# DEVELOPMENT OF PERFORMANCE GRADED BINDER SELECTION METHOD FOR CANADIAN AIRPORT PAVEMENTS

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PREPARED FOR THE 1999 FEDERAL AVIATION ADMINISTRATION
TECHNOLOGY TRANSFER CONFERENCE

April 1999

## 1.0 INTRODUCTION

Canadian airport engineering guidelines and recommended practices are maintained by the Airport Engineering Division of Public Works and Government Services Canada (PWGSC). These guidelines have historically specified the use of penetration/viscosity graded asphalt cement for the construction of flexible airport pavements.

As a result of the development of the SUPERPAVE Performance Graded (PG) asphalt cement binder specification, it was necessary for PWGSC to initiate an investigation into the suitability of using these PG binder specifications in the construction of Canadian airport pavements since SUPERPAVE was developed specifically for highways.

This paper summarizes the development of a PG Binder selection method[1] specifically for Canadian airport pavements which considers the unique technical and operational requirements of airport pavements, Canadian airport engineering methodologies and Canadian environmental conditions.

### 1.1 Background

The American Strategic Highway Research Program (SHRP) was carried out over a five year period with a budget of $50 million devoted to asphalt binders and asphalt paving mixtures. The end results from this research were asphalt binder specifications that are based on fundamental engineering properties of the asphalt binder and a mix design methodology which is expected to better replicate actual in-place pavement characteristics. The combined output was termed **SUPERPAVE** as an acronym for **Su**perior **Per**forming Asphalt **Pave**ments.

Across Canada, many Provincial transportation agencies have begun to adopt the PG binder specifications that will likely become the new Canadian standard. Asphalt cements complying with these specifications may soon become the only readily available asphalt products and penetration/viscosity asphalt grades will be phased out. Research has also shown that there may be technical and economic benefits through the use of PG Binders.

## 2.0 CURRENT CANADIAN AIRPORT GUIDELINES

### 2.1 Current Canadian Asphalt Specification

Current Canadian airport construction standards and guidelines, specify the use of asphalt cements conforming to the requirements of the Canadian standard CAN/CGSB-16.3-M90 "Asphalt Cement For Road Purposes"[2].

Under this specification, asphalt grades are defined by their penetration range and viscosity group, which reflects differences in binder stiffness and temperature susceptibility, which are used to predict performance characteristics. These specifications are summarized in Table 1 for the three asphalt grades currently recommended for airport pavement construction in Canada.

Figure 1 shows the penetration-viscosity relation for each penetration grade based on absolute viscosity. There is a similar figure in the specification based on the kinematic viscosity. In addition to these penetration and viscosity requirements, the specification includes requirements for the flash point, penetration of residue as a percentage of the original penetration after the Thin Film Oven Test, solubility in trichloroethylene, and mass loss by the Thin Film Oven Test.

**Table 1 - CGSB Specifications for Asphalt Grades used on Canadian Airports**

| **Penetration Grades** | **80-100** | | **120-150** | | **150-200** | |
|---|---|---|---|---|---|---|
| **Requirements** | **Min.** | **Max.** | **Min.** | **Max.** | **Min.** | **Max.** |
| Penetration at 25°C, 100g, 5s, 0.1 mm | 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 <u>either</u> Absolute or Kinematic Viscosity for all Asphalt Grades (Such as Figure 1) (Both Viscosity requirements 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 after Thin Film Oven Test | 47 | ---- | 42 | --- | 40 | --- |
| Solubility in trichloroethylene, % by mass | 99.0 | ---- | 99.0 | ---- | 99.0 | --- |

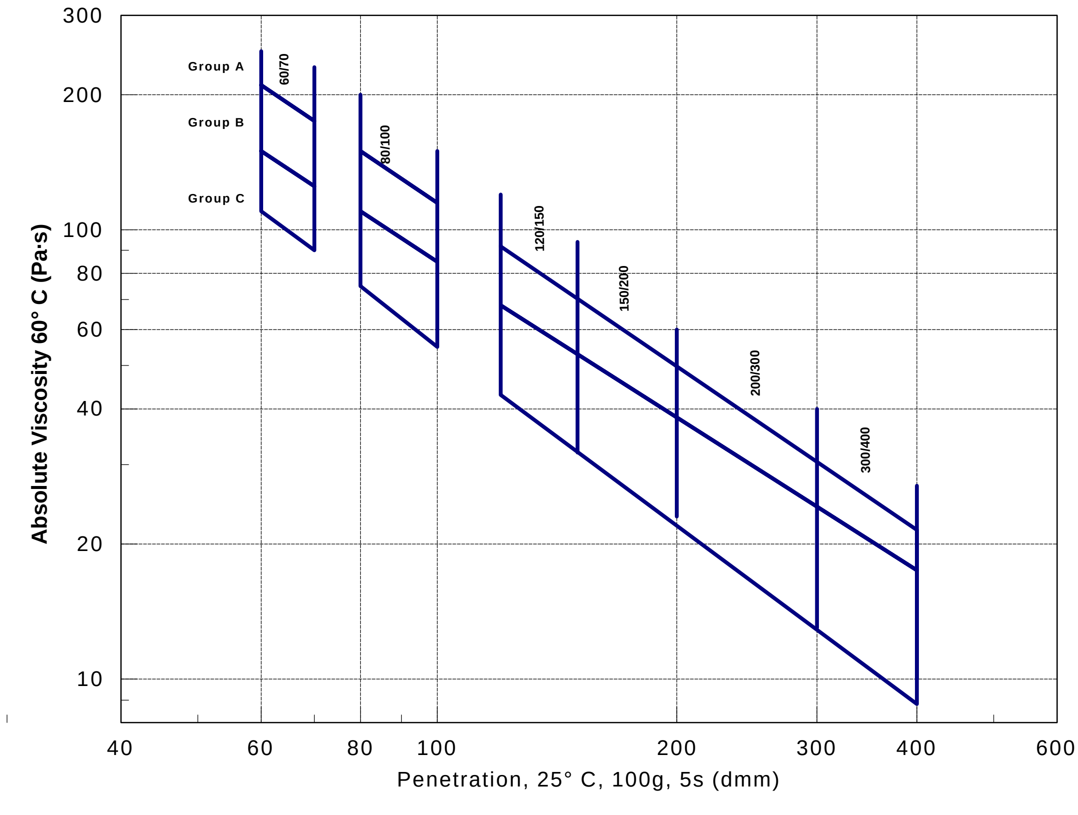



**Figure 1: CGSB Specification for Absolute Viscosity vs. Penetration**

## 2.2 Current Canadian Asphalt Selection Method for Airports

The current asphalt selection method for Canadian airport pavements is based on the airport site freezing index and facility type as specified in the PWGSC manual "Pavement Construction: Materials and Testing"[3]. Table 2 below illustrates the standard requirements for asphalt cements.

**Table 2 Standard Requirements for Asphalt Cements**

| | Asphalt Penetration Grade | |
|---|---|---|
| Site Freezing Index (°C – Days) | Runways, Taxiways and Roads | Aprons |
| <500 | 80-100 | 80-100 |
| 500 – 1400 | 120-150 | 80-100 |
| >1400 | 150-200 | 120-150 |
| For a site freezing index over 500, specify asphalt cement with a high viscosity | | |

As can be seen in Table 2 only three asphalt penetration grades are typically specified for airport construction. A stiffer binder is selected for aprons to address the standing loads. For sites with a freezing index over 500, an asphalt cement with a high viscosity is specified, typically a group A.

Although this asphalt selection method does not incorporate any direct consideration for pavement high temperature performance, it is addressed indirectly since most sites with a high freezing index would generally be expected to experience lower maximum temperatures as well.

# 3.0 SUPERPAVE PG BINDER SPECIFICATION

## 3.1 SUPERPAVE PG Binder Specification

Simply stated, the SUPERPAVE PG binder specification[4] 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 PG Binder specification for a design high pavement temperature up to 58°C and design low pavement temperature down to -28°C.

The SHRP PG binder specifications contain a number of requirements that 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. New requirements include long-term aging, determination of design temperatures, low temperature stiffness and tensile tests, high temperature complex modulus (and phase angle).

### 3.2 SUPERPAVE PG Binder Selection

The SUPERPAVE protocol for selecting a PG binder grade is the determination of the pavement design temperatures for the proposed construction site. The procedure is well documented in the Asphalt Institute's SP-1 manual[5].

A temperature database was developed by SHRP based on data from over 7500 weather stations in Canada and the United States (over 1800 in Canada)[6]. This database will calculate the high pavement design temperature at a depth 20 mm below the pavement surface for any given project location based on the following relationship:

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

where: $T_{20mm}$ = high pavement design temperature at a depth of 20 mm
$T_{air}$ = seven-day average high air temperature
Lat = the project's location in degrees latitude

The concept of reliability can be introduced by adding nΦ to the equation where n is related to the degree of reliability and Φ is the standard deviation of the seven-day average high temperatures in which case $T_{air}$ is replaced by ($T_{air}$ + nΦ).

A secondary criteria for SUPERPAVE high temperature grade selection, intended for highway applications, is based on traffic volumes and traffic speeds. For > 10 million ESALs - increase 1 grade higher; for > 30 million ESALs - increase 2 grades higher; for slow moving traffic - increase 1 grade higher; for stopped traffic - increase 2 grades higher.

Ideally, the low design temperature should be the lowest temperature that the pavement is likely to experience during its design life. Since pavement surface temperatures are rarely available, it is necessary to estimate the lowest expected pavement surface temperature from available air temperature data. The method of estimating the design low pavement temperature from air temperature is a matter of debate. Three algorithms of significance currently exist.

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

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

Work undertaken for the Transportation Association of Canada[8] in 1996 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*Φ) where Φ is the standard deviation and *n* is dependent on the desired reliability.

Substituting the reliability term provides the following form of the equation:

$T_{surf} = 0.859(T_{air}\text{-}n\Phi_{air}) + 1.7$ where: $T_{air}$ = 1-day minimum air temperature

Robertson completed additional work for TAC[9] in 1997. TAC have now published an empirical algorithm developed for the purpose of predicting appropriate low design pavement temperatures for use within the SUPERPAVE system. The TAC algorithm for the low design temperature is:

$$T_{surface} = 0.749\ (T_{air} - n\Phi_{air}) - n\Phi_{surf}$$

where: $\Phi_{surf}$ = 1.5EC
n = multiplier associated with the desired reliability determined using (1-M(n)) = $(1\text{-}R)^{0.5}$ where R is the desired reliability and M(n) is the standard normal probability corresponding to n*Φ for a normal distribution).

The FHWA's Long Term Pavement Performance (LTPP) office has also developed an algorithm for predicting low pavement temperatures based on instrumented LTPP sites [10, 11]. The LTPP algorithm is as follows:

$$T_{surf} = -1.56+0.72\ T_{air} - 0.004(\text{latitude})^2 + 6.26\ \text{Log}\ (H+25) - z(4.4+0.52\Phi_{air}{}^2)^{0.5}$$

where: z is from standard normal probability tables i.e. z = 2 for 98% reliability
H = depth to surface = 0 for surface temperature

## 4.0 DEVELOPMENT OF SELECTION METHODOLOGY FOR AIRPORTS

### 4.1 PG Binder Selection for Low Temperature Performance

Thermal cracking of asphalt pavements is prevalent in most regions of Canada and is the most common pavement distress reported at Canadian airports. Thermal cracking is influenced by the asphalt binder stiffness at low temperature more than any other factor. However, other factors that may influence the amount of cracking are the thickness of the asphalt concrete layer, the type of subgrade material (i.e. clay or sand subgrade), the age of the asphalt concrete, and the reflection of existing cracks through an overlay[12, 13].

The determination of a low design temperature to be used in selection of a binder has been highly debated in Canada since the publication of the SHRP binder selection procedures. In the original selection criteria, the pavement temperature was assumed to be equal to the air temperature which resulted in the selection of cold temperature binder grades that were unreasonable based on Canadian experience. It was identified by Canadian researchers that at cold temperatures, pavement temperatures are generally much warmer than air temperatures.

The three algorithms presented in Section 3 generally result in much warmer predicted pavement temperatures then would be obtained by considering the air temperature alone. A statistical approach is used in each of the models to result in a design temperature based on concepts of reliability. The LTPP model results in very minimal difference from air temperatures (0 + 2Φ) at the more northern areas of Canada and in some cases in the far north, calculates colder pavement than the average air temperature -2Φ. The distribution of low temperature grades for Canadian airport sites using these three algorithms are shown in Figure 2.

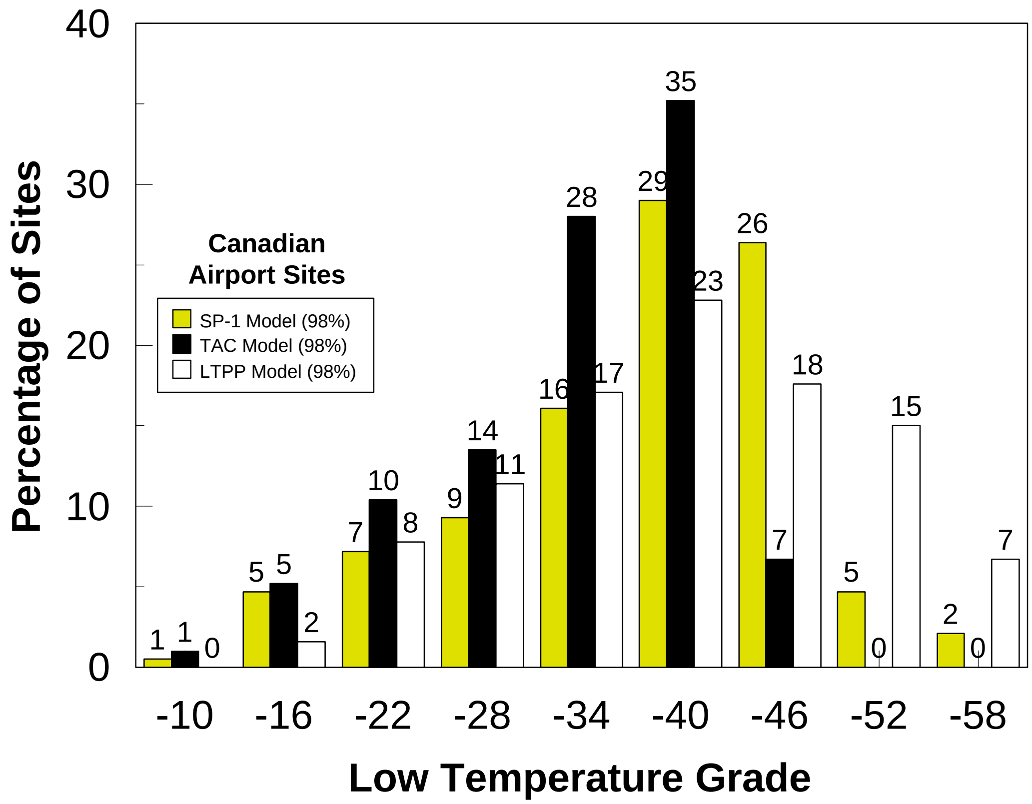



**FIGURE 2: Distribution of Low Temperature Grade for Various Algorithms**

The TAC model represents further work by Canadian researchers and effectively replaces the SP-1 model. The methodology presented in the TAC model for reliability is based on the probabilities associated with variations in both the air temperature and the model's prediction of pavement temperature (i.e. the error of the estimate). This model results in much more 'reasonable' low temperature grades and as such is seen to be the most practical model for initial use at Canadian airports.

The initial study into the suitability of PG Binders for airports[14] determined that there should be little difference with respect to the cold temperature performance of an airport pavement and a highway pavement for an equivalent pavement structure. Since thermal cracking is the predominate distress for Canadian airport pavements, it was recommended that the design cold temperature for airports be determined using the TAC algorithm at a 98% reliability. Special considerations should be made for thin overlays over thermally cracked or PCC pavements.

## 4.2 PG Binder Selection for High Temperature Performance

The selection of the high temperature grade for Canadian airport pavements has been evaluated on the basis of fatigue and instability rutting of the asphalt concrete layers. The work undertaken to develop PG selection guidelines has been carried out under the premise that neither fatigue nor instability rutting is currently a predominant distress mode for airport pavements. This information is seen as significant in the development of guidelines for asphalt grade selection as it provides a high level of confidence in the existing practices of PWGSC.

### 4.2.1 Fatigue of Asphalt Concrete Pavements

Fatigue of asphalt pavement is generally believed to be a function the tensile strain experienced at the bottom of the pavement layer, and the stiffness of the asphalt concrete.

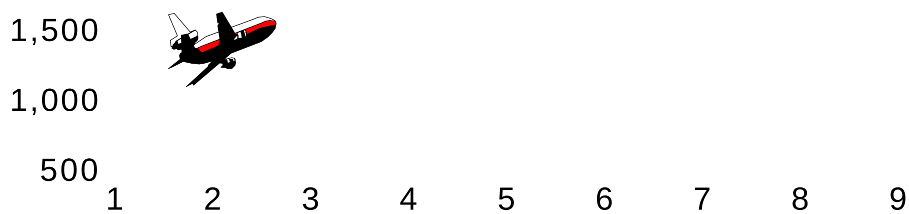
G*Sin(δ) was originally touted as the asphalt parameter to help guard against fatigue. As discussed in the PWGSC[14] 1997 study, this parameter has since been reported to correlate poorly to fatigue life of pavements. The rheological index "R" is currently being considered as the parameter which will provide a measure for fatigue performance. At this time, the FHWA and their Expert Task Groups are examining the potential of the rheological index as a specification measure[15].

Additionally, the "m" value, determined during the BBR testing at low temperatures, is considered important in low temperature fatigue.

The impact the stiffness plays on the fatigue life, including the role of seasonal variation (i.e. G*Sin(δ) and G*/Sin(δ) are tested at a specified temperature), and the significance relative to typical or expected values is difficult to quantify. There are conflicting discussions in the literature on the ability to relate binder properties to actual fatigue performance. For the purposes of this study, conventional relationships between asphalt and mix stiffness have been used to examine fatigue.

Example calculations were undertaken using a B737-300 aircraft loading on two "equivalent" structures. These calculations indicate that, for large number of repetitions, the fatigue life is greatly affected by the HMAC modulus value and also by the thickness of the HMAC in the structure[1] as shown in Figure 3.

Additional examples were examined for lighter loading conditions. For lighter loads (and appropriately lighter structures), fatigues lives are very much higher and exhibit the maximum fatigue resistance at a given HMAC modulus which changes significantly with HMAC thickness. Generally, the fatigue life calculated far exceeds the expected number of load repetitions.


1,500
1,000
500
1
2
3
4
5
6
7
8
9

Relationships between the stiffness of the asphalt binder and the asphalt concrete mix have been reported in the literature[17, 8] and include additional variables such as asphalt volume, air voids, and the percent passing the number 200 sieve size. Aggregate characteristics such as angularity, fractures and texture also provide a significant influence.

While the published research findings and experience have long supported the relationship of binder stiffness to rutting, it is also apparent that the site-specific environment, mix characteristics, and loading conditions must all be defined in order to quantify the affect of any parameter.

Work conducted by Morris for the Brampton Test Road[18] used laboratory testing to define the mixture characteristics and predict rutting. The basic form of the rutting relationship given in this work was: $\varepsilon_p = f(\sigma_1, \sigma_3, T, N)$, in which $\sigma_1$ and $\sigma_3$ are the vertical and horizontal stresses respectively, T is the temperature and N is the number of repetitions of the load.

Morris' work is significant in that the model developed directly references the loadings. Other models, such as that presented with the VESYS computer program, reference load less directly[19]. The VESYS rutting model is based on layer deflection which is a function of the HMAC modulus and the applied loading. (The model also considers permanent deformation in the base and subgrade layers).

Full scale testing is currently ongoing to validate the existing SUPERPAVE binder stiffness requirements relative to pavement rutting. The FHWA[19, 20] reported on work undertaken using accelerated loading, to, in part, "confirm that the binder properties identified by SHRP research as determinants of pavement performance are significant". This work identified some relationship between $G^*/\mathrm{Sin}(\delta)$ and rutting which was utilized to provide an indicator as to the affect of $G^*/\mathrm{Sin}(\delta)$ on rutting for increasing loadings.

The observations were summarized by Bonaquist[20], as "... at low values of $G^*/\mathrm{Sin}(\delta)$ the rutting is very sensitive to small changes in binder properties ... high values of $G^*/\mathrm{Sin}(\delta)$, the rutting behaviour is relatively insensitive to changes in binder properties. The SUPERPAVE specification limit of 2.2 kPa appears to be in the transition area between these two zones". The review undertaken for the PWGSC[14] study agreed with this finding and further identified that the sensitivity of rutting to $G^*/\mathrm{Sin}(\delta)$ increased with increasing loadings.

The FHWA accelerated loading project[19] stresses that the relationships determined are applicable to conventional binders. Similar findings were also reported by Australian researchers[21]. That is, a relatively good correlation was determined between conventional asphalts $G^*/\mathrm{Sin}(\delta)$ and wheel tracking results, however modified asphalts did not correlate well.

**Figure 4: Approximate Relationship between Rutting and G*/Sin(δ)**

### 4.2.3 Basis of Proposed Selection Criteria for High Temperature

#### 4.2.3.1 Overview

As stated at the beginning of this Section, high temperature performance in the form of rutting or fatigue has not been a significant distress on Canadian airport pavements. As a result of this observation it can generally be assumed that it will not be necessary (nor desirable) to utilize stiffer asphalts than have historically been utilized by PWGSC. To establish specific guidelines for selecting PG asphalt grades for Canadian airports, the specific grades currently used provided significant information.

For example, in Alberta, many local airports were originally constructed using 300-400A asphalt by the Provincial highway department (Alberta Transportation and Utilities). The performance of these airport pavements has been acceptable relative to high temperature related distress (distress has most typically been related to aging, raveling, cracking etc). These observations lend confidence to the use of softer asphalts for local low-loaded airports, than has previously been recommended by PWGSC.

A survey of regional offices completed by PWGSC[22] indicates that Canadian practice has generally followed PWGSC guidelines or, in some areas, used softer CGSB asphalt grades. However, it must also be noted that the softer asphalts (150-200A) have been used in regions which would not typically have significant numbers of the larger jet aircraft. The 150-200A grade was the softest asphalt reported by the PWGSC regional offices.

#### 4.2.3.2 Loadings

The Canadian airport pavement thickness design methodology[23, 24] is based upon on a series of twelve standard gear loadings (SGL) which span the range of current aircraft loadings. The aircraft characteristics which determine the SGL includes gear configuration, gross loadings, and tire pressure.

In the study[14], the loadings of typical aircraft in each of the SGL groups were examined. The loadings of SGLs 1 through 4 were considered to be of the same magnitude as might be achieved from highway truck traffic. (That is, tire pressures are within the realm of truck traffic, and gross loads, when considered in regards to the fourth power rule for truck loadings, resulted in a maximum of approximately seven equivalent single axle loads). The first grouping, because of the loose relationship to truck loadings, foreshadowed the potential of using the basic SUPERPAVE selection methodology for this grouping.

Similarly, the second and third levels defined for the SGLs attempted to distinguish logical groupings. It was noted it would probably be justified to define four groupings, however, recognizing that such further grouping would not result in different asphalt grade selections, two further groupings were selected.

A fourth level was also identified which is intended to address apron applications for high frequency or standing (i.e. apron) Level 3 loadings.

| PWGSC Standard Gear Loadings (SGL) | Load Group |
|---|---|
| 1 to 4 | Level 1 |
| 5 to 8 | Level 2 |
| 9 to 12 | Level 3 |
| High Frequency or Apron Applications | Level 4 |

#### 4.2.3.3 Affect of Reliability

A review of available temperature data for Canadian weather stations, and particularly for Canadian airport weather stations was undertaken for the purpose of evaluating the grades that would be considered appropriate based on existing SUPERPAVE criteria. This review identified that the reliability selection made no difference in the high temperature grading in almost half the Canadian sites.

There are two issues affecting this, firstly, the standard deviation is relatively low and as a result 2 times the standard deviation for 98% reliability typically does not result in more than about 2 to 4°

The use of 50% reliability was determined to be reasonable for level 1 load applications. Moving to Level 2 loading conditions from Level 1 requires the design temperature to increase by 1σ in the $T_{20mm}$ determination. The design temperature is based on the greater of the calculated $T_{20mm}$ (1σ) or 46°C plus an additional 5°C.

Moving to Level 3 loading conditions (the heaviest gear loadings), a similar rationale was applied in that modifications to the high design temperature were made on the basis of the greater of the 98% (i.e. 2σ) reliability $T_{20mm}$ temperature or 46°C. A temperature increase of 9°C was applied to the $T_{20mm}$ (98% reliability) high design temperature.

Level 4 high design temperature asphalt grade is determined on the basis of $T_{20mm}$ 99.999% (i.e. 4σ) or 46° whichever is greater, plus 12°C.

The following table summarizes the proposed criteria for selecting the high design temperature grades for PG binders for Canadian airport pavements.

| Level | Standard Gear Loading | Design High Temperature |
|---|---|---|
| 1 | 1 - 4 | $T_{20mm}$ 50% Reliability |
| 2 | 5 - 8 | Greater of { $T_{20mm}$ 83% (1σ) Reliability or 46°} + 5°C |
| 3 | 9 - 12 | Greater of { $T_{20mm}$ 98% (2σ) Reliability or 46°C} + 9°C |
| 4 | High Frequency or Apron | Greater of { $T_{20mm}$ 99.999% (4σ) Reliability or 46°C} + 12°C |

## 5.0 Validation

To validate this binder selection methodology and develop binder usage guidelines, PG binder test sections have been constructed for long term monitoring. The first test section was a 50 mm overlay constructed in 1997 at the Jean Lesage International Airport in Quebec City where a CGSB 120/150A (PG58-28) will be compared with a PG64-34.

A PG binder test section has also been incorporated into new construction at the Calgary International Airport in 1998. At this site a CGSB 150/200A (PG58-28) is being compared with a PG58-34. The pavement structure has been instrumented with thermocouples to monitor the temperature at various depths within the pavement structure. This temperature data will be used to monitor the in-service pavement temperatures and the temperature gradient throughout the width and depth of the pavement structure, such that the existing low pavement temperature algorithms can be validated for a Canadian airport pavement with a deep granular structure.

## 6.0 CONCLUSIONS

Performance graded asphalt cement binder specifications are expected to become the asphalt cement standard in Canada.

The use of PG Binders for Canadian airport pavements will allow pavement designers to more accurately predict pavement performance and optimize the selection of an asphalt binder to extend the service temperature range, maximize pavement life, minimize pavement maintenance costs and reduce facility closures.

By considering aircraft loading in the asphalt selection methodology, Canadian airport pavement designers will be able to select the most economical PG grades for the expected aircraft loading at a given site.

The TAC algorithim for estimating the low pavement temperature should be used to determine the low pavement design temperature using a reliability of 98%. For overlays over thermally cracked pavements or PCC pavements the grade selection should take reflective cracking into consideration.

Long term monitoring of airport PG binder test sections will be used to validate the binder selection methodology and the long term performance of PG binders under typical airport operations.

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