DEVELOPMENT OF A FATIGUE-BASED ASPHALT BINDER PURCHASE SPECIFICATION FOR AIRFIELD ASPHALT

By

ROSTYSLAV SHAMBOROVSKYY

A thesis submitted to the

Graduate School-New Brunswick

Rutgers, The State University of New Jersey

In partial fulfillment of the requirements

For the degree of

Master of Science

Graduate Program in Civil and Environmental Engineering

Written under the direction of

Dr. Ali Maher

And approved by

New Brunswick, New Jersey

May, 2016

ABSTRACT OF THE THESIS

Development of a Fatigue-Based Asphalt Binder Purchase Specification for Airfield Asphalt Pavements

by ROSTYSLAV SHAMBOROVSKYY

Thesis Director:
Dr. Ali Maher

The Port Authority of New York/New Jersey (PANYNJ) has recently been experiencing a wide range of service lives with their asphalt runways. In particular, PANYNJ are noting pavement lives ranging from 3 to 15+ years, with major distress observed being top down cracking. To help explain the varied degree of fatigue cracking performance, a research study was undertaken to evaluate six different asphalt runways of different fatigue cracking lives.

Airfield cores from PANYNJ were supplied to the Rutgers Asphalt Pavement Laboratory (RAPL) for forensic study. Six runways were cored to evaluate the asphalt binder cracking properties of the different mixtures placed. Each runway resulted in varying degrees of fatigue cracking performance. Therefore, an extensive study was conducted to determine what asphalt binder correlated best to the field performance.

Laboratory binder testing showed that both the Bending Beam Rheometer (BBR) ΔT_c and Double Edge Notched Tension (DENT) Critical Opening Displacement (CTOD) properties correlated to field observations. The Glover-Rowe parameters provided reasonable comparisons to the observed field performance. However, the results from the

Linear Amplitude Sweep Test (LAS) conflicted with field observations. Further binder testing showed that "fatigue" properties of extracted and recovered asphalt binders improved with depth and appears to have little aging at depth greater than 1 inch. The testing program indicated that binder tests were capable of capturing the differences in the observed field performance. Furthermore, the research study refined laboratory PAV conditioning to match field aging, and also proposed test parameters thresholds to be used as asphalt binder purchase "PG-Plus" specification for airfield asphalt pavements.

Acknowledgements

The author would like to take the time to thank Dr. Thomas Bennert, Dr. Ali Maher, and the faculty/staff of the Center for Advanced Infrastructure and Transportation (CAIT) department for supporting their graduate studies. In addition, thank you to Casimir Bognacki, Christopher Ericson, and Darius Pezeshki for their continuous support throughout this research.

Table of Contents

List of Figures

List of Tables

Introduction

Hot Mix Asphalt (HMA) material highly contributes to the investment in the infrastructure of airfield runways. It is estimated that eighty five percent of airfield pavements in America are surfaced with HMA (1). It is in the best interest for the airport managers, engineers, and federal aviation administrators (FAA) to properly design and maintain these airfield pavements in order to provide safe and effective mode of transportation to the public. A proper design of the airfield pavement surface course is crucial in order to prevent crack induced damages. These may include, water intrusion, accelerated freeze-thaw damage, and damage from de-bonded pavement materials as a result of a crack that could potentially get sucked into jet engines.

A typical airfield HMA pavement is subjected to two different modes of damage: loading from the tire pressure and gear configuration, resulting in rutting or alligator cracking of top HMA layer, and non-load-associated loading, such as oxidation and hardening due to aging. A difference in climatic effect and aging conditions has been shown to cause non-load-associated distresses, such as top-down cracking, longitudinal and transverse cracking, and raveling which is the deterioration at the surface layer (2). These distresses are related to the pavement's loss of flexibility and ductility, as a result of aging. According to the Airfield Asphalt Pavement Technology Program (AAPTP) Project 05-07, approximately twenty percent of airfield pavements in America experienced some level of non-load associated distresses in their HMA layers (1).

In order to better understand the performance-related engineering properties and to further improve empirical grading testing of asphalt binders, the Federal Highway

Administration started a nationwide research program in 1987 known as Strategic Highway Research Program (SHRP) (3, 9). As a result of this 5 year study, a development of "Superior Performing Asphalt Pavements, (Superpave)" specification was created for asphalt binders and mixture designs procedures. The new binder grading system was termed performance grade system (PG-grade), and was based on the climate conditions of the HMA pavement location, as well as aged related distresses of the asphalt pavement. This system helped to characterize the distresses of an asphalt binder pavement at high, intermediate and low temperatures for a given highway (4).

Most highway agencies adopted the performance-graded (PG) system for binder selection in the construction of HMA pavements, forcing the Federal Aviation Administration (FAA) to do the same. The current specification for selecting PG grade of a binder for airfield pavements is vague, and in many cases, counter the observed field performance, as has been shown by Airfield Asphalt Pavement Technology Project 04-02 (4). A proposed modification using LTPPBind Software has been made in order to better relate rutting distresses at high temperatures; this is done by relating the total magnitude of loading, tire pressure, and aircraft cross weight (4). However, little to no effort has been done on intermediate temperature performance grade selection of a binder to resist fatigue cracking. Therefore, The Center for Advance Infrastructure and Transportation (CAIT) conducted an extensive forensic analysis in order to provide airport managers, engineers, and federal aviation administrators (FAA) with "PG-Plus" fatigue binder parameter for purchase specification for Quality Control to promote durable asphalt binders.

Chapter 1: Asphalt Binder Background

Origin and Production

The American Society for Testing and Materials (ASTM) defines asphalt as a dark brown to black cement-like material, mostly composed of bitumen or binder, which occur naturally or through a petroleum refining process. Naturally occurring asphalt can be found in the form of a lake in the islands of Trinidad and Bermudez located in Venezuela and the La Brea pits located in California. This is the result of evaporation of natural asphalt deposits volatiles, leaving behind asphalt resin (6). Additional natural asphalt can be found in the penetrating aggregate at the Athabasca tar sands in Canada and rock asphalt in Kentucky (6).

Figure 1: Naturally Occurrence Asphalt, Rancho LaBrea (6)

During the 20th century natural asphalt was used as a primary source in construction. However, it became less favorable as the petroleum asphalt production became more common. Today, natural asphalts are only used as an add-in to petroleum-derived asphalt (8).

Currently in the United States, most asphalt binder is produced as the distillation process of crude oil, mainly through the use of atmospheric and vacuum distillation. This process starts out by pumping the crude petroleum through a heat exchanger where rapid heat is applied for initial distillation. The crude then enters an atmospheric distillation where lighter and more volatile components are removed by a series of condensers. It is then separated into other useful petroleum products like gasoline, kerosene and diesel oil as shown in figure 2. The leftover, heavy residue also known as 'toped crude' is used for fuel oil or processed into asphalt. Due to the variation in the procedure and crude source, different grades of asphalt binder can be obtained. In order to counteract the variation, the addition of polymers, waxes and fibers are introduced to achieve a preferable grade. These additives may also alter the binder performance to produce more durable mixes (6). The classification of the binder from the above modifiers can be determined using an array of testing.

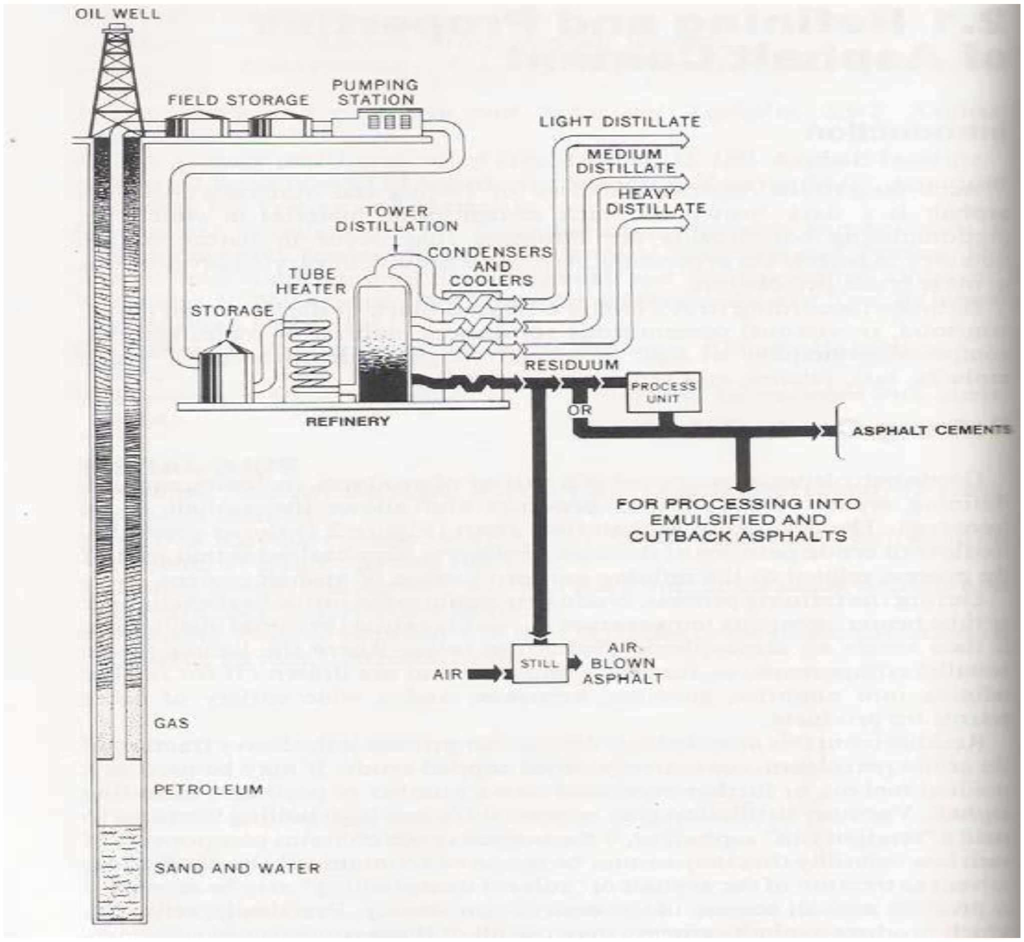

Figure 2: Petroleum Asphalt Production (6)

Asphalt Binder Development & Testing

According to the Asphalt Institute, approximately 22 million metric tons of asphalt is used annually for different applications in the United States (6). It is a very unique material that has many industrial applications, including: Highways, Airfields, Port Facilities, Parking Lots, Recreational (Bikeways, Tennis Courts, Tracks), and Hydraulic Structures (6, 7). It is estimated that 85% of asphalt is being used for paving application including highways and airfield, 10% being used for roofing shingles, and 5% for other secondary uses (6).

Prior to the new asphalt binder specification ‘Superpave’ which was developed as a result of a Strategic Highway Research Program (SHRP), the asphalt and highway industries mainly depended on empirical testing and personal experience to characterize asphalt binders. The common empirical tests included penetration and viscosity grading (4).

Penetration-Grading

The penetration test was originally developed by H.C Bowen in 1947 and further improved by A.W. Dow. Figure 3 shows the standard penetration equipment. This test records the penetration depth of the needle into a sample of asphalt at a 25 °C (Figure 4). The penetration depth of the needle is recorded in units of 0.1mm or dmm. As specified in AASHTO M 20, the binder grades are classified based on the penetration depth of a needle measured at 25 °C. Thus, an 85-100 asphalt binder grade would have a penetration depth ranging from 85 to 100 Deci millimeters (6).

Figure 3: Penetrometer (6)

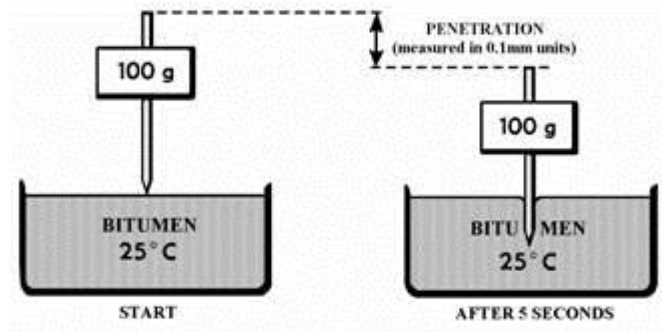

Figure 4: Penetration Test (6)

Typical standard grades would translate to 40-50 (for stiff binders), 60-40, 85-100, 120-150, and 200-300 (for soft binders) at a standard 25 °C temperature (6). Based on these grades, asphalt binder was selected for different applications. While the penetration test provided a means to evaluate stiffness, it had some limitations. Since it is an empirical test, results do not directly relate to performance related engineering properties. In addition, the test only measures the stiffness at intermediate temperature 25 °C. The stiffness of binder at different temperatures, as in summer and winter seasons can only be inferred from intermediate temperature. As illustrated in figure 5, results have shown to vary significantly for cold and hot temperature for binders with the same penetration grade (6).

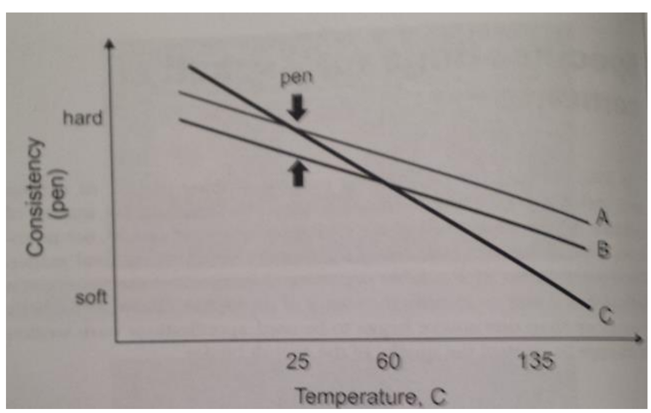

Figure 5: Penetration grading-asphalt binder properties (6)

Viscosity-Grading

Asphalt binders are visco-elastic materials, meaning that they act as viscous liquid at high temperatures, and elastic solid at cool temperatures (8). Asphalt binders that display high viscosity at a specific temperature would also have high resistance to deformation. Viscosity can also be defined as the ratio of shear stress to the rate of shear in the fluid or binder show in the equation 1 (6, 8).

$$\eta=\tau/\gamma \tag{1}$$

where,

η=viscosity
τ=shear stress
γ=shear rate

In 1970, viscosity testing became more common compared to the standard penetration grading of asphalt binders. According to AASHTO M 226 a specification criteria was established to grade physical property of asphalt binder through an absolute viscosity procedure, at 60 °C. In addition, viscosity at 135 °C was also specified. The 60 °C was chosen as an approximation reference temperature of asphalt pavement in the United States. Moreover, the 135 °C temperature was chosen to represent the construction temperature of Hot Mix Asphalt (HMA) (6). Typical viscosity testing equipment is illustrated in figure 6.

Figure 6: Absolute Viscosity Bath with Tubes (6)

The viscosity grading system utilized the grading based on un-aged asphalt binder (AC) and RTFO aged asphalt binder (AR). The Rolling Thin Film Oven (RTFO) represents the short-term aging of the material, simulating the aging effect, which would occur during the paving process on a typical roadway or airfield. The numerical values in the AC system denote the viscosity of a liquid binder in the units of poises at 60 °C (6). For instance, AC-20 would expect to have a viscosity range of 1600-2400 poises with an allowable tolerance on viscosity of ± 20 percent (6). The typical grading for an AC system is as follows: AC-2.5 (soft binders), AC-5, AC-10, AC-20, AC-30 and AC-40(stiff binders). The AR system follows the same principals; however, the absolute viscosity is tested using RTFO residue, which is slightly more aged than the original binder. The allowable tolerance on viscosity in AR system is ±25 percent. For instance, AR-8000 would include a range on viscosity, after the RTFO ageing, of 6000-10,000 poises at 60 °C (6).

The viscosity grading system shows an improvement over the traditional penetration grading. Viscosity parameter tested at 135 °C allows the producer to establish the stiffness of the asphalt binder at typical production temperature of HMA mixes (6). Moreover, the viscosity parameter at 60 °C allows the producer to establish physical properties of a binder at typical high pavement temperatures (6). Despite its benefits, the parameters for cold temperatures or severely aged binders could not be established. Additionally, polymer modified binders do not represent a proper elastic component when using the viscosity tests, thus misinterpreting the actual physical properties of the asphalt binder performance potential (6).

Analysis and Research Need

The Federal Aviation Administration (FAA) specification for airfield HMA pavements was developed based on the current Superpave Asphalt Binder criteria, with adjustments due to tire pressure, loading, and speed. These adjustments were considered to prevent rutting distresses. However, no guidelines or test parameters were developed to address fatigue cracking distresses. It is very important to address this issue, because premature fatigue cracking is being observed as early as 3 years on airfield HMA pavements. In addition, fatigue cracking can be related to the overall maintenance cost, safety, and ride quality. To resolve this issue, different laboratory “fatigue” parameters can be related to the field performance; a correlation can be developed to adopt a fatigue parameter to produce durable asphalt mixtures.

Chapter 2: PG Grading System for Highway Pavements

For many years, the asphalt and highways industry used empirical testing and professional judgment in order to characterize asphalt binder properties (4). The empirical tests were carried out at temperatures that did not necessarily represent the typical pavement temperature. It is important to carry out the tests at the proper pavement temperature because asphalt binders are highly temperature dependent (6). The Federal Highway Administration recognized the shortcoming of standard tests and therefore started a 5 year $150 million research project known as Strategic Highway Research Program (4, 9). As a result Superpave binder specification procedure was developed. The three main distresses addressed were: rutting at high temperatures, fatigue cracking at intermediate temperatures, and thermal cracking at low temperatures (11). The equipment used to characterize these distresses was: rolling thin film over (RTFO), pressure aging vessel (PAV), dynamic shear rheometer (DSR), bending beam rheometer (BBR), and direct tension tester (DTT) (4).

PG-Grade Selection

The binder grade selection is dependent on the temperature extreme in which the pavement is located. The high temperature grade is selected based on the 7 day average high temperature (4). The low grade is selected based on a single low temperature occurrence (4). For example, we expect PG70-22 binder to perform in a climate with a high average 7 day temperature being 70 °C and a low temperature being -22 °C. Depending on the speed and traffic volume these grades can be altered by using ‘grade bumping’. For a slow moving traffic condition, the rutting potential is higher, and therefore, an engineer may choose to bump or increase the high temperature grade by one (i.e. 70 to 76). In the instance of standing traffic, an engineer may choose to bump up the high temperature grade by two levels over the standard climate base grade (i.e. 70 to 82)(4). During these modifications the low temperature grade may also be altered. This grade adjustment is solely based on experience and professional judgment as specified in AASHTO M 320 (4, 5). The recommended high temperature grade adjustments using different approaches have been specified in table 1, based on traffic speed and volume.

Table 1: Recommended High Temperature Grade Adjustments (4)

Traffic Speed	Traffic Volume *MESALs*	*Grade Adjustment, °C*			
		AASHTO M 320-05	**LTPPBind 2.1/ Koch Materials**	**NCHRP 9-33 Resistivity/ Rutting**	**LTPPBind v. 3.1/ Damage Based**
Standard > 70 km/h	< 0.3	0	2	0	0.0
	0.3 to < 3	0	6	4	0.0
	3 to <10	0	8	6	6.5
	10 to <30	0*	10	9	11.3
	≥ 30	1	12	11	13.4
Slow 20 to 70 km/h	< 0.3	0	6	7	0.0**
	0.3 to < 3	6	10	12	2.6**
	3 to <10	6	12	13	8.8**
	10 to <30	6	14	16	13.5**
	≥ 30	6	16	19	15.5**
Standing < 20 km/h	< 0.3	0*	11	15	---
	0.3 to < 3	12	15	20	---
	3 to <10	12	17	22	---
	10 to <30	12	19	25	---
	≥ 30	12	21	27	---

*Consideration should be given to increasing the grade by 6°C.
**Slow traffic in LTPPBind 3.1 appears to be defined as 35 to 70 km/h.

As mentioned previously, asphalt binder classification and verification is determined using a vast array of tests. These include the aging ovens, dynamic shear rheometer, bending beam rheometer, and direct tension tester (4, 6). The binder is tested in a manner which best represents the distresses throughout its life cycle.

Asphalt Binder Aging

Asphalt binders, like any organic substances, are affected by oxidation, change in temperature, and ultraviolet radiation (6). As a result an affect known as 'aging' occurs. This alters the chemical composition of the asphalt binder and changes the mechanical and rheological properties of the binder (6, 10). The short term aging affect is the result of a loss of volatile compounds and oxidation of the binder during the mixing and laydown process of asphalt. The progressive oxidation in the pavement throughout its life is known as long term aging affect. Both aging factors cause an increase in stiffness of the binder and thus stiffening of the asphalt mixture itself making it more susceptible to fatigue intermediate temperature distresses (10).

The Strategic Highway Research Program (SHRP) developed laboratory tests for simulating the short and long term aging effects of asphalt binders to field aging conditions. The current laboratory standards for short term aging is using Rolling Thin Film Oven (RTFO) AASHTO designation T-240, and AASHTO R-28 for accelerated long term aging of binder using Pressure Aging Vessel (PAV) (6).

Rolling Thin Film Oven (RTFO)

The Rolling Thin Film Oven procedure simulates the asphalt binder's aging effect during production and placement of the pavement. It also provides measurable volatiles loss during the aging process, which is used to determine any possible contamination of the binder. The process consists of pouring 35g of binder into eight glass bottles, which are then placed in the design oven at 163 °C (6). The bottles rotate in the circular carriage and are coated with thin film of air at 400ml/min for 85min. The thin film provides the artificial short term aging effect (6).

Figure 7: Rolling Thin Film Oven (RTFO) Equipment (33)

Pressure Aging Vessel (PAV)

The pressure aging vessel (PAV) simulates aging of the binder during in-service life. The test uses the RTFO material to further age the binder in the aging vessel, by applying high pressure to increase the diffusion rate of oxygen into binder. The procedure consists of pouring 50g of binder material into an aluminum pan. It is then placed in the metal rack and loaded into the vessel at 100 °C. When the PAV reaches the test temperature it is then pressurized to 2.1 MPa for a total time of 20 hours, while maintaining constant pressure and temperature. The end result is material that has been aged to approximately 8-12 years (6).

Figure 8: Pressure Aging Vessel (PAV) equipment (8)

Dynamic Shear Rheometer (DSR)

The dynamic shear rheometer (DSR) is used to characterize the viscous and elastic behaviors of asphalt binders at medium and high temperatures (6). It measures the rheological properties, including phase angle (δ) and complex shear modulus (G*) at a loading frequency of 10 rad/sec, over a specific temperature (6). The temperature is chosen based on the yearly 7 day average high air temperature (4, 6). Complex shear modulus (G*) is a measure of material to resist deformation, and phase angle (δ) is indicator of elastic and viscous component. When $\delta=0$ the binder is purely elastic and when $\delta=90$ it is purely viscous as shown in figure 9. In terms of rutting behavior, the binder should be stiff and elastic, therefore the rutting parameter $G^*/\sin\delta$ should be maximized. In terms of fatigue resistance, the binder should be elastic, but not too stiff therefore the fatigue parameter $G^*\sin\delta$ should be minimized (6).

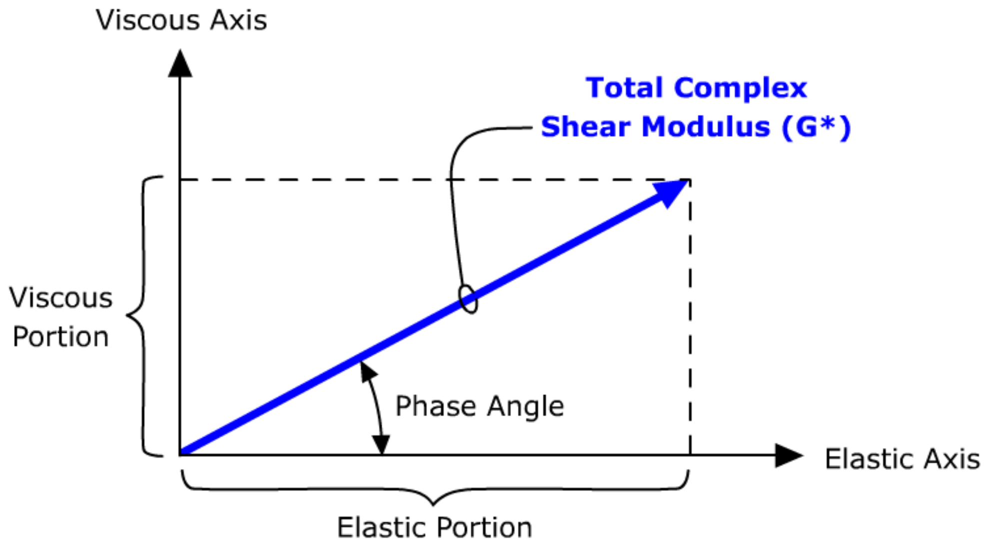

Figure 9: Shear Modulus vs. Phase Angle (34)

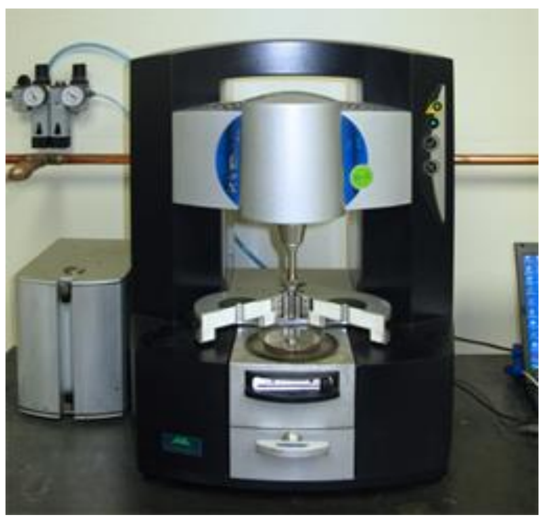

Figure 10: Dynamic Shear Rheometer (DSR) (8)

Bending Beam Rheometer (BBR)

Asphalt binders at low temperature behave similar to an elastic solid material. Therefore, they are too stiff to be evaluated on the parallel plate geometry using dynamic shear rheometer (DSR). Seeing the short comings, the Strategic Highway Research Program (SHRP) developed the bending beam rheometer (BBR) equipment in order to fully characterize the asphalt binder's rheological properties at low temperatures. The BBR uses beam theory to evaluate material stiffness under a creep load. The procedure consists of testing an asphalt beam that is simply supported, as shown in figure 11. The load is applied at the center and its deflection is measured versus time. This allows to further measure the binder stiffness (S)-value and the stress relaxation rate (m)-value by evaluating the creep response at a specific temperature. The testing temperature is chosen based on a single coldest air temperature occurrence within a year period. The threshold parameters are set at S≤300MPa for maximum stiffness at 60sec of loading and m≥0.3 for minimum stress relaxation rate at 60 sec of loading. Moreover, the test is conducted on long term aged asphalt binder material (6).

Figure 11: Bending Beam Rheometer (35)

Direct Tension Tester (DTT)

The direct tension tester is used to measure the stress and strain parameters of asphalt binder at brittle failure. It is the only asphalt binder test in Superpave specification that actually "breaks" the sample. The procedure consists of pulling a "dog bone" shaped sample apart at a constant rate of 3% per minute until failure occurs, at which point the failure strain and failure stress is recorded. The test typically requires less than one minute from load application to failure of the sample. A test result is considered valid when failure occurs at the midsection of the specimen. The test is carried out at the same low temperature as the BBR, based on the single coldest temperature occurrence of the pavements locale. The threshold minimum value of strain is set at ≥1.0%. In addition, just like the bending beam rheometer, DTT also uses PAV aged material to conduct the test (6). Lately, this test has fallen out of favor and is not being used by many state highway agencies due to its lack of repeatability.

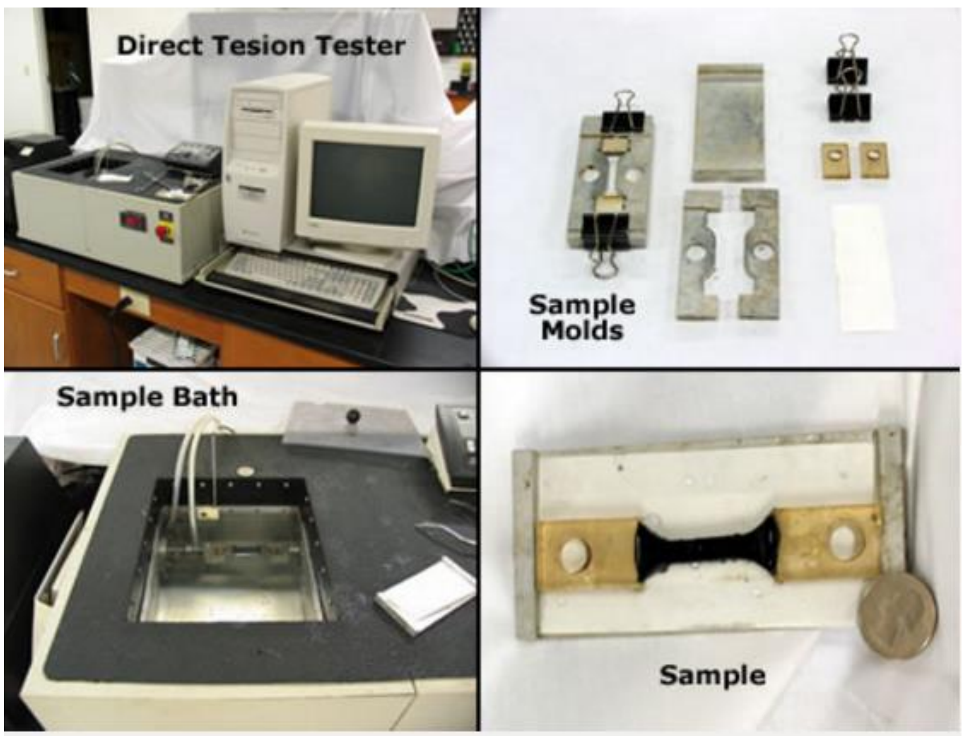

Figure 12: Direct Tension Tester (DTT) (36)

After the various binder testing, the PG grade is determined by its threshold test parameters and utilizing the tables below (4).

- Viscosity at 135 °C ≤3 Pa-s – Un-aged Binder
- G*/sinδ ≥ 1.0 kPa at 10 rad/s – Un-aged Binder
- G*/sinδ ≥ 2.2 kPa at 10 rad/s – RTFO Residue
- G*sinδ ≤ 5000 kPa at 10 rad/s – PAV Residue
- Stiffness (S-value) ≤300MPa at 60 s loading
- Relaxation (m-value) ≥ 0.30 at 60 s loading
- Failure Strain (DTT) ≥1.0% * Note-many industries solely depend on BBR results for low temperature grading *

Table 2: PG Grade Specification as Given in AASHTO M320-05 (4)

Binder Performance Grade:	**PG 46**			**PG 52**							**PG 58**				
	−34	**−40**	**−46**	**−10**	**−16**	**−22**	**−28**	**−34**	**−40**	**−46**	**−16**	**−22**	**−28**	**−34**	**−40**
Design high pavement temperature, °C:	<46			<52							<58				
Design low pavement temperature, °C:	>−34	>−40	>−46	>−10	>−16	>−22	>−28	>−34	>−40	>−46	>−16	>−22	>−28	>−34	>−40
Test on Original Binder															
Flash Point Temperature (T 48), Min., °C	230														
Viscosity (ASTM D 4402) **Maximum value of 3 Pa-s at test temperature, °C**	135														
Dynamic Shear (TP 5) **G*/sin δ, minimum value 1.00 kPa, at 10 rad/s and Test Temperature, °C**	46			52							58				
Tests on Residue from Rolling Thin Film Oven (T 240)															
Mass Loss, Maximum, %	1.00														
Dynamic Shear (TP 5) **G*/sin δ, minimum value 2.20 kPa, at 10 rad/s and Test Temperature, °C**	46			52							58				
Tests on Residue from Pressure Aging Vessel (PP 1)															
PAV Aging Temperature, °C	90			90							100				
Dynamic Shear (TP 5) **G* sin δ, maximum value 5,000 kPa, at 10 rad/s and Test Temperature, °C**	10	7	4	25	22	19	16	13	10	7	25	22	19	16	13
Physical Hardening	Report														
Creep Stiffness (TP 1) **Stiffness, maximum value 300 MPa m-value, minimum value 0.30, at 60 sec ant Test Temperature, °C**	−24	−30	−36	0	−6	−12	−18	−24	−30	−36	−6	−12	−18	−24	−30
Direct Tension (TP 5) **Failure strain, minimum value 1.0%, at 1.0 mm/min and Test Temperature, °C**	−24	−30	−36	0	−6	−12	−18	−24	−30	−36	−6	−12	−18	−24	−30

Table 2: PG Grade Specification as Given in AASHTO M320-05 (continued) (4)

Binder Performance Grade:	PG 64						PG 70					
	−10	−16	−22	−28	−34	−40	−10	−16	−22	−28	−34	−40
Design high pavement temperature, °C:	<64						<70					
Design low pavement temperature, °C:	>−10	>−16	>−22	>−28	>−34	>−40	>−10	>−16	>−22	>−28	>−34	>−40
Tests on Original Binder												
Flash Point Temperature (T 48), Min., °C	230											
Viscosity (ASTM D 4402) Maximum value of 3 Pa-s at test temperature, °C	135											
Dynamic Shear (TP 5) G*/sin δ, minimum value 1.00 kPa, at 10 rad/s and Test Temperature, °C	64						70					
Tests on Residue from Rolling Thin Film Oven (T 240)												
Mass Loss, Maximum, %	1.00											
Dynamic Shear (TP 5) G*/sin δ, minimum value 2.20 kPa, at 10 rad/s and Test Temperature, °C	64						70					
Tests on Residue from Pressure Aging Vessel (PP 1)												
PAV Aging Temperature, °C	100						100 (110)					
Dynamic Shear (TP 5) G* sin δ, maximum value 5,000 kPa, at 10 rad/s and Test Temperature, °C	31	28	25	22	19	16	34	31	28	25	22	19
Physical Hardening	Report											
Creep Stiffness (TP 1) Stiffness, maximum value 300 MPa m-value, minimum value 0.30, at 60 sec ant Test Temperature, °C	0	−6	−12	−18	−24	−30	0	−6	−12	−18	−24	−30
Direct Tension (TP 5) Failure strain, minimum value 1.0%, at 1.0 mm/min and Test Temperature, °C	0	−6	−12	−18	−24	−30	0	−6	−12	−18	−24	−30

Table 2: PG Grade Specification as Given in AASHTO M320-05 (continued) (4)

Binder Performance Grade:	PG 76					PG 82				
	−10	−16	−22	−28	−34	−10	−16	−22	−28	−34
Design high pavement temperature, °C:	<76					<82				
Design low pavement temperature, °C:	>−10	>−16	>−22	>−28	>−34	>−10	>−16	>−22	>−28	>−34
Tests on Original Binder										
Flash Point Temperature (T 48), Min., °C	230									
Viscosity (ASTM D 4402) Maximum value of 3 Pa-s at test temperature, °C	135									
Dynamic Shear (TP 5) G*/sin δ, minimum value 1.00 kPa, at 10 rad/s and Test Temperature, °C	76					82				
Tests on Residue from Thin Film Oven (T 240)										
Mass Loss, Maximum, %	1.00									
Dynamic Shear (TP 5) G*/sin δ, minimum value 2.20 kPa, at 10 rad/s and Test Temperature, °C	76					82				
Tests on Residue from Pressure Aging Vessel (PP 1)										
PAV Aging Temperature, °C	100 (110)					100 (110)				
Dynamic Shear (TP 5) G* sin δ, maximum value 5,000 kPa, at 10 rad/s and Test Temperature, °C	37	34	31	28	25	40	37	34	31	28
Physical Hardening	Report									
Creep Stiffness (TP 1) Stiffness, maximum value 300 MPa m-value, minimum value 0.30, at 60 sec ant Test Temperature, °C	0	−6	−12	−18	−24	0	−6	−12	−18	−24
Direct Tension (TP 5) Failure strain, minimum value 1.0%, at 1.0 mm/min and Test Temperature, °C	0	−6	−12	−18	−24	0	−6	−12	−18	−24

Chapter 3: Binder Grade Selection for Airfield Pavements

Consistent with the highway industry, selection of a binder for airfield pavements has progressed from penetration grading to viscosity grading systems. Prior to the strategic Highway Research Program (SHRP), which adopted the Superpave binder specification, engineers and airfield managers mainly relied on four different types of binders. Airfield pavements constructed before 2000 in the U.S. consisted of AC-30 and AC-40 binder grades for hot climates, AC-10 for cold climates, and AC-20 for the majority of the States (4).

Current Specification Standards

The current binder selection for Airfield Pavements is outlined in FAA Advisory Circular AC No: 150/5370-10G, "Standards for Specifying Construction of Airports" dated 7/21/2014. Part 5 of the manual includes the design of flexible surface courses for item P-401 and Item P-403. The item P-401 is intended for surface course, top 4 to 5 inches, and P-403 is intended for binder base and leveling course (13). The specification for the surface course is pertained to aircraft loading of gross weight greater than 12,500 pounds. For other pavements not subjected to aircraft loading, like access and perimeter roads, a state highway department specification may be used (13).

The asphalt binder performance grade should conform to ASTM D6373 specification. In addition, a binder supplier must provide a PG-grade report, called Certificate of Analysis (COA) to the engineer at each load delivery to the mix plant. Further, if any modification is made to the binder after mix plant and before use in the HMA, a report indicating new performance grade of the binder should be provided to the engineer with test results, and a statement indicating that material meets current requirements. In addition, the test results shall include viscosity charts for mixing and compaction temperatures. Moreover, to verify the quality of the materials, the engineer may request samples for testing prior to or during production (13). The modification to the binder may be a result of polymer additives, recycles asphalt pavement (RAP) contribution, contamination, or simply aging of the binder. The current specifications do allow the use of RAP up to 30% to be used only for shoulders, and leveling courses, it is not allowed in the design for surface course. The use of recycles asphalt shingles (RAS) is not permitted (13).

Current FAA specifications do include restriction on the cold temperature grade of the binder and suggest 'grade bumping' for the high temperature grade as summarized in a "note to engineer" specification section below (13, 4).

> NOTE: "Performance Graded (PG) asphalt binders should be specified wherever available. The same grade PG binder used by the state highway department in the area should be considered as the base grade for the project (e.g. the grade typically specified in that specific location for dense graded mixes on highways with design Equivalent Standard Axle Loads (ESALS) less than 10 million). The exception would be that grades with a low temperature higher than PG XX-22 should not be used (e.g. PG XX-16 or PG XX-10), unless the Engineer has had successful experience with them. Typically, rutting is not a problem on airport runways. However, at airports with a history of stacking on end of runways and taxiway areas, rutting has accrued due to the slow speed of loading on the pavement. If there has been rutting on the project or it is anticipated that stacking may accrue during the design life of the project, then the following grade 'bumping' should be applied for the top 125 mm (5 inches) of paving in the end of runway and taxiway areas: for aircraft tire pressure between 100 and 200 psi, increase the high temperature one grade; for aircraft tire pressure greater than 200 psi, increase the high temperature two grades. Each grade adjustment is 6 degrees C. Polymer-modified Asphalt, PMA, has shown to perform very well in these areas. The low temperature grade should remain the same" (4).

Additional ‘grade bumping’ based on aircraft gross weight is suggested below:

Table 3: Binder Grade Bumping Based on Aircraft Gross Weight (4)

Aircraft Gross Weight (pounds)	*High Temperature Adjustment to Base Binder Grade*	
	Pavement Type	
	Runway	*Taxiway/Apron*
Less than 12,500	--	--
Less than 60,000	--	*1*
Less than 100,000	--	*1*
Greater than 100,000	*1*	*2*

NOTES:
1. PG grades above a –22 on the low end (e.g. 64–16) are not recommended. Limited experience has shown this to be a poor performer.
2. PG grades below a 64 on the high end (e.g. 58-22) are not recommended. These binders often provide tender tendencies.
3. PG grades above a 76 on the high end (e.g. 82-22) are not recommended. These binders are very stiff and difficult to work and compact.

AAPTP Project-Proposed PG Modification for Airfield Pavements Using LTPPBind

In 1999 Pavement Systems, LLC developed PG- grade selection software for the Federal Highway Administration (FHWA). The LTPPBind software was intended to help highway engineers on selecting optimal binder grade. The results were based on climate data, and adjustments due to traffic loading and speed. The software contains close to 8000 weather climate station in the United States and Canada. The data include low, intermediate, and high temperatures values for these different locations. Users can zoom in on a particular region on the map, and receive a summary of temperature data and PG asphalt binder grade calculations. LTPPBind is an update version of the SHRPBind software that was developed during the Strategic Highway Research Program (SHRP). The program is more user-friendly and includes newer futures like; adjustment of the PG-grade for different traffic loading and speed. A free copy of the software can be obtained from the Federal Highway Administration Research and Technology website. The LTPPBind software has been proven to simplify Superpave binder selection for highway applications (4, 31). In order to use the software for airfield applications certain factors had to be normalized. These factors included difference between highway vehicle wander and aircraft wander, tire pressure between commercial trucks and aircraft, wander difference between aircraft gear configurations, wander difference between runways and taxiways, and difference between construction and composition of airfield pavements compared to highway pavements (4).

One of the most important aspects for determining an effective PG-grade for airfield pavements using LTPPBind software is to relate the equivalent single axle loads (ESAL's) or Equivalent Highway ESAL's (EHE) for a given flow of airfield traffic. The concept of ESAL was developed in 1960 by American Association of State Highway Officials (AASHO) road test. This was done to help establish a pavement damage relationship for different axle loads. The design ESAL's represents a mix of traffic having different axle configuration and loads, which is then converted into a standard 18,000 lb (80kN) single axle loads over the total design period of HMA pavement, typically 20 year period. The ESAL parameter can be thought of as a damage factor instead of a load. LTPPBind uses ESAL's as one of its parameters to characterize the damage done to the pavement due to loading. EHE can simply be defined as the overall magnitude of loading on the flexible pavement due to air traffic. In doing so, EHE parameter can be used instead of ESAL's to come up with an effective asphalt binder PG-grade for airfield pavement. The standardized ESAL's equation 2 is shown (4).

$$\text{ESAL's}=\left(\sum_{i=1}^{m} p_i F_i\right)(ADT)(T)(A)(G)(D)(L)(365)(Y) \qquad (2)$$

where,

p_i	= percentage of total repetitions for i^{th} load group
F_i	= equivalent axle load factor for i^{th} load group
	= damage caused by one load repetition of i^{th} load group relative to 18 kip axle
(ADT)	= initial average daily traffic
T	= percentage of truck in average daily traffic
A	= average number of axles per truck
G	= growth factor
D	= directional distribution factor (usually 0.5)
L	= lane distribution factor
365	= days per year
Y	= design period in years

As previously mentioned the ESAL equation does not apply to airfield pavements due to a variety of different factors. Seeing the short comings, the airfield asphalt pavement technology program (AAPTP) project 04-02 underwent an extensive analysis to relate ESAL's equation to EHE which can then be used in the LTPPBind software as a design traffic problem for a flexible pavement. It should be stressed that EHE does not compare aircraft to truck design, or airfield to highway pavement; it is simply a way to relate the overall magnitude of loading on airfield pavement in a way that will ease in PG-grade selection using standard available methods. The EHE equation and parameter factors are described in equation 3 (4).

$$\text{EHE's}=\sum_{i=1}^{m}\left[(TP_i)\left(\frac{PDR_i}{PCR_i}\right)(N_i)\right](COMP)(LC)(FC)(REL)(Y)(1+\frac{R}{100})^{0.5Y} \qquad (3)$$

where,

TP_i	= tire pressure factor for the i^{th} aircraft group
PCR_i	= pass to cover ratio for the i^{th} aircraft group
PDR_i	= pass to departure ratio for the i^{th} aircraft group
	= 1 for parallel taxiways
	= 2 for central taxiways
	= 3 for runways with central taxiways
N_i	= annual departure for the i^{th} aircraft group
COMP	= composition factor, to account for differences in rut resistance between typical airfield and highway HMA caused by differences in design VMA and mineral filler content
LC	= lab compaction factor, to account for differences in design compaction between typical airfield and highways HMA mixtures
FC	= Field compaction factor, to account for differences in field compaction between typical airfield and highway HMA pavements
REL	= reliability factor, to address any adjustments needed in reliability level of airfield pavements relative to highway pavements
Y	= design life, in years (assumed to be 20 unless otherwise noted)
R	= annual growth rate in traffic, %

Further research by Airfield Asphalt Pavement Technology Program (AAPTP), related the EHE parameter as a function of annual departures and maximum gross aircraft weight. It then suggested ‘grade bumping’ for both conventional and polymer modified asphalt binders, based on: aircraft stacking, typical speed at runways and taxiways, and the design traffic EHE. The proposed procedure has been compared to the conventional FAA procedure for selecting binder grade through an extensive study comparison. The study consisted of seven different airfield projects with different volumes and pavement designs. The proposed procedure in selecting a binder showed a very close correlation to the actual binder which has been used for the project. However, there was a big difference between the proposed procedure and the current FAA practice in selecting the binder grade (4).

The first step in the proposed procedure consists of determining the total annual departures for runway and taxiway. The maximum gross allowable weight (GAW) is determined which represents more than 10% of total annual departures. Using figure 13 the design traffic level load is determined. The LTPPBind software then uses the design traffic to determine the base PG-grade. The high temperature PG grade is then adjusted according to table 3 based on aircraft stacking, speed, design traffic load, and binder modification. In the final step the binder is compared to the common binder used in that specified state (4). Some of the common PG binders used for different states are listed in table 5 (4).

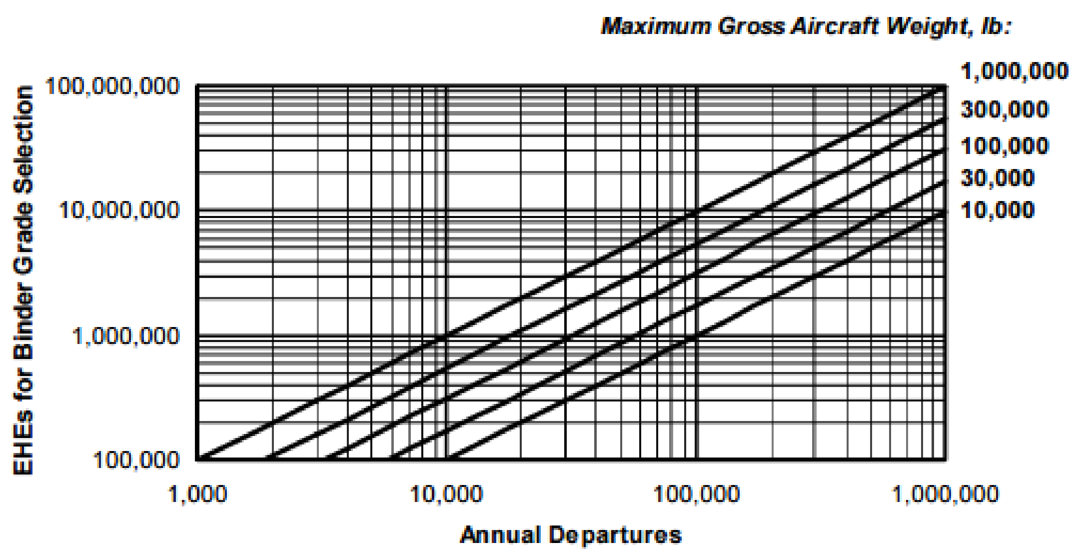

Figure 13: EHEs for Binder Grade Selection (4)

Table 4 : High Temperature PG Grade Adjustment using LTPPBind (4)

Aircraft Stacking	Typical Speed *Mph*		Design Traffic *EHEs*	Grade Adjustment ℃	
	Runway Centers	Taxiways/ Runway Ends		Non-Modified Binders	Polymer Modified Binders*
None	≥ 45	15 to < 45	< 300,000	0	
Little or none	≥ 45	15 to < 45	300,000 to < 3 million	+7	*Not Required* +4
			3 million to < 10 million	+7	*Suggested* +4
			≥ 10 million	---	*Required* +4
Occasional	---	5 to < 15	< 10 million	+14	*Suggested* +11
			≥ 10 million	---	*Required* +11
Frequent	---	< 5	Any	---	*Required* +17

Table 5 : Common PG Binder Used in Different States (4)

State	Common Binder PG Grades																						
	46	52		58			64				67	70					76				82		
	-28	-34	-28	-34	-28	-22	-34	-28	-22	-16	-22	-34	-28	-22	-16	-10	-16	-28	-22	-16	-28	-22	-16
Alabama	--	--	--	--	--	Y	--	--	Y	--	Y	--	--	--	--	--	--	--	Y	--	--	--	--
Alaska	--	--	--	--	Y	--	--	Y	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Arizona	--	--	--	--	--	Y	--	--	Y	Y	--	--	--	--	--	Y	--	--	Y	Y	--	--	--
Arkansas	--	--	--	--	--	--	--	--	Y	--	--	--	--	Y	--	--	--	--	Y	--	--	--	--
Colorado	--	--	--	Y	Y	Y	--	Y	Y	--	--	--	Y	--	--	--	--	Y	--	--	--	--	--
Connecticut	--	--	--	--	Y	--	--	Y	Y	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Delaware	--	--	--	--	Y		--	--	Y	--	--	--	--	Y	--	--	--	--	Y	--	--	--	--
Florida	--	--	--	--	--	--	--	--	Y	--	Y	--	--	--	--	--	--	--	Y	--	--	--	--
Georgia	--	--	--	--	--	Y	--	--	Y	--	Y	--	--	--	--	--	--	--	Y	--	--	--	--
Hawaii	--	--	--	--	--	--	--	--	--	Y	--	--	--	--	--	--	--	--	--	--	--	--	--
Idaho	--	--	--	Y	Y	--	Y	Y	Y	--	--	--	Y	--	--	--	--	Y	--	--	--	--	--
Illinois	Y	--	Y	--	Y	Y	--	Y	Y	--	--	--	Y	Y	--	--	--	Y	Y	--	--	--	--
Indiana	--	--	--	--	Y	--	--	Y	Y	--	--	--	Y	Y	--	--	--	--	Y	--	--	--	--
Iowa	--	Y	Y	--	Y	Y	--	Y	Y	--	--	--	--	Y	--	--	--	--	Y	--	--	--	--
Kansas	--	--	--	--	--	Y	--	Y	Y	--	--	Y	Y	Y	--	--	--	Y	Y	--	Y	Y	--
Kentucky	--	--	--	--	--	Y	--	--	Y	--	--	--	--	Y	--	--	--	--	Y	--	--	--	--
Louisiana	--	--	--	--	--	Y	--	--	Y	--	--	--	--	Y	--	--	--	--	Y	--	--	--	--
Maine	--	--	--	Y	--	--	--	Y	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Maryland	--	--	--	--	--	--	--	Y	Y	--	--	--	--	Y	--	--	--	--	Y	--	--	--	--
Massachusetts	--	--	--	Y	--	--	--	Y	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Michigan	--	Y	Y	Y	Y	Y	Y	Y	Y	--	--	--	Y	Y	--	--	--	Y	Y	--	--	--	--
Minnesota	--	--	--	Y	Y	--	Y	Y	--	--	--	Y	Y	--	--	--	--	Y	--	--	--	--	--
Mississippi	--	--	--	--	Y	--	--	--	--	--	Y	--	--	--	--	--	--	--	Y	--	--	Y	--
Missouri	--	--	--	--	--	--	--	--	Y	--	--	--	--	Y	--	--	--	--	Y	--	--	--	--
Montana	--	--	Y	--	--	--	Y	Y	Y	--	--	--	Y	--	--	--	--	--	--	--	--	--	--
Nebraska	--	--	--	--	--	--	--	Y	--	--	--	--	Y	--	--	--	--	Y	--	--	--	--	--

Table 5: Common PG Binder Used in Different States (Continue) (4)

State	Common Binder PG Grades																						
	46	52		58			64				67	70					76				82		
	-28	-34	-28	-34	-28	-22	-34	-28	-22	-16	-22	-34	-28	-22	-16	-10	-16	-28	-22	-16	-28	-22	-16
Nevada	--	--	--	--	--	--	Y	Y	Y	--	--	--	--	--	--	--	--	--	Y	--	--	--	--
New Hampshire	--	--	--	--	--	--	--	Y	Y	--	--	--	--	Y	--	--	--	Y	--	--	--	--	--
New Jersey	--	--	--	--	--	--	--	--	Y	--	--	--	--	--	--	--	--	--	Y	--	--	--	--
New Mexico	--	--	--	--	Y	--	--	Y	Y	--	--	--	Y	--	--	--	--	--	Y	--	--	Y	--
New York	--	--	--	Y	Y	--	--	Y	Y	--	--	--	--	Y	--	--	--	--	Y	--	--	--	--
North Carolina	--	--	--	--	--	--	--	--	Y	--	--	--	Y	Y	--	--	--	--	Y	--	--	--	--
North Dakota	--	--	--	Y	Y	--	--	Y	--	--	--	--	Y	--	--	--	--	--	--	--	--	--	--
Ohio	--	--	--	--	Y	--	--	Y	Y	--	--	--	--	Y	--	--	--	--	Y	--	--	--	--
Oklahoma	--	--	--	--	--	--	--	--	Y	--	--	--	Y	--	--	--	--	Y	--	--	--	--	--
Oregon	--	--	--	--	--	--	--	Y	Y	--	--	--	Y	Y	--	--	--	--	Y	--	--	--	--
Pennsylvania	--	--	--	--	Y	--	--	--	Y	--	--	--	--	--	--	--	--	--	Y	--	--	--	--
Puerto Rico	--	--	--	--	--	--	--	--	Y	--	--	--	--	Y	Y	--	--	--	--	--	--	--	--
Rhode Island	--	--	--	--	--	--	--	Y	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
South Carolina	--	--	--	--	--	--	--	--	Y	--	--	--	--	--	--	--	--	--	Y	--	--	--	--
South Dakota	--	--	--	Y	--	--	Y	Y	Y	--	--	Y	Y	--	--	--	--	--	--	--	--	--	--
Tennessee	--	--	--	--	--	--	--	--	Y	--	--	--	--	Y	--	--	--	--	Y	--	--	--	--
Texas	--	--	--	Y	Y	Y	Y	Y	Y	Y	--	Y	Y	Y	Y	--	--	Y	Y	Y	Y	Y	Y
Utah	--	--	--	Y	--	--	Y	Y	--	--	--	Y	Y	Y	--	--	--	Y	Y	--	--	--	--
Vermont	--	--	--	Y	Y	--	Y	Y	--	--	--	--	Y	--	--	--	--	--	--	--	--	--	--
Virginia	--	--	--	--	--	--	--	--	Y	--	--	--	--	Y	--	--	--	--	Y	--	--	--	--
Washington	--	--	--	Y	Y	Y	Y	Y	Y	--	--	Y	Y	Y	--	--	--	Y	Y	--	--	--	--
West Virginia	--	--	--	--	Y	--	--	Y	Y	--	--	--	--	Y	--	--	--	--	Y	--	--	--	--
Wisconsin	--	--	--	Y	Y	--	Y	Y	Y	--	--	--	Y	--	--	--	--	Y	--	--	--	--	--
Wyoming	--	--	--	Y	Y	--	Y	Y	Y	--	--	--	--	Y	--	--	--	--	Y	--	--	--	--

The low temperature grade produced by the LTPPBind software does not change, nor is there a grade adjustment made through this process. However, as explained earlier there are some restriction limits on low temperature grade selection specified in the FAA advisory circular. In addition, if thermal or fatigue cracking is known to be an issue, an engineer might suggest using high temperature PG-Grade, no stiffer than needed to withstand the anticipated loading. This will prevent excessive stiffening and reduce thermal and fatigue distresses (4). Another way to improve grade adjustments is through the use of polymer modification, as explained in the section below.

Specifying Polymer Modified Binder for Use in Airfields

The use of polymer modified asphalt (PMA) has tremendously increased over the past few years; this is primarily due to its ability to increase binder stiffness, without stiffening the low temperature grade. In doing so, we can create stiffer mixes to control permanent deformation (rutting), without reducing fatigue or thermal cracking stability. In addition, according to Glover et al (2005), polymer modified binders may improve the oxidation rate of asphalt binders, thus leading to more durable pavement mixtures (17).

A recent survey by association of modified asphalt producers (AMAP) show that approximately 23% of state highway agencies used polymer modified binders in their HMA pavement designs (18). The survey also showed that while there is a large number of polymer modifiers, the most commonly used are: Styrene-butadiene-styrene (SBS), styrene-butadiene (SB), Styrene-butadiene rubber (SBR), chemical modifiers, ground tire rubber, and oils (18).

Airfield pavements are typically subjected to heavy loading. This may result in rutting and shoving distresses in the early stages of HMA pavement. Polymer modified binders (PMA) typically stiffen the binder on the high end, thus minimizing the effect on permanent deformation or rutting. On the other hand, as the pavement ages it becomes stiffer (oxidized), losing its molecular relaxation properties making it more prone to fatigue and block cracking. Glover showed that some polymers have an effect on reducing the rate of oxidation, thereby reducing cold temperature distresses (4, 17). When selecting a polymer modified binder, an airport engineer needs to make sure that its performance and durability is beneficial to hot, intermediate, and low temperature climate distresses. Prior research has shown that while PMA binders may be beneficial to permanent deformation, they may be detrimental to fatigue performance (20). In addition, life cycle cost assessment should be performed, since PMA binders are typically 20 to 30% more expensive compared to conventional PG-Grade binders (19).

In Europe, polymer modified asphalt binder has become a common practice in HMA surface course design for airfield. For example in Denmark, the use of SBS polymer is common for taxiways surface course design. In Germany and France, it is common that HMA surface courses subjected to heavy loading be designed with PMA binder. Netherlands has also recently adopted the use of PMA in their design, based on performance and economic benefit (4). The current federal aviation administration (FAA) specifications do allow the use of PMA binders in their designs. This is mainly to provide proper high temperature grade adjustments to the base binder in order to resist rutting distresses (4). However, little to no emphasis regarding fatigue or durability resistance of polymer modified asphalt binder exists in the current FAA specifications.

Chapter 4: Innovative Asphalt Binder Fatigue/Durability Characterization Test Parameters

The industry evolved from penetration grading to viscosity grading and eventually adopting the current Superpave binder specification criteria. Superpave was intended to accurately and fully characterize the binder in terms of rutting, fatigue cracking, and thermal cracking distresses. The system has shown to perform relatively well in relating high and low temperature binder parameters to resist rutting and thermal cracking, respectively. Unfortunately, there is still a need to develop a better means to address the fatigue/durability of asphalt binders. To better understand the asphalt binder's durability in terms of flexibility and hardening, other parameters can be looked at as "PG-Plus" specification parameters. These are discussed in the sections below, and include: the difference in critical low temperature –(ΔT_c), Christensen-Anderson Model (CAM) Rheological Index –(R-value) & Crossover Frequency –(ω), Glower-Rowe Parameter –(G-R), double edge notch tension test (DENT)– CTOD, and linear amplitude sweep test– N_f.

Difference in Critical Low Temperature PG grade ΔT_c Parameter

The Critical Low Temperature ΔT_c parameter is temperature independent and is determined using the standard bending beam rheometer (BBR) procedure in accordance with AASHTO T313, *Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer (BBR).* The critical low temperature ΔT_c is defined as the temperature where exact thresholds limits are met for stiffness (S) and relaxation parameter (m-value). For Stiffness, $T_{c\ (stiffness)}$ is the exact temperature where stiffness at

60 seconds of loading equals to 300MPa. For m-value, $T_{c\ (m\text{-}slope)}$ is the exact temperature where m-value at 60 seconds of loading equals to 0.300. As a result, the test should be run at a minimum of two temperatures, from which the exact parameter limits can be interpolated. Equations for determining these parameters are listed below.

$$T_{c(stiffness)} = T_1 + [\frac{Log(300) - Log(S_1)}{Log(S1) - Log(S_2)} * (T_1 - T_2)] - 10 \quad (4)$$

$$T_{c(m-slope)} = T_1 + [\frac{0.300 - m_1}{m_1 - m_2} * (T_1 - T_2)] - 10 \quad (5)$$

where,

T_1= Temperature #1, °C
T_2= Temperature #2, °C
S_1= Stiffness at 60 seconds loading at Temperature #1, MPa
S_2= Stiffness at 60 seconds loading at Temperature #2, MPa
m_1= (m – value) at 60 seconds loading at Temperature #1
m_2= (m – value) at 60 seconds loading at Temperature #2

Anderson, et al. (2011) defined the difference between $T_{c(stiffness)}$ and $T_{c(m\text{-}slope)}$ as a good parameter for identifying non-loaded related distresses such as transverse and longitudinal cracking (22). The parameter has also been shown to correlate very well with the observed fatigue cracking on the field by Rutgers Asphalt Pavement Laboratory (RAPL) (24). The difference in critical low temperature PG-Grade can be expressed as follows:

$$\Delta T_c = T_{c(stiffness)} - T_{c(m\text{-}slope)} \quad (6)$$

In equation 6, as ΔT_c decreases and becomes more negative, indicating that binder has lost some of its relaxation properties and is not able to recover to its initial performing stage. Therefore, the binder would be more susceptible to non-load related cracking. This scenario is typically observed as the binder ages or oxidizes throughout its lifetime.

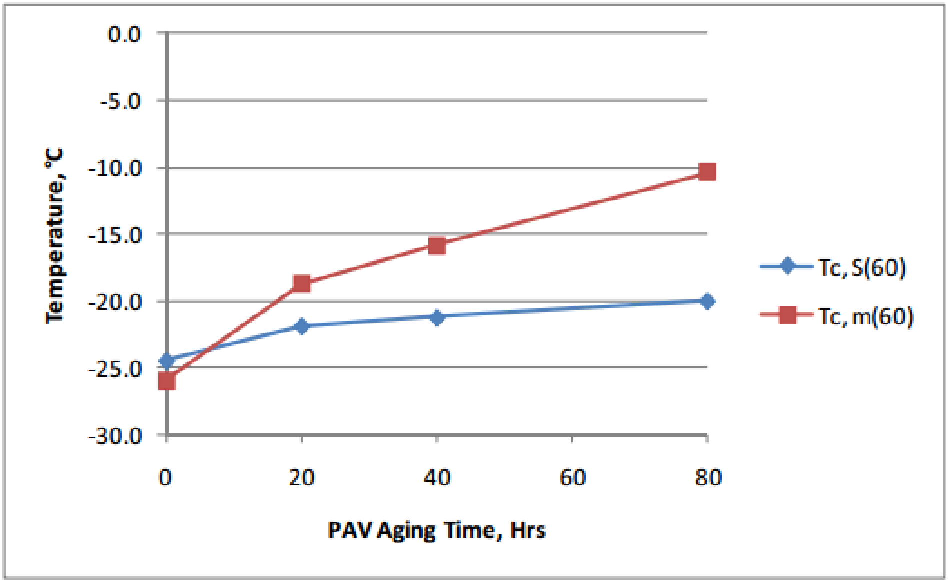

Figure 14: Effect of PAV aging on T_c (2)

From figure 14, we can conclude that as the binder ages from 0 hours to 80 hours in the PAV, the difference between stiffness and relaxation parameter is negatively affected at a greater rate. This would indicate that the binder is losing its relaxation ability as it ages. Ultimately, we can say that as ΔT_c decrease, the binder becomes more prone to fatigue cracking as it becomes more brittle. The test parameter has been proven to correlate well to field fatigue performance for both modified and conventional binders (24, 25).

The Airfield Asphalt Pavement Technology Program, Project 06-01, showed a strong correlation between the difference in critical low temperature ΔT_c and Ductility within their mixes, as shown in figure 15. As a note, the author in this specific project defined $\Delta T_c = T_{c(m\text{-}slope)} - T_{c(stiffness)}$; which would indicate that as ΔT_c increase the binder becomes more m control, therefore reducing ductility and overall reducing the durability of asphalt binders (2).

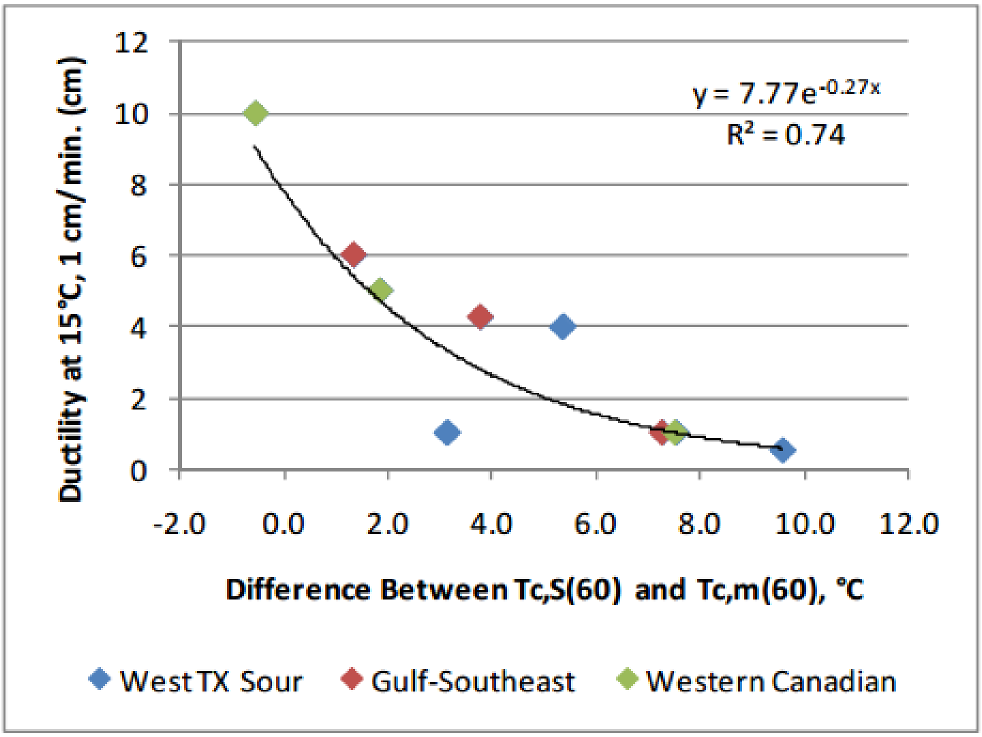

Figure 15: Relationship between ΔT_c and Ductility (2)

Initially, Anderson et al. (2011) showed that the ΔT_c correlated to non-load associated cracking as well as ductility. A limit for crack identification and visual crack initiation was set at $\Delta T_c \leq -2.5°C$ by Anderson et al. At which point a preventive maintenance was suggested to avoid further development of a crack (22). Through an extensive laboratory study, Rowe further advanced this methodology, and recommended a limit of $\Delta T_c \leq -5.0°C$ be used for immediate rehabilitation. At this point the damage to the surface pavement is considered to be significant and milling or replacement was recommended (23). Further, Rowe developed a new binder fatigue parameter, which he called 'Glover-Rove'; this parameter is discussed later in the report.

Christensen-Anderson Model (CAM)-(R-values & ω)

Christensen-Anderson Model (CAM) is very useful in determining master curve physical parameters (R, ω) of asphalt binders. The master curve sweep is conducted over a range of temperatures and frequencies using a dynamic shear rheometer (DSR). For each temperature a shear modulus (G*) is obtained through a range of frequencies as shown in figure 16. The RHEA software then utilizes each isotherm and applies appropriate shift factors to create a continuous master curve at a reference temperature of 15°C. This temperature is selected based on its relation to ductility testing (8, 27).

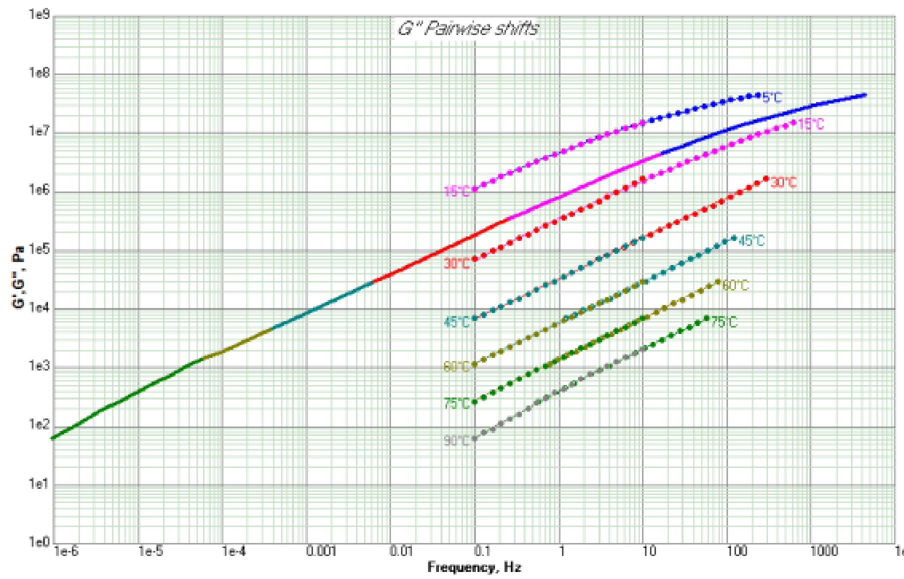

Figure 16: Isotherms and Mastercurve Construction (8)

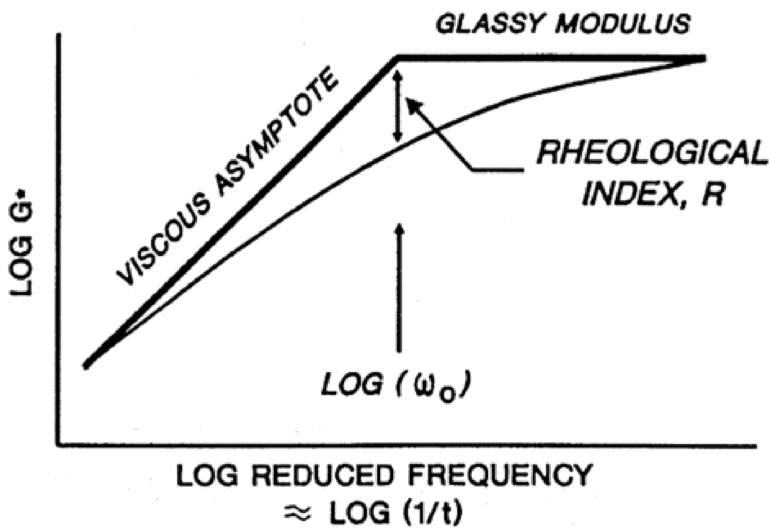

Figure 17: Rheological Parameters of Master curve (30)

For most binders, cross over frequency, ω, can be defined at the point where viscous tangent line is at 45 degrees to the master curve and crossing the glassy modulus, as shown in figure 17. As a binder become more oxidized or stiffer, the master curve becomes flatter. As the shape of the master curve gets flatter, the viscous asymptote moves to the left towards lowered reduced frequency. Therefore, as asphalt binder ages, the crossover frequency, ω, reduces. The rheological index, R-value can be defined by the difference in glassy modulus and the dynamic modulus at the crossover frequency. This index is also identified by the following equation:

$$R = \frac{(\log 2) * \log \frac{G^*(\omega)}{G_g}}{\log(1 - \frac{\delta(\omega)}{90})} \quad (7)$$

where,

G*(ω) = Measured complex modulus
Gg = Glassy modulus
δ(ω) = Measured phase angle

Based on equation 7, we can determine that as binder age, the master curve becomes flatter, resulting in higher G* and lower $\delta(\omega)$ making the R-value higher. Therefore, the Christensen-Anderson Model (CAM) suggests that as binder ages over time, cross over frequency, ω, decreases and the rheological index R-value increases (27). The CAM model has been shown to be a good aging/hardening indicator for both modified and conventional asphalt binders by Rutgers Asphalt Pavement Laboratory (RAPL) (25). Master curve shape and its properties are also very useful in determining other asphalt binder performance properties, as discussed later in this report.

Glover-Rowe – (G-R) Parameter

Many researchers have related asphalt binders durability to its measured ductility (26). Field studies by Kandhal (1977), on numerous Pennsylvania and Ohio test sections suggested that a durability limit of ≤5cm would represent the onset of crack initiation at 15°C and rate of elongation of 1cm/min. Further, Kandhal reported that when ductility was ≤3cm, major surface distresses were observed (26). Research by Glover et al. (2005) at Texas A&M further related the ductility at 15°C and 1cm/min to a DSR parameter $G'/(\eta'/G')$ measured at 15°C and frequency of 0.005 rad/s as shown in figure 18. Even though the parameter showed a good correlation with ductility, it was determined that the slow rate of loading (.0005 rad/s) would take a very long time to complete the test. As a result, researchers used time-temperature superposition principal to relate $G'/(\eta'/G')$ parameter to a standard DSR frequency of 10 rad/s. They further suggested that the same values for $G'/(\eta'/G')$ can be obtained by performing the test at higher temperature (44.7°C) and faster loading frequency (10 rad/s). The Glover parameter can be obtained from a single measurement in the DSR, making it much more practical and also requiring less asphalt binder compared to standard ductility testing.

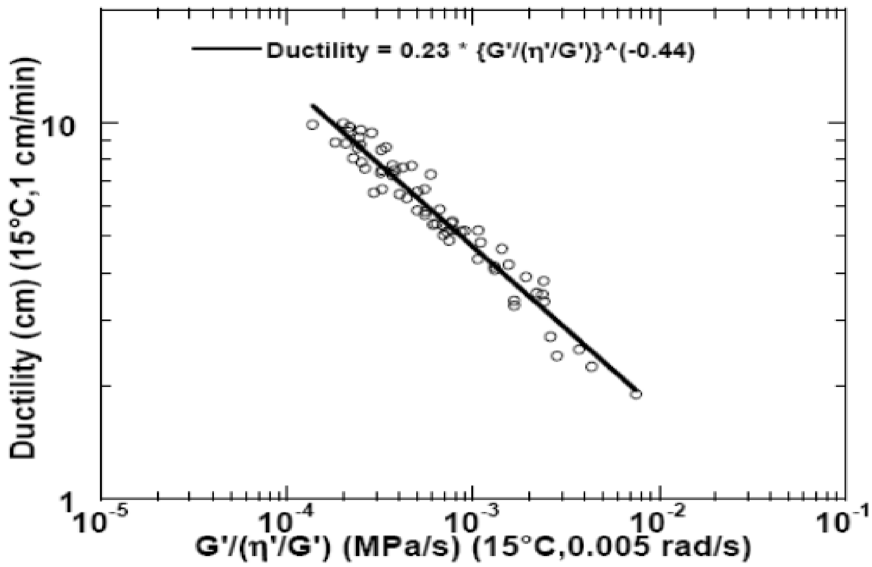

Figure 18: Ductility vs. Frequency at specific temperature and frequency (2)

Rowe further re-defined the Glover parameter G'/(η'/G') and expressed it in terms of Shear modulus G* and phase angle δ based on black space diagram shown in figure 19 and represented by the following equations below (24):

$$\eta' = \frac{G''}{\omega} \, and \; tan\delta = \frac{G''}{G'} \tag{8}$$

hence, $\frac{\eta'}{G'} = \frac{1}{\omega}\frac{G''}{G'} = \frac{tan\delta}{\delta}$ (9)

therefore, $\frac{G'}{(\frac{\eta'}{G'})} \, or \; \frac{G'}{(\frac{tan\delta}{\omega})} = \frac{G'\omega}{tan\delta}$ (10)

Putting the equation in its simples form, we have:

$$\frac{G'\omega}{\tan\delta} \, or \; \frac{G*cos\delta*\omega}{tan\delta} \, or \; \frac{G*(\cos\delta)^2 x\omega}{sin\delta} \tag{11}$$

However because the parameter is calculated at a fixed frequency (0.005rad/s), the value of ω can be ignored.

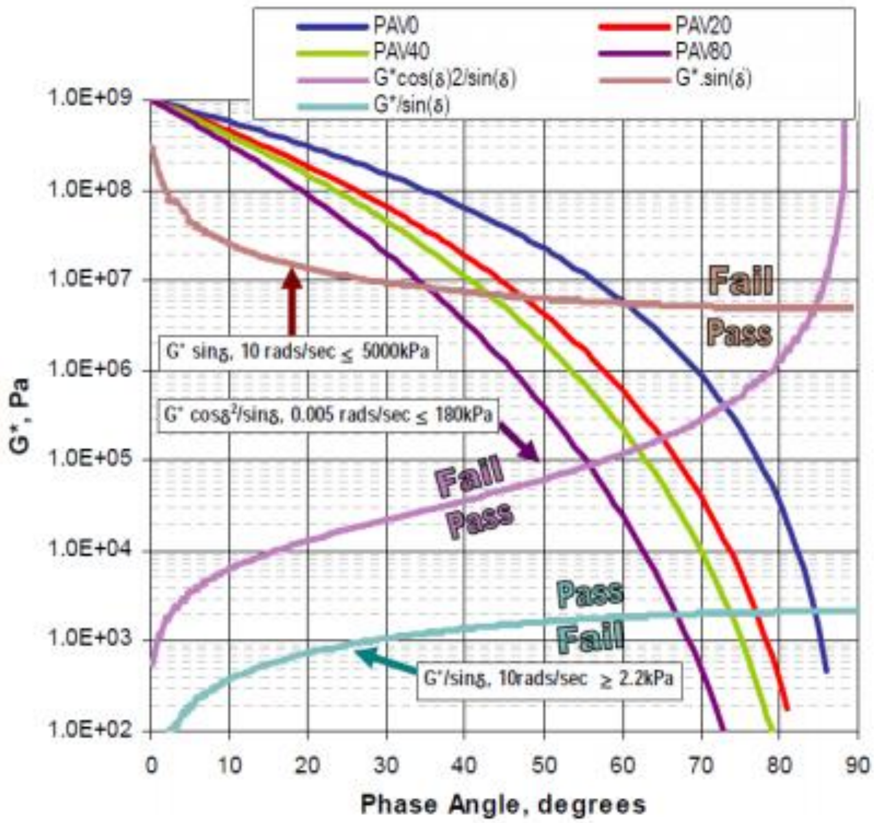

Figure 19: G-R parameter represented in black space (30)

Rowe proposed using a construction of master curve frequency sweep at 5,15, and 25 °C from 0.1 to 100rad/s and interpolating the date to find a values of G* and δ at 15 °C and 0.005 rad/sec from which G-R parameter can be calculated. A bigger value of G* and smaller value of δ would produce a larger G-R parameter. From previous discussion on ductility, we can conclude that as G-R increases, binder becomes more brittle and more prone to non-load associated cracking. The initial threshold limit for crack initiation was set at 180kPa by Glover (23), and further advanced to 450kPa for significant cracking and surface raveling by Anderson et al. (22).

It has also been shown that G'/(η'/G') and ΔT_c parameters are related to ductility of asphalt binders as suggested by Texas A&M and other researchers. Researchers then related G'/(η'/G') parameter as a function of ΔT_c and as expected, it showed a very good correlation, with R^2 value being 0.982 as shown in figure 20 (2). As a note, the author in this specific project defined $\Delta T_c = T_{c(m\text{-}slope)} - T_{c(stiffness)}$; which would indicate that as ΔT_c increase the binder becomes more m control, therefore reducing ductility and overall reducing the durability of asphalt binder.

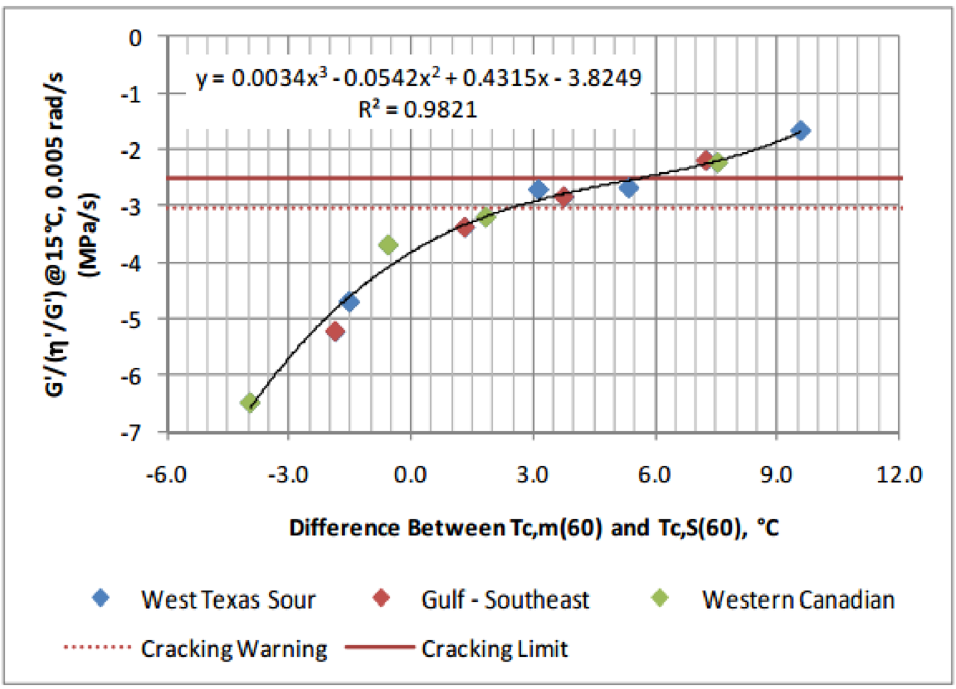

Figure 20: Relationship between G'/(η'/G') and ΔT_c (2)

Double-Edge-Notched Tension (DENT) Test-CTOD Parameter

The fracture resistance of binder properties was measured in accordance with AASHTO TP-113, *Determination of Asphalt Binder Resistance to Ductile Failure Using Double Edge Notched Tension (DENT*). The test is similar to that of the direct tension test (DTT) that is used under Superpave PG-Grade specification with the exception that notches are imposed on the sample. The test was initially proposed by Queen University and further advanced by Federal Highway Administration (FHWA) (15, 16).

After thermal conditioning of the six samples, the test is run to determine the critical crack tip opening displacement (CTOD) and the essential work of fracture (EWF) at a specific temperature (typically 25 °C) and rate of loading (typically 100mm/min). To determine these parameters, specimens are tested at 5, 10, and 15mm ligament lengths, illustrated in figure 21. Because the test is looked at the fracture resistance of asphalt binder, a typical PAV aged material is used to conduct the test (16, 21). After the six specimens are tested, the average total work of fracture (W_t) is determined for each ligament length using the area under the load – displacement curve. The specific essential work of fracture (w_e) is then determined, which is equal to the specific total work of fracture (w_t) at zero ligament length. This is, determined from the best fit line, as shown in figure 22. The critical tip opening displacement (CTOD) parameter can then be calculated using equations 13 and 14.

$$w_t = W_t/B\ell \tag{12}$$

$$\sigma_n = P_{peak}/B\ell \tag{13}$$

$$CTOD = w_e / \sigma_n \tag{14}$$

where,

w_e= the specific essential work of fracture, i.e. w_t for ℓ=0.0mm
P_{peak}= average maximum load for the 5mm ligament sample
B= sample thickness, mm
ℓ= smallest ligament length 5mm

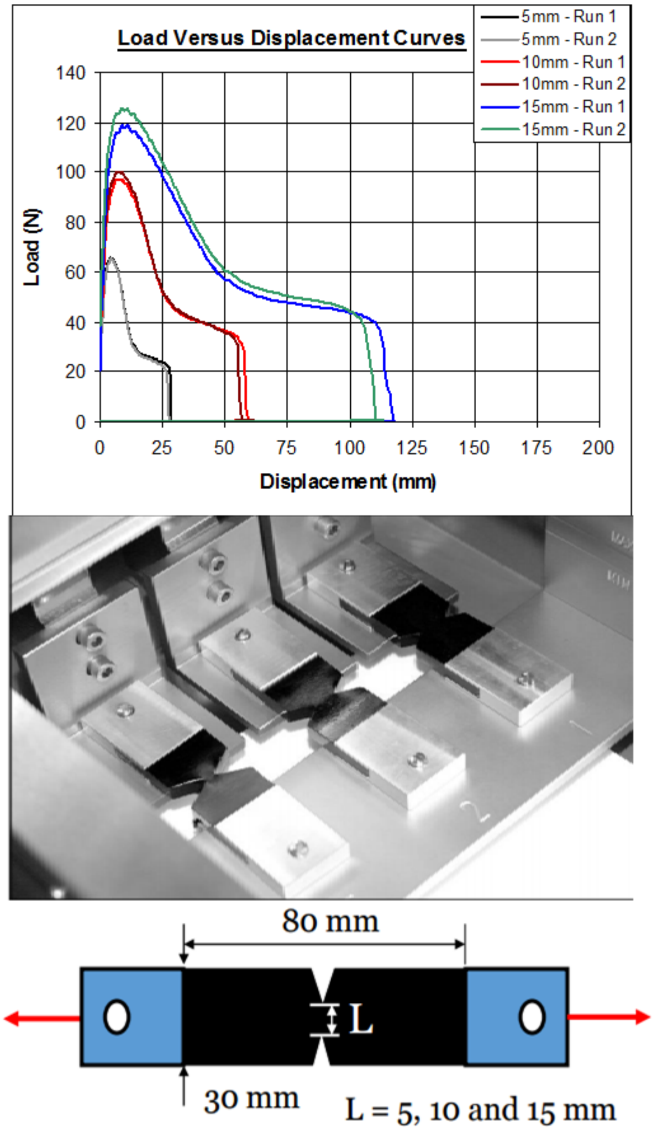

Figure 21: Load vs. Displacement Curve-Typical Raw Data (21)

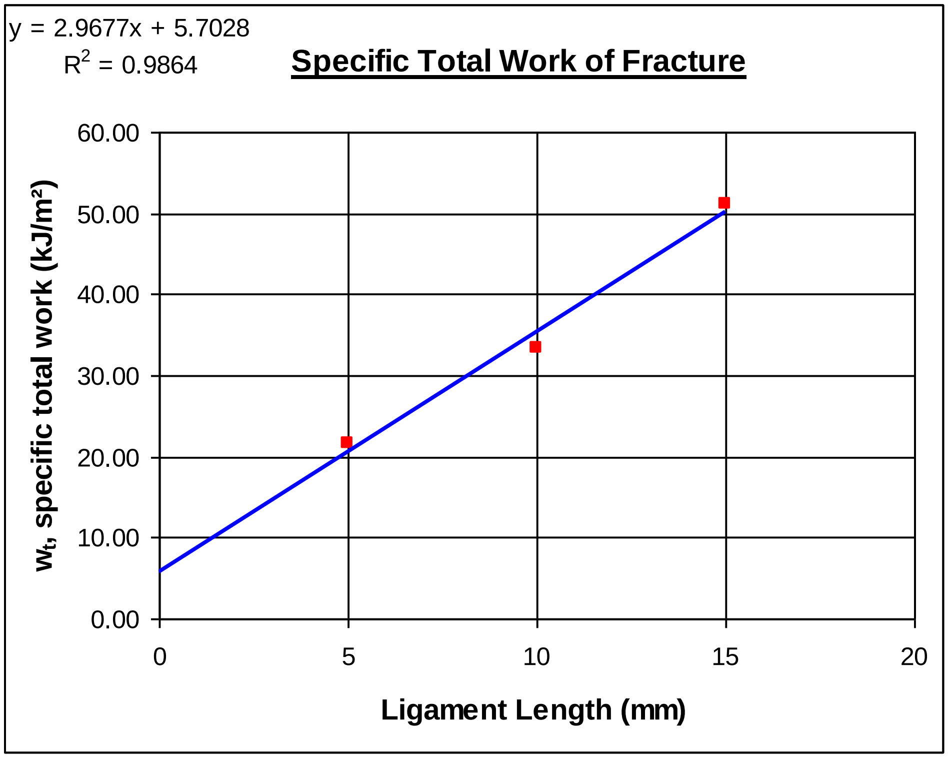

Figure 22: Specific total work vs. Ligament Length Raw Data (21)

Essentially, critical opening displacement (CTOD) parameter represents the ultimate elongation, or strain tolerance, in the vicinity of a notch or a crack at a zero ligament length. Thus, as the CTOD parameter increases, it would indicate that asphalt binder has a higher strain tolerance and therefore more resistant to fracturing. This test has been found to correlate well to field performance fatigue cracking at Federal Highway Administration (FHWA) Accelerated Loading Facility (ALF), as well as laboratory studies at Texas A&M Transportation Institute (TTI) and Rutgers Asphalt Pavement Laboratory (RAPL) (16).

Linear Amplitude Sweep Test (LAS)-N_f Parameter

Linear amplitude sweep test is used to estimate accelerated damage fatigue tolerance of asphalt binders in accordance with ASSHTO TP 101-14, *Estimating Damage Tolerance* *of Asphalt Binders Using the Linear Amplitude Sweep*. The test measures resistance to damage by means of cyclic loading, with linearly increased load amplitudes. The sweep is conducted using dynamic shear rheometer at intermediate temperature determined from PG-grade specification in AASHTO M 320. The test is conducted on PAV aged material with 8mm geometry and a 2mm gap setting. Two sets of tests are run to determine the binder fatigue performance parameter (N_f). First, a frequency sweep test is run to determine the initial undamaged parameter (α) of the specimen. This is done by applying a load of 0.1 % strain over a range of frequencies from 0.2-30 Hz. Then, complex shear modulus G* (Pa) and phase angle (δ) are recorded at each frequency as shown below (14).

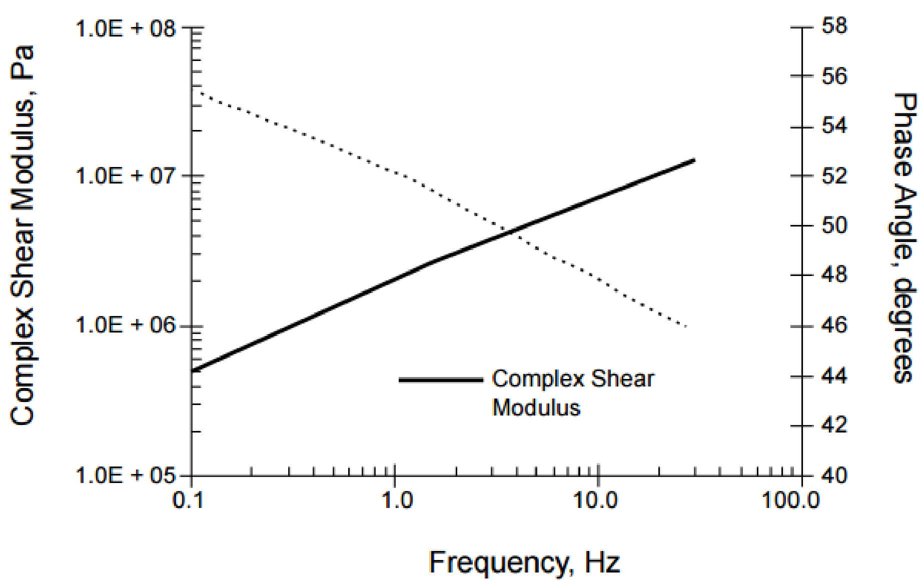

Figure 23: Frequency Sweep Test Output (14)

The second test is run using oscillatory shear in strain control mode at a selected temperature and frequency of 10 Hz. The load is linearly increased from 0 to 30% over a 3,100 number of loading cycles. Peak stress and strain are recorded every 10 load cycles, along with phase angle and shear modulus, as illustrated in figure 24. This allows for the determination of damage accumulation, as well as cycles to failure as a function of strain. Ultimately, by adjusting the strain levels for a given pavement structure and traffic density, the number of cycles to failure (N_f) can be determined (14).

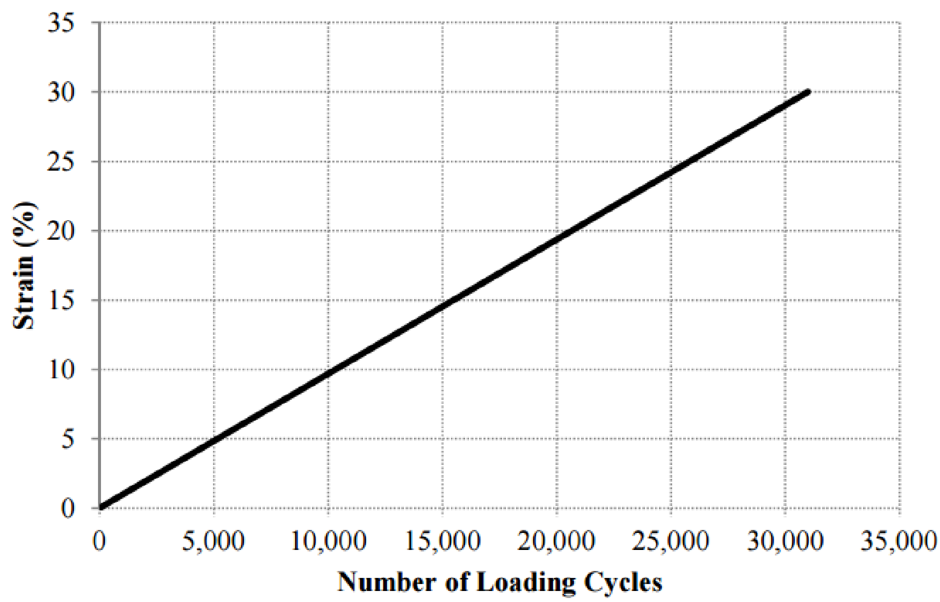

Figure 24: Loading for Amplitude Sweep Test (14)

Chapter 5: Detailed Work Plan

A thorough Literature Review on relevant technical reports, published books, journal articles, and conference presentations regarding the current asphalt binder specifications for airfield pavements, proposed binder improvement selection and innovative asphalt binder testing procedures has been conducted. Based on the literature review and recent consultations with the Port Authority of New York and New Jersey, the following work plan was developed.

Airfield Fatigue Cracking Study

The Port Authority of New York and New Jersey (PANYNJ) have recently been observing a wide range of service lives with their asphalt runways. In particular, the PANYNJ are noting pavement lives ranging from 3 and a half to 15 years, with the major distress observed being longitudinal and transverse cracking and top down cracking; cracking stops approximately 0.5 to 0.75 inches below surface. The condition of the airfields at the time of coring is show in figure 25. To help explain the varied degree of fatigue cracking performance, a research study was undertaken by Rutgers Asphalt Pavement Laboratory (RAPL), to evaluate six different asphalt runways of different fatigue cracking lives. Field cores were recovered from the runways for forensic testing using the different asphalt binder fatigue characterization tests noted earlier. The half inch increments were utilized to evaluate the change in binder properties with depth due to oxidative aging.

Figure 25 : PANYNJ Airfield HMA Pavement Conditions at time of Coring

The cores from six runways are shown in table 6. Three of the runways evaluated were from Newark Liberty International Airport (EWR), and the other three were from John F. Kennedy International Airport (JFK). Each runway specifies the cores location, number of cores, thickness, mix type, binder type, supplier, and visual observations at the time of coring.

Table 6: PANYNJ Core Specimen Location and Inventory

Runway	Core Location	Core Thickness	Mix Type	Binder Type	Supplier	Visual Observations	Aggregate Type	Date Placed (Age)	# of Cores
EWR 11-29 (Core Set 1)	Station 38+84, Offset 16 ft, Right of Centerline	2 inches	FAA #3	PG64-22 + 7% Vestoplast	Mt. Hope, Tilcon B Plant	Not performing well; Excessive cracking	Gneiss	9/20/2008 (6 Yrs, 9 Months)	11 (1 cracked)
EWR 11-29 (Core Set 2)	Station 5+99, Offset 63 ft, Right of Centerline	2 inches	FAA #3	PG64-22 + 7% Vestoplast	Mt. Hope, Tilcon B Plant	Not performing well; Excessive cracking	Gneiss	8/9/2008 (6 Yrs, 10 Months)	11 (1 cracked)
JFK 4R-22L (Core Set 3)	Station 39+50, Offset 50 ft, Right of Centerline	3 inches	FAA #3	PG76-22	Willets Pt Asphalt, Flushing, NY	Performing well; No cracking	Trap Rock (from Tilcon, Haverstraw)	9/5/2002 (12 Yrs, 9 Months)	10
JFK 4L-22R (Core Set 4)	Station -12+87, Offset 5 ft, Left of Centerline	3 inches	FAA #3	PG76-28	Willets Pt Asphalt, Flushing, NY	Performing well; Very few cracks	Trap Rock (from Tilcon, Haverstraw)	6/4/2000 (15 Yrs)	10
JFK 4L-22R (Core Set 5)	Station -10+18, Offset 27 ft, Right of Centerline	3 inches	FAA #3	PG76-28	Mt. Hope Rock Products, Flushing NY	Performing well; some cracking	Gneiss	6/4/2000 (15 Yrs)	10
Newark 4R-22L (Core Set 6)	Station 51+00, Offset 11 ft, West of Centerline	3 inches	FAA #3	PG64-22 + 7% Vestoplast	Weldon Asphalt, #11 Batch Plant, Kearny NJ	Not performing well; Excessive cracking	Fanwood Trap Rock/ Riverdale Sand	6/23/2012 (3 Yrs, 5 Months)	6

Further, table 7 specifies the volumetric and asphalt binder properties for each runway. In addition, it provides a date of HMA pavement placement, and specifies its age at the time of coring.

Table 7 : PANYNJ Volumetric and Binder Properties

Runway	Binder Type	Asphalt Content	QC Air Voids	QC VMA	QC VFA	Eff AC by Vol (%)	Flow	#200	#200/Eff AC by Vol	Visual Observations	Date Placed (Age)
EWR 11-29 (Core Set 1)	PG64-22 + 7% Vestoplast	5.37	3.4	15.8	78.8	12.4	11.8	4.5	0.36	Not performing well; Excessive cracking	9/20/2008 (6 Yrs, 9 Months)
EWR 11-29 (Core Set 2)	PG64-22 + 7% Vestoplast	5.3	3.5	15.9	77.9	12.4	11	3.9	0.31	Not performing well; Excessive cracking	8/9/2008 (6 Yrs, 10 Months)
JFK 4R-22L (Core Set 3)	PG76-22	5.14	4.9	17	71.1	12.1	13.8	4.4	0.36	Performing well; No cracking	9/5/2002 (12 Yrs, 9 Months)
JFK 4L-22R (Core Set 4)	PG76-28	5.02	4.6	17	72.9	12.4	13.3	4.8	0.39	Performing well; Very few cracks	6/4/2000 (15 Yrs)
JFK 4L-22R (Core Set 5)	PG76-28	5.05	4.6	16.4	72	11.8	14.5	3.7	0.31	Performing well; some cracking	6/4/2000 (15 Yrs)
Newark 4R-22L	PG64-22 + 7% Vestoplast	5.74	4.0	16.9	76.7	12.9	13.4	4.1	0.32	Not performing well; Excessive cracking	6/23/2012 (3 Yrs, 5 Months)

The three EWR pavements with longer in service life, lower asphalt content, and higher quality control (QC) air voids, performed better compared to the three newer JFK pavements with higher asphalt content and lower QC air voids. This contradicts the typical quality control volumetric design results of HMA pavements as illustrated in figure 26 (32). Form this we can assume that one the main causes of the fatigue cracking of the airfield HMA surface layer is from the binder rheological properties. In addition, from the binder types used, it would appear that asphalt binder pavements with addition of vestoplast polymer performed worse compared to neat binders with no additives. Therefore, an extensive forensic study was undertaken to relate different types of asphalt binders placed to their observed field performance.

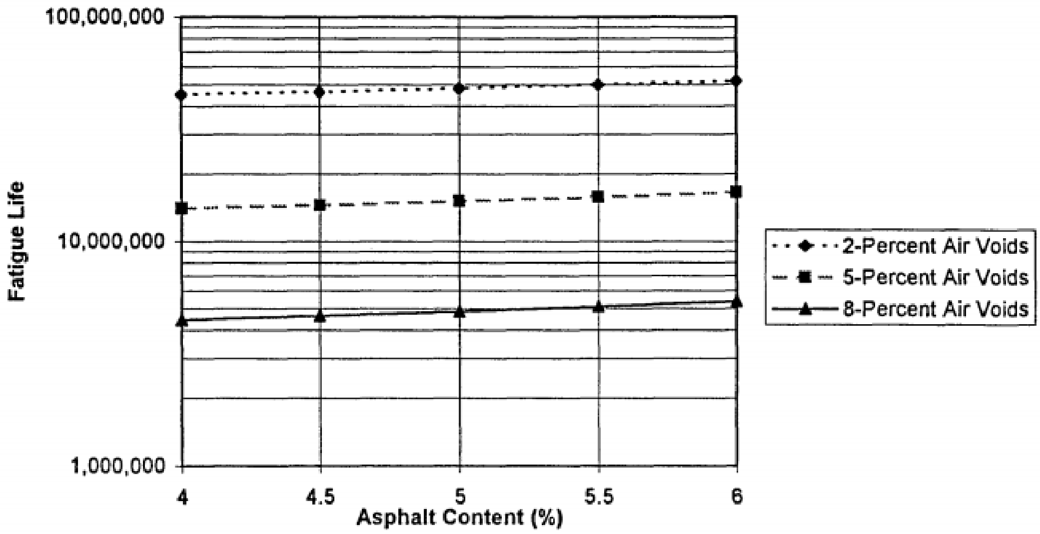

Figure 26: Effect of Asphalt & Air-Void Content on HMA Fatigue Life (32)

<u>Combining core specimens into separate lifts</u>

For each airfield location, an average of 10 cores was taken for evaluation. The cores ranged in thickness between 2 and 3 inches. After thorough visual examination, the specimens were carefully cut at ½ inch increments from top to bottom of surface layer using a wet saw. The core materials at each lift was then combined and used for asphalt binder solvent extraction and recovery to determine the degree of aging in terms of pavement depth by using an array of tests. All six airfield pavements were placed at air voids less than 6.5% and exposed to same climatic conditions during their service life.

Therefore, the degree effect due to aging and weathering is comparable within the six pavements.

Extraction and Recovery

After combining the material from each specimen lift, an extraction and recovery procedure was conducted. The four stage process allows for the separation of asphalt binder from mineral aggregates. After the binder is recovered, it is utilized for asphalt binder testing to determine its physical and mechanical performance properties. The recovered asphalt binder is treated as RTFO aged, under the assumption that equivalent aging occurred at the asphalt plant production. The four stage process is illustrated in figure 27.

Figure 27: Extraction and Recovery Process

The extraction and recovery process follows the procedures in accordance with AASHTO -T164, *Quantitative Extraction of Asphalt Binder from Hot Mix Asphalt (HMA)* and ASTM-D5404, *Recovery of Asphalt from Solution Using the Rotary Evaporator*. First, the cores were cut into lifts using a wet saw and then they were broken down in the oven at 110±5°C until the material was workable. After the sample was dried to constant mass, the initial weight (W_1) was determined. The sample was then placed in the 3000 gram extraction bowl, submerged with tri-chloroethylene (TCE) solvent and covered with a filter and the lid for up to one hour. After soaking the asphalt mixture the centrifuge was started thus separating majority of the binder and TCE solvent from mineral aggregate. This process was repeated at least 3 times or until the solution coming out of the centrifuge was not darker than light straw color. The dried weight of the sample after this primary centrifuge was recorded as W_3. The TCE binder solution was then ran through the continuous 100 gram filler centrifuge, separating the very fine mineral (passing the No. 200 sieve) matter that was initially missed by the primary centrifuge. The dried mineral matter weight was then recorded as W_4. The three weights were then used in determining the asphalt binder content of the material, as shown in the equation 15 (28).

$$AC\ \% = \frac{(W_1 - W_2) - (W_3 + W_4)}{(W_1 - W_2)} * 100 \qquad (15)$$

where,

W_1= initial mass of the sample
W_2=mass of water in the sample (assumed to be 0)
W_3=mass of extracted mineral aggregate
W_4=mass of mineral matter in the extract

The last step included the recovery of the asphalt binder from tri-chloroethylene (TCE) solvent following ASTM-D5404 specifications. Utilizing the Rotovap equipment as shown in figure 27, allowed us to separate the tri-chloroethylene (TCE) solvent from the binder through a distillation process. For repeatable results, the temperature and applied vacuum is controlled (28). After the binder has been recovered, it is then used for classification/verification of the performance grade, and other innovative binder testing.

In order to verify the extraction and recovery procedure, Rutgers Asphalt Pavement Laboratory participated in the sensitive study with 10 other AMRL accredited laboratories. The study consisted of performing an extraction and recovery, and further grading the binder for its continuous performance grade. The results show that, the grading for high, intermediate, and low temperatures all fall within the mean minimum error of testing (30). This verifies the extraction and recovery process as well as the performance grading of the binder.

Binder Testing

The recovered binder from each lift was treated as RTFO aged material, for all binder testing. This was done in order to capture the full effect of aging and binder properties within the depth of HMA pavement at its current stage. First, the binder was tested for its continuous PG-grade determination, from which the high, intermediate and low temperature grades were determined according to AASHTO M 320. The testing procedures are outlined in chapter 2 of the report, and included the utilization of dynamic shear rheometer and bending beam rheometer. Furthermore, the binder material was tested for "PG-Plus" specification parameters. This was done to better understand the aging and fatigue properties of binder with respect to pavement depth. These parameters included, the difference in critical low temperature – (ΔT_c), Christensen-Anderson Model (CAM) Rheological Index – (R-value) & Crossover Frequency – (ω), Glover-Rowe – (G-R), double edge notch tension test (DENT) – CTOD, and linear amplitude sweep test – N_f.

Laboratory PAV Calibration to Match Airfield Aging

This part of the research was developed based on literature review, and the initial finding from the airfield fatigue cracking study of this report. In order to better understand the durability or hardening of binder at different cross section of the pavement, Witczak and Mirza developed a global aging prediction model. The model shown in figure 28 was developed using field data from 40 different field projects throughout Europe and North America (29).

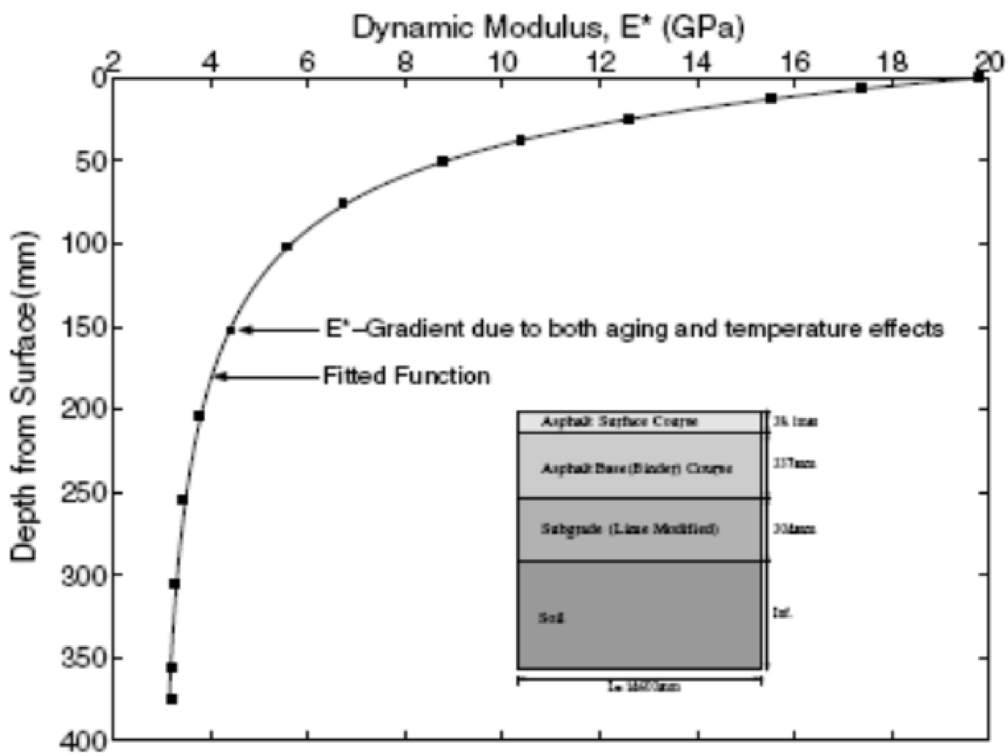

Figure 28: Aging Gradients in Asphalt Pavements

The model represents the stiffness gradient due to both aging and temperature effects. From the model it can be determined that most aging occurs at the surface layer, where pavement is more exposed to climate changes and the sun. At greater depth of pavement from the surface, the stiffness decreases and eventually plateaus at a certain depth. The aging gradient would change based on the mixture properties (29).

Based on the results from the airfield fatigue cracking study of this research, it was determined that aging appears to be insignificant at depth greater than 1 inch for all six airfield pavements. Therefore, asphalt binder from the bottom of core area was used to calibrate lab conditioning to surface conditioning of the same core. The binder material at bottom of core was assumed to be RTFO aged from plant production. The material was then aged for 20 and 40 hours in the pressure aging vessel (PAV). The PG-Grade properties from each aging intervals were then compared to the surface properties (in-situ condition) of that specific core specimen. Determining the difference between the aging properties and that at surface and knowing the actual age of the pavement at time of coring; we were able to simulate lab conditioning to field aging. In addition, we were able to relate conditioning of the asphalt binder to help assess durability and aging characteristics.

Chapter 6: Final Results

Binder "Fatigue" Testing

<u>High Temperature PG Grade</u>

High temperature PG-Grade results are shown in figure 29. As expected, the high temperature grade of the binder reduces with depth in the pavement cross section. This is because the pavement is less exposed to oxidation and environmental effects at greater depth from the surface. In addition, at depth greater than 1 inch, there appears to be little to no change in the high temperature grade for all the core sets. Therefore, the bottom of the core material can be represented as the initial material used from plant production. As previously mentioned, rutting typically occurs in the early stage of pavements life. Although rutting was not a problem for this particular project, we can assume that Newark set 2 & JFK set 5 would perform the best in terms of rutting susceptibility, because it will maximize the $G^*/\sin\delta$ rutting parameter developed by SHRP.

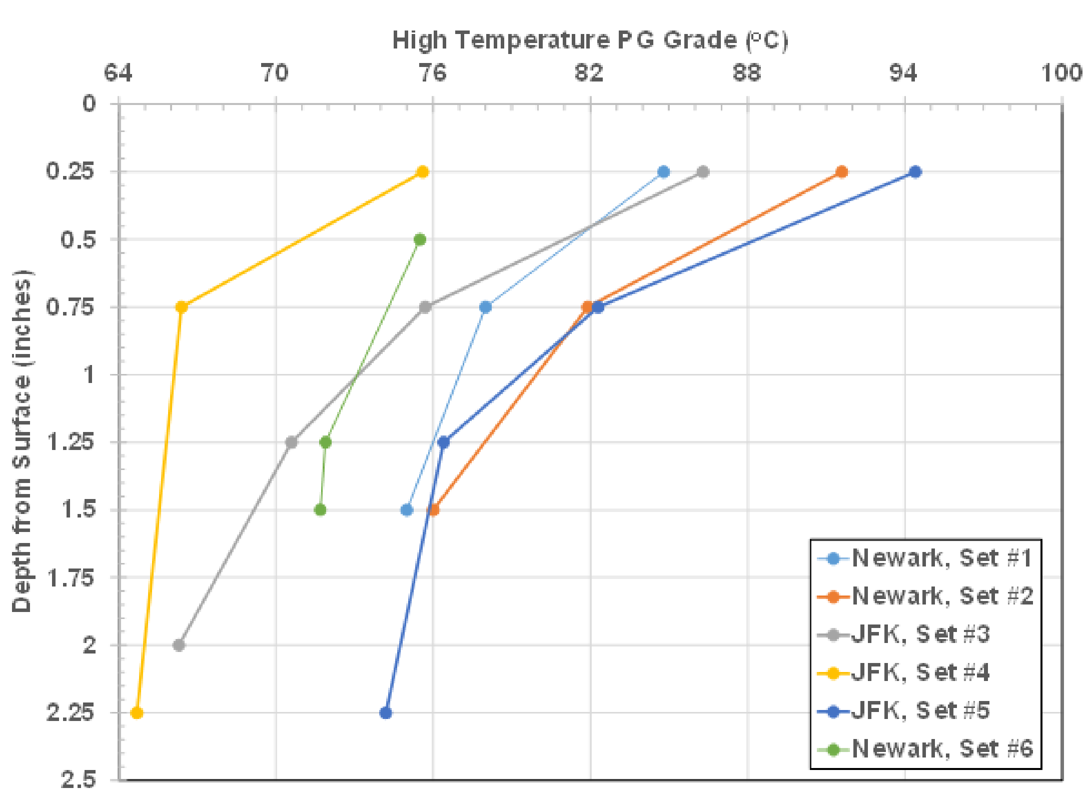

Figure 29: High Temperature PG-Grade

<u>Intermediate Temperature PG Grade</u>

Intermediate PG grade results also correlate well to aging within the pavement depth. Figure 30 shows that at greater depth from the surface, the binder grade reduces and becomes less stiff. There also appears to be insignificant aging at depth greater than 1 inch. Moreover, as previously discussed, fatigue cracking distresses occur later in the pavements life at intermediate temperatures. The distresses are typically justified by the materials stiffness and relaxation properties. Therefore, from the stiffness perspective, looking at the surface layer, we can say that JFK core set 4 would perform the best in terms of fatigues cracking performance, because it will minimize the G*sinδ fatigue parameter developed by SHRP.

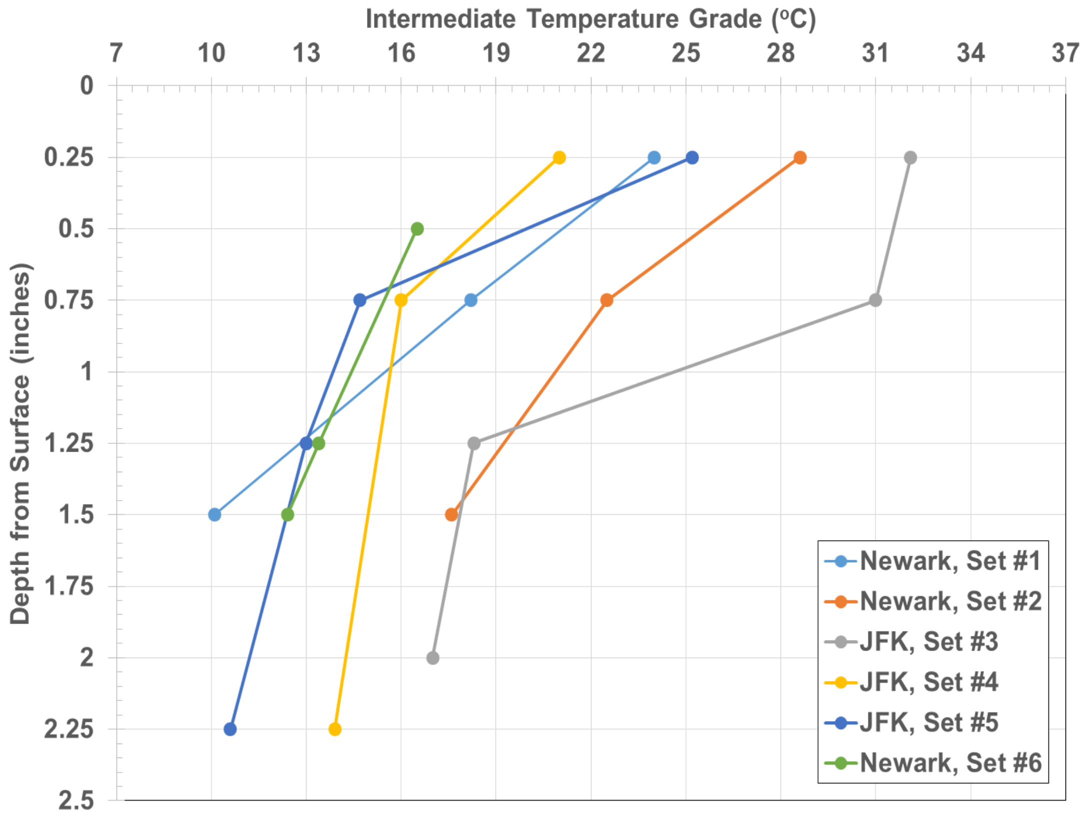

Figure 30: Intermediate Temperature PG-Grade

<u>Christensen-Anderson Model (CAM)-(R-values & ω)</u>

Christensen-Anderson Model (CAM) is a good indication tool in determining master curve physical parameters (R, ω). It utilizes the master curve shape data analysis from which it determines the rheological index (R-value) and cross over frequency (ω). Normally R-value is higher and ω-value is lower for oxidized/aged asphalt binders. Figure 31 shows the results of plotting the crossover frequency (ω) vs. rheological index(R) for different airfield pavements. We can clearly see that binders from airfield cores are aging more as they reach the surface layer. JFK core set 5 appears to show the most significant amount of aging, which would be expected because the pavement is 15 plus years old.

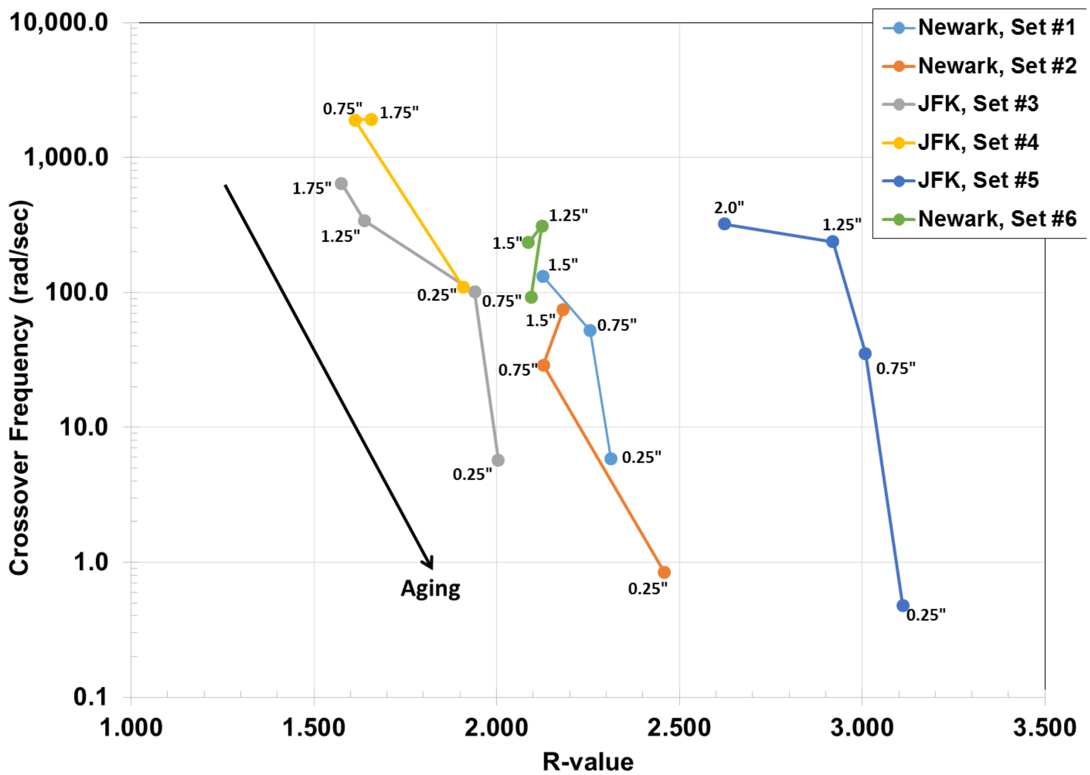

Figure 31: Rheological Index & Crossover Frequency

<u>Difference in Critical Low Temperature PG grade ΔT_c Parameter</u>

As previously mentioned in chapter 4, ΔT_c represents the difference between the critical low temperature stiffness S-value and relaxation index m-value. These parameters can be calculated directly from bending beam rheometer (BBR), which is standard equipment in determining the PG-grade of a binder, specified in AASHTO M 320. The asphalt binders closer to the surface are more aged, therefore the relaxation properties (m-value) are negatively affected at a greater rate than the stiffness (S), thus reducing the ΔT_c. The initial limit for non-load associated crack onset was set at $\Delta T_c \leq$ -2.5°C and further advanced to $\Delta T_c \leq$ -5.0°C for significant crack propagation. From figure 32, we can determine that Newark set 1 and 2, and JFK set 5 binders, performed the worst while, JFK set 4 performed the best in terms of fatigue cracking performance at the surface layer. These also show a very good correlation with the field observations. JFK airfield pavement set 5, is 15 years old and just started showing some level of cracking. The pavements parameter is significantly improved with depth.

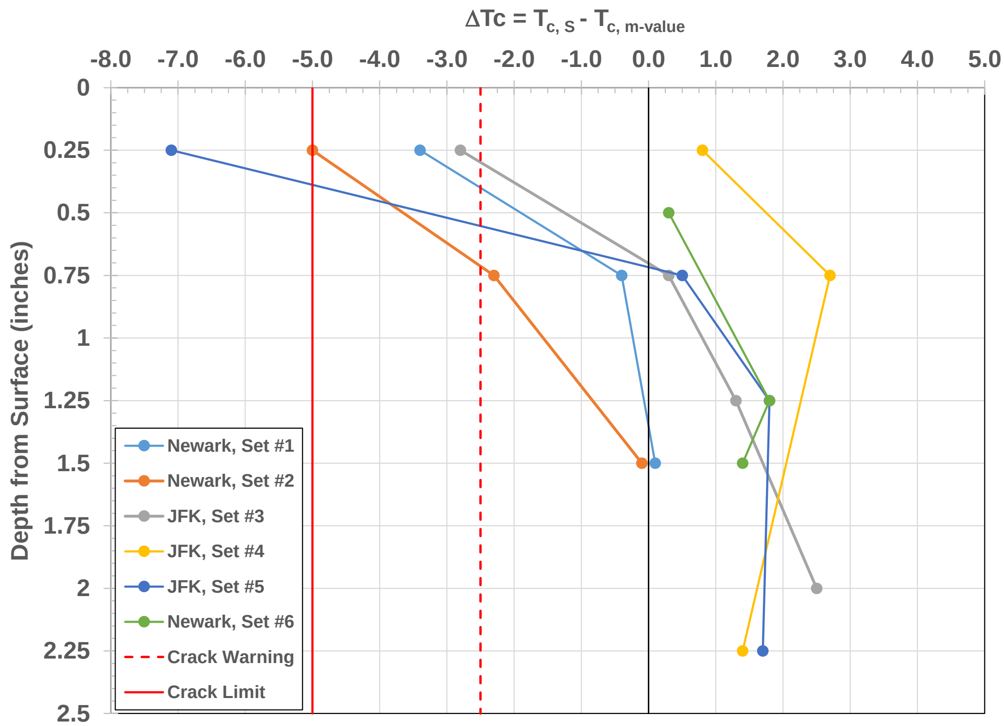

Figure 32: Difference in Critical Low Temperature ΔT_c

<u>Double Edge Notched Tension (DENT) Test</u>

The double edge notched tension evaluates the energy that is required to fracture ductile materials. It measures the essential work of fracture and critical tip opening displacement (CTOD), by utilizing the total work of fracture under the force-displacement curve. CTOD represents ultimate elongation, or strain tolerance in the vicinity of a crack or a notch. Thus, as CTOD increases it has more resistance to fracture. Figure 33 results show that JFK set 4 performed the best & Newark set 1 and JFK set 5 performed the worst in terms of fatigue cracking at the surface layer. JFK set 5 is a 15 years old pavement and just started to show signs of cracking. In addition, at a depth greater than 1 inch, there appears to be little to no change in the strain tolerance, indicating minimal aging. The test results related very well to field observations, as well as ΔT_c and G-R Parameters.

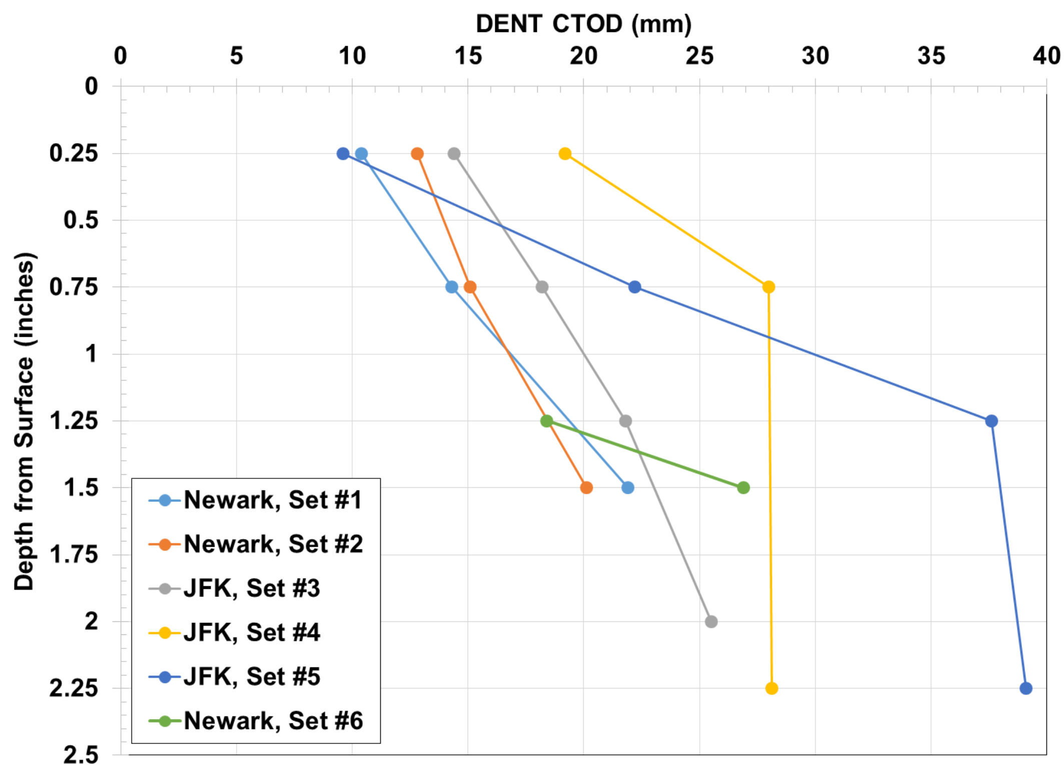

Figure 33: DENT CTOD Parameter

<u>Glover-Rowe - (G-R) Parameter</u>

Glover et al. (2005) developed the parameter based on the finding by Kandhal, who related asphalt binder oxidation hardening to its ductility at (15°C; 1cm/min). Glover rheological parameter related to a single-point measurement in the standard DSR equipment. Rowe further advanced the parameter, relating it simply as a function of G* and δ, and naming it the Glover-Rowe (G-R) Parameter. The non-load associated limits for damage onset and significant cracking were set at 180 kPa and 450 kPa respectively.

Ultimately the higher the G-R parameter the more prone the surface is to cracking. From figure 34, we can conclude that JFK set 4 performed the best and Newark set 2 performed the worst in terms of fatigue cracking. These findings relate well to the observed field performance, and also showed a very good correlation with the ΔT_c and CTOD parameters. In addition, the parameter improves with depth, but shows little to no change at depth greater than 1 inch.

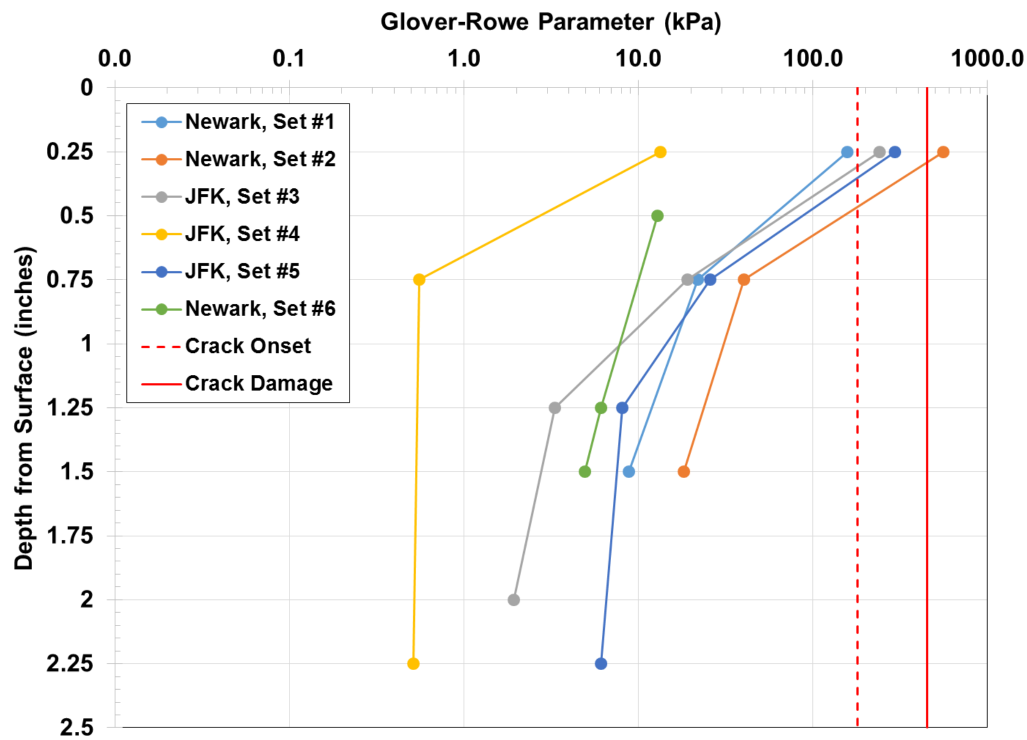

Figure 34: Glover- Rowe Parameter (G-R)

<u>Linear Amplitude Sweep (LAS) Test</u>

The linear amplitude sweep test (LAS) utilizes cyclic loading in the DSR to evaluate the undamaged and damaged condition of asphalt binders under increased accelerated damage. The analysis of the test allows for the determination of binder fatigue life in (cycles) at different shear strain levels, depending on the traffic conditions. For the testing purposes, a typical shear strain of 2.5% and 5.0% is used. For a stiffer binder, the number of cycles N_f is expected to be less than that of a softer binder. Moreover, as the binder softens with respect to depth in the HMA pavement, the fatigue life is expected to improve. However, in some instances, this was not the case from the LAS analysis of this project. Figures 35 and 36, show that for some airfield core pavements, the number of cycles actually decreased as binder got softer, like that of JFK set 5. In addition, the results did not correlate well to observed field performance and other innovative binder characterization test parameters.

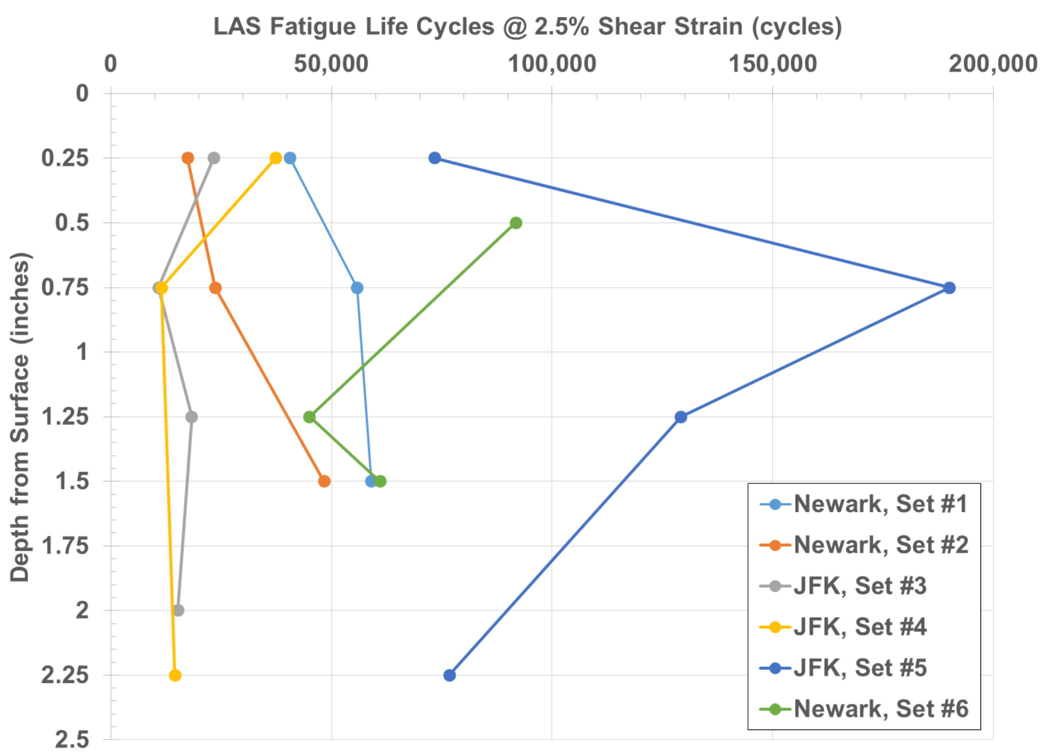

Figure 35: Linear Amplitude Sweep @ 2.5% shear strain

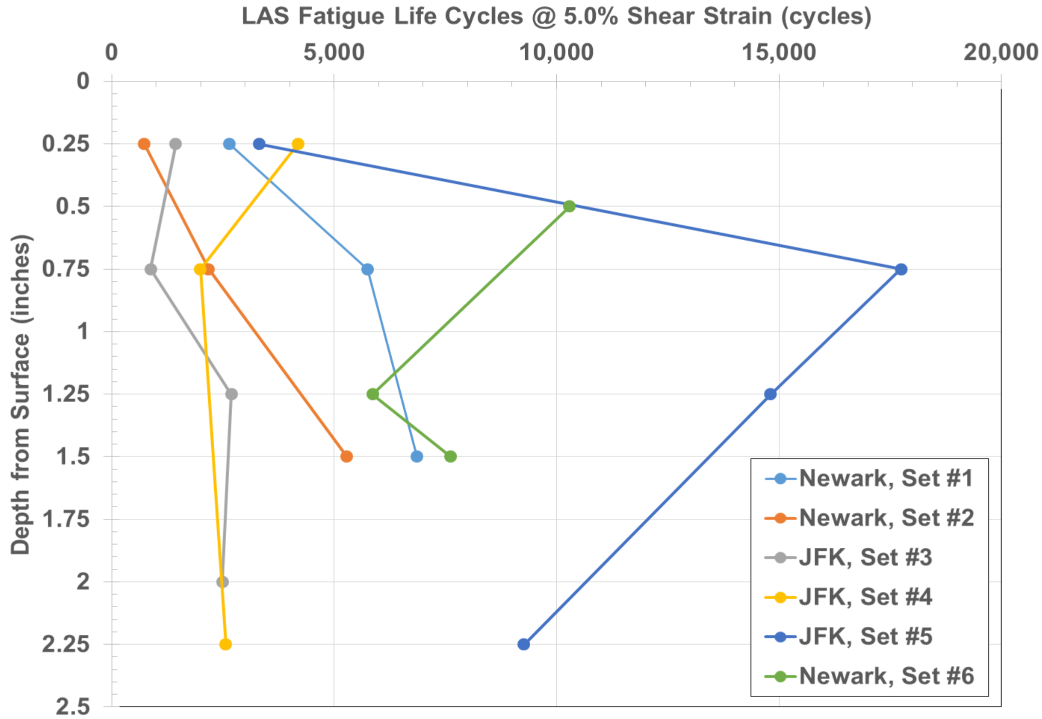

Figure 36: Linear Amplitude Sweep @ 5.0% shear strain

<u>Binder "Fatigue" Test- Ranking of Core Sets</u>

From the past research, engineers related durability of asphalt binders to its ductility properties. Further, it was related as the hypothesized property to flexibility and relaxation of asphalt binders. From the extensive binder analysis of this research it was determined that three parameters related well to ductility and expected loss of flexibility. These parameters also related well to the observed field cracking distresses. The first is the parameter suggested by Rowe, which he termed the Glover-Rove – (G-R) parameter, determined using DSR. Second, is the parameter which quantifies the difference between stiffness and relaxation properties of asphalt binder, referred to as the – ΔT_c, determined using BBR. And the last parameter which showed a strong association with the observed cracking is the critical tip opening displacement – (CTOD) parameter, determined using

DENT. The parameters were evaluated at depth of 0.25 inches, 0.75 inches, and 1.5 inches, for all six airfield pavements. A ranking score of 1 to 5 was developed to evaluate the pavement fatigue cracking performance as illustrated in table 8. A score of “1” represented an asphalt binder, which performed well in terms of fatigue cracking resistance related to that specific parameter, at which point no maintenance is recommended. A score of “5” represented an asphalt binder which exhibits the worse fatigue cracking performance for the specific parameter, at which point rehabilitation or reconstruction maintenance is recommended. The three parameters were evaluated, and the average fatigue resistance value was determined for different cross section depths in the pavement. From the binder ranking of the core set, we can conclude that for the top 0.25 inches of the HMA pavement layer, JFK set 4 performed the best (avg. score 1) and JFK set 5 performed the worst (avg. score 4.7). For the intermediate layer at a depth of 0.75 inches, JFK core set 4 again performed the best (avg. score 1) and Newark set 2 performed the worst (avg. score 4.7). And for the bottom layer of the HMA pavement at 1.5 inches, JFK sets 3, 4, and 5 equally performed well (avg. score 2), and Newark set 2 performed the worst (avg. score 5), followed by Newark set 1 (avg. score 4). The average fatigue performance values also related very well to the observed field performance, noted at the top of table 8.

Table 8: Binder "Fatigue" Test-Ranking of Core Sets

Runway	Binder Type	Visual Observations	Date Placed(Age)
Newark, Set #1	PG64-22 + 7% Vestoplast	Not performing well; Excessive Cracking	9/20/2008 (6 Yrs, 9 Months)
Newark, Set #2	PG64-22 + 7% Vestoplast	Not performing well; Excessive Cracking	8/9/2008 (6 Yrs, 10 Months)
JFK, Set #3	PG76-22	Performing well; No Cracking	9/5/2002 (12 Yrs, 9 Months)
JFK, Set #4	PG76-28	Performing well; Very Few Cracks	6/4/2000 (15 Yrs)
JFK, Set #5	PG76-28	Performing well; Some Cracking	6/4/2000 (15 Yrs)
Newark, Set #6	PG64-22 + 7% Vestoplast	Not performing well; Excessive Cracking	6/23/2012 (3 Yrs, 5 Months)

0.25" Depth	ΔTcr	CTOD (mm)	Glover-Rowe	Average
Newark, Set #1	3	4	2	**3.0**
Newark, Set #2	4	3	5	**4.0**
JFK, Set #3	2	2	3	**2.3**
JFK, Set #4	1	1	1	**1.0**
JFK, Set #5	5	5	4	**4.7**
Newark, Set #6	-	-	-	-
0.75" Depth	**ΔTcr**	**CTOD (mm)**	**Glover-Rowe**	**Average**
Newark, Set #1	4	5	3	**4.0**
Newark, Set #2	5	4	5	**4.7**
JFK, Set #3	3	3	2	**2.7**
JFK, Set #4	1	1	1	**1.0**
JFK, Set #5	2	2	4	**2.7**
Newark, Set #6	2	-	2	**2**
1.5" Depth	**ΔTcr**	**CTOD (mm)**	**Glover-Rowe**	**Average**
Newark, Set #1	4	4	4	**4.0**
Newark, Set #2	5	5	5	**5.0**
JFK, Set #3	1	3	2	**2.0**
JFK, Set #4	3	2	1	**2.0**
JFK, Set #5	2	1	3	**2.0**
Newark, Set #6	3	2	3	**2.7**

Laboratory PAV Conditioning

The results of this sections entails the means of utilizing laboratory conditioning to better simulate field aging. In addition, it helps to verify the potential use of 40 hour PAV conditioning. From the previous discussion of this report, it was noted that gradient of aging appears to be significant at depth greater than 1 inch. Therefore, the bottom of the core specimen could be used to calibrate lab conditioning to surface conditioning of the same core. Asphalt binder extracted and recovered from the bottom of the core is assumed to be RTFO aged. The material was then aged in the PAV for 20 and 40 hours. The measured aged properties were then compared to those at the surface of same core specimen. The core specimens evaluated, were from 6 different airfields with varying ages (3 to 15 years), asphalt and aggregate types. The properties evaluated were based on High Temperature (HT) PG grade, Intermediate Temperature (Int.) PG grade, Low Temperature (LT) PG grade-m-slope, Low Temperature (LT) PG grade- Stiffness, and difference in critical low temperature ΔT_c. The difference in properties of “zero” represented that lab conditioning matched field aging. The data points were plotted based on the difference in properties vs. the field age of the pavement in years. The best fit line was then utilized to determine the pavements age at a difference of “zero”. For example, from figure 37, based on HT results, we can say that 20 hours of PAV conditioning simulates approximately 9 years of field aging. The same concept was applied for other tests, as well as the 40 hour PAV conditioning. Further, the mean value and standard deviation of all tests was determined at 20 hour and 40 hour PAV conditioning. The results are as follows:

- 20 Hour PAV aging simulates 7.1 years of field aging (Std. Ded. =1.42 years)
- 40 Hour PAV aging simulates 12.2 years of field aging (Std. Dev=1.19 years)

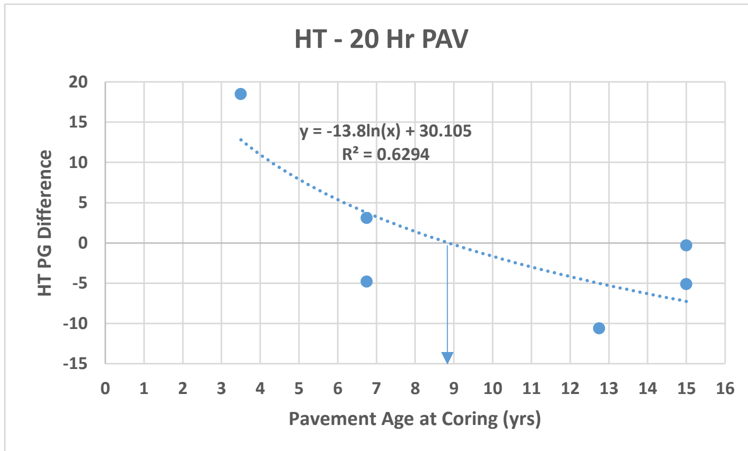

Figure 37: High Temperature 20Hr. PAV Conditioning

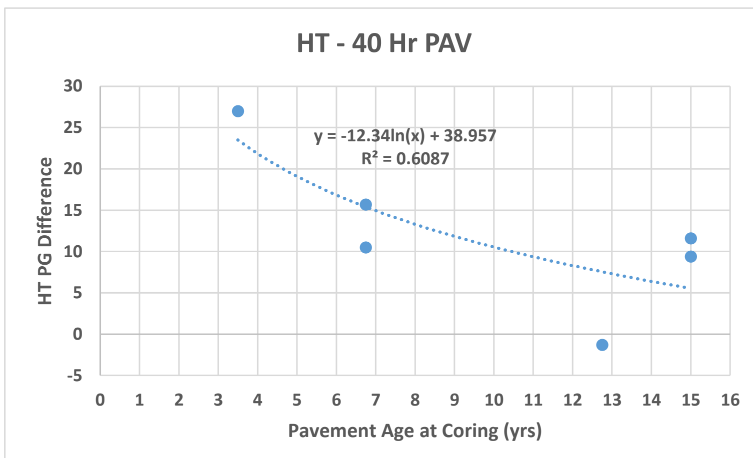

Figure 38: High Temperature 40Hr. PAV Conditioning

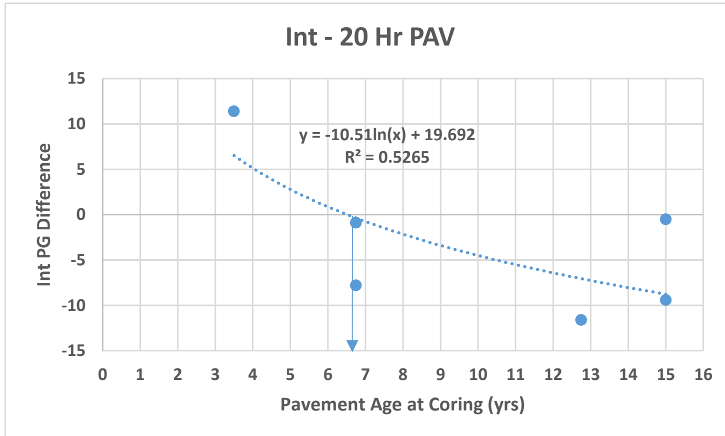

Figure 39: Intermediate Temperature 20Hr. PAV Conditioning

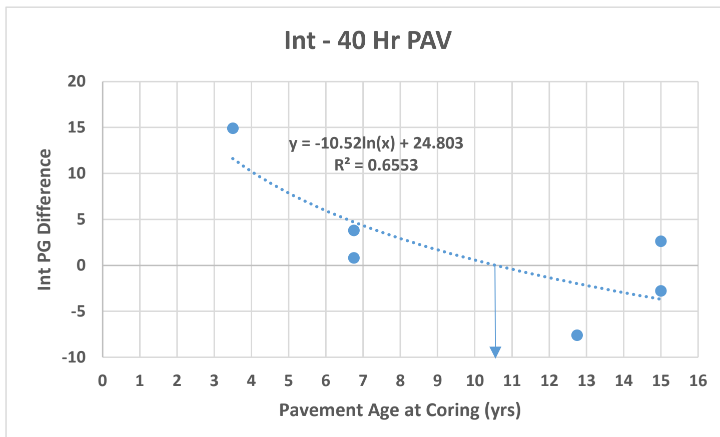

Figure 40: Intermediate Temperature 40Hr. PAV Conditioning

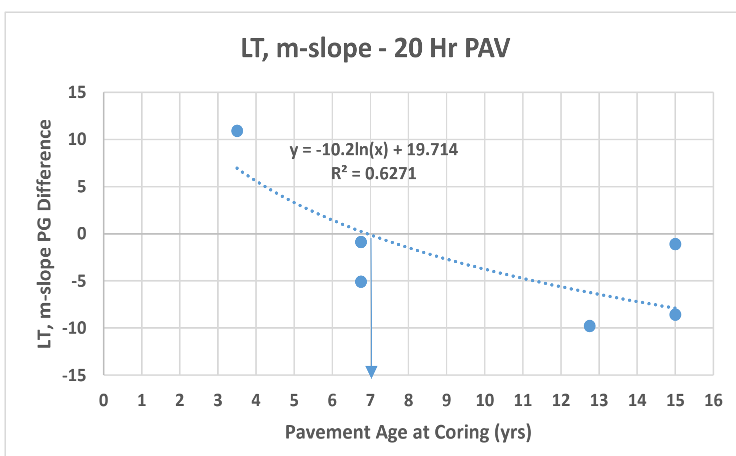

Figure 41 : Low Temperature m-slope 20Hr. PAV Conditioning

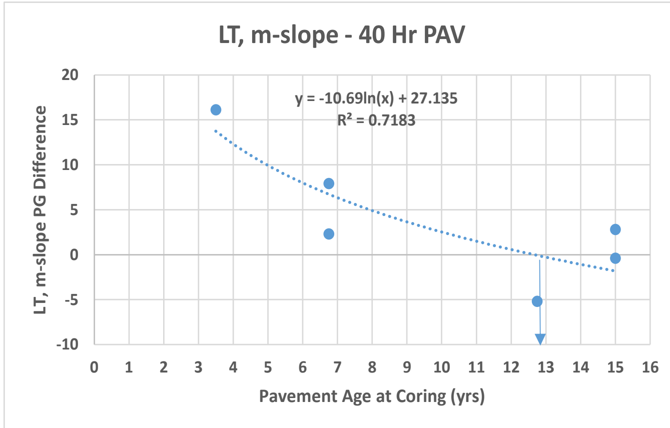

Figure 42: Low Temperature m-slope 40Hr. PAV Conditioning

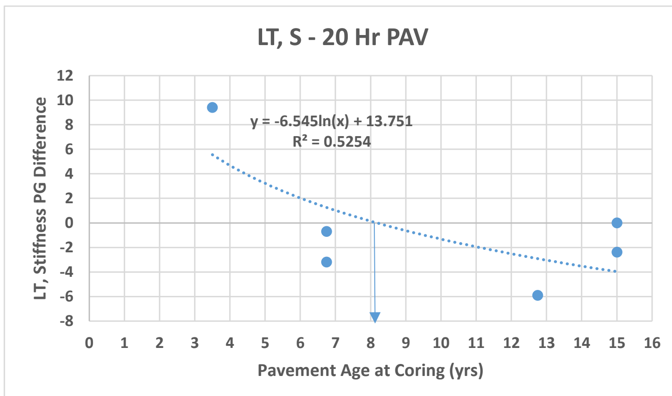

Figure 43: Low Temperature Stiffness 20Hr. PAV Conditioning

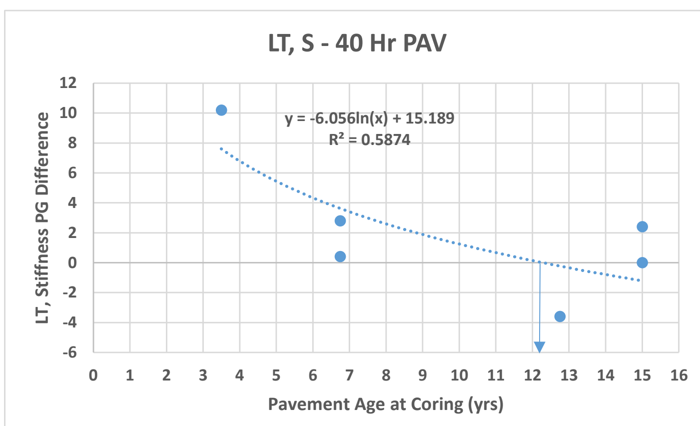

Figure 44: Low Temperature Stiffness 40Hr. PAV Conditioning

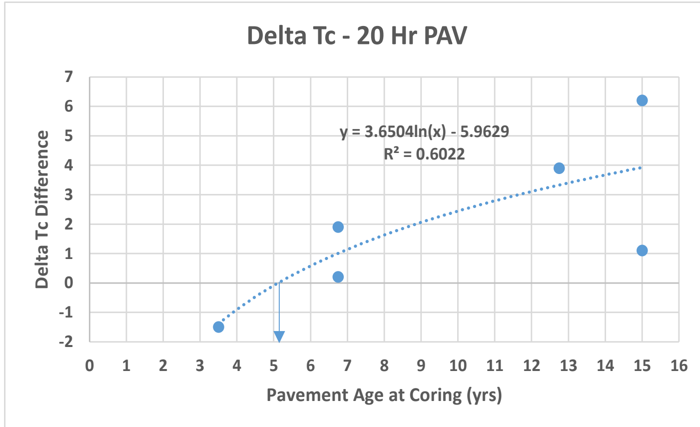

Figure 45: Critical Low Temperature ΔT_c 20Hr. PAV Conditioning

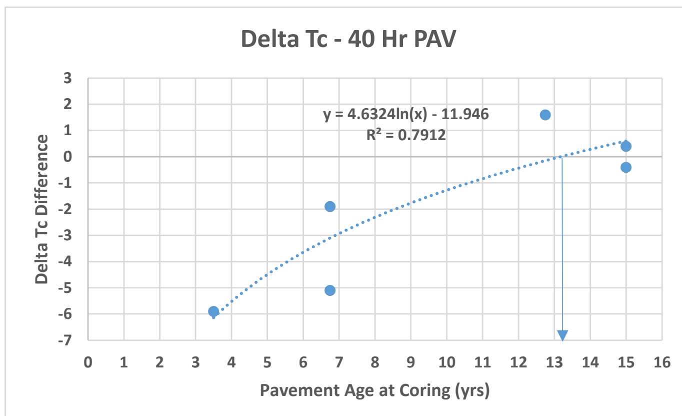

Figure 46: Critical Low Temperature ΔT_c 40Hr. PAV Conditioning

PG-Plus Specification- Recommended Parameter

This report relates three test parameters that show a strong fatigue correlation with each other, and the observed field performance distresses. The parameters looked at include: ΔT_c, CTOD, and G-R parameters. While, all three parameters have been shown to relate to ductility and loss of flexibility due to aging, not all yield an engineering property. Meaning, the temperature conditions at which the test is carried out, do not necessarily represent the climate conditions of the pavement locale. The G-R parameter was originally developed based on Kandhal's work, relating ductility at 15 °C for various test sections in Ohio and Pennsylvania. The ductility parameter is very sensitive to temperature; therefore limits for ductility at colder or warmer climates should also be met. In addition, critical tip opening displacement (CTOD) parameter is determined at two standard temperature of 25±0.5 °C or 15 ±0.5 °C, depending on ductile failure. These two temperatures do not necessary represent the intermediate pavement fatigue temperatures of different climates. To adopt the G-R and CTOD parameters for the next generation, some tie to pavement temperature through LTPPBind should be made for added validation. On the other hand, ΔT_c parameter is already tied to climate based on the standard testing procedures from BBR, which is run at the recommended pavement temperature based on the climate location. Therefore, ΔT_c parameter is chosen as the "PG-Plus" specification parameter for this project.

The ΔT_c parameter uses the standard low temperature grade procedures and is temperature independent. Regardless of the climate or asphalt binder grade, when ΔT_c reaches a certain threshold limit, it will indicate a drop of ductility and loss of durability due to aging. The threshold limits were proposed based on the PAV conditioning for 20 and 40 hours. Both aging intervals appeared to separate good and poor performing asphalt binders. For the 20 hours PAV conditioning a value greater than -1.0 °C appears to identify good and poor performing binders as illustrated in figure 47. For the 40 hours PAV conditioning a value greater than -5.0 °C appears to identify good and poor performing binders as illustrated in figure 48. JFK core set 5 did achieve a high negative ΔT_c value; however, this pavement is 15 plus years old and just started to show signs of cracking.

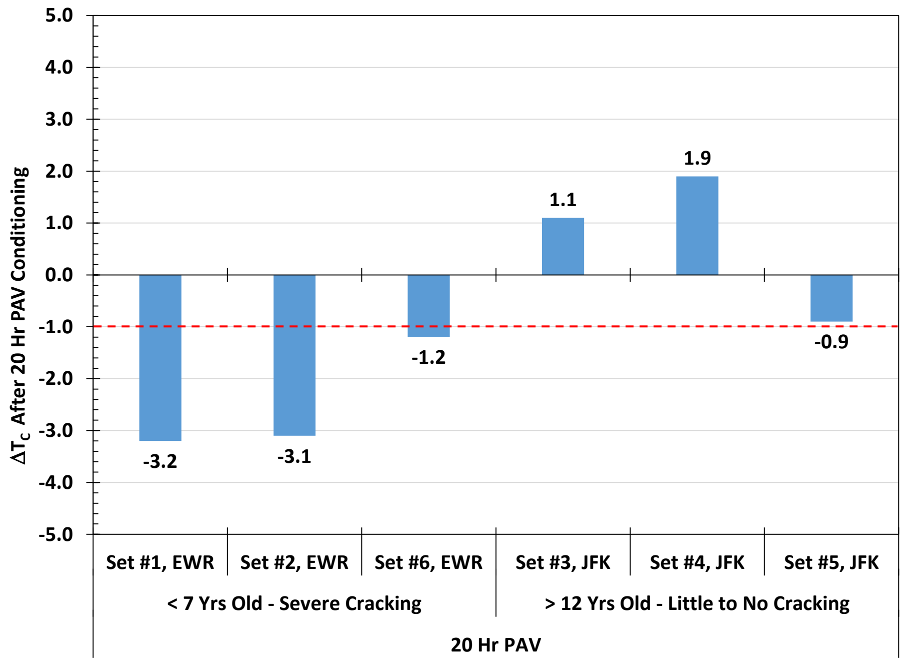

Figure 47: ΔT_c 20 Hour Recommended Parameter

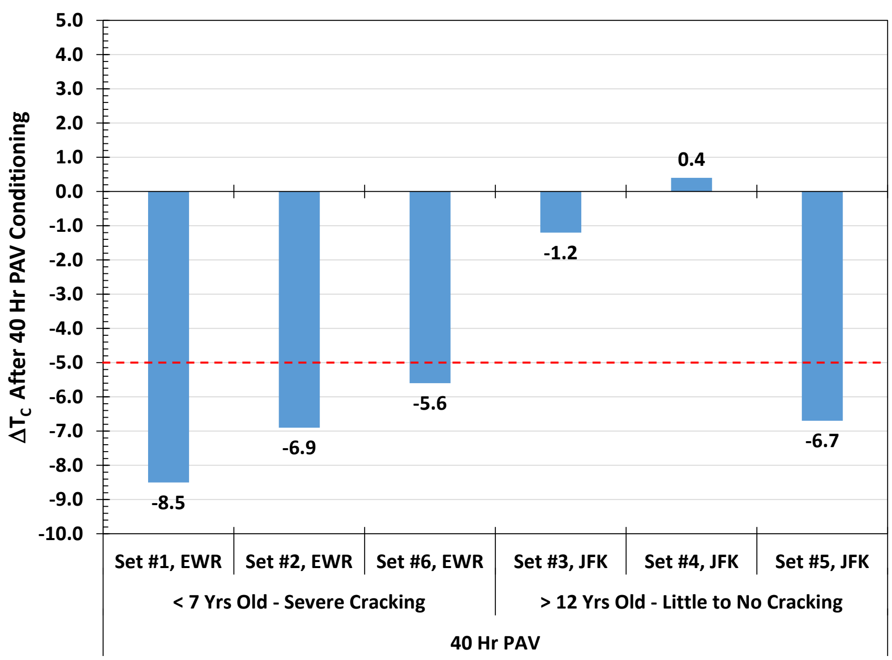

Figure 48: ΔT_c 40 Hour Recommended Parameter

Chapter 7: Final Conclusions

Limitations and Future Research

This research paper has few limitations that could alter its general use. First, the six pavements that were evaluated in the New York & New Jersey area may or may not represent the aging gradient effect of asphalt binders in the United States. Second, the mechanical and rheological properties of asphalt binders may change based on the volumetric properties of different mix designs. Third, due to equipment and material size constraints, ductility testing was not available. In addition, due to material constrains, double edge notched Tension (DENT) test was not available for the pressure aging vessel (PAV) calibration conditioning.

Additional work is needed to fully validate the effects of fatigue properties on airfield HMA mixes. This includes looking at different sources which could alter fatigue performance. The most important one includes the asphalt mixture performance properties. These could be determined by the performance testing from overlay tester and semi-circular bend (SCB) tester. The overlay tester measures the fatigue life through continuously triangular displacement, 5 sec loading and 5 sec unloading. The semi-circular bend (SCB) test evaluates the energy required to fracture the specimen and propagate a crack at the notch. Additional sources effecting fatigue performance of HMA pavement include: the construction deficiencies on airfield pavement design performance, and the effect on properties from fuel contamination and de-icing chemicals. Moreover, for future research, the bottom portion of the core specimen can be used to develop laboratory to field mixture aging protocols.

Conclusion and Recommendation

Study Conclusions

Based on the results of this research, three major factors were distinguished. One, the research team was able to relate the observed field distresses to the laboratory testing protocols using an array of tests. The three parameters that related best to field performance were: the difference in critical low temperature PG grade – (ΔT_c), Glover-Rove parameter – (G-R), and the double edge notched tension test – (CTOD). Two, the results from the high and intermediate temperature PG grades and the results from the innovative binder characterization test parameters showed that gradient of aging seemed to be significant at depth greater than 1 inch, indicating that little to no aging occurred in the vicinity of that depth (would change based on in-situ air voids). Therefore, the bottom portion of the core material was assumed to be RTFO material aged from plant production and utilized to develop laboratory PAV conditioning to match field aging. The material from the bottom of the core was extracted and recovered in accordance with AASHTO T-164, and ASTM D-5404 respectively. The results indicated that 20 hour PAV aging simulates 7.1 years of field aging (Std Dev=1.42 years) and 40 hour PAV aging simulates 12.2 years of field aging (Std Dev=1.19 years). The conditioning represented a rheological conditioning, not true kinetic or chemical aging. And lastly, the conditioning of the asphalt binders was used to help assess durability and aging characteristics, and helped to develop “PG-Plus” binder purchase specification fatigue parameter. While there were three parameters that showed a good correlation to the observed field performance, only one of them is tied to the climate condition in which the asphalt binder is being selected. This parameter is the difference between low temperature cracking –ΔT_c. In addition, regardless of the binder performance grade or climate, once the ΔT_c parameter reaches certain limit, it will indicate a drop of ductility and loss of durability due to aging, which is related to fatigue cracking. This parameter is also easily computed because it can be determined from the standard PG performance grade testing for asphalt binders.

In Europe, polymer asphalt binder modifiers are vastly used in the surface course of HMA pavement design. This is done, in order to achieve a preferable PG-grade, improve resistance to pavement distresses, and to reduce pavement design and maintenance costs. While many polymers aid in improving the pavements durability, some may be detrimental in fatigue performance distresses. From this research it was determined that pavements which included vestoplast polymer in their design, negatively affected the binder performance properties in terms of fatigue cracking.

Recommended Use

Based on the results of this study, the research team recommends that the critical low temperature cracking parameter (ΔT_c) be adopted as a "PG-Plus"- fatigue based asphalt binder purchase specification for airfield asphalt pavements. The asphalt binder should first be selected on the basis of the current PG-Grade selection methods, or that proposed by Airfield Asphalt Pavement Technology Program Project 04-02 using LTPPBind software. The airfield engineers should then coordinate with the local laboratory testing facility in order to determine the ΔT_c values at 20 and 40 hour of PAV aging. In order to satisfy the fatigue cracking performance, the limits for the ΔT_c parameter should meet the following criteria:

ΔT_c Fatigue Crack Limit Parameter

$$\Delta \boldsymbol{T_c} = \boldsymbol{T_{c(stiffness)}} - \boldsymbol{T_{c(m-slope)}}$$

- **$\Delta T_c \geq$ -1.0 °C for 20 hour of PAV**
- **$\Delta T_c \geq$ -5.0 °C for 40 hour of PAV**

Glover-Rove – (G-R), and double edge notch tension test – (CTOD) parameters should be considered for future validation and binder specification tests. An organized round-robin study should be conducted for ruggedness evaluation of G-R & CTOD parameters. LTPPBind climate data should be explored to identify different sites with fatigue performance and to characterize the binders for G-R and CTOD parameters for additional validation. This work should then be presented to the Federal Aviation Administration (FAA) for future implementations.

References

1. Hanson, Douglas I., et al. "Project 05-07 Techniques for Prevention and Remediation of Non-Load Related Distresses on HMA Airport Pavements (Phase I)." (2009).
2. AMEC Earth and Environmental, Inc., "Project 06-01 Guide for Prevention and Mitigation of Non-Load-Associated Distress (Phase II)." 2010.
3. Anderson, David A., et al. "Binder characterization and evaluation, volume 3: Physical characterization." Strategic Highway Research Program, National Research Council, Report No. SHRP-A-369 (1994).
4. Christensen, D. W. "Airfield Asphalt Pavement Technology Program Project 04-02: PG Binder Grade Selection for Airfield Pavements." Revised Final Report, Advanced Asphalt Technologies (2008).
5. Christensen Jr. Donals W,. Roy McQueen, and Hussain U. Bahia. "Selection of High-Temperature Binder PG Grades for Airfield Pavements."(2007).
6. Anderson, R. M. "The asphalt binder handbook, MS-26." (2011).
7. Hot Mix Asphalt 101. State.nj.us. (2016) retrieved from http://www.state.nj.us/transportation/eng/pavement/pdf/HotMix0709.pdf
8. Derewecki, Kristen E. Applications of the 4 mm dynamic shear rheometer geometry for the forensic evaluation of bituminous materials. Diss. Rutgers University-Graduate School-New Brunswick, 2013.
9. Wang, Hao. "Pavement Material Properties". Lecture. Rutgers University-Graduate School-New Brunswick, 2014.
10. McNally, Tony, ed. Polymer modified bitumen: properties and characterisation. Elsevier, 2011.
11. http://www.utexas.edu/research/superpave/articles/koch1.html
12. Superpave Binder Specification. Construction Division. (2006). retrieved form ftp://ftp.dot.state.tx.us/pub/txdot-info/cst/superpavebindspec.pdf
13. Circular, Advisory. "Standards for specifying construction of airports: AC 150/5370-10G Federal Aviation Administration." (2014).
14. Test, Film Oven. "Estimating damage tolerance of asphalt binders using the linear amplitude sweep." (2010): 101-14.
15. Andriescu, Adrian, Simon Hesp, and John Youtcheff. "Essential and plastic works of ductile fracture in asphalt binders." Transportation Research Record: Journal of the Transportation Research Board 1875 (2004): 1-7.
16. Gibson, N. H., et al. "Full-Scale Accelerated Performance Testing for Superpave and Structural Validation: Transportation Pooled Fund Study TPF-5 (019) and SPR-2 (174) Accelerated Pavement Testing of Crumb Rubber Modified Asphalt Pavements." Federal Highway Administration, Report No. FHWA-HRT-11-045, McLean, VA (2011).
17. Glover, Charles J., et al. Development of a new method for assessing asphalt binder durability with field validation. No. FHWA/TX-05/1872-2. 2005.
18. Casola, John. "Modified Asphalt Market Survey 2005-2006." C]//The Association of Modified Asphalt Producers Meeting. Saint Louis, USA: AMAP. 2006.
19. Selection of Asphalt Binder Grade Handout. Dot.ca.gov. 2016. Retrieved from http://www.dot.ca.gov/hq/construc/CPDirectives/CPD06-11attach2.pdf

20. Bennert, Thomas. Laboratory Evaluation of Vestoplast Modified Hot Mix Asphalt (HMA). No. PANYNJ-RU9247. 2007.
21. Zofka, Adam, and Mihai Marasteanu. "Development of double edge notched tension (DENT) test for asphalt binders." Journal of Testing and Evaluation35.3 (2006): 1-7.
22. Anderson, R. Michael, et al. "Evaluation of the relationship between asphalt binder properties and non-load related cracking." Journal of the Association of Asphalt Paving Technologists 80 (2011).
23. Rowe, G. M., G. King, and M. Anderson. "The Influence of Binder Rheology on the Cracking of Asphalt Mixes in Airport and Highway Projects." Journal of Testing and Evaluation 42.5 (2014): 1-10.
24. Bennert, Thomas, et al. "Warm Mix Asphalt Trials in New York State-Laboratory and Field Performance." Transportation Research Board. 2016.
25. Bennert, Thomas, et al. "Laboratory Performance of Re-refined Engine Oil Bottoms (REOB) Modified Asphalt." Transportation Research Board. 2016.
26. Marek, Charles R., ed. Low-temperature Properties of Bituminous Materials and Compacted Bituminous Paving Mixtures: A Symposium Presented at the Seventy-ninth Annual Meeting, American Society for Testing and Materials, Chicago, Ill., 27 June-2 July 1976. Vol. 628. ASTM International, 1977.
27. Christensen, Donald W., and David A. Anderson. "Interpretation of dynamic mechanical test data for paving grade asphalt cements (with discussion)."Journal of the Association of Asphalt Paving Technologists 61 (1992).
28. American Association of State Highway, and Transportation Officials. Standard specifications for transportation materials and methods of sampling and testing. AASHTO, 2011.
29. Mirza, Muhammad Waseem, and M. W. Witczak. "Development of a global aging system for short and long term aging of asphalt cements (with discussion)." Journal of the Association of Asphalt Paving Technologists 64 (1995).
30. Zvirblis, Donatas. Influence of high RAP on binder properties in hot mix asphalt of northeast. Diss. Rutgers University-Graduate School-New Brunswick, 2014.
31. U.S. Department of Transportation Federal Highway Administration. "LTPPBind Simplifies Superpave Binder Selection". (1999).Retrieved from tps://www.fhwa.dot.gov/publications/focus/99feb/ltppbind.cfm
32. Deacon, John A., Bor-Wen Tsai, and Carl L. Monismith. Fatigue performance of asphalt concrete mixes and its relationship to asphalt concrete pavement performance in California. No. RTA-65W485-2. University of California Berkeley, Institute of Transportation Studies, Asphalt Research Program, 1995.
33. Interactive, Pavement. “Rolling Thin Film Oven.” (2011).
34. Interactive, Pavement. “Dynamic Shear Rheometer.” (2011).
35. Interactive, Pavement. “Bending Beam Rheometer.” (2011).
36. Interactive, Pavement. “Direct Tension Tester.” (2011).