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Final Report

**SECOND PHASE STUDY OF CHANGES IN
IN-SERVICE ASPHALT
Volume I: Main Report**

**Khaled A. Galal
Thomas D. White
Adam J. Hand**

June 2000

Indiana
Department
of Transportation

Purdue
University

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**SECOND PHASE STUDY OF CHANGES IN IN-SERVICE ASPHALT
VOLUME I: MAIN REPORT**

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The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration and the Indiana Department of Transportation. This report does not constitute a standard, specification or regulation.

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16. Abstract In response to the need to improve asphalt pavement performance, the Strategic Highway Research Program (SHRP) was initiated in 1987. Products from SHRP included performance-based specifications for asphalt binders and performance-based mix design tests and criteria for asphalt mixtures. The Superpave asphalt binder specification addresses performance of in-service pavements for given environmental and traffic conditions. During the 1985 paving season in Indiana, the original asphalt binder and HMA (truck mix samples) from a number of pavements were sampled and retained. Cores were also taken from these pavements after seven or eight years of service. Condition surveys were performed on these pavements, prior to coring. The study objectives were directed toward characterizing the original and extracted/recovered asphalt binders from the field samples using Superpave test methods and equipment. Taking this approach provided an early opportunity to examine a number of features of the Superpave system, including short (RTFO) and long-term (PAV) aging. Results of this research documented the difference in stiffness between RTFO aged and asphalts extracted and recovered from loose mixtures; in addition to, the difference in stiffness between PAV aged and in-service aged asphalts, in Indiana. Additionally, asphalt stiffness was related to pavement performance. A poor relationship was found between the stiffness of RTFO aged asphalts and rutting. On the other hand, a good relationship was found between stiffness of in-service asphalts and cracking. Collected weather data from seven counties indicated agreement between calculated PG grades for these counties and recommended Superpave PG grade, based on the environmental conditions. Based on the results of this research, recommendations were made to further investigate the short-and long-term binder aging to better address variability and reliability of these procedures, to address storage conditions sometimes unavoidable for field samples, and to validate PG grades requirements for Indiana.			
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CHAPTER 1

Introduction

1.1 Background

Hot mix asphalt (HMA) pavements exhibit various types of distresses including permanent deformation (rutting), fatigue cracking, and low temperature cracking. In the last two decades, the reported occurrence of these distresses has increased in the U.S. and Canada (Nelson and Wood 1990). Asphalt technologists have associated the increase with changes in asphalt binder characteristics because of new crude sources resulting from the trade embargo in the early 1970s or changes in refining (Nelson and Wood 1990). The increased distress has also been associated with growing traffic volume and increased wheel loads (Badaruddin and White 1993).

In response to the need to improve asphalt pavement performance, the five year Strategic Highway Research Program (SHRP) was initiated in 1987 (Peterson et al, 1994 and Kennedy et al, 1994). Products from SHRP included performance-related specifications for asphalt binders and performance-related mix design tests and criteria for asphalt mixtures (Kennedy et al 1994). Those performance-related specification tests and criteria were combined into the Superior Performing Asphalt Pavement

(Superpave) system. The Superpave asphalt binder specification addresses performance of in-service pavements for given environmental and traffic conditions. Performance factors include permanent deformation, fatigue cracking, and low temperature cracking (Peterson et al, 1994).

The Superpave asphalt binder specification includes a series of tests for measuring the rheological characteristics of asphalt binders that were considered to be related to pavement construction and performance (McGennis, Shuler, and Bahia, 1994). In addition, the Superpave asphalt binder specification tests are performed at temperatures experienced by the in-service pavements. The asphalt binder tests are conducted at three different stages of binder aging:

1. Tests performed on the original asphalt binder represent characteristics during storage, transport, and handling;
2. Tests performed on the binder aged in the Rolling Thin Film Oven (RTFO) test represent potential changes in binder characteristics generated during production and construction (mix production, laydown, and compaction); and
3. Tests performed on the binder after initial aging in the RTFO and subsequent conditioning (aging) in the Pressure Aging Vessel (PAV) represent expected in-service asphalt binder characteristics (Asphalt Institute, SP-1, 1994).

During the 1985 paving season in Indiana, original/neat (i.e. original unaged/unmodified) asphalt binder, fuel oil, and HMA (truck samples) was sampled from a significant number of paving projects in Indiana. In 1988, a limited number of those projects were cored. Tests were conducted on the original asphalt binders and on asphalt binder extracted and recovered from the original HMA and subsequent cores taken from the in-service pavements (Nelson and Wood 1990). A second evaluation was initiated in 1992. It was planned that the group of pavements cored in 1988 would be re-cored. Unfortunately, none of the pavements were available because they had been overlaid or rehabilitated in 1992/1993.

As the project evolved, equipment to conduct the Superpave asphalt binder tests were generally standardized and became available. As a result, the study was directed toward using SHRP developed tests to characterize the original and extracted and recovered asphalt binders from the field sampled HMA and in-service cores. This approach provided an early opportunity to examine a number of features of the Superpave system including short and long term aging and criteria related to pavement performance.

1.2 Problem Statement

One of the primary concerns in the State of Indiana is the early appearance of flexible pavement distresses. The early distresses may result in pavement failure and/or shorter pavement service life. The recently developed SHRP Superpave asphalt

binder specifications are directed toward in-service pavement performance. Superpave tests and associated criteria were selected to control distresses that affect in-service HMA performance, specifically: permanent deformation, fatigue cracking, and low temperature thermal cracking. The Indiana Department of Transportation (INDOT) was interested in answering the question of whether or not the changes in in-service asphalt can be related to pavement distresses. The previous study, "The Effect of the Bag House Fines and Incomplete Combustion Products in a Drum Dryer on the Characteristics of Asphalt Paving Mixtures, Phase I", by Nelson and Wood, was conducted in 1990 to study in-service asphalt changes. However, none of the pavements that were studied at that time exhibited any distresses. The second phase study of in-service asphalt was initiated to document asphalt binder characteristics after seven to eight years of service. Several additional factors were included in this new study. The factors included the type and severity of distresses as documented by pavement condition surveys. Also, as noted, Superpave tests were conducted on the original and the recovered in-service binders.

1.3 Research Objectives

The Rolling Thin Film Oven (RTFO) and newly developed Pressure Aging Vessel (PAV) tests were adopted for aging asphalts in the Superpave system. During SHRP there was limited verification that the aging induced by the tests effectively simulated in-service aging. In the case of the PAV, the preliminary results were generalized or extrapolated to represent five to ten years of in-service asphalt aging.

Since the asphalt binder specification criteria are set to limit various distresses based on the expected aging, it is important that the expected aging from these adopted laboratory tests be verified. The current research has provided an opportunity for verification without waiting an additional five or ten years for new pavements to age. In addition, it is important to relate the different types of distresses with pavement performance. One important aspect is how the changes of in-service asphalts relate to distresses such as permanent deformation, fatigue cracking, and low temperature thermal cracking. Thus, the major objectives of the current research are:

- To classify asphalt binders (AC-20 viscosity grade) retained from the 1985-construction season using the Superpave performance grading (PG) system.
- To relate the extracted and recovered asphalt binder characteristics of the original truck mix samples to those of the original binders after RTFO aging.
- To relate characteristics of the extracted and recovered asphalt binders from field cores after seven to eight years in-service to the original asphalt binders subjected to combined RTFO and PAV aging.
- To evaluate the Superpave asphalt binder specifications as reflected by the results of this study.

1.4 Research Organization

This research is organized as follows: Chapter 1 is an introduction and Chapter 2 is a literature review. Chapter 3 describes the distress survey performed on the in-service sections included in this research. Chapter 4 describes the Superpave test methods, associated equipment and specifications, and Chapter 5 describes the design of experiment for this research. Chapter 6 presents the results and analyses. A discussion of the results and analyses are also presented in Chapter 6. Chapter 7 provides a summary of the research, conclusions of the results and analyses, and recommendations for future research.

CHAPTER 2

Literature Review

2.1 Introduction

Early distress reduces pavement service life and costs tax-payers money. Asphalt pavement distresses are related to the asphalt binder characteristics. Therefore, identifying changes in asphalt binders that take place in-service is important.

This chapter addresses asphalt binder specifications currently employed in the United States and Canada. Asphalt binder properties and their relation to pavement performance are discussed also. A discussion is also provided of distresses that are related to in-service pavement performance and therefore Superpave asphalt binder specifications.

2.2 Review of Current Asphalt Binder Specifications.

Penetration-graded asphalt specifications were adopted in the early 1930s by the American Association of State Highway Officials (AASHO) (Roberts et al, 1991). Viscosity-graded specifications were adopted in the 1960s. Also during the 1960s, a different viscosity-graded specification, based on the aged residue from the rolling thin film oven test, was developed by California Department of Transportation

(CALTRNAS), (Roberts, et al, 1991). The specifications included requirements for the original asphalt as well as for the residue from thin-film oven (TFO) aging and/or the residue from the RTFO test as developed by CALTRANS. Penetration and viscosity graded specifications will be reviewed in the following subsections.

Penetration ASTM D 946

The penetration test (ASTM D 5) is used to grade asphalt based on the penetration of a standard needle into a sample of asphalt (ASTM D5). The test is conducted at 25°C (77°F) which approximates the average in-service pavement temperature in the United States. The penetration test is usually considered to be a relative index test. The penetration based specifications, ASTM D 946, Standard Specification for Penetration-Graded Asphalt Cement for use in Pavement Construction, incorporates five penetration grades and requires minimum and maximum ductilities at the same temperature. Minimum values of both penetration and ductility after aging (TFOT) are also specified.

Advantages of the penetration test are that the test is fast and easy, can be used to detect asphalt cement contamination, and can be used to determine asphalt cement temperature susceptibility at intermediate temperature ranges (Roberts et al, 1991).

On the other hand, disadvantages are that the penetration test can not be used to predict in-service pavement performance at lower or higher service temperatures. Field experience and long-term observations are required to relate the penetration to pavement performance (Peterson et al, 1994 and Anderson et al, 1991).

Viscosity (ASTM D 3381)

There are several devices used to measure asphalt binder viscosity. They include the sliding plate viscometer, capillary tube, and rotational viscometer. Viscosity-graded specifications are based on the viscosity of the original asphalt cement measured at 60°C (140°F) (ASTM D3381, Tables 1 and 2) or the viscosity of the asphalt cement after aging in the RTFO Test (ASTM D3381, Table 3). Viscosity is a fundamental engineering property rather than a relative index, such as the penetration test. Some agencies amend ASTM D3381 with minimum penetration and ductility requirements at 25°C (77°F). The agencies may also require minimum and/or maximum viscosities at 135°C (275°F) before and after RTFOT aging. Cited advantages of the viscosity-grading system include the fact that the viscosity together with the penetration and ductility measurements cover a wide range of intermediate and high temperatures (25°C to 60°C). At 60°C, the viscosity of the asphalt cement represents a reasonable maximum HMA pavement surface temperature in the United States. The aged residue from RTFO conditioning approximates the asphalt properties during mixing, laydown, and compaction (Roberts et al, 1991). A disadvantage of the viscosity grading system is that there is a lack of low temperature characterization.

In summary, both penetration and viscosity measurements are inadequate to fully characterize the asphalt cements over a full range of in-service temperatures. There is also a need for tests that are able to define the characteristics of asphalt cements in fundamental engineering units (Anderson and Kennedy, 1993).

2.3 Review of Asphalt Properties: Relationship to Pavement Performance

It has been well established that HMA pavement performance is affected by rheological properties of both asphalt binder and asphalt concrete mixture. Several studies have addressed this concept. For example, Tia and Ruth, 1985, stated that the rheology of bituminous materials and bituminous mixtures defines the response of the material when subjected to loads.

In the following section, asphalt rheology will be defined and reviewed. Subsequently, asphalt properties related to pavement performance will also be reviewed.

Asphalt Rheology

Rheology of asphalt binders can be defined as the study of the time-temperature response of the asphalt when subjected to stress (Roberts et al, 1991). Several researchers have attempted to tie asphalt rheology to properties of the asphalt mixtures and in-service HMA pavement performance. Kandhal and Koehler, 1985, recommended optimization of rheological properties of asphalt binders and mixtures to minimize both load and non-load associated cracking of HMA pavements. Additionally, the same authors stated that both load and non-load associated cracking of HMA pavements cause deterioration of in-service pavements. Roque et al, 1985, reported that asphalt mixture behavior at low temperature depends on the rheological properties of the asphalt binder. In a 1990 literature review, Finn et al, summarized both chemical and physical asphalt binder properties that are believed to influence pavement performance. The properties are reviewed in the following subsections.

Penetration and Viscosity

Penetration and viscosity of original asphalt, aged residue from the TFO (short-term aging) test, and in-service asphalt (long-term aging) were of concern when studying the performance of the HMA pavements.

Results from the Zaca-Wigmore test road constructed in 1954-1955 indicated that recovered asphalt penetration decreased with increase in percent fatigue cracking (Hveem et al 1959; Zube and Skog 1969; Skog 1959, 1981). Additionally, fatigue failure was associated with a penetration of 30. In the Ste. Anne test road constructed in Manitoba in 1967 sections constructed with higher penetration asphalts exhibited less transverse cracking (Burgess et al. 1971, 1972; Gaw et al. 1974; Deme and Young 1987). Moreover, performance data collected in Pennsylvania indicated lower recovered asphalt penetration was associated with increased cracking (Kandhal, Mellott and Busso, 1984).

On the other hand, field tests in Manitoba and Ontario in 1967 indicated that pavement built with air blown, low viscosity asphalts experienced less low temperature cracking after four years (Gaw et al 1976). Field tests in Alberta suggested that a minimum viscosity of 275 poises at 140°F to be adopted (Anderson et al 1966). Data from the Zaca-Wigmore asphalt test roads suggested that the level of recovered viscosity is related to air void content in the compacted mix. A study of pavement in

West Texas indicated that the frequency of transverse cracks increased with viscosity at 60°C (Benson 1976; Anderson and Epps 1983).

Temperature Susceptibility

Temperature susceptibility can be defined as the change in consistency, stiffness, or viscosity of asphalt binder when subjected to temperature variation (Roberts et al, 1991). Commonly, temperature susceptibility is calculated from consistency measurements such as penetration and/or viscosity, at two different temperatures. Such a difference is referred to as a temperature susceptibility index. The measures will be confounded with loading times. For example, when viscosity and penetration and/or softening point and penetration are used to create a temperature index, the loading times will not be similar. Therefore, when characterizing an asphalt binder, one should be able to fully separate the rheological parameters, stress-strain as a function of loading time and temperature (Anderson et al 1991, 1993, and 1995).

Based on the Ste. Anne test road in 1967, it was concluded that “the temperature susceptibility of the asphalt had a significant effect on low temperature cracking” and as temperature susceptibility decreases pavement becomes less prone to exhibit transverse cracking (Finn et al 1990).

Stiffness

Asphalt binder stiffness can be defined as the stress-strain relationship as a function of loading time and temperature (Roberts et al 1991). This relation is also

used to represent the rheological behavior of asphalt and asphalt concrete mixtures. Studies have shown that higher asphalt stiffness is desired at high service temperatures to guard against rutting (permanent deformation), while lower asphalt stiffness is desired at low service temperatures to enhance resistance to low temperature cracking (Finn et al 1990). Generally, nomograms are used to determine asphalt cement stiffness at various test temperatures and loading times (Peterson et al 1994). Van der Poel in 1954 developed the first such nomograph using the penetration index. In 1972, McLoud updated the nomograph to accommodate both penetration and viscosity measurements. However, Peterson et al, in 1994 reported poor prediction reliability at low temperature and doubtful applicability to modified asphalt binders as two primary reasons to consider a direct method of measuring of low temperature stiffness of asphalt binders.

2.4 Summary of Asphalt Properties and Their Relationship to Pavement Performance (Pre-SHRP)

Asphalt properties that had been identified as being related to pavement performance prior to SHRP are summarized in the following paragraph:

Increased viscosity is associated with low temperature cracking, fatigue cracking, and aging. No correlation was presented relating viscosity and rutting. Lower penetration and ductility values were associated with low temperature cracking, fatigue cracking and excessive aging. No correlation was presented relating penetration and rutting. An increase in mixture stiffness and asphalt stiffness is associated with low temperature cracking. Higher asphalt temperature susceptibility is

associated with cracking. Lower asphalt temperature susceptibility is associated with less aging and cracking, but with increased rutting (Finn et al, 1990).

2.5 Review of SHRP- Superpave Asphalt Binders Specifications

In the following subsections, the need for a performance based specification, development of the Superpave specification and description of the Superpave Performance Graded (PG) asphalt specification, as well as their relation to pavement performance are discussed.

2.5.1 Need for SHRP Performance Based Specifications

It has been well documented that the penetration and viscosity specifications are not appropriate to fully characterize asphalt binders over the typical range of in-service pavement temperatures. Moreover, the associated test methods lead to mostly empirical values that cannot be related to HMA pavement performance in a rational fashion, other than through long term observation or experience (Peterson et al 1994, Anderson et al 1994, and Anderson and Kennedy 1993).

The previous authors have justified the growing need for a new performance related asphalt specification. In general, the new specifications should address several issues. Specification tests should be directly related to HMA performance and there should be no overlap of binder grades. For example, in a comparison of penetration and viscosity grading systems, two types of asphalt cements can be included in the same grade, even though, they may have different temperature susceptibility characteristics, see Figure 2-1. Current specifications do not account for long term

field (in-service) characterization of asphalt binders and they should. Finally, asphalt binders are not fully characterized over the whole range of expected in-service temperatures (low, intermediate, and high temperatures) in current specifications; they are particularly inadequate with respect to low temperature characterization.

2.5.2 SHRP Research Related to Development of PG Asphalt Specifications

In the early 1980's, the increase in premature pavement failures contributed to the initiation of the Strategic Highway Research Program (SHRP), (Peterson et al, 1994). Officially the SHRP program was implemented in October of 1987 and ended in March of 1993.

Figure 2-1 Viscosity/Penetration Consistency measurements
(After FHWA, 1996).

The products of this research were a performance based asphalt binder specification and a HMA mixture design and analysis system. Discussion of the mixture design and analysis system is beyond the scope of this work.

Long term evaluation of these specification will be accomplished through the Long Term Pavement Performance Program (LTTP), Leahy et al, 1994 and Lytton et al, 1994). The LTTP program includes the General Pavement Studies (GPS) and the Special Pavement Studies (SPS) (Leahy et al, 1994).

2.5.3 Description of Superpave Asphalt Binder Specification

The Superpave asphalt binder specification is applicable to both modified and unmodified asphalts (Kennedy, et al 1994). In addition, the Superpave asphalt binder specification utilizes two important engineering properties: the response to traffic and environmental loading (Anderson and Kennedy, 1993). Furthermore, the Superpave specification addresses three binder related in-service pavement distresses: permanent deformation (rutting), fatigue cracking, and low temperature cracking.

The Superpave asphalt binder specification is based on three aging regimes of the asphalt; original un-aged binder, short-term aging, and long-term aging. These aging regimes represent the asphalt binder during three basic stages:

1. Tests on the original binder represent the binder characteristics during storage, transport, and handling;
2. Tests on short-term aged binder, the binder after conditioning in the Rolling Thin Film Oven (RTFO) test, are intended to represent changes in the binder

characteristics due to aging in the construction process (mix production, lay down, and compaction); and

3. Tests on long-term aged binder, the binder after conditioning in both the RTFO and the Pressure Aging Vessel (PAV), are intended to represent changes in binder characteristics due to several years of in-service aging.

The environmental conditions, short- and long-term aging, and distress criteria as they apply to Superpave asphalt binder specifications will be reviewed. The associated test methods will be discussed in a separate chapter. The Superpave asphalt binder specifications and the associated criteria are illustrated in Table 2-1.

Table 2-1. Performance-Graded Asphalt Binder Specification (After Asphalt Institute, 1995)

PERFORMANCE GRADE	PG 46-				PG 52-				PG 58-				PG 64-			
	34	40	46	<46	10	16	22	28	34	40	46	52	58	64	<64	>64
Average 7-day Maximum Pavement Design Temperature, °C ^a																
Minimum Pavement Design Temperature, °C ^a	>34	>40	>46	>46	>10	>16	>22	>28	>34	>40	>46	>52	>58	>64	>64	>64
ORIGINAL BINDER																
Flash Point Temp, T48: Minimum °C	230															
Viscosity, ASTM D4402: ^b Maximum, 3 Pa·s, Test Temp, °C	135															
Dynamic Shear, TP5: ^c G'/sinδ, Minimum, 1.00 kPa Test Temp @ 10 rad/s, °C	46				52				58					64		
ROLLING THIN FILM OVEN (T240) OR THIN FILM OVEN RESIDUE (T179)																
Mass Loss, Maximum, percent	1.00															
Dynamic Shear, TP5: ^c G'/sinδ, Minimum, 2.20 kPa Test Temp @ 10 rad/s, °C	46				52				58					64		
PRESSURE AGING VESSEL RESIDUE (PAV)																
PAV Aging Temperature, °C ^d	90				100				100				100			
Dynamic Shear, TP5: ^c G'/sinδ, Maximum, 5000kPa Test Temp @ 10 rad/s, °C	10	7	4	25	22	19	16	13	10	7	25	22	19	16	13	10
Physical Hardening ^e	Report															
Creep Stiffness, TP1: ^f S, Maximum, 300 Mpa, m-value, Minimum, 0.300 Test Temp @ 60s, °C	-24	-30	-36	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	-36
Direct Tension, TP3: ^g Failure Strain, Minimum, 1.0% Test Temp @ 1.0 mm/min, °C	-24	-30	-36	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	-36

^a Pavement temperatures are estimated from air temperatures using an algorithm contained in the SUPERPAVE software program, may be provided by the specifying agency, or by following the procedures as outlined in PPX.

^b This requirement may be waived at the discretion of the specifying agency if the supplier warrants that the asphalt binder can be adequately pumped and mixed at temperature that meet all applicable safety standards.

^c For quality control of unmodified asphalt cement production, measurement of the viscosity of the original asphalt cement may be substituted for dynamic shear measurements of G'/sinδ at test temperatures where the asphalt is a Newtonian fluid. Any suitable standard means of viscosity measurement may be used, including capillary or rotational viscometry (AASHTO T201 or T202).

^d The PAV aging temperature is based on simulated climatic conditions and is one of three temperatures 90°C, 100°C or 110°C. The PAV aging temperature is 100°C for PG 58- and above, except in desert climates, where it is 110°C.

^e Physical Hardening – TP1 is performed on a set of asphalt beams according to Section 13.1, except the conditioning time is extended to 24 hrs ± 10 minutes at above the minimum performance temperature. The 24-hour stiffness and m-value are reported for information purposes only.

^f If the creep stiffness is below 300 MPa, the direct tension test is not required. If the creep stiffness is between 300 and 600 MPa the direct tension failure strain requirement can be used in lieu of the creep stiffness requirement. The m-value requirement must be satisfied in both cases.

2.5.4 Environmental Temperature Requirements of the Superpave Specifications

As shown in Figure 2-2, the Superpave asphalt binder specification is based on stiffness of the aged asphalt binder under specific traffic loading and environmental conditions (Kennedy, et al, 1994). Criteria are specified for three design temperatures: high, intermediate, and low temperatures. The annual high temperature is the average of the hottest consecutive seven-day period of temperatures at a site, over a 20-year period. The annual low temperature is the minimum pavement temperature at the site, over a 20- year period. These data are analyzed and high and low design temperatures are selected based on expected values and a desired level of reliability.

Superpave Asphalt Binder Specification

■ Grading System Based on Climate

Figure 2-2 Grading System Based on the Environmental Conditions (After FHWA,1996).

Actual pavement temperatures can be determined by installing sensors in the pavement. As an alternative, the appropriate air temperatures, high and low, can be converted into a pavement or site temperature using the following relationships (Kennedy, et al., 1994).

Pavement surface temperature may be computed as a function of several factors using the following equation:

$$1331 \alpha \tau_a (1/\cos z) \cos z + \varepsilon_a \sigma T_a^4 - h_c (T_s - T_a) - 164k - \varepsilon \sigma T_s^4 = 0 \quad \text{Equation (2-1)}$$

where

α	=	Pavement surface absorptivity
τ_a	=	Transmission Coefficient for Air
z	=	Latitude - 20°
ε	=	Pavement surface emissivity
σ	=	Stefan-Boltzman Constant (5.67×10^{-8} watts per $m^2 \text{ } ^\circ K^4$)
h_c	=	Surface coefficient of heat transfer (watts per $m^2 \text{ } ^\circ C$)
k	=	Thermal conductivity coefficient (watts per $m \text{ } ^\circ C$)
T_a	=	Air Temperature ($^\circ K$)
T_s	=	Surface Temperature ($^\circ K$)

Huber, 1994, simplified the equation to obtain high pavement surface temperature as follows:

$$\begin{aligned} \tau &= 0.80 & \alpha &= 0.9 & k &= 1.38 \text{ watts/m}^\circ C \\ \varepsilon &= 0.9 & h_c &= 19.88 \text{ watts/m}^2 \text{ } ^\circ C \end{aligned}$$

$$T_s = T_a + 0.00618\phi^2 + 0.2289\phi + 24.4 \quad \text{Equation (2-2)}$$

Where T_s = surface temperature, $^\circ C$
 T_a = air temperature, $^\circ C$
 ϕ = latitude, degrees

Also, the maximum pavement temperature at a depth “d”, which is the depth below the pavement surface, can be computed by the following equation (Kennedy et. al., 1994):

$$T_{d(max)} = [T_{s(max)} + 17.8][1 - 2.48(10^{-3})d + 1.085(10^{-5})d^2 - 2.441(10^{-8})d^3] - 17.8$$

Equation (2-3)

Where $T_{d(max)}$ = pavement temperature at depth, °C
 $T_{s(max)}$ = pavement surface temperature, °C
 d = depth from surface, mm

This equation was simplified for surface mixtures at a depth of 20mm, as follows (Kennedy et. al., 1994):

$$T_{20(max)} = 0.955 T_{s(max)} - 0.8$$

Equation (2-4)

$T_{20(max)}$ = temperature at 20 mm, °C
 $T_{s(max)}$ = temperature at surface, °C

Where as the minimum pavement temperature at depth “d” is computed as follows (Kennedy et. al., 1994):

$$T_{d(min)} = T_{s(min)} + 5.1 (10^{-2})d - 6.3 (10^{-5})d^2$$

Equation (2-5)

Where $T_{d(min)}$ = pavement temperature at depth d, °C
 $T_{s(min)}$ = pavement temperature at the surface, °C
 d = depth below the surface, mm

One of the advantages of the new PG specification is that the specification requirements remain fixed for all asphalt binder grades. However, the temperature at which these requirements must be met vary for the different binder grades. For example, as shown in Figure 2-3, PG 64-22 or PG 58-34 must exhibit minimum stiffness values of 1.0 kPa for the un-aged asphalt and 2.2 kPa for the RTFO aged asphalt, respectively (Kennedy, et al, 1994, Anderson, et al, 1994, and Anderson and Kennedy, 1993).

Avg		PG 58				PG 64				PG 70				PG 76				PG 82						
1-		-46	-16	-12	-20	-34	-40	-10	-16	-22	-28	-34	-40	-10	-16	-22	-28	-34	-40	-10	-16	-22	-28	-34
$\geq 230^{\circ}\text{C}$
$\leq 3 \text{ Pa}\cdot\text{s @ } 135^{\circ}\text{C}$
$\geq 1.00 \text{ kPa}$		46		52										70		76		82
$\geq 2.20 \text{ kPa}$		46		52										70		76		82						
20 Hours, 2.07 M		100				100 (110)				100 (110)				110 (110)
≤ 5000		DSR G																						
$S \leq 300 \text{ MPa}$ $m \geq 0.300$		(Bending Beam Rheometer) BBR "S" St "m"-value																						
Report Value		(Bending Beam Rheometer) BBR Physical H
$\geq 1.00 \%$		(Direct Tension) DT																						

Figure 2-3 How the PG Binder Specifications Works
(After FHWA, 1996).

2.5.5 Aging in the Superpave Asphalt Binder Specifications

Bell, 1989, discussed short term aging and long term aging associated with volatilization and oxidation, respectively. The Superpave asphalt binder specifications address these two major mechanisms of asphalt binder aging. Short and long term aging are addressed through the use of the RTFO and PAV binder aging procedures, respectively.

Short-Term Aging

Changes in asphalt binder characteristics which occur during the mixing, laydown, and compaction processes were associated with short term aging. In 1940, Lewis and Welborn introduced the TFO test for differentiating volatility and hardening characteristics of asphalts. In 1963 Hveem, Zube, and Skog introduced the RTFO test in specifications for the CALTRANS. Bell, 1989, summarized the test methods that have been used to simulate the short term aging on asphalt binders. He identified six test methods:

1. Thin film oven test;
2. Shell microfilm test;
3. Rolling thin film oven test;
4. Rolling microfilm oven test;
5. Tilt-oven durability test; and
6. Thin film accelerated aging test.

Initially, the Rolling Thin Film Oven (ASTM D 2872) and Thin Film Oven (ASTM D1754) tests were both being utilized for short term aging in the Superpave

asphalt binder specifications (Anderson and Kennedy, 1993). The same authors stated that further investigation of these test methods were discontinued and "attention was given to long-term field aging which is not addressed in the current specifications". In the final version of the PG specification, the RTFO test was selected to approximate short term aging. The RTFO was selected for the following reasons: temperature can be controlled more precisely, more samples can be aged in a relatively shorter time, and modified binders can be tested without "skin over" (Kennedy et al, 1994).

Long-Term Aging

Asphalt binders age due to two major mechanisms: volatilization or short term aging and oxidation or long term aging. The in-service or long term aging of asphalt binders represent slow, longer term oxidation (Bell, 1989 and Bahia and Anderson, 1994). Due to relatively moderate in-service temperatures, long term volatilization is slow. Thus, the in-service aging or long term aging is mainly related to oxidation of asphalt binders. In the Superpave asphalt binder specifications, long term aging is represented by the PAV test which reflects both the chemical and physical changes of asphalt binders in-service (Bahia and Anderson, 1994).

The PAV test is intended to approximate five to ten years of in-service aging. However, several factors affect in-service aging, which could influence this approximation (Bahia and Anderson, 1994). These factors are: local climatic conditions, aggregate type absorption, mixture type, asphalt film thickness, air-void content, traffic loading and asphalt chemical composition.

In reviewing previous efforts to select a long-term aging test, the PAV test was adopted for Superpave because it is a relatively fast and simple procedure. The PAV test is discussed in greater detail in Chapter 4.

2.5.6 Distress Modes in Superpave: Relationship to Pavement Performance

A number of researchers have discussed HMA pavement distresses that affect in-service performance (Decker and Goodrich 1989, Roberts et al 1991, Badaruddin and White, 1993; and Kennedy, et al, 1994). The primary distresses are:

- Permanent Deformation;
- Fatigue Cracking;
- Thermal Cracking; and
- Moisture Damage.

Moisture Damage will not be discussed in this work, as it is not directly addressed with the Superpave asphalt binder specification.

Permanent Deformation

Rutting or permanent deformation of HMA pavement layers is caused by a combination of densification and shear flow of the mix under repeated traffic loading. Asphalt binder, aggregate and as constructed mixture volumetric properties are related to HMA rutting potential (Leahy, et al, 1994, Anderson and Kennedy, 1993). Badaruddin and White, 1994 stated that type, gradation and percent of crushed aggregate and percent of natural sand are common factors that affect rutting in HMA pavements

Asphalt binder properties also affect mixture rutting resistance, especially in the case of polymer-modified asphalts (Anderson and Kennedy 1993). Rutting is a high temperature associated distress (Anderson et al, 1993). It has been reported that increase in asphalt mix stiffness results in enhanced resistance to rutting (Decker and Goodrich, 1989). Lower asphalt temperature susceptibility was associated with increased rutting while no correlations have been found between rutting and penetration or viscosity (Finn et al, 1990).

Initially, the non-recoverable component ($G^*\sin\delta$) of the complex shear modulus as determined in the DSR was selected as a rutting factor for the PG specification. Subsequently, however, the inverse of the loss compliance ($1/J'' = G^*/\sin\delta$) was adopted as the rutting factor for the Superpave asphalt specifications. The criteria for this factor were a minimum of 1.0 kPa for the un-aged/original (tank) asphalt and a minimum of 2.2kPa for the RTFO aged residue.

Fatigue Cracking

Fatigue cracking often referred to as alligator cracking, is classified as a load associated distress (Roberts, et al, 1991 and Huang, 1993). The usual concept of fatigue cracking is that it initiates at the bottom of the lowest asphalt layer in the pavement. Fatigue cracking can result from load that exceeds the pavement structural design, repeated traffic loading that exceeds the design repetitions or reduced bearing capacity of pavement layers. Thermal cracking as noted above is also a form of fatigue cracking (Decker and Goodrich, 1989). The authors also reported that "asphalt rheology strongly influence fatigue life". Conventionally, laboratory fatigue

testing has been conducted in either stress controlled mode or strain controlled mode. Conflicting asphalt properties have been reported to affect fatigue performance in HMA pavements. Softer asphalt are found to be more resistant to fatigue cracking when tested in strain-controlled mode (Monismith and Duncan, 1969, Van Dijk, 1975), while stiffer asphalts are found to be more resistant to fatigue cracking when tested in stress-controlled mode (Pell and Cooper, 1975), (Leahy, et al, 1994, Anderson, et al, 1993).

Based on evaluation of the flexural stiffness of mixes, the non-recoverable component ($G^*\sin\delta$) of the complex shear modulus, as determined in the dynamic shear rheometer (DSR), was selected as the fatigue criterion for the Superpave binder specification (Hicks, et al, 1993). The maximum value of this component for the aged residue from the PAV is 5000 kPa at 10 rad/sec (Hicks, et al, 1993). The asphalt binder properties related to fatigue cracking are typically measured at the intermediate design temperature.

Thermal Cracking

The prevailing opinion is that thermal cracking in HMA pavements initiates at the pavement surface. This initial surface crack then propagates into the underlying pavement layers (Burgess et al, 1971). Low temperature thermal cracking has been related to the asphalt binder characteristics (Badaruddin and White, 1994). Favorable conditions for thermal cracking occur when high rate temperature drops occur, especially during cold winter months in the northern US and Canada, inducing tensile stresses in the HMA. When these stresses exceed the HMA tensile strength, cracks are

initiated. There are two common types of cracking: shrinkage cracking that results from a single temperature below the critical cracking temperature and thermal fatigue cracking that results from repeated cooling cycles at slightly higher temperature. However, propagation of cracks (within the asphalt binder) is associated with both cracking types (Peterson, et al, 1994).

Tia and Ruth, 1985, investigated the use of lower asphalt stiffness to limit low temperature cracking in HMA pavements. This issue was also addressed by Burgess et al, 1972. The limiting stiffness moduli for asphalt binder, as determined by Gaw et al in 1974, was 10^9 N/m^2 ($1.45 \times 10^5 \text{ psi}$) at a loading time of 0.5 hour (Finn, et al, 1990). The role of temperature susceptibility has also been identified. Asphalts with higher temperature susceptibility are more prone to low temperature thermal cracking (Decker and Goodrich, 1989). Finn, et al, 1990 reported that in the Ste. Anne test, asphalt with less temperature susceptibility had no cracking after eight years. Mcleod, 1976, as documented by Finn, et al, 1990; Roberts, et al, 1990, reported that low temperature cracking increases as the Penetration-Viscosity Number (PVN) number decreases. This result was confirmed by Kandhal and Koehler, 1985. Therefore, identifying temperature characteristics of the asphalt binders is important for guarding against low temperature cracking.

Low temperature stiffness is addressed in the Superpave asphalt binder specification by limiting stiffness, as defined by Hass and Topper, 1969 and Monismith, et al, 1965 and Peterson, et al, 1994, and limiting the slope of the creep stiffness curve. The Superpave asphalt binder specification limits the stiffness of the

asphalt binder to a maximum of 300 MPa and the slope of the creep stiffness curve to a minimum of 0.300.

2.5.7 Superpave Associated Test Methods

The Superpave asphalt binder specification requires the following tests be performed: flash point, rotational viscometer, dynamic shear rheometer (DSR), rolling thin film oven (RTFO), pressure aging vessel (PAV), bending beam rheometer (BBR) and direct tension (DT). These test methods and associated equipment will be addressed in greater detail in Chapter 4.

2.6 Concerns Related to Superpave Asphalt Binder Specifications

Short-Term Aging:

Anderson and Kennedy in 1993, “Development of SHRP Binder Specification”, reported that “Aging or hardening of asphalt cement occurs during mixing and laydown process and during service. The existing methods, the Thin Film Oven Test (TFOT) ASTM D 1754 and the Rolling Thin Film Oven Test (RTFOT) ASTM D 2872, were reviewed and questions were raised with regard to the calibration of these methods for different plants, operating conditions, asphalt sources, and aggregate types-moisture conditions. An in-depth study to validate and cross-correlate the two test methods was considered, however, a comprehensive evaluation of TFOT test and RTFOT test methods would have consumed a disproportionate amount of the available resources. Therefore, further study of the two methods was discounted and attention was given to long term field aging which is not addressed in the current

specifications. In order to simplify the specification, the rolling thin film oven test was chosen as the single test for the new Superpave asphalt binder specification.

The RTFOT test can be completed more rapidly than TFOT test, the RTFOT test is preferable for polymer modified binders, and there is less between laboratory variability for the RTFOT test than for TFOT test. Hence the RTFOT test was chosen in preference to the TFOT test. If both the RTFOT and TFOT tests were retained in the SUPERPAVE asphalt binder specification the net effect would have been to double the number of grades because different asphalt cements and different asphalt binders respond differently to the two tests.”

Note 1. The general consensus among the researchers is that the TFOT or RTFOT may only simulates the mixing stage for asphalt binders. Construction stage includes: storing, transport, laydown and compaction.

Storing the mix in a silo. Indiana experience shows that this could be in some cases 4 to 12 hours, in some “odd” cases it could be to 1-3 days.

- Mixture transport is largely at mixing temperatures. A typical round trip time is one hour at mixing temperature. (i.e. 30 min. to the job site.)

- Time of lay-down and compaction is a function of the site length, but compaction should be completed before mixture temperature drops below 80C.

In 1989 Nelson and Wood reported that the TFO test was not severe enough to simulate the mixing stage for the asphalt cements which were extracted/recovered from loose mixes used in their study. These results were from mixtures produced in both drum-dryer and batch plants. In a personal communication, Professor Leonard Wood indicated that “the local limestone and/or dolomite aggregates has an adverse affect on aging of asphalt cements due to their chemical compositions and adsorption.”

R. G. Hicks, F. N. Finn, C. L. Monismith and R. B. Leahy, in 1994, “Validation of SHRP Binder Specification Through Mix Testing”, reported that “Aging of the asphalt alone, and subsequent testing does not appear to be adequate to predict mix performance because of the apparent mitigating effect aggregate has on aging.” The authors added, “The aging of certain asphalts is strongly mitigated by some aggregates but not by others. This appears to be related to the strength of the chemical bonding (adhesion) between the asphalt and the aggregate.”

Bell, 1989 also expressed concern about aging the neat (original) asphalt. He stated that asphalt binders age differently in a binder-aggregate mixture.

Long-Term Aging:

The pressure aging vessel (PAV) was developed as part of the SHRP research program to simulate long term aging of asphalt binders. However, there is concern that the PAV aging fails to predict in-service aging. Many researchers have an opinion that the PAV fails to predict 4, 8, or 15 years of in-service aging (Petersen, et al, 1994). Climate condition in particular plays a very important role in this regard. Harrigan, et al, 1993, reported that “This method is designed to simulate the oxidative

aging that occurs in the asphalt binders during pavement service. Residue from this test may be used to estimate physical or chemical properties of an asphalt binder after five to ten years of aging in the field. For asphalt binders of different grades or from different sources there is no unique correlation between the aging time and temperature in this test and in-service pavement age and temperature. Therefore, for a given set of in-service climatic conditions, it is not possible to select a single PAV aging time and temperature that will predict the properties of all asphalt binders after a specific set of in-service exposure conditions”

Similar statements can also be found in 1995 and 1996 AASHTO Provisional Standards. Moreover, Anderson, et al, 1994, stated: “It is beyond the scope of this study to establish a quantitative relationship between laboratory-aging parameters (aging temperature-time) and field aging parameters that include service temperature-time as well as mixture variable. The age of the sections whose asphalt properties are shown is approximately 8 years, and the section is located in a relatively moderate climate. Figure 5.38 also indicates that the aging levels caused by the aging in the PAV at 100C (212F) for 20 hours are very close to the aging levels occurring in that Florida section after 8 years. The same similarity between PAV aging and field aging can be seen. This section, however, is only 4 years old.”

“The two previous examples indicate that PAV aging results in aging effects that are similar to aging in the field. The examples also indicate that the relation between aging conditions in the PAV and the conditions in the field is not very simple. The results for the Florida section give equivalent to eight years in the field, while the Wyoming section, the equivalent is only 4 years. The data collected for other sections

confirm the later point. Figure 5.41 shows the master curves for another section in Florida, while Figure 5.42 shows the master curves for an asphalt used in a section in the California desert. For the section in California desert, the PAV aging falls short of simulating the aging level in the field after only 4 years. The results clearly depict the role of climate and how the equivalency of the PAV aging should be evaluated relative to local pavement climate.”

Branthaver et al, 1994, reported that “The results of the TFO-PAV aging conducted at temperatures of 60°, 70°, and 80°C (140°, 158°, and 176°F) indicated that the maximum temperature to which the pavement is exposed is an important factor in determining the eventual hardening of the pavement. The results showed that asphalts have widely ranging responses to rather small changes in temperature that occur in the pavement service temperature range. An aging test used to predict long term durability of an asphalt or pavement must take into consideration the climate to which pavement will be exposed.”

“It was also found that aging for each asphalt seems to be characterized by a response to one predominant factor, which differed among asphalts. This was not surprising because aging is a combination of oxidation, loss of volatiles, and response of the system to the first two factors. At present the findings of the statistical analysis is that aging of asphalt will probably have to be measured and can not be predicted by simple methods.”

Fatigue, Rutting and Low temperature Cracking Distress Modes:

Concern has been stated about the sole use of an asphalt binder rutting factor (i.e. $G^*/\sin\delta$) to predict rutting performance in-service. The opinion is that there are factors that influence rutting distress other than asphalt binder properties. In 1994, Leahy et al, during a laboratory wheel tracking investigation stated that “Since it was hypothesized that asphalt source would significantly affect rutting response, analysis of variance (ANOVA) was performed to determine the influence of the various factors. The ANOVA indicated each of the factors and interactions accounted for variation of rutting response by the following approximate proportions:

Rutting Response Response or Variable -----	Factor Effect % Interaction -----	Proportional -----
Rut Rate	Asphalt	26
	Aggregate	29
	Air voids	8
	Asphalt -Aggregate	27
	ANOVA Model Error	5
Rut Depth	Asphalt	31
	Aggregate	19
	Air voids	8
	Asphalt -Aggregate	28
	ANOVA Model Error	5

The results indicate that a poor relationship exists between the binder property, $G^*/\sin\delta$, and mix rutting.” At the end of this paragraph the authors reported that “Thus, most of the variation in rutting response is probably due to other variables such as aggregate characteristics and/or the testing process.” The authors added that: “The

results of this study suggest that the $G^*/\sin\delta$ is not a reliable predictor of potential rutting.”

The authors also state that: “Although the relationships between binder properties and mix shear response are weak, it appears that any binder property($G^*/\sin\delta$, G^* or G'') can be used to estimate mix shear response with the same poor degree of reliability. Thus the significance of the “ $\sin\delta$ ” term in $G^*/\sin\delta$ is questionable, although it may have more of an effect with modified asphal.” “The correlation between $G^*/\sin\delta$ and the various measures of permanent deformation response were generally poor. The weak correlations are partly the result of the dominant effect of aggregate characteristics on permanent deformation response. However, in cases where mix characteristics are such that interparticle friction is low (e.g. RH aggregate and 7 percent air voids) and the mix is subjected to harsh environmental and loading conditions (e.g. 60°C and CH shear test), the influence of the binder becomes more readily apparent.” “It appears that the value of $G^*/\sin\delta$ may be used to screen binders that will provide inferior performance in such cases. The results of these studies underscore the importance of mix testing, in addition to binder testing, for evaluation of permanent deformation in pavements” “Overall the findings to date are encouraging for fatigue and low temperature cracking, but less so for permanent deformation.” “ $G^*\sin\delta$, as well as G^* , and G' , all result in relationships of equivalent strength with mix fatigue response. Hence, one may conclude that the effect of the “ $\sin\delta$ ” term of $G^*\sin\delta$ is negligible, and any of these terms could be used in the SHRP binder specification. However, the effect of $\sin\delta$ may still be important for

modified asphalts.” “ In the prediction of fatigue cracking in pavement structures it appears that the asphalt binder properties are again important but pavement structure effects may be equally or more important.” “Overall asphalt binder properties play a critical role in the fatigue response of asphalt- aggregate mixes. However, other mix characteristics such as air voids levels and aggregate characteristics can also have a significant impact on fatigue response.”

The same findings were reported by Leahy and Quintus, 1994: “This comparison does demonstrate, however, the possibility of accepting an asphalt according to the specification limit that may result in rutting, or rejecting an asphalt that would provide acceptable performance.” “The A-003 contractor attempted to validate the findings and recommendations of the A002A contractor relative to the influence of asphalt on the three key distresses incorporated in the SHRP asphalt research program. The findings are encouraging for fatigue and low temperature cracking, but less so for permanent deformation.”

Leahy and Von Quintus, 1994 stated “A requirement that $G^*/\sin\delta$ be greater than 1kPa was added to the binder specification to protect against the possibility that the asphalt binder would contribute to tenderness during mixing and laydown.”

Lytton, et al, 1994 stated “Binder performance-based specifications limits for thermal cracking were found based upon patterns of binder stiffness and log slope of the compliance as they affect the extent of cracking. No such patterns could be found to relate binder properties to the rate of appearance of load-related distresses, leading to the conclusion that the binder properties alone do not control fatigue cracking or

rutting performance of asphalt concrete. Instead, specifications for both the binder and mixture properties are necessary for quality assurance against load-related distress.”

CHAPTER 3

Condition Surveys, Pavement Sections and Coring Methods

This chapter describes the condition surveys performed on pavement sections included in this study. The condition surveys were conducted during the summer of 1992 and 1993. Pavement sections and distresses exhibited on these sections are described and evaluated in terms of severity and extent. Pavement Condition Index (PCI) is described for each project section. Coring methods that were used to obtain in-service samples are also discussed. The distress surveys presented were procured and revised from the original draft report titled “Second Phase Study of Changes In-service Asphalts” written by Robin Fontaine in 1994.

3.1 Condition Surveys

The Pavement Maintenance Management System (PAVER) can be effectively utilized to conduct pavement inspections, set maintenance and rehabilitation priorities, and evaluate and analyze economic alternatives for maintenance and repair work (Shahin and Kohn, 1981; Pavement Maintenance Management, 1982). The PAVER system was selected to conduct pavement condition surveys (inspections) for this study. The PAVER system provides a detailed evaluation of the condition of the pavement as well as a reasonable estimate of the pavement condition index (PCI). PCI is a combined pavement condition index, which is a function of several individual surface distresses. Moreover, the PAVER system has a 95 % confidence level (i.e. the PCI rating is within +/- 5 points of the mean PCI).

Several steps were followed in conducting pavement condition surveys. They can be summarized as follows. A pavement was divided into branches with similar identification. For example, Interstate 65 (I-65) or State Road 231 (SR-231). The pavement branches were then divided into sections with similar characteristics. For example, project number R-15502-I and project number R-15502-A were considered two different sections because they were paved on two different days. Sections were selected to encompass 2500 +/- 1000 square feet¹. Figure 3-1 was used to determine the minimum number of sample units to be surveyed on each section.

Note¹: The following conversion factor was used throughout this Chapter:
One foot = 0.3048 meter

**Figure 3-1 Determination of Minimum Number of Sample Units to be Surveyed
(After Shahin and Kohn, 1981).**

3.2 Pavement Condition Index (PCI)

The average PCI rating of each project was computed as follows:

- Required sample units were inspected, distresses and their severity were determined;
- Density of each distress was determined and a deduct value was obtained;
- Total deduct value for all distresses were determined;
- Total adjusted deduct value was computed; and
- PCI rating was calculated according to the following formula:

$$\text{Pavement Condition Index} = 100 - \text{Corrected Deduct Value}$$

A pavement section was given an Excellent rating if its PCI value ranged between 100 and 85.1; Very Good if its PCI value was between 85 to 70.1; Good if its PCI value was between 70 to 55.1; Fair if its PCI value was between 55 to 40.1; and Poor if its PCI value was between 40 to 25.1. Since the sample units were chosen at random, the PCI rating of a pavement section was obtained by averaging the PCI of its sample units. Deduct values were obtained from several curves, an example of which is illustrated in Figure3-2.

Figure 3-2 Severity of Distresses, the PCI, and the Rating
(After Shahin and Kohn, 1991).

3.3 Results of Condition Surveys

1. Project R-15502I

Project R-15502I was located on Route 67 in Morgan County, Indiana. The pavement, HAC #11 modified surface, consisted of 6.3 kilometers (20900 feet) paved from station 1908+00 to station 1699+00 (English system stationing). The liquid asphalt used on this project was provided by Asphalt Materials, Inc. The Average PCI rating for the project was 71.3% (“Very Good” rating). Figure 3-3 represents a typical section of the project pavement.

Figure 3-3 Typical Section of Project 15502I Pavement.

Distress exhibited on project R-15502I

The condition survey conducted on this project section revealed the following distresses:

Low severity fatigue cracking, low severity block cracking, low severity bumps and sag, low severity edge cracking, low severity longitudinal and transverse cracking, low severity patching, low rutting (permanent deformation), low severity slippage, and low severity weathering and raveling. Figure 3-4 shows a low severity fatigue crack in the pavement section.

Figure 3-4 A Low Severity Fatigue Cracking of the 15502I Pavement Section.

2. Project R-15404

Project R-15404 was located at the intersection of County-line Road and the 6th Street in Johnson County, Indiana. The pavement, HAC # 9 surface, consists of 1.6 kilometer (5520 feet) from station 0+00 to station 55+20 on CountyLine Road and 0.2 kilometer (726 feet) paved from station 0+00 to station 7+26 on 6th Street.

Distresses exhibited on project R-15404 CountyLine Road West Bound:

The condition survey conducted on this project section revealed the following distresses:

Low and medium severity fatigue cracking, low severity block cracking, low severity bumps and sags, low severity edge cracking, low and medium severity lane/shoulder drop-off, low and medium severity longitudinal and transverse cracking, and low severity weathering and raveling. The average PCI rating of the project was 62.4 percent which gave the project a “Good” rating. Figure 3-5 shows typical block cracking on CountyLine Road.

Figure 3-5 Block Cracking on CountyLine Road.

Distress exhibited on project R-15404 Countyline Road East Bound:

The condition survey conducted on this project section revealed the following distresses:

Low and medium severity fatigue cracking, low and medium severity block cracking, low, medium, and high severity edge cracking, low, medium, and high severity lane/shoulder drop-off, low and medium severity longitudinal and transverse cracking, and medium severity rutting. The average PCI rating for the project was 68.6 percent, thus project rating was “Good”. Figure 3-6 illustrates the lane shoulder drop off on 6th street.

Figure 3-6 The Lane Shoulder Drop Off on 6th Street.

Distress exhibited on project R-15404 6th Street:

The condition survey conducted on this project section revealed the following distresses:

Low and medium severity fatigue cracking, low and medium severity edge cracking, low and high severity lane/shoulder drop-off, low and medium severity longitudinal and transverse cracking, and medium severity weathering and raveling. The average PCI rating for the project was 49.3 percent ("Fair" rating). Figure 3-7 illustrates the edge cracking on Countyline Road.

Figure 3-7 Edge Cracking on Countyline Road.

3. Project R-15308D

Project R-15308D was located on Interstate 74 (I-74) in Shelby County. The pavement, HAC #11 modified surface, consisted of 7.0 kilometers (22732 feet) from station 141+00 to station 480+00. The average PCI rating of the project is 55.7 percent (“Good” rating).

Distresses exhibited on project R-15308D:

The condition survey conducted on this project section revealed the following distresses:

Low severity fatigue cracking, low severity block cracking, low and medium severity edge cracking, low severity longitudinal and transverse cracking, and low, medium, and high severity weathering and raveling. The average PCI rating for the project was 55.7 percent, giving the project a “Good” rating. Figure 3-8 shows typical block cracking in the passing lane of the pavement.

Figure 3-8 A Typical Block Cracking in the Passing Lane.

4. Project R-15173

Project R-15173 was located on Route 56 in Dubios County. The pavement, HAC #11B surface, consisted of 4.5 kilometers (14500 feet) from station 243+00 to station 388+00. The average PCI rating for the project was 48.4 percent. The project rating was “Fair”. Figure 3-9 shows a typical section of this pavement.

Figure 3-9 A Typical Section of R-15173 Pavement.

Distress exhibited on project R-15173:

The condition survey conducted on this project section revealed the following distresses:

Medium severity bumps and sags, low and medium severity edge cracking, low, medium, and high severity lane/shoulder drop-off, low and medium severity longitudinal and transverse cracking, polished aggregate and low rutting. Figure 3-10 illustrates the polished aggregate on the pavement section.

Figure 3-10 A Typical Polished Aggregate on the Pavement Section.

5. Project R-15174B

Project R-15174B was located on Route 241 in Knox County. The pavement, HAC #9 surface, consisted of 3.4 kilometers (11100 feet) from station 354+00 to station 465+00. The average PCI rating of the project was 54.1, which gave the project a “Fair” rating. Figure 3-11 shows a typical section of the pavement.

Figure 3-11 A Typical Pavement Section of R-15174 B.

Distress exhibited on project R-15174B:

The condition survey conducted on this project section revealed the following distresses:

Medium severity bumps and sags, low and medium severity edge cracking, low, medium, and high severity lane/shoulder drop-off, low and medium severity longitudinal and transverse cracking, polished aggregate, and low rutting. Figure 3-12 shows typical alligator cracking observed on a steep curve on the pavement.

Figure 3-12 Alligator Cracking on a Steep Curve on the Pavement.

6 & 7. Project R-15165A East/WestBound

Project R-15165A was located on Route 44 in Shelby County. The pavement, HAC #9 Type A surface, consist of 1.1 kilometers (3624 feet) from station 50+10 to station 63+06 eastbound and from station 48+11 to station 84+35 westbound. The average PCI rating of the eastbound was 58.9 which gave the section a “Good” rating. The average PCI rating in the westbound direction was 70.5 (“Very Good” rating).

Distresses exhibited on project R-15165A EastBound:

The condition survey conducted on this project section revealed the following distresses:

Low alligator cracking, low block cracking, low and medium severity lane/shoulder drop-off, low severity longitudinal and transverse cracking, low patching, low severity potholes, and low and medium severity weathering and raveling.

Distresses exhibited on project R-15165A WestBound:

The condition survey conducted on this project section revealed the following distresses:

Low alligator cracking, low block cracking, low and medium severity lane/shoulder drop-off, low severity longitudinal and transverse cracking, and low and medium severity weathering and raveling.

8 & 9. Project R-15165B East/WestBound

Project R-15165B was located on Route 44 in Shelby County. The pavement, HAC #9 surface, consisted of 1.4 kilometers (4595 feet) from station 135+55 to station 181+05 eastbound and from station 139+45 to station 185+40 westbound. The average

PCI rating in the eastbound direction was 55.6. The average PCI rating in the westbound direction was 54.6. Both sections obtained “Good” ratings.

Distresses exhibited on project R-15165B EastBound:

The condition survey conducted on this project section revealed the following distresses:

Low alligator cracking, low block cracking, low, medium, and high severity lane/shoulder drop-off, and low severity weathering and raveling.

Distresses exhibited on project R-15165B WestBound:

The condition survey conducted on this project section revealed the following distresses:

Low alligator cracking, low block cracking, low edge cracking, low, medium and high severity lane/shoulder drop-off, low severity longitudinal and transverse cracking, low rutting, and low and medium severity weathering and raveling.

10. Project R-15617A

Project R-15167A was located on Route 234 in Fountain County. The pavement, HAC #9 surface, consisted of 5.3 kilometers (16696 feet) from station 3+80 to station 194+50. However, only 4.6 kilometers (15112 feet) were surveyed due to a bridge construction. The average PCI rating of project R-15167 A section was 59.4 (“Good” rating).

Distresses exhibited on project R-15167A:

The condition survey conducted on this project section revealed the following distresses:

Low alligator cracking, low block cracking, low severity bumps or sags, low, medium, and high edge cracking, low, medium, and high severity lane/shoulder drop-off, low and medium longitudinal and transverse cracking, low rutting, and low and medium severity weathering and raveling. Figure 3-13 depicts a typical transverse crack in the section. Figure 3-14 shows a bump observed in the section.

Figure 3-13 A Typical Transverse Crack in the Section.

Figure 3-14 Bump Observed in this Section.

11. Project R-15617B

Project R-15167B was located on Route 234 in Fountain County. The pavement, HAC #9 surface, consist of 4.8 kilometers (15539 feet) from station 417+33 to station 625+37. The average PCI rating of project R-15167B section was 68.4 which also gave this section a “Good” rating.

Distresses exhibited on project R-15167 B:

The condition survey conducted on this project section revealed the following distresses:

Low alligator cracking, low block cracking, low depressions, low, medium, and high edge cracking, low, medium and high severity lane/shoulder drop-off, and low and medium severity longitudinal and transverse cracking. Figure 3-15 shows a typical section of the pavement.

Figure 3-15 Typical Section of the Pavement.

12. Project R-15630

Project R-15630 was located on Route 37 in Crawford County. The pavement, HAC #11 surface, consisted of 0.9 kilometers (3027 feet) from station 54+87 to station 76+90 and station 36+60 to station 53+00. The average PCI rating for project R-15630 was 84.4, which gave this section a “Very Good” rating.

Distresses exhibited on project R-15630:

The condition survey conducted on this project section revealed the following distresses:

Low severity edge cracking, low severity longitudinal and transverse cracking, and low severity weathering and raveling. Figure 3-16 shows a typical section of the pavement.

Figure 3-16 A Typical Section of the Pavement R- 15630.

13. Project R-15619A

Project R-15619A was located on Route 25 in Tippecanoe County. The pavement, HAC #9 surface, consisted of 2.7 kilometers (8829 feet) from station 524+89 to station 481+80 and from station 481+47 to station 436+27. The average PCI rating of project R-15619A was 67.1 which gave this section a “Good” rating.

Distresses exhibited on project R-15619A:

The condition survey conducted on this project section revealed the following distresses:

Low alligator cracking, low severity block cracking, low and medium severity edge cracking, low severity joint reflected cracking, low and medium severity lane/shoulder drop-off, low severity longitudinal and transverse cracking, polished aggregate, low and medium potholes, low severity rutting, and low severity weathering and raveling.

14. Project R-15619D

Project R-15619D was located on Route 25 in Tippecanoe County. The pavement, HAC #9 surface, consisted of 2.7 kilometers (8638 feet) from station 476+34 to station 489+96. The average PCI rating of project R-15619D was 50.3 giving this section a “Fair” rating.

Distresses exhibited on project R-15619D:

The condition survey conducted on this project section revealed the following distresses:

Low severity alligator cracking, low severity block cracking, low severity edge cracking, low joint reflected cracking, low and medium severity lane/shoulder drop-

off, low severity longitudinal and transverse cracking, polished aggregate, low potholes, low severity rutting, and low severity weathering and raveling.

15. Project R-15605

Project R-15605 was located on Route 40 in Wayne County. The pavement, HAC #11 surface, consisted of 2.9 kilometers (9262 feet) from station 136+80 to station 197+17 and from station 199+90 to station 232+15. The average PCI rating of project R-156605 was 51.1, which gave the section a “Fair” rating.

Distresses exhibited on project R-15605:

The condition survey conducted on this project section revealed the following distresses:

Low and medium severity alligator cracking, low and medium severity edge cracking, low joint reflected cracking, low and medium severity lane/shoulder drop-off, low severity longitudinal and transverse cracking, low severity patching, polished aggregate, low potholes, low severity rutting, and low severity weathering and raveling. Figure 3-17 shows alligator cracking in this pavement section.

Figure 3-17 Medium Severity Alligator Cracking in the 15605 Pavement Section.

16. Project R-15309E

Project R-15309E was located on Route 3 in Rash County. The pavement, Type IV surface, consisted of 3.6 kilometers (11705 feet) from station 258+70 to station 141+65. The average PCI rating of project R-15309 was 75.8, which gave the section a “Very Good” rating.

Distress exhibited on project R-15309E:

The condition survey conducted on this project section revealed the following distresses:

Low and medium severity alligator cracking, low and medium severity edge cracking, low joint reflected cracking, low and medium severity lane/shoulder drop-off, low severity longitudinal and transverse cracking, low severity patching, polished aggregate, low potholes, low severity rutting, and low severity weathering and raveling.

17. Project R-14643

Project R-14643 was located on Route 27 in Union and Wayne Counties. The pavement, HAC Type IV surface, consisted of 37.4 kilometers (121286 feet) from station 221+80 to station 343+665. The average PCI rating of project R-14643 was 77.4 which gave this section a “Very Good” rating.

Distresses exhibited on project R-14643:

The condition survey conducted on this project section revealed the following distresses:

Low alligator cracking, low and medium severity edge cracking, low and medium severity longitudinal and transverse cracking, low severity patching, polished aggregate, and low potholes. Figure 3-18 shows a typical pavement section.

Figure 3-18 A Typical Pavement Section of R-14643.

3.4 Coring Methods

The number, size, and location of in-service cores, that would be required to generate sufficient asphalt binder for testing, were determined in accordance with SHRP guidelines for the Long Term Pavement Performance Program (LTPP 1989, 1992) and by previous research performed by Badaruddin and White in 1994.

Three different pavement locations were of interest in this research study. In-service cores were taken from outside the wheel paths (OWP approximately 0.5 feet from the shoulder line), in the wheel paths (IWP approximately 2 feet from the shoulder line), and between wheel path (BWP approximately 6 feet from the shoulder line). It was of interest at the beginning of the study to detect and study the effect of contamination materials, such as: gasoline, oil, or antifreeze (if they existed), in the cores sampled BWP. Therefore, in-service cores OWP were selected to serve as a control, since it was very unlikely that this area of a pavement would exhibit contamination. In-service cores sampled from IWP and BWP were expected to reflect the greatest changes in asphalt binder characteristics. In-service cores IWP were expected to show the changes that occurred under repeated traffic loading. 100 mm (4 inches) and/or 153mm (6 inches) cores were used for asphalt extraction/recovery purposes. The cores were sampled from one and two lane sections. In the two lane sections, cores were sampled from the traveled lane. Figures 3-19 and 3-20 show 4 inch and 6 inches coring plans for one and two lane sections.

Note (6inches = 153mm, 2feet=60cm, and 6 feet = 1.8 meter)

Figure 3-19 Coring Method Used for Two Lane Roads.

Note (6inches = 153 mm, 2feet = 60 cm, and 6feet = 1.8 meter)

Figure 3-20 Coring Methods Used in Four Lane Roads.

CHAPTER 4

SUPERPAVE Test Methods, Associated Equipment, and Criteria

Superpave asphalt binder test methods and the associated equipment that were used to carry out the experimental work in the study are discussed in this chapter. The purpose of each test, the significance and the description of the test method, as standardized by the American Association of State Highway and Transportation Officials (AASHTO), are discussed and outlined in this chapter. Additionally, the description and the significance of the adopted Superpave asphalt binder equipment are also presented in this chapter. The Superpave asphalt binder tests include: Dynamic Shear Rheometer (DSR), (AASHTO-TP5); Bending Beam Rheometer (BBR), (AASHTO-TP1); Rolling Thin Film Oven Test (RTFOT), (AASHTO T240); and Pressure Aging Vessel (PAV), (AASHTO PP1); and extraction and recovery procedure, (SHRP B-006).

4.1 Dynamic Shear Rheometer (DSR), AASHTO TP5

Purpose and significance of use:

The DSR test, (AASHTO - TP5, 1993, 1994, and 1995), measures the rheological characteristics of an asphalt binder; the complex shear modulus (G^*) and

the phase angle (δ). These characteristics represent the viscous (non-recoverable, $G^*\sin(\delta)$) and elastic (recoverable, $G^*\cos(\delta)$) behavior of the asphalt binder. A sample of asphalt is sandwiched between two parallel disks. The sample is subjected to an oscillation through a fixed rotation at 10 radians per second (1.59Hz). This frequency was selected because it represents a loading time of 0.1 second and simulates the traffic loading (Anderson et al 1994). The DSR test is performed at temperatures associated with intermediate and high pavement service temperatures.

Superpave asphalt binder specifications utilize DSR results to assess two criteria: rutting at elevated service temperatures ($G^*/\sin(\delta)$), and fatigue at intermediate service temperatures ($G^*\sin(\delta)$). Rutting criteria are a minimum of 1.0 kPa for the original/tank asphalt and a minimum of 2.2 kPa for short-term (RTFO aged) material. The fatigue criterion is specified as a maximum of 5000 kPa on long-term (RTFO +PAV aged) material.

Concept of Operation

Figure 4-1 shows the BOHLIN DSR equipment, a similar device was used to characterize the project asphalts in earlier stages of this study. In summary the DSR test can be explained as follows.

An asphalt disk is sandwiched/placed between the DSR plates, as shown in Figure 4-2. Figure 4-2 also shows the two plates; the upper plate oscillates and the lower plate is fixed. The oscillating plate rotates through a fixed angle following a sine

function and the applied torque is measured, as shown in Figure 4-3. The equation used to compute the complex shear modulus and phase angle using the DSR test results follow. Figure 4-4 illustrates the DSR measurements.

The complex shear modulus is computed as follows:

$$\begin{aligned}\tau &= 2T/\pi r^3 \\ \gamma &= \theta r/h \\ G^* &= (\tau_{\max} - \tau_{\min})/(\gamma_{\max} - \gamma_{\min})\end{aligned}\quad (4-1)$$

Where,

- T = maximum applied torque;
- r = radius of the sample;
- θ = rotational angle;
- h = sample height;
- τ = shear stress; and
- ω = angular frequency.

The phase angle is computed as the time lag between the applied shear stress and resulting shear strain, as shown in Figure 4-4.

The DSR test is either conducted in a constant-stress or a constant-strain mode. In the constant-stress mode, the fixed torque is applied and the rotation of the upper plate is measured. In the constant-strain mode, the upper plate is rotate a fixed amount and the resulting torque is measured.

Figure 4-1 The Bohlin DSR (After FHWA, 1996).

Note: Asphalt sample is sandwiched between the two plates

Figure 4-2 Schematic of the DSR Plates (After FHWA, 1996).

Note: AB, BA, AC, CA is one cycle the plate is moving a fixed distance.

Figure 4-3 Schematic of the DSR Plate Movement (After FHWA, 1996).

Figure 4-4 DSR Measurements (After FHWA, 1996).

DSR Test Procedures

The DSR test can be summarized as follows. Asphalt is heated until sufficiently fluid to pour. The asphalt is poured into a silicon mold, the use of which minimizes specimen trimming. The asphalt is left in the mold to cool. The distance between the upper and lower plate (Gap) is adjusted to 1 mm for tests on original or RTFO aged asphalts and 2 mm for PAV aged asphalts. Subsequently, the specimen is mounted between the parallel plates and excess asphalt is trimmed. Tests are software controlled and depending on the type of DSR device can be conducted in the constant stress or constant strain mode. The complex shear modulus and phase angle are reported. In addition, the rutting factor, $G^*/\sin\delta$, is reported for the original and the RTFO aged asphalt and the fatigue factor, $G^*\sin\delta$, is reported for PAV aged asphalt.

DSR Device Description

A computer and associated software are used to control the test and data acquisition. Either water or an air heating/cooling system controls temperature during the DSR test. Rotation torque is applied by an electric motor. Stress and/or strain is measured, recorded, and used as a feed back to control the test. Figure 4-5 shows the two DSRs employed in this study.

Figure 4-5 BOHLIN (Top) and RAA (Bottom) DSR Used on this Study.

4.2 Bending Beam Rheometer (BBR), AASHTO TP1

Purpose and Significance of Use:

The Bending Beam Rheometer (BBR) test, (AASHTO - TP1, 1993, 1994, 1995), is used to measure the flexural stiffness of asphalt binder at very low temperature. Two properties are obtained from the test; creep stiffness which can be defined as the resistance of asphalt binder to application of a constant load and m-value which can be defined as the rate of change of the asphalt binder stiffness under load. A Cannon BBR similar to the one used in this study is shown in Figure 4-6.

Concept of Operation

A schematic of the Bending Beam Rheometer is shown in Figure 4-7. In summary, the BBR test can be explained as follows. A mid-point load is applied to an asphalt beam of known dimensions while immersed in a fluid bath as depicted in Figure 4-8. The deflection of the beam under the constant load is measured throughout the duration of the test as shown in Figure 4-9. The bath controls test temperatures constant throughout the test. Load, beam dimensions, and recorded deflections are used to compute flexural creep stiffness as indicated in the following equation:

$$S(t) = PL^3/4bh^3\delta(t) \quad (4-2)$$

where,

$S(t)$ = Creep stiffness at any time t ;

P = Applied load (constant load of 980 to 1030g);

L = Distance between beam supports in mm;

b = width of asphalt beam 12.5 millimeters;

h = thickness of asphalt beam 6.25 millimeters; and

$\delta(t)$ = deflection at any time t.

The m-value is the slope of the log creep stiffness curve versus the log time curve. The m-value represents the rate of change of the stiffness over time.

Figure 4-6 Cannon Bending Beam Rheometer (After FHWA, 1996).

Figure 4-7 Schematic of the Bending Beam Rheometer (After FHWA,1996).

Figure 4-8 Bending Beam Rheometer – Loading Mechanism (After FHWA).

Figure 4-9 Schematic of the Bending Beam Rheometer at Loading
(After FHWA, 1996).

BBR Test Procedures

The BBR test can be summarized as follows. A sample of asphalt is heated to a temperature at which is sufficiently fluid to pour. An aluminum mold assembly, as shown in Figure 4-10, is used to prepare the asphalt beam. The fluid asphalt is poured into the mold assembly as shown in Figure 4-11. The asphalt mold assembly is then left to cool for a period of 45 to 60 minutes, at which point the excess asphalt above the top of the mold assembly is trimmed. The mold assembly is then cooled in a freezer for a period of 5 to 10 minutes prior to demolding, as shown in Figure 4-12. The asphalt beam is then placed in the BBR test bath for a period of 60+/- 5 minutes prior to loading. The test is then initiated and creep stiffness, m-value and deflection

readings are electronically recorded over a period of 240 seconds. The stiffness and the m-value of the asphalt are reported at 60 seconds.

Figure 4-10 BBR Mold Assembly (After FHWA, 1996).

Figure 4- 11 Pouring Asphalt into BBR Mold (After FHWA, 1996).

Figure 4 - 12 BBR Asphalt Beam (After FHWA, 1996).

BBR Device Description

As shown in Figure 4-7, the BBR device consists of three major components. A computer and software are used for test control and data acquisition. A cooling unit is incorporated that controls the test temperature in the bath through a fluid. A small loading frame, which resides in the bath, is used to maintain a constant load on the beam throughout the test. Figure 4-13 shows the two BBR devices used in this study.

Figure 4-13 Cannon (Top) and ATS (Bottom) BBR Used in This Study.

4.3 Short term Aging-RTFO Test, AASHTO T240

Purpose and significance of test

The Rolling Thin Film Oven test (RTFOT), (AASHTO – T240, 1995), is performed on the original asphalt binder to simulate the effects of the construction process on the asphalt binder (mixing, laydown and compaction). In the early 1960s, CALTRANS introduced the RTFO for its proposed aged-residue viscosity grading system. An RTFO is shown in Figure 4-14.

Concept of Operation

Glass jars are filled with 35 grams of asphalt binder and placed in the RTFO oven rack. While the RTFO sample rack is rotated, a jet air is blown into the sample jars. This air along with elevated temperature (163°C) and test duration of 80 minutes causes the asphalt binders to age. Figure 4-14 shows the jet nozzle, bottle rack, and an RTFO jar. Figure 4-15 illustrates the schematic/concept of operation of the RTFO test.

Figure 4-14 Rolling Thin Film Oven – Short Term Aging (After FHWA,1996).

Figure 4-15 Schematic/concept of operation of Rolling Thin Film Oven (After FHWA, 1996).

RTFO Test Procedure:

The RTFO test can be described as follows. An asphalt sample is heated until sufficiently fluid to pour. The heated asphalt is poured into glass jars (35 +/- 0.5 gram). The contents of two jars are used to determine mass loss, which is limited to 1.0% in the Superpave specifications. The jars are heated in the RTFO oven (163°C) for 80 +/- 5 minutes. After test duration, the asphalt residue is poured into one container to ensure homogeneity. The asphalt in the container can be used for either testing or additional aging in the Pressure Aging Vessel. Figure 4-16 shows an RTFO jar used to check mass loss, RTFO jars, and 8oz asphalt container after the RTFOT.

RTFO Oven Description

Figure 4-17 shows the RTFO used in this study. The RTFO controls (air jet control and temperature control), heating elements, sample rack, and jet nozzle can also be seen. A schematic of the RTFO was presented earlier in Figure 4-15.

Figure 4-16 Mass Loss, RTFO Bottles, and 8oz asphalt container after the RTFOT.

Figure 4-17 Rolling thin Film Oven Test Used in this Study.

4.4 Long Term Aging-PAV Test, AASHTO PP1

Purpose and Significance of test

The Pressure Aging Vessel (PAV) test, (AASHTO – PP1, 1995), is performed on the asphalt binder that has been subjected to the RTFO conditioning to simulate long-term in-service aging (five to ten years). The PAV test does not consider different mix parameters that affect in-service aging such as aggregate type, voids content, and film thickness.

Concept of Operation

The PAV test utilizes both temperature and pressure to simulate in-service aging. To accelerate test time and while the pressures are maintained at 2.1 MPa, three different temperatures are used: 90, 100 or 110°C, depending on asphalt binder grade and expected in service environmental conditions. The asphalt ages as a result of the high temperature and pressure. This process causes the oxidation process to be accelerated. Figure 4-18 illustrates the PAV equipment.

PAV Test Procedure

A schematic of the PAV is shown in Figure 4-19. The PAV test procedure can be described as follows. An RTFO aged asphalt sample is heated until sufficiently fluid to pour. 50 +/- 0.5 grams of binder is poured into each Thin Film Oven (TFO) pan. TFO pans are placed in a pan holder, subsequently, the pan holder is placed in the aging vessel. Figure 4-20 shows the aging vessel, TFO pans, and TFO pan holder.

The pressure line and temperature monitoring devices are then connected to the pressure vessel. The PAV oven temperature is allowed to equilibrate at the test temperature. Pressure in the vessel is increased to 2.1MPa to the specified limit over ten minutes. The asphalt binder is aged over the twenty hour test period. The pressure is released slowly over a ten minute period at the completion of the test. The TFO asphalt pans are placed in an oven for a period of thirty minutes and stirred occasionally to release any trapped air. The aged asphalt is then poured into tins for future testing (DSR and BBR).

Figure 4-18 Pressure Aging Vessel (PAV) – Long Term Aging
(After FHWA, 1996).

Figure 4-19 Schematic of the Pressure Aging Vessel (After FHWA, 1996).

Figure 4-20 Pressure Vessel, TFO pans, and TFO pan holder (After FHWA, 1996).

PAV Device Description

Major components of the PAV can be described as follows. The pressure vessel holds the TFO pans and TFO pan holder. A pressure line connected to the pressure vessel that maintains the pressure constant throughout the test. A temperature probe that measures the temperature inside the pressure vessel is attached also. The entire assembly is held in a PAV oven. The two PAVs used in this study are shown in Figure 4-21. Figure 4-22 shows the pressure vessel inside the PAV oven, the pressure line connected to the pressure vessel, and the temperature probe.

Figure 4-21 PAV Used In this Study.

Figure 4-22 PV inside the PAV oven, the pressure line connected to the PV, and the temperature probe.

4.5 Extraction of Asphalt Binders from Field Samples, SHRP B-006

Purpose and Significance of Use

The extraction and recovery procedure, (SHRP B-006, 1995), is performed to extract and recover asphalt binder from bituminous field samples. Subsequently, physical and/or chemical tests can be performed on the extracted and recovered asphalt. The asphalt content of the field samples can be determined as a result of this procedure also. However, it is not recommended to use the remaining aggregate for sieve analysis.

Concept of Operation

HMA samples (field cores or loose mixtures) are repeatedly washed using solvents. Filters are used throughout the test to recover aggregate fines. A rotary evaporator is used to distill the asphalt under vacuum. Finally, nitrogen is introduced to “drive-off” any remaining traces of solvents, (SHRP B-006, 1995).

Extraction and Recovery Procedure

The extraction and recovery procedure can be described as follows. HMA samples are heated and placed in a high-speed centrifuge. The HMA is repeatedly washed with solvent. The resulting solution is distilled in a rotary evaporator. Nitrogen is introduced to drive-off remaining traces of solvents. The recovered asphalt is poured into containers for subsequent testing.

Extraction and Recovery Equipment

Figure 4-23 shows a schematic of the SHRP extraction procedure and Figure 4-24 shows the major components of the extraction and recovery equipment.

Figure 4-23 Schematic of the SHRP Extraction Procedure.

Figure 4-24 Rotary Evaporator set-up of the Extraction and Recovery Procedure.

CHAPTER 5

Testing Program

The experimental work associated with this project and subsequent analyses are described in this chapter. Steps in the analysis are also described. This chapter describes three stages of the experiment that all together encompass five parts.

Stage-I of the testing program included three parts. In this stage, tests were conducted to classify the original asphalts by Superpave performance grades. Asphalts extracted and recovered from field samples (both truck mix samples and in-service cores) were also characterized using the Superpave techniques based on the Superpave binder specifications.

In summary, the three parts were:

- Part-1: tests performed on the original asphalt cements retained from 1985 paving projects in Indiana;
- Part-2: discussion the tests performed on the original truck mix samples retained from the 1985 paving projects in Indiana; and
- Part-3: discussion the tests performed on in-service cores taken in 1992/1993.

In Stage-II, Part-4, tests were conducted to study the effect of storage time on asphalt stiffness. Superpave tests were performed on original asphalt binders and on

asphalt extracted from cores collected after construction in the 1996 paving season. The stiffness of asphalt in these cores included stiffening during the period of mixing, laydown, and compaction. In Stage-III, Part-5, stiffness tests were conducted to study the effect of solvents used in asphalt extraction. Extractions were performed using two solvents. An outline for the analysis will be presented as each part is discussed. Figure 5-1 shows the five Parts of this study.

5.1 Stage-I: Part-1 Experimental Work Performed on the Neat Asphalt From 1985

During the 1985 paving season in Indiana, original/tank asphalt binder, and fuel oil from a number of paving projects were sampled and retained. The samples were put in sealed containers and stored at a temperature averaging between 25 and 30°C at Purdue University.

The purpose of the study was to determine long term in-service stiffening of asphalts. The Superpave asphalt binder tests, equipment and associated criteria were standardized and become available in the early 1990s. The asphalts sampled during the 1985 construction season were characterized/classified using the Superpave asphalt binder specification. The test protocol employed is outlined in Figure 5-2. The tests included DSR tests on original asphalts, RTFO aged asphalts (asphalt aged in RTFO) and RTFO+PAV aged asphalts (asphalt subjected to PAV after RTFO aging). In addition, BBR tests were performed on the RTFO+PAV aged asphalts. The purpose of these tests was to classify the original asphalt into Performance Grade asphalts (PG grades). The classification process was performed in accordance with Superpave specifications, (AASHTO - MP1, 1995).

Figure 5-1 The Testing Program / Experimental work conducted in this Study.

Figure 5- 2: Test Protocol Used for PG Classification.

5.2 Stage-1: Part-2 Experimental Work Performed on the Original Truck Mix Samples

One of the goals of this study was to develop data to compare the stiffnesses of the asphalts extracted and recovered from field mixes to the stiffnesses of the asphalts aged in the RTFO test. The assumption is that the RTFO test stiffens asphalt cement in the laboratory an amount comparable to the stiffening that occurs during the mixing and laydown and compaction processes (Anderson and Kennedy, 1993). Loose mixtures (original truck mix samples) from the 1985 projects were collected and retained. Asphalts from these mixtures were extracted and recovered in 1994 and 1995. The stiffness of extracted and recovered asphalts was compared to asphalt stiffness of the same original asphalts after RTFO aging. DSR tests were conducted over a range of temperatures to obtain stiffness data. Figure 5-3 shows the test protocols performed on the field mixes. Mean comparisons (t-tests) to determine if significant difference existed between the two groups were conducted.

Figure 5-3: Test Performed on Truck Mix Samples.

5.3 Stage-1: Part-3 Tests on In-service Cores

In 1992/1993, a series of cores were taken at longitudinal stations along the study pavement sections. Each series consisted of cores at three transverse locations: outside the wheel path (OWP), in the wheel path (IWP) and between the wheel paths (BWP). Outside the wheel path cores were taken 15 cm (6 inch) from the edge of the outside shoulder line. Similarly, in the wheel path cores were taken 60 cm (2-feet) from the edge of the outside shoulder line and between the wheel path cores were taken 182cm (6-feet) from the same reference. For each project section 14 cores were taken from each transverse location. Longitudinally, the cores were spaced 60cm (2-feet) apart. Figure 5-4 illustrates the number, location and size of cores taken from each transverse location.

Cores were extracted from the three previously mentioned locations; OWP, IWP and BWP. A total of fourteen cores were taken at each transverse location. This resulted in a total of forty two cores from each pavement location. The top 9.5mm (3/8 inch) of each core was removed by sawing. The asphalt from this top section and the remainder of the surface layer (approximately 2.5 cm) were extracted and recovered separately. The reason the top section of the core was removed and treated separately was to try to detect near surface aging.

Figure 5-4: Number, Location and Size of Cores Taken from each Transverse Location.

The goal of this effort was to compare complex shear moduli (G^* or stiffness) of the asphalts extracted and recovered from in-service cores with the corresponding values measured on PAV aged asphalt. The test protocol followed for the extracted and recovered asphalts from in-service cores after seven or eight years is illustrated in Figure 5-5. Mean comparisons (t-tests) were performed to determine if differences in asphalt stiffness existed between transverse and vertical pavement locations. Mean comparisons were also performed to determine if differences existed between in-service asphalts from these locations and PAV aged asphalts.

Figure 5-5: Test Protocol Applied to In-service Cores after 7 or 8 years.

Stage-1: Layout of the Experiment

Three groups of tests were performed during the course of this stage. Group I tests represent the test protocol used to determine the neat asphalt performance grades (Figure 5-2). Group II and Group III tests represent test protocols applied to the field samples (truck mix samples and in-service cores – Figure 5-3 and Figure 5-5, respectively). Complex shear modulus (G^* or stiffness) and phase angle (δ) were determined for the original asphalt, RTFO aged asphalt, and the RTFO plus PAV aged asphalt, respectively. In addition, the creep stiffness $S(t)$ and the slope of the creep stiffness curve or m -value were determined for the RTFO plus PAV aged asphalts. Similarly, stiffnesses were determined for the extracted and recovered asphalt from the truck mix samples (HMA) and the field cores. The difference of the means analysis was conducted to relate the results and to determine significance.

The objectives of the analysis were to contrast/compare the results in the following fashion:

- Evaluate the performance of asphalts used in past construction in terms of their PG grading;
- Compare the stiffness of the conditioned asphalt binder after the RTFO aging to the stiffness of asphalt extracted/recovered from the original HMA;
- Compare the stiffness of the RTFO + PAV aged asphalt binder to the stiffness of the extracted/recovered asphalt from in-service cores; and

- Correlate overall pavement condition index (PCI ratings) and individual distresses to binder stiffness.

5.4 Stage-II: Part-4 Effect of Storage Time on Stiffness

An experiment was conducted to study the effect of storage time on the stiffness of the extracted and recovered asphalts from the HMA. The original truck mix samples (loose HMA) were collected and retained during the 1985 paving season in Indiana. However, asphalt extraction and recovery procedures were not conducted until 1992/1993 seven or eight years later. A difference in asphalt stiffness was observed between the extracted and recovered asphalts from these mixes and aged asphalt from the RTFO test. However, there was a question about the effect of the extended storage time on the asphalt.

To address the question, a decision was made to collect new mixtures samples (Superpave mixes) and the original asphalt binders used to produce these mixes. Extraction and recovery procedures were conducted on the new Superpave mixes just after construction (i.e. on cores collected just after construction). DSR tests were performed on both the asphalt binders extracted and recovered from the Superpave mixes and the aged asphalt after the RTFO test. The results reflected the actual stiffness difference that existed between field asphalt (asphalt after mixing and construction) and laboratory (RTFO aged) asphalt without the age time. Storage time (seven or eight years) had been avoided in this experiment.

Stage-II: Layout of Experiment:

Two groups of tests were conducted during the course of this stage. Group I tests represent test protocol used to classify original asphalt binders into performance grades, similar to Stage-I, Figure 5-2. Group II tests represent protocol applied to the Superpave mixtures, again similar to Stage-I, Figure 5-3.

The stiffness difference between the RTFO asphalt binders and extracted and recovered asphalt binders from the Superpave mixture were determined through mean comparisons.

5.5 Stage-III: Part-5 Effect of Solvent Type on Extracted and Recovered Asphalt Stiffness

In this stage, an experiment was conducted to study the effect solvent type used in the extraction and recovery of asphalt binders on measured binder stiffness. Two projects were selected from the 1985 construction season. Truck samples and in-service cores were split for testing. Extractions and recoveries were conducted using two different solvents: toluene (TOL) and trichloroethylene (TCE). Only DSR tests were used to evaluate the effect of solvent type on stiffness.

Stage-III: Layout of the Experiment:

Test protocol applied on the in-service cores is illustrated on Figure 5-3 and Figure 5-5. Mean comparisons were performed to evaluate the effect of solvent type on recovered asphalt stiffness.

CHAPTER 6

Results and Analysis

During the 1985 paving season in Indiana, the original asphalt binder, fuel oil, and HMA (truck mix samples) from a significant number of projects were sampled and retained. Both batch and drum mix plants were used to produce the HMA. In 1988, during phase one of this study, tests were conducted on the neat asphalt binders and on the extracted/recovered asphalt from the original truck mix samples and in-service cores. No distresses were exhibited on those pavements at that time. In a subsequent study initiated in 1992 (phase two), it was planned that the group of pavements cored in 1988 would be re-cored. Unfortunately, none of those pavements were available because they had all been overlaid or rehabilitated.

As the project evolved, equipment to conduct the Superpave asphalt binder tests was generally standardized and became available. As a result, the study was directed toward characterizing the neat and extracted/recovered asphalt binders from the field samples, HMA, and in-service cores, using these test methods. This approach provided an early opportunity to examine a number of features of the Superpave system, including short and long-term aging and criteria related to pavement performance.

The Superpave asphalt binder specification incorporated the Rolling Thin Film Oven (RTFO) and newly developed Pressure Aging Vessel (PAV) tests with limited validation/verification. In the case of the PAV, the preliminary results were generalized/extrapolated to represent five to ten years of in-service aging. Since the asphalt binder specification criteria were set to limit various distresses based on the expected aging, it is important that the expected aging from these adopted laboratory tests be verified. The current research provided an opportunity for this verification. Results of this study include the following:

- PG classification of the original binders retained from the 1985 paving season in Indiana. Comparison of PG grade classification results and recommended Superpave PG grades;
- Evaluation of short (RTFO test) and long term (PAV test) binder aging;
- Study of the effect of storage time on the stiffness of asphalts extracted and recovered from stored loose truck mix samples retained for nine to ten years;
- Comparison between the Rotovap Abson extraction and recovery results using TCE and toluene. The later solvent was recommended by SHRP (Petersen et al, 1994);
- Determination of PG grades for 7 counties in Indiana, based on 9 years of weather data and comparison of those grades to SHRP/Superpave recommended PG grades for the same counties;
- Correlation of rutting or permanent deformation, at high temperature (64°C), and the stiffness (G^*) of the in-service asphalts; and

- Correlation of cracking, at the mean annual pavement temperature in Indiana, and the stiffness (G^*) of the in-service asphalts.

6.1 PG Classification/Characterization of Asphalt Retained from the 1985 Season.

Asphalt binders retained from the 1985-construction season were tested according to the Superpave binder specifications to determine their performance grades, the testing program was described in details in Chapter 5. Tests were conducted at high, intermediate, and low temperatures. These test results are shown in Figures 6-1 through 6-13 for the study projects. Figures 6-1 through 6-13 show the stiffnesses of the neat (original), RTFO aged, and PAV aged asphalts, as well as the stiffnesses of asphalt extracted and recovered from truck mix samples and in-service cores. Generally, these figures show the average values of the pavement transverse locations (i.e. top and bottom of OWP, IWP, and BWP). However, if the top or the bottom asphalt binder stiffness was lower than other locations, as with project 14643 (Figure 6-1), the stiffness results were not included in calculating the average in-service values shown on the plots.

Table 6-1 shows the PG classification of the original binders from the same projects. Table 6-1 was developed based on the Superpave specification shown in Table 2-1. Asphalt binders that pass the Superpave criteria at the specified temperatures are classified into their respective PG grades. Tables 6-2 through 6-5 show recommended SHRP PG grades for the same projects based on the environmental conditions at the 50% and the 98% reliability levels. The tables were

developed/extracted from the “Weather Database for the SUPERPAVE™ Mix Design System” report, (Huber, 1994). Appendix A contains the detailed test results discussed in this chapter.

Analysis/Conclusions

From the classification/characterization results shown in Figures 6-1 through Figure 6-13, Tables 6-1 through 6-5, and stiffness results provided in Appendix A, the following observations were made:

1. All asphalts tested met or exceeded the SHRP/Superpave high temperature grade. Based on a minimum of twenty years temperature data, SHRP researchers developed maps of the required PG grades for the fifty States and the counties within each State. Table 6-2 provides a portion of this database.
2. None of the project asphalts met the Superpave 98% reliability level low temperature grade requirements. Eight project asphalts met the 50% reliability level and the asphalts from three projects did not meet either the 50% or 98% reliability levels.
3. Figures 6-1 through 6-13 show that pronounced differences in stiffness were observed between the RTFO aged asphalts and the extracted and recovered asphalts from the loose truck mix samples retained from the 1985 construction season. Mean comparisons were performed to determine

if the differences were significant. The comparisons are presented in Section 6-2.

4. Figures 6-1 through 6-13 also show that pronounced differences in stiffness were observed between the PAV (RTFO+PAV) aged asphalts and the extracted and recovered asphalts from the in-service cores taken in 1992 and 1993. The PAV aging should produce stiffnesses approximately equal to in-service stiffnesses after five to ten years. Mean comparisons were performed to determine whether the differences were significant. The results of the comparisons are presented in Section 6-3.
5. High variability in recovered asphalt stiffness was observed, both within and between pavement locations, in the core test results. There is a need to account for the in-service stiffness variability based on location within a pavement (i.e. top/bottom of OWP, IWP, and BWP). An analysis is presented in Section 6-3 which addresses the high in-service stiffness variability.

Figure 6-1 Characterization of Project 14643 Asphalt.

Figure 6-2 Characterization of Project 15165a Asphalt.

Figure 6-3 Characterization of Project 15165b Asphalt.

Figure 6-4 Characterization of Project 15165a1 Asphalt.

Figure 6-5 Characterization of Project 15165b1 Asphalt.

Figure 6-6 Characterization of Project 15308 Asphalt.

Figure 6-7 Characterization of Project 15502 Asphalt.

Figure 6-8 Characterization of Project 15404 Asphalt.

Figure 6-9 Characterization of Project 15630 Asphalt.

Figure 6-10 Characterization of Project 15173 Asphalt.

Figure 6-11 Characterization of Project 15174 Asphalt.

Figure 6-12 Characterization of Project 15309 Asphalt.

Figure 6-13 Characterization of Project 15617 Asphalt.

Table 6-1 PG Classification for Asphalts Retained from 1985 Construction Season.

Project	PG Classification
14643	64-22
15165a	64-22
15165b	64-22
15165a1	64-22
15165b1	64-22
15308	64-22
15502	64-28
15404	64-22
15630	64-28
15173	70-22
15174	64-16
15309	64-22
15617	58-22

Table 6-2 Projects tested Identified by Counties and Nearest SHRP Weather Station (Huber, 1994).

ST	County ID	Project no.	Station*
IN	Morgan	15502	Martinsville 2 sw
IN	Crawford	15630	English
IN	Shelby	15165 & 15308	Shelbyville Sewage Plt
IN	Tippecanoe	15619	West Lafayette
IN	Knox	15174	Edwardsport Power Plant
IN	Dubois	15173	Dubois s Ind Forage Frm
IN	Wayne	14643	Cambrige City
IN	Johnson	15404	Franklin 2 Nine
IN	Rush	15309	Rushville Sewage Plant

*SHRP-A-648A, (Huber, 1994)

Table 6-3 Longitude, Latitude and Elevation – Low and High Air Temperatures for Projects Tested (Huber, 1994).

ST	County ID	Project no.	*	*	*	Air temp*			
			Long	lat	elev	Low		High	
						avg	Std	avg	Std
IN	Morgan	15502	86.45	39.4	186	-23	4	33	2
IN	Crawford	15630	86.47	38.28	156	-23	7	33	2
IN	Shelby	15165 & 15308	85.78	39.52	229	-22	5	33	2
IN	Tippecanoe	15619	86.93	40.42	183	-24	3	33	2
IN	Knox	15174	87.23	38.8	140	-21	4	34	2
IN	Dubois	15173	86.7	38.45	210	-22	4	33	2
IN	Wayne	14643	85.17	39.82	290	-24	4	32	2
IN	Johnson	15404	86.07	39.52	235	-21	4	33	2
IN	Rush	15309	85.45	39.6	293	-23	4	33	2

*SHRP-A-648A, (Huber, 1994)

Table 6-4 SHRP Recommended 50% Reliability and PG Grades for Projects Tested (Huber, 1994).

ST	county ID	Project no.	50% reliability*						
			Temperatures				Binder Grade		
			max air	max pvt	min air	min pvt	PG	HT	LT
IN	Morgan	15502	33	53	-23	-23	PG	58	-28
IN	Crawford	15630	33	54	-23	-23	PG	58	-28
IN	Shelby	15165 & 15308	33	53	-22	-22	PG	58	-22
IN	Tippecanoe	15619	33	53	-24	-24	PG	58	-28
IN	Knox	15174	34	55	-21	-21	PG	58	-22
IN	Dubois	15173	33	54	-22	-22	PG	58	-22
IN	Wayne	14643	32	52	-24	-24	PG	58	-28
IN	Johnson	15404	33	53	-21	-21	PG	58	-22
IN	Rush	15309	33	53	-23	-23	PG	58	-28

*SHRP-A-648A, (Huber, 1994)

Table 6-5 SHRP Recommended 98% Reliability and PG Grades for Projects Tested (Huber, 1994).

ST	county ID	Project no.	98% reliability*						
			Temperatures				Binder Grade		
			max air	max pvt	min air	min pvt	PG	HT	LT
IN	Morgan	15502	37	57	-31	-31	PG	58	-34
IN	Crawford	15630	37	58	-37	-37	PG	58	-40
IN	Shelby	15165 & 15308	37	57	-32	-32	PG	58	-34
IN	Tippecanoe	15619	37	57	-30	-30	PG	58	-34
IN	Knox	15174	38	58	-29	-29	PG	64	-34
IN	Dubois	15173	37	57	-30	-30	PG	58	-34
IN	Wayne	14643	36	56	-32	-32	PG	58	-34
IN	Johnson	15404	37	57	-29	-29	PG	58	-34
IN	Rush	15309	37	57	-31	-31	PG	58	-34

*SHRP-A-648A, (Huber, 1994)

6.2 Evaluation of RTFO Aging

RTFO aging is used to simulate aging during HMA production and construction. Thus, the stiffness of original binders after RTFO aging should be comparable to the stiffness of asphalt extracted and recovered from loose truck mix samples collected for this research in 1985. Figure 6-14 shows the difference in stiffnesses between RTFO test results on the project asphalts and the extracted and recovered asphalts from the truck mix samples at 64°C. It is clear from the figure that aging occurring during construction was greater than that induced by the RTFO test. Larger variability was observed between the stiffness of the asphalt extracted and recovered from the truck mix samples compared to the stiffness of the binders after the RTFO aging.

Analysis/Conclusions

Mean comparisons (t-tests) were conducted to determine if significant differences existed between stiffnesses of RTFO aged asphalts and extracted and recovered asphalts from truck mix samples. Table 6-6 summarizes the results of this analysis. This analysis was performed on the pooled data at 64°C. As shown in Table 6-6 the difference in means is significant at the 95% reliability level (P-Value = 7.44E-8). Appendix B provides detailed results of the statistical analysis. The SAS program, including the original stiffness data of the two groups (stiffnesses of RTFO aged asphalts and extracted and recovered asphalts from truck mix samples), and the SAS output are also provided in Appendix B.

The common belief is that the RTFO laboratory aging procedure is comparable to aging occurring during production and construction. The data suggest that this is not the case. In Section 6-4 further investigation of the effect of storage time on the stiffness of the extracted and recovered asphalt, from truck mix samples retained from 1985 construction season, will be presented.

Figure 6-14 Stiffnesses of the Aged Asphalts After RTFO Aging and the Extracted and Recovered Asphalts from the Loose Truck Mix Samples, at 64°C.

**Table 6-6 Comparison of Means Between Stiffnesses of RTFO aged Asphalts
and Asphalts Extracted and Recovered from Truck Mix Samples
at 64°C.**

t-Test: Paired Two Sample for Means*		
	<i>RTFO@64C</i>	<i>MIX@64C</i>
Mean	3.57044025	4.05172028
Variance	0.01285109	0.028318354
Observations	12	12
Pearson Correlation	0.54822869	
Hypothesized Mean Difference	0	
Df	11	
T Stat	-11.7151498	
P(T<=t) one-tail	7.4427E-08	
T Critical one-tail	1.79588369	
P(T<=t) two-tail	1.4885E-07	
t Critical two-tail	2.20098627	

*64°C is the maximum expected pavement temperature in Indiana.

6.3 Evaluation of PAV Aging

The PAV aging test was developed to simulate the changes in rheological characteristics of asphalt binders after several years in-service (ASSHTO – PP1, 1995). Thus, the stiffness of PAV aged asphalts (i.e. RTFO+PAV) should be comparable to the stiffness of the extracted and recovered asphalt after seven or eight years in-service. Figure 6-15 shows the stiffness of the PAV aged asphalts and the stiffness of the extracted and recovered asphalts from the in-service cores. The figure clearly shows that the stiffness of the PAV aged binder was less than that of the field aged binders after seven to eight years of service. In the following sections, analysis and conclusions are offered with regard to the difference between the PAV aging and the in-service aging followed by a reliability analysis and conclusions relative to PAV and in-service variability.

Analysis/Conclusions

Mean comparisons (t-tests) were performed to determine if significant differences existed between the stiffness of PAV aged asphalts and extracted and recovered asphalts from in-service cores. Table 6-7 summarizes the results of this analysis. This analysis was performed on the pooled data at 16°C, which approximately represents the mean annual pavement temperature in Indiana. Table 6-7 shows the difference in means to be significant at the 95% reliability level (P-Value = 0.0007). Appendix C provides detailed results of the statistical analysis. The SAS program, including the raw stiffness measurements, and the SAS output are also

provided in Appendix C. The following conclusions were drawn from the statistical analyses:

- There was significant difference between in-service asphalt stiffness (i.e. Top and Bottom of OWP, IWP, and BWP) and the stiffness of the PAV aged binders;
- Significant differences in binder stiffness were observed between transverse pavement locations;
- There was not a significant difference in the stiffness of the asphalt between the top and bottom of the cores;
- There were no interactions between sample, top/bottom, project, and transverse pavement locations; and
- There was some interaction between transverse pavement locations and top/bottom of the sample.

The AASHTO provisional standard (PP1-1995) states that the PAV aging represents five to ten years of in-service aging. The projects included in this study were in-service seven or eight years. It is clear that the PAV aging does not simulate five to ten years of in-service aging in Indiana. As presented previously in Chapter 2, SHRP researchers acknowledged that the PAV aging did not always produce aging comparable to five to ten years of in-service aging. Additionally, the aging occurring at different pavement locations is different and the Superpave system does not account for this.

Figure 6-15 Stiffnesses of the PAV Aged Asphalts and Extracted and Recovered Asphalts from In-service Cores at 16°C.

Table 6-7 Comparison of Means Between Stiffnesses of PAV aged Asphalts and Asphalts Extracted and Recovered from In-service Cores at 16°C.

t-Test: Paired Two Sample for Means		
	Pav@16	In-Service@16
Mean	7.071753779	7.405745539
Variance	0.024110033	0.039491911
Observations	13	13
Pearson Correlation	-0.343064524	
Hypothesized Mean Difference	0	
Df	12	
t Stat	-4.135962798	
P(T<=t) one-tail	0.000690366	
t Critical one-tail	1.782286745	
P(T<=t) two-tail	0.001380732	
t Critical two-tail	2.178812792	

*16°C is the approximate average annual pavement temperature in Indiana

Reliability Analysis/Conclusions

The current Superpave system assumes that aging occurring in asphalt binders is not affected by in-service asphalt binder locations (transverse and/or vertical pavement locations). However, the statistical analysis summarized above showed significant in-service stiffness variability between pavement locations. Additionally, there were significant differences in stiffnesses between the PAV aged asphalts and asphalts extracted and recovered from in-service cores, both within and between projects.

The averages and the standard deviations were computed for PAV and in-service asphalt stiffnesses. These calculations were made at 28°C and 16°C. The calculations are summarized in Table 6-8. Table 6-8 shows the computed bias estimates or the difference in stiffness between the PAV aged asphalts and asphalts extracted and recovered from the in-service cores. The calculations are the actual reliability of the PAV prediction of the in-service asphalt stiffnesses, based on the field data after seven or eight years in Indiana.

The total “PAV” variance does not include variability in in-service aging due to pavement location (i.e. top and bottom of OWP, IWP, and BWP). Recall that previous statistical analysis showed that in-service location had a significant effect on binder stiffness. Therefore, it is essential that this variability be included in the total variance.

The following question had to be answered. What would the predicted PAV stiffness be, if the PAV accounted for the in-service stiffness variability.

The following analysis was performed to answer the question. The average PAV stiffness added to the average computed bias estimates plus twice the standard deviation of the bias estimates would represent the PAV predicted stiffness at the 98% reliability level. This estimate accounts for in-service variability in pavement location in Indiana. This is expressed in the following equation:

$$PAV \text{ Estimate (98\%)} = \text{Average PAV} + \text{Average Bias} + 2 \text{ Standard Deviation of the Biases.}$$

(Equation 6-1)

In other words, the “PAV estimate” is the prediction of the required PAV stiffness at the 98% reliability level taking the in-service stiffness variability, in both transverse and vertical pavement location, into consideration. Table 6-8 shows the PAV estimate as calculated from Equation 6-1 presented above, at the 98% reliability level at 28°C and 16°C (7.5, 7,924). Additionally, Table 6-8 shows the actual reliability level of PAV aged asphalt when the effects of pavement location are incorporated into the reliability estimate. These reliability levels are the ratio of the average PAV stiffness and estimated PAV stiffness at the 98% reliability level. This represents a reduction in reliability of ten to fourteen percent.

Figure 6-16 shows the distribution of the in-service asphalt stiffnesses from transverse and vertical pavement locations and the distribution of the PAV aged asphalts in accordance with the current AASHTO PPI procedure. It is clear from Table 6-8 and Figure 6-16 that there is a need to incorporate the effect of pavement location on overall reliability.

**Table 6-8 Average, Standard Deviation, and Bias Estimates of PAV
and In-Service Stiffness**

Temperature 28°C	
Average Biases	0.5167
Standard Deviation of Bias	0.3391
Average PAV	6.2933
Standard Deviation of PAV	0.1833
Variance Bet. PAV	0.0336
Average In-service	6.8100
Standard Deviation In-service	0.2636
Variance In-service	0.0695
Total PAV Variance Between/Within Projects	0.0899
Total In-service Variance Between/Within Projects	0.7349
PAV at 98% =	7.5
PAV Reliability =	83.90%
Temperature 16°C	
Average Biases	0.3929
Standard Deviation of Bias	0.2630
Average PAV	7.0058
Standard Deviation PAV	0.1943
Variance Bet. PAV	0.0377
Average In-service	7.3987
Standard Deviation In-service	0.2077
Variance In-service	0.0431
Total PAV Variance Between/Within Projects	0.6083
Total In-service Variance Between/Within Projects	0.7302
PAV at 98% =	7.9240
PAV Reliability =	88%

Figure 6-16 PAV Aged Asphalt and In-service Aged Asphalt Distributions.

6.4 Storage Time Effects

As indicated in Section 6.2, stiffnesses of RTFO aged asphalts were significantly different from stiffnesses of extracted and recovered asphalts from the loose truck mix samples retained from the 1985 construction season. However, the asphalt samples and truck mixture samples had been in storage for seven to eight years. A small study was conducted to address whether the storage by itself contributed to the difference in stiffnesses of the RTFO and in-service aged binders.

For this study, samples from two Superpave projects were randomly selected during the 1996 construction season. Original asphalts and core samples (just after construction) were collected from these two projects. Recall that RTFO aging is used to simulate aging occurring during HMA production and construction. Thus, the stiffness of the binders after RTFO aging should be comparable to the stiffness of the asphalt extracted and recovered from core samples just after construction. The test protocol applied to mix samples described in Chapter 5 was followed. Project asphalts were also characterized and classified as in accordance with AASHTO MP1 described in Section 6-1.

The results of the characterization/classification are presented in detail in Appendix D. Figures 6-17 and 6-18 show the results of characterization of the binders used on the Superpave projects, identified as projects A and B, respectively. A mean comparison was performed to assess the effect of storage time on binder stiffness. The detailed statistical analysis is presented in Appendix D. Table 6-9 presents a summary

of the mean comparison performed using stiffness data obtained on the materials from the two Superpave Projects.

Figure 6-17 Characterization of Superpave Project A Asphalt.

Figure 6-18 Characterization of Superpave Project B Asphalt.

Table 6-9 Summary of Mean Comparison Performed on the Two Superpave Projects from 1996.

t-Test: Paired Two Sample for Means*		
	<i>RTFO-96</i>	<i>Mix-96</i>
Mean	5768.01704	6880.191205
Variance	59471084.9	88054781.7
Observations	32	32
Pearson Correlation	0.9934803	
Hypothesized Mean Difference	0	
Df	31	
t Stat	-3.25356338	
P(T<=t) one-tail	0.0013762	
t Critical one-tail	1.69551868	
P(T<=t) two-tail	0.00275241	
t Critical two-tail	2.03951458	

*Pooled data at all temperatures

t-Test: Paired Two Sample for Means**		
	<i>RTFO-96 @ 64C</i>	<i>Mix-96 @64C</i>
Mean	1589.90231	1835.103092
Variance	48350.6717	5036.711662
Observations	4	4
Pearson Correlation	0.8438046	
Hypothesized Mean Difference	0	
Df	3	
t Stat	-2.98164259	
P(T<=t) one-tail	0.02926005	
t Critical one-tail	2.35336302	
P(T<=t) two-tail	0.0585201	
t Critical two-tail	3.18244929	

**Pooled data at all temperatures

Analysis/Conclusions

There was a significant difference in stiffnesses when the data at all temperatures from both projects were pooled for comparison ($P=0.0013$). Similarly, there was a significant difference in stiffnesses when the data at 64°C from both projects were pooled for comparison ($P=0.0292$). This suggests that the RTFO aging does not produce the degree of aging occurring during production and construction of HMA in Indiana.

Figure 6-19 shows the average differences in stiffnesses between RTFO aged asphalts and asphalts extracted and recovered from cores after production and construction in 1996. It is clear that the stiffness of the loose mix binder was greater than the RTFO aged binder at all temperatures. However, there was a difference in stiffness between materials sampled in 1985 and those sampled in 1996. Figure 6-20 shows the difference in stiffness between the 1985 and 1996 samples. The stiffness differences between RTFO aged asphalts and the asphalts extracted and recovered from samples taken in 1996 is less than the stiffness differences between the RTFO aged asphalts and the asphalts extracted and recovered from the truck mix samples from 1985. The difference in stiffness between 1996 and 1985 can be attributed to the “storage time effect.” As shown from Figure 6-20 the binder from the 1985 mixes had approximately six times the stiffness of the binder from the mixes sampled in 1996.

Based on these findings, the best policy would have been to test both the neat (original) asphalt and asphalt extracted and recovered from loose truck mixture as

soon as possible. Unfortunately, researchers associated with the original study decided the storage conditions, not realizing the effects of such. However, even short storage times (i.e. two weeks) can have an effect on asphalt binder in loose mixtures. This is important because material preparation and testing is time consuming and storage is unavoidable. This leads to the suggestion that other storage conditions should be investigated, i.e. cold storage, storage in containers with inert gas.

Figure 6-19 Stiffnesses Differences Between RTFO aged asphalts and Asphalts Extracted and Recovered from Cores after Production and Construction in 1996.

Figure 6-20 Stiffnesses Differences Between RTFO Aged Asphalts and Asphalts Extracted and Recovered from Mixes in 1996 and 1985.

6.5 Effect of Extraction Solvent on Stiffness

Questions were also raised about the effect of the type of solvent used in the asphalt extraction and recovery processes on the stiffnesses of asphalts extracted and recovered from loose mix samples and in-service cores. A small study was conducted to determine whether the solvent (TCE) used contributed to the difference in stiffnesses of the asphalts extracted and recovered from these field samples.

Materials from two projects sampled during the 1985 construction season were used to assess the effect of extraction solvent type on asphalt stiffness. In-service samples from these two projects (truck mix samples and in-service cores) were split into two separate samples for testing. The centrifuge/rotary evaporator procedures were used to extract and recover the asphalts from the field samples using two different solvents, namely trichloroethylene (TCE) and Toluene. Traditionally, extraction and recovery procedures have used TCE, while, the SHRP program (SHRP B-006, 1995) recommended the use of Toluene (TOL). The testing protocol described in Chapter 5 (Figure 5-3 and Figure 5-5) was followed on these field samples. Asphalts from half of the samples were extracted using TCE, while, asphalts from the other half were extracted using Toluene.

Figure 6-21 shows the stiffnesses of the asphalts extracted and recovered from loose truck mixtures from projects 15165a and 15165b at 64°C. Figures 6-22 and 6-23 show the stiffnesses of the asphalts extracted and recovered from in-service cores from projects 15165a and 15165b at 25°C, respectively. The detailed results of the statistical analysis performed on the field samples are presented in Appendix E. Table

6-10 summarizes the mean comparison performed on the truck mix samples and Table 6-11 summarizes the mean comparison performed on the in-service cores.

Figure 6-21 Stiffnesses of Asphalts Extracted and Recovered from Truck Mix Samples on projects 15165a and 15165b at 64°C.

Table 6-10 Stiffnesses Mean Comparison in Means Analysis Performed on the Truck Mix Samples.

<i>t-test: Paired Two Sample for Means</i>	<i>TOL</i>	<i>TCE</i>
Mean	97917.74216	113626.3098
Variance	20176998657	25310428566
Observations	32	32
Pearson Correlation	0.984582979	
Hypothesized Mean Difference	0	
Df	31	
t Stat	-2.827917245	
P(T<=t) one-tail	0.004068897	
t Critical one-tail	1.695518677	
P(T<=t) two-tail	0.008137793	
t Critical two-tail	2.039514584	

Analysis/Conclusions

Figure 6-21 shows a consistent difference in stiffnesses of the asphalts extracted and recovered from loose truck mixtures with TCE and TOL. The stiffness of the asphalts extracted using TOL are less than those extracted with TCE. The mean comparison (Table 6-10) showed the difference to be significant ($P=0.0041$).

The same observation was not made for the field cores however. Figures 6-22 and 6-23 show the stiffnesses measured on the asphalt extracted using both TOL and TCE as a function of transverse and vertical pavement location for Projects 15165a and 15165b, respectively. In five of the eight cases the stiffness of asphalts extracted using TCE are less than the stiffness associated with the use of TOL. A mean comparison of the stiffnesses associated with each solvent was performed. There was no significant difference between stiffnesses of asphalts extracted and recovered, using TCE and TOL, from in-service cores. Table 6-11 is a summary of the mean comparisons performed for the material tested from each set of field cores. Note that significant difference is not observed for either site ($P=.4637$, $P=.2909$).

In general, the variability of stiffnesses of asphalts extracted and recovered using TOL was lower than those extracted and recovered using TCE. This obvious from the lower variance associated with TOL test results shown in Tables 6-10 and 6-11. Both solvents (TCE and TOL) can be utilized to extract and recover asphalts from field samples, because no significant statistical difference was observed between the solvents. However, TOL provided lower variability in stiffnesses. An additional study is recommended to verify the conclusions presented herein. The new study should

Figure 6-22 Stiffnesses of Asphalts Extracted and Recovered, Using TCE and TOL, from In-Service Cores from Project 15165a.

Figure 6-23 Stiffnesses of Asphalts Extracted and Recovered, Using TCE and TOL, from In-Service Cores from Project 15165b.

encompass more projects, several PG grades, and extraction and recovery procedure should be performed on “blank” (i.e. neat asphalt mixed with solvents) samples using different soaking times.

Table 6-11 Summarizes Stiffnesses Difference in Means Analysis Performed on the In-Service Cores Extracted and Recovered Using TCE and TOL.

t-Test: Paired Two Sample for Means, 15165a.		
	<i>TCE@25</i>	<i>TOL@25</i>
Mean	1.1E+07	10456749
Variance	3.4E+13	8.74E+12
Observations	6	6
Pearson Correlation	0.5507	
Hypothesized Mean Difference	0	
Df	5	
t Stat	0.09578	
P(T<=t) one-tail	0.46371	
t Critical one-tail	2.01505	
P(T<=t) two-tail	0.92742	
t Critical two-tail	2.57058	
t-Test: Paired Two Sample for Means, 15165b.		
	<i>TCE@25</i>	<i>TOL@25</i>
Mean	1.3E+07	12150000
Variance	1.8E+13	5E+09
Observations	2	2
Pearson Correlation	1	
Hypothesized Mean Difference	0	
Df	1	
t Stat	0.29094	
P(T<=t) one-tail	0.40988	
t Critical one-tail	6.31375	
P(T<=t) two-tail	0.81976	
t Critical two-tail	12.7062	

6.6 Weather Data – PG Calculations for Seven Counties in Indiana

The purpose of this task was to compare PG grades recommended by SHRP procedures to those that would be required by the same process using actual weather data observed near each project over the time period the project pavement had been in-service. Weather data were collected from seven Indiana counties over the nine year period the pavements had been in-service. The calculated PG grades required based on this in-service climatic data, were compared to recommended SHRP PG grades for the same counties based on environmental conditions at the 50% and the 98% reliability levels. Figure 6-24 shows locations of the counties in Indiana. These counties represent the bulk of the project locations considered in this research. The SHRP PG grades for these counties were developed/extracted from the “Weather Database for the SUPERPAVETM Mix Design System” report, (Huber, 1994).

Figures 6-25 through 6-31 show the high and low air temperature distributions of these seven counties. Tables 6-12 through 6-18 summarize the temperature data and the computed PG grades of these counties. The (NOAA) data presented in Tables 6-12 through 6-18 is in degrees Fahrenheit as prepared by NOAA. The temperature distributions presented in Figures 6-25-through 6-31 have been presented in degrees Celsius to be consistent with Superpave. The distributions presented are simply normal distributions determined from means and standard deviations of the air temperatures. Table 6-19 shows the comparison between the computed PG grades and the recommended SHRP PG grades. The weather data summary is presented in Appendix F.

Analysis/Conclusions

The PG grade requirements determined from the nine years of in-service climatic data are in agreement with the SHRP PG grade requirements in nine of fourteen cases when both 50% and 98% reliability levels are considered. Computed PG grades that differ than the recommended SHRP PG grades are bolded/shaded in Table 6-19. Differences were observed at both high and low temperature grades and at both 50% and 98% reliability levels for two of the seven locations. SHRP recommended PG grades are based on twenty years of data. However, the computed PG grades are based only on nine years of weather data. This could be part of reason for the discrepancies.

Optimum PG grades should be computed based on both air-temperature data and pavement temperature data. The best method of obtaining pavement temperatures is through the installation of sensors at multiple depths in a pavement. Based on this limited study, it is suggested that recommended SHRP PG grades for Counties in Indiana be verified. Also, loading conditions and traffic speed should be taken into considerations when selecting PG grades. The Superpave binder specifications recommended using a higher high temperature grade for slower traffic or heavier loading conditions, using two grades higher for stop and go type traffic, and using two grades higher for the combination of slower traffic and heavier loading conditions.

Figure 6-24 Locations of Selected Counties for Weather Data Study in Indiana.

Figure 6-25 High and low Temperature Distribution from English Weather Station in Crawford County.

Table 6-12 High and Minimum Air Temperature of English City in Crawford County and Computed PG Grades.

English	7 Day Average		Minimum
	Maximum	St. Dev.	
1985	93.14285714	0.899735411	-19
1986	95.42857143	0.786795792	-4
1987	96.71428571	1.603567451	-2
1988	100.2857143	1.253566341	-9
1989	91.85714286	0.690065559	-21
1990	95.28571429	1.253566341	1
1991	97.14285714	1.463850109	-2
1992	92.85714286	1.463850109	-2
1993	94.57142857	0.786795792	-4
AVG.	95.25396825		-6.888888889
ST. DEV.	2.580121989		7.912718314
Computed PG Grades			
98%		64	-34
50%		58	22

Figure 6-26 High and low Temperature Distribution from Martinsville Weather Station in Morgan County.

Table 6-13 High and Minimum Air Temperature of Martinsville City in Morgan County and Computed PG Grades.

Martinsville	7 Day Average		Minimum
	Maximum	St. Dev.	
1985	90.14285714	0.899735411	-21
1986	95.28571429	1.380131119	0
1987	92.57142857	1.133893419	-7
1988	100	0.577350269	-4
1989	90.71428571	0.755928946	-22
1990	91.28571429	1.253566341	-2
1991	93.57142857	0.975900073	-3
1992	87.42857143	0.975900073	-7
1993	91.85714286	1.069044968	-11
	92.53968254		-8.555555556
	3.561971606		8.017342314
Computed PG Grades			
98%		58	-34
50%		58	-28

Figure 6-27 High and low Temperature Distribution from Perrysville Weather Station in Fountain County.

Table 6-14 High and Minimum Air Temperature of Perrysville City in Fountain County and Computed PG Grades.

Perrysville	7 Day Average		
	Maximum		Minimum
1985	91.28571429	1.112697281	-26
1986	94.14285714	0.690065559	-6
1987	94.42857143	0.786795792	-11
1988	102.7142857	1.380131119	-13
1989	94.28571429	1.704336206	-23
1990	95.42857143	1.133893419	-7
1991	97	1.527525232	-3
1992	91	2.449489743	-7
1993	92.57142857	0.534522484	-7
AVG	94.76190476		-11.44444444
SD	3.54346193		7.970431467
Computed PG Grades			
98%		64	-34
50%		58	-28

Figure 6-28 High and low Temperature Distribution from Richmond Weather Station in Wayne County.

Table 6-15 High and Minimum Air Temperature of Richmond City in Wayne County and Computed PG Grades.

Richmond	7 Day Average		
	Maximum		Minimum
1985	90.85714286	1.214985793	-25
1986	93	0.577350269	-2
1987	88.57142857	0.975900073	-3
1988	98.85714286	1.069044968	-6
1989	91.85714286	1.573591585	-22
1990	92.57142857	1.511857892	1
1991	94.14285714	1.069044968	1
1992	88.42857143	1.397276262	-8
1993	91.71428571	1.112697281	-9
AVG	92.22222222		-8.111111111
SD	3.130748681		9.439868172
Computed PG Grades			
	98%	58	-34
	50%	58	-28

Figure 6-29 High and low Temperature Distribution from Rushville (SHRP) Weather Station in Rush County.

Table 6-16 High and Minimum Air Temperature of Rushville City in Rush County and Computed PG Grades.

Rushville	7 Day Average		Minimum
	Maximum		
1985	90.28571429	1.112697281	-24
1986	92.14285714	0.377964473	0
1987	91	0.577350269	-4
1988	99.71428571	0.487950036	-5
1989	91.71428571	0.951189731	-23
1990	91.71428571	1.799470822	1
1991	93.14285714	1.214985793	0
1992	87.85714286	1.214985793	-7
1993	90.85714286	0.690065559	-8
AVG	92.04761905		-7.777777778
SD	3.228539823		9.457507306
Computed PG Grades			
98%		58	-34
50%		58	-28

Figure 6-30 High and low Temperature Distribution from Shelbyville (SHRP) Weather Station in Shelby County.

Table 6-17 High and Minimum Air Temperature of Shelbyville City in Shelby County and Computed PG Grades.

Shelbyville	7 Day Average		
	Maximum		Minimum
1985	90.71428571	0.487950036	-22
1986	93.85714286	0.899735411	9
1987	90.42857143	0.786795792	17
1988	100.4285714	0.786795792	-4
1989	92.14285714	0.899735411	-22
1990	94	2.380476143	1
1991	94.57142857	1.133893419	1
1992	88.42857143	0.786795792	-6
1993	91.57142857	1.272418021	-8
AVG	92.9047619		-3.777777778
SD	3.442679308		12.88194257
Computed PG Grades			
98%		58	-40
50%		58	-22

Figure 6-31 High and low Temperature Distribution from Vincennes Weather Station in Knox County.

Table 6-18 High and Minimum Air Temperature for Vincennes City in Knox County and Computed PG Grades.

Vincennes	7 Day Average		
	Maximum		Minimum
1985	92.57142857	1.133893419	-19
1986	95.42857143	0.534522484	-1
1987	95.57142857	1.511857892	0
1988	102.1428571	1.069044968	-2
1989	94.28571429	0.755928946	-21
1990	96.57142857	1.397276262	2
1991	97.85714286	1.214985793	4
1992	91.71428571	0.755928946	0
1993	94.71428571	0.755928946	-5
AVG	95.65079365		-4.666666667
SD	3.07484122		9.055385138
Computed PG Grades			
98%		64	-34
50%		58	-22

Table 6-19 Comparison between the Computed PG grades and the Recommended PG grades, at the 98% and 50% Reliability levels.

	98% Reliability Level	50 Reliability Level
	English Weather Station	
Computed PG Grades	64-34	58-22
SHRP PG Grades	58-40	58-28
	Martinsville Weather Station	
Computed PG Grades	58-34	58-28
SHRP PG Grades	58-34	58-28
	Perrysville Weather station	
Computed PG Grades	64-34	58-28
SHRP PG Grades	58-34	58-28
	Richmond Weather station	
Computed PG Grades	58-34	58-28
SHRP PG Grades	58-34	58-28
	Rushville Weather Station	
Computed PG Grades	58-34	58-28
SHRP PG Grades	58-34	58-28
	Shelbyville Weather Station	
Computed PG Grades	58-34	58-28
SHRP PG Grades	58-34	58-28
	Vincennes Weather Station	
Computed PG Grades	64-34	58-22
SHRP PG Grades	58-34	58-28

6.7 Correlation to Pavement Performance.

The correlation of fundamental material properties with field performance is the ultimate objective in any pavement materials study. Therefore, correlations between in-service asphalt stiffnesses and rutting and cracking were a major objective of this task. The condition of the field pavements considered in this research were correlated to asphalt stiffness. The details of the condition surveys performed on pavement sections are presented in Appendix G. The projects included in this study are shown in Figure 6-32. Simple relationships between asphalt stiffness and rutting and cracking deduct values, determined from the condition surveys, were developed and analyzed.

6.7.1 Correlation between Stiffness and Rutting Deduct Value

Figure 6-33 shows the relationship between the stiffnesses of the RTFO aged asphalts and rutting at 64°C. Figure 6-34 shows the relationship between the stiffnesses of the asphalts extracted and recovered from loose truck mix samples and rutting at the same temperature. The RTFO and loose mix stiffnesses were selected for rutting analysis because if rutting occurs in HMA layers, it normally occurs early in the pavement life when the stiffness of the HMA is relatively low. It was assumed that the observed rutting had occurred in the HMA.

Analysis/Conclusion

It is widely accepted that rutting or permanent deformation is a mix-related distress (Galal and White, 1997). Only in the case of a weak aggregate matrix does the asphalt binder play an important role in resisting the rutting potential in HMA pavements. Other factors such as aggregate gradation, particle shape, and surface texture; HMA asphalt content and volumetric properties (e.g. VMA); and in-place density have more pronounced affects on rutting. Figures 6-33 and Figure 6-34 show very poor relationships between asphalts stiffness and rutting deduct value. However, the figures do indicate that with an increase in asphalt stiffness, rutting deduct value decreases. The trends are consistent with conventional thinking. The poor relationships could partially be due to the assumption that all observed rutting was due to permanent deformation in the HMA surface layer. If some permanent deformation were occurring in lower unbound structural layers the relationship would be expected to be poor.

Selecting an asphalt binder that posses an acceptable stiffness definitely would enhance matrix capabilities to resist rutting potential in HMA pavements. Higher stiffnesses are preferable to guard against rutting potential in HMA mixes.

Figure 6-32 Projects Locations Included in this Study.

Figure 6-33 Relationship between RTFO Stiffnesses and Rutting at 64°C.

Figure 6-34 Relationship between Truck mix Stiffnesses and Rutting at 64°C.

6.7.2 Correlation between In-service Stiffnesses and Cracking

Figures 6-35 and 6-36 show correlations between the stiffness of the asphalts extracted and recovered from in-service cores and cracking deduct value at the mean annual pavement temperature in Indiana (i.e. 16°C). Figures 3-37 and 6-38 show the same correlations at the average annual air temperature in Indiana (i.e. 19°C). Figures 6-36 and 6-38 were generated after removing potential outliers. It is important to note that for all forms of cracking, not just fatigue cracking, were used in this analysis. The reason in-service stiffness (after seven or eight years) was correlated with cracking deduct value was simply because cracking is not expected early in a pavements life. It is expected to occur when the binder has aged (stiffened) and became more brittle.

Analysis/Conclusions

The figures show strong relationships between the stiffness of asphalt extracted and recovered from in-service cores and cracking deduct value. All figures indicate that lower asphalt stiffnesses is associated with lower cracking, at the mean annual pavement temperature and average annual air temperature. Regardless of which set of relationships (with or without potential outliers) are considered, the conclusions remains the same. The relationships suggest that for the binders used in Indiana, at least in 1985, the binder stiffness after several years of in-service should be limited to 2.5 – 3.0MPa. The mixtures that incorporated binders that possessed stiffnesses approaching 5.0MPa as, recommended by SHRP, showed excessive cracking. This observation should be validated in future research, after an appropriate long-term binder aging method is identified.

There is a need to balance asphalt stiffnesses at high and low temperature. The asphalt should be selected based on both climatic and loading conditions. Selecting PG grades at higher reliability levels would ensure higher asphalt stiffnesses at high temperatures and lower asphalt stiffnesses at low temperature.

Figure 6-35 Correlation Between Stiffnesses and Deduct Value of Cracking at the Mean Annual Pavement Temperature in Indiana at 16°C, With Outliers.

Figure 6-36 Correlation Between Stiffnesses and Deduct Value of Cracking at the Mean Annual Pavement Temperature in Indiana at 16°C, Without Outliers.

Figure 6-37 Correlation Between Stiffnesses and Deduct Value of Cracking at the Average Annual Air Temperature in Indiana at 19°C, With Outliers.

Figure 6-38 Correlation Between Stiffnesses and Deduct Value of Cracking at the Average Annual Air Temperature in Indiana at 19°C, Without Outliers.

CHAPTER 7

Conclusions and Recommendations

Hot mix asphalt (HMA) pavements exhibit various types of distress including permanent deformation (rutting), fatigue cracking, and low temperature cracking. In past several decades, reports of these types of distress have increased in the U.S. and Canada. Asphalt technologists have associated the increases with changes in asphalt binder characteristics because of new crude sources resulting from the trade embargo in the early 1970's and/or changes in refining processes. The increased distress has also been associated with growing traffic volumes and increased wheel loads.

In response to the need to improve asphalt pavement performance, the five year Strategic Highway Research Program (SHRP) was initiated in 1987. Products of SHRP included performance-based specifications for asphalt binders and performance-based mix design tests and criteria for asphalt mixtures. The specifications, tests, and criteria were combined into the Superior Performing Asphalt Pavement (Superpave) system. The Superpave asphalt binder specification addresses performance of in-service pavements for site specific environmental and traffic conditions. Performance factors include permanent deformation, fatigue cracking, and low temperature cracking.

The Superpave asphalt binder specification adopted a series of test methods for measuring the rheological characteristics of asphalt binders that were considered to be related to pavement construction and performance. In addition, the Superpave asphalt binder specification tests are performed at temperatures similar to those experienced by the in-service pavements. The asphalt binder tests address three different binder aging condition; specifically unaged (original), short-and long-termed aging.

Tests are performed on the original asphalt binder to characterize it during storage, transport, and handling. Tests performed on binder aged in the Rolling Thin Film Oven (RTFO) test are performed to measure potential changes occurring during mix production, lay-down and compaction. The RTFO test is used to simulate the aging occurring through construction. Finally, tests are performed on binder after initial aging in the RTFO and subsequent conditioning (aging) in the Pressure Aging Vessel (PAV) to estimate expected in-service changes to the asphalt binder after long-term (five to ten years in-service) aging.

During the 1985 paving season in Indiana, the neat (original/tank) asphalt binder, fuel oil, and HMA (truck mix samples) from a significant number of pavements were sampled and retained. These truck mix samples (loose-mixtures) were collected and put in sealed containers (cans) by the Indiana Department of Transportation (INDOT). The sealed containers were grouped and stored in boxes, which were retained and stored at a temperature averaging between 25 and 30°C at Purdue University.

In 1988, cores were taken from a limited number of the projects. Tests were conducted on the neat asphalt binders and extracted/recovered asphalt from the original loose mix HMA and subsequently sampled in-service cores. A second evaluation was initiated in 1992. It was planned that the group of pavements cored in 1988 would be re-cored, but unfortunately none of the pavements were available because they had all been overlaid or reconstructed.

As the project evolved, equipment to conduct the Superpave asphalt binder tests was generally standardized and became available. As a result, the study was directed toward characterizing the neat and extracted/recovered asphalt binders from the loose mix field samples, and in-service cores, using the Superpave test methods. This approach provided an early opportunity to examine a number of features of the Superpave system including short and long term aging and criteria related to pavement performance.

The Superpave asphalt binder specification adopted the Rolling Thin Film Oven (RTFO) test and newly developed Pressure Aging Vessel (PAV) test with limited verification. In the case of the PAV, the preliminary results were generalized/extrapolated to represent five to ten years of aging of in-service asphalts. Since the asphalt binder specification criteria are set to limit various distresses based on the expected aging, it is important that the expected aging from these adopted laboratory tests be verified. The current research has provided an opportunity for verification without waiting an additional five or ten years for new pavements to age.

The major objectives of the research were to:

- Determine PG classifications for the neat binders retained from the 1985 paving season in Indiana;
- Compare PG grades of the original binders from 1985 to recommended Superpave PG grades;
- Evaluate the Superpave short-(RTFO test) and long-term (PAV test) binder aging procedures;
- Study of the effect of storage time on the stiffness of the loose mixes from truck mix samples retained from the 1985 construction season;
- Study the effect of solvent type (TCE and TOL) on stiffness of the asphalt extracted and recovered from field samples;
- Calculate PG grades, for seven counties in Indiana, based on nine years of weather data and compare them to Superpave recommended PG grades for the same counties;
- Correlate pavement performance (rutting and cracking) at the mean annual pavement temperature in Indiana and the (stiffness) G^* of in-service asphalts; and
- Assess the effect of transverse and vertical pavement location of overall binder reliability.

Results of this study offer the following conclusions:

1. The PG grades recommended at the 98% reliability level are adequate for use in Indiana based on the environmental conditions. However, consideration should be given to both speed and loading when selecting the high temperature grades. The PG grade should be selected a grade higher for slower traffic or heavier loads. In “stop-go” traffic situations and/or a combination of slower traffic and heavier loading conditions, the PG grade should be selected two grades higher to safeguard against rutting potential in HMA pavements.
2. Selecting the PG grades at the 98% reliability level for the low temperature grades should be adequate to safeguard against crack potential in HMA pavements. For this study, none of the projects met the SHRP/Superpave 98% reliability level, eight projects met 50% reliability levels, and three projects did not meet either the 50% or 98% reliability levels. Therefore, the observed cracking would be expected according to the Superpave PG system.
3. The RTFO aging does not produce the degree of aging occurring during production and construction of HMA in Indiana. There was a significant difference in the aged asphalts from the RTFO test and the extracted and recovered asphalts from the loose truck mix samples retained from 1985 construction season. However, the asphalt samples and truck mixture samples had been in storage for seven to eight years.

4. A small study was conducted to address whether the storage by itself contributed the difference in stiffnesses of RTFO and in-service (mix) aging. Neat asphalts and cores after production and construction were collected from the 1996 construction season. The study confirmed that the RTFO aging does not produce the degree of aging occurring during production and construction of HMA in Indiana. However, there was a difference in stiffness between materials sampled in 1985 and those sampled in 1996. The stiffness differences between RTFO aged asphalts and the asphalts extracted and recovered from samples taken in 1996 was less than the stiffness differences between the RTFO aged asphalts and the asphalts extracted and recovered from the truck mix samples from 1985. Therefore, storage time should be limited to the shortest time possible.
5. Short storage times (i.e. two weeks) can have an effect on asphalt binder in loose mixtures. This is important because material preparation and testing is time consuming and storage is unavoidable. Other conditions should be investigated, i.e. cold storage, storage in containers with inert gas.
6. The stiffnesses of the asphalts extracted and recovered from the in-service cores (i.e. top and bottom of OWP, IWP, and BWP cores) was significantly different than stiffnesses of the asphalts after the aging. In other words, the current combined RTFO and PAV aging does not correspond to the stiffness of asphalt

extracted and recovered from in-service cores. The PAV test fell short of estimating aging of the extracted and recovered asphalts from in-service cores after seven and eight years. Future research should be conducted to refine the PAV test conditions such that the aging will more correctly simulate actual field aging.

7. Transverse pavement location (i.e. OWP, IWP, and BWP) had a significant impact on asphalt stiffness. The stiffness of the asphalts extracted and recovered from in-service cores varied significantly with transverse pavement locations. This suggests the reliability levels in Superpave should incorporate factors other than pavement temperature.
8. It was of interest to detect stiffness differences between the top and the bottom of cores. However, statistical analyses showed no significant statistical difference between stiffnesses of asphalt extracted and recovered from the top and bottom of in-service cores. Actual differences may be masked by test variability however.
9. The BWP (Between Wheel Path) location showed significantly lower stiffnesses than other transverse pavement locations. This may have been due to contaminants from travelling vehicles (e.g. dripping oil).
10. The current AASHTO standards assume that aging occurring in asphalt binders is not affected by in-service asphalt binder location (transverse and/or vertical).

Thus, the PAV variance does not include in-service aging variabilities due to pavement locations (i.e. top and bottom of OWP, IWP, and BWP). However, it was demonstrated that in-service asphalt binder location is significant. Thus, it is essential to account for this variability in the total variance, and therefore overall reliability.

11. Because in-service binder stiffness is sensitive to its pavement location an analysis was conducted to estimate the error associated with using only temperature to estimate binder reliability relative to the Superpave specification on PAV aged material. Data from Indiana projects suggest that the reliability level of PAV aged asphalt, when the effects of pavement location are incorporate into the reliability estimate, range from 84 and 88%. This represents a reduction in reliability of ten to fourteen percent.
12. Significant difference was observed between the stiffnesses of asphalts extracted and recovered, from truck mix samples using TCE and TOL. The use of TOL resulted in lower asphalt stiffnesses. However, there was no significant difference between stiffnesses of asphalts extracted and recovered, using TCE and TOL, from in-service cores (i.e. Top and Bottom of OWP, IWP, and BWP).
13. The variability of stiffnesses of asphalts extracted and recovered using TOL was lower than those extracted and recovered using TCE.

14. Poor correlation between asphalt stiffness and rutting deduct value was observed.

However, rutting deduct value decreased with increase in asphalt stiffness.

Selecting an asphalt binder that possesses an acceptable stiffness would enhance matrix capabilities to resist rutting potential in HMA pavements. Higher stiffnesses are preferable to guard against rutting potential in HMA mixes.

15. A good correlation between in-service asphalt stiffness and cracking deduct value was observed. Lower asphalt stiffness was associated with lower cracking. Higher stiffness will make the asphalt binder brittle and prone to both load associated cracking and low temperature cracking. It appears as though the Superpave PAV stiffness specification limit should be set to a maximum of 3.0 MPa for Indiana.

16. The results of this study were only directed to evaluate binders retained from the 1985-construction season in Indiana. There are several mixture properties that must also be considered when evaluating performance of HMA pavements.

The results of this study lead to the following recommendations:

1. Asphalt binders retained from the 1985 construction season in Indiana were evaluated in this study. However, at that time Indiana employed a viscosity grade

specification. Repeating the evaluation on PG asphalt binders is certainly desirable to verify/confirm the findings of this study.

2. The Superpave system includes both binder and mixture specifications. Further study to evaluate both binder and mixture performance is desirable to evaluate the Superpave system.
3. Accelerated Pavement Testing (APT) should be utilized to evaluate performance of high temperature PG grades relative to rutting potential in HMA pavements. An attempt to utilize the APT to evaluate cracking at mean annual pavement temperatures should also be made.
4. Short storage times (i.e. two weeks) can have an effect on asphalt binder in loose mixtures. This is important because material preparation and testing is time consuming and storage is unavoidable. Different storage conditions should be investigated, i.e. cold storage, storage in containers with inert gas.
5. RTFO aging does not produce the degree of aging occurring during production and construction of HMA in Indiana. There is a need to investigate the degree of aging in asphalt binder after production and construction.

6. The PAV aging does not produce the degree of aging occurring after seven or eight years of in-service aging in Indiana. There is a need to match the degree of in-service aging and to account for in-service transverse pavement variability.

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