STATE-OF-THE-KNOWLEDGE

USE OF THE

DELTA T_c PARAMETER

TO CHARACTERIZE ASPHALT BINDER BEHAVIOR

asphalt institute

IS-240

ASPHALT INSTITUTE TECHNICAL ADVISORY COMMITTEE

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1.0 INTRODUCTION AND PURPOSE

Delta T_C (ΔT_C) is a derived asphalt binder property that has been gaining attention for the last decade. It has become a topic of focus to both researchers in the asphalt binder technical community and user agencies seeking physical property parameters that will improve hot mix asphalt pavement performance. It is generally accepted that ΔT_C targets cracking behavior that is affected by asphalt binder durability related to aging of the binder in an asphalt mixture. More specifically, ΔT_C provides insight into the relaxation properties of a binder that can contribute to non-load related cracking or other age-related embrittlement distresses in an asphalt pavement. It is a calculated value using the results of the bending beam rheometer test determined on laboratory-aged asphalt samples or samples recovered from pavements. At the time this document was developed (mid-2019), ten user agencies have or soon will implement ΔT_C as part of their purchase specification, with two more expecting to do so in the near future. In addition, several national level research projects are actively considering ΔT_C as part of their studies. In fact, ΔT_C was first hypothesized via a research project led by the Asphalt Institute.

Considering these factors, at the 2019 spring meeting of the Asphalt Institute, its Technical Advisory Committee decided that there was substantial need for a state-of-the-knowledge, engineering report to describe ΔT_C and its relevance in characterizing the behavior of asphalt materials. It is intended that this document fulfill that purpose. Prior to this document, relevant information pertaining to ΔT_C was scattered among research papers, presentations, various meeting minutes, and personal communications among individuals with detailed interest in ΔT_C. Consequently, it was difficult to find and sort through relevant sources of information. Hopefully, this document provides a single, up-to-date reference on the topic of ΔT_C. It is also hoped that this report will serve as a focal point for dialog among agency users, industry producers, academia, and others with a need to have a more detailed understanding of ΔT_C.

The second section describes the research that led to the development of ΔT_C. The third section provides a detailed explanation of how ΔT_C is computed along with a review of the testing involved to capture information needed to calculate ΔT_C. Information is presented to help explain the physical meaning of ΔT_C. To offer perspective on ΔT_C, typical values are shown for a wide range of PG binders. The fourth section provides information pertaining to factors that affect ΔT_C. Laboratory aging, both normal and extended, is discussed and data is offered that illustrates the change in ΔT_C with various forms of laboratory aging. Information is presented describing the effect of other materials on ΔT_C. These include materials such as recycled asphalt and elastomeric polymers. The fifth section presents practical considerations related to the measurement of ΔT_C. Factors such as precision and the effect of ΔT_C on laboratory workflow are described. Because ΔT_C is viewed as an asphalt binder property that is related to asphalt pavement performance, the estimated effect of ΔT_C on various asphalt pavement distress types is also described. The sixth section describes ΔT_C data derived from full scale projects. The seventh section describes the use of ΔT_C in forensic and specification environments. Because there are numerous national-level research projects that are considering ΔT_C, the eighth section provides a brief summary of that research that pertains to ΔT_C. Summary and references used to produce this document are covered in the ninth and tenth sections. Lastly, a list of frequently asked questions (FAQ's) pertaining to ΔT_C is presented in Section 11.

2.0 ORIGIN OF ΔT_C

ΔT_C was conceptualized in a research project sponsored by the Airfield Asphalt Pavement Technology Program (AAPTP), Project 06-01, "Techniques for Prevention and Remediation of Non-Load-Related Distresses on HMA Airport Pavements" a summary of which was presented by Anderson, et al. in 2011 (1). To understand the relevance of ΔT_C as an indicator of pavement performance it is desirable to review some details from that study. Those in the pavements and materials technical community should seek out the paper and the discussion that followed its presentation to gain a more thorough understanding of the ΔT_C concept. What follows is a summary of the more important features of the AAPTP study.

The goal of the Project 06-01 study was to identify "simple binder and/or mixture tests which can predict imminent cracking or raveling so that pavement preservation strategies can be timed to delay or prevent damage of HMA pavements on general aviation airports." In other words, it was envisioned that in-place binder or mixture at airports could be analyzed to determine the optimum time to place, for example, a thin overlay, fog seal, microsurfacing, or other pavement preservation treatment.

In that study, Anderson, et al. (1) evaluated three asphalt binders that were expected to represent divergent aging characteristics based on the crude source from which they were derived. These were binders based on West Texas sour crude (WTX), a Gulf Southeast crude (GSE), and a Western Canadian crude (WC). The authors surmised that the WTX and WC binders would indicate the worst and best aging and brittleness characteristics, respectively, with the GSE in the middle. Their experimental plan involved evaluating the three binders using several physical property tests as follows:

- Dynamic Shear Rheometer (DSR) test via AASHTO T315 at 44.7°C (Texas A&M University procedure to determine G' and η'),
- DSR mastercurves G' and η' determined from G^* and d,
- DSR monotonic binder fatigue test via AASHTO TP-101,
- Ductility via AASHTO T51 at 15°C,
- Force ductility via AASHTO T300 at 15°C, and
- Bending Beam Rheometer (BBR) creep stiffness (S) and creep rate (m) via AASHTO T313 at various low temperatures.

The binders were tested in an unaged condition. But because the authors knew that aging was important to define long-term, brittle asphalt behavior, they also conditioned each binder using the pressure aging vessel (PAV, AASHTO R28) at three levels of aging:

- PAV20 (20 hours of aging, i.e., the normal AASHTO R28 amount),
- PAV40, (40 hours of aging), and
- PAV80 (80 hours of aging).

The authors relied on the landmark research conducted in 1977 by Kandhal (2) which showed that raveling and block cracking were highly related to loss of binder ductility when measured at 15°C. Therefore, by relating the results of the various physical property tests to 15°C ductility, the authors believed they could select the test or tests that best predicted the "imminent" development of cracking due to age-related embrittlement. Examples of this type of non-load associated cracking are shown in Figure 1.

Figure 1. Example Block Cracking From Age-Related Embrittlement in Two-Year Old Surface

Of the results from the six physical property tests that were conducted (see list above), the authors concluded that two were thought to show promise as identifying asphalt binders with differing block cracking potential because they best related to ductility. Those were (a) $G'/(\eta'/G')$ determined using results of the mastercurve evaluations and (b) BBR S and m results at various low temperatures converted into a new parameter "ΔT_C," which represented the difference between the test temperatures at which a binder is exactly at the specified limit for S and m. (The exact method of calculating ΔT_C will be demonstrated in Section 3.0 of this document.)

$G'/(\eta'/G')$ was a parameter developed at Texas A&M University by Glover, et al. (3) and thought to be a surrogate for ductility. The authors computed $G'/(\eta'/G')$ using the original Glover procedure that measures G' and η' by DSR testing at 44.7°C. Similarly, they also determined G′ and η′ using the DSR mastercurve method. Of the two methods, the DSR mastercurve method was best related to ductility. Figure 2 shows the relationship between $G'/(\eta'/G')$ and 15°C ductility. The authors' analysis showed that 85 percent of the variability in ductility of the binders tested could be explained by variability in $G'/(\eta'/G')$ computed by the mastercurve method. Thus, they concluded that $G'/(\eta'/G')$ from the mastercurve method, as expected, was a very good predictor of ductility.

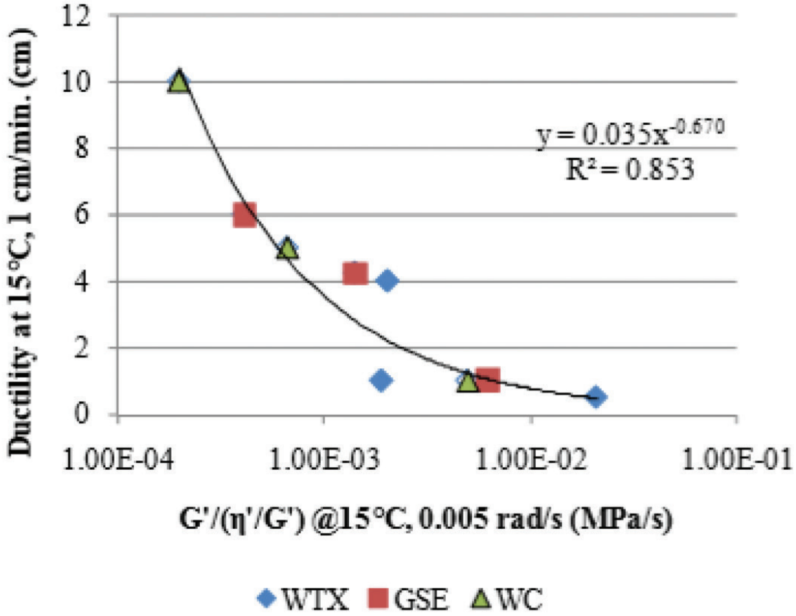


Figure 2. Relationship Between G′/(η′/G′) and 15°C Ductility (1)

Likewise, the authors determined ΔT_c to also be highly predictive of ductility. Figure 3 demonstrates 15°C ductility as a function of ΔT_c. In this case, 74% of the variability in ductility of the binders tested could be explained by variation in ΔT_c. The authors computed ΔT_c from BBR tests at sufficient temperatures to bracket the two temperatures at which the binders exhibited a creep stiffness (S) and creep rate (m) at their AASHTO M320 specified limits. The authors hypothesized that ΔT_c was an indicator of loss of relaxation properties.

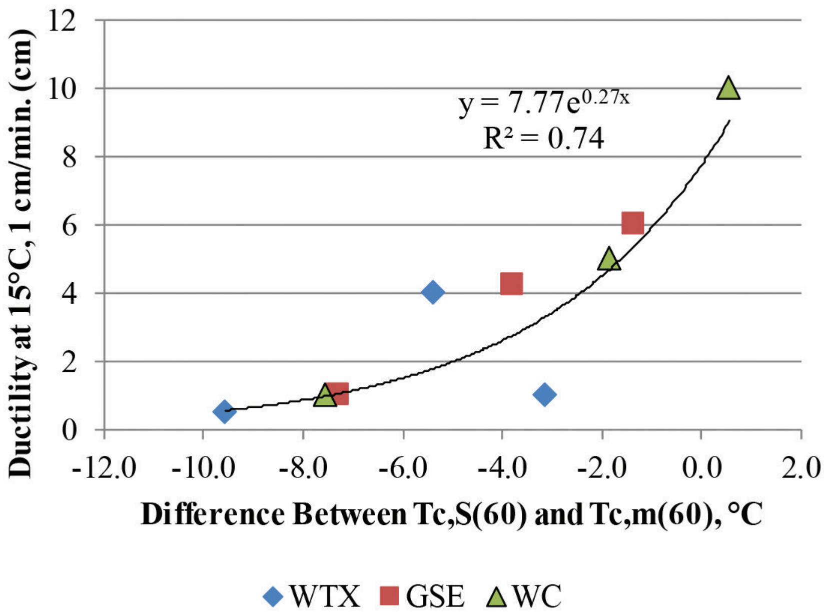


Figure 3. Relationship Between ΔT_c and 15°C Ductility (1)

As a historical note for those that review the original report from Anderson et al. (1), the original AAPTP project defined ΔT_C as being equal to $T_{C,m}$ - $T_{C,S}$ which meant that most computations indicated positive values for ΔT_C. In later years, determination of ΔT_C evolved such that ΔT_C was computed from $T_{C,s}$ - $T_{C,m}$ and this is the standard convention used in this report and elsewhere. Section 3.0 presents more thorough information on ΔT_C calculations.

After determining what they believed were the two parameters most aligned with the goal of the study, the authors studied the relationship between G′/(η′/G′) and ΔTc. That relationship is shown in Figure 4. As expected, the relationship between the two parameters that were the best predictors of ductility were themselves highly related.

An interesting feature of Figure 4 are the two limiting values of G′/(η′/G′) shown. These limits came from the Glover's research that showed a G′/(η′/G′) corresponding to a 15°C ductility of 5 cm as indicative of a pavement that is near to cracking, while a ductility of 3 cm indicated a pavement that already exhibited cracking. In other words, Glover established cracking limits of G′/(η′/G′) that related back to Kandhal's work. When generating Figure 4, the authors plotted the Glover G′/(η′/G′) cracking limits as horizontal lines. By plotting Figure 4 the authors clearly demonstrated that it would be possible to arrive at cracking limits for ΔT_C by using the limiting values of Glover's parameter, G′/(η′/G′), which itself related back to Kandhal's study. Projecting down from where the horizontal lines intersect the relationship between ΔT_C and G′/(η′/G′), ΔT_C cracking limits roughly in the range from about 3° to 6°C are evident.

However, the authors found it most accurate to model the relationship between G′/(η′/G′) and ΔT_C by relating the log of G′/(η′/G′) versus ΔT_C. That relationship is shown in Figure 5, which includes the deterministic relationship between log G′/(η′/G′) versus ΔT_C.

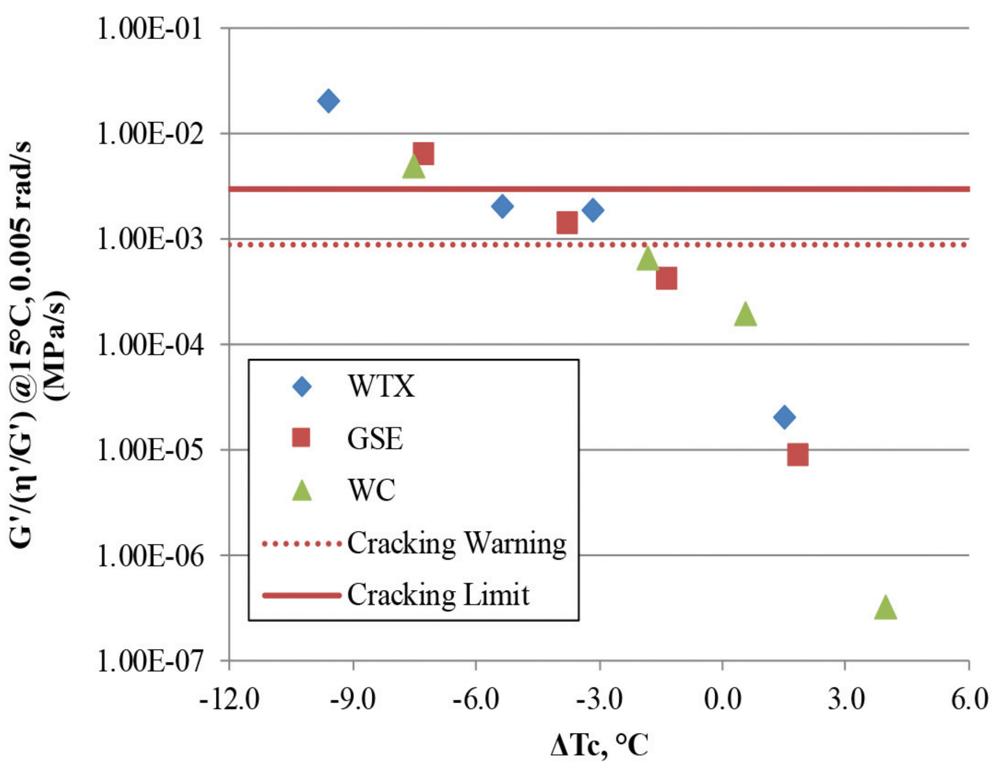


Figure 4. Relationship Between Log G′/(η′/G′) and ΔT_C (1)

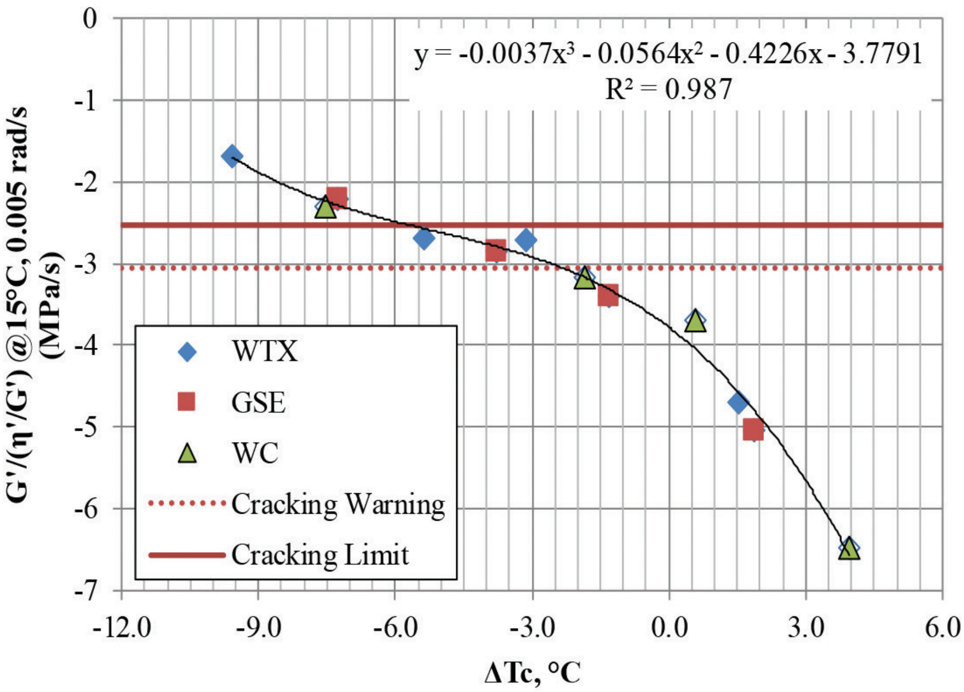


Figure 5. Relationship Between Log G'/(η'/G') and ΔT_C (1)

Next, the authors inserted values of G′/(η′/G′) corresponding to Glover's limiting ductility values of 5 cm and 3 cm into the equation shown in Figure 5 to arrive at warning and cracking limits of ΔT_C. Using the binder data generated in the authors' experiment, those ΔT_C limits were 2.5°C and 5.0°C, respectively. This analysis was the genesis of the 5.0°C (now -5.0°C) ΔT_C value that is sometimes used in forensic analyses and specification limits.

According to the original purpose of this study, an in-place asphalt sample could be extracted and evaluated for ΔT_C. If that sample indicated a ΔT_C of about 2.5°C, then it is likely that the pavement would need a preventive maintenance treatment because cracking would be imminent. Likewise, if the sample indicated a ΔT_C of 5°C or greater, then the pavement was likely already exhibiting cracking and thus, a maintenance treatment more targeted to this condition would be necessary.

It is worth noting that within the context of this study, ΔT_C was viewed by the authors as being more useful than G′/(η′/G′). That is for two reasons. First, G′/(η′/G′) was determined at 15°C. That is the temperature Kandhal used in his study, which focused on test sections in Pennsylvania. The authors point out that for the aging that occurs in different climates, different ductility test temperatures might be needed. Second, and conversely, the ΔT_C parameter is independent of climate. That is, when ΔT_C reaches about 5.0°C, ductility is likely at a level where block cracking will commence regardless of climate.

To validate their hypotheses regarding the usefulness of G′/(η′/G′) and ΔT_C in predicting the timing of pavement preservation treatments, the authors evaluated cores taken at three airfield pavements, one in Montana and two in New Mexico. The Montana airfield had a recent overlay that was reported to be over a cracked pavement. Thus, the Montana airfield offered two data points, the overlay representing relatively unaged, non-brittle binder and the underlying pavement representing a brittle, highly aged binder. Both New Mexico pavements indicated low to moderate durability-related raveling.

The authors extracted asphalt binder from the four pavement layers (old and new layers at Montana airfield plus two New Mexico airfields) and determined G′/(η′/G′) and ΔT_C. They superimposed those four data points on the data for the three asphalt binders evaluated at the four aging conditions. That plot is shown in Figure 6.

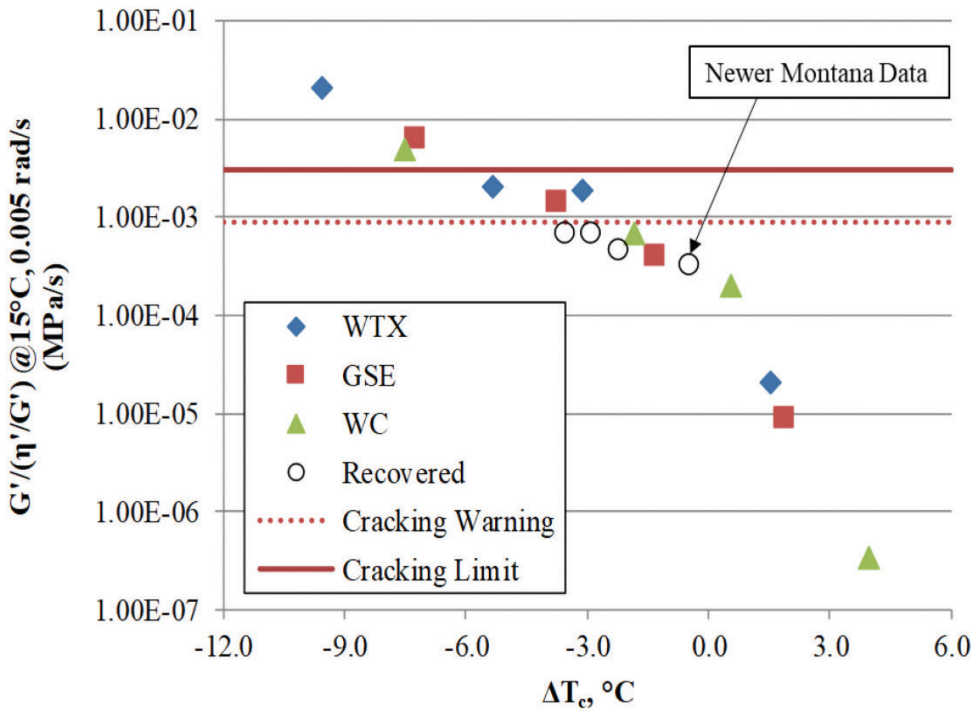


Figure 6. Relationship Between G′/(η′/G′) and ΔT_C with Recovered Binders from Airfield Pavements in Montana and New Mexico (1)

The recovered binder data (shown in the graph as open circles) fit very well with the binders previously evaluated. In other words, the newer Montana recovered binder had values of G′/(η′/G′) and ΔT_C that indicated less aging and less brittleness when compared with the older Montana and New Mexico recovered binders.

Comments on Originating Study: AAPTP Project 06-01

It is again important to note that the AAPTP study had a targeted purpose, which was to identify a laboratory technique that could be used to select an optimum time for preventive maintenance treatments. Thus, the authors' intent was to use ΔT_C in a forensic role. Furthermore, the authors point out that the three binders evaluated (WTX, GSE, and WC), in addition to the extracted cores for the three locations, did not include any modified asphalt. In fact, Kandhal's original study that arrived at a critical ductility value at 15°C also did not include modified asphalt since at that time modified asphalt binders in paving applications were exceedingly rare. Likewise, Glover's et al. (3) ductility prediction using G′/(η′/G′) was found to be accurate only for unmodified binders, and even then for ductility values less than 10 cm. Thus, an important research need would be to determine the efficacy of using ΔT_C (or even G′/(η′/G′)) as an indicator of aging characteristics and preventive maintenance needs for pavements containing modified binders.

Since the milestone AAPTP Project 06-01 study was originally published, ΔT_C has been included as an experimental feature in a significant number of studies by a variety of researchers. In fact, some of this research remains ongoing at the time this publication is under development. A good summary of the evolution of ΔT_C as a property of interest is contained in a circular published by the Transportation Research Board (4). The circular was a product of a technical session at the 2017 Annual Meeting of the TRB. It included four papers that presented varying degrees of information and data related to ΔT_C.

3.0 THE MECHANICS OF ΔT_C

Calculation of ΔT_C

As previously stated ΔT_C is determined using the results of AASHTO T313 (bending beam rheometer) determined at multiple low temperatures. By running the BBR test at multiple low temperatures, the temperatures at which creep stiffness (S) and creep rate (m) are exactly at their AASHTO M320 limiting values (300 MPa and 0.300, respectively) can be determined by interpolation. In other words, the BBR test is executed at temperatures that bracket specified values of S and m. "Critical temperatures" are thus determined by interpolating between passing and failing temperatures and are called $T_{c,S}$ and $T_{c,m}$. Figure 7 illustrates the concept of $T_{c,S}$ and $T_{c,m}$.

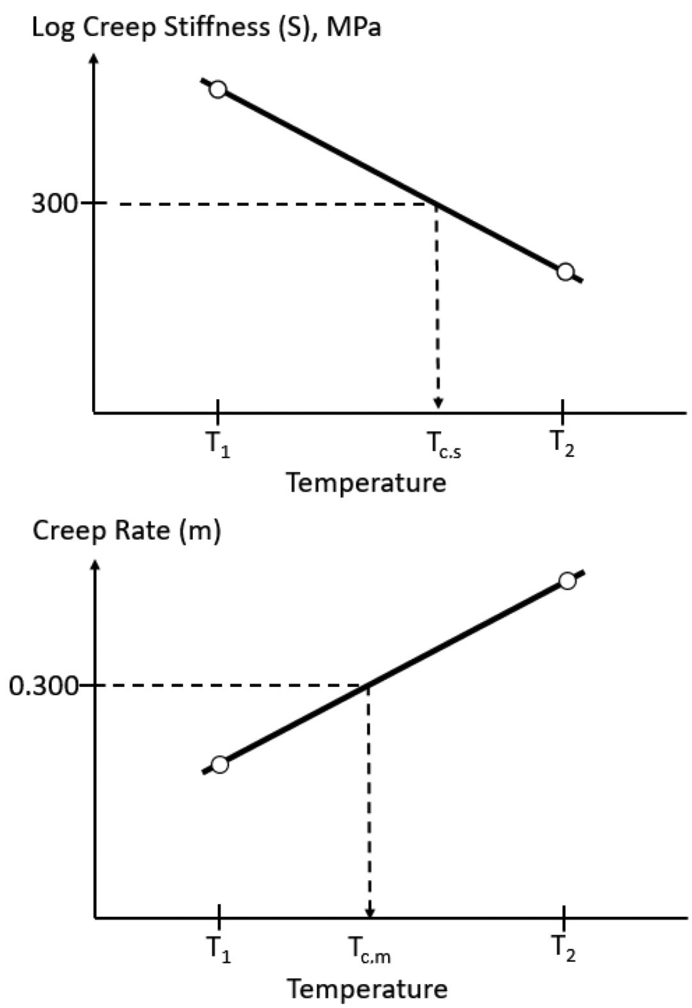


Figure 7. Graphical Concept of $T_{c,S}$ and $T_{c,m}$

A graphical solution to determining $T_{c,S}$ and $T_{c,m}$ such as Figure 7 would certainly be possible, although awkward. A much more convenient method is to use deterministic equations developed by Anderson, et al. (1) which interpolate $T_{c,S}$ and $T_{c,m}$. Those equations are:

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

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

where,

S_1 = creep stiffness at T_1, MPa,
S_2 = creep stiffness at T_2, MPa,
m_1 = creep rate at T_1,
m_2 = creep rate at T_2,
T_1 = temperature at which S and m passes, °C, and
T_2 = temperature at which S and m fails, °C.

Therefore, the equation for ΔT_c becomes:

$\Delta T_c = T_{c,S} - T_{c,m}$.

To demonstrate the calculations for ΔT_c, consider a binder that exhibits S-values of 248 MPa and 466 MPa at -18°C and -24°C, respectively and m-values of 0.324 and 0.290 at -12°C and -18°C, respectively. Then,

$T_{c,S}$ = -18 + ((-18 - -24) x (Log 300 - Log 248)/(Log 248 – Log 466)) – 10 = -29.9°
$T_{c,m}$ = -12 + ((-12 - -18) x (0.300 – 0.324)/(0.324 – 0.290)) = -26.2°.

ΔT_c = -29.9° - (-26.2°) = -3.7 C

Typical PG compliance testing only requires one BBR test at the specified grade temperature to determine if the material meets the specification limit. The critical temperature determination requires two BBR tests be conducted that bracket the T_c. Occasionally, certain binders will indicate different ranges of passing and failing temperature for S and m. For example, a PG 64-22 binder might indicate passing and failing values for S at -18°C and -24°C, respectively, while indicating passing and failing m-values at -12°C and -18°C. Traditionally, this binder would only have been evaluated at -12°C and -18°C and therefore it would be tempting to determine a failing value of S via extrapolation. Anderson recommends (5) that under those circumstances a third temperature be employed to bracket passing and failing critical temperatures and that extrapolation be avoided.

What the ΔT_c Number Means

For an asphalt binder being evaluated, the sign on ΔT_c, either positive or negative, indicates whether the performance grade of the binder is governed by its creep stiffness S (+ΔT_c) or creep rate m (-ΔT_c). The absolute magnitude of ΔT_c indicates the degree to which the binder is governed by either creep stiffness or creep rate. Whether the low temperature grade of an asphalt binder was governed by its S- or m-value has been a topic of discussion since the 1990's when the PG binder specification (AASHTO M320) and the attendant use of the BBR began. Yet with the rise in interest of ΔT_c, the topic of "S-control" versus "m-control" has gained renewed interest.

S-controlled binders (+ΔT_c) are those that fail the 300 MPa limit at a warmer temperature than the m-value temperature. Alternately, m-controlled binders (-ΔT_c) fail the 0.300 m-value at a warmer temperature than the S-value temperature. Thus, to gain an understanding of what ΔT_c means, one must review the meaning of BBR S and m. The following paragraphs present a brief summary.

Since its incorporation into the PG binder specification the BBR has proven to be a valuable tool to understand the low temperature behavior of asphalt. To determine low temperature asphalt properties, the BBR combines simple beam and viscoelastic theories of engineering mechanics.

To run the BBR test, an aged sample of asphalt binder is molded into a small beam and placed on two simple supports in a liquid testing bath. The bath provides very precise control of test temperature. A pneumatic loading shaft applies a constant load to the center of the beam for 240 seconds. Figure 8 shows a schematic of the BBR apparatus. During the 240-second loading period, the BBR measures and stores the deflection of the beam under the test load.

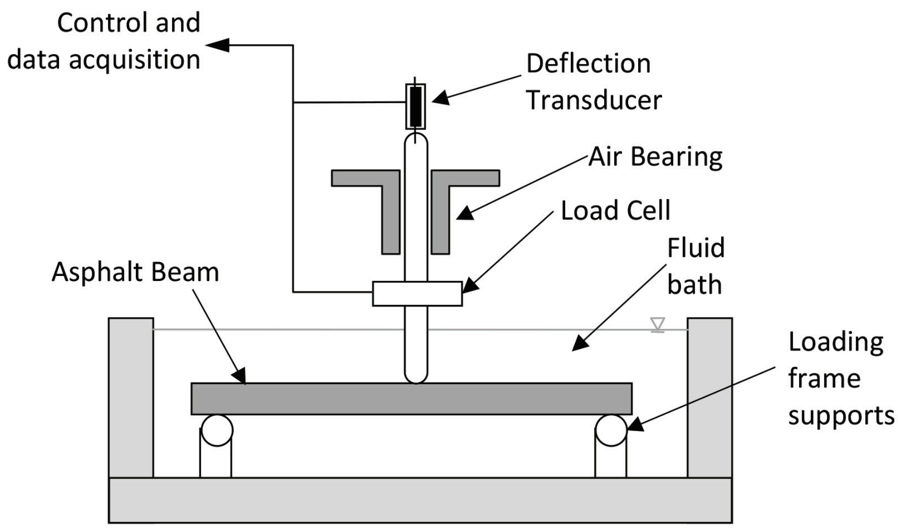


Figure 8. Simple Schematic of BBR Test Apparatus

An equation from engineering classic beam theory is used to compute the stiffness of the asphalt beam at any point in time while it deflects under the load. That equation is

$$S(t) = \frac{PL^3}{4bh^3(t)}$$

where,

S(t) = creep stiffness at any given time,
P = applied constant load,
L = span between beam supports,
b = beam width,
h = beam thickness, and
(t) = deflection of beam at time at any given time t under the test load.

Thus, by means of the BBR instrument measuring beam deflection, and by knowing the shape of the beam, the applied load, and span length, it becomes a very easy matter for the BBR software to calculate the stiffness of the asphalt beam at any time within the 240-second loading period. During that period, the asphalt beam deflection increases. Since beam deflection is in the denominator of the above equation, the beam stiffness diminishes as the test proceeds through the 240-second loading period. Figure 9 shows this conceptually.

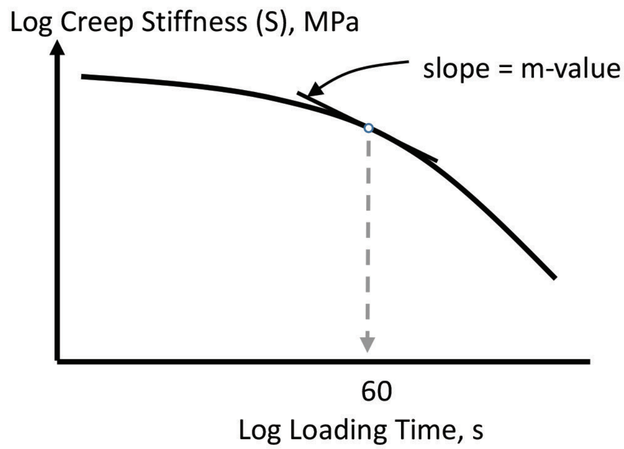


Figure 9. Effect of Loading Time on Creep Stiffness (S)

At the very low temperatures at which the BBR test is performed, asphalt binder becomes very stiff and loses flexibility. It is axiomatic that if the asphalt binder stiffness becomes too large, then it will crack. In the context of the PG asphalt binder specification, critical stiffness is assumed to be exactly 300 MPa. That is the upper limit for asphalt binder stiffness calculated at 60 seconds of loading time in the BBR while being tested 10°C warmer than the anticipated low pavement temperatures.

Yet creep stiffness does not present a complete picture of cracking tendency of asphalt binder at low temperatures. Because asphalt is a viscoelastic material, it has the ability to relax applied stresses. To put it another way, if given sufficient time, asphalt binder will shed the stresses that build up when a load is applied. It makes no difference whether the source of the stress is an applied load or stress that builds up as temperature conditions change.

A good way to visualize the concept of shedding of stresses is by considering the two asphalt beams shown in Figure 10. Beam A is fixed at one end. When subjected to a lowering of temperature, the beam contracts and changes length. Because one end of the beam is free to move, no stresses build up as it contracts. However, when subjected to the same drop in temperature, Beam B tries to contract but is restrained from doing so because it is fixed at both ends. This causes buildup of tensile stresses. If those stresses become too great, then the beam will crack. However, so long as the tensile stresses in the beam do not exceed the tensile strength of the beam no cracking will occur. Under that condition, if given sufficient time the asphalt will experience viscous flow which causes the stresses to “relax” back to residual stress. At a given temperature, an asphalt that experiences the viscous flow faster will shed the stresses quicker. This ability to more quickly relax stresses is the asphalt characteristic measured in the BBR test known as creep rate or m-value.

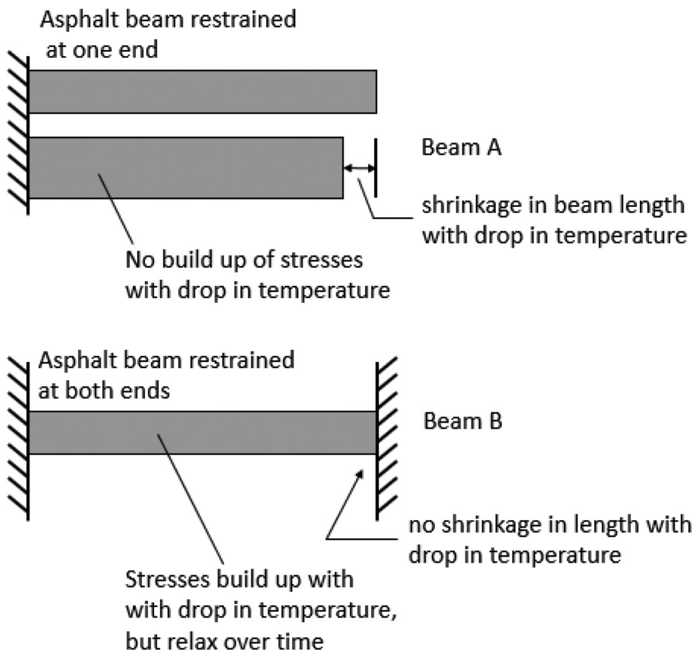


Figure 10. Fixed Asphalt Beams to Illustrate Concept of Stress Buildup and Relaxing

In addition to S, the BBR also uses data collected during the 240-second loading period to compute m-value. Within the framework of the BBR test, m-value is the rate of change of S as a function of loading time. In other words, m-value represents how fast stiffness drops off during the 240-second loading period and is analogous to the fixed beam example in Figure 10.

As a practical matter, the software that controls the BBR computes a mathematical equation to relate log S with log loading time (see Figure 9). The first derivative of that equation is solved at 60 seconds and that is the slope of the relationship at 60 seconds, which is the m-value reported from the test. A large slope or m-value means that the stiffness changes relatively fast and thus, has better ability to shed thermal stresses. In the context of the PG asphalt binder specification, the critical m-value is taken to be exactly 0.300, which is the lower limit for asphalt binder calculated at 60 seconds of loading time when tested 10°C warmer than the anticipated low pavement temperatures. In other words, if m-value is greater than 0.300 at 60 seconds, it is assumed that the binder has sufficient ability to quickly shed stresses that build up due to a drop in temperature.

Based on the foregoing review of S and m, it becomes clear that ΔT_C is a parameter that represents how well the low temperature cracking behavior of an asphalt binder is balanced between its stiffness and its ability to shed stresses at low pavement temperatures. At a given low temperature performance grade, a more negative value of ΔT_C means that the binder's ability to shed stress is not sufficient regardless of stiffness.

What ΔT_C Looks Like

Table 1 shows ΔT_C data provided by the Kansas Department of Transportation (DOT) from their 2018-2019 database of 72 asphalt binder prequalification samples. This illustrates a practical range of ΔT_C for typical asphalt binders that are commercially supplied to a given geographic area.

Table 1. ΔT_c Values for Various Commercial Binder Grades and Aging Conditions

	20-hour PAV		40-hour PAV		Aging
Product	Theoretical Grade	ΔTc, °C	Theoretical Grade	ΔT_C, °C	Difference, °C
PG 52-34	PG 65-37	1.7	PG 65-35	2.5	+0.8
PG 52-34	PG 55-36	1.7	PG 55-35	0.6	-1.1
PG 52-34	PG 55-36	1.9	PG 55-35	1.1	-0.8
PG 58-28	PG 61-29	1.2	PG 61-28	-1.1	-2.3
PG 58-28	PG 60-30	1.0	PG 60-27	-1.9	-2.9
PG 58-28	PG 60-29	-3.3	PG 60-23	-7.7	-4.4
PG 58-28	PG 59-32	3.4	PG 59-26	-5.9	-9.3
PG 58-28	PG 60-30	-0.2	PG 60-27	-1.4	-1.2
PG 58-28	PG 61-31	-0.8	PG 61-26	-3.2	-2.4
PG 58-28	PG 60-30	-0.6	PG 60-27	-2.1	-1.5
PG 58-28	PG 60-30	1.2	PG 60-29	0.1	-1.1
PG 58-28	PG 59-30	2.1	PG 59-27	-1.2	-3.3
PG 58-28	PG 60-32	-0.1	PG 60-26	-5.2	-5.1
PG 58-34	PG 62-35	2.2	PG 62-34	1.9	-0.3
PG 58-34	PG 64-36	0.0	PG 64-34	-1.9	-1.9
PG 58-34	PG 64-35	-1.1	PG 64-33	-1.9	-0.8
PG 58-34	PG 66-35	2.2	PG 66-33	0.8	-1.4
PG 58-34	PG 62-36	1.6	PG 62-33	-2.1	-3.7
PG 64-22	PG 66-27	0.6	PG 66-24	-1.5	-2.1
PG 64-22	PG 66-25	-0.7	PG 66-21	-4.3	-3.6
PG 64-22	PG 68-31	-7.4	PG 68-17	-21.9	-14.5
PG 64-22	PG 66-26	-1.9	PG 66-21	-5.4	-3.5
PG 64-22	PG 66-24	-1.1	PG 66-23	-1.9	-0.8
PG 64-22	PG 68-25	-2.9	PG 68-18	-8.1	-5.2
PG 64-22	PG 66-25	-4.7	PG 66-14	-14.7	-10.0
PG 64-22	PG 66-23	-5.7	PG 66-19	-6.8	-1.1
PG 64-22	PG 66-27	-1.0	PG 66-23	-4.4	-3.4
PG 64-22	PG 66-25	1.1	PG 66-23	-0.6	-1.7
PG 64-22	PG 67-24	1.4	PG 67-20	-2.1	-3.5
PG 64-22	PG 65-27	-1.0	PG 65-22	-3.9	-2.9
PG 64-22	PG 66-25	-1.9	PG 66-22	-4.4	-2.5
PG 64-22	PG 67-26	-0.2	PG 67-22	-3.0	-2.8
PG 64-28	PG 69-31	-0.1	PG 69-25	-4.8	-4.7
PG 64-28	PG 68-35	-0.5	PG 68-31	-2.0	-1.5
PG 64-28	PG 70-30	0.1	PG 70-27	-2.0	-2.1
PG 64-28	PG 69-30	0.6	PG 69-27	-1.5	-2.1
PG 64-28	PG 66-30	1.5	PG 66-27	-1.6	-3.1

PG 64-28	PG 67-29	-4.7	PG 67-24	-9.4	-4.7
PG 64-28	PG 66-29	-2.0	PG 66-25	-5.0	-3.0
PG 64-28	PG 67-31	1.4	PG 67-28.0	-1.5	-2.9
PG 64-28	PG 67-31	0.8	PG 67-28	-1.9	-2.7
PG 64-34	PG 71-36	1.5	PG 71-34	0.2	-1.3
PG 64-34	PG 69-36	-0.3	PG 69-34	-1.2	-0.9
PG 64-34	PG 70-37	-0.5	PG 70-33	-3.5	-3.0
PG 64-34	PG 68-36	0.4	PG 68-34	-1.2	-1.6
PG 64-34	PG 70-36	-1.4	PG 70-31	-4.7	-3.3
PG 64-34	PG 68-35	1.4	PG 68-34	-0.6	-2.0
PG 70-22	PG 70-30	0.3	PG 74-27	-1.9	-2.2
PG 70-22	PG 77-25	-1.3	PG 77-21	-4.5	-3.2
PG 70-22	PG 72-32	0.0	PG 72-29	-0.9	-0.9
PG 70-22	PG 72-32	0.4	PG 72-29	-1.2	-1.6
PG 70-22	PG 73-27	-0.8	PG 73-23	-3.5	-2.7
PG 70-28	PG 75-31	1.0	PG 75-29	-0.7	-1.7
PG 70-28	PG 75-31	1.8	PG 75-30	-0.4	-2.2
PG 70-28	PG 75-31	0.1	PG 75-27	-3.2	-3.3
PG 70-28	PG 74-32	0.4	PG 74-28	-2.3	-2.7
PG 70-28	PG 75-35	-5.2	PG 75-18	-22.0	-16.8
PG 70-28	PG 74-30	-4.2	PG 74-25	-8.5	-4.3
PG 70-28	PG 76-29	0.7	PG 76-26	-2.4	-3.1
PG 70-28	PG 72-32	0.3	PG 72-29	-1.7	-2.0
PG 70-28	PG 73-30	-0.4	PG 73-25	-4.3	-3.9
PG 70-28	PG 72-32	0.8	PG 72-29	-1.6	-2.4
PG 70-28	PG 75-30	-0.7	PG 75-26	-3.6	-2.9
PG 70-28RCI	PG 79-37	0.8	PG 79-34	-1.7	-2.5
PG 70-34	PG 78-36	1.8	PG 78-34	0.7	-1.1
PG 76-22	PG 79-28	-0.7	PG 79-24	-4.0	-3.3
PG 76-22	PG 77-25	-1.3	PG 77-21	-4.5	-3.2
PG 76-22	PG 81-32	1.4	PG 81-30	-1.1	-2.5
PG 76-28	PG79-30	-11.1	PG 79-18	-22.5	-11.4
PG 76-28	PG 80-31	-0.6	PG 80-29	-1.8	-1.2
PG 76-28	PG 78-29	0.7	PG 78-27	0.7	0.0
PG 76-28	PG 80-32	1.7	PG 80-19	-7.5	-9.2

The Aging Difference in Table 1 represents the difference between ΔT_c values after standard PAV aging (20 hours) and after extended PAV aging (40 hours) with a negative value indicating that ΔT_c became worse (more negative). Observed values of ΔT_c from the Kansas DOT data range from about +3.4°C to -11.1°C for 20 hours and +2.5°C to -22.5°C for 40 hours. One of the binders (PG 52-34) indicated an improvement (increase) in ΔT_c with additional aging. This is highly unusual and should not be expected. Overall there was a wide variety of low temperature behavior, even within a single performance grade.

The data in Table 1 shows that the degree to which a binder is S- versus m-controlled is strongly influenced by the amount of aging. Of the 72 binders listed, for 20 hours of PAV aging there was about an even split between S- and m-control: 36 exhibited S-control, 34 exhibited m-control, and two binders indicated equal balance (i.e., ΔT_C equal to zero). For 40 hours of PAV aging, m-control became strongly dominant: 8 exhibited S-control and 64 exhibited m-control.

While the data shown in Table 1 represents a database of commercial binders in a given geographic area, Figures 11 and 12 show ΔT_C data assembled from a database of asphalt binders from very different sources and manufacturing processes that are part of the NCHRP 9-60 research project (5).

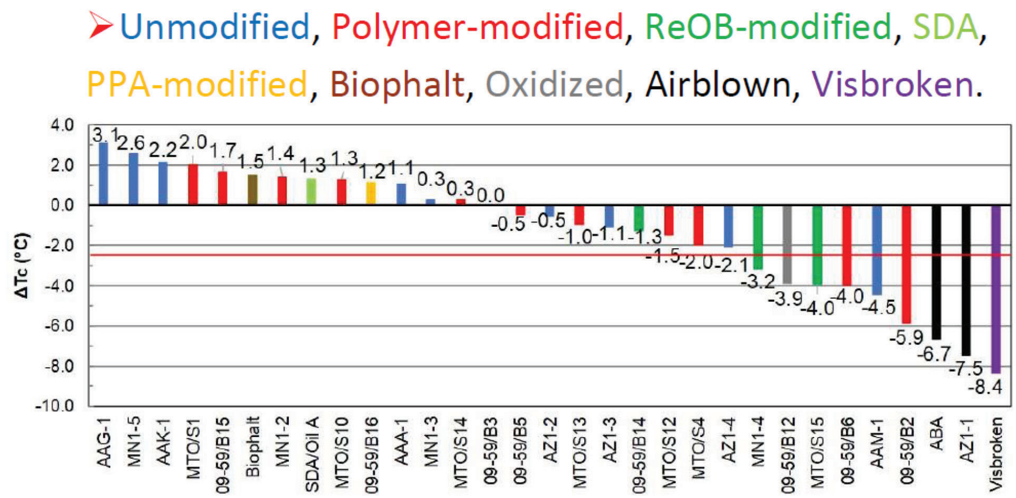


Figure 11. ΔT_C (PAV20) of Various Binders from NCHRP 9-60 Research Project Database (4)

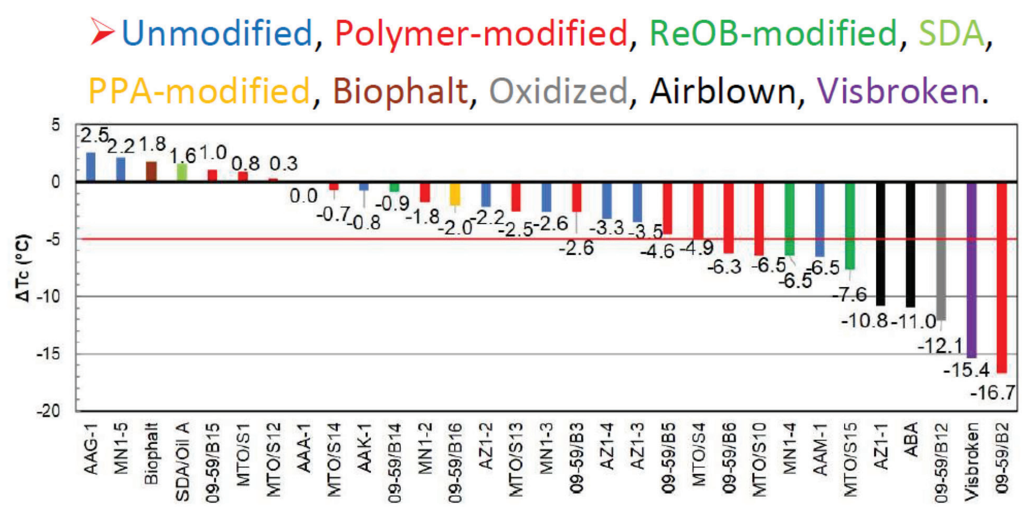


Figure 12. ΔT_C (PAV40) of Various Binders from NCHRP 9-60 Research Project Database (4)

Figure 11 indicates a range in ΔT_C from 3.1°C to -8.4°C while Figure 12 indicates a range from 2.5°C to -16.7°C. As with the Kansas DOT data, this clearly shows that 40-hour PAV results in more negative values of ΔT_C compared to 20-hour PAV. Comparing Figures 11 and 12 shows a trend consistent with the Kansas data in that more binders switched from S-control to m-control with additional aging. Although performance grades are not shown as part of this database, both figures illustrate the wide range of ΔT_C exhibited by binders produced by different manufacturing processes and modification strategies.

4.0 WHAT AFFECTS ΔT_c?

Laboratory Aging

Since ΔT_c targets durability types of asphalt pavement distress, laboratory aging is a key component of any discussion pertaining to ΔT_c. Figure 13 is replotted from the original AAPTP Project 06-01 study by Anderson et al. (1) and uses the current convention for calculating ΔT_c, i.e., $\Delta T_c = T_{c,S} - T_{c,m}$. It illustrates that the length of PAV aging influences measured values of ΔT_c for the three binders evaluated in that study. A very clear and expected trend is that as level of aging increases, ΔT_c becomes more negative. Note that in an unaged condition, all three binders indicate a positive value of ΔT_c. At 20 hours of PAV aging, which is the normal amount used in PG binder analyses according to AASHTO M320, the WC binder remains positive and thus, S-controlled. Yet 20 hours of PAV aging causes the WTX and GSE binders to be negative and become m-controlled. Extended PAV aging causes even the WC binder to become m-controlled while WTX and GSE become more so.

Another significant finding from the data in Figure 13 is that with oxidative aging, ΔT_c becomes more negative, which means the asphalt binder becomes more m-controlled. This indicates an asphalt binder that cannot shed stresses and is becoming less flexible, more brittle, and more prone to crack if given the opportunity via applied stresses. It is expected since cracking is generally a distress type that occurs in older, more aged pavements.

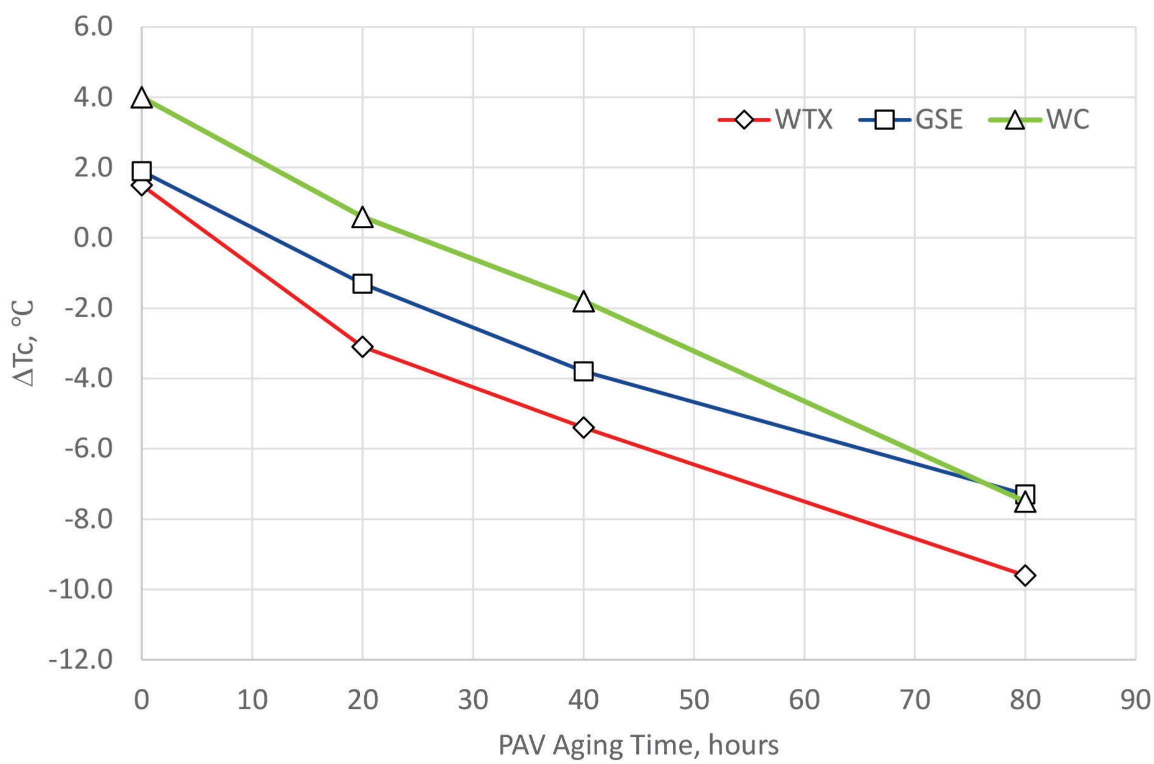


Figure 13. Effect of PAV Aging Time on ΔT_c (1)

Yet there remains questions regarding how much laboratory aging needs to be accomplished to adequately evaluate ΔT_C. The answer to that question depends on the nature of the ΔT_C analysis that is being undertaken.

Reinke, et al. (6) first explored this issue while considering the eight-year performance history of various test sections on County Trunk Highway (CTH) 112 in Olmsted County, MN. In this study, cores were taken from four test sections after eight years of service, each containing binder from a different source. The mixtures contained no RAP or RAS. (Additional features of this project will be discussed later in this report.) A comparison was made between the properties of binders extracted from the top ½-inch of the cores and ΔT_C values from testing of virgin binders that had been archived when the test sections were originally constructed. The virgin binders were tested using 20 and 40 hours of PAV aging. Figure 14 shows the results of this analysis. The line of equality in Figure 14 represents laboratory and pavement ΔT_C as equal. This data shows that 20 hours of PAV aging tends to under predict pavement aging while 40 hours over predicts aging for Minnesota pavement in-place for eight years.

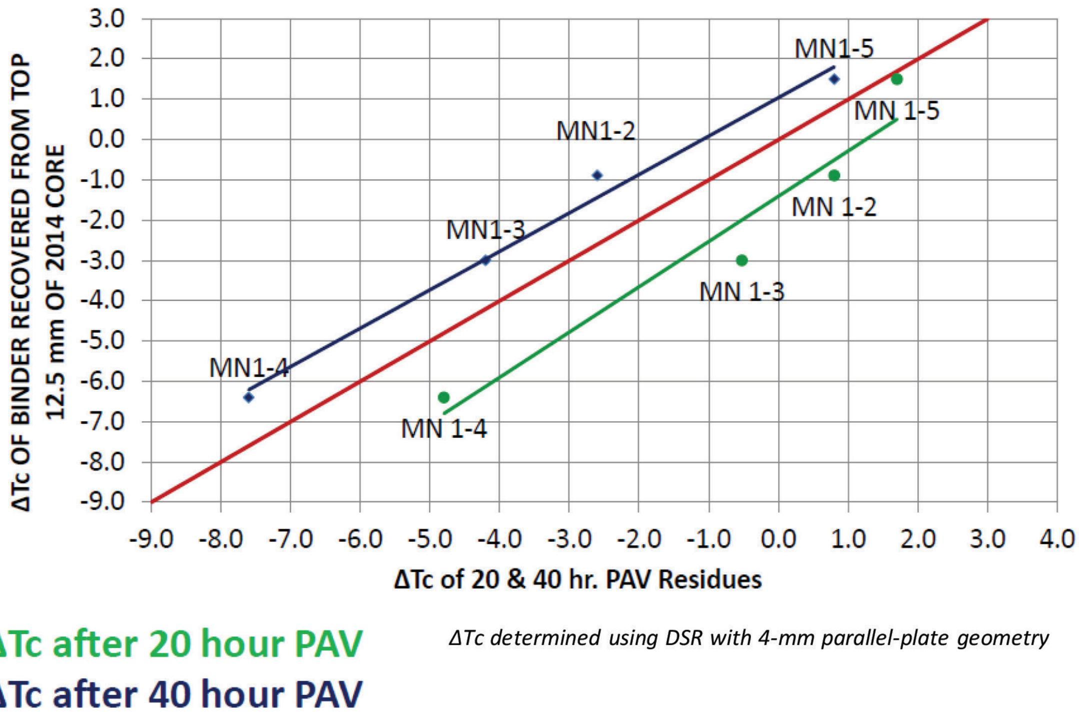
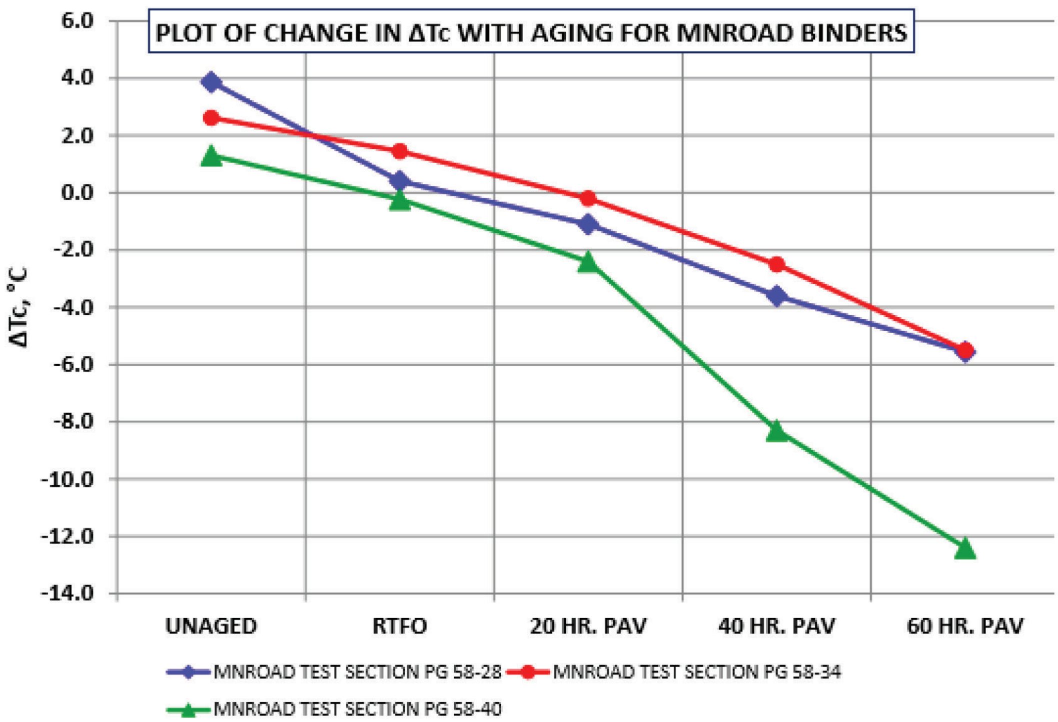


Figure 14. Comparison of 8-Year In-Service Aging with Laboratory Aging for CTH 112 Project (6)

Reinke et al. (6) also evaluated MnROAD binders and showed that the rate of change of ΔT_C was not the same for different binders. Figure 15 shows ΔT_C for three MnROAD binders as a function of laboratory-aged condition. For all PAV aging conditions, the rank order of the three binders remain consistent with respect to ΔT_C. However, when increasing from 20 to 40 hours of PAV aging, the rate of decline in ΔT_C was much greater for the PG 58-40 binder than for the PG 58-28 and PG 58-34 binders. It should be noted that the PG 58-40 test section of MnROAD displayed poor cracking performance.

This data illustrates that in some cases, extended PAV aging is needed to gain clear discrimination among binders with varying cracking characteristics. Consequently, the effect of additional laboratory aging needs to be taken into account. Consideration needs to be given that 20 hours of extended aging (i.e., 20 additional hours added to the existing 20-hour PAV aging period for a total of 40 hours) would require additional resources to complete the analysis including additional time for sample preparation and perhaps additional pieces of laboratory equipment.



ΔTc determined using DSR with 4-mm parallel-plate geometry

Figure 15. Effect of Aging Condition on ΔT_C for Various MnROAD Binders (6)

Another factor to consider is that without upgraded controls, many PAV's now in service can only run 20-hour cycles and do not have the capability to run a continuous, extended aging cycle. In recognition of that, the Asphalt Institute organized an interlaboratory study (7) among Northeast Asphalt User Producer Group members to evaluate the effect on ΔT_C of various methods to achieve extended aging. Twenty-seven laboratories participated including 13 suppliers, 10 DOT's, the FHWA, and three universities. The study evaluated three asphalt binders: PG 58S-28, PG 64E-22, and PG 64S-22. The following five aging protocols were used:

- Method A – standard 20-hour PAV
- Method B – 20-hour PAV followed immediately by another 20-hour PAV
- Method C – 20-hour PAV, wait 4 hours, then another 20-hour PAV
- Method D – extended 40-hour PAV
- Method E – 20-hour PAV using only 12.5 grams per pan

Table 2 shows the results of the interlaboratory study.

Table 2. Northeast Asphalt User Producer Group Extended Laboratory Aging Study (7)

BBR Property	PG 58S-28				
	Method A	Method B	Method C	Method D	Method E
Avg m-value @ -18°C	0.322	0.286	0.285	0.285	-
Avg stiffness @ -18°C	243	283	280	278	-
Avg m-value @ -24°C	0.261	0.242	0.235	0.241	-
Avg stiffness @ -24°C	494	530	531	541	-
Avg ΔT_C, °C	+0.3	-3.2	-2.6	-2.6	-
	PG 64E-22				
Avg m-value @ -12°C	0.347	0.307	0.306	0.304	-
Avg stiffness @ -12°C	157	195	191	197	-
Avg m-value @ -18°C	0.287	0.264	0.258	0.258	-
Avg stiffness @ -18°C	327	363	361	383	-
Avg ΔT_C, °C	-0.5	-3.5	-3.6	-3.2	-
	PG 64S-22[A]				
Avg m-value @ -12°C	0.318	0.305	0.293	0.302	0.299
Avg stiffness @ -12°C	103	71	82	77	79
Avg m-value @ -18°C	0.278	0.271	0.262	0.273	0.268
Avg stiffness @ -18°C	197	135	146	139	142
Avg ΔT_C, °C	-7.4	-13.1	-15.1	-14.3	-15.0

[A] test temperatures for Methods B, C, D, and E were -6 and -12°C

The table cells in yellow are highlighted for comparison. In general, the ΔT_C values were reasonably similar for a given asphalt binder across the different aging protocols. This suggests that the ΔT_C values resulting from the various methods of extended aging are similar.

The PG 64S-22 was evaluated with a smaller film at 20 hours of PAV aging. In that one case, the aging that resulted from thinner-film aging for only 20 hours compared favorably with the 40-hour values of ΔT_C. While this represents only one data point, it suggests that there is optimism it will be possible to develop a protocol for extended aging but over a shorter interval of time. These developments would enhance the efficiency of ΔT_C evaluations when extended aging is necessary.

According to Wakefield (8), the Ontario Asphalt Pavement Council (OAPC) partnered with the University of Waterloo's Center for Pavement and Transportation Technology (CPATT) to conduct a research study to provide a framework for evaluating asphalt binder properties in plant-produced asphalt mixes. Seven binders were included in the study that had unmodified and modified grades most commonly used in Ontario, including PG 58-28, PG 58-34, PG 64-28, and PG 70-28. The study included a small interlaboratory study to compare the standard deviations of testing of physical properties of original asphalt binder and recovered asphalt extracted from plant produced mix. Five labs participated in the study. The labs were instructed to determine the continuous "true grade" of the samples, which allowed ΔT_C to be calculated. To investigate the impact of extended aging, labs were asked to follow the Ministry of Transportation or Ontario (MTO) laboratory procedure LS-228 R31 Method C (9) for accelerated aging of asphalt that increases the aging time in the PAV from one 20-hour cycle to two 20-hour cycles for a total of 40 hours. Method C specifies that the vessel will be depressurized between cycles, the sample remains in the vessel, and the second cycle begins within 30 minutes of the end of the first cycle. Three labs were able to provide data for the 40-hour PAV part of the study. Table 3 shows the results of the original binder testing from the OAPC/University of Waterloo experiment and compares the results of 20 and 40 hours of PAV aging.

Table 3. Results of OAPC and University of Waterloo Aging Experiment (8)

Calculated ΔT_c from Ontario PG's @ 20hr PAV								
Lab ID	Y1	Y2	Y3	Y4	Y5	Average	Stdv	COV
58-34 8-1031	1.3	0.0	1.3	0.8		0.9	0.6	0.7
58-34 4-1003	0.4	0.5	1.4	1.1		0.8	0.5	0.6
58-34 3-0915	2.0		1.2	0.8		1.3	0.6	0.5
58-28 6-1006	1.1	0.3	1.6	2.9		1.5	1.1	0.8
64-28 7-1010	1.3	0.3	0.9	1.0		0.9	0.4	0.5
70-28 1-0708	0.3	0.0	0.6	1.9	0.9	0.7	0.7	1.0
70-28 2-0809	-2.0	0.2	0.5	5.0	0.2	0.8	2.6	3.3

Calculated ΔT_c from Ontario PG's @ 40hr PAV								
Lab ID	Y1	Y2	Y3	Y4	Y5	Average	Stdv	COV
58-34 8-1031	-2.6	-2.1	0.8			-1.3	1.8	-1.4
58-34 4-1003	-3.1	-2.9	-2.0			-2.7	0.6	-0.2
58-34 3-0915	-1.0	-0.8	-0.1			-0.6	0.5	-0.7
58-28 6-1006	-1.0	-1.2	-1.0			-1.1	0.1	-0.1
64-28 7-1010	-1.9	-1.7	-1.0			-1.5	0.5	-0.3
70-28 1-0708	-3.0	-2.1	-1.6			-2.2	0.7	-0.3
70-28 2-0809	-2.4	-3.3	-1.1			-2.3	1.1	-0.5

The average ΔT_c among the participating labs at 20 hours for each of the seven binders ranged from +0.7°C to +1.5°C, with the lowest value from a single lab and single binder being 0.0°C. The average ΔT_c among the participating labs at 40 hours for each of the seven binders ranged from -0.6°C to -2.7°C, with the lowest value from a single lab and single binder being -3.1°C. The standard deviations of the two aging protocols do not appear to be significantly different. There are a few individual results that appear as outliers, for example Lab Y4, 20 hours, sample 2-0808 and Lab Y3, 40 hours, sample 8-1031. As with other evaluations cited in this report the OAPC/University of Waterloo data clearly demonstrates the effect of extended aging on ΔT_c.

Within the asphalt technology community there has been discussion concerning whether the ΔT_C from 20 hours of PAV aging could be used to predict ΔT_C from 40 hours of PAV aging. For practical reasons, it would be advantageous to achieve a reliable relationship between 20- and 40-hour PAV aging to improve laboratory workflow. Golalipour and Mensching (10) examined the hypothesis that 20 hours of PAV aging could be used to predict ΔT_C from 40 hours of PAV aging and rejected it. Figure 16 shows the data from which they drew this conclusion.

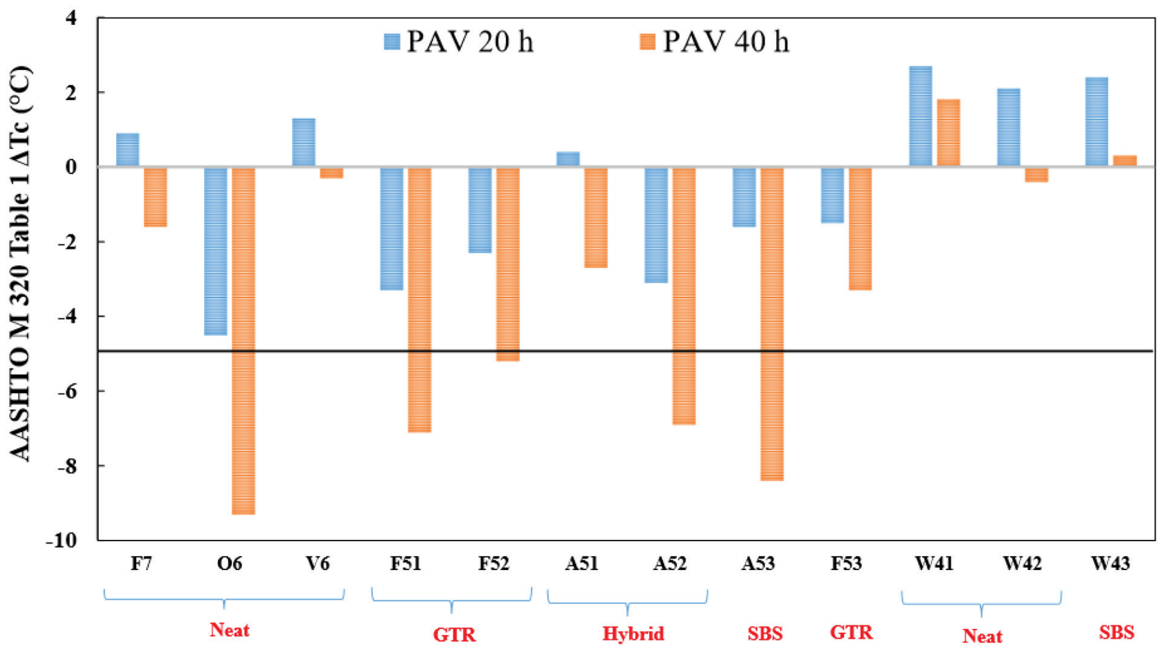


Figure 16. Comparison of ΔT_C from 20 versus 40 Hours of PAV Aging (10)

In Figure 16, there was not a consistent trend in the change in ΔT_C from 20 to 40 hours of PAV aging. Golalipour and Mensching concluded that this relationship was not linear and that the data did not exhibit a simple doubling of ΔT_C with a doubling of aging period.

Figure 17 shows a plot of ΔT_C (40 hours PAV aging) versus ΔT_C (20 hours of PAV aging) for the Kansas DOT results previously listed in Table 1. The coefficient of correlation is reasonably high at 0.74. The slope of the correlation equation is 1.73, which somewhat supports the observation by Golalipour and Mensching that there is not necessarily a simple doubling of ΔT_C results from a doubling of PAV aging time.

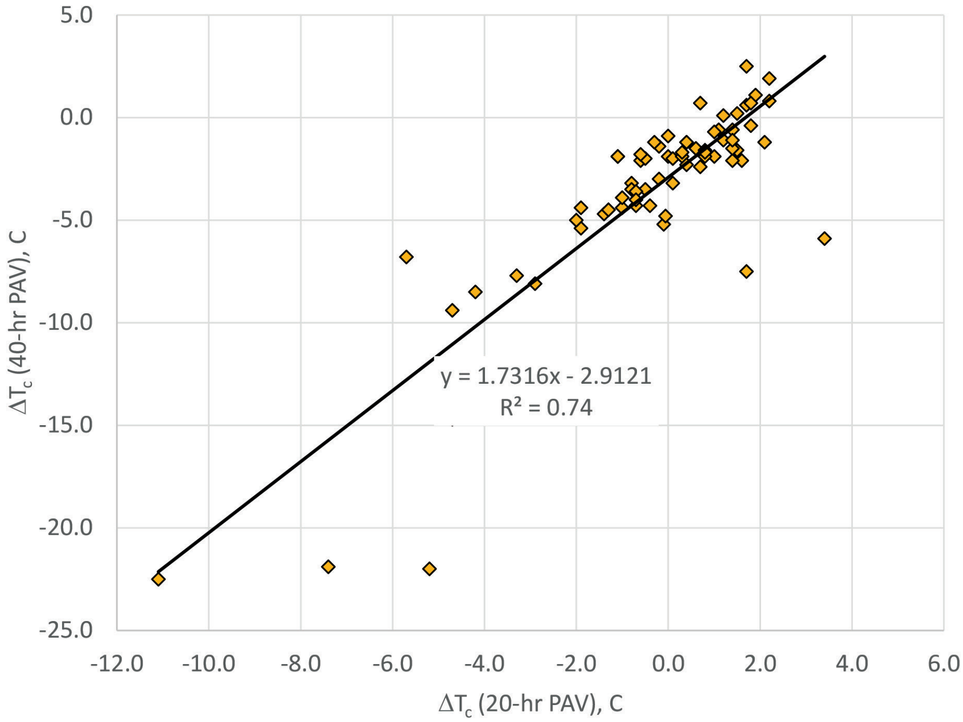


Figure 17. Correlation Between ΔT_C (40-hour) and ΔT_C (20-hour) for Kansas DOT Binders

Figure 18 shows a similar comparison of ΔT_C from 20 and 40 hours of PAV aging developed at the Western Research Institute (WRI) (5). Data from 31 binders is stratified to show polymer modified, unmodified, and other modified binders. "Other" includes REOB modified, solvent deasphalting unit binders, polyphosphoric (PPA) acid modified binders, a biobinder, oxidized binders, and binders from visbreaking. The regression equations show a slope of 1.12 (unmodified binders) and 1.44 (overall data) and is less than the regression slope of 1.84 shown in Figure 17 for the Kansas data. Once again this shows there is not a consistent relationship between ΔT_C resulting from 20- and 40-hour PAV aging.

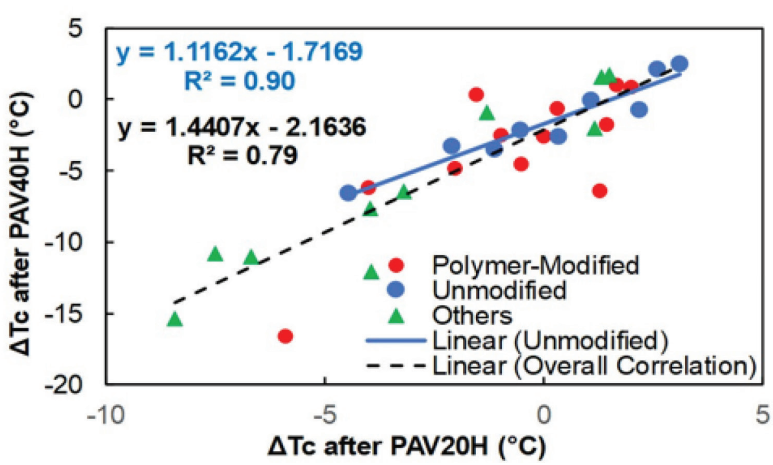


Figure 18. WRI Correlation of ΔT_C (20- vs 40-hour) for Modified and Unmodified Binders (5)

D'Angelo (11) proposed a methodology to predict ΔT_C from 40 hours of PAV aging as a function of the ΔT_C from 20 hours of PAV aging. His technique arrived at an "indicator" of 40-hour ΔT_C and not a direct measure. His predictive model used ΔT_C determined after RTFO aging along with ΔT_C from 20 hours of PAV aging according to the following equation:

$$\Delta T_C \text{ (40-hour)} = \Delta T_C \text{ 20-hour} - (\Delta T_C \text{ RTFOT} - \Delta T_C \text{ 20-hour})$$

In this equation, D'Angelo posited that the change in ΔT_C between RTFO and 20-hour PAV conditions would be the same as the change from 20- to 40-hour PAV. He reasoned that if the estimated 40-hour ΔT_C was acceptable, then an actual 40-hour test would not be necessary. Likewise, if the estimated 40-hour ΔT_C was not acceptable, then an actual ΔT_C determination at 40 hours would be necessary. Table 4 shows an analysis of test results where D'Angelo tested his hypothesis.

Table 4. Prediction of 40-hour ΔT_C (°C) Based on D'Angelo's Methodology (11)

RTFO	20-hour PAV	Measured 40-hour PAV	Estimated 40-hour PAV	Measured -Estimated ΔT_C
0.5	-3.3	-6.1	-7.1	-1.1
1.0	-0.9	-1.4	-2.8	-1.4
-2.6	-7.0	-12.4	-11.4	+1.0
1.7	-1.0	-2.3	-3.8	-1.5
2.8	1.7	0.8	0.6	+0.2
2.3	-0.5	-4.7	-3.3	+1.4
-1.3	-4.8	-7.6	-8.3	-0.7
1.3	-0.9	-2.6	-3.0	-0.4
0.6	-2.7	-5.8	-6.0	-0.2
1.9	0.8	-2.6	-0.3	+2.3
0.4	-3.1	-8.7	-6.6	+2.1
1.3	-0.5	-2.9	-2.3	+0.6
1.7	-0.7	-2.2	-3.1	-0.9
1.6	-2.3	-8.4	-6.2	+2.2

D'Angelo concluded that the estimated 40-hour ΔT_C provided clear indication of whether the measured ΔT_C would be less than a certain value. However, he also pointed out that measurement of the RTFO ΔT_C was problematic given its relatively soft condition.

Finally, aging temperature also can have an effect on the BBR results and the determination of ΔT_C. In recent work conducted by McGennis, a straight-run PG 64-22 produced from western Canadian crude vacuum residuum (i.e., not blended) was tested after PAV aging using 20 and 40 hours and 100°C and 110°C. Results are shown in Table 5.

Table 5. Effect of Aging Temperature and Time on ΔT_c Values

	PAV Aging Temperature and Time			
	100°C		110°C	
BBR Property	20 hours	40 hours	20 hours	40 hours
BBR S(60), MPa				
-6°C				111
-12°C	144	183	164	207
-18°C	312	352	320	368
$T_{c,S}$ (°C)	-27.7	-26.5	-27.4	-25.9
BBR m(60)				
0°C				0.335
-6°C				0.293
-12°C	0.352	0.305	0.325	0.271
-18°C	0.299	0.268	0.280	0.242
$T_{c,m}$ (°C)	-27.9	-22.8	-25.3	-15.0
ΔT_c (°C)	+0.2	-3.7	-2.1	-10.9

The data in Table 5 shows that aging time and temperature affect ΔT_c values, with increased time and increased temperature resulting in lower (more negative) ΔT_c values. The ΔT_c value of the binder sample aged at 110°C for 20 hours is comparable in magnitude to the ΔT_c value of the binder sample aged at 100°C for 40 hours. This may offer an option for consideration by the asphalt industry to generate more severe aging than the standard PAV aging without doubling the aging time. It should be noted, however, that the data shown in Table 5 represents a straight-run asphalt and the trends for modified binders may be different.

Reclaimed Asphalt Pavement (RAP)

ΔT_c is calculated using BBR results on aged binder samples. The PG laboratory aging procedures (RTFO and PAV) age the sample through oxidative hardening. Thus, it would seem natural that combining pre-aged materials such as RAP with a virgin asphalt binder would cause a change to the ΔT_c of the virgin binder by itself. Anderson (12) mined data from the NCHRP 9-12 research project[1] to demonstrate the effect of RAP on ΔT_c.

Two binders, PG 52-34 and PG 64-22, were combined with binder extracted from RAP sampled in Connecticut and Arizona. The Connecticut and Arizona RAP binders represent relatively soft and hard RAP binder, respectively. As such, the virgin binders are being "modified" with oxidized components. Figures 19 and 20 illustrate how RAP affects ΔT_c using these materials that were evaluated in NCHRP 9-12 experiments.

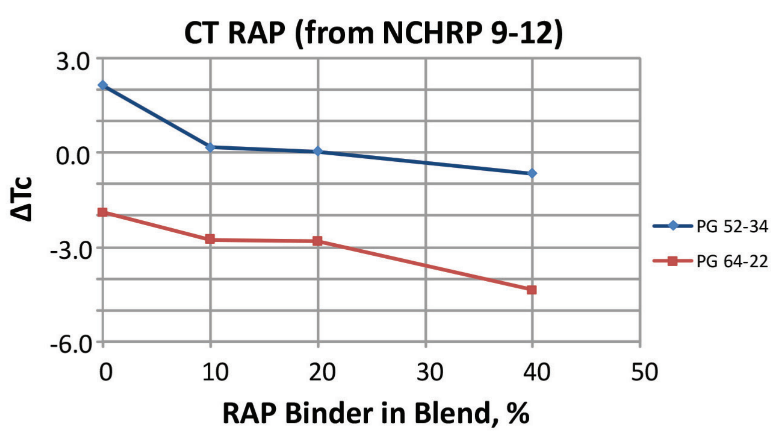


Figure 19. Effect of Relatively Soft RAP on ΔT_c (12)

1 NCHRP 09-12, Incorporation of Reclaimed Asphalt Pavement in the Superpave System, completed 9/30/2000

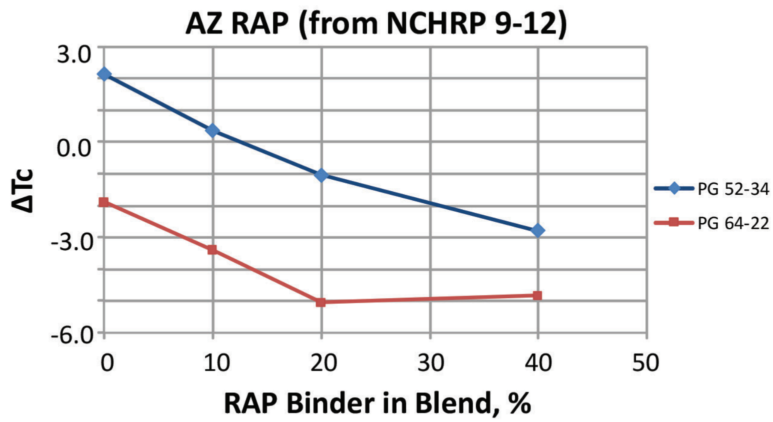


Figure 20. Effect of Relatively Hard RAP on ΔT_c (12)

As shown in the figures, the ΔT_c of the virgin PG 52-34 and PG 64-22 were +2°C and -2°C, respectively. In all cases, addition of RAP binder, whether hard or soft, caused a decay in ΔT_c. Further, the harder Arizona RAP binder caused a greater loss in ΔT_c as compared with the softer Connecticut RAP binder. The most important observation from this data is that an aged component (i.e., RAP binder aged in a pavement) added to a virgin binder resulted in a reduction in ΔT_c.

Using NCHRP 9-12 derived data, Anderson (12) also evaluated the relationship between ΔT_c and the fatigue behavior of an asphalt mixture containing the virgin binders and RAP-modified binders. This relationship is shown in Figure 21.

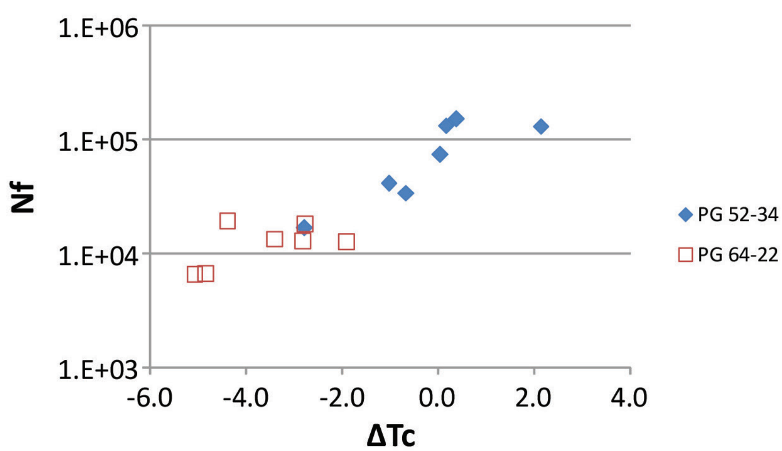


Figure 21. The Relationship Between ΔT_c and Fatigue Life (N_f) of Asphalt Mixtures Containing RAP (12)

The number of cycles to failure (N_f) shown in Figure 20 was measured at 20°C via flexural beam fatigue testing on short-term oven aged mixtures at a constant 8 percent microstrain. In Figure 21, the softer binder, PG 52-34, exhibits higher cycles to failure than PG 64-22, which is the expected trend. Also plotted on Figure 21 are the ΔT_c values for the two virgin binders modified with hard and soft RAP binder and their associated cycles to failure. The trend shown is that lower values of ΔT_c are associated with a lower number of cycles to failure.

Another trend shown in this data is that m-controlled binders (i.e., more negative ΔT_c) exhibit diminished fatigue life. Anderson suggests this behavior shows that fatigue life is influenced by the ability of the asphalt mixture to relax stresses and not just the mixture stiffness.

Recycled Asphalt Shingles (RAS)

As with RAP, it would seem intuitive that replacing a virgin asphalt binder with a heavily oxidized material like reclaimed asphalt shingle (RAS) binder would cause a change to the ΔT_C of the virgin binder by itself. Yet it is experimentally difficult to directly estimate the effect of RAS binder on ΔT_C. That is because extraction of binder from hot mix asphalt would homogenize the virgin and RAS binder. Similarly, blending recovered RAS binder with virgin binder in a laboratory setting may not be realistic, although it may have validity since it represents a worst case. In other words, the RAS binder is so stiff that only part of it becomes intimately combined with the virgin asphalt during manufacture of hot mix asphalt. Musselman (13) suggests that the degree to which the two binders become homogeneous in a mix is termed the "RAS binder availability ratio" and places this at about 0.70 to 0.85.

Anderson (12) conducted an experimental evaluation of materials collected by Willis and Turner (14) in a study for the Federal Highway Administration. The materials evaluated were three types of waste RAS including (a) post-consumer waste (PC), (b) manufacturing waste (MW), and (c) a mixture (blend) of these. Anderson extracted the RAS binders and conducted a normal ΔT_C analysis on each. The results of the ΔT_C evaluations are shown in Table 6.

Table 6. NCAT RAS Study (12, 14)

RAS Source	T_C – High	T_C – Low	ΔTc
NH (PC)	163	+12	-33
OR (blend)	152	+14	-37
TX (MW)	122	-7	-23
WI (MW)	146	+16	-40

The most obvious feature of the NCAT RAS material is that the materials are extremely stiff with high critical temperatures, far in excess of 100°C. Likewise, ΔT_C of the RAS binder is very negative, about one order magnitude lower than laboratory aged paving asphalt. BBR testing of these materials exposed a problem in determining ΔT_C on very stiff, highly oxidized materials. That is, at temperatures where stiffness brackets 300 MPa, m-values are extremely low and much less affected by changes in temperature. If temperatures are increased to achieve m-values that bracket 0.300, then beams become so soft that they exceed the displacement limit allowed in the BBR test procedure (AASHTO T313). Given this situation it becomes necessary to use extrapolation to arrive at $T_{C,m}$ and the resulting value of ΔT_C becomes much less reliable.

Reinke and Hanz (15) reported results of study that included the effect of various rejuvenators on the aging characteristics of RAS-modified hot mix asphalt. Their hypothesis was that materials such as rejuvenators that soften asphalt may not necessarily have the desired effect when extended aging is considered. The RAS used in their study exhibited the properties shown in Table 7.

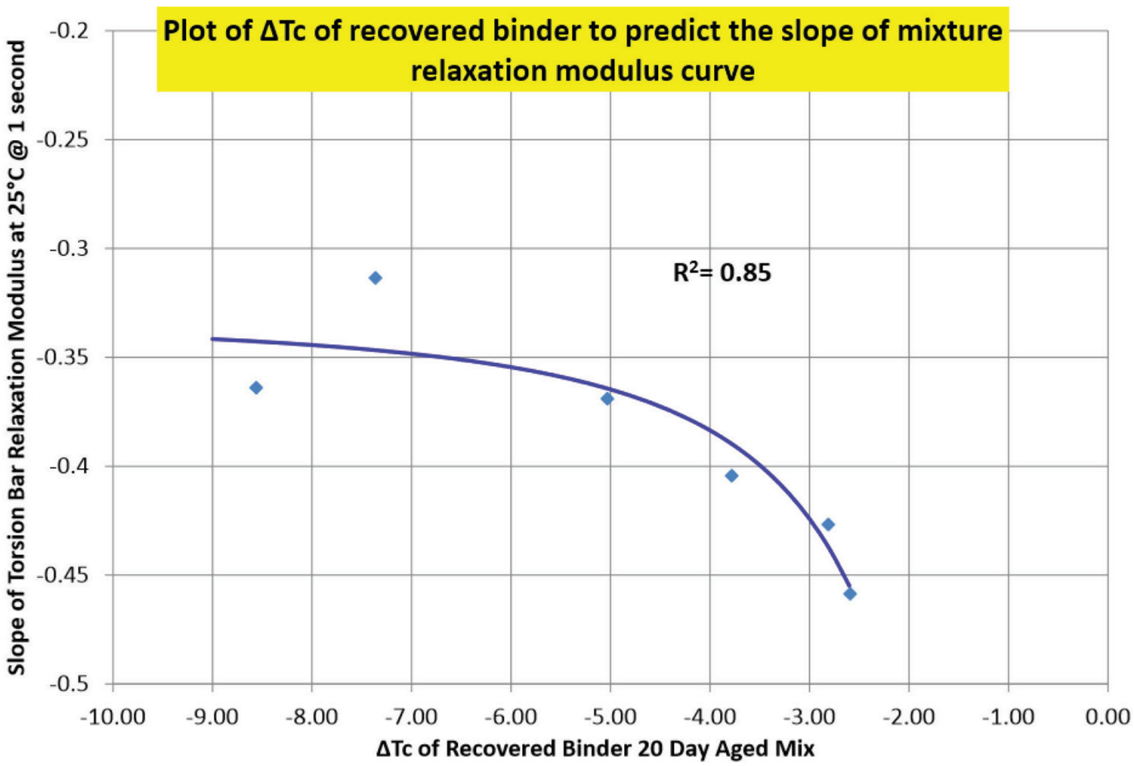
Table 7. Central Wisconsin RAS Binder Properties (15)

RAS Source	T_C – High	T_C – Low	ΔT_C
Central Wisconsin	146	+6	-31.4

Comparing the ΔT_C results in Tables 6 and 7 indicates that the recovered RAS binder properties are very similar. For their mixture study, the RAS binder comprised 22.1% of total binder. Several important conclusions resulted from the Hanz and Reinke study (15).

First, the authors measured the relaxation properties of the mix using a torsion bar test on mixtures long term aged for 20 days at 85°C. The slope of the relationship between torsion bar relaxation modulus with loading frequency is a mixture analog for binder m-value, although in this case a lower slope indicates a mixture that can shed stresses faster and thus be less cracking prone. For mixtures containing RAS, Reinke and Hanz regressed the slope of the relaxation modulus on ΔT_C and discovered that about 85 percent of the variability in slope was explained by ΔT_C. This relationship is shown in Figure 22 and is further validation that ΔT_C offers some indication of the ability of asphalt materials to shed stresses.

Second, because the aged binder and mixture data were so highly related (R^2 = 0.85 in Figure 21) Reinke and Hanz concluded that this suggested the RAS contained in the mix was contributing to aging behavior and not just functioning as a "black rock." This supports Musselman's concept (13) of a RAS binder availability ratio and also caused them to conclude that RAS has a detrimental effect on aging behavior of asphalt.



ΔTc determined using DSR with 4-mm parallel-plate geometry

Figure 22. Relationship Between Slope of Mixture Relaxation Modulus With ΔT_C (15)

Sharma, et al. (16) evaluated physical and chemical properties of asphalt materials containing RAP and/or RAS. Although it was only a small part of their study, they evaluated ΔT_C of tank binders as well as binders recovered from mixtures containing the same tank binders but with RAS included as a mixture modifier. Both PG 58-28 and PG 64-22 were evaluated. Table 8 shows a subset of Sharma's experimental data.

Table 8. Effect of RAS on ΔT_C (16)

Binder Condition	Asphalt Binder	Asphalt Binder Replaced, %	ΔT_C (PAV20), °C	ΔT_C (PAV40), °C
Tank	PG 58-28	0	+0.4	-7.7
Tank	PG 64-22	0	-0.3	-6.0
Extracted	PG 58-28	21.2	-17.7	-
Extracted	PG 58-28	29.8	-8.4	-
Extracted	PG 64-22	0	-1.3	-
Extracted	PG 64-22	10.5	-3.4	-

Unfortunately, Sharma, et al. did not conduct any extended aging (i.e., PAV40) on the RAS modified mixtures. Also missing from their experimental results is extracted ΔT_C for the PG 58-28. Nevertheless, two interesting observations are evident with the data available. First, for the tank binders, there was a significant decline in ΔT_C when extended aging was applied. The ΔT_C of the tank PG 58-28 declined by about 8 degrees, while the ΔT_C of the PG 64-22 declined by over 6°C. Second, for the extracted PG 64-22, the addition of RAS caused only a modest decline in $\Delta T_{C.}$ Addition of RAS to the PG 58-28 seemed to cause a larger decline in ΔT_C, most likely because extracted PG 58-28 utilized a higher percentage of RAS. It is also possible that being softer, the PG 58-28 was able to combine more thoroughly with the RAS binder.

Diefenderfer (17) also offered this explanation when evaluating some RAS-modified SMA mixtures in Virginia. In that study, the effect on ΔT_C of RAS binder was greater for PG 64-22 compared with PG 70-22. Table 9 presents relevant data from Diefenderfer's study.

Table 9. Effect of RAS Binder on ΔT_C of PG 64-22 and PG 70-22 (17)

Binder Condition	Asphalt Binder Grade	ΔT_C, °C (PAV20)
Tank	PG 64-22	-1.0
Extracted	PG 64-22	-7.8
Tank	PG 70-22	-2.3
Extracted	PG 70-22	-4.3

Note: all mixtures contained 5% RAS

Although it is problematic to directly compare tank and extracted binder results, Diefenderfer's data does indicate a decline in ΔT_C with the addition of RAS.

Diefenderfer did not report reclaimed binder ratio (RBR) data for the SMA mixtures evaluated. In general, SMA mixtures possess higher asphalt contents compared to dense graded mixtures. Thus, it is possible that if the RBR was comparatively low, then that might explain why the relatively high RAS content (5%) did not have a larger unfavorable effect on the observed ΔT_C values.

Corrigan and Golalipour (18) reported the results of a project in Wisconsin that utilized both RAP and RAS. The RAP comprised 13 to 40 percent of total mix while the RAS comprised three to six percent of total mix. These amounts of RAP and RAS comprised from about 25 to 65 percent asphalt binder replacement. Four binders were used in a variety of mix combinations. Figure 23 shows the effect on ΔT_C as the amount of asphalt binder replace increases.

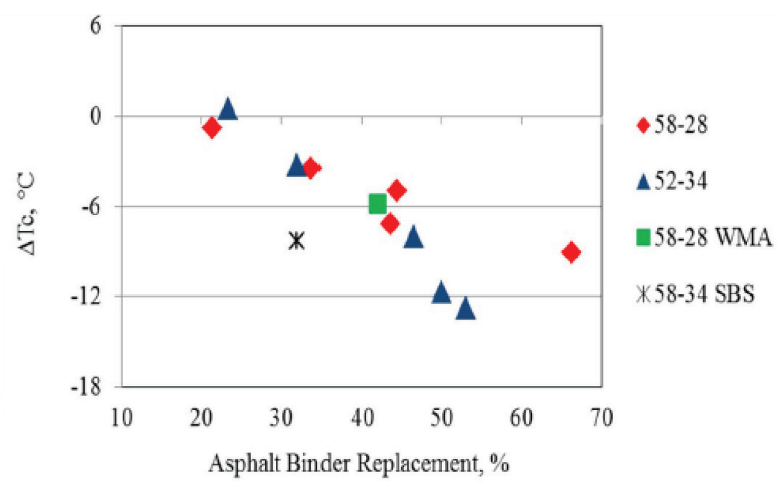


Figure 23. Change in ΔT_C with Asphalt Binder Replaced by RAP and RAS (18)

Corrigan and Golalipour concluded that for the recycled materials used, ΔT_C decreases at the rate of about 0.2°C per percent asphalt binder replacement.

Re-refined Engine Oil Bottoms (REOB)

In 2016, the Asphalt Institute published a state-of-the-knowledge report (19) on the use of re-refined engine oil bottoms (REOB), also known as vacuum tower asphalt extender (VTAE). (It should be noted that since 2016, the expression “REOB” has become standard lexis for these materials, thus REOB is used throughout this report.) Those seeking more information on the influence of REOB on performance properties of asphalt binders and mixtures are encouraged to review Chapter 5 of that report. Several of the more relevant studies contained in IS-235 are presented briefly as follows. Their original source also is presented in the list of references for this document.

Not every REOB source will have the same effect on ΔT_C. Bennert (20) mixed REOB from two sources with the same PG 70-22 to produce PG 64-22 and then evaluated ΔT_C of the resulting blends. For comparison purposes, Bennert also evaluated ΔT_C for an unmodified PG 64-22. Figure 24 shows results of this experiment. The most obvious result is that both REOB-modified binders exhibit more negative values of ΔT_C compared with the standard PG 64-22. Additionally, both REOB sources indicated the same ΔT_C after 20 hours of PAV aging, yet after 40 hours of PAV aging REOB Source #2 indicated a sharper decline in ΔT_C compared to REOB Source #1.

Bennert (20) also demonstrated (Figure 25) the intense effect of high REOB dosage rate on ΔT_C.

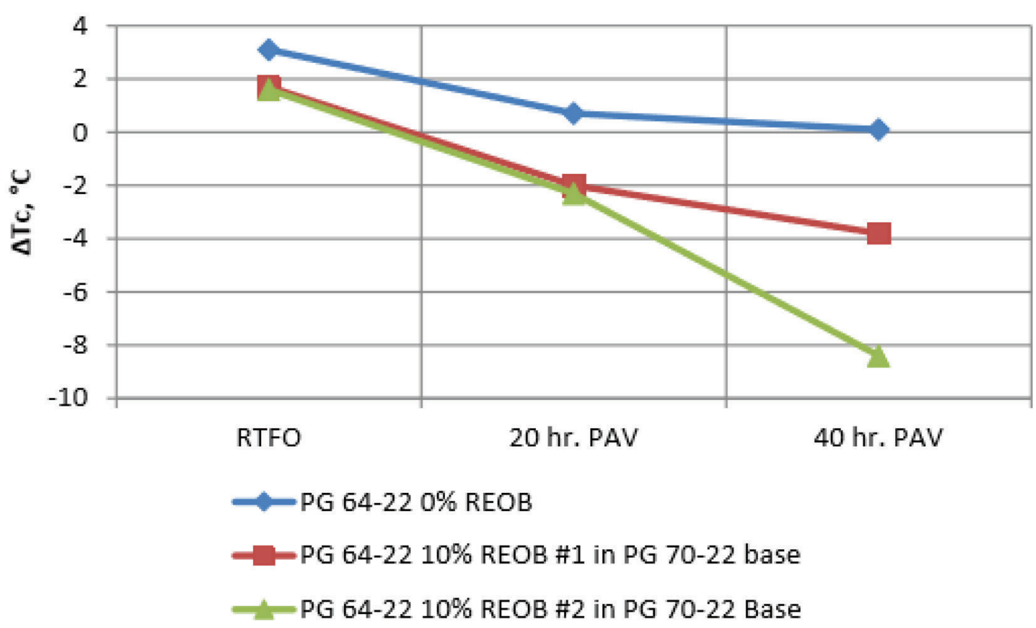


Figure 24. Effect of REOB Source on ΔT_C (20)

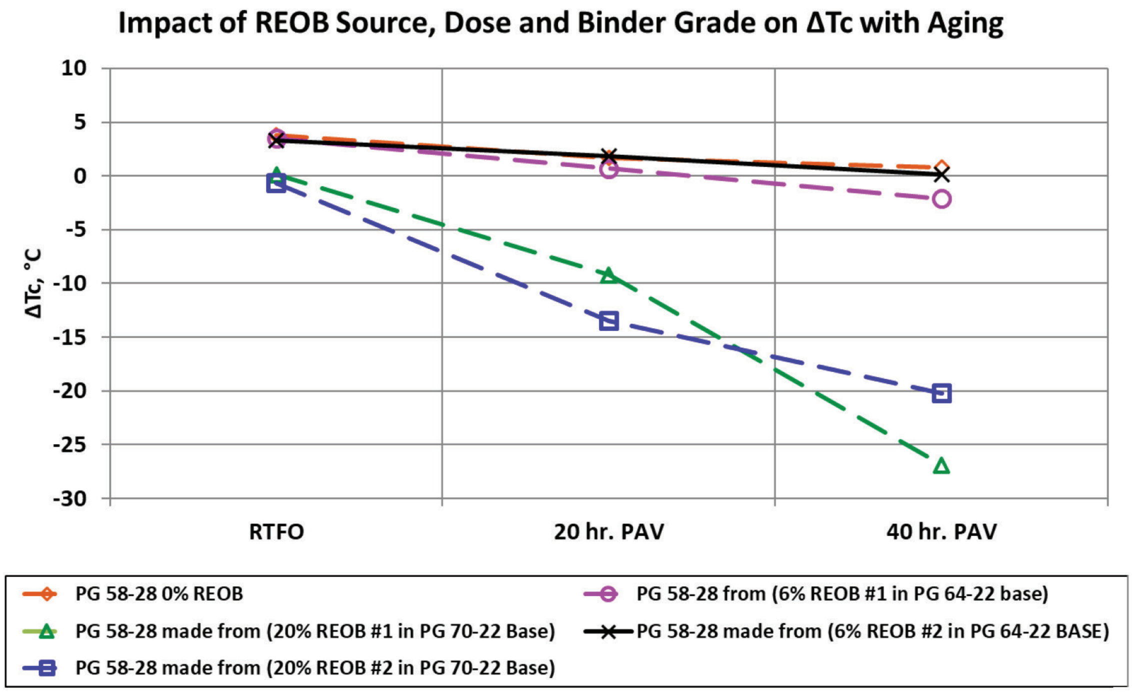


Figure 25. Effect of REOB Dosage Rate on ΔT_C (20)

In Figure 26, Bennert shows ΔT_C for five PG 58-28 binders. Two were created by dosing PG 70-22 with 20% of each of the two REOB sources. Another two were created by dosing PG 64-22 with 6% of each of the two REOB sources. For comparison, the ΔT_C properties of a straight run PG 58-28 are also shown. The most obvious trend is that lower REOB dosed PG 58-28 binders (i.e., REOB added to PG 64-22) share similar ΔT_C characteristics with the straight run PG 58-28. Yet PG 58-28 binders with the higher REOB dosage (i.e., REOB added to PG 70-22) exhibit a more intense decline in ΔT_C at 20 hours of PAV aging and even more at 40 hours of PAV aging.

To illustrate the effect of REOB on pavements with known cracking performance, Reinke, et al. (6) evaluated materials collected from cores taken from test sections on County Trunk Highway (CTH) 112 in Olmsted County, MN. The test sections were placed on top of aggregate base, contained the same aggregate, and were paved on the same day by the same paving crew. The only difference in the test sections was the asphalt binder. The cores were secured 8 years post construction. Four binders were used and designated as:

- MN1-2 (polymer modified PG 58-34),
- MN1-3 (PG 58-28),
- MN1-4 (PG 58-28 containing estimated 8% REOB), and
- MN1-5 (PG 58-28).

Reinke, et al. recovered the binder from the top ½-inch and evaluated the ΔT_C of the recovered binder. The results are shown in Figure 26. The red regression line represents the relationship between ΔT_C and transverse cracking. Because the line is flat, relatively little of the variation in ΔT_C from the four binders explains the variation in transverse cracking. The green regression line represents cracking performance where transverse crack length is removed from total crack length and the red line is total crack length. Total crack length includes load- and non-load-associated cracking as well as transverse cracking. (Even though transverse cracking is considered non-load-associated cracking, it was treated separately in this analysis.) In either case, ΔT_C is very predictive of observed cracking performance with the notable exception that ΔT_C was not related to transverse cracking. Nevertheless, the test section containing MN-4 binder with REOB indicated the greatest extent of cracking in all cases.

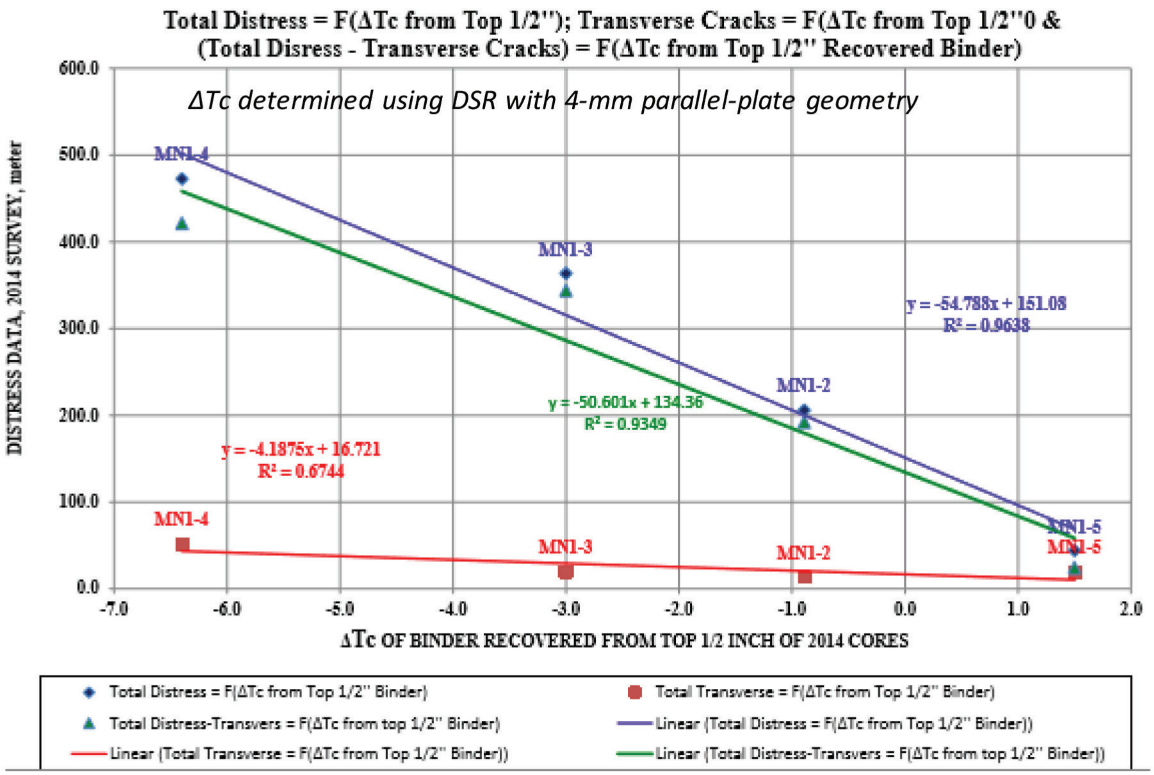


Figure 26. Relationship Between ΔT_c and Cracking Performance (7)

Figure 14 was previously presented to illustrate that different asphalt binders can exhibit different rates of change in ΔT_C with increased laboratory aging. The data in Figure 13 came from three sections at the MnROAD pavement test track (6). Each section had a unique PG binder. The PG 58-28 and PG 58-34 exhibited similar rate of decline in ΔT_C behavior when subjected to increasing levels of laboratory aging. By comparison, the PG 58-40 contained REOB and exhibited the characteristic steeper decline in ΔT_C between 20 and 40 hours of PAV aging.

Li, et al. (21) conducted an experiment to evaluate the effect of REOB dosage rate on ΔT_C. The experiment involved dosing a PG 58-28 with 2.5, 6.0, and 15.0 percent REOB and evaluating the effect of these dosage rates on ΔT_C. The results are shown in Figure 27. At lower dosage rates, for both 20 and 40 hours of PAV aging, ΔT_C values were similar. Also, both aging protocols indicated a decline in ΔT_C with higher amounts of REOB. However, the decline in ΔT_C was significantly greater for 40 hours as compared to 20 hours of PAV aging. Thus, Li, et al. concluded that it was necessary to utilize 40 hours of PAV aging to adequately detect the presence of high amounts of REOB.

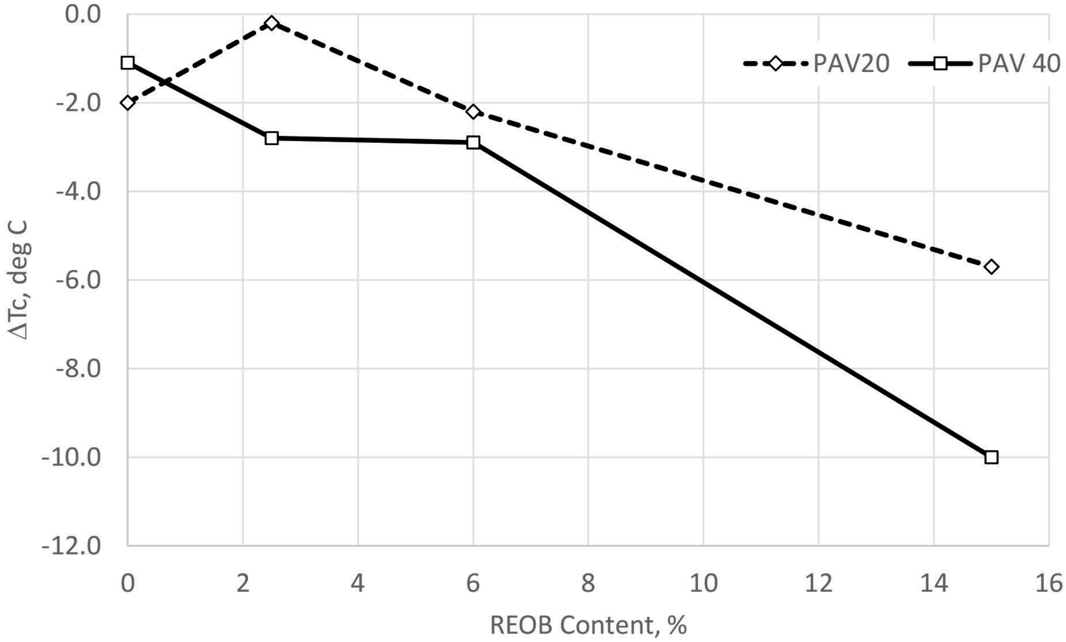


Figure 27. Effect of Extended Aging on ΔT_C for PG 58-28 Containing REOB (21)

Combined Effects

In addition to individual factors which affect ΔT_C, a combination of factors can also affect the relaxation properties of the effective asphalt binder.

In an internal study (22), the Asphalt Institute generated data from two projects in Kentucky where RAP and RAS were being used in the asphalt mixtures. Table 10 summarizes the details of both projects.

Table 10. Asphalt Binder Characteristics for Bullitt County and Fleming County Projects (22)

Project	RAP/RAS, % of mix	Recycled Binder Ratio	Virgin Asphalt Used	Estimated Zin_C Content in asphalt, ppm	Estimated REOB Content, % of binder	Virgin Asphalt ΔTc, °C (20-hr PAV)
Bullitt	10/3	0.25 total 0.12 RAP 0.13 RAS	Source A PG 58-28	11	0	1.8
Fleming	10/3	0.19 total 0.09 RAP 0.10 RAS	Source B PG 58-28[1]	1085	18	-14.3

[1] later found to be PG 58-25.7

In both mixes, RAP was being used at 10 percent by weight of the mix and RAS was being used at 3 percent by weight of the mix. The Bullitt County mix had a total reclaimed binder ratio (RBR) of 0.25, split approximately evenly between contribution from the RAP binder (0.12) and contribution from the RAS binder (0.13). The Fleming County mix had a total RBR of 0.19, split almost evenly between the RAP binder (0.09) and the RAS binder (0.10). Both mixes were produced with PG 58-28 asphalt binders, although supplied from different sources

The virgin asphalt binder used in the Bullitt mix was a PG 58-28 from Source A with a ΔT_C of -1.8°C after standard PAV aging (i.e., 20 hours). The virgin asphalt binder used in the Fleming mix was identified as a PG 58-28 (although Asphalt Institute testing determined the continuous grade as -25.7°C) from Source B with a ΔT_C of -14.3°C after standard PAV aging. Because of the very low value of ΔT_C for Source B, XRF testing was conducted on both binders. The results showed that the Source A virgin binder had 11 ppm zinc (with ΔT_C of -1.8C) and the Source B virgin binder had 1085 ppm zinc (with ΔT_C of -14.3°C). According to Arnold and Shastry (23), the amount of zinc present in an asphalt binder may be related to the amount of REOB/VTAE used through the following equation:

Zinc (ppm) = 59.834*(REOB%) +13.247

Although the equation was based on the asphalt binders tested at the time, and may not apply to all materials, it should provide a relative estimate of the amount of REOB/VTAE present. Using the data above, Source A would be estimated to have effectively no REOB/VTAE while Source B would be estimated to have approximately 18% REOB/VTAE present.

Black Space curves were generated for the RTFO and PAV-aged virgin binder, the recovered RAP binder, the recovered RAS binder, and the recovered binder from the loose asphalt mixture for both the Bullitt and Fleming mixes. This data is illustrated in Figures 28 and 29. For reference, the procedure used the DSR with 8-mm parallel plate geometry at 1% shear strain, 3 or more temperatures between 5 and 30°C, and loading frequencies of 0.1 to 100 rad/s (spaced evenly on a log scale at 10 points per decade) with a reference temperature of 20°C for mastercurve determination. The curves are smoothed data derived from the mastercurve equations.

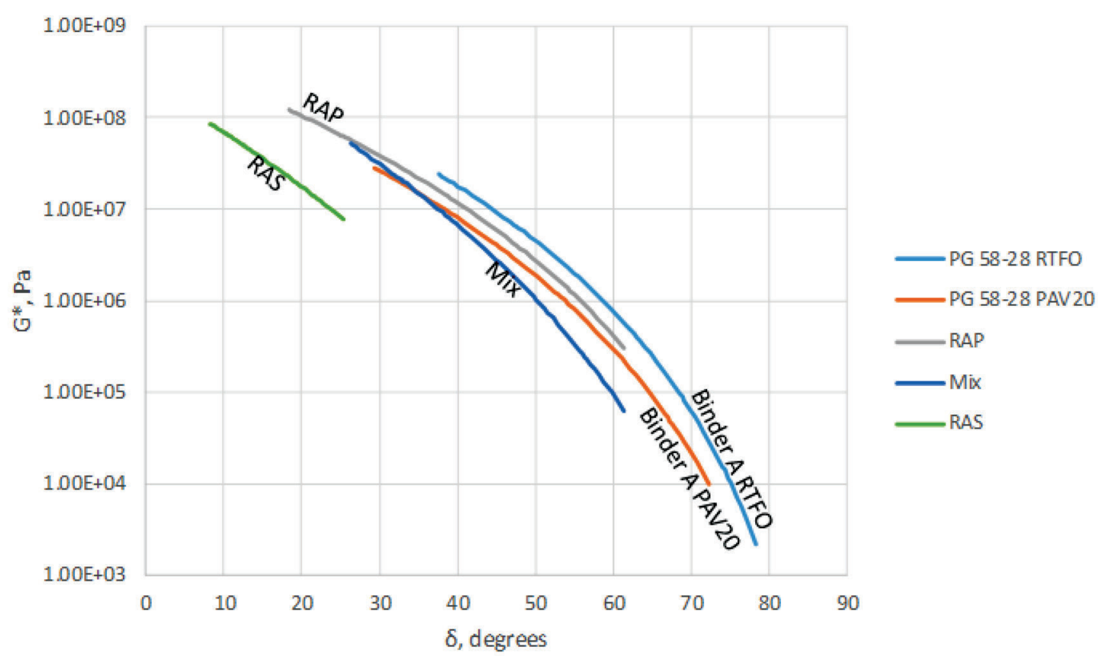


Figure 28. Black Space Curves from Bullitt County Project (22)

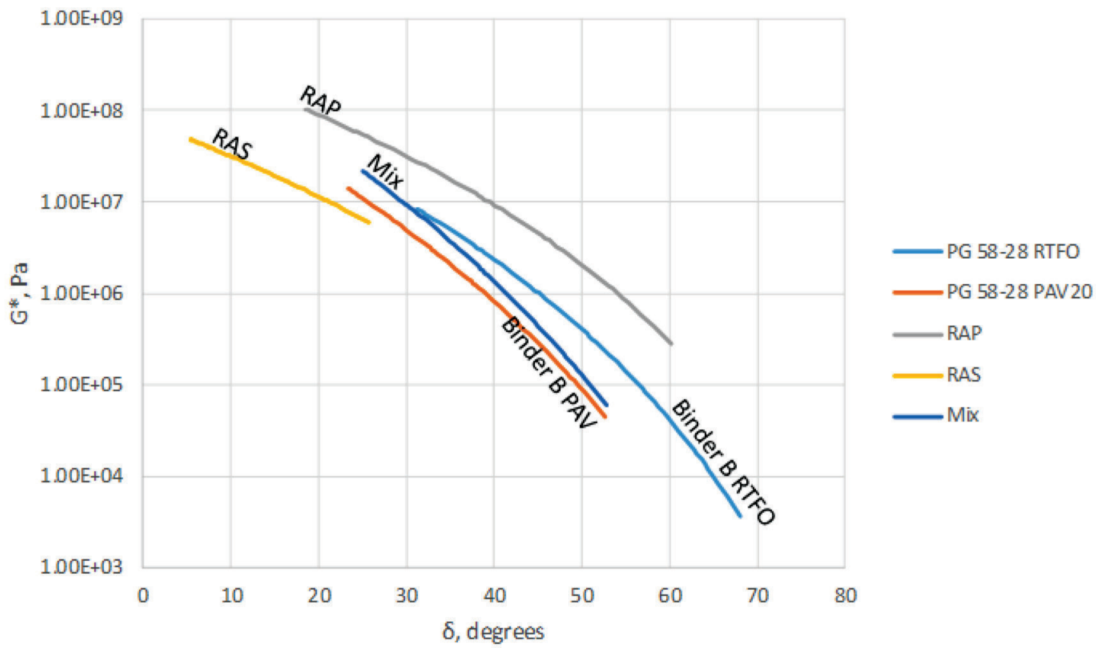


Figure 29. Black Space Curves from Fleming County Project (22)

In Figure 28, the recovered asphalt binder from the loose mix looks to have worse relaxation properties in the as-recovered state than even an asphalt binder that was subjected to standard PAV aging. This is largely driven by the RAS binder which is far to the upper left corner. The recovered RAP binder looks very similar in relaxation properties, albeit stiffer, to the PAV-aged virgin asphalt binder.

By contrast, in Figure 29, the recovered asphalt binder from the loose mix is actually helped a bit by the RAP since the virgin asphalt binder (Source B) has such poor relaxation properties.

Although ΔT_c values aren't shown, general experience indicates that with all other conditions being equal, Black Space curves that are shifted to the left have lower ΔT_c values.

Finally, Figure 30 is a Black Space graph showing the Gulf Southeast (GSE) asphalt binder from the AAPTP 06-01 study at 20, 40, and 80 hours of PAV aging (1). The recovered loose mix curves (plant-produced, no additional aging) from the Bullitt and Fleming projects are also shown. If the RAP and RAS binder is considered to be active, or mostly active, in the mix and not just a black rock then the curves suggest that these mixes, which in theory should produce blended PG 64-22 asphalt binders, are being placed with relaxation properties that simulate 80 hours of PAV aging or more in a virgin mix. In other words, the mixes have properties that make them appear from a relaxation standpoint to be 8-10+ years old when initially placed. The use of an asphalt binder with poor ΔT_c (Fleming, Source B) makes the relaxation properties look even worse.

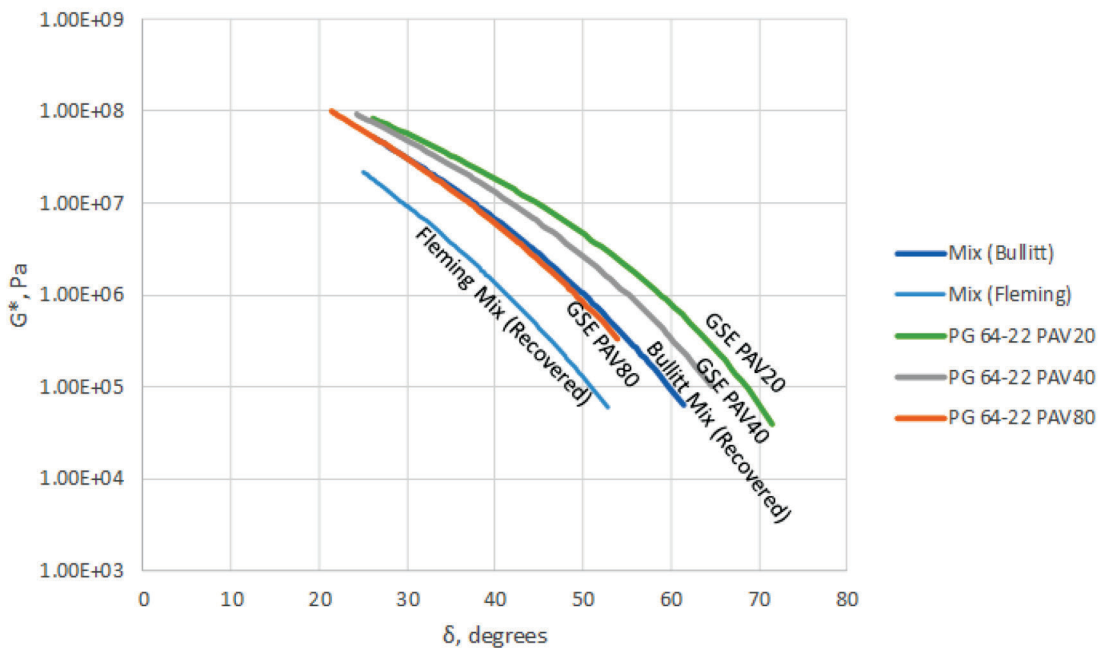


Figure 30. Black Space Curves Illustrating Combined Effects of RAP, RAS and Virgin Asphalt Binder (22)

Elastomeric Polymer Modification

At the time this document was developed there was an ongoing dialog among asphalt technologists concerning the validity of characterizing the durability of polymer modified binders using ΔT_c. It is generally accepted that elastomeric polymer modification improves durability in asphalt pavements. Yet Kluttz (24) has pointed out that certain features of polymer modification may have a worsening effect on ΔT_c and therefore make it appear as if polymer modified binders will exhibit diminished durability.

It is generally accepted by asphalt technologists that reduced laboratory aging (both RTFO and PAV) occurs in polymer modified binders because of their higher viscosity at standard aging temperatures. This phenomenon is readily apparent in situations when G*/sin δ of RTFO residue governs the high temperature grade of a polymer modified binder. This reduced aging will cause a slightly improved BBR S, which would result in a favorable decrease in $T_{c,S}$. However, Kluttz (24) points out that because styrene-butadiene-styrene (SBS) is elastic this characteristic will inhibit viscous flow, even at very low temperatures and result in less relaxation. Recall the example of the fixed beam shown in Figure 10. A highly elastic polymer modified binder would slow down the shedding of stresses, thus exhibiting a lower m-value. Less relaxation causes m-value to decrease, which in turn causes an unfavorable increase in $T_{c,m}$. Considering the equation $\Delta T_c = T_{c,S} - T_{c,m}$ the effect of polymer modification on ΔT_c will be decided by the net effect of the decrease in $T_{c,S}$ and increase in $T_{c,m}$. Figure 31 illustrates this potentially unfavorable but possibly misleading effect of polymer modification on ΔT_c.

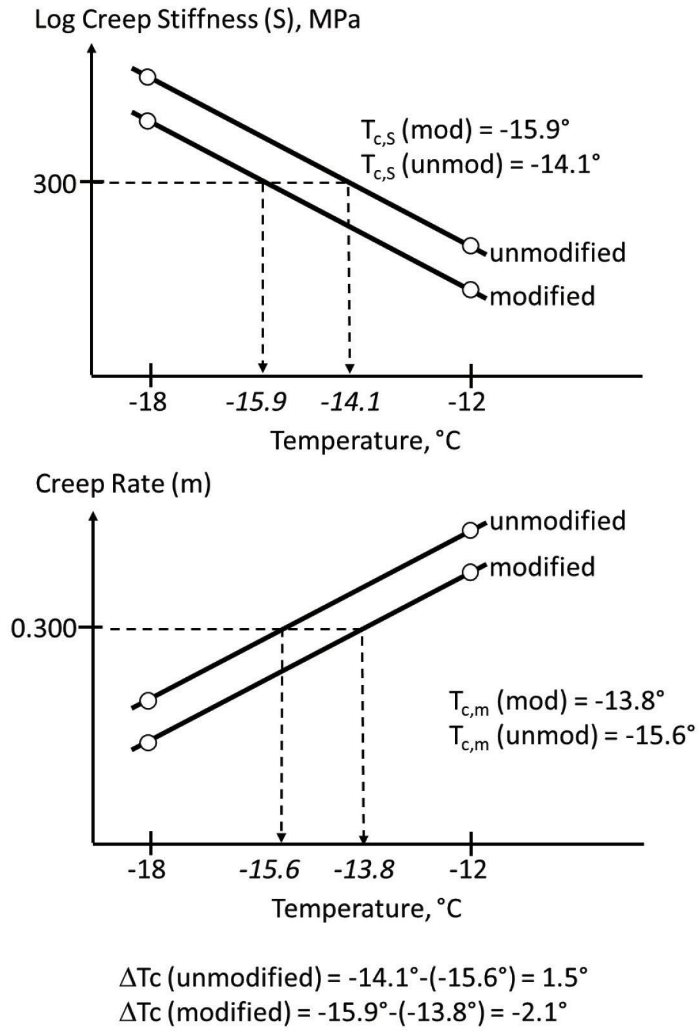


Figure 31. Possible Effect of Polymer Modification on ΔT_C

The degree to which scenario demonstrated in Figure 31 likely depends on several factors. It would be expected that very highly polymer modified binders would exhibit this trait. The quality of compatibility between the asphalt and polymer could also be expected to play a role. It is possible that a highly compatible combination of polymer and asphalt would suffer more from the scenario illustrated in Figure 31 and indicate a low ΔT_C. On the contrary, poor compatibility either from poor manufacturing or lack of affinity between asphalt and polymer could result in a comparatively favorable ΔT_C.

Kluttz (24) presented experimental data that showed ΔT_C is influenced by high levels of elastomeric polymer. In this experiment, three asphalt binders were evaluated:

- PG 64-22,
- PG 64-22 with 3% SBS, and
- PG 64-22 with 7.5% SBS.

Each binder was evaluated for various rheological properties. Table 11 shows the results.

Table 11. Rheological Properties of PG 64-22 Modified with SBS (24)

Asphalt Binder	$T_{c,Low}$, °C	G*sin d @ 25°C, kPa	$T_{c,S}$, °C	$T_{c,m}$, °C	ΔT_c, °C
PG 64-22	-24.0	3930	-27.6	-24.0	-3.6
PG 64-22+3% SBS	-25.9	3320	-29.8	-25.9	-3.9
PG 64-22+7.5% SBS	-24.0	2450	-31.5	-24.0	-7.5

Most notable is that the ΔT_C value for the highly modified PG 64-22 is about twice as low (more negative) compared to the base asphalt or base asphalt with lower SBS content. Overall this data would suggest that ΔT_C is either unaffected or negatively affected by the presence of an elastomeric polymer depending on polymer content. This is a counterintuitive trend as it is an article of faith among the asphalt pavement and materials engineering community that elastomeric polymer modification enhances the performance characteristics of asphalt materials.

Kluttz also points out (24) that several rheological properties also appear to be negatively influenced by polymer modification. These include R-value and the Glover-Rowe parameter, as well as ΔT_C. Crossover temperature (the temperature at which phase angle is exactly at 45°) is increased due to the shift toward a more elastic binder. He attributes this counterintuitive behavior to a flattening of mastercurves for elastomeric-modified binders, a trait that is shared with binders that are prone to excessive aging. In other words, a flattening of the master curve (more negative, unfavorable ΔT_C) for an unmodified binder would indicate age-related embrittlement. On the contrary, Kluttz contends that the normal and expected effect of polymer modification on the mastercurve and the effects of aging may lead to a worsening of ΔT_C despite the expected improvement in toughness and cracking resistance.
Anderson (12) supports Kluttz' position by pointing out that polymer modified binders generally exhibit a higher elastic component as evidenced by lower phase angle at a given temperature or stiffness. Because of the effect on phase angle, he believes that some modified binders will exhibit lower (more negative) values of ΔT_C.

Kriz (25) suggests it is possible that polymer modified binders may exhibit lower ΔT_C but still not suffer from age related embrittlement. He attributes this to the fact that polymer modified binders possess a strong network that itself resists age-related cracking. He also posits that the binder will relax stresses faster within the network and likewise not suffer from age-related cracking.

Examination of the Kansas DOT data presented in Table 1 offers some insight on the effect of polymer modification on ΔT_C. Kansas DOT specifications (26) require increasing levels of elastic recovery depending on PG temperature spread according to Table 12. The practical effect of the provision stated in Table 12, note 1, is that Kansas DOT excludes the use of acid modification.

Table 12. Kansas DOT Elastic Recovery Requirements (26)

Temperature Spread, °C	Minimum Elastic Recovery[1,2]	Separation[3], °C maximum
92	60	2
98	65	2
104	75	2
110	80	2

1. Performa all tests after adding 0.5% high molecular weight amine antistripping agent.
2. ASTM D6084, Procedure A.
3. ASTM D7173, run on original binder.

Within this specification environment, it is reasonable to assume that binders with a temperature spread of 92°C or greater contain elastomeric polymers. Alternatively, temperature spreads of less than 92°C are not modified, at least with elastomeric polymers or PPA. Using these assumptions, Table 13 shows ΔT_C values for the Kansas DOT prequalification data from Table 1 sorted by presence of polymer modification.

Table 13. Average ΔT_C Values for Kansas DOT Samples

Type of Binder	PAV Aging		Loss in ΔT_C (PAV20 - PAV40)
	20 hours	40 hours	
Unmodified	-0.6	-4.0	-3.4
Modified	-0.3	-3.3	-3.0
Overall	-0.4	-3.6	-3.2

When comparing unmodified versus modified binders, there is not a practical difference in ΔT_C for either 20 or 40 hours of PAV aging. Likewise, the loss in ΔT_C from extended aging was practically the same for unmodified and modified binders.

As part of the task force effort to produce this document, Heptig (27) prepared Figure 32 that contains data for the 72 asphalt binders shown in Table 1. Thirteen of them do not meet the 40-hour PAV ΔT_C requirement of ≥ -5.0. Eight of the 13 binders that did not meet this requirement are unmodified and the other five are polymer modified.

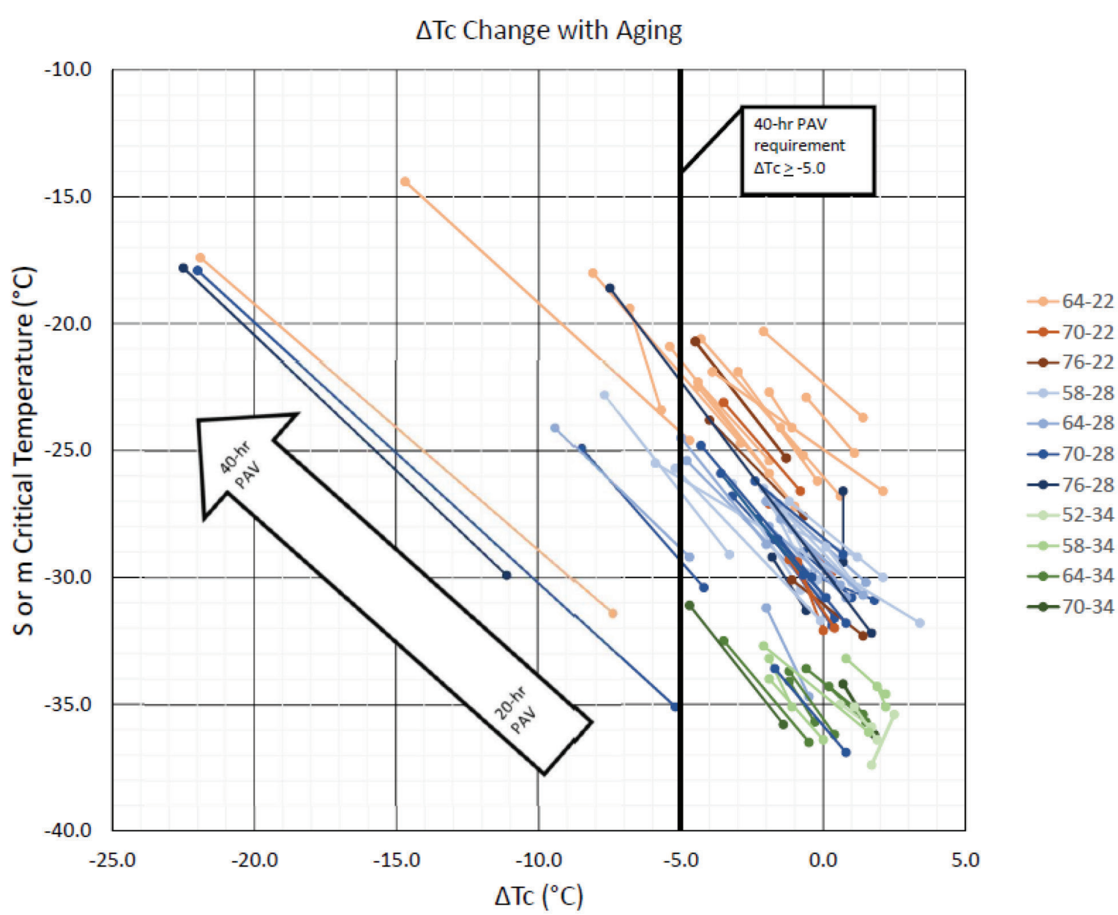


Figure 32. Effect of Binder Grade and PAV Aging on ΔT_C (27)

Overall, the Kansas DOT data does not support the contention that polymer modification of asphalt influences ΔT_C, either favorably or unfavorably. Although some of the modified binders analyzed by the Kansas DOT have indicated low ΔT_C values, the majority of their data to date indicates favorable values of ΔT_C. However, it should be noted that if the Kansas DOT ΔT_C data were sorted, for example by presence of other components, then the effect of polymer modification on ΔT_C might gain more clarity. A previous section in this report showed that REOB, particularly at high dosage rates, affects ΔT_C. Kansas DOT personnel (27) suspect that some of the binders contain REOB or other type of softening agents, especially the binders with more negative 40-hour PAV ΔT_C values. It is possible that the presence of these materials is overwhelming the effect of polymer modification on ΔT_C and confounding the comparison.

Woo, et al. (28) point out that polymer modification causes improvements in ductility to the asphalt upon which the binder is based. They also point out, however, that this ductility improvement is degraded over time due to two factors: (1) oxidative aging of the base asphalt and (2) polymer degradation ("polymer size reduction") caused by oxidation. Therefore, the ability of laboratory aging protocols to model this two-step reduction in ductility is important to fully understand the effect of polymer modification on ΔT_C. In other words, it is possible that to some extent the effect of polymer modification on ΔT_C is being confounded by the laboratory aging protocols, a point that has also been made by Kluttz (24).

Given the information in this section, it becomes obvious that the net effect of elastomeric polymer modification on ΔT_C needs a clearer understanding. It is hoped that existing and future research will offer clarity on this important topic.

Other Asphalt Characteristics

Kriz, et al. (29) related asphalt composition to ΔT_C. Phase instability – the imbalance of asphaltenes, resins, and oils in the asphalt binder molecular structure – has been observed in asphalt binders with low values of ΔT_C. These asphalt binders may also exhibit a significant increase in stiffness from the unaged (original condition) to the RTFO-aged condition, meaning they will have aged more significantly during production and construction than other asphalt binders. Premature aging may also lead to premature cracking in the asphalt mixture. The most typical example cited for this type of condition is oxidized asphalt. Table 14 shows the results of a ΔT_C analysis on a PG asphalt binder produced via oxidation (30).

Table 14. ΔT_C Analysis of Oxidized PG Binder (30)

BBR Property	20-hour PAV			40-hour PAV			
	BBR Test Temperature						
	-12°C	-18°C	-24°C	-6°C	-12°C	-18°C	-24°C
Creep Stiffness (S), MPa	96.6	196	365	54.6	107	200	385
Creep rate (m)	0.313	0.288	0.244	0.313	0.285	0.263	0.234
$T_{c,S}$	-32.1			-31.7			
$T_{c,m}$	-25.1			-18.7			
ΔTc	-7.0			-13.0			

The BBR results for the oxidized binder exhibit pronounced m-control and a large decrease in ΔT_C with additional PAV aging. However, when compared with other types of oxidized binders (see Tables 8 and 9) the effect of oxidation on ΔT_C for this paving asphalt is less significant.

Furthering their discussion of phase instability, Kriz, et al. (29) also related ΔT_c to asphalt binder phase angle. Although ΔT_c is determined from low temperature testing using the BBR, they point out that it is related to the phase angle at intermediate temperatures. Figure 33 shows the relationship between ΔT_c and phase angle at a constant stiffness. A phase angle lower than 45 degrees at a given stiffness means that the asphalt binder exhibits more elastic solid behavior than viscous behavior. As the phase angle decreases and the elastic solid behavior increases, the ability to dissipate energy through viscous flow (i.e., relax applied stresses) is diminished, which some believe is related to non-load related cracking seen at intermediate temperatures.

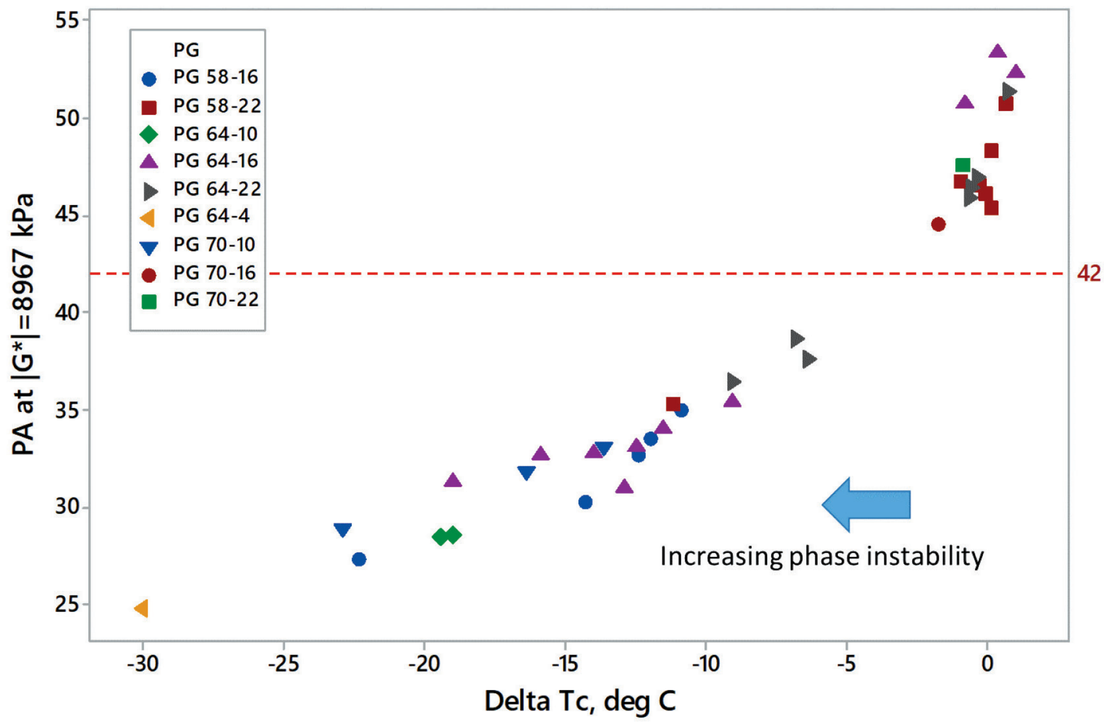


Figure 33. Relationship Between ΔT_c and Phase Angle at Intermediate Temperatures (23)

Erskine, et al. (31) also evaluated the effect of composition on low temperature properties of aged asphalt binders. Although they did not identify ΔT_c as a stated independent variable in their experiment, they did use the concept of $T_{c,S}$ and $T_{c,m}$. Their experiment evaluated 11 asphalt binders produced from at least four crude oil sources and various modification strategies including PPA, styrene-butadiene diblock, air blowing, REOB, reactive elastomeric terpolymer, and combinations of these. Various forms of PAV aging were used involving 20- and 40-hour periods with varying amounts of asphalt in the PAV pans. In one case, moist air was use during a 20-hour PAV aging period. The experiment involved a partial factorial experiment involving 11 binders of different composition and 5 aging protocols for a total of 44 combinations. Figure 34 shows results of the Erskine, et al. experiment. In Figure 34, harsher aging conditions proceed up and to the right.

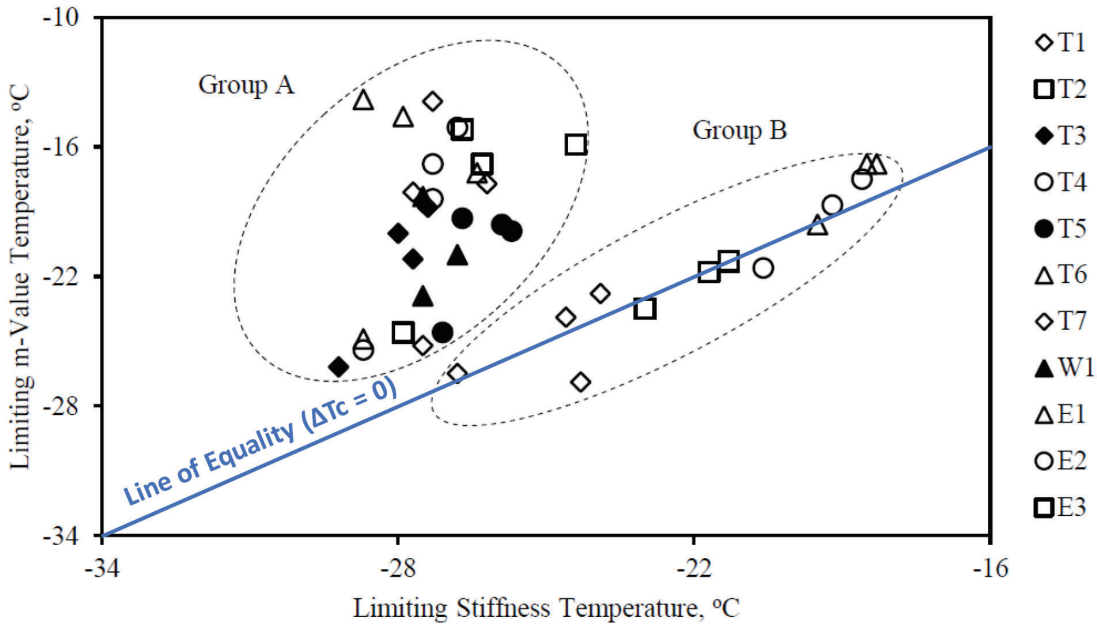


Figure 34. Limiting Temperatures ($T_{c,S}$ and $T_{c,m}$) for Binders of Different Composition and Aging (31)

Erskine et al. identified two groupings of data. Group A binders "rapidly lose their relaxation ability" and are characterized as gel asphalts with very low stiffness. In other words, their $T_{c,m}$ is declining faster than their $T_{c,S}$ and as such, the binders are becoming more and more m-controlled and indicate more negative ΔT_c. Erskine, et al. point out that the worst performers – Binders T2, T4, T6, and T7 (all in Group A) – are either air blown or modified with REOB. Planche (32) notes that the negative effect of REOB on ΔT_c is more pronounced when added to a gel asphalt. In that situation, the impact of REOB is similar to paraffins by causing phase separation of asphaltenes.

Group B binders exhibit less loss of stiffness and relaxation upon harsher aging and indicate less negative ΔT_c. Binders T1 and E1-E3 were identified as made with Lloydminster or Cold Lake crude oil, which are generally considered to be very durable asphalt binders. Erskine, et al. attribute this desirable characteristic to their high naphthenic hydrocarbon content and low linear paraffin content. They exhibit less asphaltene formation under more or harsher aging conditions.

5.0 CONSIDERATIONS WHEN USING ΔT_C

Distress Types that ΔT_C Addresses

As previously stated, ΔT_C was originally envisioned to be a parameter that could be measured on an in-service, aged pavement to determine the optimum time for a pavement preservation treatment. Even though ΔT_C originally targeted block cracking caused by age-related embrittlement, ΔT_C is sometimes considered as a parameter that will indicate the role of asphalt binder in addressing other types of asphalt pavement distress. In that case, it is necessary to understand how low values of ΔT_C influence the various asphalt pavement distress types. Table 15 shows an estimate of the efficacy of ΔT_C in terms of its ability to preclude various distress types when used in a purchase specification environment. The distress types shown are from the Distress Identification Manual for the Long Term Pavement Performance Project (33). The table shows three potential effects of ΔTc. Direct effect means that ΔT_C directly influences the distress type. Indirect effect means the distress type is predominately caused other factors, but ΔT_C may play a supporting role. No effect means that the distress type is unrelated to a low value of ΔT_C. The effects listed in Table 15 were developed via a survey of the Asphalt Institute task force members charged with developing this document. That survey is discussed in greater detail in Section 7.0 of this document.

Table 15. Efficacy of ΔT_C in Precluding Various Asphalt Pavement Distress Types

Distress Type	Effect of ΔT_C
Block Cracking	Direct effect
Fatigue Cracking	Indirect effect
Edge Cracking	Indirect effect
Longitudinal Cracking	Indirect effect
Reflection Cracking	Indirect effect
Transverse Cracking	Indirect effect
Potholes	Indirect effect
Raveling	Indirect effect
Rutting	No effect
Shoving	No effect
Bleeding	No effect
Polished Aggregate	No effect
Lane-to-shoulder drop off	No effect
Water Bleeding and Pumping	No effect

In general, this data suggests that asphalt technologists believe that various forms of cracking damage are, to a greater or lesser extent, influenced by ΔT_C due to its ability to predict age embrittlement. However, it should also be pointed out that cracking damage is also influenced by many other factors such as low voids in the mineral aggregate, low effective binder content, poor compaction leading to high air voids, thin asphalt layers in combination with excessive deflection, the dust fraction that forms the mastic, etc. Said differently, the ΔT_C parameter should not be considered a panacea for favorable asphalt pavement cracking performance.

Recovered Binder in Relation to ΔT_c

Knowing the ΔT_c of a binder that will be used on a project is a worthwhile piece of data, but it cannot guarantee long term performance. There are too many other factors that must be considered. If the project does not contain recycled material, then the ΔT_c of the job binder is most important because there are no other binder sources to interact with the virgin binder. Of course, mix design properties, binder content, air voids of the pavement, base and subgrade properties are all factors that can result in pavement distress regardless of the binder quality.
In addition to the factors just mentioned, the presence of RAP and/or RAS in the mix can have a significant impact on long term pavement performance regardless of the quality of the ΔT_c of the virgin binder used on a project. As previously mentioned, RAP and/or RAS will degrade ΔT_c of the virgin binder with which it is blended. Therefore, it is advisable that the quality of the total binder (i.e., virgin binder plus RAP and/or RAS) be evaluated for ΔT_c to ensure that the total binder does not have a detrimental effect on long-term durability.

If one wants to make the best use of ΔT_c it is recommended that a sample of lab mix be produced that contains the RAP that is to be used on the project (34). The lab sample should also contain the amount of virgin binder to be used in the mix as determined from the mix design. The laboratory-produced mix should be subjected to the standard two-hour loose mix aging at 135°C. Finally, the binder from the aged mix should be recovered for determination of ΔTc.

It is strongly advised to conduct the extraction using toluene as a solvent (preferably not trichloroethylene) and then recover the binder using a rotary evaporator procedure as specified in ASTM D7906. If trichlorethylene must be used, it is imperative that the ASTM D7906 be followed to produce recovered binder. The Abson method, no matter how carefully conducted, will excessively age the recovered binder and adversely affect the ΔT_c value. After the binder is recovered, subject it to 20 and 40 hours PAV aging. This will require recovering about 10 grams more than 100 grams of binder so that there is sufficient material for two 50-gram PAV pans for aging. If the amount of recovered binder is sufficient for only one PAV pan, then PAV age the binder for 40 hours and determine ΔT_c on that sample.

Precision

Because ΔT_c is a property derived from BBR testing, the precision of ΔT_c is summation of the precision of the BBR test, which itself is subject to the precision of the RTFO and PAV tests. Anderson (12) used the single operator and multiple laboratory d2s% precision statements for creep stiffness and creep rate in AASHTO T313 to estimate how ΔT_c varies within those measures of repeatability and reproducibility, respectively. Using typical S- and m-values for an unmodified PG 64-22 he calculated that ΔT_c precision for a single operator is 0.8°C and for multiple laboratories is 1.8°C. To put these numbers in perspective, Anderson points out that the limit on ΔT_c in AASHTO PP78, "Provisional Practice for Design Considerations When Using Reclaimed Asphalt Shingles in Asphalt Mixtures", is -5°C. The single and multiple laboratory precision values are 16 and 36 percent of that limit, respectively.

Practical Considerations

Two factors inhibit routine measurement of ΔT_c. First, more aged binder is required for the BBR test to be conducted at two temperatures to define $T_{c,s}$ and $T_{c,m}$. Second, extended aging in the PAV delays test results for an additional 20 hours beyond normal PAV aging.

The first issue potentially could be solved by the use of the DSR using 4-mm sample geometry. Scientists at the Western Research Institute (WRI) conceptualized and developed this method. According to Elwardany, et al., (35), use of 4-mm DSR geometry does not exist as a standard but is occasionally used in research and industrial applications. For example, based on the WRI work, Reinke and Hanz (15) developed a related methodology that captures $T_{c,s}$ and $T_{c,m}$ using the DSR by executing the test at a variety of test temperatures and loading rates. This technique saves time in that one test is used to capture low temperature continuous grading. It also requires far less material compared to BBR determinations.

Although 4-mm sample geometry shows promise in reducing the amount of sample needed for testing, Reinke (34) also points out that this must be moderated by the fact that such use is not yet widely validated for ΔT_c determinations. No coordinated studies have been conducted to validate such use and a ruggedness experiment has not been conducted to determine whether allowed variations in the test procedure have a significant effect on test results.

Table 16 shows a comparison of ΔT_c values measured on the binders that were part of the original AAPTP study (1) compared with ΔT_c values measured at WRI on the same binders using the 4-mm DSR approach (36).

Table 16. Comparison of ΔT_c Values Using BBR and 4-mm DSR Approaches (36)

Asphalt Binder	Expected Block Cracking Performance	20-hour PAV		40-hour PAV	
		BBR	4-mm DSR	BBR	4-mm DSR
WTX	Poor	-3.1	-6.2	-5.4	-10.5
GSE	Average	-1.3	-3.7	-3.8	-4.9
WC	Good	+0.6	-2.0	-1.8	-2.4

Both methods of determining ΔT_c rank the binders the same according to their expected block cracking performance. However, the two methods exhibit ΔT_c values that are about 1.0 to 5.0°C different with the 4-mm DSR method providing more highly negative values. In observing this data, Reinke (34) points out a general trend that as binder quality improves, the differences between ΔT_c from the two methods decreases. He points out that as relaxation of the binder becomes poorer, the task of getting reliable m-value results using 4-mm geometry becomes more difficult.

The second issue, impracticality of extended laboratory aging (e.g., 40 hours of PAV aging) is potentially being solved by aging thinner films of asphalt binder in a variety of apparatuses. One such approach was developed by WRI for the Federal Highway Administration called the Universal Simple Aging Test (37) or USAT. The USAT uses thin film (300 mm) short- and long-term aging as a surrogate for the RTFO and PAV tests. USAT short-term aging is accomplished by 50 minutes in a convection oven at 150°C. USAT long-term aging is accomplished by 8 hours in a PAV at 100°C. In both cases, the asphalt sample is contained in a special plate with three slots. Each slot contains 1 gram of binder that has been spread into a film 300 mm thick. The USAT only produces small samples compared to the conventional methods of laboratory aging. Yet that is less of a concern since it is the intent that residue from the USAT be tested in the DSR using 4-mm geometry.

6.0 FULL-SCALE PROJECTS AND ΔT_C

Reinke, et al. (6) have presented extensive data from test sections at the MnROAD test track and another full-scale experimental project on CTH 112. That data was previously discussed in the document to demonstrate factors that affect ΔT_C. Because pavement performance is carefully monitored and control sections are included, these and similar projects present valuable opportunities to evaluate ΔT_C properties. Two other full-scale test sites that offer insight on the use of ΔT_C: an Indiana DOT test pavement on State Route 13 and the National Center for Asphalt Technology (NCAT) test track and local pavements. In addition, pavement performance and binder test data gathered at airports managed by the Port Authority of New York and New Jersey (PANYNJ) also offer notable information pertaining to the use of ΔT_C.

Huber, et al. (38) reported the results of a test project in northern Indiana that measured ΔT_C on materials that had been in-place for five years. The objective of the study was to determine the effect of design and in-place air void content on various performance properties of asphalt binders and mixtures. Two asphalt mixtures were evaluated:

- a normal Superpave mixture (called "Superpave4") designed at 4 percent air voids and compacted in-place to 7 percent air voids (the control section) and
- a Superpave5 mixture designed at and compacted in-place to 5 percent air voids.

Both mixtures contained 7 percent RAS, which accounted for about a 20 percent RBR. Each of these mixtures were placed on State Route 13 near Middlebury, IN in 2013. Cores were taken from both sections in 2018.

After five years both sections indicated a large extent of transverse, longitudinal, and diagonal cracking. All of these were determined to be reflection cracks. However, the control mix also exhibited extensive "map" cracking (i.e., block cracking), while the Superpave5 mix exhibited none. Cores were secured at three locations in both the control and Superpave5 sections. In-place air void content in the control mix averaged 8.3 percent whereas the Superpave5 section indicated 3.6 percent air voids. Binder was extracted from the cores and evaluated for several properties including ΔT_C. Figure 35 shows the ΔT_C of the binder extracted from cores as a function of in-place air void content.

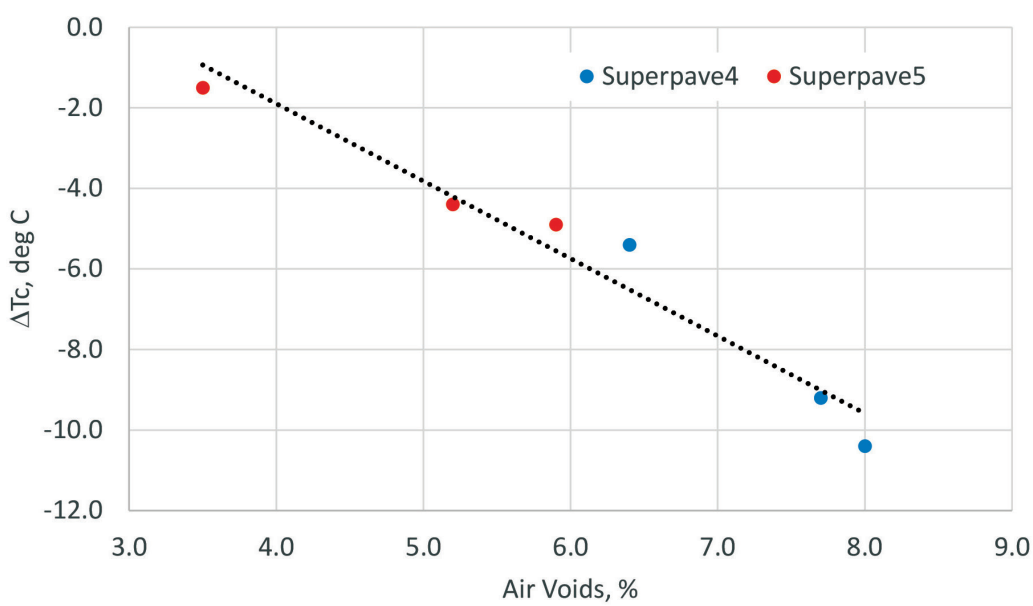


Figure 35. Influence of Air Voids on ΔT_C (38)

This data is notable for several reasons. Figure 35 shows the clear effect that asphalt mixture in-place air voids has on ΔT_C. Huber, et al. measured permeability of the cores and found at about 7.5 percent, the air voids become interconnected and water permeability rapidly increased. Thus, the higher air void content allowed air to better infiltrate the mixtures and age the asphalt binder. The result was a more negative ΔT_C. It should be noted that there was no clear indication of correlation with field observations of reflection cracking, which was identified as the dominant distress type.

Another notable feature of this study was that the Superpave5 mixtures exhibited no map cracking while the Superpave4 mixtures showed extensive map cracking. The Superpave4 mixtures exhibited ΔT_C at or below about -5°C while the Superpave5 mixtures exhibited ΔT_C at or above -5°C. This observation supports the conclusions of the original AAPTP airfield pavement study by Anderson, et al. (1). An important observation, however, is that the presence of other forms of cracking were not correlated to the observed values of ΔT_C.

Another interesting feature of this data is the fact that all the mixtures contained a relatively high RBR from RAS. Yet, when compared to the reported values of binder containing RAS (see Tables 6 and 7), the observed values of ΔT_C on the Indiana project were considerably more favorable, i.e., less negative. Huber, et al. did not comment on this feature of their data. Unlike other experiments that targeted age-related cracking, this study extracted asphalt from the entire core and not just the upper surface. It is generally accepted that binder aging is more severe in the upper portions of a surface layer. Had the authors evaluated just the upper surface, it is possible the ΔT_C values would have been more negative and more reflective of the high RAS content of the mixes.

Bennert, et al. (39) conducted a forensic study to evaluate the causes of top down fatigue cracking on airfield pavements managed by the PANYNJ. They collected cores from pavements at John F. Kennedy International Airport (JFK) and Newark Liberty International Airport (EWR). The pavements exhibited two types of cracking behavior: less than seven years old with severe cracking and greater than 12 old with little to no cracking. Through analysis of asphalt extracted from cores, this study concluded that binder aging was insignificant at depths greater than about 1-inch. As shown in Figure 36, the study also concluded that ΔT_C and the Glover Rowe parameter properly ranked the observed top down fatigue cracking performance of the JFK and EWR pavements.

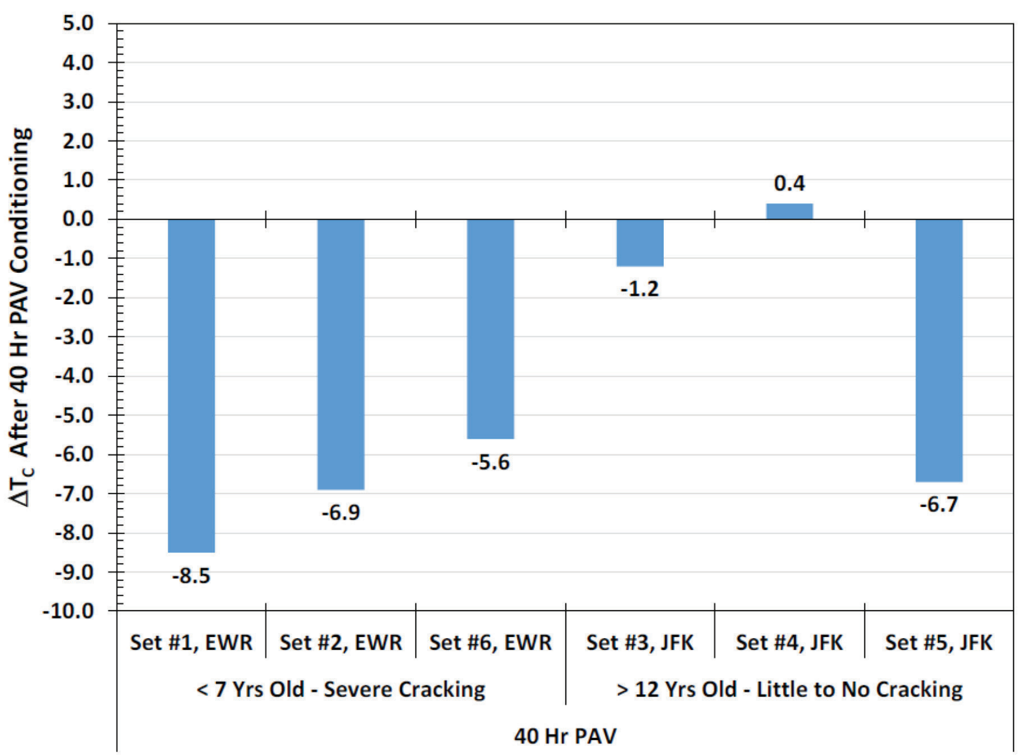


Figure 36. ΔT_C of Asphalt Binders Extracted from PANYNJ Airfield Pavements (39)

As seen in Figure 36, there was a fairly good, although not perfect, relationship between ΔT_C and observed cracking. Bennert et al. noted that JFK Set #5 did exhibit a more negative value of ΔT_C but pointed out that pavement had been in service for 15 years and was just beginning to exhibit cracking when the cores were taken. Data from the study also showed that 20- and 40-hour PAV aging simulated about seven and ten years of field aging, respectively, for the New York/New Jersey climate conditions. The results of this research project influenced the development of a ΔT_C specification by the PANYNJ.

Turner (40) evaluated ΔT_C for asphalt binder associated with materials that were collected in 2015 from untrafficked test sections originally constructed in 2000. Cracking performance for all test sites have carefully monitored. She evaluated the ΔT_C for original binder (aged using both 20- and 40-hour PAV aging) and extracted from cores using only the top ½-inch of mixture. Table 17 summarizes Turner's analysis.

Table 17. NCAT Evaluations of ΔT_C for Pavements of Known Cracking Performance (40)

			ΔT_C, °C		
Section	**Binder Type**	**Percent Cracking**	**Extracted from Cores**	**Original Binder (PAV20)**	**Original Binder (PAV40)**
E3	76-22 SBR	24.7	-4.2	-0.7	-2.9
E6	67-22	1.6	-3.8	1.5	1.0
E7	76-22 SBR	16.9	-9.3	0.7	-2.9
E9	67-22 SBS	0.2	-2.6	-1.2	-1.8

Sections E3 and E7 indicated a ΔT_C of -2.9°C, which would normally be considered to indicate non-cracking prone materials, yet those sections exhibited significant cracking. Turner suggested that for those sections, mix factors may have come into play to overwhelm the effect of binder aging on cracking tendency. Sections E6 and E9 exhibited very little cracking and likewise indicated ΔT_C values that would support that condition. When comparing ΔT_C values from original binder with that extracted from cores, Turner pointed out that 40 hours of PAV aging did not simulate 15 years of pavement aging.

7.0 PERCEIVED UTILITY OF ΔT_c

Asphalt Institute Survey

The Asphalt Institute supports the use of tests and related specification requirements that result in improved asphalt pavement performance and better life cycle costs. To gain opinions on the utility of ΔT_c, a blind, on-line survey of the task force members that developed this document was performed. The survey consisted of three questions related to the use of ΔT_c and an additional question pertaining to the distress types addressed by ΔT_c. Table 15 shows the previously discussed results pertaining to distress types addressed by ΔT_c. Table 18 shows the results of the survey pertaining to the utility of ΔT_c.

Table 18. Task Force Survey Results on the Utility of ΔT_c

Question No. 1	Answer	No. of Responses
What is your opinion on the use of ΔT_c as an asphalt binder parameter related to durability?	Don't Support; there is not enough data to support its use at this time.	2
	Conditionally support; there is evidence that it may be related to durability and is needed by users and producers to evaluate the quality of asphalt binders.	9
	Support; there is sufficient evidence to suggest that it is related to durability and is a parameter that is needed by users and producers to ensure the quality of asphalt binders.	3
	No opinion at this time.	0
Question No. 2	**Answer**	**No. of Responses**
Assuming it is used as either a research or specification parameter, what level of aging do you consider appropriate to use in evaluating ΔT_c?	PAV20 with standard conditions per AASHTO R28.	6
	PAV40 with standard conditions per AASHTO R28 except with time increased to 40 hours.	3
	Thin film PAVx (where x<40), assuming research indicates that it is a viable option to use thinner films for extended aging.	3
	No opinion	2
Question No. 3	**Answer**	**No. of Responses**
In your opinion, what is the minimum value of ΔT_c you think is appropriate for use as a durability indicator?	Evaluation: -2°C	
	-3°C	
	-4°C	
	-5°C	8
	-6°C	
	-8°C	1
	no opinion/don't know	
	Specification: -2°C	
	-3°C	
	-4°C	
	-5°C	4
	-6°C	1
	-8°C	
	no opinion/don't know	

A total of 14 task force members completed the survey. The responses in Table 18 suggest there is clear consensus that ΔT_C is a property that indicates asphalt durability and its role in age-related cracking. There is an even split between those that believe the current aging level in R28 is sufficient and those that preferred some form of extended aging, either through an extended PAV aging period or thinner films in PAV pans. About 2/3 of the responses indicated ΔT_C should remain an evaluation tool with -5°C as a limiting value. The remainder of the responses indicated ΔT_C could be used as a specification tool and again -5°C was the most cited limiting value.

There does not appear to be strong consensus among task force members on the effectiveness of ΔT_C as a specification parameter. About 2/3 of the task force believe ΔT_C has more use as an evaluation tool, whereas 1/3 feel ΔT_C could be used in a specification environment. However, there does appear to be strong consensus that ΔT_C is a property that is useful in evaluating the durability of asphalt pavements. The results indicated that laboratory aging, its ability to mimic field aging, and its influence on ΔT_C remains an issue to be addressed.

Agency Specifications

The results of the survey seem to mirror the overall sentiment of agency practitioners and others in the asphalt pavement technical community when this document was prepared. That is, some engineers favor implementation of ΔT_C as a "plus" requirement to the existing AASHTO M320 or M332 specifications. At the other extreme, some seem to have no interest in ΔT_C as a specification parameter. Among those that favor ΔT_C as a specification parameter, there is no consensus on degree of PAV aging and limits on ΔT_C. A common concern to all seems to be the need to better simulate field aging in the lab juxtaposed with the need to obtain quickly the results of quality control and quality assurance testing.

Table 19 shows the status of adoption of ΔT_C as a specification parameter among various purchasing agencies. It includes agencies that have currently implemented a ΔT_C specification along with those that expect to do so starting in 2020.

Table 19. Current[1] Status of Adoption of ΔT_C as a Specification Parameter

Agency	ΔT_C Requirement, °C	PAV Aging Duration, hrs.	Status
Florida DOT	≥ -5.0	20	Current
Utah DOT	≥ -2.0	20	Current[2]
PANYNJ	≥ -5.0	40	Current
Vermont DOT	≥ -5.0	40	Current
Maryland DOT	≥ -5.0	40	Current
Kansas DOT	≥ -5.0	40	Current
Ontario MTO	≥ -5.0	20	Current
Texas DOT	≥ -6.0[4]	20	Current[4]
Oklahoma DOT	≥ -6.0	20	2020[3]
Delaware DOT	≥ -5.0	40	2020[3]

[1] Consult Asphalt Institute web site for current asphalt binder specification database (www.asphaltinstitute.org)
[2] Only applies to binders with ≥ 92°C temperature spread; BBR creep stiffness ≥ 150 MPa
[3] Applies to project tendered for bid beginning 1/1/2020
[4] Only applies to Balanced Mix Design projects. For comparison, TxDOT requirement is shown using ΔT_C computed by $\Delta T_C = T_{c,S} - T_{c,m}$.; actual requirement is $\Delta T_C \leq 6$°C using the equation $\Delta T_C = T_{c,m} - T_{c,S}$. (41)

The Utah Department of Transportation recently completed a research project aimed at the implementation of a ΔT_C plus specification. According to Anderson, et al. (42), the use of tensile failure strain and stress measured using the direct tension test as part of their purchase specification has helped mitigate thermal cracking in Utah DOT pavements. Unfortunately, the direct tension test equipment is no longer supported by its manufacturer and no other similar instruments are available at a reasonable cost. Thus, engineers at the Utah DOT implemented ΔT_C as a substitute for tensile failure strain and strength measured from the direct tension test. In their study of binders supplied to UDOT projects, Anderson, et al. noted that a majority of those binders indicated ΔT_C values between -2.0° and +2.0°C using 20 hours of PAV aging. They attribute this favorable historical range to their use of the direct tension test and its related test parameters.

Anderson, et al. observed that it would be possible to achieve a passing ΔT_C value but with an accompanying BBR stiffness of less than 100 MPa and a high BBR m-value of greater than 0.350. UDOT data showed that these types of binders typically indicate low failure strength. That is why UDOT also adopted a minimum limit for BBR creep stiffness of 150 MPa concurrent with adopting a limiting ΔT_C of -2.0°C. The UDOT research also indicated that the use of the direct tension test had resulted in modified asphalt binders with a relatively high elastic recovery. Thus, as part of the change to ΔT_C as a plus specification parameter, UDOT raised limits on elastic recovery to further ensure that there would be no decline in asphalt quality. Those limits are 80 and 85 percent for binders with temperature spreads greater than 92° and 98°, respectively.

To implement ΔT_C, the Florida DOT evaluated 57 binders that consisted of all grades commercially used in Florida (43). Their reason for embracing ΔT_C was industry concern with REOB, but also believed there would be an overall improvement in quality as a consequence of ΔT_C. Initially FDOT proposed the use of 40-hour PAV aging, along with a ΔT_C limit of -5°C and maximum REOB limit of eight percent. Because of industry concern over testing timeliness, FDOT evaluated both 20- and 40-hour PAV conditioning. Figure 36 shows a plot of ΔT_C for the 57 binders evaluated.

The top and bottom of Figure 36 shows the results of the ΔT_C evaluation using 20- and 40-hour PAV aging, respectively. For 20 hours practically all of the data points with less than eight percent REOB were greater than the -5.0°C specification limit. However, for 40 hours a significant number of binders were below the ΔT_C limit. Because FDOT did not want to disallow binders that were not using REOB, it was decided to move forward with 20 hours of PAV aging, -5.0°C for ΔT_C, and maximum REOB content of eight percent.

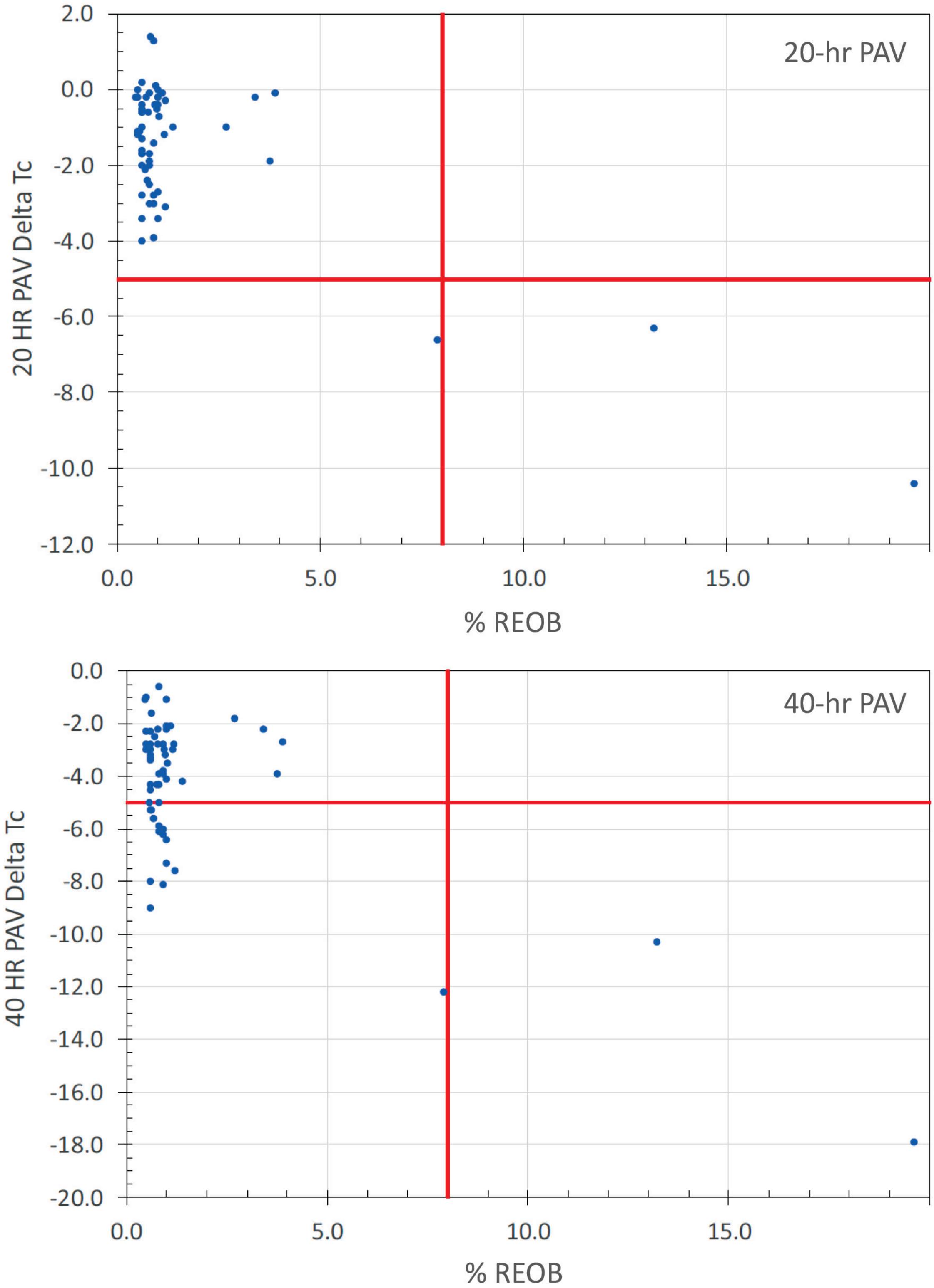


Figure 37. Florida DOT Evaluations of ΔT_C

While ten agencies (see Table 19) have or soon will adopt ΔT_c as a specification parameter, at the time this document was developed there existed two sources of information on standardized determination of ΔT_c:

- ASTM D7643, Standard Practice for Determining the Continuous Grading Temperatures and Continuous Grades for PG Graded Asphalt Binders, Section 6.3 "Calculation of DTC"
- AASHTO PP 78-17, Standard Prectice for Design Considerations When Using Reclaimed Asphalt Shingles (RAS) in Asphalt Mixtures, Section 7, "Binder Quality Requirements for Binder Embrittlement"

Recently, a draft standard was developed at the Asphalt Institute (44) and submitted to AASHTO for review, "Standard Practice for Characterizing the Relaxation Behavior of Asphalt Binders Using the Delta T_c (ΔT_c) Parameter." The draft standard describes the calculations necessary to arrive at ΔT_c and discusses factors that affect ΔT_c such as laboratory aging, RAP, RAS, and determining ΔT_c on asphalt recovered from pavements.

Considerations for Implementation of ΔT_c as a Specification Parameter

Should a purchasing agency decide to embrace ΔT_c as a specification requirement, there are several steps that should be carefully considered. Practical examples of this systematic process include the original AAPTP Project 06-01 (1), the work of Heptig (27) at the Kansas DOT, the work by Bennert, et al. (39) supporting the PANYNJ, the study by Anderson, et al. (42) at the Utah DOT, and the study described by Moseley (43) conducted by the Florida DOT. A brief summary of these steps are described as follows.

The first and most important step is to clearly identify the pavement performance challenge that ΔT_c is intended to solve. Without this clear statement of purpose, it is possible that a specification change will not mitigate the intended problem. ΔT_c is primarily aimed at asphalt pavement distress that is related to a lack of durability exhibited by asphalt binders. The most prominent distress form that ΔT_c targets is block cracking of age-embrittled asphalt pavements, which was the damage for which ΔT_c was originally developed to evaluate. However, referring to Table 15 it is believed that lower values of ΔT_c may also have at least an indirect effect on other forms of cracking.

The second step is to determine whether ΔT_c is the most favorable alternative to solve the problem identified in the first step. A good example of this is the original Anderson, et al. study (1) that was conducted for the FAA that targeted distress types (block cracking and raveling) that are known to be troublesome, yet common to airfield pavements. A following section lists some of the potential alternatives should an agency not want to consider the use of ΔT_c.

The third step is to consider what form of laboratory aging needs to be used to simulate pavement aging so that ΔT_c (or other) measurements are made on representative samples. This step, along with the fourth step, ensures that the ΔT_c specification is relevant. To accomplish this, a variety of PG binders should be laboratory aged, typically using variations on AASHTO R28. At present, 20 and 40 hours of pressure aging are most commonly used for ΔT_c determinations.

The fourth step is to secure cores from existing asphalt pavements that exhibit a range of cracking behavior caused by age-related embrittlement. The cores should represent not just a range in cracking behavior, but also a range in age from as little as two years up to as many as 15 years. The study by Bennert, et al. (39) provides a good example of this approach. ΔT_c would be measured on these materials with no additional laboratory aging. Because aging occurs most rapidly in the materials most exposed to the elements, it is essential that asphalt binder from the top ½-inch (12.5 mm) of the core should be extracted and recovered for determination of ΔT_c. This is considered a conservative approach in determining ΔT_c because it represents the worst case for aging. However, if in the first step it is decided that ΔT_c needs to target the asphalt binder contribution to load-associated damage, it might be desirable to evaluate ΔT_c for lower asphalt layers. When extracting/recovering asphalt materials, strong consideration should be given to using ASTM D7906 binder recovery procedure with toluene as this method is well-suited for the evaluation of polymer-modified asphalt binders and eliminates the age stiffening of halogenated solvents.

The fifth step is to evaluate the ΔT_C test results obtained in the previous step to arrive at the aging protocol necessary to simulate the ΔT_C values obtained in service. Laboratory aging data from step three should be employed for this purpose. The last step is to conduct a discriminant analysis to arrive at a ΔT_C value that distinguishes between good and poor pavement performance with respect to age-related embrittlement or other distress type under consideration.

Agencies and industry are in the process of embracing the concept of balanced mix design (BMD), which is based on mixture performance testing. Unfortunately, there appears to be a lack of data demonstrating practical correlation between ΔT_C and mixture tests (45). In fact, there are no performance related tests currently in use that have been demonstrated to address block cracking, which likely happens over a wide variety temperatures from low to intermediate. This is potentially problematic if both ΔT_C and BMD are to be implemented by agencies. The introduction and adoption of appropriate mixture aging protocols and a test that models block cracking in BMD specifications will be a good step toward closing this gap. Furthermore, as with ΔTc, the rate of change of mixture performance with increased aging may be as important, if not more important, than the absolute performance at any single aging level, and therefore, be a better measure to correlate between ΔT_C and mixture performance testing.

Finally, when implementing important specification changes, such as ΔT_C, the Asphalt Institute encourages agencies to work together regionally (such as in User-Producer Groups) to facilitate uniform transition for the asphalt industry.

Alternatives to ΔT_C for Addressing Block Cracking

Agencies may desire to address the phenomenon of block cracking from age-related embrittlement through specification but do not wish to do so using ΔT_C. There are several approaches that can be taken.

While the original purpose of ΔT_C was to serve as a forensic parameter to stage the application of a preventive maintenance treatments, research following the development of ΔT_C identified that it could also be employed as a physical property parameter to identify the presence non-bituminous components thought to be deleterious to pavement performance. It is probable that some agencies are employing ΔT_C for that purpose. If that is the goal, then one alternative to the use of ΔT_C would be to simply require certification that supplied binders do not contain certain undesired additives. The Asphalt Institute specification database cites some of these types of compositional provisions (http://asphaltinstitute.org/engineering/specification-databases/us-state-binder-specifications/).

Another approach would be to place compositional limits but also employ a test to verify adherence to those limits. For example, the Texas Department of Transportation has successfully implemented x-ray fluorescence testing and a related specification to preclude over use of binder additives that their engineers believe have a negative effect on asphalt durability (46).

Rowe (47) in his prepared discussion of the Anderson et al. paper (1) points out that correlations exist between ΔT_C and other rheological parameters including Glover's original parameter ($G'/(\eta'/G')$) and the more recently developed Glover-Rowe parameter ($G^*(\cos \delta)^2/ \sin \delta$) at 15°C and 0.005 radians/second. Anderson, et al. (1) also related Glover's parameter ($G'/(\eta'/G')$) and ΔTc. Thus it is possible that the use of a rheological parameter, measured on an appropriately aged sample, would be an alternative to ΔTc. It was pointed out, however, that sole use of a rheological parameter (Glover, Glover-Rowe, ΔT_C, etc.) may also need an additional parameter (e.g., a strength test) to account for low temperature behavior that is not completely addressed by rheological parameters (for example, see section on Elastomeric Polymer Modification).

Kriz (48) has identified a good correlation between ΔT_C and phase angle and suggests this is sensible because both are related to relaxation rate. Therefore, it may be possible to specify a minimum phase angle at a given binder stiffness for long-term aged residue to facilitate binders that are not prone to age-related cracking.

One of the significant challenges that has caused considerable dialog over the use of ΔT_C as a specification parameter is the need expressed by some to use binders that have been aged for 40 hours in a PAV. This lengthy aging period is viewed by some as impractical and is possibly the cause of some of the resistance to the use of ΔT_C. Thus, an alternative specification would be to retain use of ΔT_C, but adopt what would be considered a more practical aging protocol. Section 4 identified alternatives such as thin film PAV aging and a modestly higher aging temperature in concert with the normal 20 hours of PAV aging as examples of approaches that could be considered to overcome this barrier.

8.0 RECENT NATIONAL RESEARCH PROJECTS AND ΔT_C

Several national research projects conducted through the National Cooperative Highway Research Program (NCHRP) have considered the use of ΔT_C as a parameter for intermediate temperature durability. A brief summary of these projects follows.

NCHRP 09-59, "Relating Asphalt Binder Fatigue Properties to Asphalt Mixture Fatigue Performance", has been conducted under the direction of Dr. Don Christensen at Advanced Asphalt Technologies. Its principal objectives were to determine asphalt binder properties that are significant indicators of the fatigue performance of asphalt mixtures and to identify or develop a practical, implementable binder test (or tests) to measure those properties for use in a performance-related binder purchase specification such as AASHTO M 320 and M 332.

Although the focus of the research was on fatigue performance of asphalt binders, as noted in Table 15 the responses indicated a belief that ΔT_C had at least an indirect effect on fatigue cracking performance. As such, the findings from the NCHRP 09-59 research may be related to ΔT_C.

In his recent paper, Christensen et al. (49) recommended that "An effective improvement in the current binder fatigue specification can be made by replacing the current specification parameter $|G^*| \sin \delta$ with the Glover-Rowe Parameter, and establishing a maximum limit for R- value, as calculated from bending beam rheometer data." Christensen further notes that "A reasonable alternative to using the R-value is have suitable limits on ΔTc…" and "Binders with high R-values (more negative ΔT_C) therefore have three serious issues that will tend to decrease resistance to fatigue and thermal cracking: relatively low failure strain, poor healing potential and low temperature grading errors due to physical hardening."

NCHRP 09-60, "Addressing Impacts of Changes in Asphalt Binder Formulation and Manufacture on Pavement Performance through Changes in Asphalt Binder Specifications", has been conducted under the direction of Dr. Jean-Pascal Planche at Western Research Institute. Its principal objective was to propose changes to the current performance-graded asphalt binder specifications, tests, and practices to remedy gaps and shortcomings related to the premature loss of asphalt pavement durability in the form of cracking and raveling.

As noted in a recent paper by the research team (35), "Black space functions, such as the Glover-Rowe parameter which was derived as a rheological function to mimic failure strain, may enable single point DSR measurements to predict potential for surface damage within climate-based PG specifications. Numerous other functions related to Black Space, including ΔT_C, crossover parameters, TIR (Range of Intermediate Temperatures), a low temperature Glover-Rowe function, R-value in combination with a constant stiffness, and others remain under consideration as potential candidates to bolster predictive cracking protocols that can be included in future PG binder specifications." NCHRP 9-60 conclusions are expected to become available by late 2019 or early 2020.

NCHRP 9-61, "Short- and Long-Term Binder Aging Methods to Accurately Reflect Aging in Asphalt Mixtures" is being conducted under the direction of Dr. Ramon Bonaquist at Advanced Asphalt Technologies. This research targets accurate simulation of short-term (construction-related) and long-term (field) aging. The research plan recognizes that aging of different binders is not consistent and that factors such as warm mix asphalt, RAP, and chemical and polymer modifiers will affect laboratory predicted aging. The relevance of this research to the use of ΔT_C as a parameter is that the project is considering optimizing the amount of binder in the PAV to make 40 hours of aging in no more than 20 hours.

9.0 SUMMARY

This document is intended to serve as state-of-the-knowledge, engineering report to describe ΔT_c and its relevance in characterizing the behavior of asphalt materials and also as a focal point for dialog among agency users, industry producers, academia, and others with a need to have a more detailed understanding of ΔT_c. ΔT_c is an asphalt binder property that was conceived during an Asphalt Institute research project for the purpose of timing preventive maintenance treatments for airfield pavements. Since publication of the results of that study in 2011, the asphalt technical community has embraced ΔT_c as a property of great interest. Its use has been extended in numerous ways including as a forensic tool and more recently a specification parameter.

A significant challenge with the use of ΔT_c is the level of laboratory aging that must be used for ΔT_c to be useful, particularly in a specification role. Work by several researchers in the past eight years has suggested the normal long-term asphalt binder aging protocol (AASHTO R28 PAV protocol at 20 hours) may not be sufficient to allow ΔT_c to identify binders that will age excessively in a pavements without extended aging. On the other hand extended aging such as 40 hours of PAV aging is viewed by some as burdensome or impractical and therefore, may inhibit more widespread use of ΔT_c in either a forensic or specification role. Research in areas such as thinner films used in conjunction with the existing 20-hour PAV aging protocol show promise to make ΔT_c evaluations more practical.

ΔT_c has been shown in some cases to have the ability to identify reclaimed additives that are occasionally used in asphalt binders. REOB and RAS are examples of such components that are thought by most to have a deleterious effect on asphalt pavement performance when overused. RAP also has been shown to negatively affect ΔT_c. An important relationship was presented that showed the effect of ΔT_c on mixture fatigue life. It demonstrated that mixture fatigue life is influenced by the ability of an asphalt mixture to relax stresses and not just the mixture stiffness.

There is concern by some asphalt technologists whether ΔT_c fully captures the behavior of elastomeric polymer modification without an attendant property such as a strength parameter. It has been suggested that the favorable effect that polymers have on the temperature susceptibility of asphalt will negatively influence ΔT_c. It was noted, however, the Kansas DOT data set used for illustrative purposes in this report does not support this concern. This issue needs additional research to clearly understand the relationship between polymer modification and ΔT_c.

Forensic and test section experience with ΔT_c has generally validated that ΔT_c is a reasonable indicator of asphalt durability. Reinke's evaluation of MnRoad and CTH 112 materials (6) showed that ΔT_c of in-place materials generally matched cracking performance. Bennert's evaluation of airfield pavements managed by PANYNJ (39) indicated ΔT_c was a good predictor of top down fatigue cracking. An experimental project in Indiana by Huber (38) showed that ΔT_c was a reliable indicator of map cracking but not a reliable indicator of other types of cracking. Test sections in Alabama evaluated by Turner at NCAT (40) indicated a similar scenario where ΔT_c was predictive of cracking in some cases, although not others.

Among Asphalt Institute Technical Advisory Committee professionals, there is a clear consensus that ΔT_c is a worthwhile parameter to gauge asphalt durability. There is less consensus over the use of ΔT_c as a specification parameter. Among that group, it is believed that the development of block cracking is highly related to more negative values of ΔT_c. Opinions were somewhat mixed on other forms of cracking. It was pointed out, however, that ΔT_c should not be viewed as a panacea for favorable asphalt pavement cracking performance and that mixture characteristics and construction practices often play a dominant role.

SUMMARY

At the time this document was developed ten agencies in North America had or soon will adopt ΔT_C as a specification parameter. There is about an even split between agencies using 20- and 40-hour PAV aging protocols. Most (but not all) agencies have adopted a minimum limit for ΔT_C of -5.0°C. The basis for that specification value is the AAPTP research project conducted by Anderson, et al. (1) which built on the research of Glover, et al. (3), which in turn, used research by Kandhal (2) in Pennsylvania in the 1970s.

Several agencies conducted research to arrive a ΔT_C specification strategy. For example, as a consequence of its prior use of the direct tension test and validated via an internal research project, the Utah DOT had adopted a minimum limit of -2°C (PAV20) along with a minimum BBR stiffness of 150 MPa. The PANYNJ used research information gathered by Bennert, et al. (39) to implement a ΔT_C specification.

A suggested framework for implementation of ΔT_C was presented. That framework entailed clearly identifying the problem for which ΔT_C hypothesized to be a solution. A strategy for sampling and testing lab produced and in-place cores was suggested for the purpose of gaining sufficient data to ensure that a proposed ΔT_C specification would be relevant. Potential alternatives to a ΔT_C specification were presented.

Finally, several national-level research projects were mentioned. These projects are variously investigating the use of ΔT_C a parameter for intermediate temperature durability.

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11.0 FREQUENTLY ASKED QUESTIONS

1. **What is Delta T_C (ΔT_C)?**
   ΔT_C is a parameter that provides insight into the relaxation properties of an asphalt binder which can contribute to non-load related cracking or other age-related embrittlement distresses in an asphalt pavement. It is a calculated value using the results (S and m) from the BBR test. It is intended to be used on binder that has been short and long-term aged (RTFO plus PAV), but can also be used on binder recovered from asphalt pavements.

2. **What led to the development of ΔT_C as an asphalt binder property of interest; where did it come from?**
   ΔT_C was conceptualized as an potential indicator of pavement performance in a research project sponsored by the Airfield Asphalt Pavement Technology Program (AAPTP), Project 06-01, Techniques for Prevention and Remediation of Non-Load Distresses on HMA (Hot Mix Asphalt) Airport Pavements". The goal of the study was to identify simple binder and/or mixture testing which could predict imminent block cracking or raveling so the that pavement preservation strategies could be timed to delay or prevent damage to HMA pavements. The study concluded that a new binder parameter called ΔT_C had promise as a tool that could be used to analyze the durability-related properties of aged asphalt airfield pavements. Since then, ΔT_C has evolved into a parameter that can be used to predict age-related embrittlement of asphalt binders and is even being used by some agencies as a specification requirement. The use of ΔT_C in ongoing research continues.

3. **This document states that ΔT_C can measure the relaxation properties of a binder. What is relaxation and how does binder relaxation properties relate to mixture performance?**
   Asphalt exhibits a bit of viscous behavior, even at low temperatures when its behavior is mostly considered elastic. Therefore, when thermal stresses build up as a pavement gets colder, the asphalt binder will gradually experience viscous flow and the stresses will greatly reduce. This reduction of stresses over time is what is known as relaxation. In general, as a binder ages, its relaxation properties are diminished. An asphalt pavement that has a binder with good relaxation properties will be less likely to have durability-related cracking than a pavement containing a binder with poor relaxation properties.

4. **Can ΔT_C be used to predict cracking?**
   Yes. ΔT_C is thought to be principally related to block cracking. However, fatigue, edge, longitudinal, reflection, and transverse cracking may indirectly be related to ΔT_C of the binder. These distress types are typically caused by other factors, yet ΔT_C can play a supporting role in their development. *Reference: Table 15*

5. **Besides cracking, are there other types of pavement distress that can be affected by ΔT_C?**
   Although there are many factors more likely to influence a pavement's tendency to experience raveling or potholes, both of these are thought to be only indirectly affected by ΔT_C of the binder. *Reference: page 51; Table 15*

6. **Which binder testing procedure is used to determine ΔT_C?**
   BBR test (AASHTO T313) results are used to calculate ΔT_C.

7. **How is ΔT_C calculated using BBR results?**
   First the critical (or continuous) temperature for both creep stiffness (S) and creep rate (m) at the AASHTO 320 limiting values of 300 MPa and 0.300 respectively are calculated. ΔT_C is simply the mathematical difference between these two critical temperatures, expressed in degrees C to one decimal point. The equation is:

   $$\Delta T_C = T_{c,s} - T_{c},m.$$

   For example, when the critical temperatures of T_c,S equals -19.3°C and T_c,m equals -22.7°C, ΔT_C is calculated as follows:

   $$\Delta T_C = -19.3° - (-22.7°)$$
   $$\Delta T_C = +3.4°C$$

8. **Are there any AASHTO or ASTM standards that discuss ΔT_C?**
   Yes, ASTM D7643 Section 6.3 "Calculation of DT_C", and AASHTO PP 78-17 Section 7 "Binder Quality Requirements for Binder Embrittlement", both discuss how to calculate ΔT_C.

QUESTIONS

9. **What does the binder's ΔT_C value tell me?**
   The sign on ΔT_C, either positive or negative, indicates whether the binder's PG low-temperature grade is governed by its creep stiffness S (+ΔT_C) or governed by its creep rate m (-ΔT_C). When ΔT_C is positive, the binder is referred to as being "S-controlled" (failing the S criterion at a warmer temperature than the m criterion), while a negative ΔT_C value indicates the binder is "m-controlled" (fails m criterion at a warmer temperature than the S criterion). The absolute magnitude of the ΔT_C value indicates the degree to which the binder is S- or m-controlled.
10. **Can I use ΔT_C to evaluate a virgin asphalt binder?**
    Yes.
11. **Can I use ΔT_C to evaluate a virgin binder that may contain PPA, REOB/VTAA or other asphalt additive?**
    Yes.
12. **Can I use ΔT_C to evaluate a recovered binder that contains aged RAP, RAS or RAP/RAS binder?**
    Yes. It is advisable that the quality of the total binder (i.e., virgin binder plus RAP and/or RAS) be evaluated for ΔT_C to ensure that the total binder does not have a detrimental effect on long-term durability. However, it should be noted that the recovery process will completely blend the virgin, RAP and/or RAS binder which does not accurately represent the incomplete field blending that likely takes place on plant-produced materials. It is strongly advised to conduct extractions using toluene as a solvent and then recover the binder using a rotary evaporator procedure as specified in ASTM D7906.
13. **As a binder ages, how does it affect ΔT_C?**
    As a binder ages, either in the lab or the field, ΔT_C will decrease.
14. **How do RAP and RAS binders affect ΔT_C?**
    For all practical purposes, RAP and RAS binders will always cause ΔT_C to decrease, which is an unfavorable trend.
15. **What are some things to consider before implementing ΔT_C in a purchase specification?**
    Before a purchasing agency implements ΔT_C as a specification requirement, there are a number of steps that should first be carefully considered. A brief summary of these steps is described in the "Considerations for Implementation of ΔT_C as a Specification Parameter" section of this document. When implementing important specification changes, AI encourages agencies to work together regionally (such as in User-Producer Groups) to facilitate uniform transition for the asphalt industry.
16. **What is the precision of typical ΔT_C values?**
    Precision estimates for ΔT_C are 0.8°C for a single operator and 1.8°C for multiple laboratories.
17. **Why use 40 instead of 20 hours of PAV aging?**
    Some believe that 40 hours of aging better reflects the aging that occurs in real pavements that have been in service for longer periods of time. Using 40 hours, the agency has less risk of accepting material that may be subject to premature binder embrittlement than if it used 20 hours. However, at 40 hours binder suppliers have more risk of having acceptable binder rejected. It is important to balance the buyer's and seller's risks.
18. **Do I have to modify test equipment or test procedure to determine ΔT_C?**
    No, the equipment requirements and test procedure outlined in AASHTO T313 are not changed. However, S and m values need to be determined at a minimum of two different temperatures in order to interpolate the exact critical temperatures needed for ΔT_C. Older PAV models have controls that only allow a 20-hour aging cycle. To age a sample for 40 hours it is necessary to stack 20-hour cycles. Some PAV's can be retrofitted with new controls which will allow for 40 hours of continuous aging, which some laboratory personnel find more convenient.
19. **Can I use ΔT_C to evaluate an asphalt binder that contains a polymer modifier?**
    Yes. However, there remains dialog in the asphalt technology community concerning the validity of characterizing polymer modified binders using ΔT_C. Certain features of polymer modification could possibly have a worsening effect on ΔT_C and therefore make it appear as if polymer modified binders will exhibit diminished durability. *Reference: Table 11*
20. **Are agencies using ΔT_C in their specifications?**
    Yes. At the time this document was developed, ten agencies in North America had or soon will adopt ΔT_C as a specification parameter in some manner. There is about an even split between agencies using 20- and 40-hour PAV aging protocols. Most (but not all) agencies have adopted a minimum limit for delta Tc of -5.0°C. The basis for that specification value is the AAPTP research project mentioned in FAQ #2.

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