(δ) minimum requirement at 58°C but did at 52°C.

It can be seen from the results that the ‘A’ grade asphalt grades to a higher temperature grade than does the ‘B’ grade asphalt (ie: PG58-XX versus PG52-XX); however both products grade to the same low temperature (ie: -28).

Other examples from initial work conducted on grading CGSB asphalts to the SHRP PG specifications show 200-300A and 300-400A asphalts from Alberta crude sources both grade to -34. The test data shows that these grades are within about 2°C, at the limiting stiffness value of 300MPa. The two softer grades do grade differently at the high temperatures.

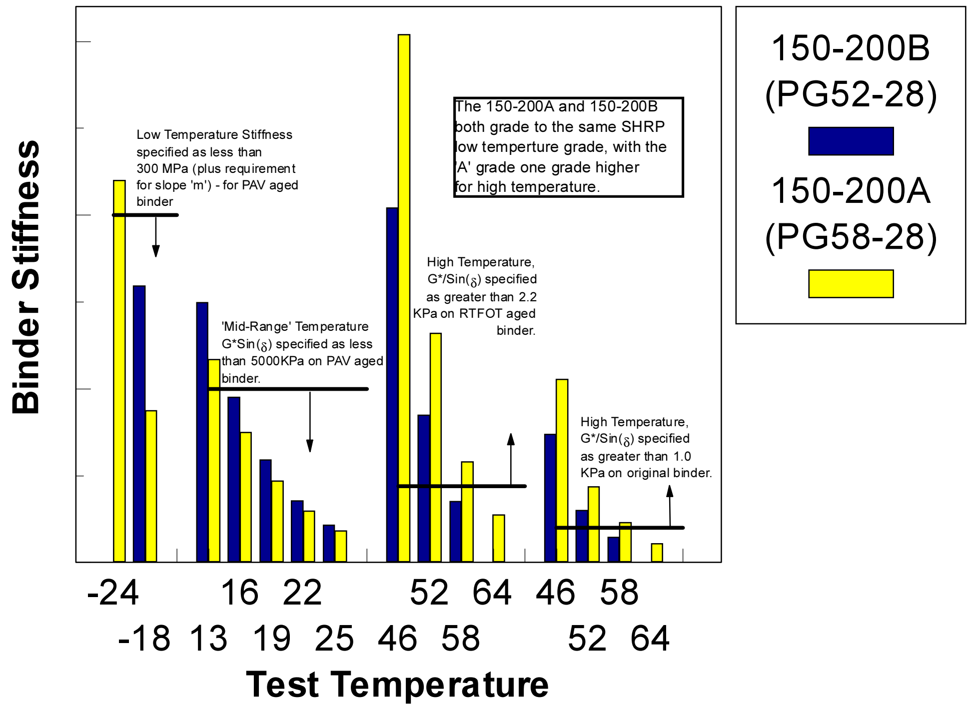

Figure 6-4 Grading of Asphalts To SHRP Specifications

Experience with the 150-200 asphalts show that the low temperature performance is not equivalent[(24)]. The reason for the difference in performance can be gleaned from both Figure 6-4 and Figure 6-5 as the low temperature stiffness for the ‘A’ grade asphalt cement can be seen to be significantly lower than that for the ‘B’ grade asphalt, at the same test temperature.

Figure 6-5, which includes stiffness measurements converted from Complex Shear modulus results as described previously, allows an estimate to be made of the temperature at which the asphalt binder meets the maximum allowed stiffness. For the specific asphalts presented here it can be seen that the ‘B’ asphalt reaches the critical stiffness at about -19°C while the ‘A’ asphalt reaches the critical stiffness at approximately -23°C. Each asphalt is close to being classified as a different SHRP PG grade.

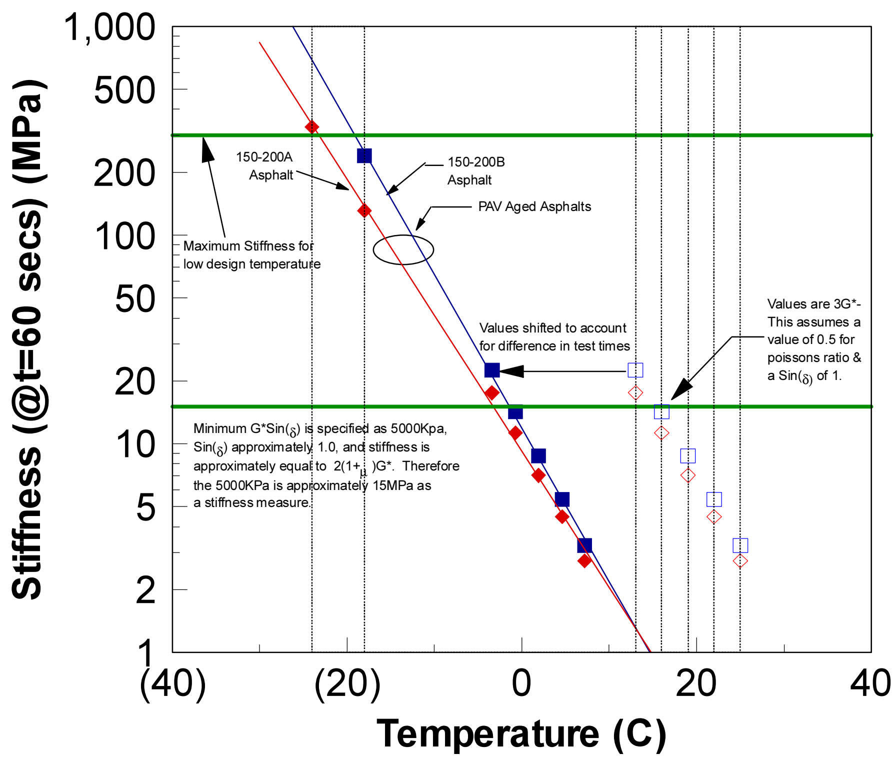

Figure 6-5 Stiffness Measurements For Two Asphalts

The preceding illustrates that a SHRP PG grade may possibly include asphalts that would currently fit into two different CGSB asphalt grades. Likewise, CGSB graded asphalts could be expected to possibly meet different SHRP PG grades within a single CGSB graded asphalt. Notwithstanding, there are some trends in the current CGSB asphalts[8] relative to SHRP grades, and although it is emphasized that a given CGSB grade cannot be directly correlated to a SHRP grade, the trends identified in the TAC report can be used to examine the current asphalt selection guidelines for airports.

The specific sites identified in Figure 6-3 are shown in Table 7.

Table 7 Asphalt Grades For Select Airport Sites

Site	CGSB Binder Grade[1]	SHRP PG Binder Grade[2]
Vancouver	80-100	PG46-16
Calgary	120-150	PG52-34
Winnipeg	150-200	PG52-40
Thunder Bay	150-200	PG52-34
Toronto	80-100	PG58-28
Montreal	120-150	PG58-34
Fredericton	120-150	PG52-34
Halifax	120-150	PG52-22
Yellowknife	150-200	PG46-46

[1] CGSB Binder Grades selected based on site freezing index and PWGSC airport standards.
[2.] SHRP PG binder grades selected based on site temperature at 98% and low temperature equation as referenced in SP-1.

Based on ***trends*** identified by initial SHRP testing in Alberta and Quebec, the CGSB grades tend to correspond to the following high temperature grading: (Again it is emphasized that these trends are being examined for illustrative purposes only and should not be considered applicable to all CGSB asphalts).

80-100A ➔ PG64-YY
120-150A ➔ PG58-YY
150-200A ➔ PG58-YY

These trends can be used to compare the airport asphalt requirements with SHRP requirements. It can be seen that most airport sites would currently use an asphalt grade that is one or two grades harder (or three for Vancouver) on the high temperature end than the SHRP PG binder specification would require. Montreal seems the only exception to this trend, for the few sites examined. However, the PG grade selected for Montreal was done so at the pure interpretation of the 98% reliability needs. The actual high temperature design value is 52.1°C - a PG52-YY binder would theoretically be less than the 98% reliability. This example illustrates the need to understand both the performance level required of a PG grade asphalt and the performance level that is currently being received with CGSB specified asphalts in order to specify an appropriate asphalt for an airport pavement.

These few examples illustrate that the current practice, based on experience with CGSB asphalt grades, is to utilize a stiffer binder on the high temperature end than SHRP would dictate based on temperature information alone.

The trend observed is supported by SHRP’s current guidelines. Within the SHRP PG binder specification as documented in the Asphalt Institute’s SP-1, there is provision for adjusting the asphalt binder grade selection for traffic speed and loading. SHRP recommends going to one grade harder for slower moving vehicles and two grades harder for standing loads. For airport operations, slow moving and stopped aircraft are normal.

SHRP’s guidelines were identified for highway traffic and loading conditions. A similar approach for airports would logically make the expected aircraft loadings the governing factor for selecting asphalt grades higher than dictated by temperature considerations alone.

Modified Asphalts

Because the SHRP binder selection is expected to require modified asphalts to meet many of the grades required in Canada it is appropriate to discuss issues associated with modified asphalts.

The selection of asphalts based on the SHRP grading system results in a significantly increased number of asphalts grades (37 possible grades) compared to the current CGSB specification (6 grades X 3 Groups). Asphalts specified and used within the CGSB specification have historically been specified on the basis of what is available in the marketplace.

The range (in °C) between the high temperature grading and the low temperature grading in essence defines the temperature susceptibility of the asphalt binder. As a rule of thumb, a range of greater than about 90° can only be achieved using a modified asphalt binder; some grades identified in the SHRP PG binder specification may not be commercially available for some time.

As modified asphalts can represent a significant cost, (eg: an approximate doubling of the cost of the binder to meet a PGXX-40 requirement) it is important for the specifying agency to recognize the costs and assure that the use of modified asphalt is cost effective. It can be seen from Table 7 that if the high temperature portion of the binder requirement is increased even one grade, almost all of the sites used in the example would require a modified asphalt.

Modified binders also raise questions regarding the applicability of the SHRP PG binder specification and testing protocols. It is considered beyond the scope of this project to evaluate the use of the SHRP PG binder specification for modified asphalts. It is known that the NCHRP has a current contract to evaluate testing protocols for modified binders and to provide recommendations for accommodating modified binders in the SHRP PG binder specification. It can also be stated that while there is some known concerns with the SHRP testing protocols for modified binders, the CGSB testing protocols are even less suited.

7. Discussion

As presented previously, the three distress modes influenced by the properties of the asphalt binder are:

- thermal cracking,
- fatigue cracking and
- permanent deformation.

As the asphalt binder stiffness contributes to the overall asphalt concrete mix stiffness it follows that the binder will play a role in the resulting strains at the base of the pavement layer and in the top of the subgrade layer, as well as the overall resistance of the pavement to shear forces. It is also true that the asphalt binder characteristics will govern thermal cracking behavior of a pavement.

The previous Sections have identified the background to the existing CGSB specification and the SHRP PG binder specification and provide the basis for evaluating the impact of the asphalt binder on pavement performance. The following sections compare and evaluate the significance of the asphalt binder on the distress mode and compare the SHRP PG binder specifications to the current use of the CGSB specifications for airport pavements.

7.1.1 Thermal Cracking

Thermal cracking is, overall, influenced more by the asphalt binder stiffness at low temperatures than any other factor. The significance of even 1 or 2 degrees C can greatly impact the low temperature performance of an asphalt. As was presented in the previous sections, the difference between an ‘A’ and ‘B’ grade asphalt at the critical stiffness value may be 4°C, which still maintains them in the same PG grade - this difference leads to a difference in thermal cracking.

Other factors, specific to airports have been considered but do not readily suggest any significant impact on low temperature performance. These factors include relatively high asphalt film thicknesses as a result of 50 blow Marshall designs, runway geometry (ie: significantly wider than a highway pavement), and practice of deep granular layers which minimize HMAC use and guard against frost action.

However, the literature[(11)] has consistently identified an increased cracking frequency in pavements constructed on non-cohesive soils. Robertson’s[(9)] most recent work with low pavement temperatures indicated that the pavement temperature of one site was discarded from the analysis because it did not fit the data. It was suggested that the difference for that site might be related to the coarse 'bouldery' subgrade. While the contribution of the subgrade does not appear to be well understood, pavements constructed on thick granular layers may also be impacted. It is hypothesized that the affect from the subgrade layers may be related to thermal conditions and therefore the algorithms developed for predicting low pavement temperatures may have to be reevaluated for pavements with thick granular layers, such as many airport pavements.

The overall conclusion is that for low temperature performance an airport pavement should not be expected to perform differently than a highway pavement. Therefore, the SHRP PG binder specification relative to low temperature is felt to be applicable to airport pavements to the same degree as it is to highway pavements.

7.1.2 Fatigue Cracking

It has been discussed that the binder's role in fatigue cracking on airport pavements is not likely significant because of the structural design approach which basically results in a structure adequate for unlimited repetitions of aircraft loadings. The example presented in Section 6.1 was intended to examine the influence of stiffness on fatigue and should not be interpreted in terms of the total number of allowable loads calculated for the example.

It has also been noted that the G*Sin(δ) parameter which is intended to provide a measure of fatigue resistance is a matter of significant debate within the SHRP implementation group. This parameter is likely to be replaced by a different parameter and may include specific requirements depending on the pavement structure[13].

While it can be concluded that the SHRP fatigue parameter (G*Sin(δ)) is not applicable to airport pavements, it still may have significance for asphalt pavements structures different than the 100mm used in the example. As well, it is noted that because the testing is conducted on PAV aged binder at ‘ambient’ type temperatures, it provides another method of guarding against excessive age hardening.

7.1.3 Permanent Deformation

Permanent deformation or rutting which occurs on high trafficked highway pavements is the result of the applications of many loads concentrated in the wheelpaths of the roadway. For airport pavements permanent deformation is caused by fewer repetitions of heavier loads, and also loads which increase due to slow speeds during taxiing and while waiting for clearances.

The SHRP parameters for high temperature stiffness are based on what has performed for highway loading conditions; similarly, it is presumed, as the current PWGSC asphalt binder standards have evolved for airports.

As can be noted from the example grades for airport locations discussed previously, there appears to be some differences on the high temperature end requirements of SHRP than is the current practice. That is, CGSB grades used for the various airports would appear to have stiffer high temperature characteristics than SHRP would recommend on the basis of temperatures. This indicates that adjustments or adaptations of the SHRP criteria are required for high temperature performance.

8. Conclusions and Recommendations

Canadian airports span the range of geographic locations and aircraft loadings. As such pavement design and maintenance is a difficult undertaking. PWGSC has a significant amount of experience in airport construction and the performance of materials that have been used historically. This knowledge base should be exploited by examining it in greater detail as the issue of binder specifications is studied further.

The SHRP PG binder specification is based on sound engineering principles and directly measures the asphalt's fundamental properties and as such is considered a major step forward for asphalt technology.

However, SHRP is brand new, virtually untried and untested. While millions of dollars were spent developing, and many more are being spent validating the SHRP PG binder specification's parameters, the specification can be expected to evolve well into the next decade. The FHWA is coordinating the implementation of SHRP products and localized modifications are being discouraged in the interest of maintaining a uniform specification. There is virtually no work being conducted to validate the use of SHRP's Superpave (binder and mixtures) for airport pavements.

The conclusions and recommendations presented here are based on the current state of the SHRP specification and are intended to provide PWGSC with direction relative to the potential use of the SHRP PG binder specification. A summary of the main issues and conclusions relative to the parameters discussed in this report are shown in Table 8. Conclusions and recommendations for further work are documented in the following sections.

Table 8 Summary of Superpave Binder Specification and Testing Protocol Issues

Issue	Description	Significance	Discussion	Conclusions/Recommendations (Potential adoption or adaptation)
Flash Point	The SHRP Binder specification has a 230°C limit which is higher than the current requirements for the softer CGSB grades.	Not felt to be significant.	A cursory review of test results on Alberta asphalts over the last 10 years indicated that about 3% of the tests would fail the SHRP requirements	Issue affects the entire industry and not just airports.
Solubility in Trichloroethylene	The SHRP specification does not include a requirement for solubility in trichloroethylene.	The lack of this specification may allow the use of fillers in asphalt cements which may not be desirable.	ASTM is examining the introduction of solubility into the SHRP specification	Although this is not an issue specific to airports it is concluded that solubility in TCE should be added to the SHRP specification.
Loss on Heating	SHRP has a maximum loss on heating (after RTFOT) of 1% which is less than allowed by CGSB for softer asphalts.	For the production of asphalts capable of meeting low temperature performance requirements it is necessary to utilize soft asphalts; the SHRP specification may be too restrictive.	As it is known that softer asphalts, such as those required to meet low temperature parameters, will be at risk to meet the required loss specification, even though the after PAV aged material meets the physical test requirements. This SHRP parameter needs to be reviewed prior to adopting SHRP specifications	Loss on heating should be used to guard against the "innovative" addition of light materials to improve low temperature properties. In Canada where we have a high demand of very low temperature grades it will likely be necessary to modify this specification to a value of 1.5% for grades PGXX-34 and lower, similar to the CGSB standard. As this is not specific to airports it is expected that the industry will have to resolve this in the near future.
RTFOT Test	CGSB currently specifies the TFOT which is generally considered less severe than the RTFOT.	The use of the RTFOT has implications in the mass loss of softer asphalts and therefore is of significance for the mass loss specification.	In addition to the question of mass loss, there are some indications that the RTFOT may change the properties of some modified asphalts more than they change during mixing in an asphalt plant.	The RTFOT is not a specific issue to airports.
PAV Aging	The Pressure Aging Vessel is used to age asphalt binders in an accelerated manner in order to simulate long term aging.	The PAV aged asphalt forms the basis for some SHRP parameters used for determining physical properties (ie: stiffness and complex shear modulus) and as such affects the performance of the asphalt.	A number of influences have been identified which impact the rate at which an asphalt will age in the field. One of the largest impacts is the initial plant aging. The PAV has been compared to in-service highway pavements for comparison of the relative aging and has been found to represent somewhere up to ten years aging. While the temperature regime of an airport pavement may be different enough from a highway pavement to affect the rate of in-service aging of an asphalt binder, other influences such as initial plant aging, air voids, and in-service temperature differences caused by such subtle differences as aggregate color are expected to be more significant.	It is concluded that PAV aging should be considered as representative for airfield pavements as it is for highway pavements.
Bending Beam Rheometer	The BBR measure of binder stiffness at low temperature is intended to prevent low temperature cracking.	The BBR test by itself is significant in that it provides a means of directly measuring the binders low temperature properties.	Airport pavements may experience different temperature regimes than Highway pavements due to geometry and structure. If such differences do exist then the testing temperature would be affected. This is an issue only in the manner in which pavement temperatures are determined.	The Bending Beam Rheometer is considered applicable to airfield pavements as for highway pavements. (Although it is noted that the limiting stiffness, originally developed for highways is currently being reviewed as being too high).

Direct Tension Test	The DTT is considered an complimentary test for asphalts which fail the BBR requirements but still exhibit stiffnesses less than 600MPa.	In the case where the asphalt is greater than 300MPa and less than 600MPa, the DTT requires an asphalt to exceed 1% strain to failure in order for it to be considered acceptable.	The cracking mechanism for pavements is considered to be a function of binder tensile strength and ability to relax stresses. Therefore the mechanism is the same for airfield pavements and highway pavements.	The DTT parameters are considered equally valid for airfield pavements.
DSR Testing (General)	The loading frequency for the DSR testing is 10rad/s at high and intermediate temperatures.	The loading frequency has been found to be equivalent to slow moving traffic and was adopted by SHRP not as representative of traffic movements but as a convenient rate of testing in the laboratory.	While testing often does not mimic in-service conditions the DSR rate of testing would be more representative of slow moving aircraft than highway traffic. The limiting value for the $G^*/\mathrm{Sin}(\delta)$ parameter is discussed below.	The loading frequency of the DSR appears to be more applicable to airfield pavements where the critical loading will be slow moving aircraft as compared to highway truck traffic. The potentially larger deformations may, however, introduce larger error into the testing. Testing with larger deformations should be considered.
@ Medium Temperature	At the medium temperatures the DSR measures the properties of PAV aged asphalts.	This parameter is currently being criticized in terms of its ability to guard against fatigue failures in asphalt pavements and it is expected to be removed from the specification requirements in the near future.	As it was noted in the text, this parameter ,$G^*(\mathrm{Sin}(\delta))$, may provide a further check in guarding against excessive age hardening.	The $G^*(\mathrm{Sin}(\delta))$ parameter is not significant to airfield pavements. As it has been recently shown to not correlate to fatigue on highway pavements it is concluded that the parameter only be considered in terms of age hardening concerns. The example calculations help to demonstrate that binder characteristics would not be expected to be a concern to guard against fatigue for airport pavements.
@ High Temperature (on tank and RTFOT asphalt)	At high temperature the DSR measures the potential for rutting concerns in asphalt pavements.	The SHRP specification limit for the $G^*/\mathrm{Sin}(\delta)$) parameter is based on highway loadings and observations of performance on highways. As such it is not considered directly applicable to aircraft loads.	The DSR limiting values at high temperatures have been selected based on the performance of highway pavements. For highways, slow moving loads call for the increase in the high temperature grade - there is no conclusive data to help select a high temperature grade for airport pavements.	The grades selected in accordance to the SHRP protocols compared to the current CGSB grades selected by PWGSC suggest that airfield pavements would typically use harder asphalts than SHRP would suggest, however, because of the lack of information on the DSR values of current CGSB asphalts it is difficult to predict what appropriate values are Further research and/or review of current grades in terms of DSR results will allow more definitive conclusions to be drawn. It is considered desirable to pursue a format which would link the asphalt grade to the SGL of the aircraft using the facility.
High Design Temperatures	High Design temperatures are not considered in the current PWGSC guidelines.	The high design temperature selection in the SHRP procedure allows the ambient in-service conditions to be considered directly.	The high temperature selection, which identifies the test temperature at which the DSR results must be acceptable is considered appropriate for airfield pavements. As discussed above, because the DSR parameters are based on highway traffic the loading conditions of an airfield are not considered to be properly represented.	The selection of the high temperature for design purposes is considered appropriate for airfield pavements.
Low Design Temperatures	Low design temperatures are currently considered in the PWGSC standards by means of the freezing index.	The method of determining low pavement temperatures as presented in this report is considered an improvement over the SHRP methods and more directly accounts for low temperatures than does the freezing index.	The low design temperature dictates the temperature at which the asphalt must meet the low temperature stiffness requirements. While it has been concluded that the low temperature stiffness values would be appropriate for airfield pavements it is not known if the airfield pavements differences in terms of structure would affect the low temperatures experienced by the pavement.	While the algorithms presented in this paper reflect the current state-of-the-art for determining asphalt pavement temperatures it is not known if this is as applicable to asphalt pavements which are constructed on deep granular layers. Further research is required with low pavement temperatures to quantify any significant differences. Such work should include a review of the U.S. algorithm being developed from the LTPP sites.

8.1 Conclusions

1. The SHRP specification is new and unproved. However, it is based on the concept of binder stiffness which Canadian researchers have long concluded is the most fundamental property of a (conventional) binder which relates to pavement performance at high and low temperatures.

2. The SHRP PG binder specification directly accounts for the high and low temperature issues and as such is easy to apply and understand; in this way the SHRP PG binder specification is superior to the CGSB specification.

3. The range in PG binder grades is 6°C at both the high and low temperature ends. This range in temperature is significant and represents a ‘broader’ range of asphalts for each range than does the CGSB specification. (As an example, depending on the crude source and refining practice, 80-100, 120-150,and 150-200 CGSB grade asphalts could all potentially be graded as SHRP PG58-28 binders.)

4. Airport pavement design is empirical based and does not directly consider binder stiffness issues; although the experience that the designs are based on should provide a significant indicator of the acceptability of the current asphalt selection. (ie: are the currently specified asphalts performing?).

5. Although there are numerous questions regarding the appropriateness of various of the SHRP PG binder specification parameters and their limits, there is no clear information which would suggest that alternate limits/parameters be adopted for airport pavements. It is expected, throughout the industry, that changes to the SHRP PG binder specification

will continue well into the next decade.

6. The practice of replacing frost susceptible native soil with granular material may impact the propensity of an asphalt to develop low temperature (thermal) cracking. This supports conservatism with regards to the selection of the minimum design temperature.

8.2 Recommendations

1. It is recommended that PWGSC establish a program to evaluate and document the following items which are outside the scope of this study:
    - The range of stiffnesses currently being supplied for CGSB specified asphalt cements, (refiners should have this information)
    - The performance of asphalts specified under the CGSB specifications, (based on PWGSC experience) and
    - The significance of ‘overlap’ of CGSB grades supplied in different parts of the country when compared to SHRP PG binder specifications.

    This recommendation will allow PWGSC to identify PG grades which will provide at least the same performance as is currently achieved and will identify the potential for improving binder performance.

2. Further research is recommended to identify appropriate values of G*/Sin for loading associated with airport pavements. It is envisioned that values appropriate for different ranges of SLGRs would be identified. While initial values may be obtained on the basis of current practice (such as would be determined form recommendation #1), advanced laboratory testing to evaluate larger deformations may be needed to better describe the behaviour of airport pavements. Such work may identify the need for different testing equipment and different (non-linear viscoelasticity) models to properly

characterize the asphalts for airports.

3. Research using equipment such as ALF (Accelerated Load Facility) is recommended to identify pavement design requirements (including appropriate $G^*/\sin(\delta)$ limits) for loads and tire pressures representative of aircraft loading. Such specialized research may be possible in a collaborative effort between PWGSC and the FAA with the cooperation of the FHWA and their Pavement Testing Facility at the Turner-Fairbank Highway Research Center.

4. The SHRP specification should be adopted as appropriate for the part of the country for which the airport is to be constructed (ie: the prevalent specifications should be used which may be dictated by the local highway agencies) - with the provision that both high and low temperature requirements be reviewed relative to what has historically been provided to assure that the current standard is maintained.

5. For high temperature loadings it is recommended that the high temperature grade be increased based on the speed and size of aircraft. As was demonstrated with the relationship between loading frequency and stiffness (Complex modulus), standing loads greatly reduces the asphalt modulus. SHRP currently recommends an adjustment of one grade for slow traffic and two grades for standing traffic.

   For airport loadings, based on the CGSB grades currently used, an increase of two grades is recommended as an interim measure, for airport pavements when the airport is utilized by large jet aircraft. (Recommendation #1 would be expected to help PWGSC refine this recommendation based on its experiences.)

6. When applying the SHRP specification a degree of conservatism in the selection of the minimum design temperature is recommended when new pavement construction is undertaken. A reliability of 98% is considered warranted because low temperature cracking can occur with a single occurrence of the low temperature.

7. When overlaying cracked asphalt or PCC pavements, the cracking will generally be controlled more by the underlying pavement and conservatism in the selection of the low pavement temperature is not warranted. For overlays, the low temperature selection should include consideration of the overlay thickness as a major factor in reflective cracking. The low temperature reliability should then be selected on the basis of the binders potential in contributing to thermal cracking severity. This recommendation is made recognizing that the binder selection alone will not prevent reflective cracking but needs to provide some resistance to significant deterioration of reflected cracks.

Respectfully Submitted,
EBA Engineering Consultants Ltd.

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