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Past, Present, and Future of Asphalt Binder Rheological Parameters

Synopsis of 2017 Technical Session 307 at the 96th Annual Meeting of the Transportation Research Board

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Past, Present, and Future of Asphalt Binder Rheological Parameters

Synopsis of 2017 Technical Session 307 at the 96th Annual Meeting of the Transportation Research Board

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Sponsored by
Standing Committee on Asphalt Binders

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Standing Committee on Non-Binder Components of Asphalt Mixtures
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Introduction

This e-circular was developed from presentations made during the 96th Annual Meeting of the Transportation Research Board, in a session titled “Black Space is Not a Black Hole: Past, Present, and Future of Asphalt Binder Rheological Parameters.” David Mensching of FHWA guided the session, which was sponsored by the Standing Committee on Asphalt Binders (AFK20) and co-sponsored by the Standing Committees on Non-Binder Components of Asphalt Mixtures (AFK30) and Structural Requirements of Asphalt Mixtures (AFK50).

The notion of an aging and under-funded infrastructure in the United States is widely accepted. Current legislation calls for performance targets to be established to aid not only in the administering of funds, but also to assist states in identifying high-risk practices and roadways within their own networks. In addition to the performance targets, agency officials are looking to design and construct longer-lasting pavements through performance specifications.

In the late-1980s and early-1990s, the Strategic Highway Research Program developed a performance-based purchase specification for asphalt binders. High, intermediate, and low temperature performance is considered at single time or frequency points evaluated at different temperatures to meet American Association of State and Highway Transportation Officials (AASHTO) thresholds. Over the last 20 years, members of the asphalt pavement industry have expressed concerns over the ability of the performance-grading thresholds to accurately assess field performance. That concern has been magnified in recent years due to the increasing representation of polymers, recycled materials, and other products in asphalt pavements.

As industry experts explore refinements to the performance-grading system, rheological analysis of asphalt binders is coming to the forefront. This analysis may include interconversions between other viscoelastic properties (i.e., creep compliance to relaxation modulus) or master curve determination. This more comprehensive look at material behavior is likely needed because modifiers and other additives may render some of the more basic rheological assumptions invalid. While this is not necessarily indicative of poor performance, the behavior across the expected temperature and frequency conditions is beneficial nonetheless. More information is provided on these methods throughout the E-Circular.

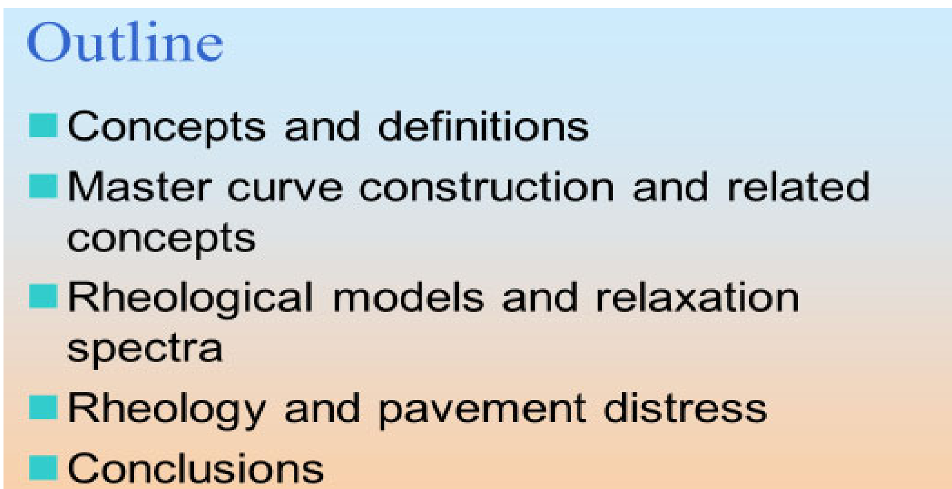
Contents

Asphalt Rheology Introduction

DON CHRISTENSEN
Advanced Asphalt Technologies

DAVE ANDERSON
Consultant

Four important topics are covered in this presentation (Slide 1). We first discuss the basic concepts and definitions needed to understand rheology and how it applies to asphalt technology. The next topic covered is that of the master curve—what is meant by this term, how to construct a master curve and how to interpret it. This is followed by information on rheological models and relaxation spectra. The final topic covered is how the rheological concepts discussed in this presentation potentially affect asphalt pavement performance.

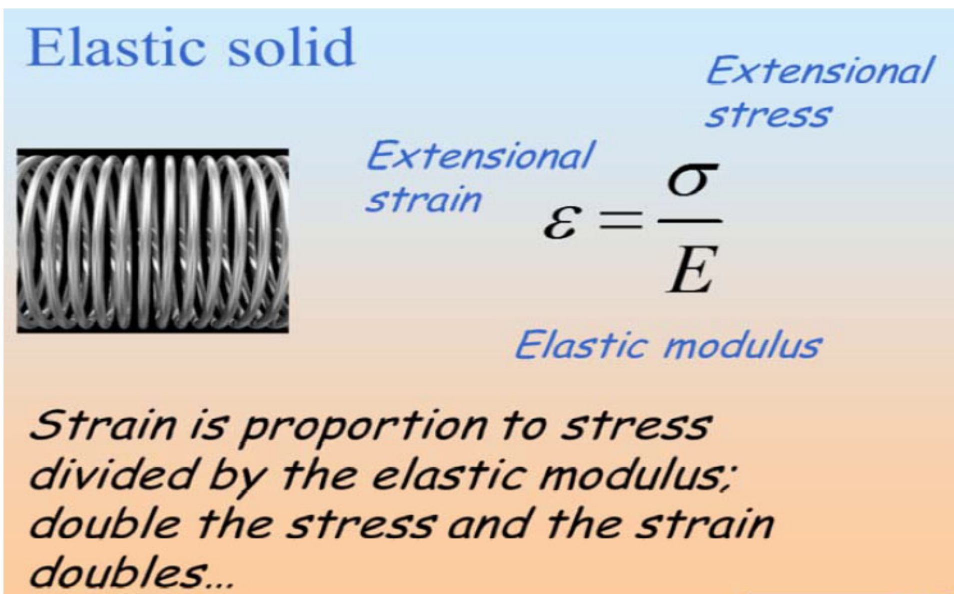


SLIDE 1

It is surprising how many materials that don't at first appear to be fluids can flow like water under the right conditions (Slide 2). Asphalt, pitch, and tar are usually semi-solid at room temperature and will flow very slowly when under stress. On the right of this slide is a photograph of the pitch drop experiment showing that even very stiff, glass-like pitch will flow if given enough time. We'll talk more about this demonstration later in the presentation.

In order to understand rheology and discuss it with other scientists and engineers, some basic concepts and definitions must be understood (Slide 3). An elastic solid is a material in which an applied stress produces a strain that is proportional to that stress. A steel spring is a simple example—if you double the load applied to a steel spring, the extension of the spring will also double. No matter how long you load an elastic solid, it will never flow like a fluid.

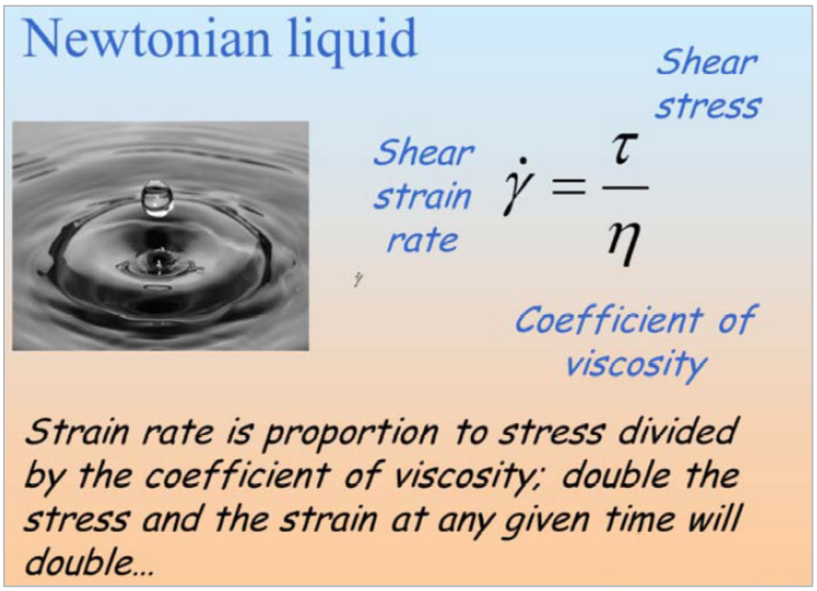
SLIDE 2



SLIDE 3

Water under many conditions can be considered a Newtonian fluid (Slide 4). The rate of flow of water (at low stresses and flow rates) is proportional to the applied shear stress. The material property that describes the relationship between shear stress and flowrate (shear strain rate) is the coefficient of viscosity η, often referred to simply as viscosity. Viscosity has the units of stress-time, such as Pa-s.

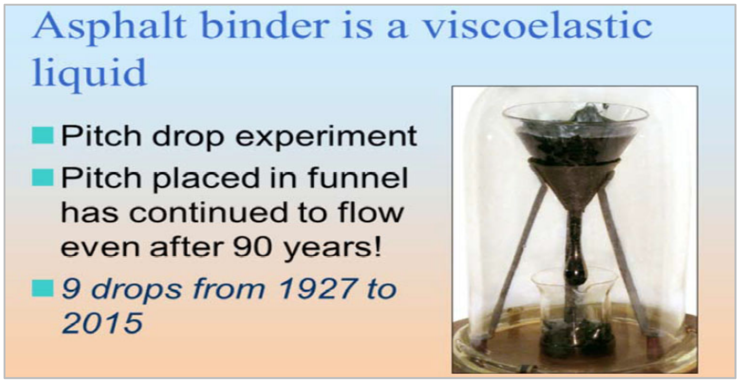
Many everyday materials are neither elastic solids nor Newtonian liquids, but instead are viscoelastic, meaning their mechanical behavior exhibits aspects of both types of materials (Slide 5). Many foods, such as honey, syrup, ketchup and chewy candies are viscoelastic. Many rubbers and plastics are also viscoelastic.



SLIDE 4

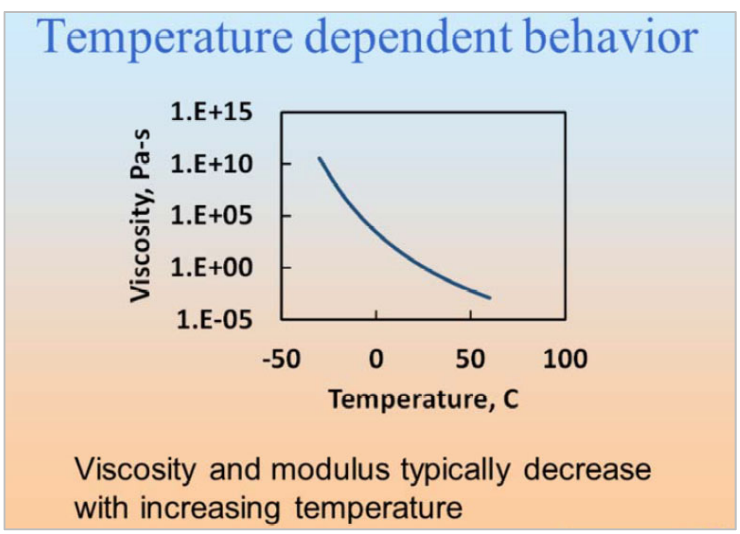
SLIDE 5

The pitch-drop experiment (mentioned a few slides ago) was devised by Professor Thomas Parnell of the University of Queensland in Brisbane, Australia. Professor Parnell set up this funnel of pitch—a stiff, asphalt-like material (Slide 6). In the 87 years since this experiment was started, a total of nine drops of pitch have flowed from the funnel. Even though the pitch used in this experiment appears to be a glass-like solid, given enough time it flows in the same way as water, but much more slowly; its viscosity is estimated to be 230 billion times that of water at room temperature.



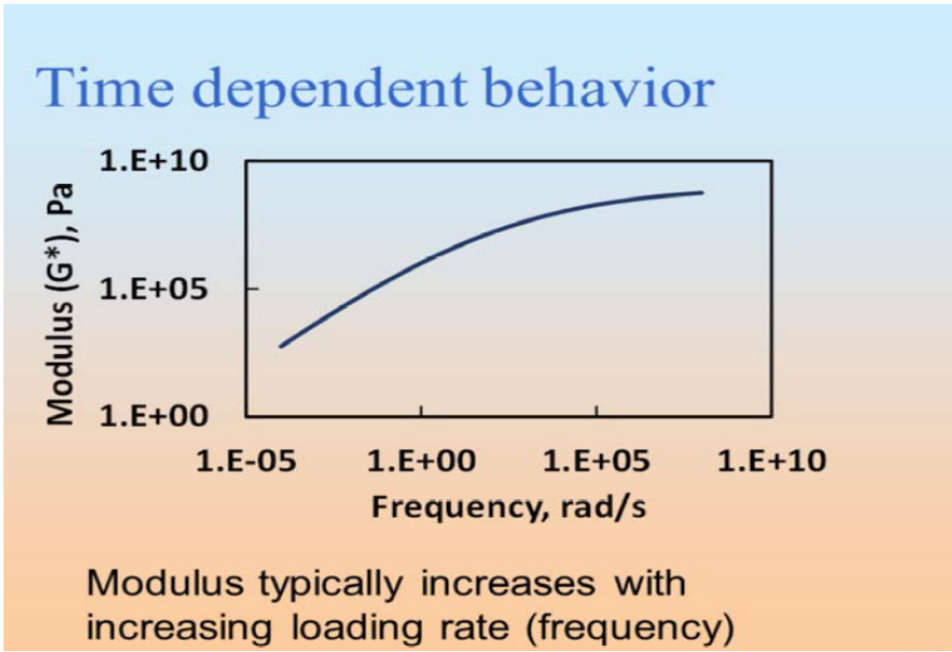
SLIDE 6

Most viscoelastic materials, including asphalt binders, exhibit temperature dependent behavior (Slide 7). This means that the mechanical properties change significantly when the temperature of the material changes. Usually the viscosity (and modulus) decrease as the temperature increases. The viscosity of asphalt cement typically drops by about a factor of 1,000 or more just by going from freezing to room temperature. The temperature susceptibility of asphalt binders makes devising effective tests and specifications a challenge.



SLIDE 7

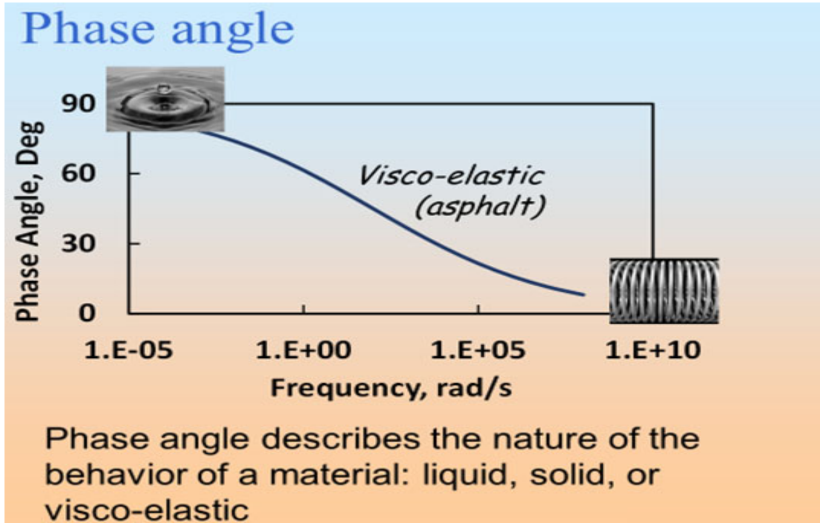
The mechanical properties of viscoelastic materials are also affected by changes in loading time or loading rate (Slide 8). At room temperature, the modulus of typical asphalt will increase by about a factor of 10,000 when the loading time goes from an hour to two-hundredths of a second—about the rate of loading caused by traffic on highway. Again, this makes testing and specifying asphalt binders for paving applications complicated.



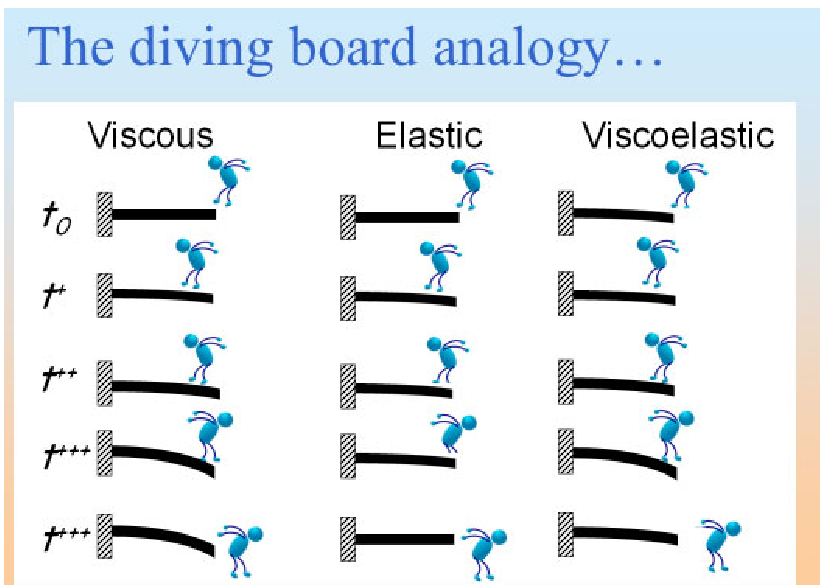
SLIDE 8

Modulus describes how much a material deflects or distorts under an applied load and is an important engineering property. For viscoelastic materials, the phase angle is also an important characteristic (Slide 9). The phase angle increases as the response of a material becomes more like a liquid. For purely elastic materials, the phase angle is zero, while for Newtonian liquids the phase angle is 90 degrees. Viscoelastic materials exhibit phase angles between these extremes. Typical paving grade asphalts have a phase angle of about 45 to 55 degrees at room temperature under rapid loading, but like modulus the phase angle of asphalt binders changes dramatically with temperature and rate of loading. Usually, the lower the temperature and the faster the loading rate, the lower the phase angle will be for a given asphalt binder. This means that at low temperatures and short loading times, asphalts tend to behave like elastic solids. At high temperatures and long loading times, the phase angle of asphalt binders is much higher, and their behavior often approaches that of a Newtonian fluid.

In understanding the difference between viscous, elastic and viscoelastic behavior it is useful to visualize how an imaginary diving board behaves when someone jumps on it and dives into a swimming pool (Slide 10). If the board is made of a purely viscous material, it will bend when the diver jumps on it, and after he dives into the pool the board will remain in the same bent position—it will not return at all to its original, horizontal configuration. If the board is made of an elastic material, after the diver launches himself off the board into the water the



SLIDE 9

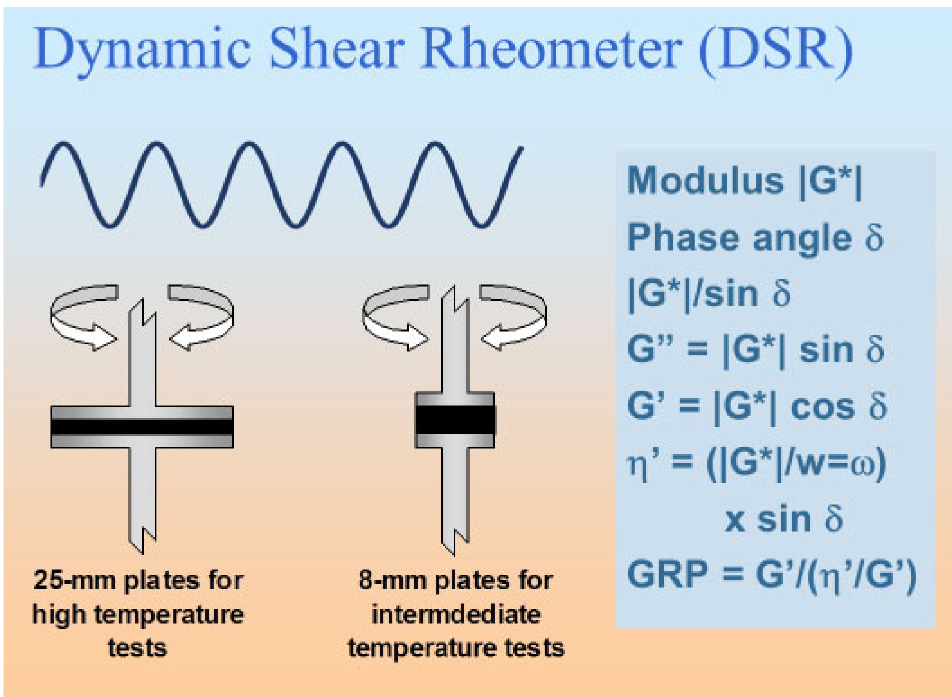


SLIDE 10

board will quickly and completely return to its original position. If the diving board is made of a viscoelastic material, after the diver jumps off of it the board will gradually return towards its original horizontal position, but only part of the way.

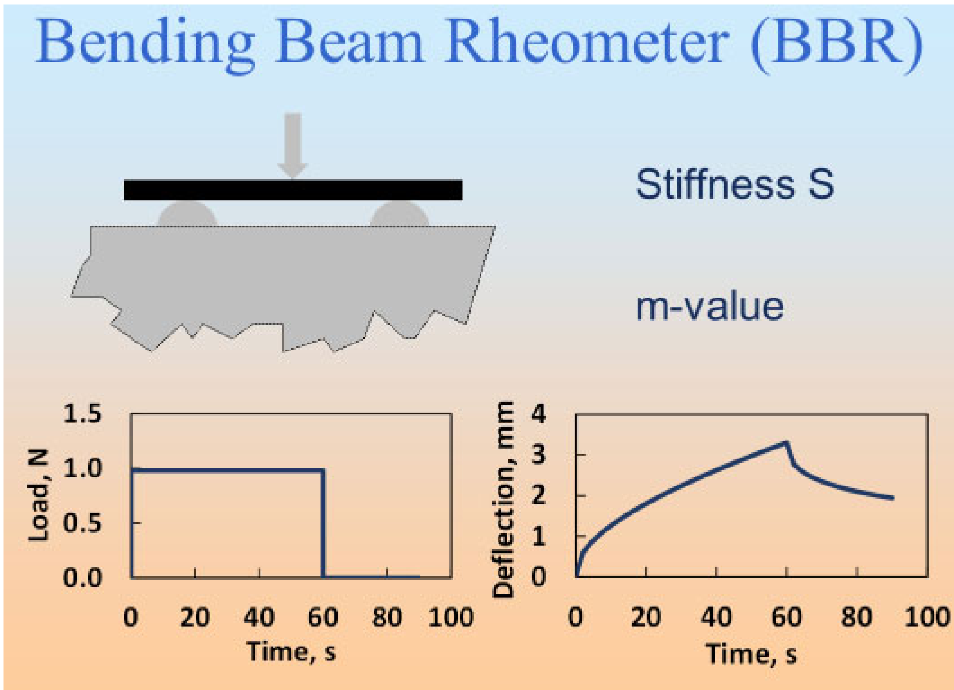
The dynamic shear rheometer (DSR), is used to measure the modulus and phase angle of paving grade binders at temperatures ranging from about 5°C to 70°C and higher (Slide 11). The DSR determines modulus by shearing an asphalt specimen between two circular plates and measuring the torque and deflection, which can then be used to calculate shear stress and strain.

The modulus measured with the DSR is called the dynamic shear modulus. The overall magnitude of the dynamic shear modulus is called the complex modulus |G*|, or “G-star.” Because asphalt is a viscoelastic material, it has not only a modulus but a phase angle. The modulus and phase angle together can be expressed as an in-phase and out-of-phase component of the modulus. The in-phase component of the modulus is called the storage modulus G′ (G prime), and is calculated by multiplying the complex modulus by the cosine of the phase angle δ (delta). The out-of-phase component of the modulus is called the loss modulus G″ (G double-prime), and is calculated by multiplying the complex modulus by the sine of the phase angle. Current specifications for asphalt binders include a minimum value of |G*|/sin δ at high temperatures to help prevent rutting in asphalt pavements. The specification also includes a maximum value of |G*| sin δ (the loss modulus G″) at intermediate temperatures, which helps prevent excessive fatigue cracking due to traffic loading. Other rheological parameters that can be calculated from DSR data include the storage viscosity η′ and the Glover–Rowe parameter (GRP), which a number of researchers have related to various aspects of pavement performance.



SLIDE 11

The bending beam rheometer (BBR) is used to measure the stiffness of asphalt binders at low temperatures, typically from about –30°C to –0°C, depending on the binder grade (Slide 12). At these low temperatures most asphalt binders are quite stiff and can be tested as if they were solid materials, although they will still slowly deflect under load. The BBR applies a load to a small beam of asphalt (about 100 mm or 4 in. long) for 1 min and measures the deflection in the beam during this time. From the applied load and measured deflection, stress and strain are calculated and used to calculate the creep stiffness of the asphalt at a 60-s loading time. In addition to creep stiffness, the m-value is also calculated; m is the log-log slope of stiffness with respect to time. Current specifications limit the creep stiffness S at 60 s to a maximum of 300 MPa, and limit the m-value to a minimum of 0.300. These limits help ensure that asphalt pavements won't crack because of excessive thermal stresses caused by extreme low temperatures during winter weather.



SLIDE 12

Recently some pavement engineers and researchers have suggested that premature pavement failures might be related to the parameter ΔT_c ("delta tee see") (Slide 13). This parameter is calculated from BBR tests data by subtracting the temperature at which m = 0.300 from the temperature where S = 300 MPa. The value of ΔT_c tends to be more negative for certain unusual asphalt binders, including ones that have been oxidized during the refining process. Aging, either in the pavement or using laboratory methods such as the pressure-aging vessel (PAV), will further decrease ΔT_c. Several examples of rapid, premature asphalt pavement failures have been linked to binders that have highly negative values of ΔT_c, especially after laboratory aging in the PAV.

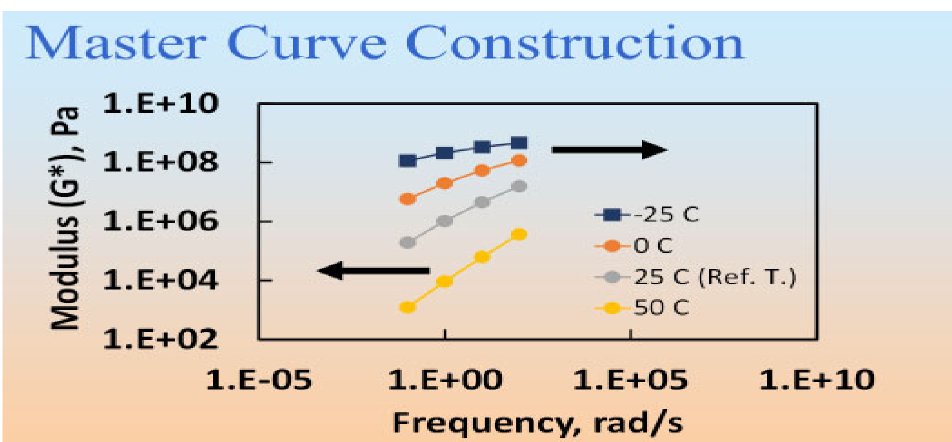
ΔT_c ("delta tee see")

- Difference between BBR critical temperature from stiffness S(t) and m-value m(t).
- Highly negative ΔT_c linked to poor performance
- ΔT_c more negative for heavily oxidized binders
- ΔT_c change with aging varies among asphalt binders

SLIDE 13

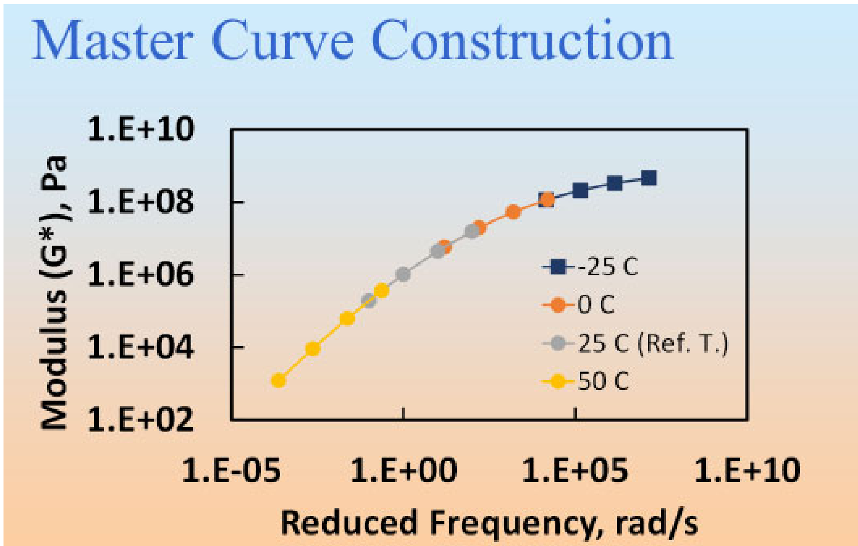
MASTER CURVE CONSTRUCTION AND RELATED CONCEPTS

A master curve for an asphalt binder is a method of analyzing DSR data (Slide 14). To construct a master curve, |G*| data from different temperatures are plotted as a function of frequency on a log-log scale. This plot shows data from four such frequency sweeps at temperatures of –25°C, 0°C, 25°C, and 50°C. To form a master curve, the frequency sweep data at each temperature are shifted horizontally to form a single curve showing |G*| as a function of temperature. One of the temperatures is selected as the reference temperature, which is not shifted but stays in its original location. In this example, 25°C is the reference temperature.



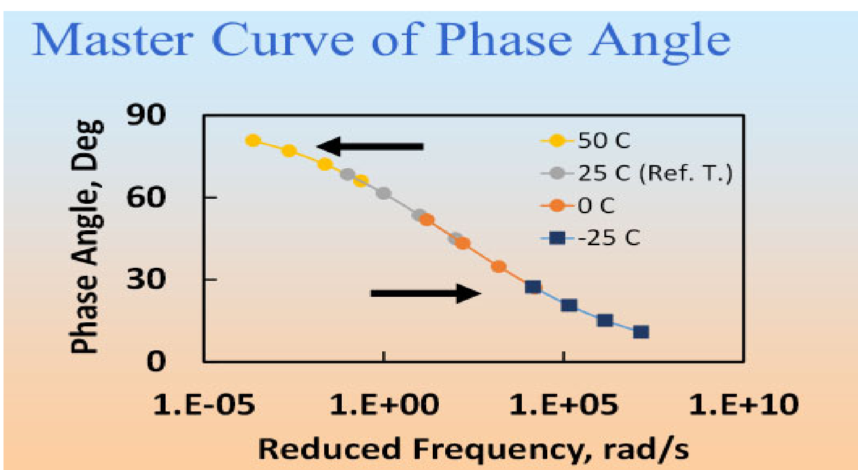
SLIDE 14

This plot in Slide 15 shows the completed master curve of |G*| from the previous set of data. In a properly constructed master curve, a single, smooth curve is formed without any noticeable "tails" or misfit data. Notice that the horizontal axis is labeled "reduced frequency," which simply means frequency that has been calculated through shifting of data to construct a master curve. Master curves typically cover many log decades of frequency and modulus, although caution should be used when estimating modulus data from a master curve at very high or very low frequencies.



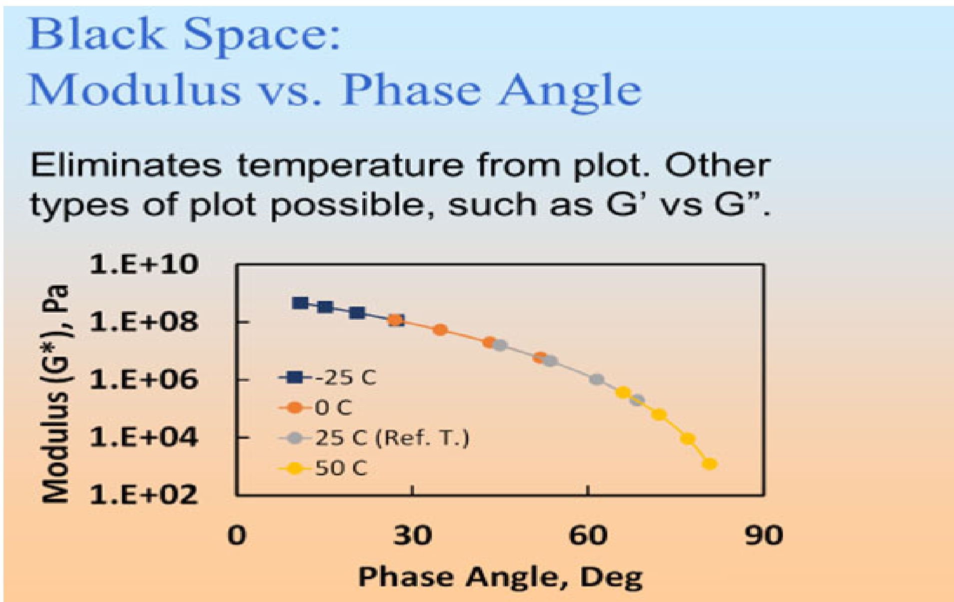
SLIDE 15

A master curve will also show phase angle as a function of reduced frequency (Slide 16). The same amount of shifting is used to construct master curves of modulus and phase angle. Some amount of shifting of modulus is often needed to construct good quality master curves. The amount of shifting required to form the master curve is called a shift factor; shift factors are often plotted as a function of temperature to show how the properties of an asphalt binder change with temperature.



SLIDE 16

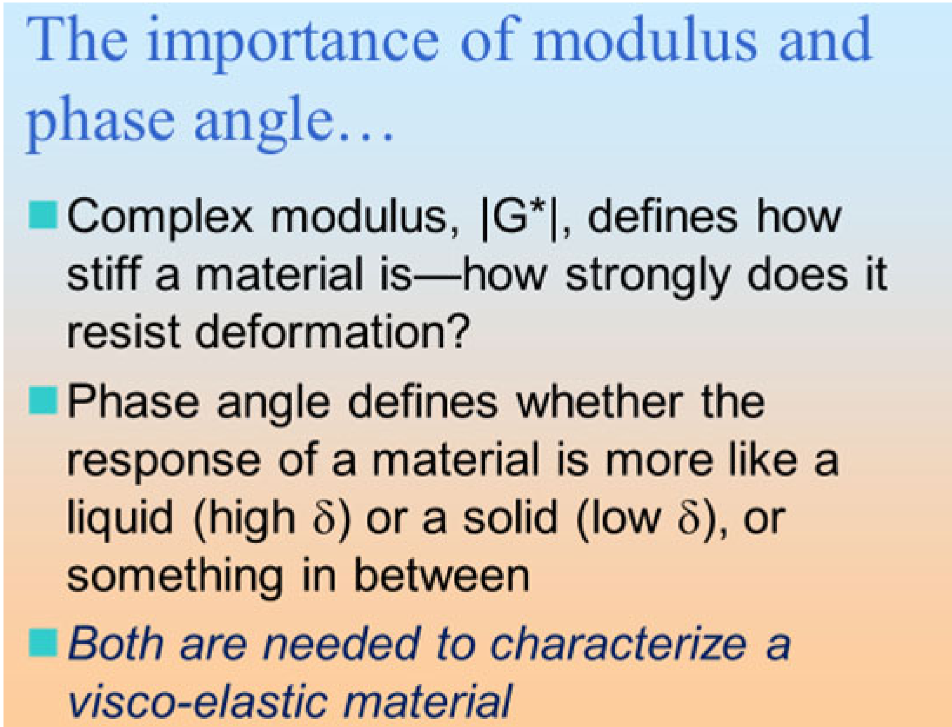
Sometimes when analyzing DSR data from asphalt binders a black space diagram is constructed; this provides a plot of modulus as a function of phase angle (Slide 17). A black space plot can help evaluate the quality of DSR data and also the nature of the asphalt binder. Normally, good quality DSR data of a nonmodified asphalt binder will plot as a single, smooth function in black space, although some slight vertical shifting of the data might be needed. If data at different temperatures do not form a single line, it can be because of poor quality DSR data, such as data generated using stresses and strains that are too high and outside of the region where stress and strain are proportional, or linear. Black space data that does not form a single line can also occur for some polymer modified binders, when the binder forms a strong network within the binder and exhibits temperature dependency different from that of the asphalt binder.



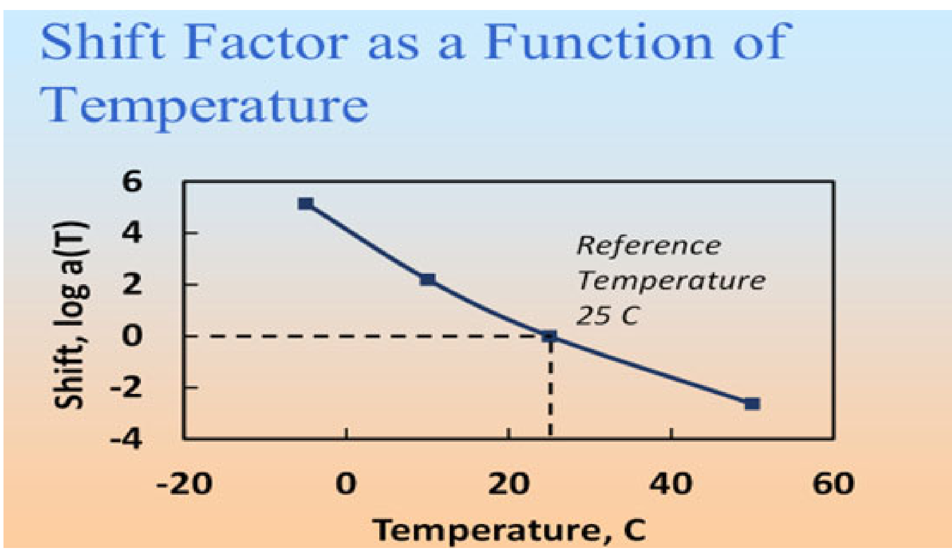
SLIDE 17

To thoroughly characterize viscoelastic materials like asphalt binder, measurements of both modulus and phase angle are needed (Slide 18). Modulus tells us how much the asphalt will deform under stress, while phase angle will tell us about the nature of that deformation—is it permanent, like a fluid? Or will the deformation recover like an elastic solid? Low phase angles indicate a material is behaving mostly like a solid, while high phase angles—close to 90 degrees—indicate a material is behaving like a simple fluid.

As mentioned a few slides ago, when a master curve is constructed from DSR data collected for an asphalt binder, the amount of shifting used in the analysis is often plotted as a function of temperature (Slide 19). This slide shows a plot of the log of the shift factor, log a(T), as a function of temperature for a typical asphalt binder. The shift factor at the reference temperature, in this case 25°C, is always one, so the value of log a(T) at the reference temperature is always zero. Plots of log a(T) versus temperature are an effective, fundamental way of characterizing how the mechanical properties of an asphalt binder change with temperature.



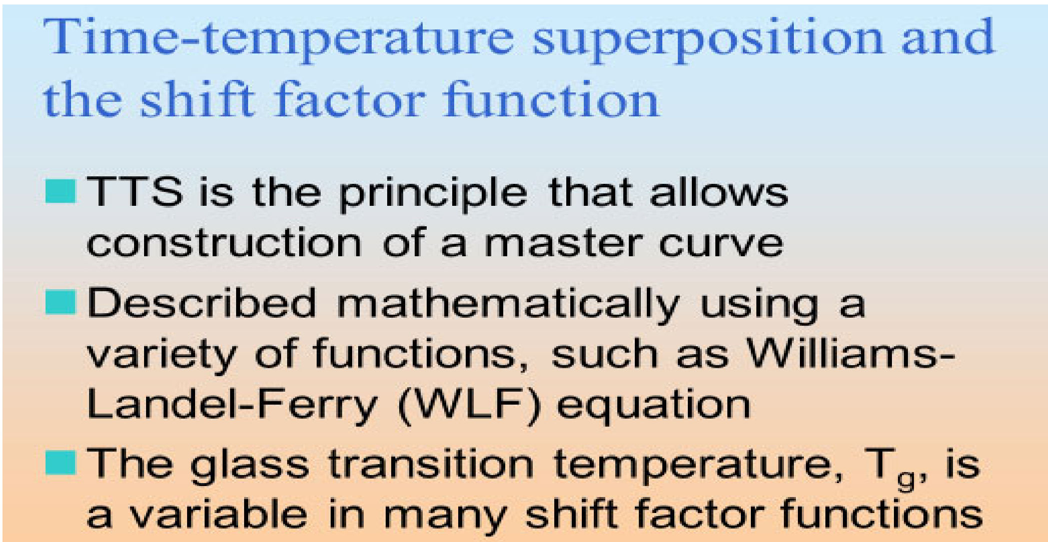
SLIDE 18



SLIDE 19

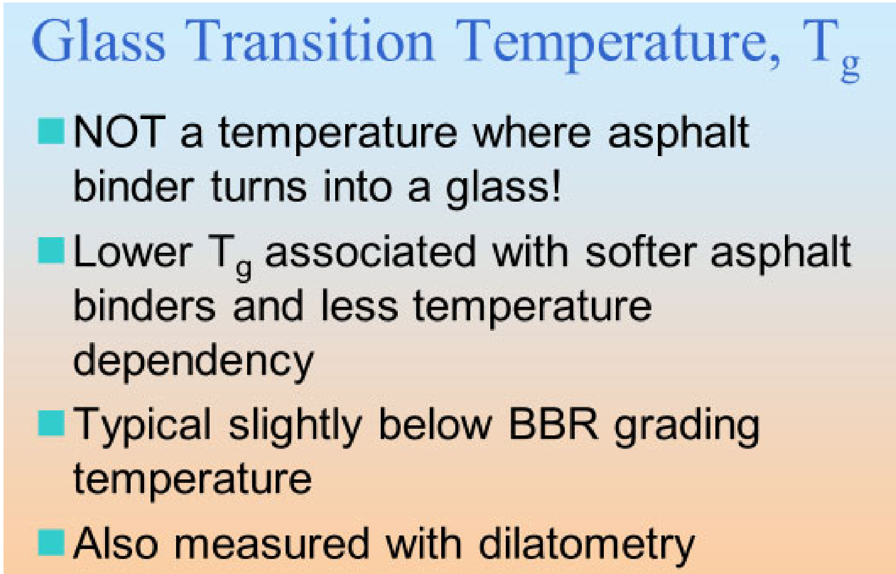
The theory underlying construction of master curves from DSR and similar data is called time–temperature superposition (Slide 20). In simple terms, this theory states that for linear viscoelastic materials under loading, the response under a specified loading frequency and temperature is equivalent to the response at some other lower frequency and lower temperature.

For example, the modulus of a given asphalt binder at 25°C and 10 rad/s might be the same at 10°C and a frequency of 0.1 rad/s. The change in shift factors with temperature is often modeled mathematically, using functions such as the Williams–Landel–Ferry (WLF) equation. The WLF equation and many other similar methods for characterizing log a(T) as a function of temperature often use the glass transition temperature T_g as an important predictor variable.



SLIDE 20

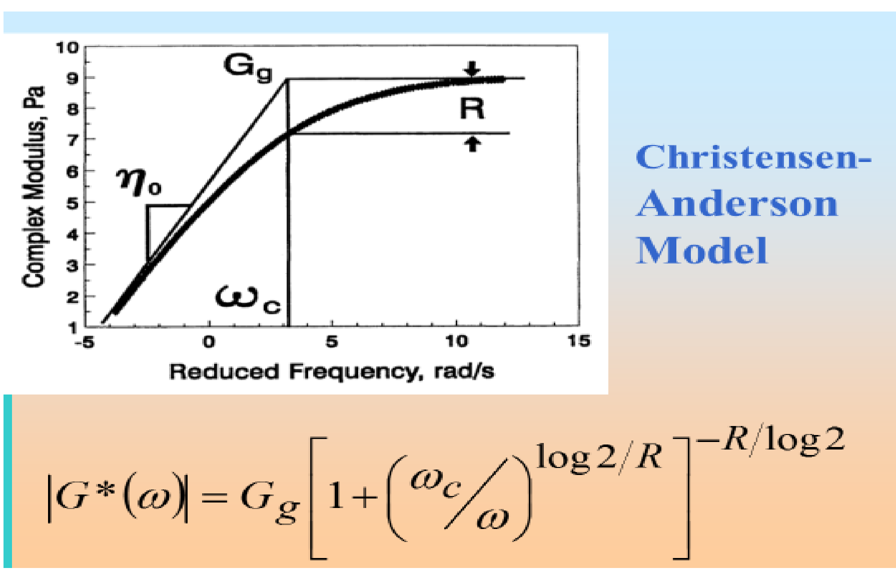
The glass transition temperature, sometimes called the defining temperature T_d for asphalt binders, is not the temperature where the asphalt turns to glass or becomes glasslike, although asphalt binders at or near T_g will generally be quite brittle (Slide 21). The glass transition temperature is a characteristic temperature related to the viscoelastic behavior of an asphalt binder. Usually lower values of T_g are associated with softer binders and less temperature dependency. Lower glass transition temperatures also suggest better resistance to low temperature cracking, but this is not always the case. The glass transition temperature for an asphalt binder is usually slightly below the critical BBR grading temperature. The value of T_g can be determined in a number of ways, including analysis of shift factor data, and careful measurement of the change in the volume of the asphalt binder with temperature, a technique called dilatometry.



SLIDE 21

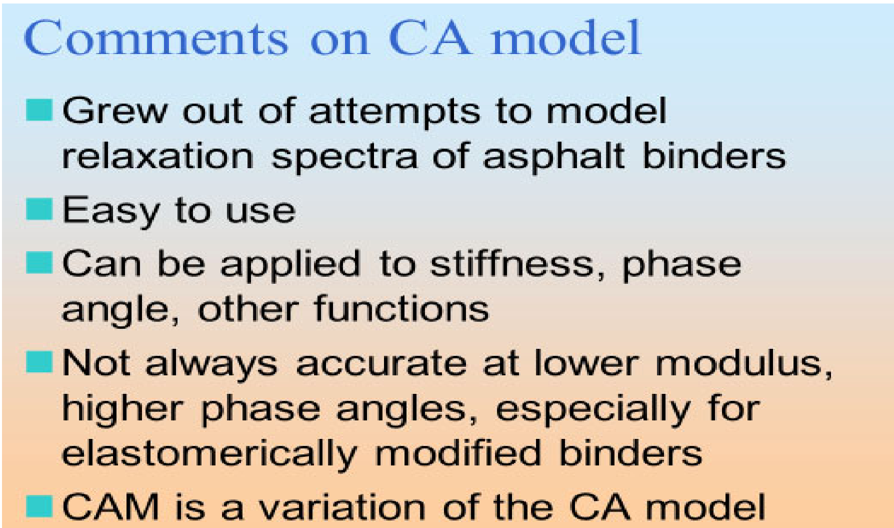
RHEOLOGICAL MODELS

Mathematical models are sometimes used to describe asphalt binder master curves—how the modulus and phase angle vary with frequency and temperature. One widely used model is the Christensen–Anderson (CA) model, shown in Slide 22. A variation of this model, called the Christensen–Anderson–Marasteanu (CAM) model, uses a slightly different equation that better fits data at lower frequencies and/or higher temperatures.



SLIDE 22

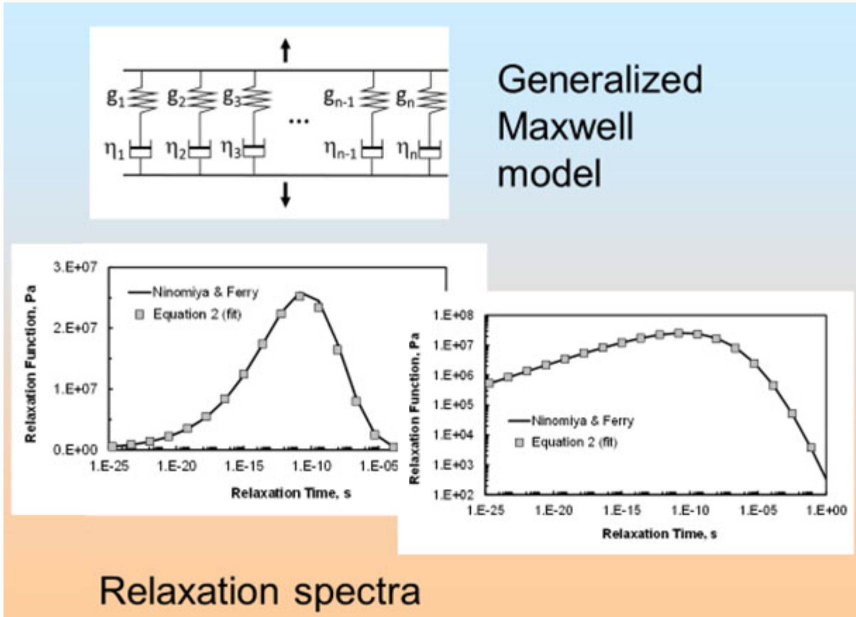
Although it might seem to be purely empirical, the CA model (and the CAM model) in fact grew out of efforts during the Strategic Highway Research Program (SHRP) to model the relaxation spectra of asphalt binders (Slide 23). Relaxation spectra are one of the most fundamental ways of characterizing the behavior of viscoleastic materials, and will be discussed in more detail later in this presentation. Recent research has shown direct relationships between the CA model and the properties of the relaxation spectra for a wide range of asphalt binders, demonstrating the theoretical underpinnings of the CA and CAM rheological models.



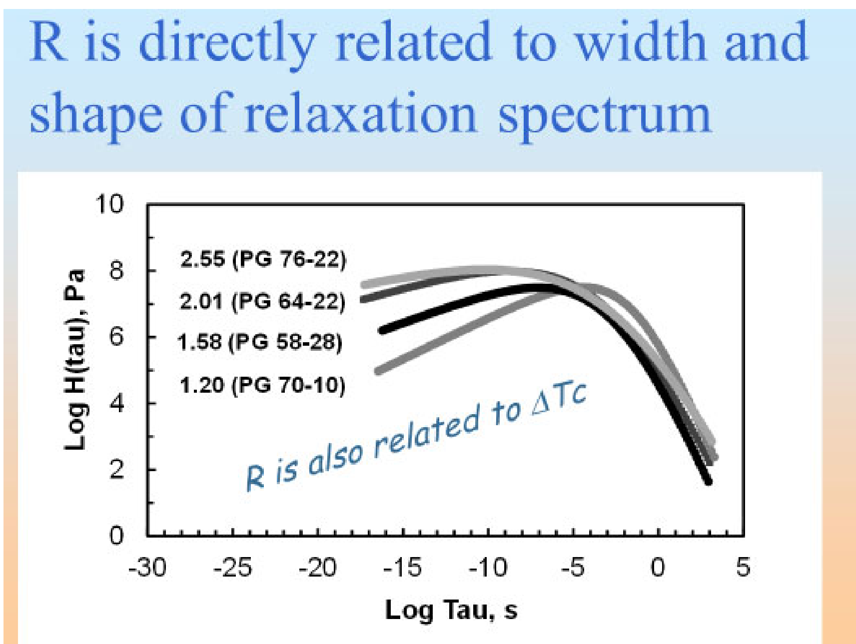
SLIDE 23

Another approach to modeling not just asphalt binders but any linear viscoelastic material is the generalized Maxwell model, which can be visualized as a number of springs and dashpots connected in parallel (Slide 24). Associated with each of these elements is a spring constant or modulus (g_i), a viscosity (η_i), and a relaxation time calculated by dividing the viscosity by the spring constant. The number of elements used in a Maxwell model varies from a few to several dozen or more. The Maxwell model for a given viscoelastic material can be described graphically by plotting the spring constant for each element against its relaxation time. This is the basis of the relaxation spectrum—it is a plot showing how the relaxation function [H(tau), spring constants in the Maxwell model] vary with relaxation time (tau). The two plots at the bottom of this slide show relaxation spectra for an asphalt binder; the one on the left is on a semi-log scale, while the one on the right is on a log-log scale.

The relaxation spectra of asphalt binders on a log-log scale have a characteristic shape, with the relaxation function H(tau) slowly increasing to a peak value and then more rapidly decreasing with increasing relaxation times tau (Slide 25). Both the width and the shape of the relaxation spectrum are directly related to the R-value of the CA model: the higher the R-value, the broader the relaxation spectrum and the more skewed it is towards shorter relaxation times. This plots shows relaxation spectra for four different asphalt binders. The relaxation spectra and R-value are also related to ΔT_c, mentioned earlier in this presentation. Higher R-values and



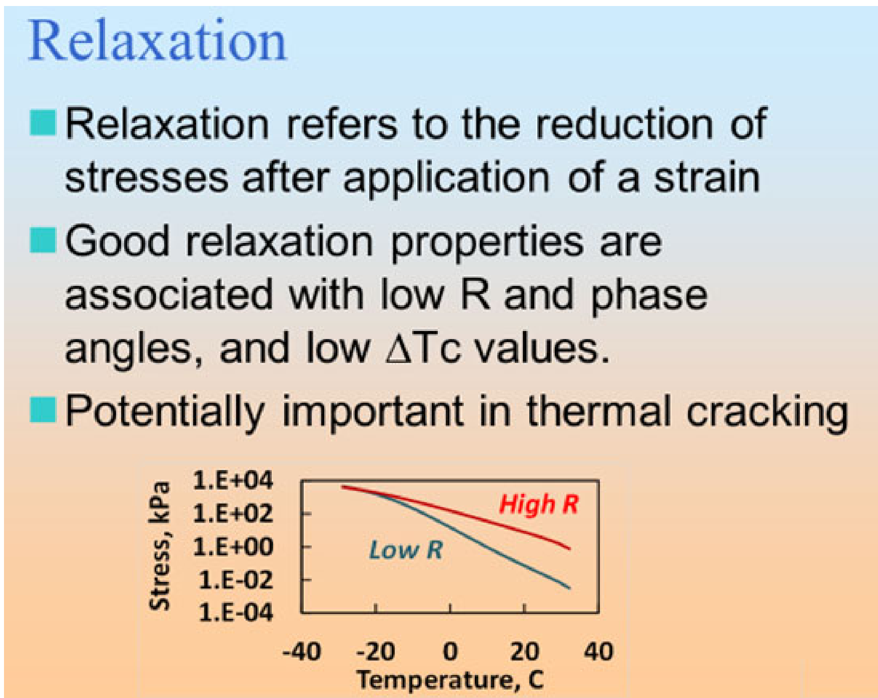
SLIDE 24



SLIDE 25

broader relaxation spectra are associated with more negative ΔT_c values. During long-term oxidative aging in pavements, and during laboratory aging in the PAV, the width of the relaxation spectrum generally increases substantially, while the R-value increases and ΔT_c becomes more negative.

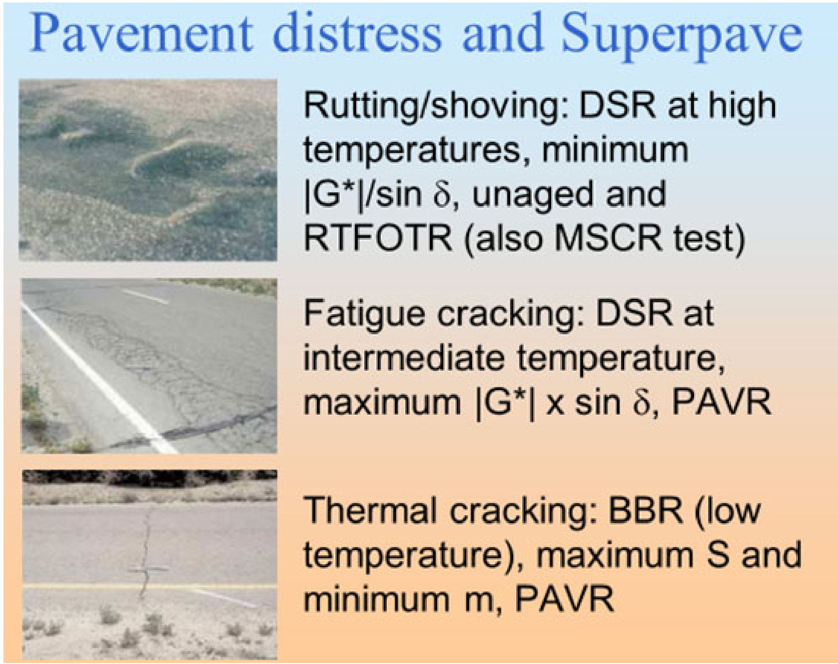
The term relaxation is used to describe how a viscoelastic material responds to being deformed or placed under strain; good relaxation properties (low absolute value of ΔT_c) mean that the stresses produced under strain rapidly decay (Slide 26). This helps prevent failure, for instance, during rapid cooling in winter weather which causes thermal contraction and tensile stresses in pavements. Asphalt binders with high R-values tend to have poor relaxation properties compared to binders with lower R-values. The R-values and related parameters for asphalt binders, such as ΔT_c are potentially related to other aspects of pavement performance, although researchers don't yet fully understand all of these relationships.



SLIDE 26

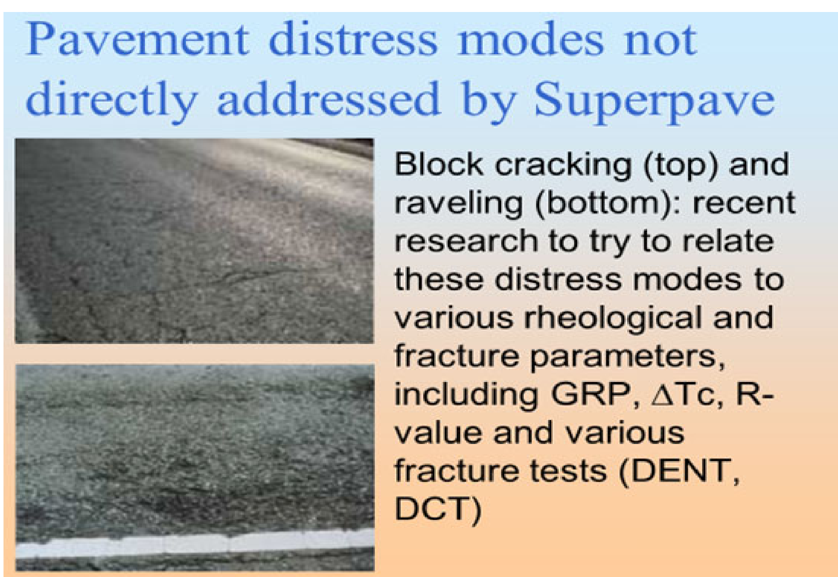
RHEOLOGY AND PAVEMENT DISTRESS

Current Superpave specifications for asphalt binders were designed to address various aspects of pavement performance (Slide 27); this is what the PG stands for in a binder grade—performance graded. Rutting and shoving in asphalt pavements at high temperature are addressed in DSR testing, with a minimum value of $|G^*|/\sin\delta$, both in an unaged condition and after aging in the rolling thin-film oven (RTFO). Fatigue cracking is also addressed with the DSR, with a maximum value for $|G^*| \sin\delta$ after aging in the RTFO followed by the PAV. Low-temperature or thermal cracking is controlled using the BBR test, with a maximum value for stiffness and a minimum value for m, again after RTFO and PAV aging.



SLIDE 27

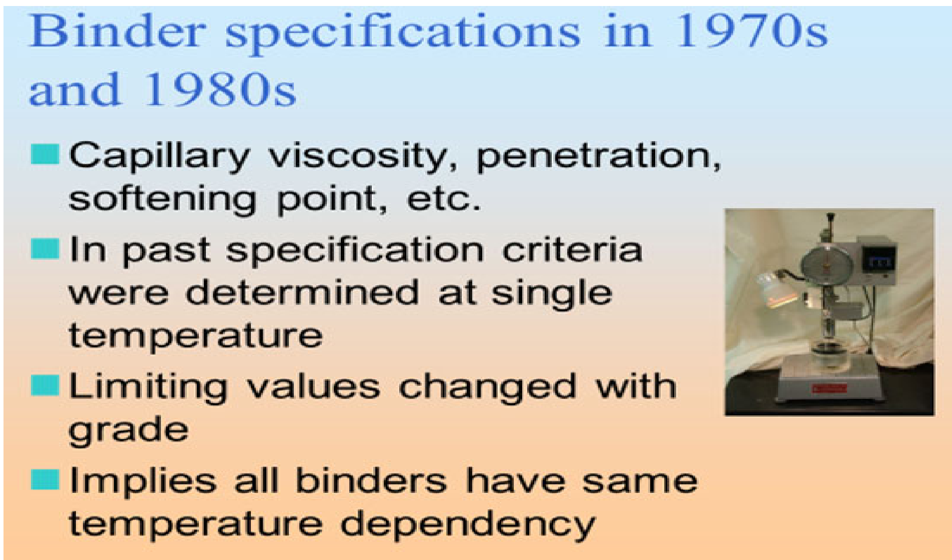
There are no specific tests in the Superpave system that are meant to address block cracking and raveling in asphalt pavements (Slide 28). Researchers are currently trying to determine what asphalt properties relate to these forms of distress. Properties and parameters that have been proposed as relating to these and other related forms of asphalt pavement distress include the GRP, ΔT_c, the R-value from the CA rheological model, and a variety of other binder tests, including the double-edged notch tension test.



SLIDE 28

CONCLUSIONS

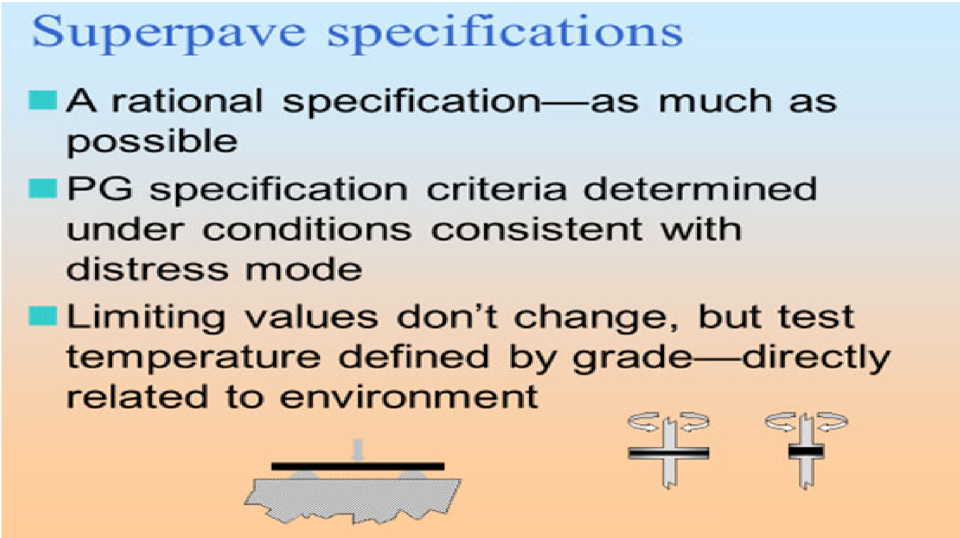
Asphalt binder specifications have gone through several big changes over the past 50 years. In the 1970s and 1980s, binders were specified using capillary viscosity, penetration (pen), softening point and ductility tests (Slide 29). All of these tests except for viscosity are empirical tests that don't directly provide information on engineering properties. Typically, a given test such as penetration was performed at a single temperature and upper and/or lower limits were given in a specification. These limits varied with the asphalt grade. For penetration grading, a 60/70 pen asphalt had a penetration at 25°C of between 60 and 70 dm (6 to 7 mm). A 120/150 pen asphalt had a penetration of between 120 and 150 cm (12 and 15 mm).



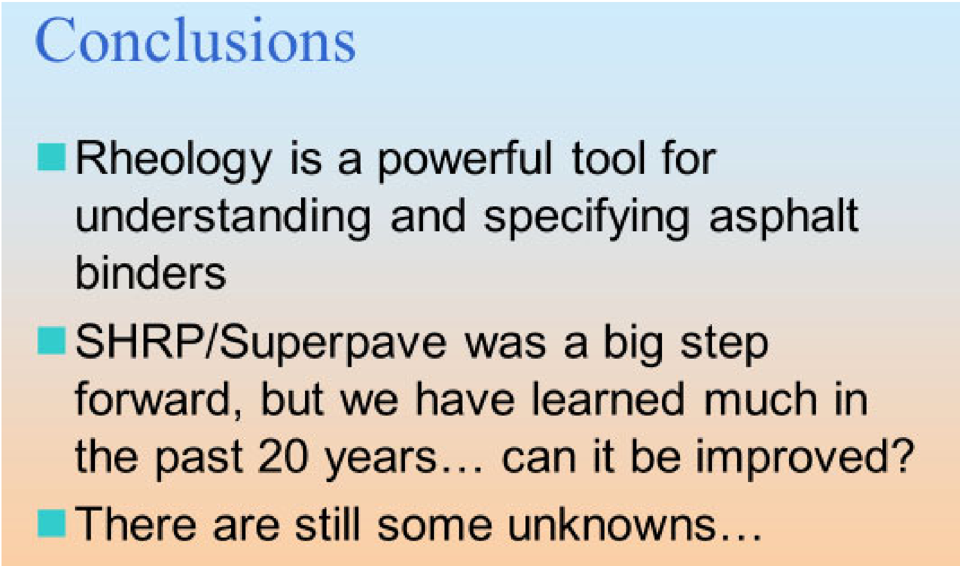
SLIDE 29

The Strategic Highway Research Program (SHRP) began in the late 1980s and continued through the mid-1990s, focused on developing more rational asphalt binder specifications that would provide information on engineering properties and would also relate to performance (Slide 30). Because the resulting Superpave standards were meant to relate directly to pavement performance, asphalt binders specified under this system were called performance graded. A PG 64-22 binder is a performance-graded asphalt binder, for which the maximum service temperatures are 64°C and –22°C, as determined using the DSR and BBR tests and the appropriate laboratory aging procedures.

Rheology—the study of how materials flow—is a very useful tool for measuring and specifying the performance-related properties of asphalt binders (Slide 31). The Superpave binder tests and specifications developed during SHRP were a significant improvement over the largely empirical tests and specifications used previously to control asphalt binder properties. It has been 20 years since the end of the SHRP program and the initial implementation of Superpave. During that time, the asphalt industry has changed significantly and pavement engineers and researchers have continued to improve their understanding of how asphalt binder properties effect pavement performance. There are a number of research efforts currently underway to determine if the existing Superpave binder specifications can be improved.



SLIDE 30

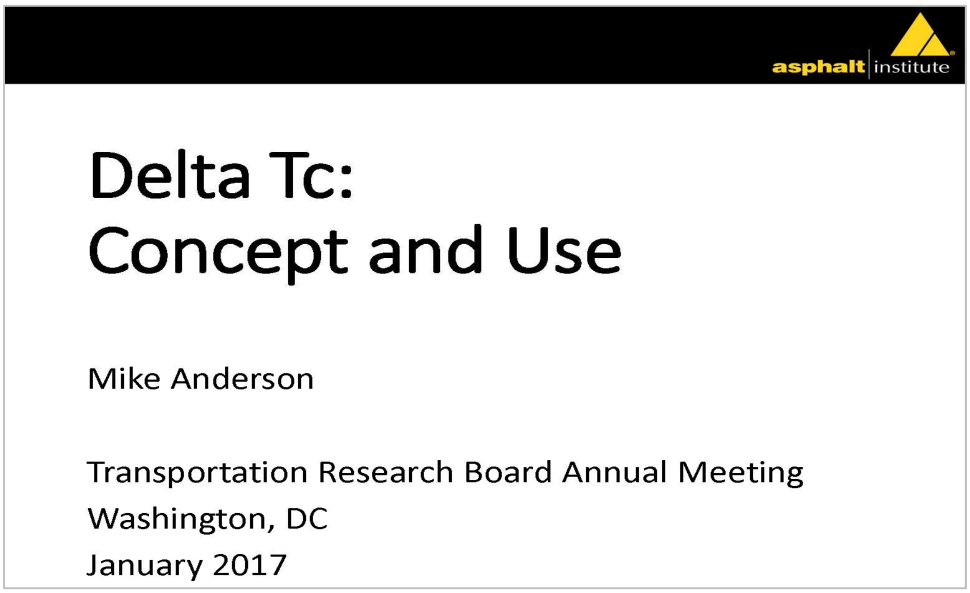


SLIDE 31

Delta T_c

Concept and Use

R. MICHAEL ANDERSON
Asphalt Institute



SLIDE 1

ACKNOWLEDGMENTS

The work presented was directly conducted in parts of several projects including, in order of earliest use (Slide 2 and Slide 3):

- Airfield Asphalt Pavement Technology Program (AAPTP) Project 06-01
- Federal Highway Administration Cooperative Agreement DTFH61-08-H-00030
- Pooled Fund Study, TPF-5(153), led by Minnesota DOT
- Federal Highway Administration Cooperative Agreement DTFH61-11-H-00033

This presentation represents the author's interpretation of results and associated opinions and does not necessarily represent the views or policies of the sponsors or panel of Project 06-01 of the Airfield Asphalt Pavement Technology Program, U.S. Department of Transportation, Minnesota Department of Transportation, the Asphalt Institute or its individual Member Companies.

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products or manufacturers. Any trade or manufacturers' names that appear herein do so solely because they are considered essential to this presentation.

SLIDE 2

SLIDE 3

AAPTP 06-01 RESEARCH OBJECTIVES

- Objectives
  - Develop a practical guide identifying means to prevent and mitigate cracking caused by environmental effects.
  - Develop one or more test procedures that could be used by a pavement manager to determine when preventative maintenance is needed to prevent the development of cracking (specifically block cracking).

SLIDE 4

TPF-5(153) RESEARCH OBJECTIVES

- Primary Objective
  - to develop and validate technology that can be used by highway agencies to determine the proper timing of preventive maintenance in order to mitigate damage caused by asphalt aging

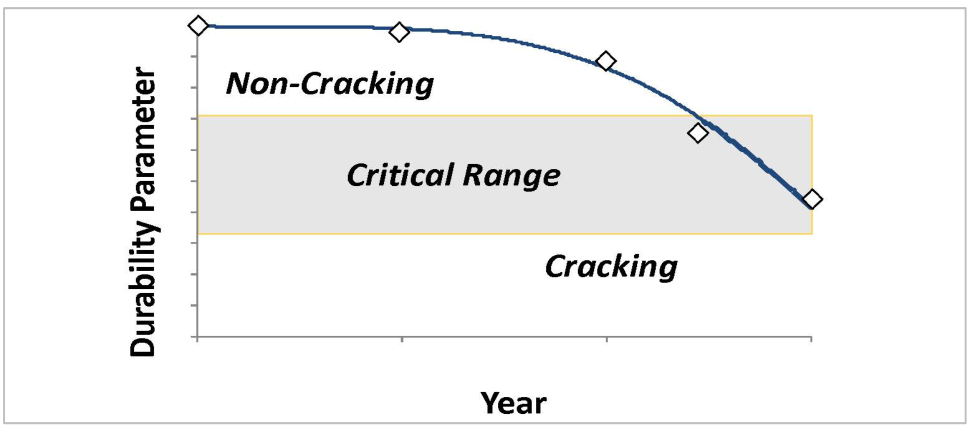
SLIDE 5

The author's first experience with using Delta T_c (ΔT_c) as an indicator of aging was in the AAPTP 06-01 project (Slide 4). Research team member Gayle King (consultant) suggested examining asphalt binders with different low temperature properties—specifically testing asphalt binders where the BBR stiffness at 60 s controlled the low-temperature grade (referred to as S-controlled asphalt binders) and asphalt binders where the BBR m-value at 60 s controlled the low-temperature grade (referred to as m-controlled asphalt binders). The objectives of the AAPTP 06-01 project were similar to the TPF-5(153) study—to determine when preventive maintenance might be needed to inhibit the development of cracking, specifically the type of cracking that is not load-related but is a function of the environment and the asphalt pavement properties (Slide 5). The asphalt binder is affected by both temperature and oxidation and will change its properties (becoming stiffer with less relaxation) as it ages in-service.

CONCEPT

General Concept

Slide 6 shows a general concept graphic used to indicate pavement deterioration with time. Preventive maintenance suggests that one would want to apply a treatment to restore the durability of the asphalt pavement at an appropriate time before the onset of damage. The concept illustrated is that in an ideal case there would be a "durability parameter" that could easily be measured and would relate to cracking at intermediate temperatures resulting from environmental factors (i.e., oxidation). The subsequent slides discuss the effects that environmental factors, specifically oxidation during aging, has on asphalt binder stiffness and loss of flexibility (Slide 7 and Slide 8). If one or more parameters can be identified relating to the effects of aging, then it might be possible to monitor when an asphalt pavement is reaching a critical state so that action can be taken.



SLIDE 6

- In-service aging leads to oxidation and loss of flexibility at intermediate and low temperatures
  - Block-cracking
    - when environmental (non-load) conditions create thermal stresses that cause strain in the asphalt mixture that exceeds the failure strain

SLIDE 7

- In-service aging leads to oxidation and loss of flexibility at intermediate and low temperatures
  - Preventing or mitigating distress
    - identify a property of the asphalt binder or mixture that sufficiently correlates with its flexibility
    - provide a procedure to monitor when flexibility reaches a state where corrective action is needed

SLIDE 8

ASPHALT DURABILITY

J. Claine Petersen discussed what constituted a "durable" asphalt in a paper in 1984 (Slide 9). The complete reference is: Petersen, J. C. Chemical Composition of Asphalt as Related to Asphalt Durability: State of the Art. *Transportation Research Record 999*, 1984, pp. 13–30.

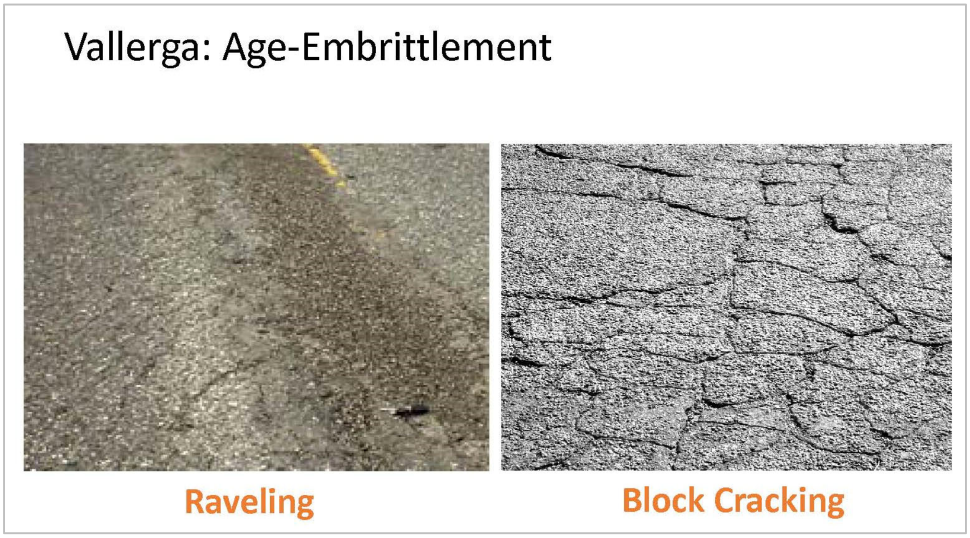
- A durable asphalt:
  - has physical properties necessary for desired initial product performance, and
  - is resistant to change in physical properties during long-term, in-use environmental aging

Petersen, J.C., "Chemical Composition of Asphalt as Related to Asphalt Durability-State of-the-Art", TRR. 999, 1984

SLIDE 9

ASPHALT OXIDATION

Barney Vallerga, former Asphalt Institute Engineer, is credited with the two photographs in Slide 10 showing raveling and block cracking as a result of age-embrittlement.



SLIDE 10

DUCTILITY AND DSR PARAMETER

Although the concept for ΔT_c was advanced during the AAPTP 06-01 project, the research team built upon the work by Charles Glover and associates at Texas A&M University in the early 2000s, who built upon the work by Prithvi (Ken) Kandhal in 1977 (Slide 11). Kandhal's work looked at block cracking in asphalt pavements and related it to asphalt binder ductility at 15°C. His findings suggested that cracking worsened as the ductility decreased below 10 to 5 cm (surface cracking observed) to 3 cm or less (serious surface cracking observed). Glover et al built upon Kandhal's hypothesis and looked for a rheological parameter that was related to ductility (Slide 12 and Slide 13). Their finding was that ductility at 15°C and a displacement rate of 1 cm/min correlated well with a DSR parameter at the same temperature and a loading frequency of 0.005 rad/s when ductility was 10 cm or less. Note that the loading frequency is very slow relative to standard DSR testing, which is conducted at 10 rad/s. If a standard test was conducted at a frequency of 0.005 rad/s instead of 10 rad/s it would take approximately 3.5 h to complete. As such, Glover's team—and the AAPTP 06-01 research—initially focused on performing temperature–frequency sweep tests to generate master curves at a reference temperature of 15°C from which the 0.005 rad/s data could be determined.

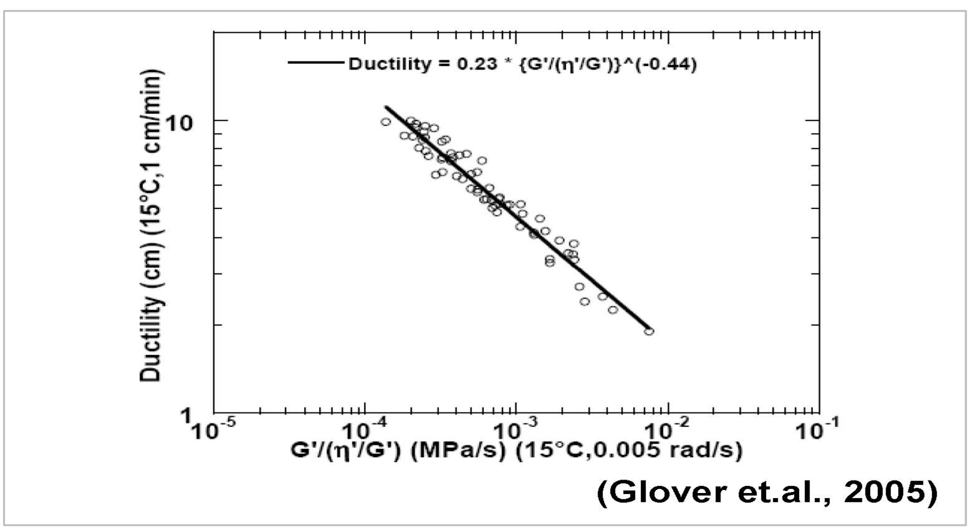
- Physical Changes – Ductility
  - Block cracking severity related to ductility at 60ºF (15ºC) – Kandhal (1977)
    - "Low-Temperature Ductility in Relation to Pavement Performance", ASTM STP 628, 1977
  - Loss of surface fines as ductility = 10 cm
  - Surface cracking when ductility = 5 cm
  - Serious surface cracking when ductility < 3 cm

SLIDE 11

RECENT AGING RESEARCH.

- Texas A&M Research (Glover, et.al.)
  - 2005
  - "Development of a New Method for Assessing Asphalt Binder Durability with Field Evaluation"
  - Build on work by Kandhal suggesting block cracking and raveling is related to low binder ductility after aging
  - Identified rheological parameter related to ductility

SLIDE 12



SLIDE 13

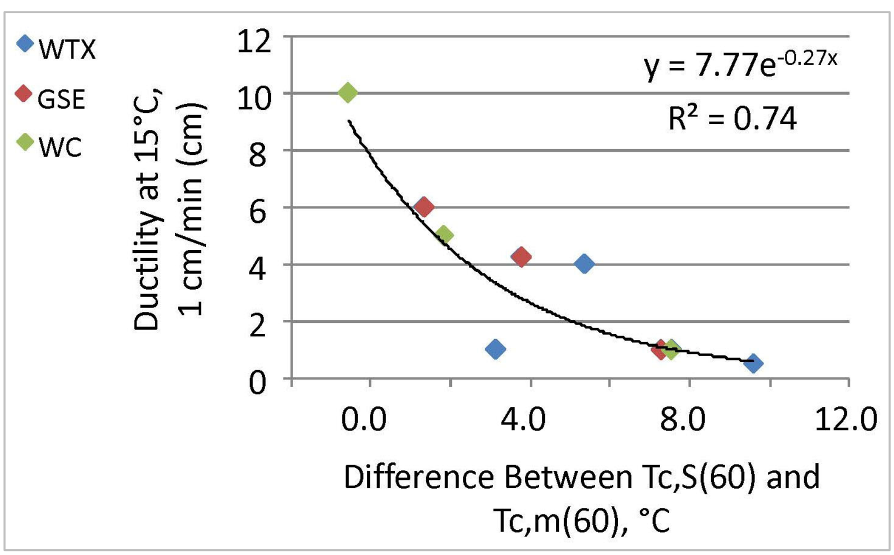
Sources

Kandhal, P. S. Low-Temperature Ductility in Relation to Pavement Performance. *ASTM STP 628: Low-Temperature Properties of Bituminous Materials and Compacted Bituminous Paving Mixtures* (C. R. Marek, ed.), American Society for Testing and Materials, Philadelphia, Pa., 1977.

Glover, C. J., R. R. Davison, C. H. Domke, Y. Ruan, P. Juristyarini, D. B. Knorr, and S. H. Jung. Development of a New Method for Assessing Asphalt Binder Durability with Field Evaluation. Report # FHWA/TX-05/1872-2. Federal Highway Administration and Texas Department of Transportation, 2005.

RELATIONSHIP BETWEEN ΔT_c AND DUCTILITY

As noted earlier, the AAPTP 06-01 research team focused on the aging of three asphalt binder sources with different low-temperature properties (not just different low-temperature grades) (Slide 14). Aging was conducted to simulate plant mix aging and aging in-service. The RTFO test (AASHTO T240) was used to simulate the aged properties of the asphalt binder as it was initially used in an asphalt mixture and placed on the road. The PAV procedure (AASHTO R28) was used to simulate the standard long-term aging of an asphalt binder in-service. To simulate even more aging, the PAV procedure was conducted for twice as long (40 h) and four times as long (80 h) under the same temperature and pressure conditions. The four conditions were designated as PAV0, PAV20 (standard), PAV40, and PAV80. The graph from the report shows a comparison of ductility and for the PAV20, PAV40, and PAV80 conditions for the three asphalt binders. As can be seen, as ductility decreases, the absolute value of the difference between $T_{c,s}$ and $T_{c,m}$ (ΔT_c) increases. It is important to note that the values are shown as positive numbers although current convention now is to show the direction of the difference, with negative values indicating more aging and lower ductility values. This graph is also important because it can be used to make the connection between the Glover rheological parameter, $G'/(\eta'/G')$, and ΔT_c.



SLIDE 14

Hanson D.I., P.B. Blankenship, G.N. King, and R.M. Anderson. “Techniques for Prevention and Remediation of Non-Load-Related Distresses on HMA Airport Pavements – Phase II”, Final Report, Airfield Asphalt Pavement Technology Program, Project 06-01, December 2010.
Anderson R.M., G.N. King, D.I. Hanson, and P.B. Blankenship. “Evaluation of the Relationship between Asphalt Binder Properties and Non-Load Related Cracking”, Asphalt Paving Technology, Volume 80, Association of Asphalt Paving Technologists, 2011.

WHAT IS DELTA T_c?

Slides 15 through 17 discuss what ΔT_c is and what it is expected to represent. In asphalt binder testing one can determine a “true grade” or “continuous grade” by calculating the temperature where the specification value is exactly met for any parameter. The procedure for doing this calculation is discussed later. When testing at low temperature using the BBR, two values can be determined by calculating the temperature where the stiffness (S) meets 300 MPa and the temperature where the m-value (m) meets 0.300. In both cases, S and m-value are determined after 60 s of loading in accordance with AASHTO T313 and M320. The temperatures where those specification values are met are referred to as critical temperatures (T_c) and are represented by $T_{c,S}$ for stiffness and $T_{c,m}$ for m-value (Slide 15).

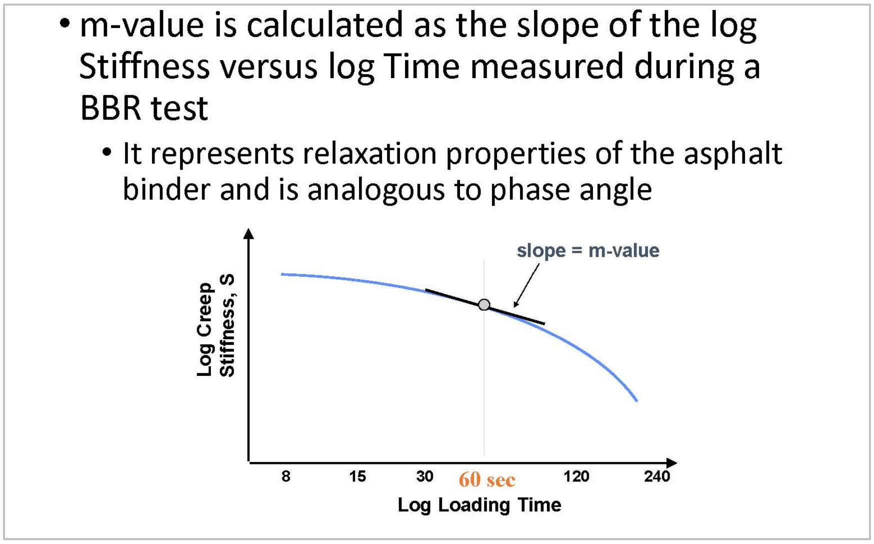
When considering what ΔT_c represents, one has to recall what the components represent. Stiffness is self-evident. The m-value is the slope of the stiffness versus time curve represented on a logarithmic scale. It represents relaxation properties (Slide 16); in other words, how quickly does stiffness change in response to loading? At low temperatures, rapid change (high m-value) is better than slow change (low m-value). It is analogous to phase angle, with lower m-value corresponding to lower phase angle or more elastic solid behavior.

As an asphalt binder ages, stiffness increases and m-value decreases, but the rate at which each change may not be the same (Slide 17). A similar response can be seen at intermediate temperatures for G* (increases with aging) and phase angle, δ, (decreases with aging). What is desirable is that there is some balance between the change in stiffness and the proportion of viscous and elastic properties.

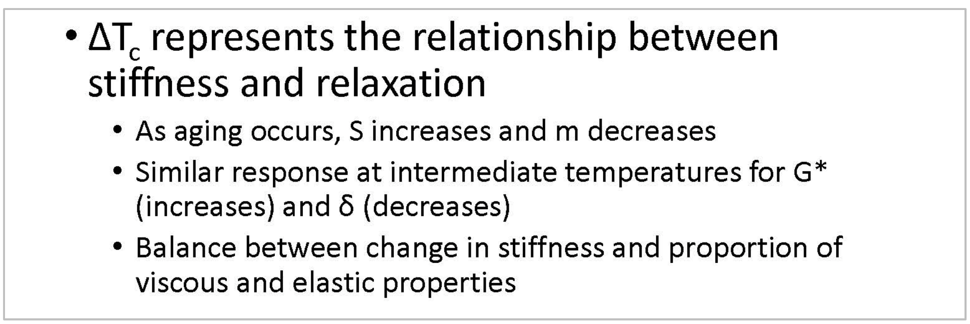
- Delta Tc (ΔT_c) is the difference between the critical low temperatures of the asphalt binder, determined using the Bending Beam Rheometer (BBR), where the stiffness (S) at 60 seconds of loading time is exactly equal to the specification value of 300 MPa and the m-value (m) at 60 seconds of loading time is exactly equal to the specification value of 0.300.

SLIDE 15

WHAT DOES ΔT_c REPRESENT?



SLIDE 16



SLIDE 17

DETERMINING ΔT_c

Slides 18 through 24 describe the mechanics of determining ΔT_c and guidelines for best practices. The steps should be self-explanatory, but are repeated as follows:

1. Perform the BBR on an asphalt binder sample at an appropriate low temperature (determined based on grade or experience). The sample can be an asphalt binder that is aged to any desired condition or a recovered asphalt binder sample from an asphalt mixture.
2. Based on the results for S and m-value from the first temperature, choose a second test temperature (usually six degrees warmer or colder) and perform the BBR test on the sample asphalt binder sample (though not the same specimen).
3. Using S and m-values at two temperatures calculate the critical temperatures for S ($T_{c,S}$) and m ($T_{c,m}$). Don't forget that test temperatures are in AASHTO M320 are warmer than the grade temperature, thus causing us to subtract 10 degrees at the end of the equation.

- First Step
  - After aging the asphalt binder, run the BBR test at the appropriate temperature for the grade as defined in AASHTO M320 (or at an appropriate starting temperature if the grade is unknown). Designate this temperature as T_1. Determine the stiffness (S) and m-value (m) at 60 seconds of loading time. Designate these values as S_1 and m_1

SLIDE 18

- Second Step
  - If $S_1 < 300$ MPa, choose the second test temperature (T_2) to provide a stiffness value that will be ≥ 300 MPa. If S_1 is ≥ 300 MPa, choose T_2 to provide a stiffness value that will be < 300 MPa.
  - Determine the stiffness (S) and m-value (m) at 60 seconds of loading time. Designate these values as S_2 and m_2.

SLIDE 19

- Third Step
  - Calculate the values of $T_{c,S}$ and $T_{c,m}$ as follows:

$$T_{c,S} = T_1 + \left(\frac{(T_1 - T_2) * (\text{Log } 300 - \text{Log } S_1)}{\text{Log } S_1 - \text{Log } S_2}\right) - 10$$

$$T_{c,m} = T_1 + \left(\frac{(T_1 - T_2) * (0.300 - m_1)}{m_1 - m_2}\right) - 10$$

SLIDE 20

- Fourth Step
  - Calculate the value of ΔT_c :

    $\Delta T_c = T_{c,S} - T_{c,m}$
- Positive values of ΔT_c indicate an S-controlled asphalt binder
- Negative values of ΔT_c indicate an m-controlled asphalt binder

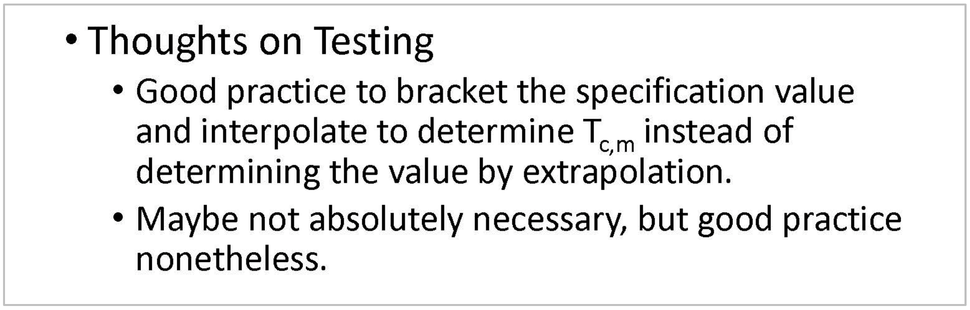
SLIDE 21

- Thoughts on Testing
  - For many asphalt binders, the selection of two temperatures based on S values will be sufficient to also provide m-values that bracket the specification value of 0.300.

SLIDE 22

- Thoughts on Testing
  - If the m-values are on the same side of the specification value (i.e., both higher or both lower) then the proper procedure is to run another test at a different temperature (warmer or colder depending on the measured values) and determine the m-value (m_3) at that temperature (T_3).
  - To calculate $T_{c,m}$ substitute m_3 and T_3 in place of either of the other paired terms (T_1 and m_1, or T_2 and m_2).

SLIDE 23



SLIDE 24

4. Calculate ΔT_c by subtracting $T_{c,m}$ from $T_{c,S}$. The resulting number may be 0, but will usually be either positive (an S-controlled asphalt binder) or negative (an m-controlled asphalt binder).

Note that the calculations for $T_{c,S}$ and $T_{c,m}$ that are used are shown in several sources, including the following:

- AASHTO M323-13: Standard Specification for Superpave Volumetric Design, American Association of State Highway and Transportation Officials, 2016.
- MS-26. *The Asphalt Binder Handbook*, Asphalt Institute, Lexington, Ky., 2011.

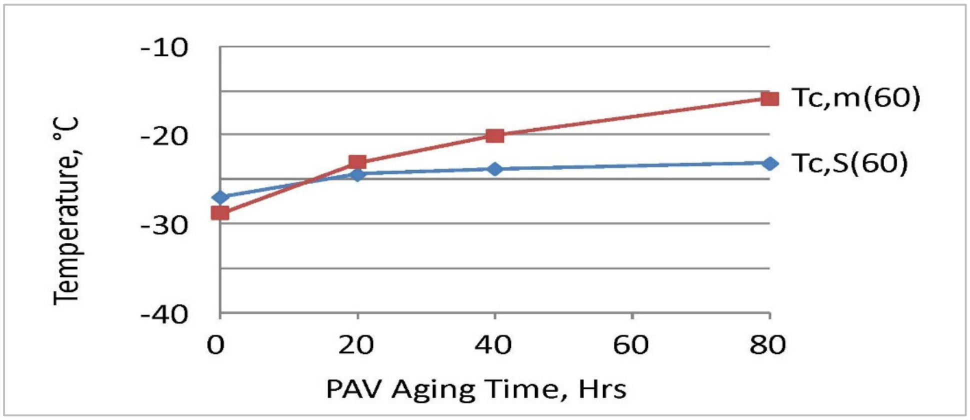
Although only two temperatures are needed to calculate ΔT_c the best practice is to select the two temperatures so that they result in S and m-values that bracket the specification values of 300 and 0.300 MPa (i.e., temperatures that generate both passing and failing results). If an asphalt binder is "balanced" in S and m, then this should be possible. The more S-controlled or m-controlled the asphalt binder is, the more likely it is that when testing at the two selected temperatures at least one of the parameters (S or m) will have values that are either both passing or both failing. Although it is possible to calculate T_c from two temperatures with values that either both pass or both fail, it is a better approach to interpolate whenever possible. In that case, it is good practice to perform testing at a third temperature. It should be noted that this recommended practice may not be possible with very m-controlled asphalt binders because the Stiffness will be too low to be reliably determined using AASHTO T313 at the temperature when the m-value will pass the criterion of 0.300. In that case, extrapolation may be the preferred alternative.

BBR: GULF–SOUTHEAST

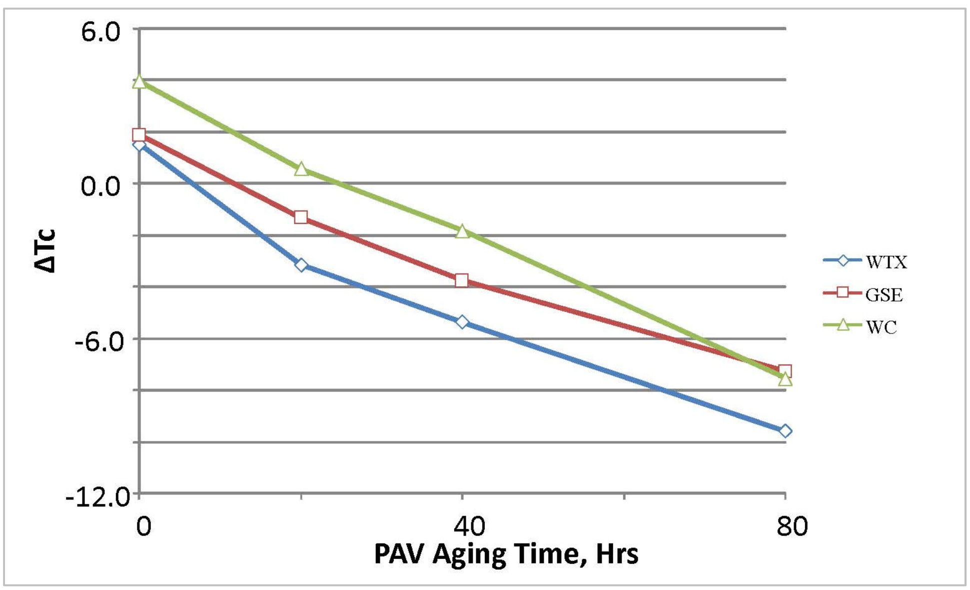
ΔT_c as an Indicator of Oxidative Aging

What does ΔT_c represent? ΔT_c measured at several aging conditions can be an indicator of oxidative aging. In Slide 25, the Gulf Southeastern asphalt binder used in the AAPTP 06-01 study is shown at various aging times. Note that as aging progresses, both the $T_{c,S}$ and $T_{c,m}$ increase (get warmer) as expected, but the rate at which they increase is not the same. The gap between the two curves is ΔT_c. Slide 26 shows the value of ΔT_c as a function of PAV aging time.

The progression is approximately linear for the three asphalt binders used in the AAPTP 06-01 study showing lower (more negative) values of ΔT_c as aging increases.



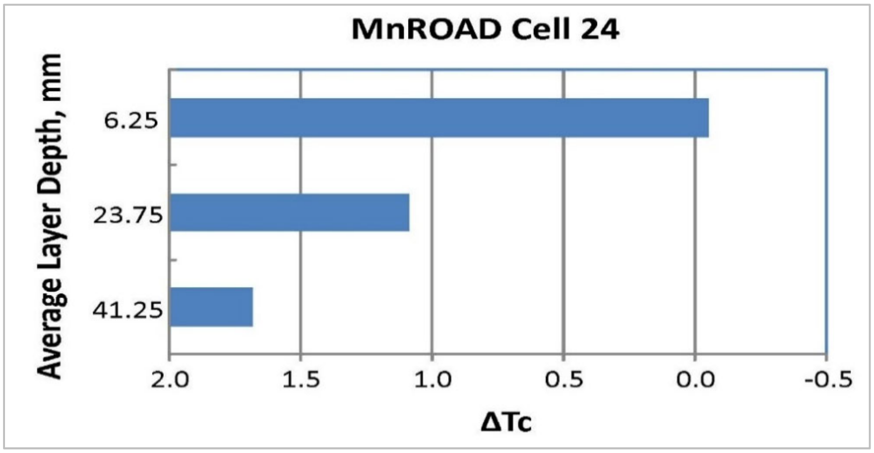
SLIDE 25



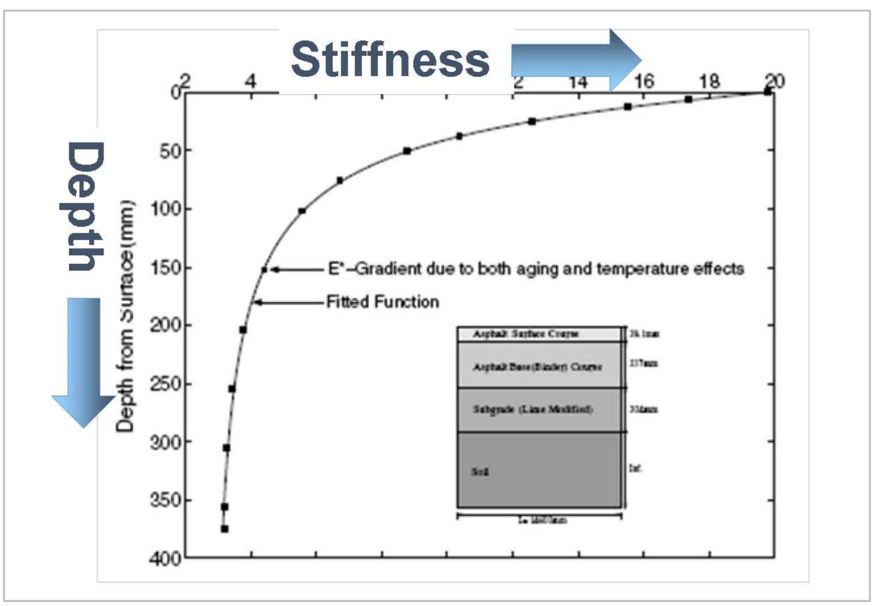
SLIDE 26

In Pooled Fund Study TPF-5(153): Optimal Timing of Preventive Maintenance for Addressing Environmental Aging in Hot-Mix Asphalt Pavements, cores were taken from Cell 24 of the Low-Volume Road section of MnROAD (a pavement test track owned by MnDOT), a section designated to study the optimal timing for preventive maintenance due to aging. Cores were cut into 12.5-mm thick layers starting at the top (surface) and progressing downward approximately 50 mm into the core. Each layer was then subjected to solvent extraction followed by recovery of the asphalt binder. Various physical properties of the recovered asphalt binder were determined including ΔT_c. Slide 27 shows ΔT_c data for cores taken from a section that was

2 years old and was sealed immediately after construction. As shown in the slide, ΔT_c values were lowest near the surface and were progressively higher as the layer depth increased. This matches expectations for aging where the asphalt mixture closest to the surface will be more aged than asphalt mix that is deeper in the pavement structure. This behavior mimics the behavior suggested by Witczak and Mirza in their proposed global aging model for asphalt mixtures where stiffness is highest at the surface and decreases significantly as the depth from the surface increases (Slide 28).



SLIDE 27



SLIDE 28

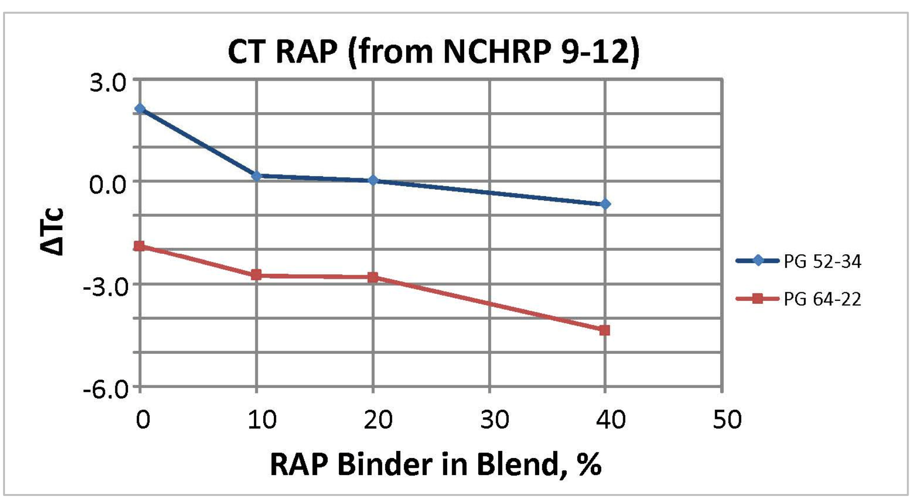
Sources

Anderson, R. M., P. B. Blankenship, A. Zeinali, G. N. King, and D. I. Hanson. Optimal Timing of Preventive Maintenance for Addressing Environmental Aging in Hot-Mix Asphalt Pavements. Report MN/RC 2014-45. Minnesota Department of Transportation, St. Paul, 2014.

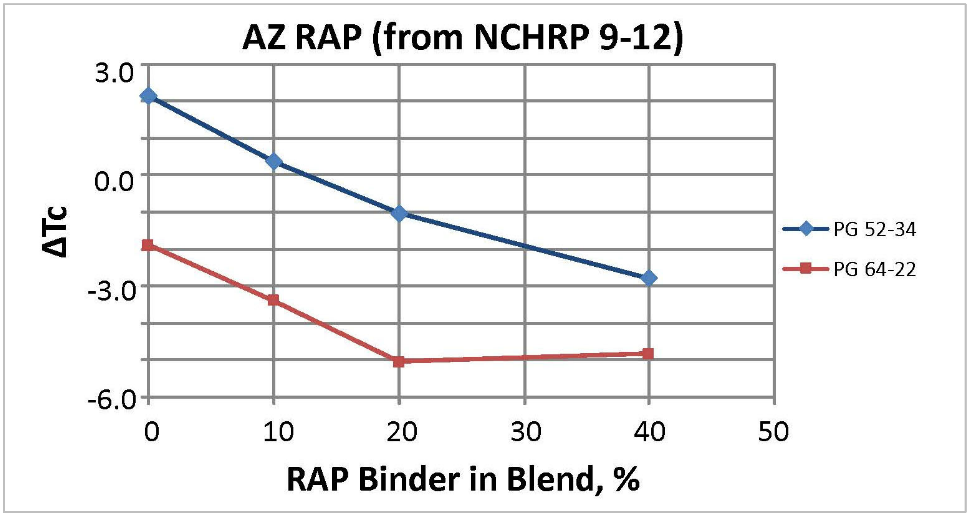
Mirza, M. W., and M. W. Witczak. Development of a Global Aging System for Short- and Long-Term Aging of Asphalt Cements. *Asphalt Paving Technology*, Vol. 64, Association of Asphalt Paving Technologists, 1995.

Houston, W. N., M. W. Mirza, C. E. Zapata, and S. Raghavendra. *NCHRP Web-Only Document 113: Environmental Effects in Pavement Mix and Structural Design Systems*. National Cooperative Highway Research Program, 2005.

One of the advantages of using ΔT_c as a parameter is that in many cases no new testing is needed; just an analysis of existing BBR data. Slide 29 and Slide 30 provide examples of new analysis of old data by examining the data from the Binder Effects Experiment in the NCHRP 9-12 project. In this experiment, recycled asphalt pavement (RAP) binder was recovered from different sources (Connecticut in Slide 29 and Arizona in Slide 30) and blended with virgin asphalt binder in the proportions shown—generally 10, 20, and 40% RAP binder. The physical properties of the asphalt binder were tested to compare with the estimates obtained using linear blending equations (or charts) from data obtained from the recovered RAP binder and virgin binder. BBR testing was conducted at multiple temperatures to determine the critical low temperatures for S ($T_{c,S}$) and m ($T_{c,m}$), which in turn allowed for a determination of ΔT_c for each combination of virgin asphalt binder, RAP binder and percentage. The results show that the ΔT_c of the virgin binder matters in the final blended value of ΔT_c, with the softer virgin binder grade (PG 52-34) having a ΔT_c value that is 4 degrees greater than the stiffer virgin binder grade (PG 64-22). Similarly, adding a greater percentage of aged RAP binder to the blend resulted in a general decrease in ΔT_c value. Finally, the properties of the aged RAP binder also affect the value of ΔT_c, with the stiffer RAP (Arizona) producing generally lower values than the RAP from a more moderate climate (Connecticut). This response confirms that ΔT_c is an apparent indicator of oxidative aging.



SLIDE 29



SLIDE 30

Sources

McDaniel, R. S., H. Soleymani, R. M. Anderson, P. Turner, and R. Peterson. *NCHRP Web-Only Document 30: Recommended Use of Reclaimed Asphalt Pavement in the Superpave Mix Design Method*. National Cooperative Highway Research Program, 2000.

NCHRP PROJECT 9-12 MIXTURE EFFECTS STUDY

The purpose of the Mixture Effects Experiment in the NCHRP 9-12 project was to validate the findings of the Binder Effects Experiment and examine how different virgin binder grade, RAP binder grade, and RAP stiffness affects mixture properties like fatigue (Slide 31). Flexural beam fatigue testing was conducted in accordance with AASHTO T321 (although it might have been the provisional version of the standard at that time) at 20°C and two strain levels: 400E-06 and 800E-06 mm/mm. In general, responses were rational, with lower number of cycles to failure associated with mixtures with stiffer asphalt binders. This can also be seen in Slide 32 as cycles to failure is plotted as a function of the initial value of flexural loss stiffness (S*sin φ).

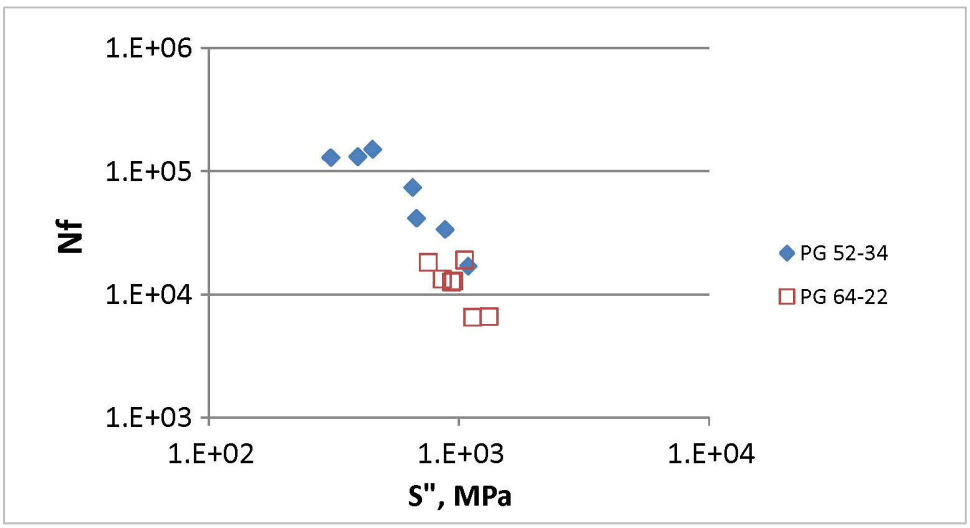
How does this relate to ΔT_c? The working hypothesis by the author and research team members is that the number of cycles to failure in a fatigue test is affected not only by stiffness, but also by the relaxation of the mix. Just as G*sin δ is not considered an ideal indicator of fatigue cracking performance for asphalt binders, the concern is that initial flexural loss stiffness may also not be the only factor affecting fatigue cracking performance for mixtures. This hypothesis is explored in the next few slides.

In Slide 33, two pairs of data are highlighted from the NCHRP 9-12 Mixture Effects Study. The blue-shaded cells show the number of cycles to failure for the PG 52-34 virgin binder blended with 20% RAP from Connecticut and Arizona. The initial flexural stiffness of these two mixes were very similar at 931 MPa (Connecticut) and 976 MPa (Arizona) but the cycles to failure were dramatically different at 73,767 (Connecticut) and 41,259 (Arizona). The orange-

- Validate Binder Effects Study
  - Beam Fatigue testing on STOA mixes
  - 20°C, 800E-06 mm/mm strain
- Generally Rational Responses

Binder	RAP	10%	20%	40%
PG 52-34	CT	131,121	73,767	33,533
	AZ	150,530	41,259	16,892
PG 64-22	CT	18,164	12,822	19,043
	AZ	13,332	6,527	6,608

SLIDE 31



SLIDE 32

- Flexural Beam Fatigue Nf
  - Affected by Initial Stiffness

Binder	RAP	10%	20%	40%
PG 52-34	CT	131,121	73,767	33,533
	AZ	150,530	41,259	16,892
PG 64-22	CT	18,164	12,822	19,043
	AZ	13,332	6,527	6,608

- 52CT20 = 931 MPa 52AZ20 = 976 MPa
- 52CT40 = 1313 MPa 64CT10 = 1291 MPa

SLIDE 33

shaded cells show two mixes with similar initial flexural stiffness at 1313 MPa (PG 52-34 virgin binder with 40% CT RAP) and 1291 MPa (PG 64-22 virgin binder with 10% CT RAP) but very different number of cycles to failure—33,533 and 18,164 cycles, respectively.

In Slide 34, the values of ΔT_c are added to support the hypothesis that relaxation is important in fatigue performance in addition to stiffness. In both comparisons, the lower number of cycles to failure is associated with a lower value of ΔT_c, despite the similar stiffness values.

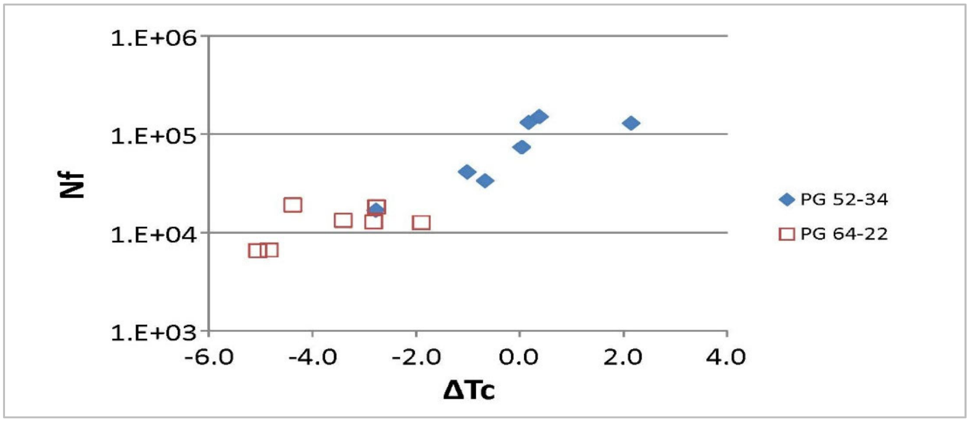
Slide 35 shows the relationship between cycles to failure and ΔT_c. the relationship looks similar as the data in Slide 32.

- Flexural Beam Fatigue Nf
  - Affected by Initial Stiffness

Binder	RAP	10%	20%	40%	
PG 52-34	CT	131,12 **0.0**	73,767	33,533	**-0.7**
	AZ	150,5 **-1.0**	41,259	16,892	
PG 64-22	CT	18,164	**-2.8** ,822	19,043	
	AZ	13,332	6,527	6,608	

- 52CT20 = 931 MPa 52AZ20 = 976 MPa
- 52CT40 = 1313 MPa 64CT10 = 1291 MPa

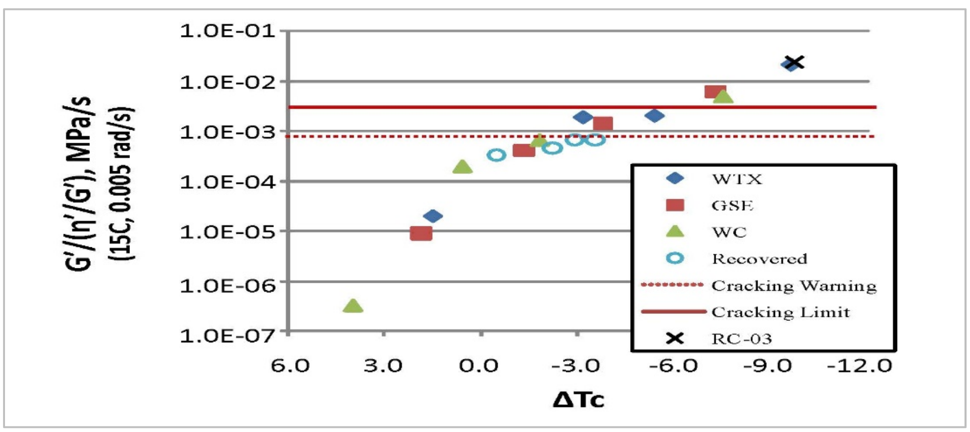
SLIDE 34



SLIDE 35

RELATIONSHIP BETWEEN G′/(η′/G′) and ΔT_c

This graphic (Slide 36) was developed during the AAPTP 06-01 research and was reported on in the 2011 Association of Asphalt Paving Technologists (AAPT) paper referenced earlier. In addition to the three paving grade asphalt binders, a data point was added for a roofing asphalt binder on which there was BBR data. The data point represented by an "X" is a roofing coating—an asphalt binder produced by air blowing a flux—and is very stiff compared to paving grade asphalt binders. As expected, the value of ΔT_c is much more negative, indicating a loss of relaxation properties compared to other materials.



SLIDE 36

ΔT_c WITH RECLAIMED ASPHALT SHINGLES

Continuing the thought about roofing asphalt binders, this slide presents data derived from testing conducted on reclaimed asphalt shingles (RAS) supplied by the National Center for Asphalt Technology as part of research being conducted through a cooperative agreement with the Federal Highway Administration (Slide 37). The data represents testing conducted by the Asphalt Institute on RAS from different sources including:

(a) Post-consumer waste, or tear-offs, (PC);
(b) Manufactured waste (MW); and
(c) A blend of both MW and PC.

From that study, a couple of observations were made. First, the values of ΔT_c were somewhat variable and did not necessarily line up as expected. It was expected that the MW RAS would have higher (less negative) values of ΔT_c than either the PC or Blend RAS because the MW RAS would not have been exposed to additional oxidation in-service on the roof. While this was true for the Texas RAS (which had the least negative value of ΔT_c) it was not true for the Wisconsin RAS (which had the most negative value of ΔT_c). It is difficult to draw a conclusion based on only four materials, so not too much should be made of the results. Nevertheless, the range of ΔT_c was –23 to –40. These values are much lower than are normally seen with paving asphalt binders.

The second observation was that the determination of ΔT_c was difficult to make for the recovered RAS binders because at test temperatures where the stiffness brackets 300 MPa the m-values are very low and not as affected by changes in temperature. Thus, $T_{c,S}$ can be easily determined, but an extrapolation to determine $T_{c,m}$ can yield unreasonable results. Likewise, as temperatures are increased to try and get m-values closer to 0.300, the stiffness decreases to the point that there is concern that the beam will exceed the maximum allowable deflection per AASHTO T313. As such, $T_{c,m}$ must be determined from extrapolation, using data at temperatures that are significantly colder than the expected temperature to produce an m-value of 0.300.

- NCAT Materials
  - RAS study conducted by Richard Willis and Pamela Turner
    - Four RAS sources (MW, PC, Blend) supplied
    - ΔT_c values variable
    - Difficult to test BBR at elevated temperatures

RAS	Tc,High	Tc,Low	ΔTc
NH (PC)	163	+12	-33
OR (Blend)	152	+14	-37
TX (MW)	122	-7	-23
WI (MW)	146	+16	-40

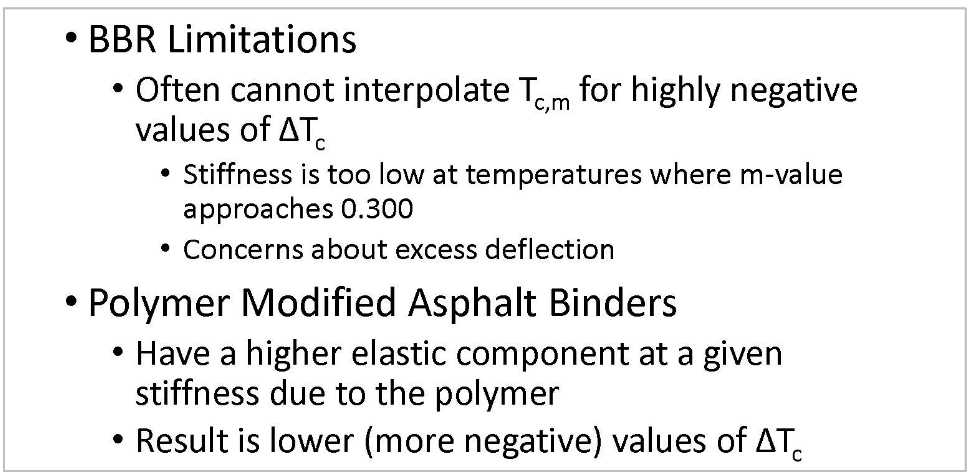
SLIDE 37

Source

Willis, J. R., and P. Turner. Characterization of Asphalt Binder Extracted from Reclaimed Asphalt Shingles. NCAT Report 16-01. National Center for Asphalt Technology, Auburn, Ala., 2016.

ΔT_c POTENTIAL ISSUES

The determination and use of ΔT_c as a parameter does have some potential issues (Slide 38). The first that is noted is what was previously discussed on Slide 37—that interpolation to determine $T_{c,m}$ values is often not possible for highly oxidized materials such as RAS. The alternative is extrapolation, which is an estimate at best and, depending on the extent of extrapolation, could produce numbers that are excessively high for $T_{c,m}$ leading to excessively low values of ΔT_c.



SLIDE 38

The second potential issue is with the value of ΔT_c for polymer-modified asphalt binders. Since polymer modified asphalt binders usually have a higher elastic component (lower phase angle) at a given stiffness, and ΔT_c responds to this type of behavior by appearing to be more negative, it is expected that many polymer-modified asphalt binders will have lower (more negative) values of ΔT_c than many conventional (unmodified) asphalt binders. The implication is that lower values of ΔT_c may be associated with increased potential for durability cracking, which is counterintuitive for many polymer-modified asphalt binders.

VARIANCE IN ΔT_c

Variability of any test parameter should also be considered. Since ΔT_c is derived from BBR testing, the variability in the value of ΔT_c is a function of the variability in determining stiffness and m-value from BBR testing (AASHTO T313). The standard, published precision estimates for single-operator d2s% are 7.2% for stiffness and 2.9% for m-value (Slide 39). Using these values, one is able to create cases of BBR results to see how ΔT_c varies as a function of that repeatability. The same can be done for multilaboratory variability, where d2s% values are 17.8% and 6.8% for stiffness and m-value, respectively (Slide 40).

Variance in ΔTc as a function of Single Operator d2S%

		S	m	S ++	m ++	S - -	m - -
	-12	200	0.33	207.2	0.3348	192.8	0.3252
	-18	400	0.27	414.4	0.2739	385.6	0.2661
		-25.5	-25.0	-25.2	-25.4	-25.8	-24.6
ΔTc			-0.5		0.2		-1.3

		S	m	S ++	m ++	S - -	m - -
	-12	280	0.305	290.1	0.3094	269.9	0.3006
	-18	560	0.245	580.2	0.2486	539.8	0.2414
		-22.6	-22.5	-22.3	-22.9	-22.9	-22.1
ΔTc			-0.1		0.6		-0.9

Single Operator d2S%
S = 7.2%
m = 2.9%

SLIDE 39

- Variance in ΔT_c
  - Single Operator d2s% = 0.8°C
  - Multi-Laboratory d2s% = 1.8°C
  - Rough estimate based on single analysis. May be different if assumptions are different
    - S(60) changes by a factor of two for a change of 6°C
    - m(60) changes by 0.060 for a change of 6°C
    - Single Operator d2s%
      - S = 7.2% m=2.9%
    - Multi-Laboratory d2s%
      - S = 17.8% m=6.8%

SLIDE 40

Using these two sets of values and the assumptions that stiffness changes by a factor of two and m-value changes by 0.060 for a change of 6 degrees, the single-operator d2s% and multilaboratory d2s% can be estimated for ΔT_c as 0.8 and 1.8 degrees, respectively. This variability should be considered if seeking to use ΔT_c in the evaluation of asphalt binders. Putting these numbers in perspective, AASHTO PP78: Provisional Practice for Design Considerations When Using Reclaimed Asphalt Shingles in Asphalt Mixtures, suggests that when the ΔT_c value is –5.0 or lower "a significant loss of cracking resistance occurs." The single operator and multilab variability is estimated to be 16% and 36% of that limit, respectively.

SUMMARY

- ΔT_c can be easily calculated using standard BBR test data
  - Depending on lab and practices, 1-2 additional BBR tests at different temperatures may be needed
- ΔT_c appears to be an indicator of oxidative aging and loss of relaxation properties
  - Related to intermediate, durability cracking even though the tests are performed at low temperatures
  - Greater aging results in lower (more negative) values of ΔT_c

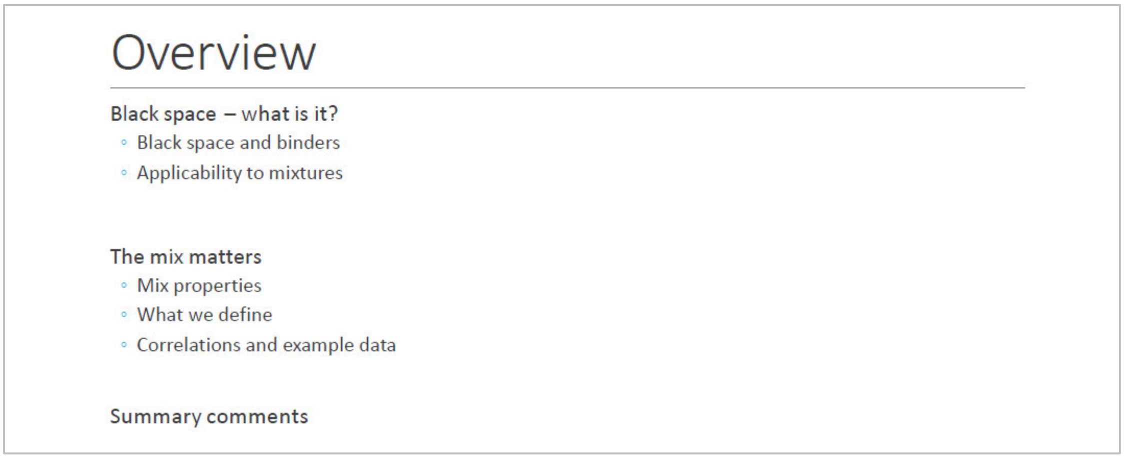
SLIDE 41

- Variability of ΔT_c appears reasonable, as it is a product of BBR testing
  - Consider variability if establishing guidance for use
- Asphalt binders with very negative values of ΔT_c may be more difficult to test in the BBR
  - RAS binders
- Caution when using ΔT_c with polymer modified asphalt binders
  - Higher elastic component may make ΔT_c more negative

SLIDE 42

Black Space Characterization of Binders and Applications to Mix

Geoffrey M. Rowe
Abatech

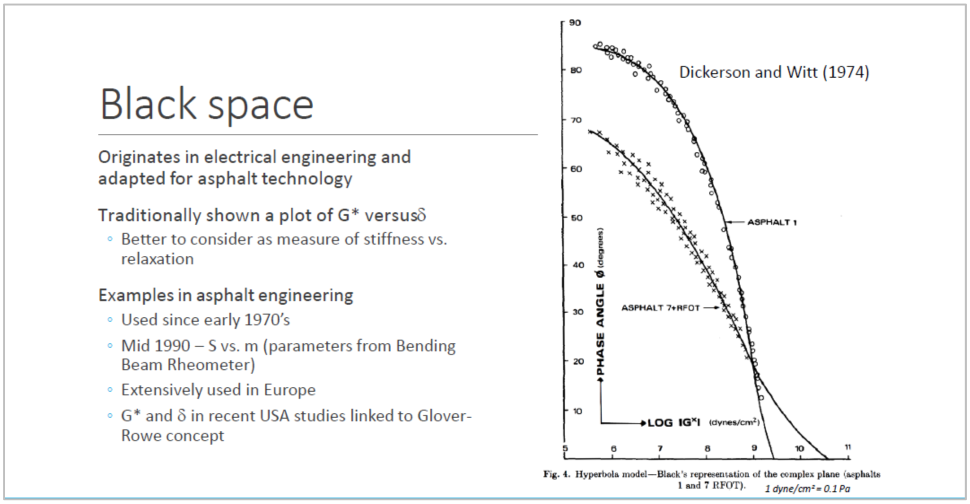


SLIDE 2

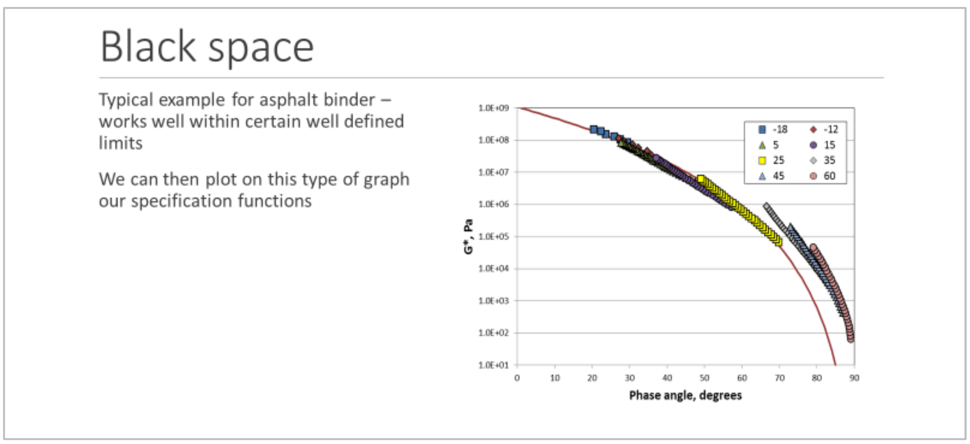
This presentation discussed the black space characterization of binders and explored the applications to mixtures (Slide 2). The mix matters at the end of the day, since we drive on roads that are built of asphalt mixtures not asphalt binders. We have to consider the mix properties and what we define is very important for performance of our highway network. The presentation gave some examples of parameters and correlations that have been developed with various data sources. Summary comments are given at the end of the presentation.

Black space representations have existed in asphalt technology for some time. The slide that shows the plot from Dickason and Witts (1974) paper demonstrates that black space has been part of the asphalt industry for some considerable time (Slide 3). Examples go back in asphalt engineering to the early 1970s. In the mid-1990s the S versus m parameter from the BBR is also a form of Black Space that has been extensively used. The use of G*and phase angle (δ) in the United States and the linkage to parameters such as the Glover Rowe (G-R) concept is all based around the black space. The original idea of black space originates from electrical engineering and this has been adapted for asphalt technology. We believe the original publication dates back to the 1920s, but we haven't been able to track down the original source.

The figure in Slide 4 shows a typical black space for an asphalt binder, shown as G* versus δ. If you look carefully at the curves you will see that the results from –18°C to +25°C form a smooth curve. There is some separation between the 25°C result and the 35°C result. It has been suggested that this separation is a function of components in the binder which may be wax or some other material. When we see separations of this kind in a black space plot it suggests that the binder is not behaving in a thermo-rheologically simple manner. The material can be considered thermo-rheologically simple for the lower temperature range.



SLIDE 3

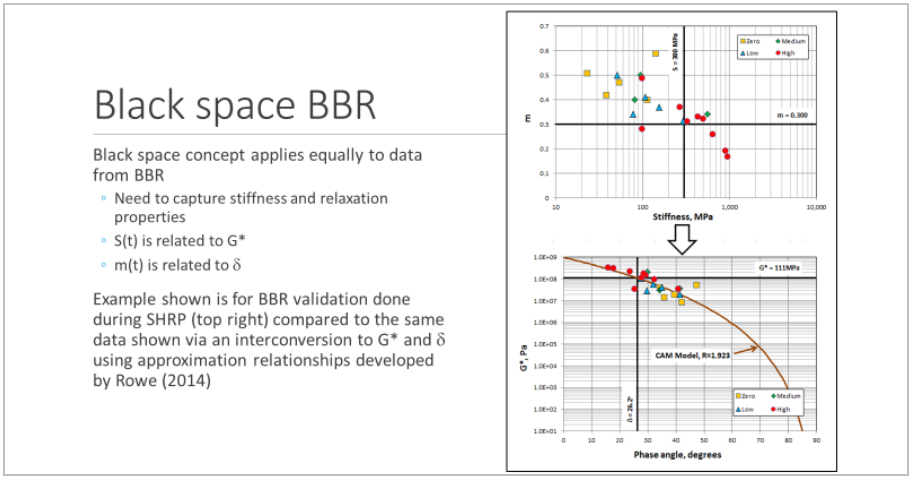


SLIDE 4

The black space concept applies equally to data in the BBR, the stiffness with respect to time, S(t), in the BBR is related to the G*in the DSR test. The m(t), [d log S(t)/ d log t] is related to the δ obtained from DSR testing.

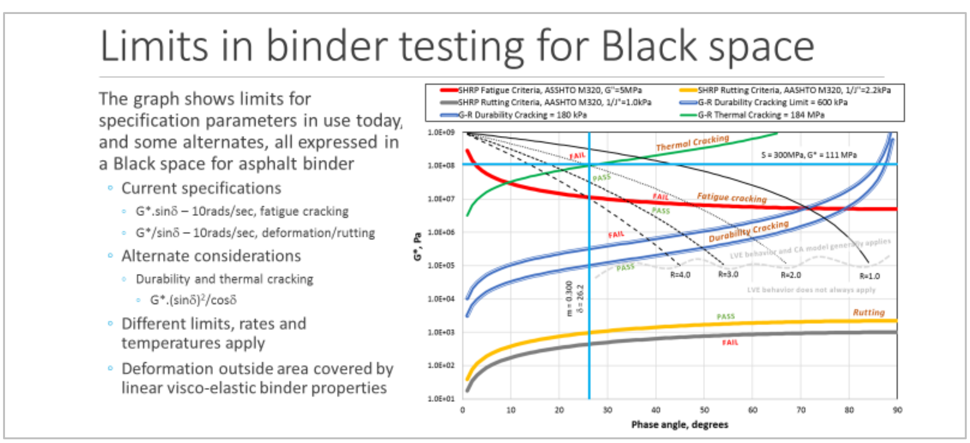
The example shown in the figure on Slide 5 is data that which was developed and presented at a Canadian Technical Asphalt Association meeting. It shows the original validation data points used for the S equals 300 MPa, m is equal to 0.300. These data items are converted to a G* of 111 MPa and a phase lag (δ) of 26.2° in the lower graph. The line that curves through the intersection point of the two black lines represents a CA model if the R-value was 1.923 and the glassy modulus equals 1e9 Pa.

This curve is important since data points above that curve would represent binders that are S controlled whereas data points below the curve would represent binders that are m-controlled. That is to say that when we have a large R-value the binders tend to be m-controlled and when the R-value is smaller the binders tend to be S-controlled.



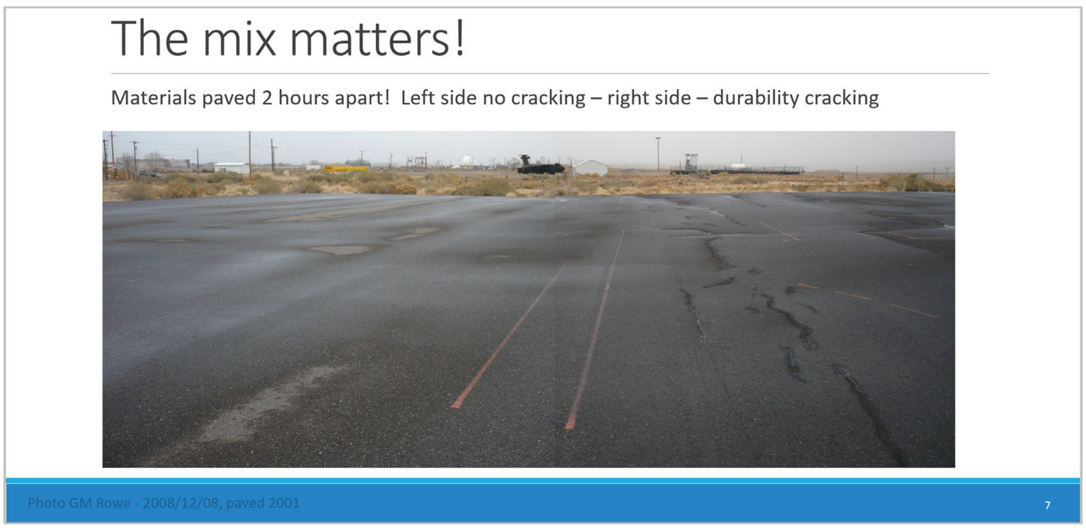
SLIDE 5

Slide 6 shows various cracking limits used in our specification and the deformation criteria (G*/sinδ) in a black space plot. The cracking parameters in this graph relate to thermal cracking, fatigue cracking, durability cracking, and permanent deformation. Note that on this graph there is a gray line below the limit of 1e5 Pa. When the G* is lower than this value we must be very cautious about applying the CA model to those values. Consequently, this graph shows full curves representing different values of R-value for the CA model with a glassy modulus set at 1e9 Pa for each of those curves. Typically, asphalt binders in common usage have an R-value somewhere between 1.0 and 2.5. The R-values of 3 and 4 would be more representative of oxidized binders. The area in the black space below 1e4 Pa is more associated with the deformation behavior of binders and are outside the stiffness range which can be modeled by the CA model. In addition, we are now using parameters such as the nonrecoverable creep compliance (J_{nr}) which also captures to some extent the nonlinear viscoelastic material properties associated with deformation response. However, we can say that all cracking parameters whether it be thermal, fatigue, or durability cracking are within the range covered by linear viscoelastic behavior and that defined by the CA-model and the stiffness range one each of the 1e5 to 1e9 Pa.



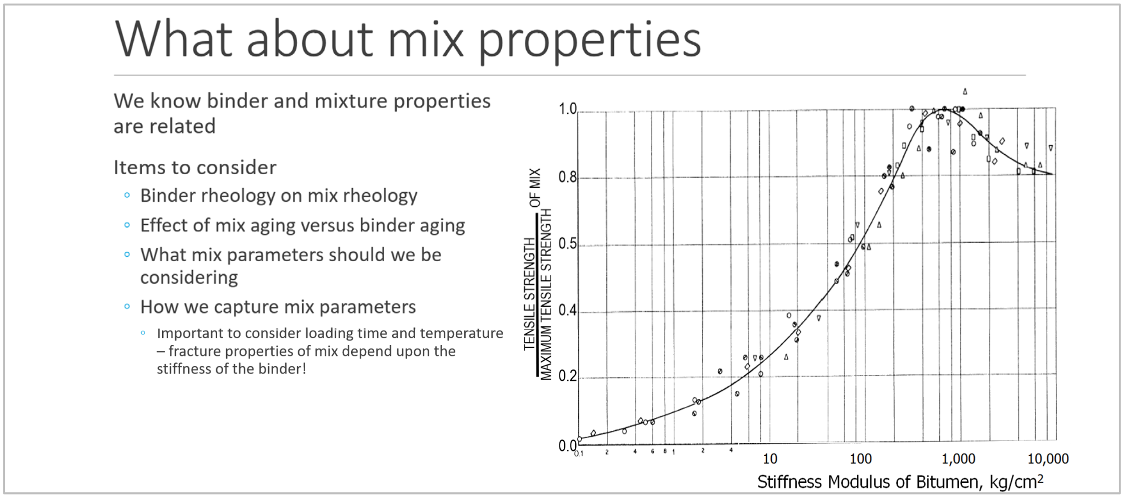
SLIDE 6

The mixture does matter. The photograph on Slide 7 shows a project that I was involved with in New Mexico a few years back. If you look at this photograph you will see that top-down surface cracking exists on the right side of the picture whereas the material on the left side has no surface cracking and looks perfectly satisfactory. This photograph was taken approximately 7 years after the material was laid. The material laid on the right side was laid about 2 h after that which was laid on the left side. The importance of this is to note that in between laying the left and the right the plant ran out of the filler specified in the mix design. An alternate filler was procured by the plant without knowledge of the quality control technicians and not in accordance with the specification. The material that was laid on the right side containing this nonconforming filler ended up having lower durability than the material on the left side. The significant point here is that while we can do our best to understand the binder properties we also cannot neglect understanding the mix properties and the way the mixture ages. In this case the binder properties did not change but two entirely different outcomes are seen under identical site conditions point to the importance of mix properties.



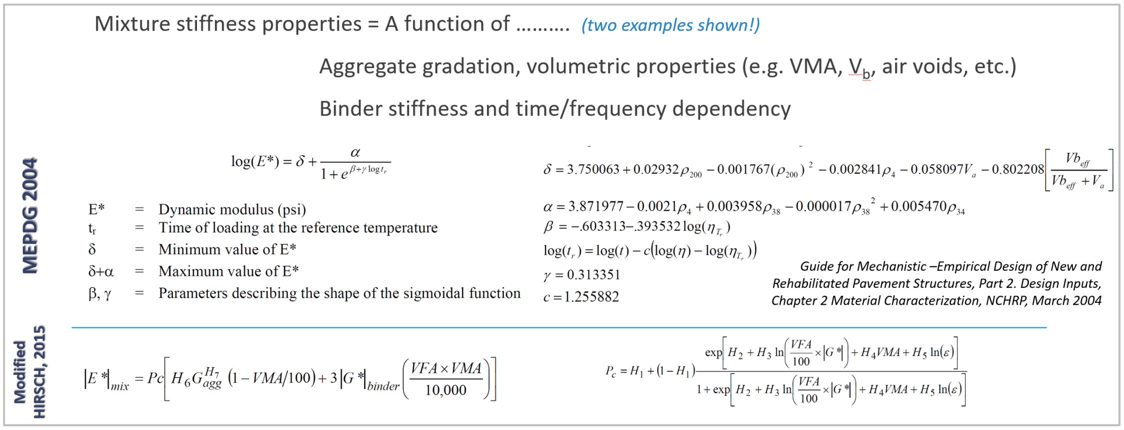
SLIDE 7

Slide 8 shows a plot from Heukelom's 1966 AAPT paper. This graph shows on the horizontal axis the stiffness modulus of the bitumen plotted against a normalized tensile strength of various mixtures. The interesting thing here is that when the tensile strength is normalized in this manner all these mixtures have a similar relationship when compared to the stiffness modulus of the bitumen. This leads to the conclusion, for the most part, the cracking behavior of the mix can be related to the stiffness behavior of the bitumen–asphalt binder. The other important thing to note here is that the stiffness of the bitumen is a time–temperature-adjusted parameter insomuch that the stiffness depends on both time and temperature of loading. If we consider normalization of data whether it be strength or fracture property we can produce an ultimate property master curve if we plot those results against the stiffness modulus of the bitumen–asphalt binder. This is a fact in asphalt engineering and this was established approximately 50 years ago but is been used very little in our interpretation of stress, strain, or energy at break, or ultimate properties of any kind.

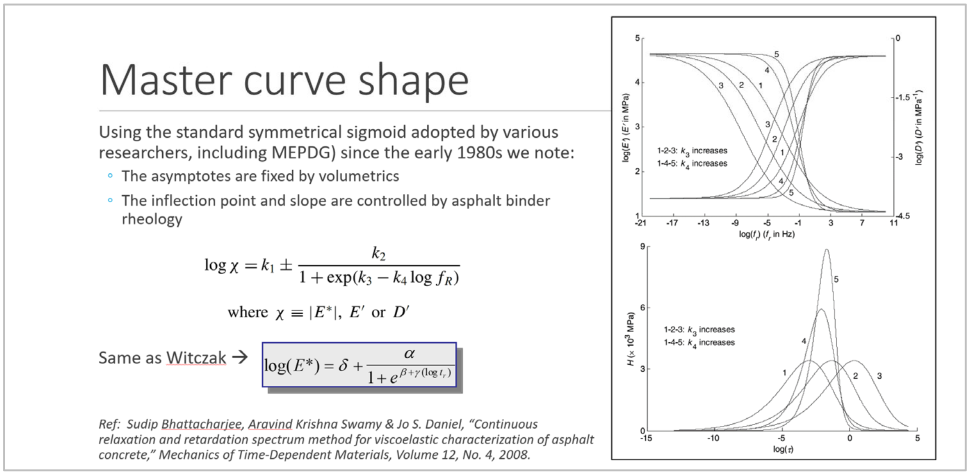


SLIDE 8

The next question that we must consider is how all the binder and the mix are related. Slide 9 shows some equations developed by Matt Witczak (and his team) and Don Christensen (and his coworkers) in the development of the Hirsch model. If you look carefully at these equations you will see that the binder properties are used to describe the temperature susceptibility and time dependency in both of these equations. The form of the dependency is a little different from one equation to another. The first equation used in the *Mechanistic–Empirical Pavement Design Guide* (MEPDG) equation uses an exponential form in the sigmoid equation and a couple of factors in that term are dependent upon the viscosity of the binder. So effectively, both time and temperature susceptibility is dealt with via the viscosity aspect of the binder. In the second equation (the Hirsch model) we have the direct input of the complex shear modulus (G*) of the binder. The G* is dependent on both time of loading and temperature in the same manner that bitumen stiffness (S_b) as used by Heukelom in his representation of the data back in 1966.



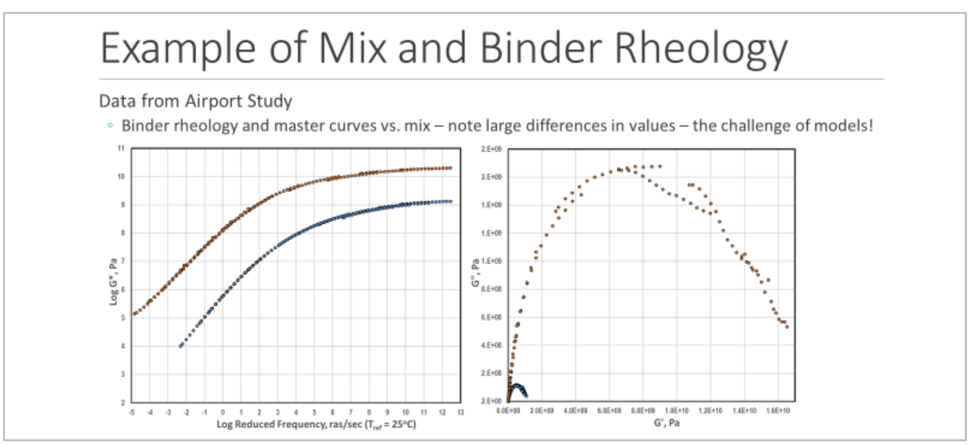
SLIDE 9



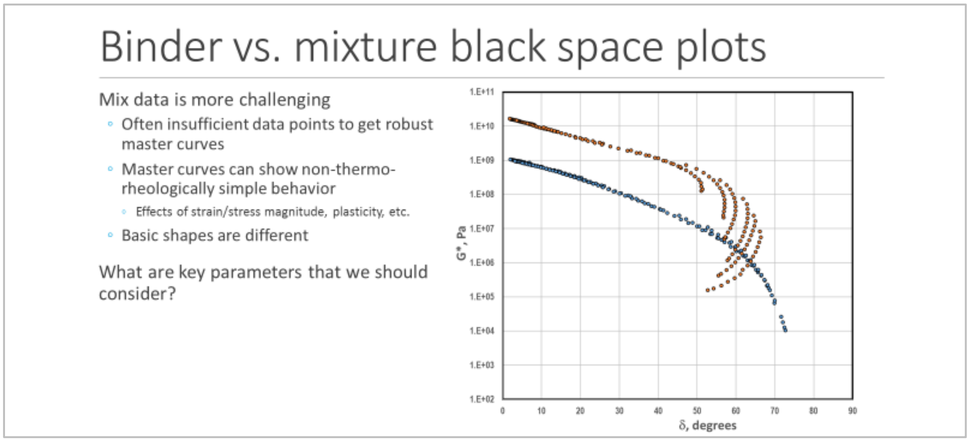
SLIDE 10

Some analyses conducted by the University of New Hampshire are presented in Slide 10. In this slide, they have the same equation form as used in the MEPDG—the model developed by Witczak et al. They conducted an analysis of the relaxation spectra and in the representation of that data in this plot, they have varied the k_3 and k_4 parameter in their equation which corresponds to the β and γ parameters in the Witczak equation and show how these change the shape of the curves. The β and γ parameters affect the slope of the central portion of the curve the position of the inflection point. In the symmetric sigmoid represented by the Witczak equation the β and γ parameters are effectively similar to the crossover frequency (ω_c) and the R-value in the C-A model and they are directly related.

When we look at values from data developed for master curves for binder and mixture we see very large difference in numerical values. The example in Slide 11 comes from some binder rheology work and mixture rheology work conducted on an airport study in Denmark. We can observe large differences in the numerical values. However, it's important to note that there is a direct translation in the slopes and the shapes of these curves that can be made by using the analysis on the previous slide.



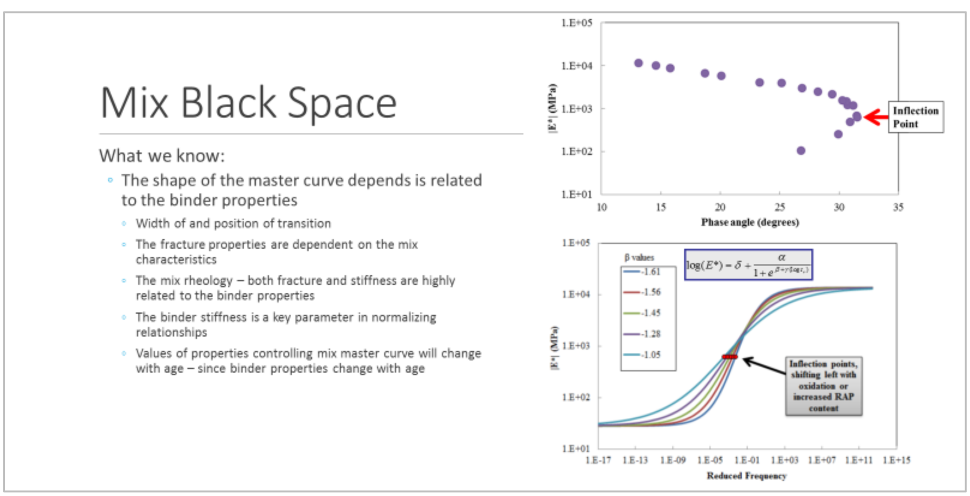
SLIDE 11



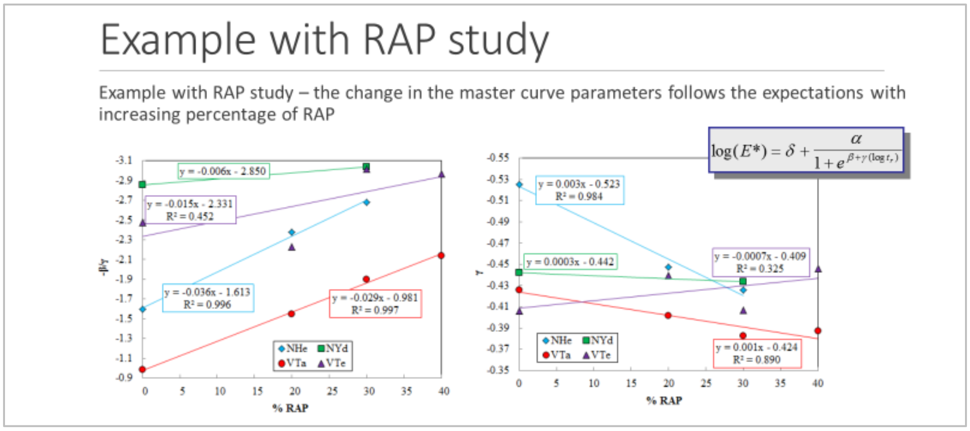
SLIDE 12

Slide 12 shows the same data was on the previous plot (Slide 11) but this time in a Black Space plot. It can be seen that with a mixture (the brown-colored set of data) there is some non–thermo-rheologically simple behavior as the lines do not neatly lineup. The basic shapes appear to be different but one of the key parameters we should consider we need to look very carefully at the high stiffness portion of the master curve and the area from the inflection point and higher because this area corresponds directly to the cracking regime in the binder master curve.

Slide 13 shows how the mixture master curve shape changes with changing values of β and it in shows how we can deduce an inflection point on we can see that from understanding a black space parameter in a mixture master curve. The width and the position of the transition is dependent upon the binder properties. The fracture properties are dependent upon the mixture characteristics the mix rheology. However, both fracture and stiffness are highly related to the binder properties and the binder stiffness is a key parameter in normalizing the relationships for mixture stiffness. The values of properties controlling the mix master curve will change with age so we need to understand how the binder properties change with age.



SLIDE 13

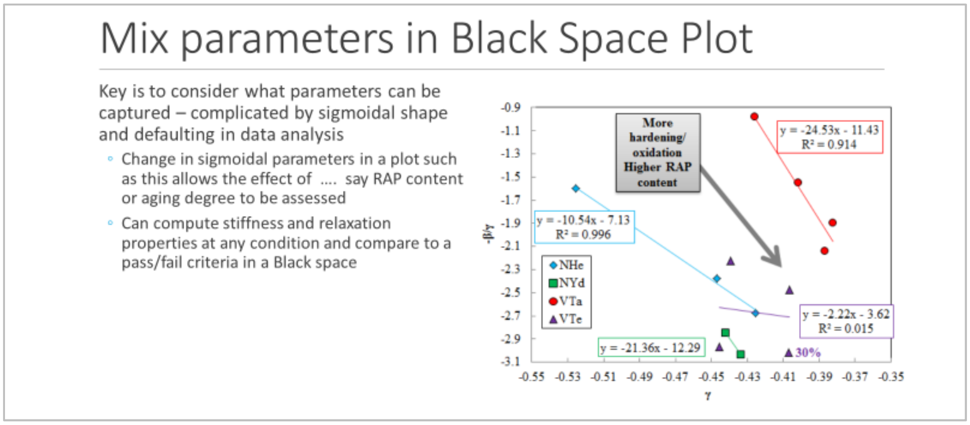


SLIDE 14

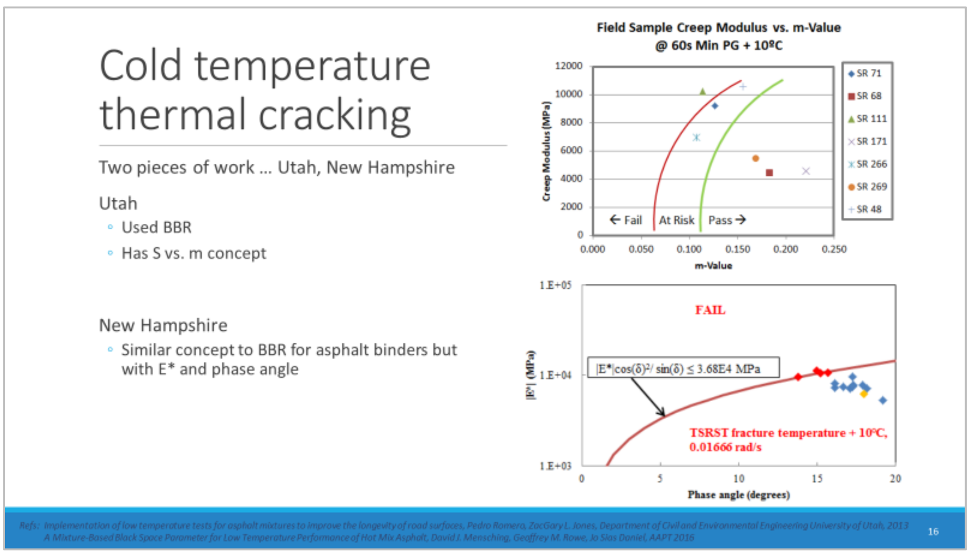
In Slide 14 we show how critical parameters –β/γ (which is the inflection point) and γ (which is the slope of the central portion) change with different percentages of RAP this data comes from a New Hampshire study reported by Mensching et al. (2016). In this data set we can see that the –β/γ increases with RAP content. Remember here that this is the inflection point so the inflection point is basically changing to a more negative value which means the master curve is shifting to the left which is representing a hardening of the system associated with higher RAP. The slope also changes. The changes are a little more difficult to discern because sometimes the quality of the mix data is less than desirable and also with dealing with real materials and probably some experimental error. Generally, we expect the slope to decrease with the percentage of RAP since the RAP contains more oxidized binder we would expect a lower slope as we transition from the glassy asymptote to the equilibrium modulus. Generally, we see this trend in this data and with the exception of one of the results this trend holds true.

Both of these parameters can be combined and this is shown in Slide 15. If we plot –β/γ on the y-axis versus the γ on the horizontal axis this type of plot is effectively the same as looking at R-value versus ω_c. We can see that a change in the sigmoid parameters in such a plot goes in a certain direction with higher RAP contents, higher aging would also move the material in the same direction. Rejuvenation would do the opposite. If you wanted to rejuvenate these products you would move from the bottom right towards the top left. Oxidation goes one direction and rejuvenation goes in the other direction. This type of plot can help us with understanding our mix properties at any condition within a black space and can help us in understanding and developing pass fail criteria in a black space.

Now of interest is contrasting the work done at New Hampshire to work done in other places (Slide 16). On the top right, here we show some data developed by the University of Utah that used mixture BBR tests. On the vertical axis, we have the stiffness modulus (S) whereas on the horizontal axis we have the m-value. This plot is essentially the same type of plot as G*and δ (black space) as it is a representation of the stiffness and relaxation properties, but using data from a BBR test on mixtures. This work by Utah supports the concept of stiffness and relaxation properties and the use of a black space concept. In the bottom plot is the data from the University of New Hampshire and the plot shows the results of complex extensional modulus (E*) versus phase angle and the proposed failure criteria. This work was published by Mensching et al. in AAPT (2016).

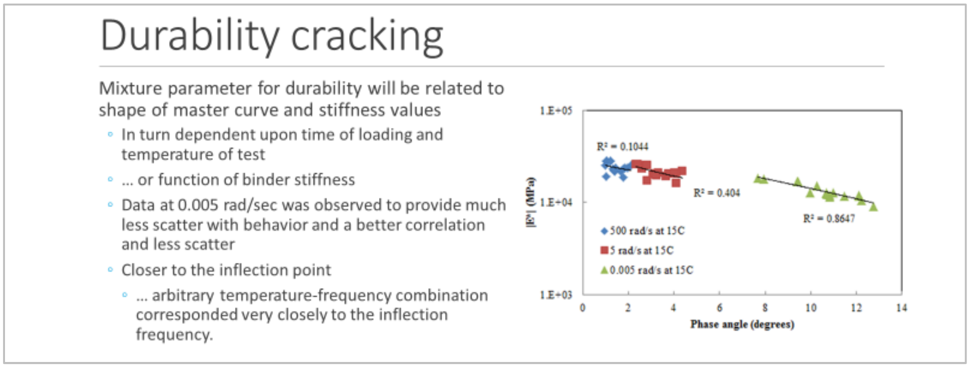


SLIDE 15



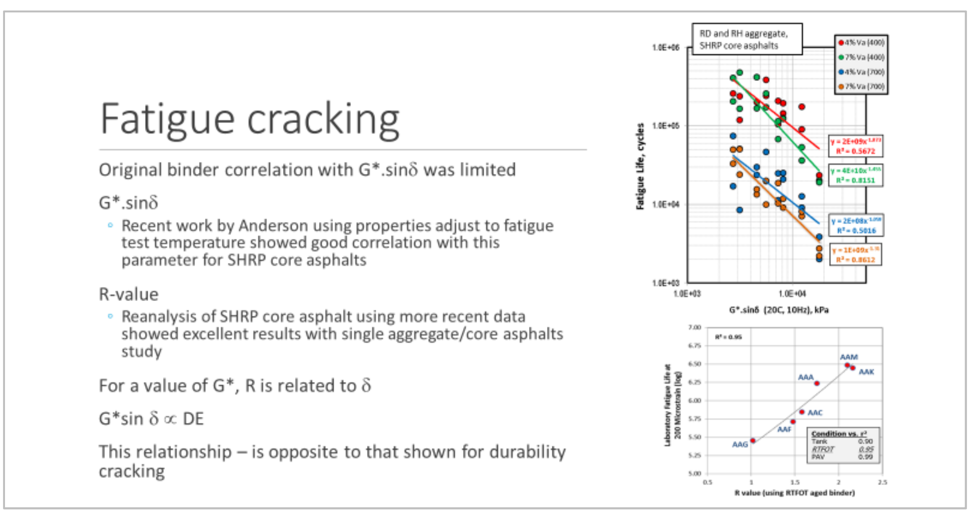
SLIDE 16

Durability cracking parameters can be developed for mixtures with black space concepts just as they can for us for binders. One of the concepts (Slide 17) was to use data at 0.005 rads/s and to compare this to data collected at say 500 rads/s. What was observed was that the 0.005 rads/s at 15°C gave the best correlation and had less scatter. The reason we feel that this one is important is because it's close to the inflection point that occurs in the mix master curve for the data studied. The mixture black space parameter that is used must consider what is needed to get the best indication of the slope factor and the inflection point. To have a number selected close to the inflection point is probably where we need to be in a black space type plot.



SLIDE 17

With regard to fatigue cracking two graphs are shown in Slide 18. The data sets on the top right considers the original fatigue data that was collected during the SHRP work and the original G*sinδ parameter. It can be observed that when the data was compared at the same temperature and at the same frequency as applicable to both test types then a reasonable correlation results. With bending beam fatigue this data would suggest that the lower the value of G*sinδ the better the fatigue life. This is suggesting a low G*sinδ is better. Now the second plot (lower right) is shows the R-value versus laboratory fatigue life (at 200 microstrain) for some of the same binders from the SHRP study (AAG, AAF, AAC, AAA, AAM, and AAK). These binders are what we referenced as the SHRP core asphalts. The results show the higher the R-value the longer the fatigue life becomes. The G*sinδ and R-value are both black space parameters and are related. The important observation here is that the fatigue life correlations do not line up with the binder parameters needed for durability performance. We see that we have in this plot here is some opposites in relationships compared to what will expect to see for durability cracking.



SLIDE 18

So the importance of this presentation is that we have to recognize stiffness and relaxation in both binder and mixture performance. The relationship between stiffness and relaxation is of key performance when understanding cracking. Stiffness can be characterized by G*, E*, S(t) and the relaxation is characterized by the phase angle (for a binder δ or for a mix often shown as ϕ) or "m" for if we use a BBR for either binder or mix testing. The range of properties related to fracture lie within the region of the binder master curve that can be described by the CA model that is within the stiffness range 1e5 to 1e9Pa. In this range, the properties can be represented by linear viscoelastic behavior and properties can be inspected in the black space within this range. Generally all binders will produce a smooth master curve. Interrelationships exist, for example if the data is interconverted using the relaxation spectra analysis of a BBR, the stiffness of 300 MPa equates to a G*of 111 MPa. The m-value of 0.300 would equate to a phase angle of approximately 26.2°. It should be noted that the phase angle relationship is not quite as robust as the stiffness relationship. Note, some other relationships exist such as those used with 4-mm DSR testing and comparisons to BBR. These relationships may be further developed as this testing develops towards full standards.

In summary, the development of tests and concepts for cracking should consider that cracking is within the region of binder stiffness that can be characterized by a linear viscoelastic behavior. This covers the brittle-ductile (or instability flow transitions) that occur. Stiffness can be used as a normalizing parameter to assess the quality of products and ultimate property master curves exists for materials. These concepts are becoming more well-known and asphalt materials specifications could be improved if they are included. Material pass–fail criteria can be developed in a black space plot. The failure criteria may vary with modified binders and will most likely need some adjustment for different climatic zones etc. Mixture black space failure criteria have been demonstrated for cold temperature cracking (for example the Utah and New Hampshire studies) and also intermediate cracking.

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Field Experience with Rheological Parameters

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Field Experience with Binder Rheological Parameters

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MTE

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Transportation Research Board Meeting
Session 307
Washington D.C.
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SLIDE 1

Acknowledgements

- MTE Staff
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- Mathy Construction Staff
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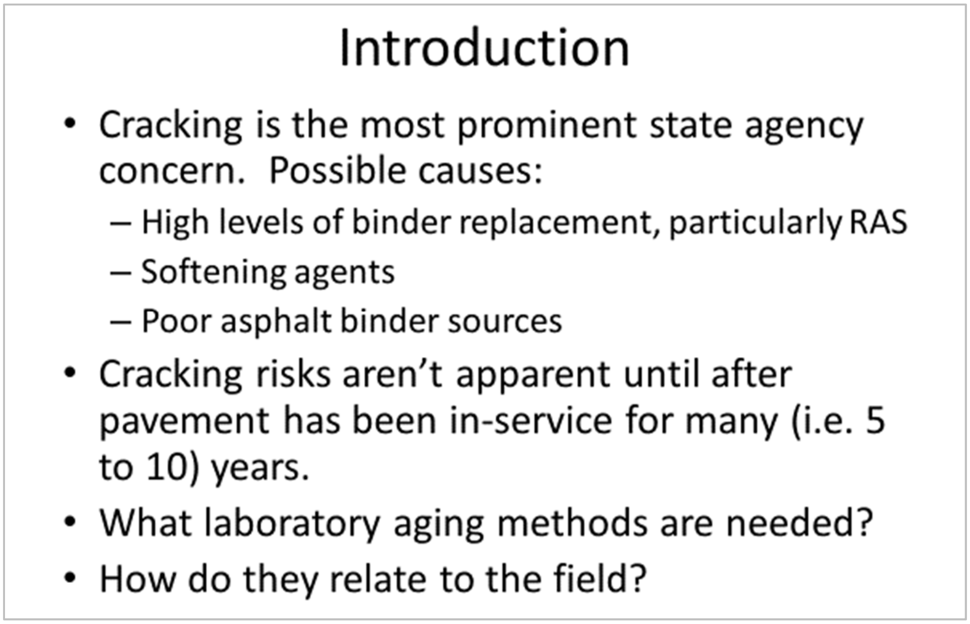
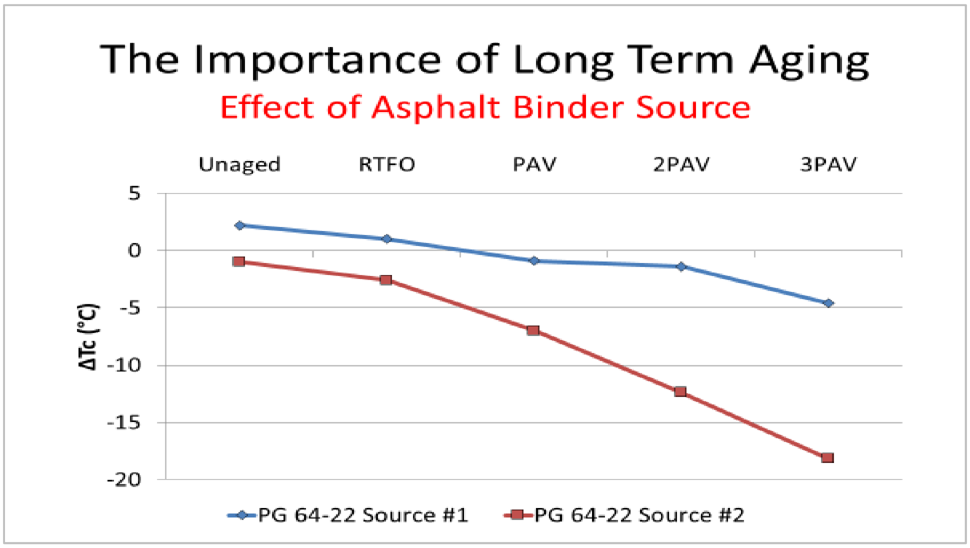
SLIDE 2



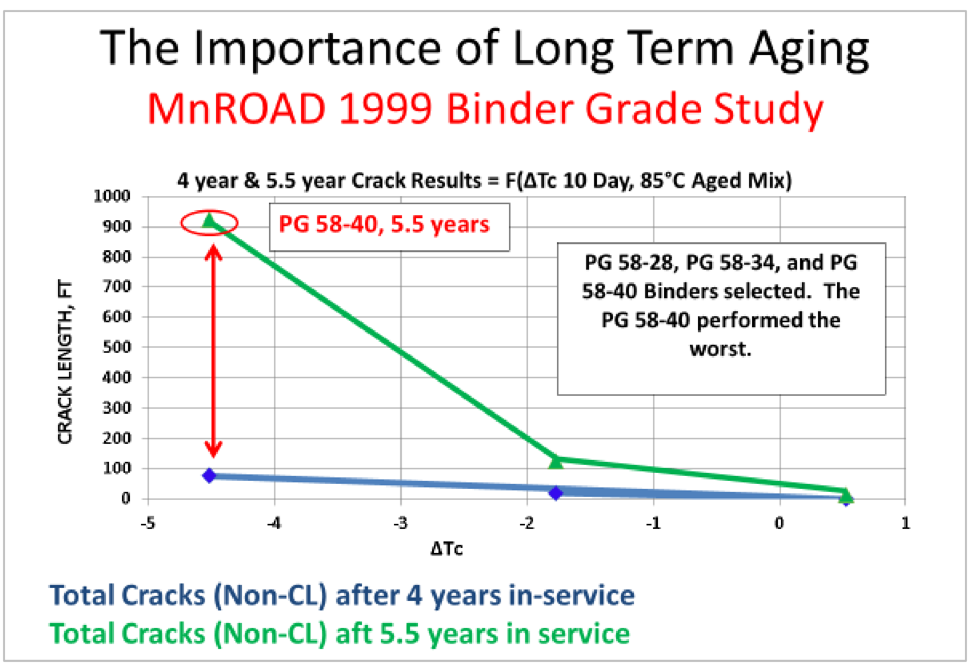
SLIDE 3

The implementation of the SHRP asphalt mix design and asphalt binder performance grading system in the 1990s addressed rutting issues by changing mix composition and providing guidelines for use of polymer-modified asphalts (Slide 3). As rutting performance improved cracking distress became more prominent. Reasons for the increased cracking distress cited include reduced asphalt contents, particularly for higher traffic mixes due to the gyration levels used in the SuperPave mix design method and the increased use of RAP and more recently RAS. This slide identifies softening agents and aging susceptible asphalt binder sources as two other asphalt binder related factors that can influence cracking performance. The concept that sufficient aging is needed to differentiate cracking susceptible materials is also introduced. There is an interactive effect between aging and materials used that have practical implications. Regarding asphalt source, durability properties and aging resistance are directly related to properties of the crude oil slate used for production. As a result asphalts that have the same PG grade but are from different sources can have very different performance characteristics and aging susceptibilities. Softening agents have become more prominent as a result of agencies specifying softer binder grades to address cracking and the concept of "grade dumping" to accommodate the continued interest in increased use of recycled products. The influence of softening agents on performance and aging susceptibility as well as their interaction with recycled asphalts is a primary area of interest for researchers and practitioners. Two issues related to long term aging are addressed in this presentation, alternative asphalt mix or binder long term aging methods, and initial relationships between laboratory and field aging using select performance test sections from Minnesota and Wisconsin.

The importance of including long-term aging in evaluation of cracking susceptibility is demonstrated using examples of how different binder sources or PG grades using softening agents react to laboratory and field aging (Slide 4 and Slide 5). For this presentation the ΔT_c parameter was used to evaluate asphalt binder durability properties. The parameter is defined as $T_{critical}$ S(60) – $T_{critical}$ m(60), and is derived from low temperature continuous grade data. The parameter was introduced by Mike Anderson of the Asphalt Institute and the details of its origins and evolution were summarized in a previous presentation in this session. The reference used for this work is Anderson et al. (Anderson, R. M., G. N. King, D. I. Hanson, and P. B. Blankenship. Evaluation of the Relationship between Asphalt Binder Properties and Non-Load Related Cracking. *Asphalt Paving Technology*, Vol. 80, 2011).



SLIDE 4



SLIDE 5

This publication and the supporting research projects proved instrumental for future investigation of cracking potential because in addition to defining a cracking parameter the work presented the need for extended aging and introduced tentative warning ($\Delta T_c = -2.5°C$) and failure ($\Delta T_c = -5.0°C$) limits. These limits were used as a benchmark to assess performance for the data generated in the various studies at MTE Corp. summarized in these proceedings.

Slide 4 compares the evolution of the ΔT_c parameter with aging for PG 64-22 binders from two different sources. In addition to the standard aging used in AASHTO M320 and M332, extended aging including 40-h PAV (2PAV) and 60-h PAV (3PAV) were included. The results highlight the effect of binder source on aging susceptibility. Source 1 is resistant to aging maintaining a ΔT_c value above the –5.0°C failure threshold through 60 h of PAV aging, whereas Source 2 exceeds this threshold after one PAV cycle. Source 2 is a highly aging susceptible binder that is not commonly used for paving in the United States, Source 1 is commonly used for asphalt paving in the Midwest. The difference in performance between Source 1 and Source 2 represents the range in properties between paving grade asphalts, particularly when the effects of recycled binders and recycling agents are considered. Current specifications do not have the ability to differentiate between these materials.

Results of an asphalt binder grade study conducted at MnROAD in 1999 demonstrate that long-term laboratory aging must sufficiently stress the material in order to identify cracking susceptible materials. The MnROAD 1999 study was commissioned to investigate binder grade alternatives to reduce thermal cracking by constructing test sections with three different binder grades, PG 58-28, PG 58-34, and PG 58-40. After 5 years of performance monitoring the PG 58-40 section had significantly more cracking than the others, a result that was opposite of expected behavior. MTE recently completed a forensic analysis of these test sections, as shown in Slide 5. In the slide total crack length is plotted on the y-axis and the ΔT_c values of compacted mix sampled during production aged at 85°C for 10 days is plotted on the x-axis. For long-term mixture aging the AASHTO R30 protocol was used with extended aging. Total cracking is defined as the sum of transverse cracking, wheelpath/non-wheelpath cracking, and fatigue area. Centerline cracking was excluded. The relationship between ΔT_c and cracking distress is plotted after 4 and 5 years in service. After 4 years all three materials behaved similarly, whereas after 5 years a significant increase in cracking was observed for the PG 58-40 test section. The large change in performance after 5 years field aging provides anecdotal evidence of the need for sufficient laboratory aging. Furthermore, field performance related well to the extended aging protocol used as the ΔT_c value of the PG 58-40 section was significantly more negative than both the PG 58-34 and PG 58-28. Again this aspect of performance is not addressed in current the current binder grading systems.

Further forensic analysis was conducted on binders to identify the reason for the poor performance of the PG 58-40 binder. Results of X-ray fluorescence (XRF) testing identified the presence of recycled engine oil bottoms (REOB) in the sample, this was later confirmed through discussion with those involved in the project. REOB isn't the only softening agent that has adverse effects on performance, similar behavior was observed by MTE when paraffinic base oils were used in laboratory studies to formulate a PG 58-28 from a PG 64-22 base asphalt. This data was presented at a fall 2015 FHWA Binder Expert Task Group meeting. There is also corroborating field data from a project constructed in 1994 on STH 53. Data recovered from cores taken after 4 years in-service is presented in Table 1. There is a clear dose response with the presence of paraffinic base oil and only after 4 years in-service both modified asphalts had ΔT_c values below the –5.0°C failure limit. Unfortunately detailed field cracking data was not tracked for this project.

TABLE 1 Recovered Binder Results from 4-Year-Old Cores Taken from STH 53

BINDER	BBR, S CRITICAL (°C)	BBR m CRITICAL (°C)	ΔT_c (°C)
58-28	-34.5	-35.4	0.9
58-34 (58-28 + 3% PARAFFINIC OIL)	-38.1	-33	-5.1
58-40 (58-28 + 5% PARAFFINIC OIL)	-43.1	-34.9	-8.2

The cooperation of Minnesota Department of Transportation (MnDOT) in supplying cracking performance data and materials for this investigation is acknowledged, further information related to these test sections and the work conducted by MTE Corp. are available in the following references.

Worel, B. J., T. R. Clyne, T. R. Burnham, D. M. Johnson., and D. M. Tompkins. Low-Volume-Road Lessons Learned: Minnesota Road Research Project. *Transportation Research Record: Journal of the Transportation Research Board, No. 1989*, 2007, pp. 198–207.

Reinke, G., A. Hanz, D. Herlitzka, S. Engber, and M. Ryan. Further Investigations into the Impact of REOB and Parafinnic Oils on the Performance of Bituminous Mixtures. Presented at FHWA Binder Expert Task Group Meeting, Fall River, Mass., 2015.

The three primary objectives of this presentation are presented in Slide 6. These include laboratory investigation of long-term binder and mixture aging using both rheology and chemical composition, applications of concepts discussed to forensic analysis of field sections, and verification of trends using the torsion bar testing geometry. The mixture aging investigation is focused on evaluation of recovered binder properties after various aging protocols which included both different aging temperatures and times on mixes that included approximately 25% binder replacement from RAS. Torsion bar testing was used to verify that the trends observed in recovered binder grading results were representative of mixture performance and not an artifact of the 100% blending of virgin and recycled binders caused by the recovery process.

The test procedures and parameters used to evaluate asphalt binder and mixture rheology, as well as asphalt binder chemical composition are summarized in Slides 7 through 11. Rheological characterization of the asphalt binders was conducted using the 4-mm DSR parallel plate geometry analysis parameters included R-value, low-temperature binder grade, and ΔT_c. The 4-mm parallel plate was developed by Western Research Institute (WRI) as a tool to generate asphalt binder master curves and estimate low-temperature PG grade. The small geometry allows for direct measurement of rheological properties at low temperatures and the small sample size is conducive to testing of binder recovered from mixes or field cores. The R-value is a well-established shape parameter of the mastercurve that was introduced in the CA model as part of the original SHRP research and has been related to cracking. Schematics demonstrating the change in R-value and low-temperature grade with aging were included in the presentation. References for derivation of the R-value and the 4-mm parallel plate development and application to low-temperature grading are provided in the references section (*4*–*6*). Detailed discussion of the *R*-value was also included by Geoff Rowe in this workshop.

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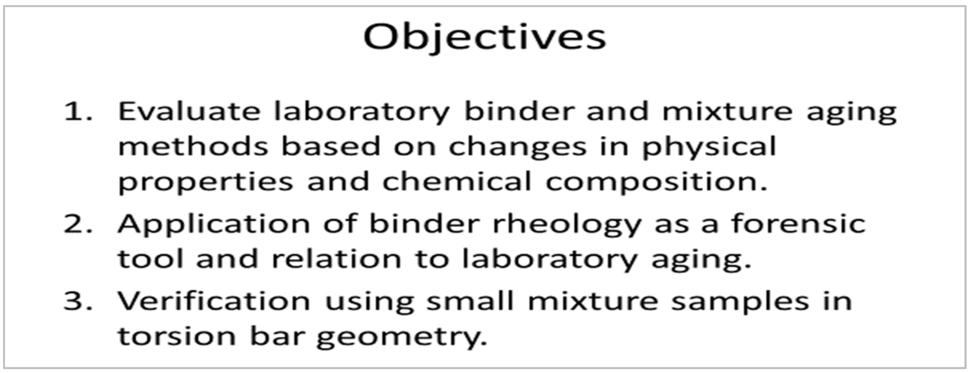
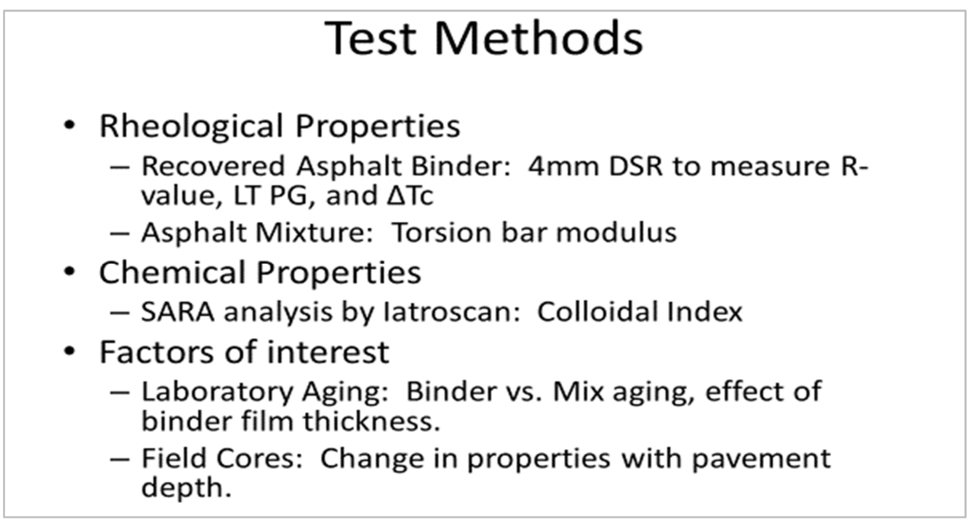
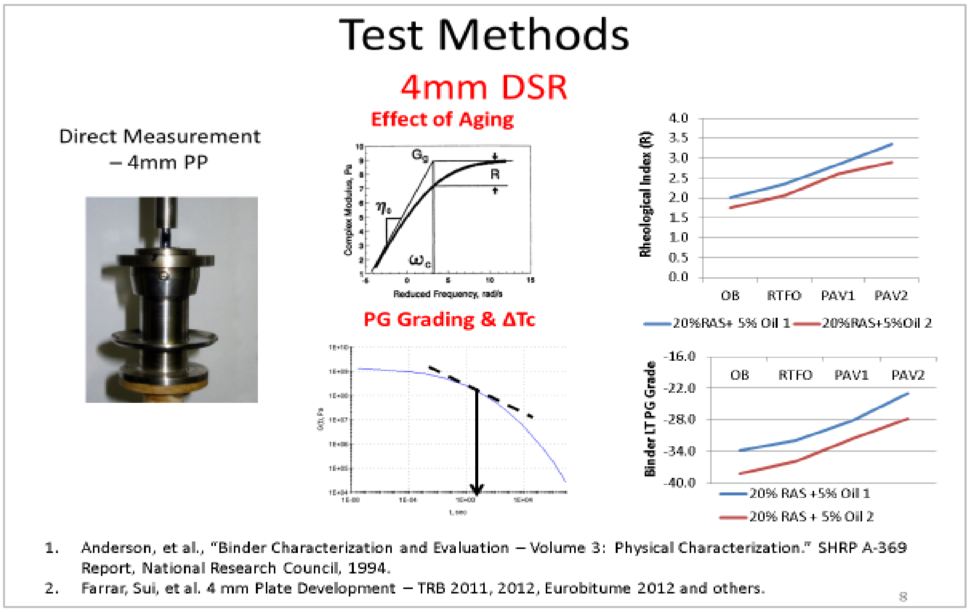


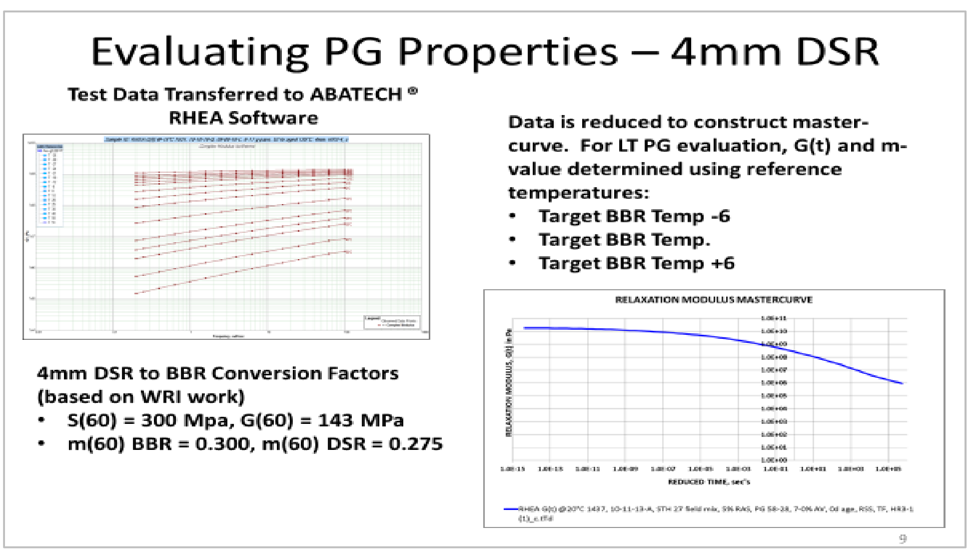
SLIDE 6



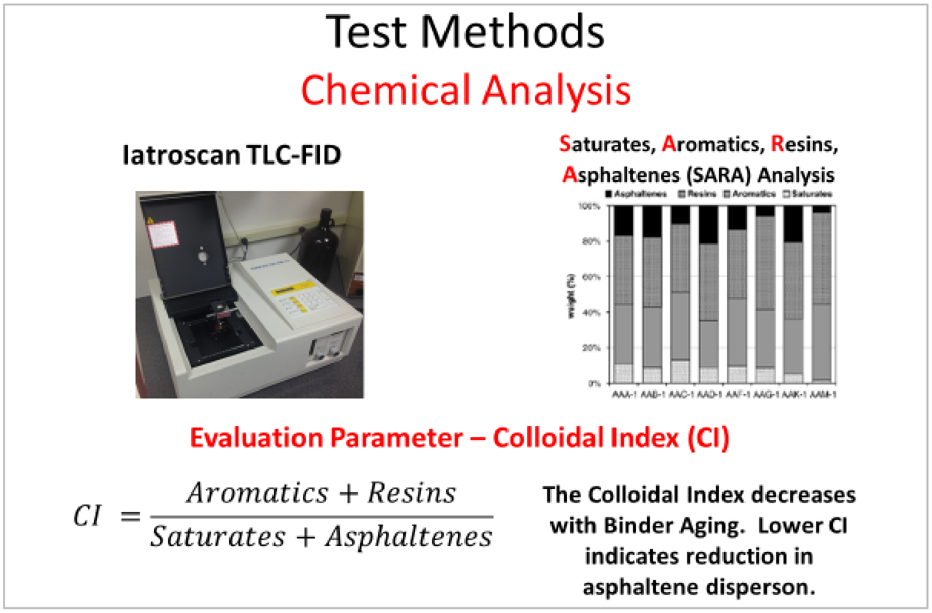
SLIDE 7



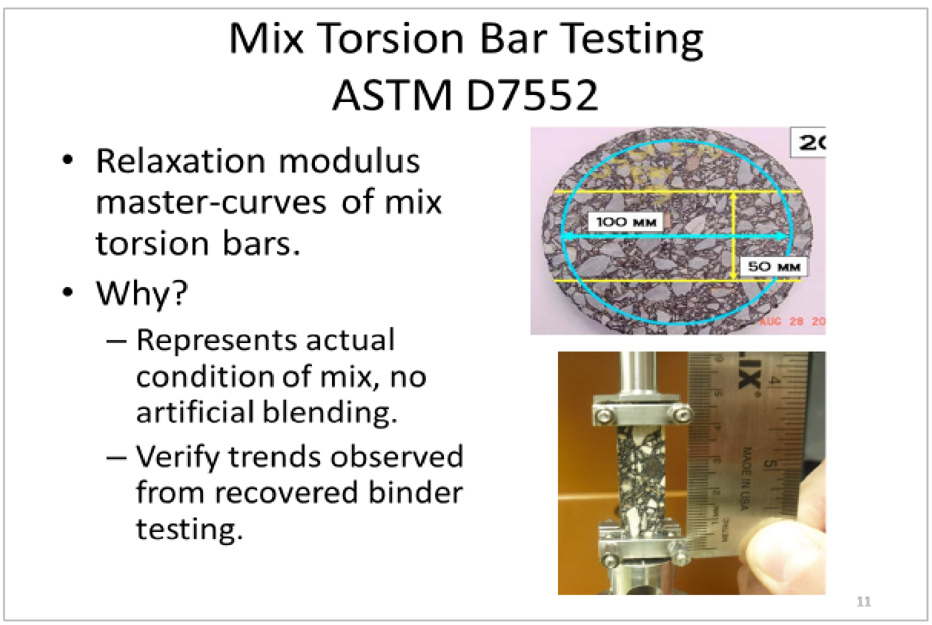
SLIDE 8



SLIDE 9



SLIDE 10



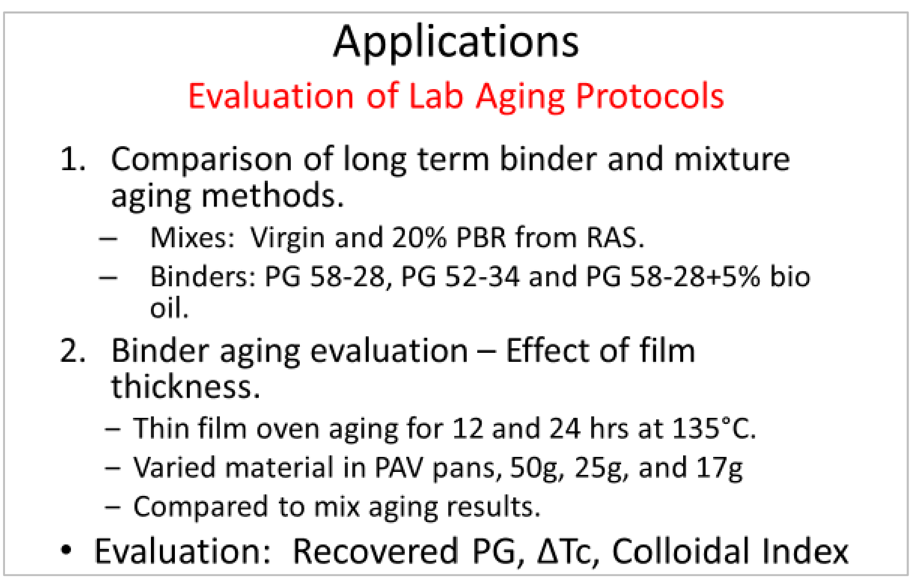
SLIDE 11

Based on the WRI publications MTE has developed an internal test and analysis procedure for the 4-mm parallel plate geometry, which has been shared with the FHWA Asphalt Binder Expert Task Group. The procedure involves a frequency sweep from 0.2 to 100 rad/s at isotherms ranging from –36°C to 50°C, above 0°C isotherms are in 10°C increments with a test included at 25°C. Below 0°C tests are in 6°C increments from 0°C to –18°C and 3°C increments at colder temperatures for better estimation of low-temperature PG. Strain levels are also varied by isotherm so testing remains in the linear visco-elastic region. To generate the master-curve the data is reduced using the RHEA software developed by Abatech. R-value is determined at a reference temperature of 25°C. Low-temperature PG is estimated using the relaxation modulus and m-value at three reference temperatures centered at the low PG testing temperature (LT PG +10°C). Conversion between relaxation modulus [G(t)] and BBR properties is conducted using the following relations: for S(60) at 300 MPa, G(60) = 143 MPa and for BBR m(60) = 0.300, m(60) –4 mm = 0.275. These factors vary slightly from those published by WRI but were found to provide a better correlation between low-temperature continuous grade measured by the BBR and 4-mm test geometries. The utility of the 4-mm plate is it allows for low-temperature continuous grading with one test, given that calculation of the ΔT_c parameter requires both the S(60) and m(60) failure temperatures this represents a considerable time savings as the spread between S-controlled and m-controlled failure increases due to aging or the materials studied.

Chemical composition was measured using the Iatroscan TLC-FID to determine the distribution of the saturates, aromatics, resins, and asphaltenes of the asphalt binder either aged or extracted from aged loose mix. Changes in chemical stability with aging were quantified using the Colloidal Index, which is a measure of asphaltenes dispersion, as materials age the Colloidal Index decreases. This parameter was included to investigate relationships between changes in asphalt binder rheology and chemistry with aging and the effects of different aging methods.

Torsion bar testing was conducted in accordance with ASTM D7552 to develop relaxation modulus mastercurves. The test procedure is similar to what was discussed previously for the 4-mm DSR in regards to temperature ranges of testing and varying strain levels to remain in the visco-elastic region.

An overview of laboratory based efforts to evaluate aging protocols for asphalt binder and mixture aging is provided (Slide 12). To evaluate the effects of recycled materials and the use of softening agents the study included a virgin mix and mix that had 20% binder replacement from RAS. The mixes were prepared with three asphalt binders, PG 58-28, PG 52-34, and a PG 58-28 + 5% binder oil with a resulting grade of PG 52-34. In practical applications, the use of PG 58-28 represents a conventional binder in some states, for example Wisconsin allows up to



SLIDE 12

20% binder replacement with RAS without adjustment to binder grade. The PG 52-34 binders derived from either straight asphalt or using a softening agent were selected to represent the concept of "grade dumping" to better accommodate the use of recycled materials. The primary analysis was conducted on recovered binder rheological and chemical properties. A small subset of the data set was investigated using torsion bar relaxation modulus properties to verify findings. The data presented in this section pertains to Objectives 1 and 3 in Slide 6.

The study included three different laboratory aging protocols: recovered binder + PAV aging, loose mix aging, and compacted mix aging (Slide 13). For all three protocols extended aging was investigated to evaluate differentiation between materials. Recovered binder was subjected to 20- and 40-h PAV aging to represent the current aging protocols in M320/M332 and the extended aging used in the developmental work for the ΔT_c parameter by the Asphalt Institute previously referenced. Compacted mix aging was extended to durations beyond the standard 5 days in AASHTO R30 based on relationship with 10-day aging noted in the MnROAD 1999 study and the general concern that the current AASHTO R30 protocol was not sufficient to represent long-term aging. Loose-mix aging was conducted at two different temperatures and time scales. Accelerated loose-mix aging at 135°C on a time scale of hours was selected based on work by Braham and subsequent efforts by the Asphalt Institute led by Phil Blankenship. Loose-mix aging at a lower temperature (95°C) and an extended duration of 20 days was included in the study based on in-progress research for NCHRP 9-54 and the concern that accelerated loose mix aging may cause changes in chemistry or rheology that do not represent field behavior. All loose mix aging was conducted in the same oven using a 2-in. layer of mix. The reference to the loose mix aging protocol used by Braham is: Braham A., W. Buttlar, and T. Clyne. The Effect of Long-Term Laboratory Aging on Hot-Mix Asphalt Fracture Energy. *Journal of the Association of Asphalt Paving Technologists,* Vol. 78, 2009, pp. 371–402.

Evaluation of Lab Aging Protocols

Long Term and Extended Aging Procedures

Aging Method	Aging Condition
Loose Mix + PAV	As-Recovered + PAV (Blending Chart)
	As-Recovered + 2PAV
Loose Mix	12 hrs at 135°C
	24 hrs at 135°C
	480 hrs (20 days) at 95°C
Compacted Mix	10 days at 85°C
	20 days at 85°C

- All mixes were aged in the same oven.

13

SLIDE 13

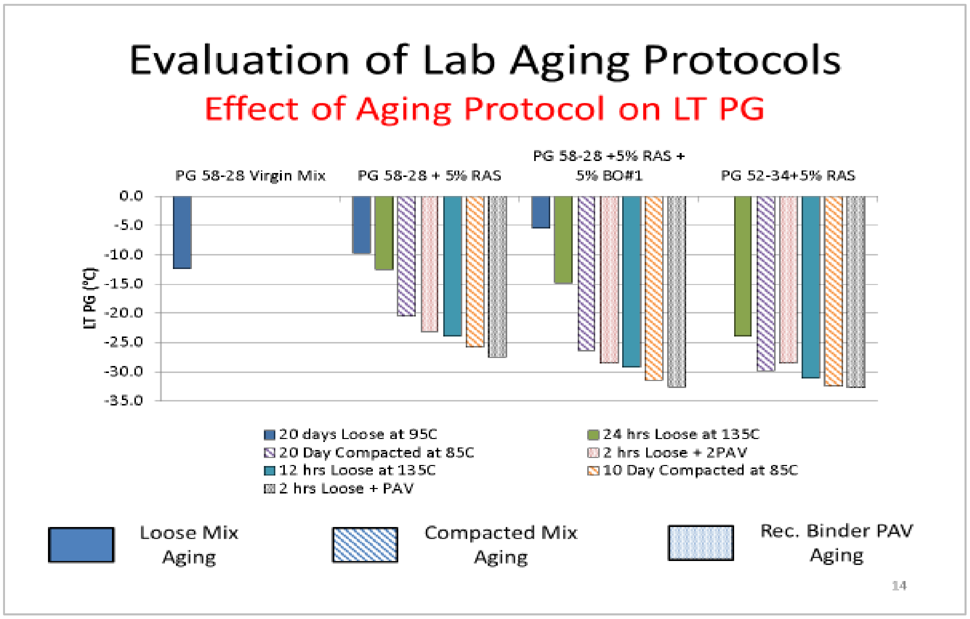
Recovered binder properties after the prescribed laboratory aging protocols are presented for low-temperature PG (Slide 14) and the ΔT_c parameter (Slide 15). The data are presented as bar charts with the performance parameter on the y-axis and the different mix types evaluated on the x-axis. The response to different long-term aging protocols are grouped for each mix, with each loose-mix aging type, loose mix + PAV, compacted mix, and loose mix assigned a different pattern in the legend key. For the mixes including RAS three different groupings related to the severity of the aging protocol are evident, ordered from least severe to most severe are (1) Rec. Binder + PAV and 10-day compacted mix aging at 85°C; (2) Rec. Binder + 2PAV, 20-day compacted mix aging at 85°C and 12-h loose-mix aging at 135°C; and (3) 24-h loose-mix aging at 135°C and 20-day loose-mix aging at 95°C. For the purposes of discussion these three groups will be defined as standard, intermediate, and extreme aging. This study was designed assuming a base climate grade of PG 58-28, results of low-temperature grading confirm the need to adjust binder grade for increased use of recycled materials as mixes aged up to the intermediate condition with the PG 52-34 or PG 58-28 + 5% bio oil binders meet or exceed the –28°C low-temperature grade requirement, whereas the unmodified PG 58-28 fails the requirement by ½ to a full PG grade. Significant increases in low-temperature binder grade and corresponding decreases in ΔT_c were observed for the extreme aging conditions. In regards to the two extreme aging conditions the loose mix aging for 20 days at 95°C was more severe than the 24-h loose-mix aging at 135°C for the three mixes tested.

With extended aging the PG 52-34 binder proved more stable than the PG 52-34 that was obtained by modifying the PG 58-28 with bio oil, differentiation in ΔT_c between the materials begins with the intermediate aging condition with the ΔT_c values of the bio oil-modified binder becoming more negative and having similar performance of the unmodified PG 58-28. At extreme aging conditions the largest change in both low-temperature grade and ΔT_c were observed for the bio oil-modified binders and the final properties are again comparable to the unmodified PG 58-28, whereas the performance of the PG 52-34 was substantially better. Based on these findings, for this particular formulation there is a diminishing improvement in low-temperature and relaxation properties for the bio oil-modified materials. However, defining the duration that these laboratory-aging protocols represent in the field is a topic of in-progress research.

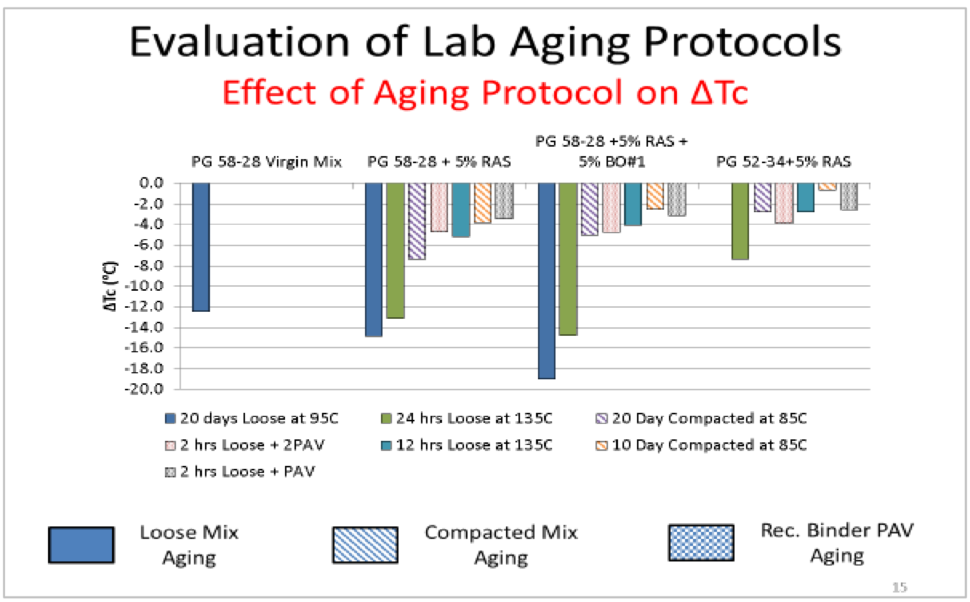
The relationship between chemical composition and asphalt binder rheology as measured by the Colloidal Index and ΔT_c parameters is presented (Slide 16). The most significant outcome of this plot is that all of the different aging protocols evaluated follow the same exponential relationship, save for one outlier in the 24-h loose-mix aging condition. Based on this relationship the changes in rheology correspond to changes in chemistry including loose mixes aged at both 95°C and 135°C. For the materials studied the accelerated loose-mix aging protocol did not cause an extreme change in response to aging.

Slide 17 summarizes the observations of the laboratory-aging protocol investigation that should be self-explanatory based on the presentation and discussion provided above.

Torsion bar testing was employed on the compacted mix specimens aged for 20 days at 85°C to verify that the observations from the laboratory aging study were representative of mixture behavior and not an artifact of forced blending caused by the extraction and recovery process (Slide 18). Six mixes were selected all had 20% binder replacement from RAS and used different binder formulations including PG 58-28, PG 52-34, and PG 58-28 modified with various bio-based additives to make a PG 52-34 grade. The figure shows the relaxation modulus master curves at a reference temperature of 25°C for all the materials tested and highlights two mixes with different behavior, PG 58-28 and PG 52-34 with 20% RAS binder replacement. These mixes were highlighted



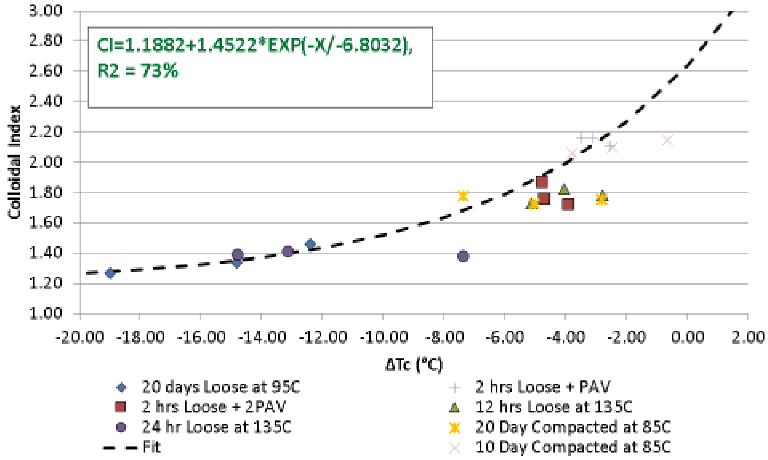
SLIDE 14



SLIDE 15

Evaluation of Lab Aging Protocols

Relationship Between Composition and ΔTc



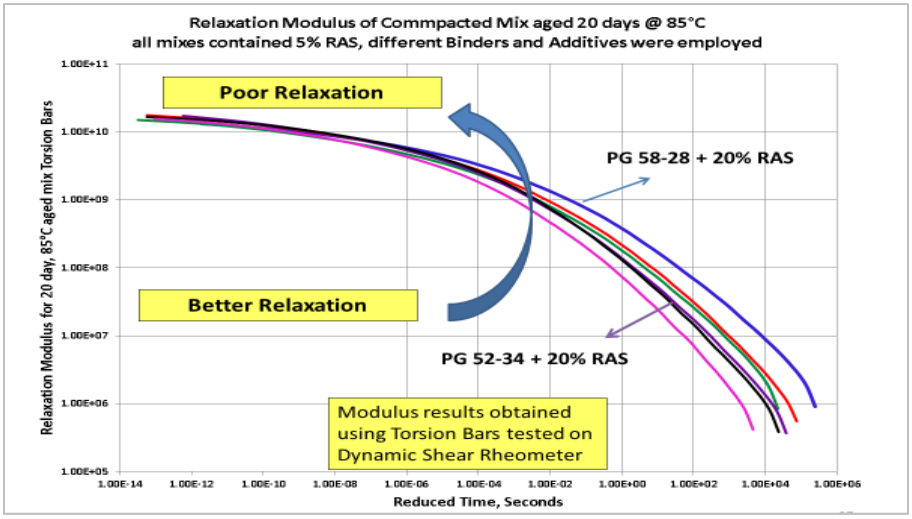
SLIDE 16

Evaluation of Lab Aging Protocols

Observations

- Extended mixture aging was better able to discriminate between mixes than "intermediate aging."
  - Loose mix aging was more severe than compacted mix aging.
  - 20 Day aging at 95°C was more severe than accelerated (135°C for 24 hours) aging.
- Loose mix aging at intermediate and extended conditions was found to discriminate more between materials than PAV aging of recovered binder.
- No benefit of additives observed in maintaining low temperature PG with extended aging. PG 52-34 was marginally better than BO #1 at intermediate aging and substantially better after extended aging.
- A general trend was observed between ΔTc and Colloidal Index across all aging protocols.

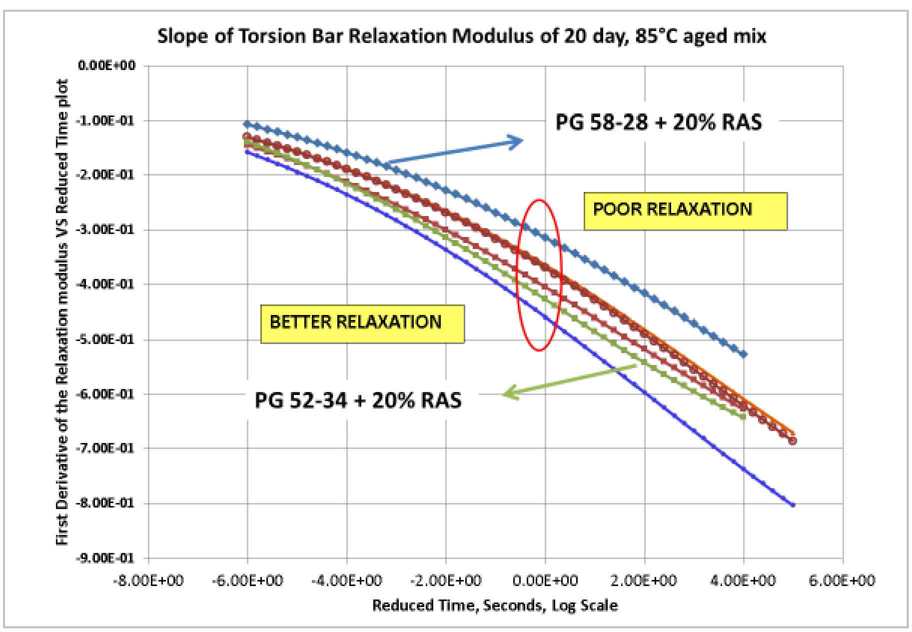
SLIDE 17



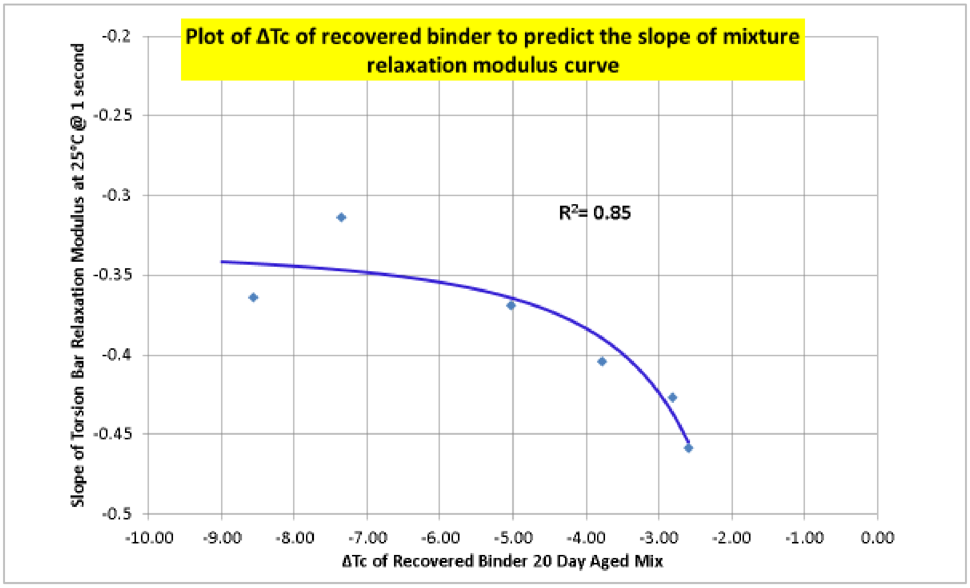
SLIDE 18

to accent a schematic of how to interpret relaxation modulus curves, where poor relaxation properties are identified as a positive rotation of the relaxation modulus curve. Therefore as the slope of the relaxation modulus decreases the relaxation properties of the material worsen.

For direct comparison of the materials the first derivative of the relaxation modulus was determined and plotted versus reduced frequency (Slide 19). Results for all mixes are presented along with specific reference to the PG 58-28 and PG 52-34 binder called out in the previous slide. This method of data analysis clearly demonstrates the range of relaxation slopes observed for the materials tested after extended compacted mix aging. At the loading time of 1s highlighted in the figure relaxation slopes varied by a factor of 1.5 from –0.3 to –0.45 Pa/s.

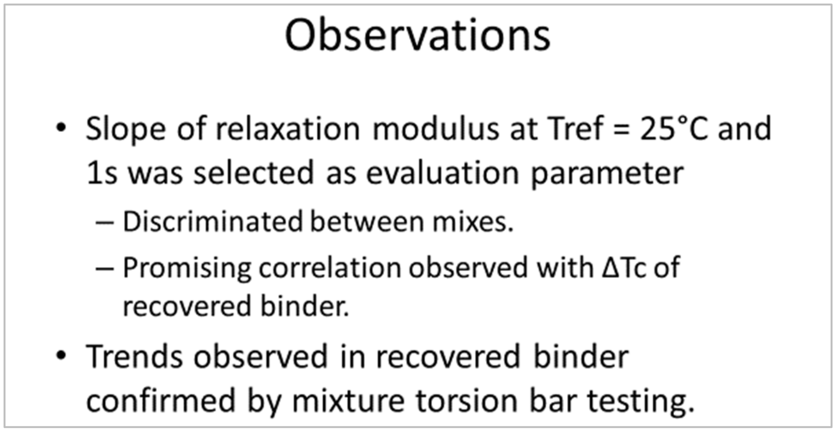


SLIDE 19



SLIDE 20

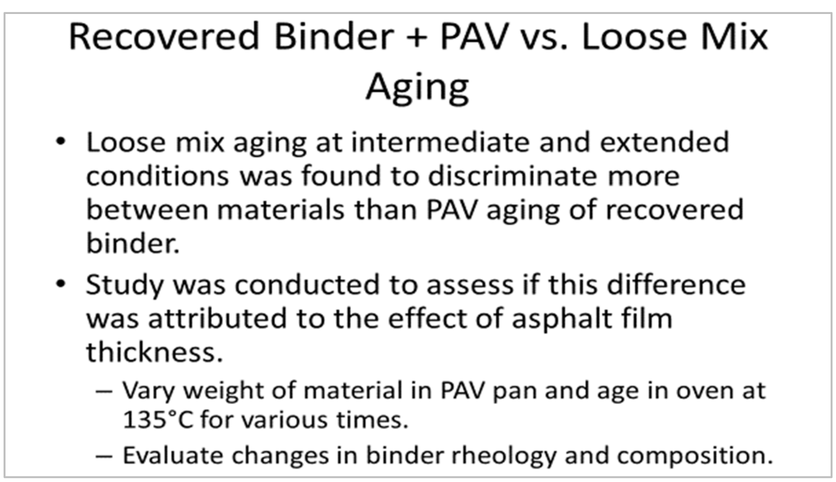
The relationship between the torsion bar relaxation slope, plotted on the y-axis, and the recovered binder ΔT_c, plotted on the x-axis, was used to verify the findings of the laboratory aging study. An exponential relationship between mixture torsion bar relaxation slope and recovered binder ΔT_c with a R^2 value of 0.85 was observed for the six data points tested (Slide 20). Furthermore, the torsion bar relaxation slope verifies the ΔT_c warning and failure limits of –2.5°C and –5.0°C originally proposed in the AAPT publication by Anderson previously referenced. At ΔT_c values below –5.0°C relaxation slope trends to a value of –0.35 Pa/s indicating poor relaxation properties and thus increased potential for cracking. For ΔT_c greater than –5.0°C relaxation modulus slope values range from –0.36 Pa/s to –0.45 Pa/s and improve (decrease) as ΔT_c increases from –5.0°C to –2.5°C. Further work on this topic has been completed since this presentation and was recently summarized at the FHWA May 2017 Asphalt Binder Expert Task Group (ETG) meeting in Ames, Iowa (Reinke, G., and A. Hanz. Extended Aging of RAS Mixes with Rejuevnator—An Update. *Proc., FHWA Asphalt Binder ETG*, Ames, Iowa, 2017. http://www.asphaltpavement.org/PDFs/Engineering_ETGs/Binder_201705/12_Reinke&Hanz_UpdateExtendedAgingofRAS.pdf).



SLIDE 21

Self-explanatory, Slide 21 summarizes the observations from torsion bar relaxation modulus testing and investigation of the relationship between relaxation modulus slope and ΔT_c.

Data generated in the comparison of laboratory aging protocols of mixes with 20% binder replacement from RAS demonstrated that the loose-mix aging protocols were more severe than extended (up to 40 h) aging of recovered binder in the PAV (Slide 22). In the original experimental design it had been assumed that 40-h PAV aging would produce similar results to 24-h loose-mix aging. A study was conducted to investigate the impact of film thickness on aging and to compare binder aging of various film thicknesses to the loose-mix aging results previously discussed. For the study PG 58-28 binder was blended with 20% RAS recovered RAS binder and aged in PAV pans at a variety of film thicknesses and aging times in a forced draft oven at 135°C. Similar to the mixture study PG 58-28 binder with no RAS replacement was included and aged for 20 days at 95°C. Film thickness was varied by controlling the amount of material in the pan. Specifically masses of 50, 25, and 17 g and aging times of 12, 24, and 52 h were used. A detailed summary of the combinations of materials, weights, and aging times is provided in Slide 23.



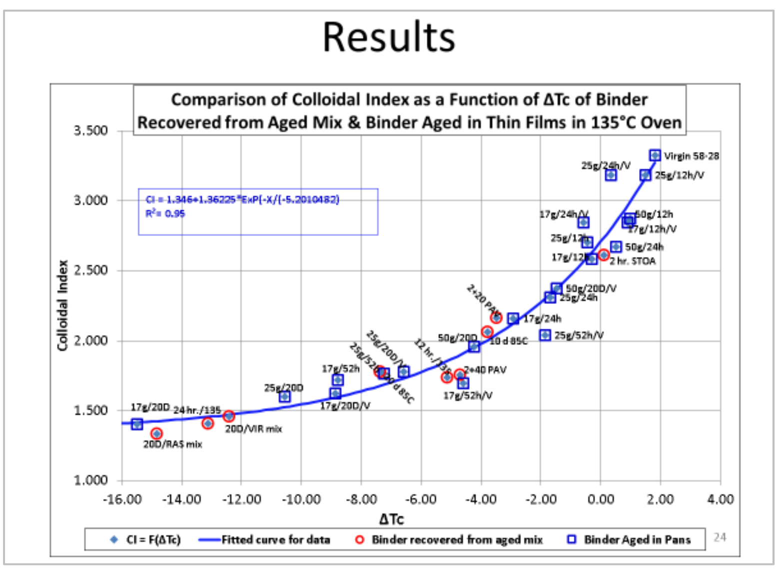
SLIDE 22

Summary of Film Thickness Study

PG 58-28 + 5% RAS, Mix, various aging, binder recovered from mixes and tested or PAV aged and tested		PG 58-28 + 20% Binder from tear off shingle aged in PAV pan, 135°C, forced draft oven	
Lable Name	Lable Info	Lable Name	Lable Info
2 hr	2 hr. STOA @ 135°C	50/12	50 g aged 12 hrs.
2 +20 PAV	2 hr. STOA + 20 hr. PAV	25/12	25 g aged 12 hrs
2 + 40 PAV	2 hr. STOA + 40 hr. PAV	17/12	17 g aged 12 hrs.
12 hr	12 hr. loose mix age @ 135°C	50/24	50 g aged 24 hrs.
24 hr	24 hr. loose mix age @ 135°C	25/24	25 g aged 24 hrs.
10 d 85C	Pills aged 10 days @ 85°C	17/24	17 g aged 24 hrs.
20 d 85C	Pills aged 20 days @ 85°C	25/52	25 g aged 52 hrs.
		17/52	17 g aged 52 hrs

SLIDE 23

Similar to Slide 16, the relationship between Colloidal Index and ΔT_c was plotted to assess the effects of all binder and mixture aging conditions evaluated (Slide 24). In the figure results from binders recovered from loose mix aging are indicated by red circles and results of thin film binder aging are identified using blue squares. As shown results follow an exponential relationship for all binder and loose mix aging conditions, indicating that the changes in rheology are associated with changing chemical composition. The relationship presented also demonstrates the adjustments in binder aging time and/or film thickness required to match a given loose-mix aging condition. At the film thickness that corresponds to a weight of 50 g in the PAV pan 24-h aging is required to match the chemistry and rheology of a short-term aged mixture. To represent an extended aging condition, thinner binder films are needed. Specifically aging 25 g of binder for 52 h produces similar results as binder recovered from loose mix + 2 PAV aging, loose mix aging at 135°C for 12 h, or compacted mix aged at 85°C for 10 days.

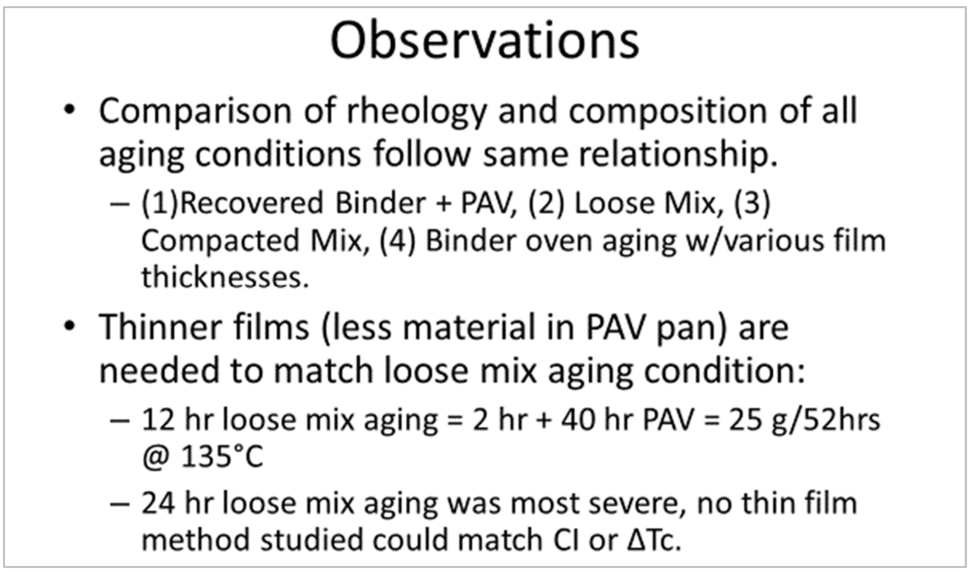


SLIDE 24

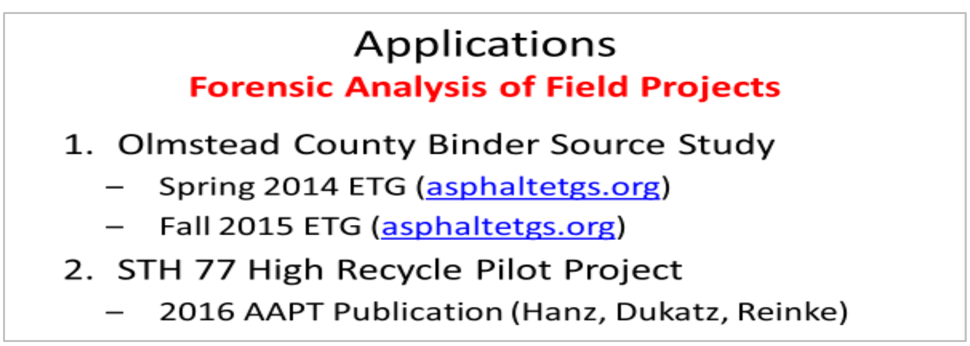
Similar relationships can be drawn to compare any binder and loose-mix aging condition of interest. The main takeaway from this figure is that the binder film thickness experiments verified the finding of laboratory mix aging work that there is potential that accelerated aging can be used to assess aging stability with minimal risk that the aging conditions will cause severe changes in chemical composition or rheological response.

Slide 25 includes summary of findings of the film thickness study, discussed previously.

Objective 2 of Slide 26 was to discuss how the concepts presented were applied to forensic analysis of field sites. This slide introduces two projects the Olmstead County Binder Source study and the Wisconsin State Highway (STH) 77 High Recycle Pilot project. Additional details related to the projects and associated research is available in the following references:



SLIDE 25



SLIDE 26

Olmstead County Binder Source Study

Pauli, A. T., M. J. Farrar, and P. M. Harnsberger. Material Property Testing of Asphalt Binders Related to Thermal Cracking in a Comparitive Site Pavement Performance Study. *Proc., 7th RILEM Conference on Cracking in Pavements Mechanisms, Modeling, Testing, Direction, and Prevention Case Histories*, Delft, Netherlands, 2012, pp. 233–242.

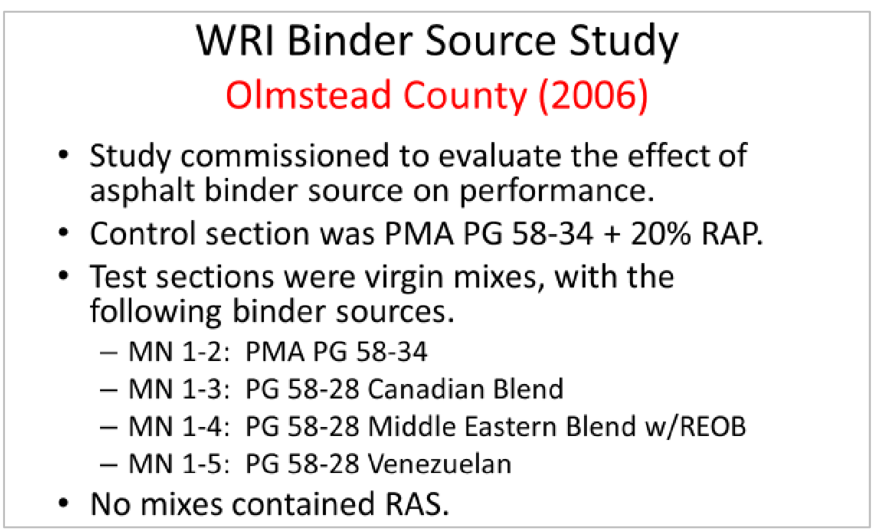
Presentation of initial MTE Research at 2014 FHWA Mixture ETG. Available at http://www.asphaltpavement.org/PDFs/Engineering_ETGs/Mix_201409/Reinke_Performance%20of%20Binders%20Blended%20with%20Additives%20for%20Reducing%20Low%20Temp%20Properties%20of%20Asphalt%20Binders.pdf.

Follow up Presentation of MTE Research at 2015 FHWA Binder ETG. Available at http://www.asphaltpavement.org/PDFs/Engineering_ETGs/Binder_201504/03%20REINKE%20ETG%20PRESENTATION%20INTO%20REOB%20&%20OTHER%20PARAFFINIC%20OILS%20BINDER%20ETG%204-9-15%20with%20notes.pdf.

Wisconsin STH-77 High Recycle Pilot Project

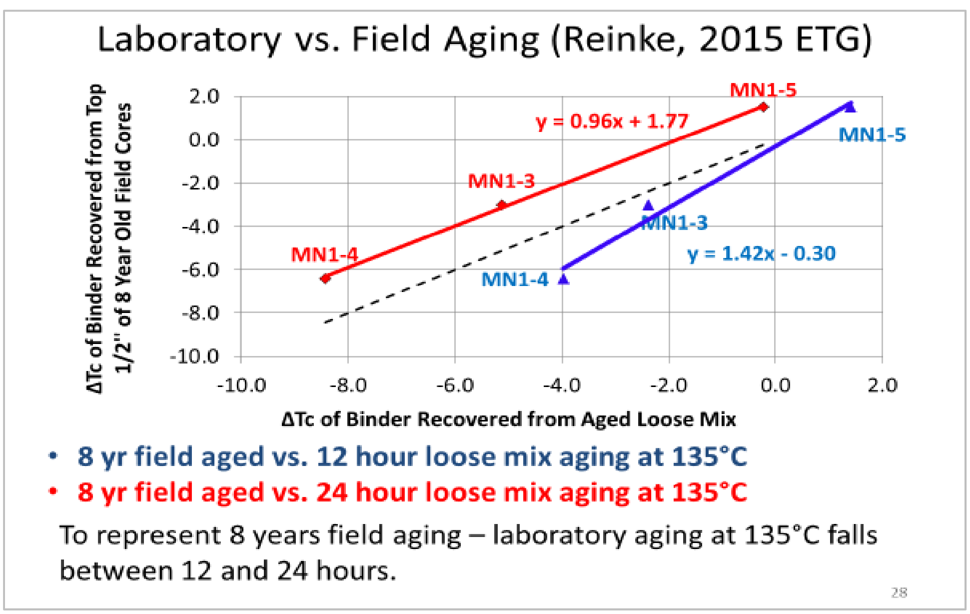
Hanz, A., E. Dukatz, and G. Reinke. Use of Performance Testing for High RAP Mix Design and Production Monitoring. *Asphalt Paving Technology*, Vol. 85, 2016, pp. 449–484.

The Olmstead County project was constructed by Mathy Construction as part of a research effort led by WRI in 2006 as a study to investigate the impacts of asphalt binder source on performance (Slide 27). The control section was a PMA PG 58-34 with 20% RAP. The four test sections were virgin mixes that used asphalt from different sources including a PG 58-34 (Canadian blend) (MN1-2) and PG 58-28 from three different crude sources, Canadian (MN1-3), Middle Eastern (Kirkuk) (MN1-4), and Venezuelan crude blend (MN1-5). Subsequent discussion and evaluation identified that the Middle Eastern source included REOB. Asphalt binder was sampled during the project by MTE and loose mix sampled by WRI during construction was provided to MTE for the follow-up research effort summarized in this presentation.

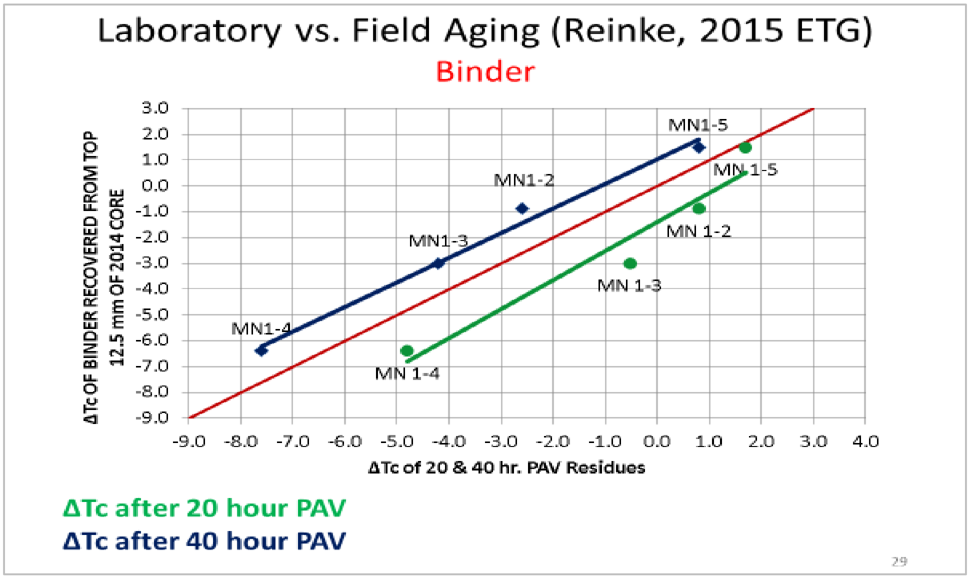


SLIDE 27

A field distress survey was conducted and each test section was cored in fall 2014, representing 8 years of field performance. For comparison of laboratory to field aging asphalt binder was extracted and recovered from the top half-inch of the field cores and characterized using the 4-mm DSR. Relationships between field aging and laboratory loose mix and binder aging are presented in a similar manner in Slides 28 and 29. The ΔT_c of the binder extracted from the 8-year old field cores is presented on the y-axis and the ΔT_c values of the laboratory-aged loose mix or binder samples are presented on the x-axis. On each plot, the line of equality between laboratory and field aging is provided. Loose mix was aged for 12 and 24 h at 135°C which are represented by blue and red lines in the Slide 28 and blue and green lines in Slide 29. The two laboratory aging conditions fall on either side of the line of equality, meaning that 12-h loose-mix aging under predicts 8 years of field aging and the field aging is over predicted by 24-h loose-mix aging. The slope of the 12-h loose mix line is steeper than that of the line of equality. As a result, for the two binders (MN 1-5 and MN 1-3) with less negative ΔT_c values the 12-h aged and field-aged properties are similar; for binder MN 1-4, ΔT_c is under predicted by approximately 2.0°C. The 24 loose mix line is parallel to the line of equality and ΔT_c is over predicted by approximately 1.5°C for all three binder sources. Asphalt binder was PAV aged for 20 and 40 h and compared to field aging in Slide 29. In this plot both aging conditions are relatively parallel to the line of equality, 20-h PAV aging under predicts field aging by 2.0°C to 3.0°C and 40-h PAV aging over predicts ΔT_c by approximately 2.0°C. Results of binder aging and mixture aging were consistent in that to represent 8 years field aging in Minnesota a laboratory conditioning time between the aging treatments selected is required.



SLIDE 28



SLIDE 29

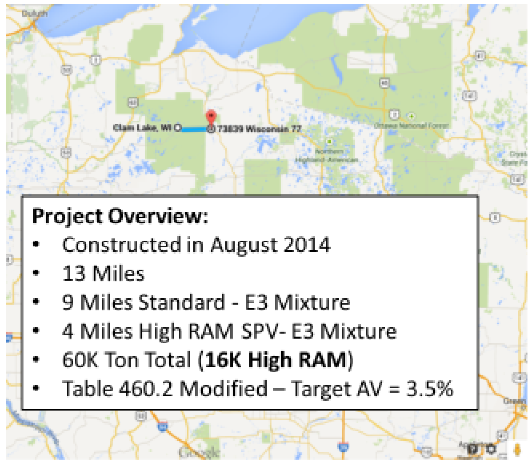
Differing in-service performance was observed in the test sections that ranked well with the range of ΔT_c values of the binder recovered from the field cores, detailed data is in the ETG presentation referenced above. This corroborates the concept presented in Slide 4 that the durability properties of asphalt binders that meet the same PG grade and thus their performance can vary significantly based on source. It is also interesting to note the similarity in ΔT_c values between loose mix and binder aging observed for the virgin mixes used in the Olmstead County study compared to the higher severity observed for loose mix aging of mixes containing RAS in the laboratory aging data discussed in Slides 13 through 25.

An overview of the STH 77 high recycled asphalt materials (RAM) pilot project is provided (Slide 30). The project was constructed in 2014 as part of a pilot organized by the Wisconsin Department of Transportation to investigate the use of high recycled mixes. Two test sections were included the control which contains 25% binder replacement and the high recycle section that has 37% binder replacement, in both sections all binder replacement was from locally sourced RAP. The project was located in northern Wisconsin and is in a PG 58-34 climate. For the high RAP mix a PG 52-40 binder that included a bio-based additive was supplied. The high RAM section was also subject to a pilot mixture performance testing specification. Mix was sampled during production and the project was cored in 2015 and 2016, representing 1 and 2 years after construction.

Forensic analysis of the STH-77 project followed an approach similar to what was used for the Olmstead County project previously summarized (Slides 31 through 33). The properties of the top ½ in. of the binder recovered from field cores taken 15 and 22 months after construction were compared to binder recovered from plant produced loose mix aged at 135°C for 12 and 24 h. Recovered binders were tested using the 4-mm DSR to determine the low-temperature grade and ΔT_c. Results are displayed as bar charts, with the performance property on the y-axis and the field or lab aging condition on the x-axis. The high RAM and control mixes are plotted together for a given coring time or loose-mix aging condition. After 22 months field performance the low-temperature grade of the control and high RAM mixes are similar and both

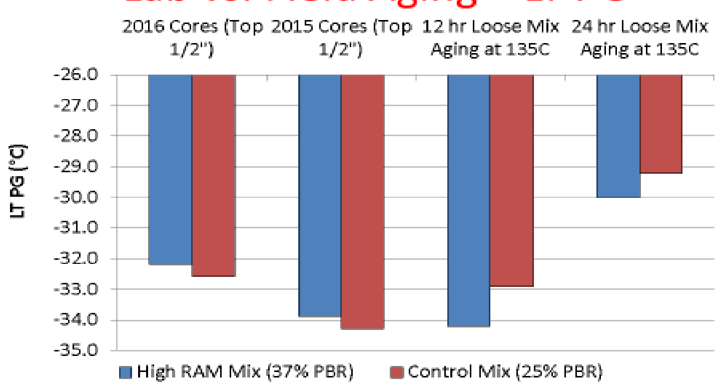
STH 77 High RAM Pilot Project
Overview

- Two mixes
  - All binder replacement from RAP
  - High RAM (37% PBR)
  - Control (25% PBR)
- Plant produced mix lab aged for 12 and 24 hours at 135°C
- Two Coring Times
  - November 2015 (15 months)
  - June 2016 (22 months)



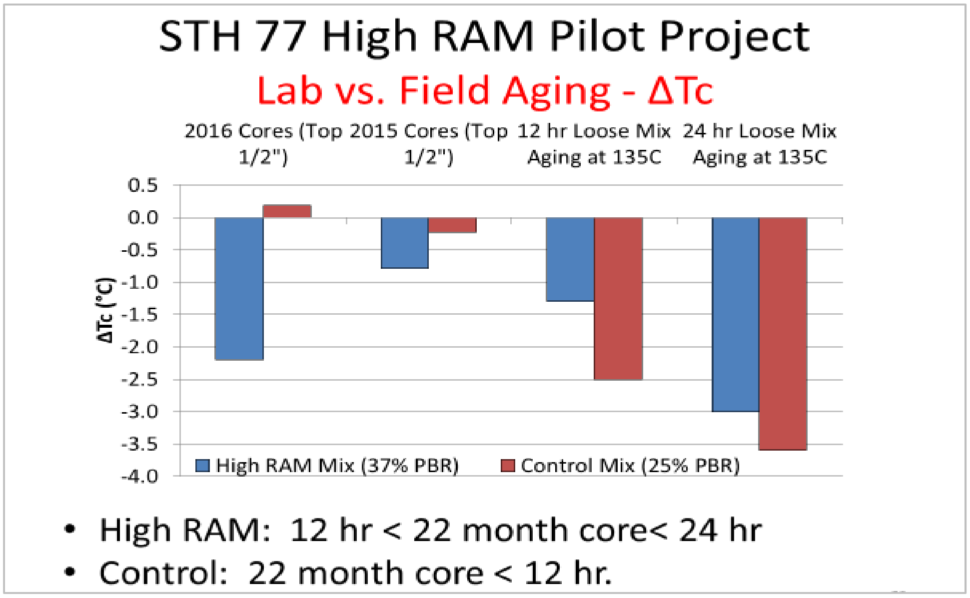
SLIDE 30

STH 77 High RAM Pilot Project
Lab vs. Field Aging – LT PG

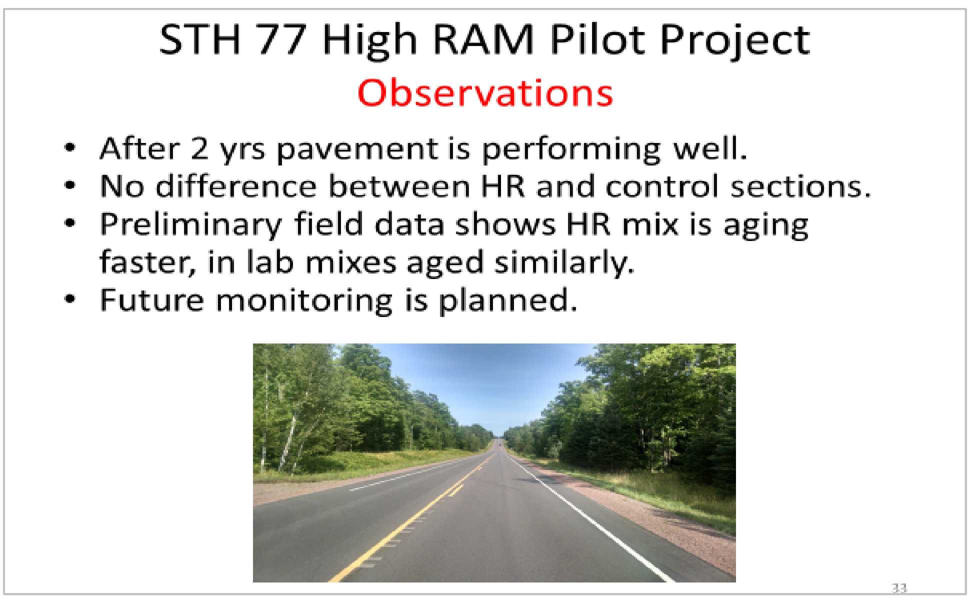


- High RAM: 12 hr Loose Mix = 15 month core.
- Control: 12 hr Loose mix = 22 month core.

SLIDE 31



SLIDE 32

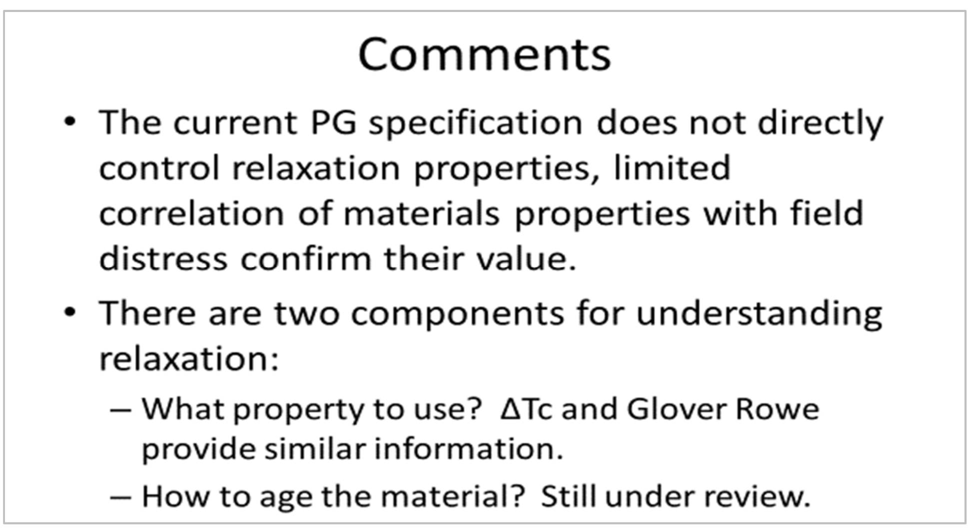


SLIDE 33

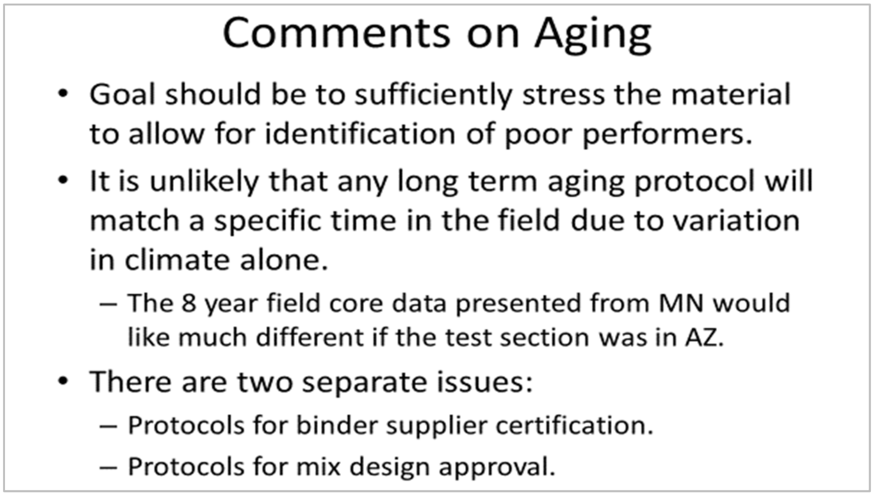
2°C warmer than the –34°C climatic target. Comparison of laboratory and field aging shows that loose-mix aging for 12 h at 135°C produces similar properties to the high RAM mix after 15 months in service and to the control mix after 22 months in service. The low-temperature grade of the 22 month field cores is approximately 3°C colder than the loose mix aged for 24 h in the laboratory. Significantly more differentiation between the control and high RAM mix was observed in comparing the ΔT_c. After 22 months in-service aging the control mix remained slightly S-controlled and had a positive ΔT_c. Conversely, the high recycle mix had a ΔT_c value of

–2.0, approaching the cracking warning limit. Relative to laboratory aging the ΔT_c of the 22-month high RAM mix fell between the results for 12- and 24-h loose-mix aging. Although preliminary data collected from field cores indicates that the high RAM mix is aging at a faster rate than the control, no performance differences were observed in a windshield survey of the two sections.

Cracking distress remains the primary performance concern for asphalt pavements, it would be short-sighted to attribute all of these issues to “dry” mixes caused by overly severe gyration levels in the mix design process or increased use of recycled materials. There is an opportunity to improve current asphalt PG binder specifications by including testing protocols to better control asphalt durability through use of a measure of relaxation properties at the appropriate aging condition. The limited data discussed in this presentation provides specific examples of controlled studies where materials that met or exceeded current PG requirements demonstrated significantly different field performance. Parameters including ΔT_c or G-R exist, both have cracking warning and failure limits but have yet to be incorporated into specifications (Slide 34). The most significant barrier to implementation of durability-related specifications appears to be aging as research attempts to quantify how an extended laboratory aging protocol in the PAV or on loose mix relates to aging in the field (Slide 35). This duration will vary significantly based on climate. As an intermediate step specification improvements should emphasize selecting an aging protocol that sufficiently stresses the material to differentiate between aging resistant and aging susceptible materials. Given the definition of a durability parameter and an aging protocol, the concept could be applied to both asphalt binder supplier certification to control aging susceptible sources or additives and a mix design approval test to address concerns with increased uses of recycled materials, particularly RAS.

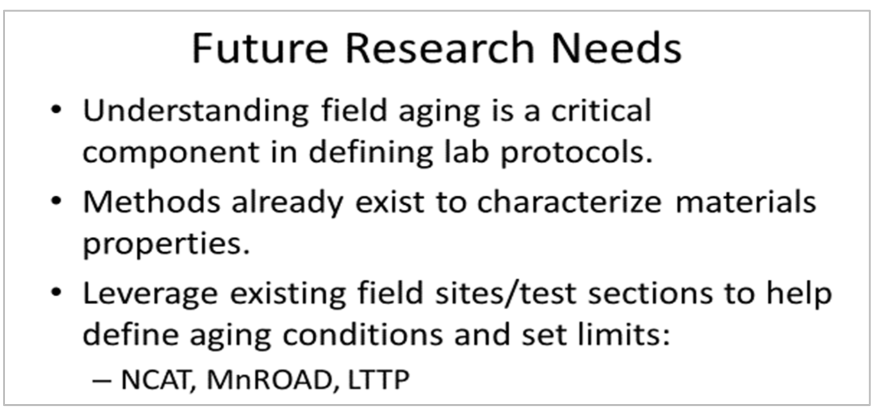


SLIDE 34



SLIDE 35

Although there has been significant progress made in this area research opportunities remain in refining laboratory testing protocols and limits associated with durability parameters. For example, the 4-mm DSR parallel plate geometry was used extensively in this study and is still in need of significant development and standardization for widespread adoption. Perhaps the most critical need is improved understanding of the relationship between laboratory and field aging, particularly how aging varies by climates. The concept of degree-days to normalize climatic variations is promising and between the numerous test sections available through MnROAD, the NCAT test track, and the Long-Term Pavement Performance Program there is an opportunity to leverage existing test sections to support further specification improvement (Slide 36).



SLIDE 36

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The **National Academy of Sciences** was established in 1863 by an Act of Congress, signed by President Lincoln, as a private, non-governmental institution to advise the nation on issues related to science and technology. Members are elected by their peers for outstanding contributions to research. Dr. Marcia McNutt is president.

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