Benchmarking Delta Tc (ΔTc) for Wisconsin Materials

Thomas Bennert, Ph.D.
Geoff Rowe, Ph.D., P.E.
Christopher Ericson
Jake Tulanowski

Center for Advanced Infrastructure and Transportation (CAIT)
Rutgers University
100 Brett Road
Piscataway, NJ 08854

Abatech, Inc.
PO Box 356
Blooming Glen, PA 18911

WisDOT ID no. 0092-23-01

May 2025

Research & Library Unit

Wisconsin Highway Research Program

WISCONSIN DOT
PUTTING RESEARCH TO WORK

DISCLAIMER

This research was funded through the Wisconsin Highway Research Program by the Wisconsin Department of Transportation and the Federal Highway Administration under Project 0092-23-01. The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views of the Wisconsin Department of Transportation or the Federal Highway Administration at the time of publication.

This document is disseminated under the sponsorship of the Department of Transportation in the interest of information exchange. The United States Government assumes no liability for its contents or use thereof. This report does not constitute a standard, specification or regulation.

The United States Government does not endorse products or manufacturers. Trade and manufacturers' names appear in this report only because they are considered essential to the object of the document.

TECHNICAL DOCUMENTATION PAGE

<table>
<tr><td colspan="2">1. Report No.
WHRP -0092-23-01</td><td colspan="2">2. Government Accession No.</td><td colspan="2">3. Recipient's Catalog No.</td></tr>
<tr><td colspan="4" rowspan="2">4. Title and Subtitle
Benchmarking Delta Tc (ΔTc) for Wisconsin Materials</td><td colspan="2">5. Report Date
May 2025</td></tr>
<tr><td colspan="2">6. Performing Organization Code</td></tr>
<tr><td colspan="4">7. Author(s)
Thomas A. Bennert, Ph.D., Geoff Rowe, Ph.D., P.E., Christopher Ericson, and Jake Tulanowski</td><td colspan="2">8. Performing Organization Report No.
If applicable, enter any/all unique numbers assigned to the performing organization.</td></tr>
<tr><td colspan="4" rowspan="2">9. Performing Organization Name and Address
Center for Advanced Infrastructure and Transportation (CAIT), Rutgers University</td><td colspan="2">10. Work Unit No.</td></tr>
<tr><td colspan="2">11. Contract or Grant No.
WHRP 0092-23-01</td></tr>
<tr><td colspan="4" rowspan="2">12. Sponsoring Agency Name and Address
Wisconsin Department of Transportation
Research & Library Unit
4822 Madison Yards Way Room 911
Madison, WI 53705</td><td colspan="2">13. Type of Report and Period Covered
Final Report
October 2023 to May 2025</td></tr>
<tr><td colspan="2">14. Sponsoring Agency Code</td></tr>
<tr><td colspan="6">15. Supplementary Notes</td></tr>
<tr><td colspan="6">16. Abstract
The purpose of the study was to evaluate the ΔTc parameter for potential inclusion in the WisDOT asphalt binder specifications for cracking potential. The research encompassed a literature review and extensive laboratory program consisting of both asphalt binder and mixture testing. Using the information and data collected, it was determined that ΔTc should not be selected as an asphalt binder cracking parameter due to variability issues, as well as its difficulty to properly rank polymer modified asphalt binder performance. Instead, the study provided two parameters that can be determined at a single test temperature and loading frequency; Glover-Rowe Parameter at 15C (GRP_{15C}) and R-value at 15C. Criteria for the parameters were calibrated against an IDEAL-CT Index value of 30 after WisDOT long-term mixture aging. The parameters were found to be sensitive to asphalt binder supplier, different PG grades, inclusion of recycled asphalt binder and aging. Currently approved WisDOT asphalt binders were found to meet the requirements of the procedure after 20 hour PAV conditioning, but many had issues meeting the requirements after 40 hour PAV conditioning. When blended with typical RAP binder from Wisconsin, the test parameters would limit most asphalt binders to 35% asphalt binder replacement before failing the proposed criteria. Lastly, the criteria was compared to recovered asphalt binder from the WisDOT BMD Implementation test sections and could identify Test Section #4, which was the only section designed and produced to have an IDEAL-CT Index less than 30.</td></tr>
<tr><td colspan="3">17. Key Words
Asphalt, asphalt binder, fatigue cracking, specification, Glover-Rowe Parameter (GRP), IDEAL-CT Index, rheological, Delta Tc</td><td colspan="3">18. Distribution Statement
No restrictions. This document is available through the National Technical Information Service.
5285 Port Royal Road
Springfield, VA 22161</td></tr>
<tr><td colspan="2">19. Security Classif. (of this report)
Unclassified</td><td colspan="2">20. Security Classif. (of this page)
Unclassified</td><td>21. No. of Pages
137</td><td>22. Price</td></tr>
</table>

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

EXECUTIVE SUMMARY

The purpose of this report is to document the results of the WHRP research project, *Benchmarking Delta Tc (ΔTc) for Wisconsin Materials*, as performed over the past several years by the Center for Advanced Infrastructure and Transportation (CAIT) at Rutgers University. The purpose of this research is described in the RFP:

> *The objective of this research project was to evaluate the use of ΔTc parameter to help predict the no-load related cracking susceptibility of Wisconsin asphalt mixtures using past research to standardize, validate, and recommend an aging procedure prior to measurement of ΔTc. The results of the study should be compared to thresholds recommended by past researchers to determine the risk of early non-load related cracking in Wisconsin, while recommending a means for implementation as the preferred performance measure for cracking susceptibility in WisDOT specifications.*

Early in the literature review, as well as during the laboratory evaluation, it was evident that ΔTc may not be the best parameter for WisDOT implementation. Issues with testing variability and poor ability to rank polymer modified binders made the ΔTc parameter a less than desirable candidate for future specifications. Instead, the research concluded that the combined use of the Glover-Rowe Parameter at 15°C (GRP_{15C}) and the R-value at 15°C were currently the best option for Wisconsin to implement an asphalt binder testing protocol that was sensitive to mixture cracking performance (in this case the IDEAL-CT Index) and simple to measure and implement into current specifications. In fact, the proposed methodology uses a similar approach to the intermediate temperature PG grade but only requires a single test temperature and loading frequency. The GRP_{15C} parameter was also found to be highly correlated to the low temperature PG grade as determined by the BBR m-value. A finding that could be used to quicken the low temperature assessment of asphalt binders in Wisconsin. The GRP_{15C} and R-value at 15°C were found to be able to differentiate the same PG grades from different asphalt suppliers, while showing current suppliers' asphalt binders would meet the proposed criteria after 20 hour PAV conditioning with most having difficulties meeting the criteria after 40 hour PAV conditioning. The inclusion of recycled binder replacement (RBR) greater than 35% would also result in most asphalt binder grades and sources failing the proposed criteria. Lastly, the methodology was briefly validated using the IDEAL-CT Index performance testing of the WisDOT BMD Implementation Test Sections and could identify the single test section purposely designed and produced to have an IDEAL-CT Index less than 30. The proposed methodology is an improvement over the current procedure used by WisDOT, more accurate and repeatable than ΔTc, and easily implementable, greatly improving and strengthening the WisDOT materials specifications against asphalt binders prone to cracking and durability issues.

TABLE OF CONTENTS

FIGURES

TABLES

BACKGROUND

The Wisconsin Department of Transportation (WisDOT) is interested in looking at the potential use of the Delta Tc (ΔTc) parameter as a means of benchmarking the asphalt binders currently supplied to the state and is encouraged by previous work under WHRP project 0092-19-04, *Recycled Asphalt Binder Study* (Rodezno et al., 2021) that successfully used ΔTc to evaluate the cracking susceptibility of high recycled mixtures. This study aims at evaluating whether or not ΔTc is appropriate for use with conventional asphalt binders supplied to WisDOT, and if so, what type of criteria should exist to differentiate between good and poor performing asphalt binders. Currently, WisDOT relies on the intermediate temperature PG grade as a means of capturing poor intermediate temperature cracking performance, even though it is well known intermediate temperature PG grade has its limitations.

To help address these needs, a research study was initiated with the following objectives;

1. Evaluate the use of the ΔTc parameter to help predict the non-load related cracking susceptibility of Wisconsin asphalt mixtures;
2. Use past research to standardize, validate, and recommend an aging procedure prior to the measurement of ΔTc;
3. Compare the benchmarking study results against ΔTc thresholds recommended by past researchers to determine the risk of early non-load related cracking in Wisconsin; and
4. Recommend a plan for implementing ΔTc as a preferred performance measure for cracking susceptibility into WisDOT specifications.

As part of the study, a Literature Review was conducted to collect the most updated information pertaining to ΔTc and how it relates to asphalt mixture performance.

INITIAL DEVELOPMENT OF ΔTc

The formal development of the ΔTc approach was proposed by Blankenship et al. (2010) under the Airfield Asphalt Pavement Technology Program (AAPTP) Project, 06-01, *A Laboratory and Field Investigation to Develop Test Procedures for Predicting Non-Load Associated Cracking of Airfield HMA Pavements*. The authors utilized three different asphalt binders with historical performance and chemical characteristics. The authors aged the asphalt binders at four different conditioning times in the pressure aging vessel (PAV); 0, 20, 40, and 80 hours. After each of the conditioning times, the asphalt binders were evaluated under various test methods, including the bending beam rheometer (BBR). The researchers noted;

> "… as PAV aging time increases, the critical temperature for S(60) and m(60) both increase. However, the critical temperature for m(60) increases at a much more rapid rate indicating a loss of relaxation properties in the asphalt binder as aging increases. To quantify this change, the difference between Tc,m(60) and Tc,S(60) was determined."

The test data the authors presented in the report to illustrate this effect is shown as Figures 1 and 2. Blankenship et al. (2010) originally defined the ΔTc parameter as the difference between the Tc,m(60) and Tc,S(60), as shown in Figure 2. However, it was eventually decided to reverse the difference to show that more negative values result in worse relaxation and ductility performance.

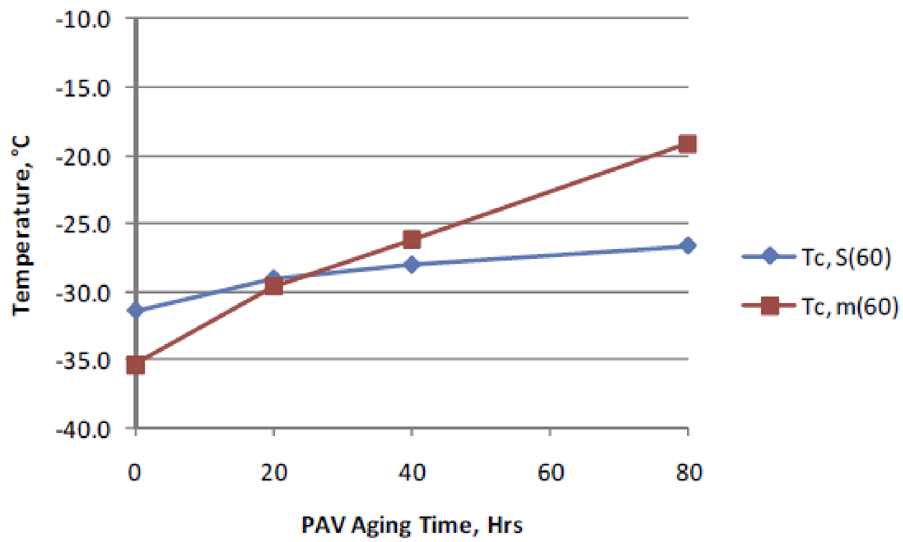


Figure 1 – Effect of PAV Aging Time on Tc – Western Canadian Crude (After Blankenship et al., 2010)

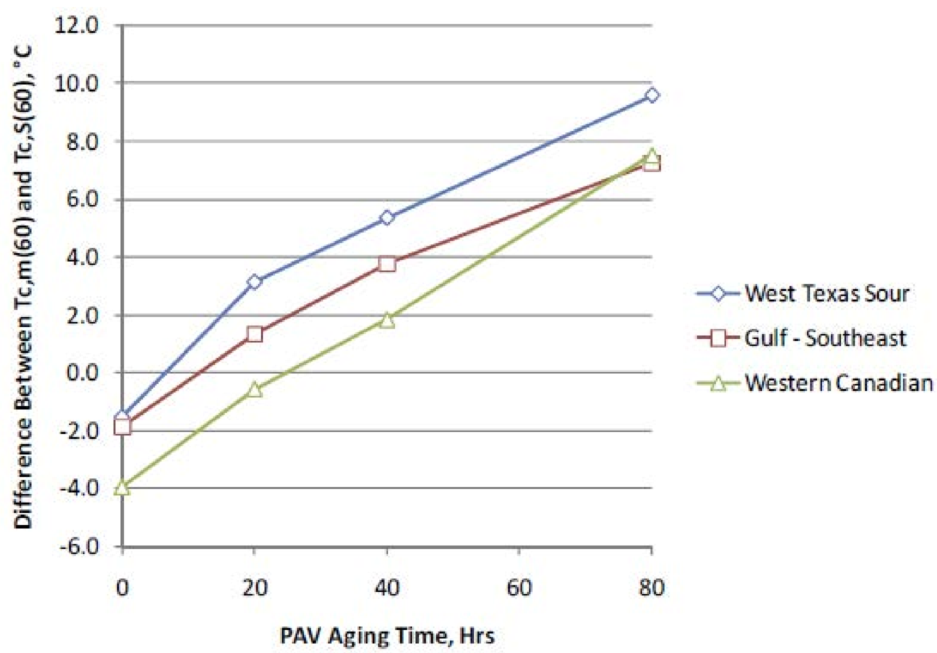


Figure 2 – Effect of PAV Aging Time on the Difference Between Tc,m(60) and Tc,S(60) (After Blankenship et al., 2010)

The researchers went on to demonstrate that ΔTc is related to DSR parameter G'/(η'/G') developed by Glover et al., (2005), which was shown to be directly related to the ductility of asphalt binders (Figures 3 and 4). Using the relationship shown in Figure 3 and the criteria originally proposed by Glover et al., (2005), Blankenship et al., (2010) recommended a ΔTc value of 2.5°C and 5.0°C for the "Warning" when non-load associated cracking is a potential and "Limit" where non-load associated cracking will occur. As noted earlier, the sign of these values was eventually changed to be negative based on how the ΔTc was recommended to be calculated.

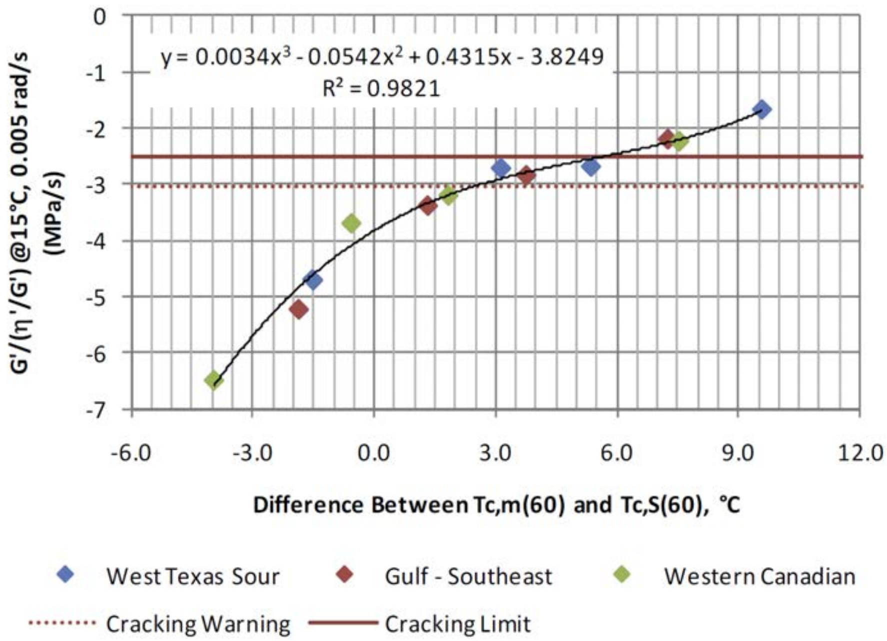


Figure 3 – Relationship Between ΔTc and Glover DSR Parameter (After Blankenship et al., 2010)

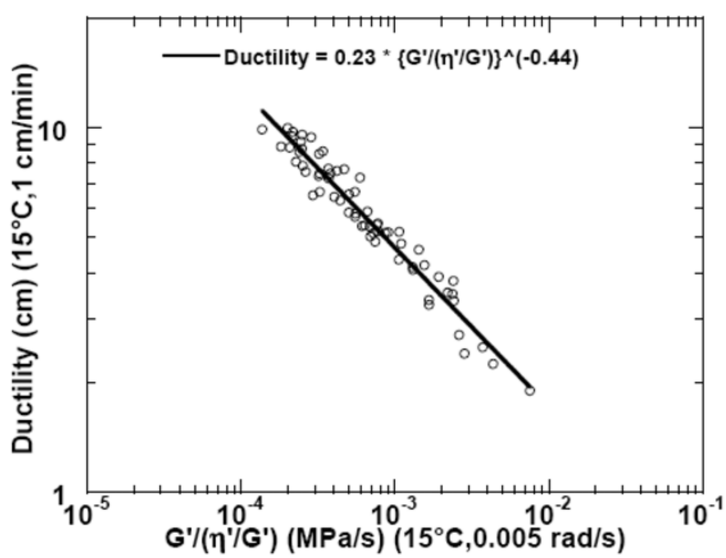


Figure 4 – Relationship Between Ductility and DSR Parameter (Glover et al., 2005)

The authors did note that there were several limitations to the proposed ΔTc procedure and protocols;

1. Only three asphalt binders were originally evaluated in the laboratory study;
2. No modified asphalt binders were included; and
3. Only four airport asphalt pavements were studied in the field validation portion of the study from three airports and two sets of environmental conditions.

Although there were obvious limitations to the study, the conceptual theory of ΔTc is valid. As asphalt binders age, the stiffness of the asphalt binder increases. This can be observed during intermediate PG temperature testing where the G* will increase and the phase angle (δ) will decrease with aging. As aging continues, the relaxation properties of the asphalt binder also decrease and can be directly measured by the BBR m-value.

Appropriate Calculation for ΔTc

As mentioned earlier, the critical cracking low temperatures (Stiffness, S and m-value, m) from the bending beam rheometer (BBR) are used to calculate ΔTc. The critical cracking temperatures are determined by interpolating between the passing and failing temperatures using the following equations;

$$T_{C,S} = T_1 + \left(\frac{(T_1 - T_2)\times(\log 300 - \log S_1)}{\log S_1 - \log S_2}\right) - 10 \qquad (1)$$

$$T_{C,m} = T_1 + \left(\frac{(T_1 - T_2)\times(0.300 - m_1)}{m_1 - m_2}\right) - 10 \qquad (2)$$

Where,

T_1 = temperature at which S and m passes, ^oC
T_2 = temperature at which S and m fails, ^oC
m_1 = creep rate at 60 seconds at T_1
m_2 – creep rate at 60 seconds at T_2
S_1= creep stiffness at 60 seconds at T_1
S_2 = creep stiffness at 60 seconds at T_2

The difference between $T_{C,S}$ and $T_{C,m}$ is then calculated to determine ΔTc, as shown in Equation 3.

$$\Delta T_C = T_{C,S} - T_{C,m} \qquad (3)$$

In some cases, the difference between $T_{C,S}$ and $T_{C,m}$ is large, resulting in different ranges of passing and failing temperatures for the Stiffness and m-value. Anderson (2017) recommends that under these circumstances, a passing and failing temperature must be measured for each parameter (Stiffness and m-value) to avoid extrapolation and possible errors.

Table 1 shows an example of the potential error when extrapolating BBR data. The testing for this asphalt binder was required to be conducted at three test temperatures. The passing and failing temperatures for the Stiffness (S) and the m-value were -18 and -24°C and -12 and -18°C, respectively. The correct Stiffness data is highlighted in the blue box and the correct m-value data is highlighted with the red box. When properly tested, the ΔTc is determined to be -4.7°C. However, when the $T_{C,S}$ data was extrapolated, the ΔTc was determined to be -4.0°C. Meanwhile, when the $T_{C,m}$ was extrapolated, the ΔTc was determined to be -4.8°C. Ultimately, the magnitude of the error will be dependent on how much extrapolation occurs.

Table 1 – Example of Associated Errors in Extrapolation of BBR Data

	Temp (°C)	S (MPa)	m-value	Tc,s	Tc,m	Tc,s (Extrap)	Tc,m (Extrap)
T3	-12	108	0.324		-26.0	-30	
T2	-18	232	0.289	-30.7			-25.9
T1	-24	416	0.256				
			ΔTc =	**-4.7**		-4.0	-4.8

Repeatability of the ΔTc Measurement

Asphalt Institute Work

Two separate round robin studies were conducted by Asphalt Institute researchers. Under the direction of Michael Anderson of the Asphalt Institute, two neat asphalt binder grades (PG58-28, PG64-22) supplied by the Illinois DOT were evaluated by 18 different asphalt binder laboratories with the ΔTc value measured after 40 hours conditioning in the PAV. The second study, under the direction of Greg Harder from the Asphalt Institute, utilized over 20 different laboratories associated with the Northeast Asphalt User Producer Group (NEAUPG) to measure the ΔTc value of a neat (PG58-28) and polymer modified binder (PG76-22). The NEAUPG study looked at a 20 hour PAV conditioning, as well as two different 40 hour PAV conditioning methods; 1) 20 hours of PAV conditioning with complete depressurizing, repressurizing, 20 hours of PAV conditioning and 2) 40 hours of PAV conditioning with no interruptions. In both studies, laboratories were requested to follow current AASHTO specifications for conditioning and testing.

Table 2 shows the results of both studies with the resultant 2ds statement (allowable range of two results).

- On average, the 2ds for the 20 hour PAV conditioned binder was 1.35°C, while the 2ds for the 40 hour PAV conditioned binder was 2.47°C.
    - This essentially means that if WisDOT specifies a value of -5°C for a 40 hour PAV ΔTc, replicate testing by a second lab could range between -2.53°C and -7.47°C and be statistically equal to the -5°C criteria. This range in ΔTc can be quite concerning

when attempting to enforce a specification. The data also suggests that the way the asphalt binder is conditioned for 40 hours can also induce repeatability issues.

- In the NEAUPG study, when conditioning the asphalt binder for two consecutive 20 hour periods, the average 2ds was 3.1^{o}C. As opposed to running the PAV for 40 hour continuously, which resulted in a 2ds of 1.6^{o}C.
  - It was not clear whether the variability in the ΔTc results is due to extrapolation of BBR results for calculation purposes, laboratory conditioning, testing error or a combination of all the above.

Table 2 – ΔTc Repeatability Studies

Study	Binder Type	ΔTc (^{o}C)		Aging Method
		Average	2ds [1]	
Illinois DOT [2]	PG58-28	-3.9	2.8	40 Hr PAV [3]
	PG64-22	-6.5	2.6	
NEAUPG [4]	PG58-28	0.4	1.6	20 Hr PAV
		-3.0	3.3	40 Hr PAV (20 Hr + 20 Hr)
		-2.5	2.0	40 Hr PAV
	PG76-22 (PG64E-22)	-0.6	1.1	20 Hr PAV
		-3.5	2.9	40 Hr PAV (20 Hr + 20 Hr)
		-3.2	1.2	40 Hr PAV

[1] 2ds is the allowable range of two results for multiple labs
[2] Study conducted by Asphalt Institute under Mike Anderson (2021)
[3] Not known how 40 Hrs were achieved in PAV
[4] Study conducted by Greg Harder (2019) under Northeast Asphalt User Producer Group (NEAUPG) Asphalt Binder Technical Group

Recommended Changes to the Precision Estimates of T313 (Azari and Akisetty, 2024)

Pavement Systems, LLC, in conjunction with the Maryland State Highway Administration (MDSHA), conducted an interlaboratory evaluation of AASHTO T313 with seven different AASHTO accredited laboratories. The main emphasis of the study was to address the precision of the passing and failing temperatures in the bending beam rheometer, but the collected data also allowed for evaluating the repeatability of the ΔTc measurement. The asphalt binders utilized in the study consisted of the following neat asphalt binders; PG64-22, PG58-28, and PG52-34; and the following polymer modified asphalt binders; PG64E-22, PG64-28, and PG58-34. Each laboratory was responsible for RTFO and 20 hour PAV conditioning in accordance with AASHTO specifications. Four BBR beams were prepared for each binder – two to be tested at the Passing low temperature grade and two to be tested at the Failing low temperature grade. The BBR stiffness and m-value for each beam at each temperature was collected and used for analysis. A total of 168 tests were performed overall in the study.

Results of the study concluded that the 2ds (acceptable range of two test results) for the Single Operator precision was 0.8^{o}C. However, the multiple laboratory 2ds was three times that, at a value of 2.4^{o}C.

ΔTc vs ASPHALT MATERIAL PERFORMANCE

The Asphalt Institute collected a significant amount of information on the ΔTc parameter and how it relates to laboratory and field performance of asphalt materials (Asphalt Institute, 2019). The document contains information from technical presentations, reports and journal articles up to the end of 2018. Some of the more pertinent material will be covered within the following sections, however, most of the information presented is based on material after the Asphalt Institute document was developed and solely relative to the main premise of this study.

Laboratory Performance

Laboratory Performance of Re-Refined Engine Oil Bottoms (REOB) Modified Asphalt (Bennert et al., 2016)

Bennert et al. (2016) evaluated the impact of REOB modification of asphalt binders utilizing both asphalt binder and mixture performance testing in the laboratory. The ΔTc parameter was found to be both sensitive to level of laboratory conditioning (20 vs 40 hr PAV) and dosage rate of the REOB. The relationship between the Overlay Tester cycles to failure and the ΔTc value of the asphalt binder is shown as Figure 5. The figure suggests that a positive ΔTc provides better fatigue cracking performance as opposed to a negative ΔTc based on the materials evaluated in the study. However, no distinct relationship was noted. This may be because the asphalt binders were not recovered from the tested mixtures, solely conditioned and tested as would be the case within a purchase specification environment.

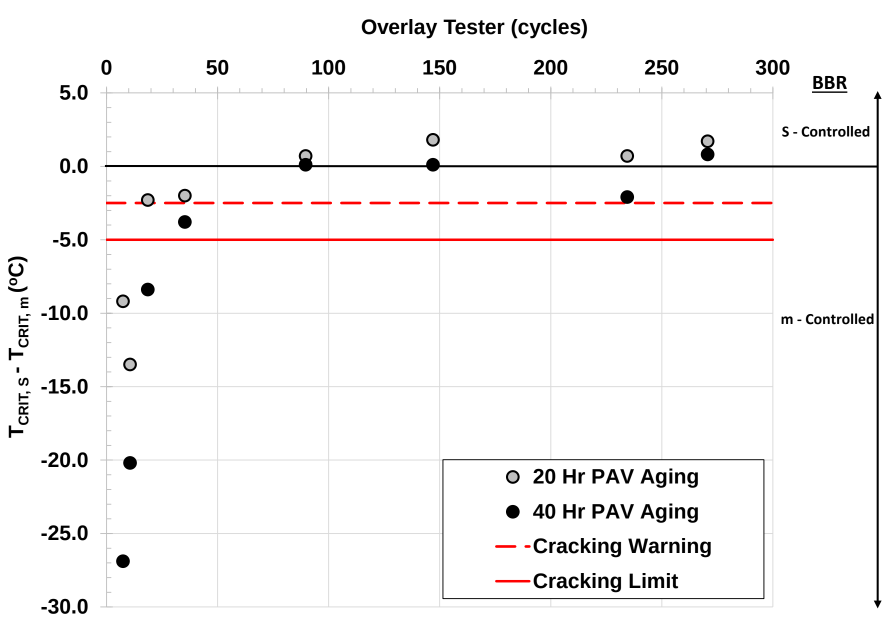
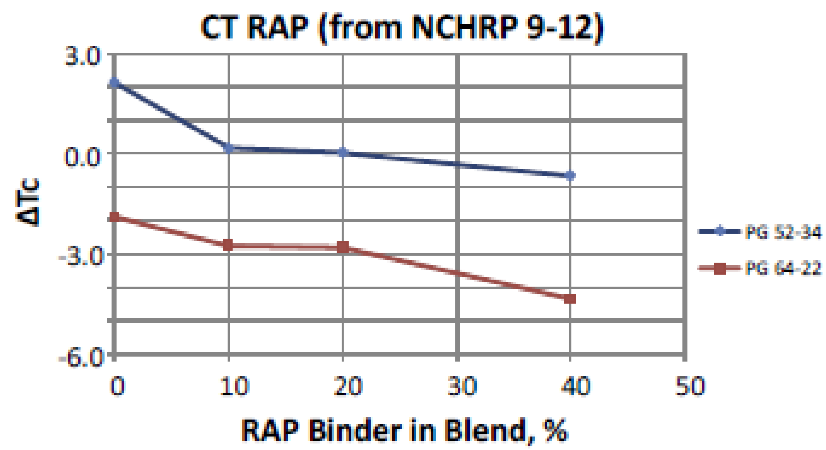


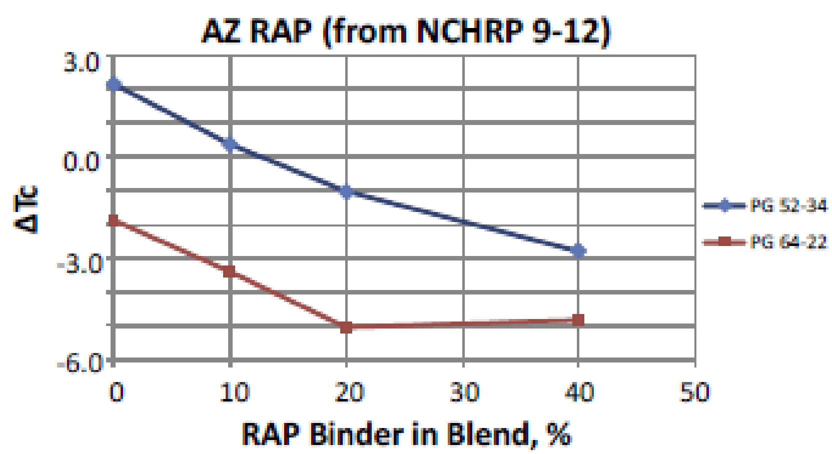
Figure 5 – ΔTc Parameter Compared to Overlay Tester Cycles to Failure for REOB Modified Asphalt (Bennert et al., 2016)

Δ Tc: Concept and Use (Anderson, 2017)

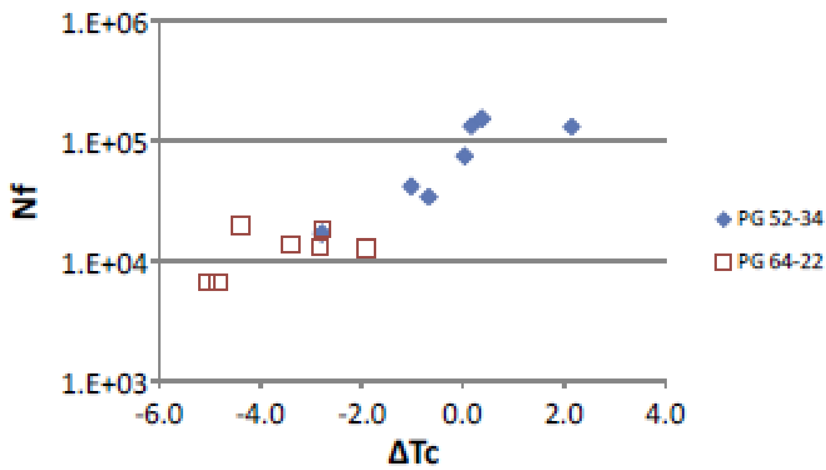
Using test data developed during NCHRP 9-12, Anderson (2017) showed the sensitivity of the ΔTc parameter with recycled asphalt pavement (RAP) content. The study looked at two RAP sources blended with two different asphalt binder grades. The test data clearly shows that as RAP content increased, the ΔTc value became more negative. Anderson (2017) went on to also show that there appeared to be a relationship between the number of cycles to crack initiation from the flexural beam fatigue and the ΔTc parameter itself (Figure 6).



(a)



(b)



(c)

Figure 6 – ΔTc Parameter Sensitivity to Aging and Flexural Fatigue Crack Initiation (After Anderson, 2017)

<u>Universal and Practical Approach to Evaluate Asphalt Binder Resistance to Thermally-Induced Surface Damage (Elwardany et al., 2020)</u>

Under NCHRP Project 9-60, the researchers evaluated a wide variety of asphalt binders of different chemistries, unmodified and modified using a wide array of modifiers and dosage rates. The authors concluded that ΔTc alone, although capable of tracking relative changes in the relaxation of the asphalt binder due to aging, is in fact a rheologically, low strain parameter, and without inducing actual damage to the asphalt binder sample, failure properties and strain tolerance may not be accurately predicted for complex and modified asphalt binders. One issue in particular the researchers noted was the potential problem with SBS modified asphalt binders. Figure 7 is from the authors' paper showing that as SBS content in a base binder increased, the ΔTc value decreased and became more negative. Originally starting with a ΔTc value of 1.5^{o}C at 0% SBS, the ΔTc parameter at 5% SBS decreased to -4.2^{o}C. The change in ΔTc was not due to the SBS "aging" the base binder, but due to the increase in elastic stiffness, making the binder less prone deflection/relaxation in the BBR test.

These findings resulted in the researchers pushing forward the concept of including fracture toughness of the asphalt binder using the Asphalt Binder Cracking Device (ABCD) in combination with the ΔTc parameter. The ΔTc parameter is essentially used to evaluate the relative age hardening of the asphalt binder while the ABCD based parameter, ΔTf, addresses the fracture toughness of the asphalt binder to ensure polymer modified binders that have historically shown good field performance due to strong fracture toughness properties but may have marginal ΔTc values, are not classified as poor asphalt binders. Figure 8 is a performance space developed for asphalt binders after 20 hours conditioning in the pressure aging vessel (PAV).

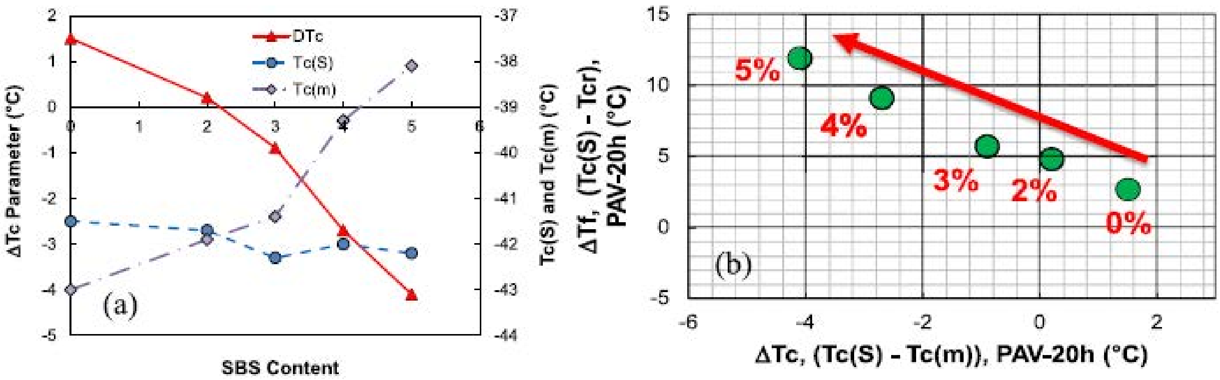


Figure 7 – Impact of SBS Polymer Modification on ΔTc for a Base Binder with Varying Polymer Dosage Rates (After Elwardany et al., 2020)

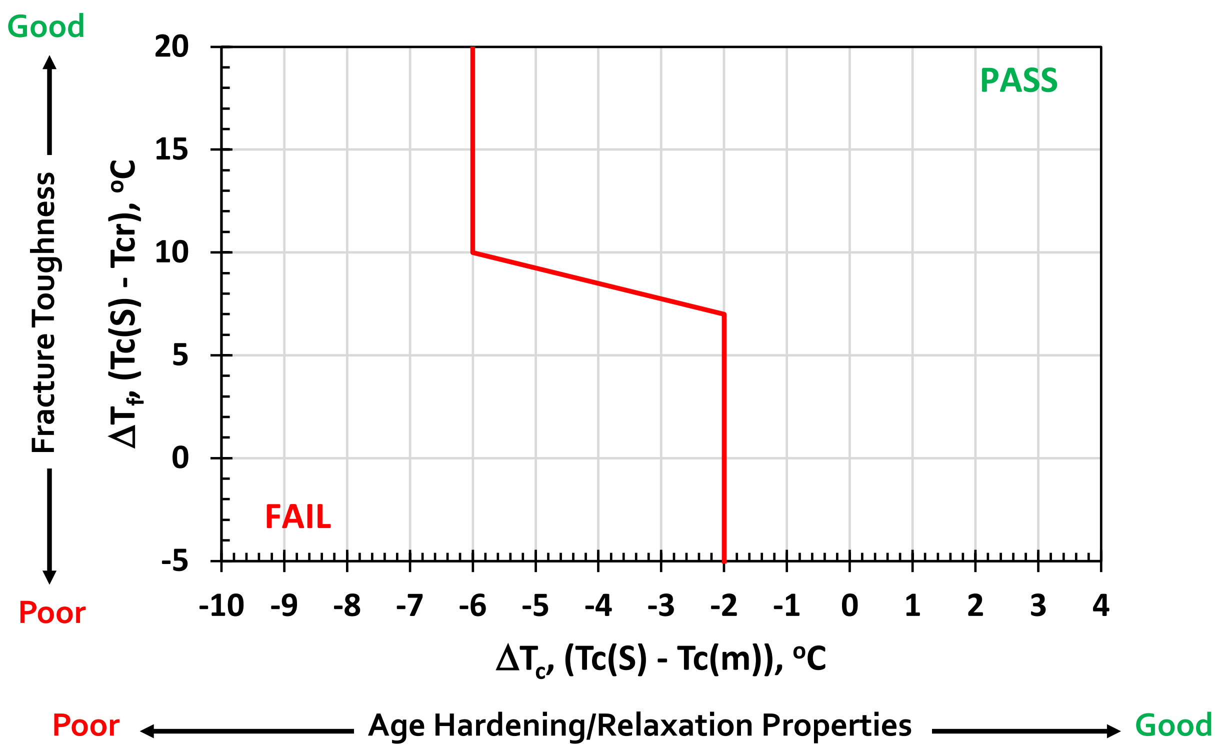


Figure 8 – Asphalt Binder Performance Space that Includes Fracture Toughness and Relaxation from NCHRP Project 9-60

<u>Recycled Asphalt Binder Study (Rodezno et al., 2021)</u>

Rodezno et al (2021) evaluated how the quantity and quality of recycled asphalt materials (RAM) affect the performance of the resultant asphalt binder blends. The researchers utilized different RAP and RAS sources, recovered and blended into base asphalt binders of different grades at different percentages. Overall, the study showed that the ΔTc parameter was sensitive to recycled asphalt content, recycled asphalt type (RAP vs RAS), and recycling agent (RA) type. The following figures were created for the literature review from the data in the final report to illustrate this conclusion.

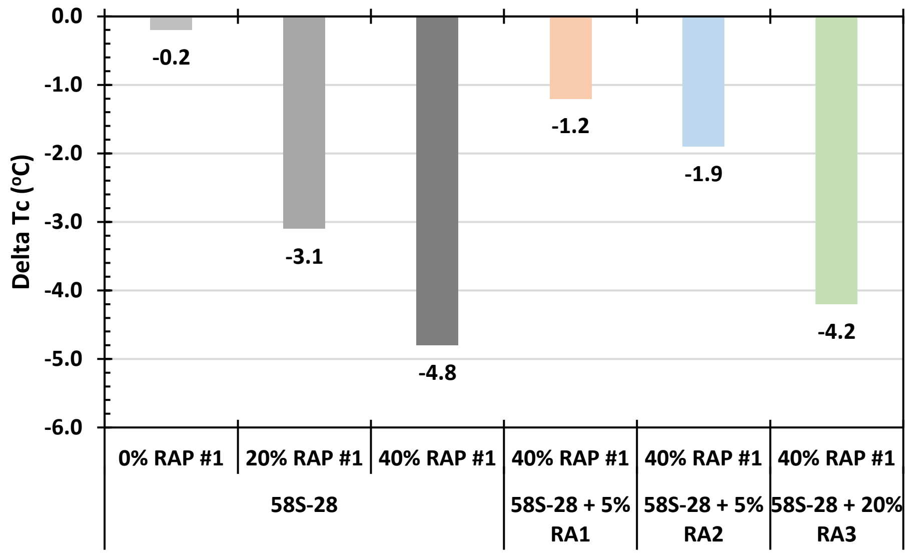


Figure 9 – ΔTc Value from Different RAP Percentages (RAP Source #1) and Recycling Agents (RA1 to RA3)

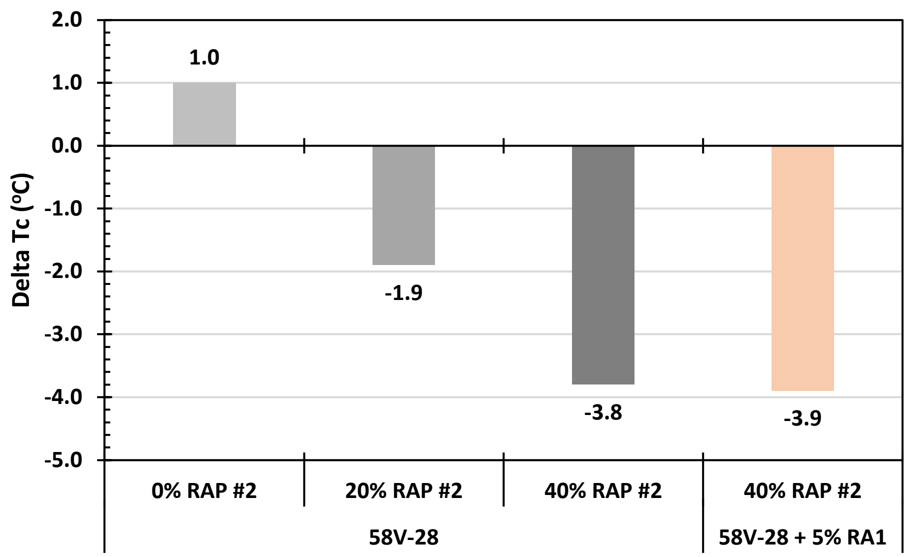


Figure 10 – ΔTc Value from Different RAP Source (Source #2)

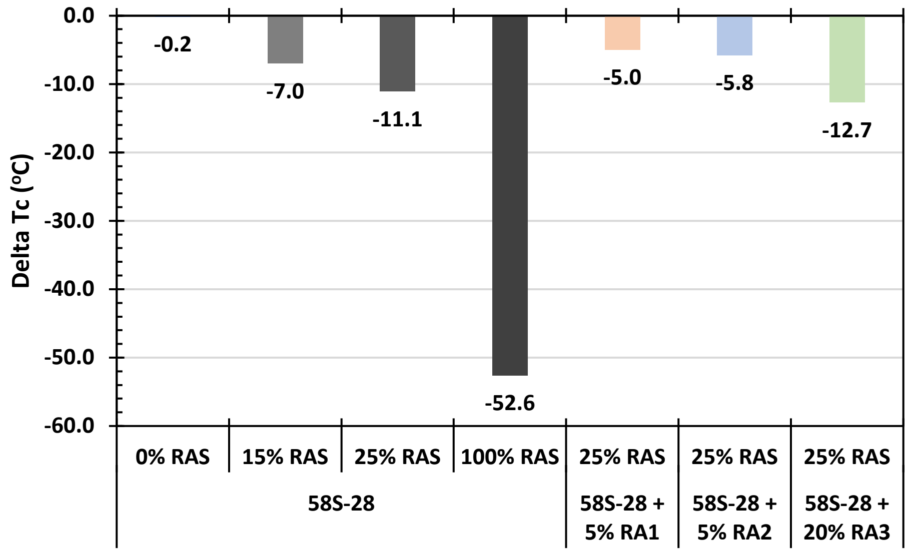


Figure 11 – ΔTc Value from Different RAS Percentages and Recycling Agents

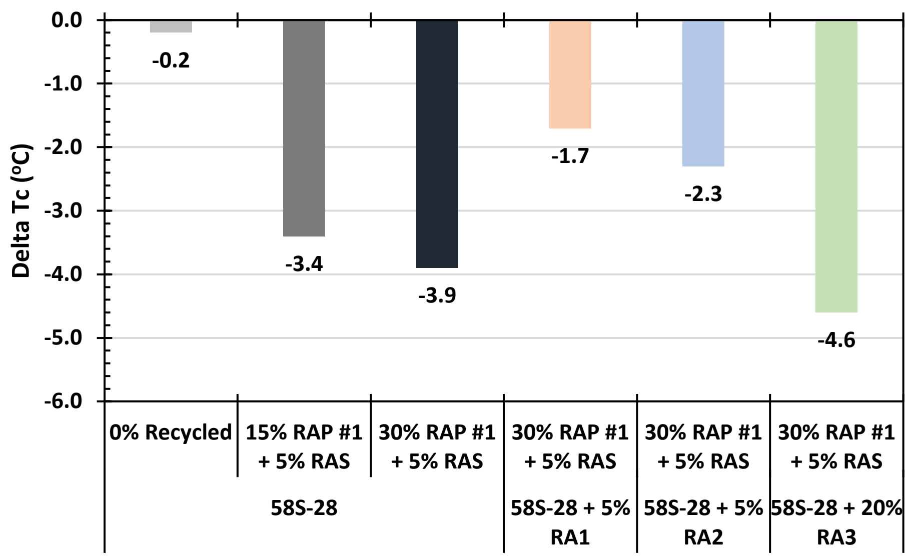


Figure 12 – ΔTc Value from Different Blended Percentages of RAP and RAS with and without Recycling Agents

The researchers then looked at the mixture performance of the different asphalt mixtures, although not as extensively as the asphalt binder testing. Figure 13 was developed by taking the results of the asphalt binder testing and the asphalt mixture testing presented in the final report. The figure shows that the ΔTc parameter did not correlate to either the IDEAL-CT Index or to the DCT Fracture Energy. This may indicate that a difference exists when evaluating recycled asphalt from extracted, recovered and blended (100% blended) to loose mix (blended/black rock condition).

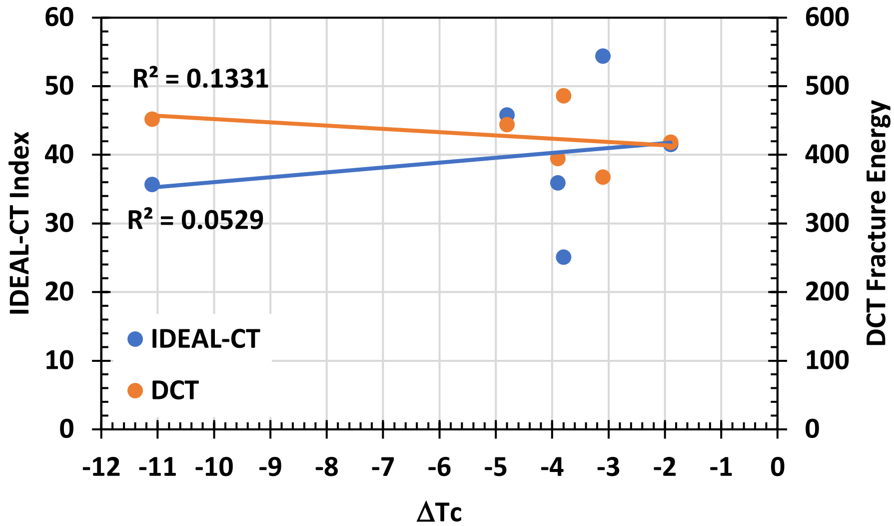


Figure 13 – ΔTc Compared to Mixture Cracking Tests Using Data from WisDOT Report 0092-19-04

<u>Comprehensive Laboratory Evaluation of Recycling Agent (RA) Treated Plant-Produced Asphalt Mixtures (Zhang et al., 2022) and Laboratory Evaluation of Rheological, Chemical and Compositional Properties of Bitumen Recovered from RAP Mixtures Treated with Seven Different Recycling Additives (Reinke et al., 2022)</u>

Data from the on-going National Road Research Alliance (NRRA) study on recycling agents for recycled asphalt mixtures was evaluated and established a relationship between ΔTc and asphalt mixture cracking tests. The two studies were conducted by the same set of researchers from two different perspectives: mixture characterization and asphalt binder characterization. In the mixture study, the collected loose mix was conditioned at a short-term and long-term laboratory aging conditions and tested for their respective fatigue cracking performance using the IDEAL-CT Index and SCB Flexibility Index. In the asphalt binder study, the asphalt binder was recovered from field cores and conditioned at 4 different PAV levels; 0, 20, 40 and 60 hours. After the appropriate conditioning, the asphalt binders were evaluated under a variety of tests that included ΔTc. Overall, 9 different asphalt mixtures/binders were produced and evaluated at different conditioning levels.

Figures 14 and 15 show the comparison of the test results. In Figure 14, the IDEAL-CT Index shows a poor relationship to the ΔTc for each of the four conditioning types. However, Figure 15 shows that no relationship exists between the SCB Flexibility Index and the ΔTc parameter. Like the work presented by Rodezno et al (2021), high recycled asphalt mixes may behave differently than ΔTc predicts due to the potential lack of complete blending between the virgin and recycled asphalt binders. Figure 16 shows, once again, that ΔTc was sensitive enough to trend with the amount of laboratory conditioning for each of the recovered asphalt binders.

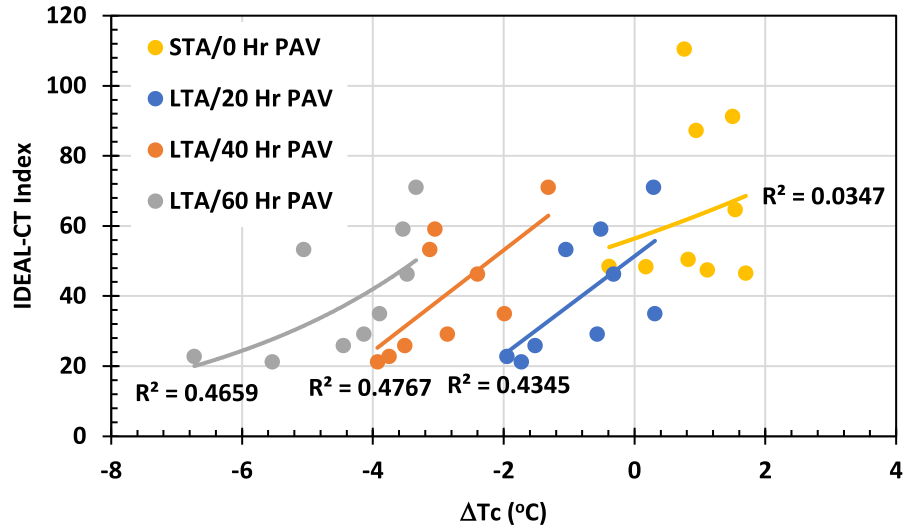


Figure 14 – ΔTc and IDEAL-CT Index Compared at Different Conditioning Levels for NRRA Recycled Asphalt Test Sections

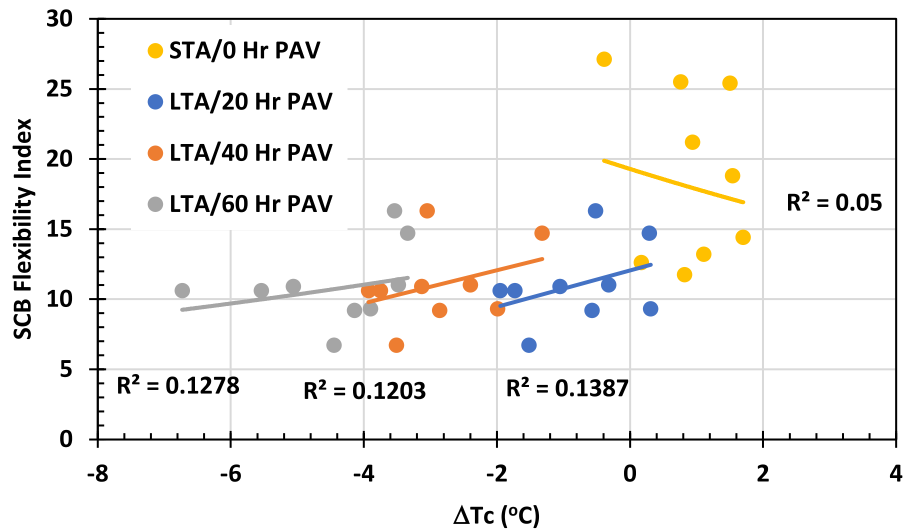


Figure 15 - ΔTc and SCB Flexibility Index Compared at Different Conditioning Levels for NRRA Recycled Asphalt Test Sections

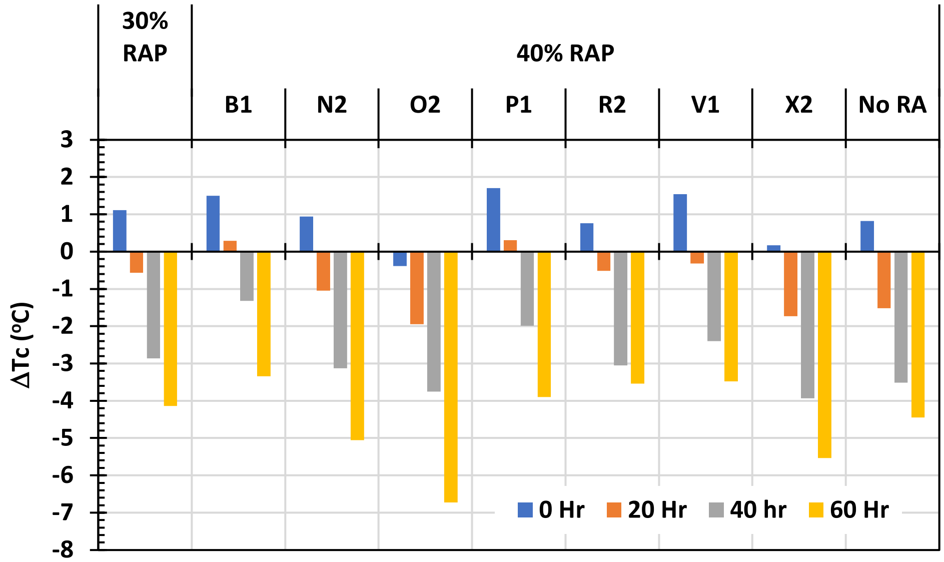


Figure 16 – Change in ΔTc with Laboratory PAV Conditioning for Different Recycled Asphalt Binders from the NRRA Recycled Asphalt Study

<u>Evaluation of Test Methods to Identify Asphalt Binders Prone to Surface Initiated Cracking (Bennert et al., 2023)</u>

In a research study conducted for the Federal Aviation Administration (FAA) to provide better laboratory characterization methods for fatigue cracking performance of asphalt binders, the researchers examined various neat and polymer modified binders at varying levels of laboratory conditioning. The asphalt binders were used in two asphalt mixtures of different effective asphalt contents to compare the asphalt binder and mixture performance. The JFK P401 mix had an optimum asphalt content of 5.5% and an effective asphalt content by volume of 12.2%. The EWR P401 mix had an optimum asphalt content of 6.5% and an effective asphalt content by volume of 14.5%. Figures 17 and 18 show the SCB Flexibility Index and IDEAL-CT Index tested at 25°C for the EWR asphalt mixture. The results show a moderate relationship between ΔTc and the mixture cracking tests. Figures 19 and 20 show a very similar relationship for the JFK mix.

An interesting finding from the study was the impact of the effective asphalt content on the fatigue cracking performance. Figure 21 shows the comparison between the EWR and JFK IDEAL-CT Index measured values. Both mixtures utilized the same asphalt binders, same level of conditioning but the JFK asphalt mixture had 1% lower total asphalt (2.3% less effective asphalt content by volume). The results show that a decrease of 43% in the measured IDEAL-CT Index was found simply due to the change in binder content, even when utilizing the identical asphalt binders. This clearly indicates that although asphalt binder performance is important, perhaps even more critical is the volume of effective asphalt binder in the mixture itself.

It should be noted that all asphalt mixtures in this study were virgin asphalt mixes containing no recycled asphalt.

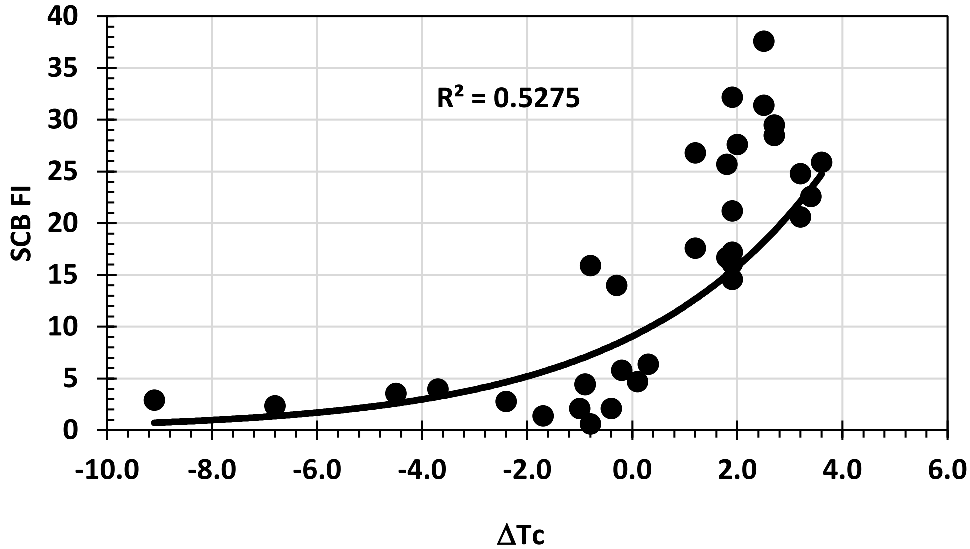


Figure 17 – SCB Flexibility Index vs ΔTc for EWR P401 Asphalt Mixture

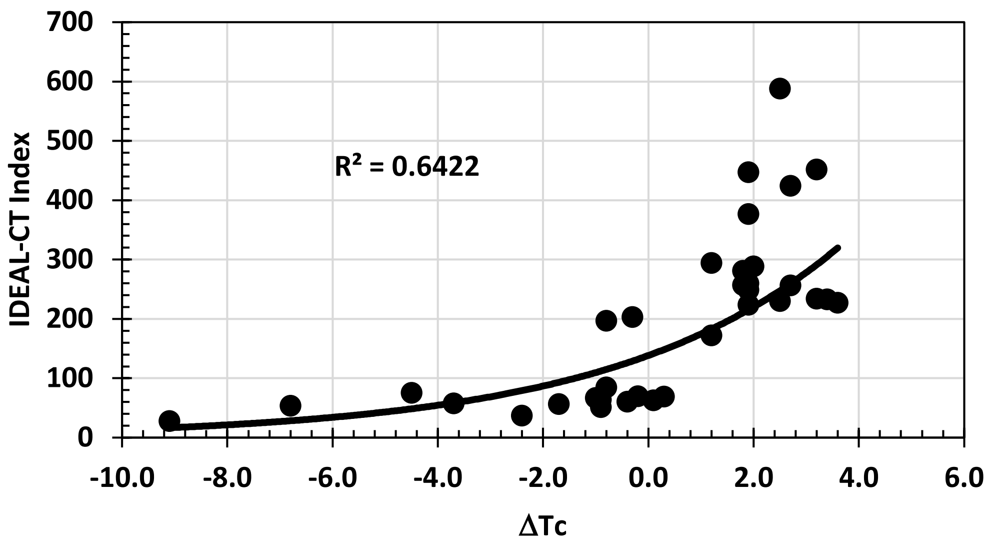


Figure 18 – IDEAL-CT Index vs ΔTc for EWR P401 Asphalt Mixture

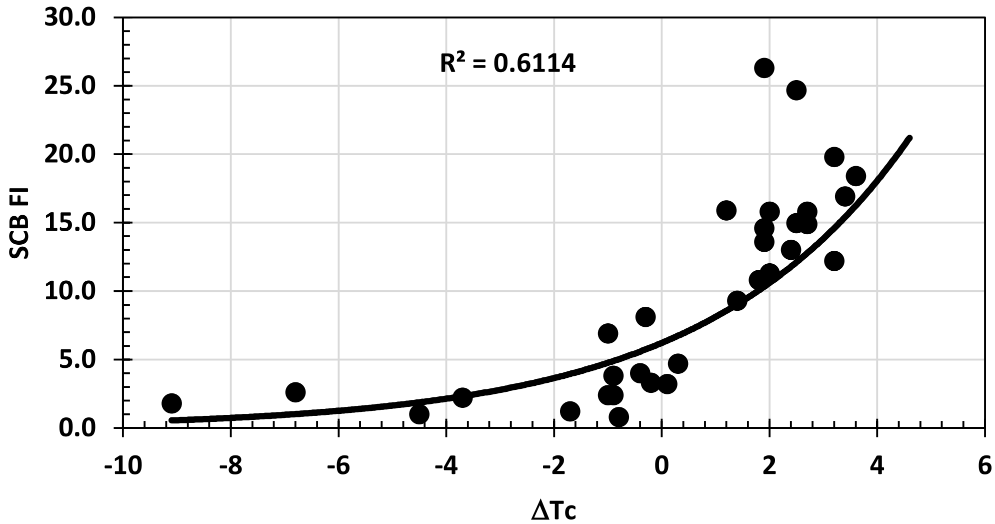


Figure 19 - SCB Flexibility Index vs ΔTc for JFK P401 Asphalt Mixture

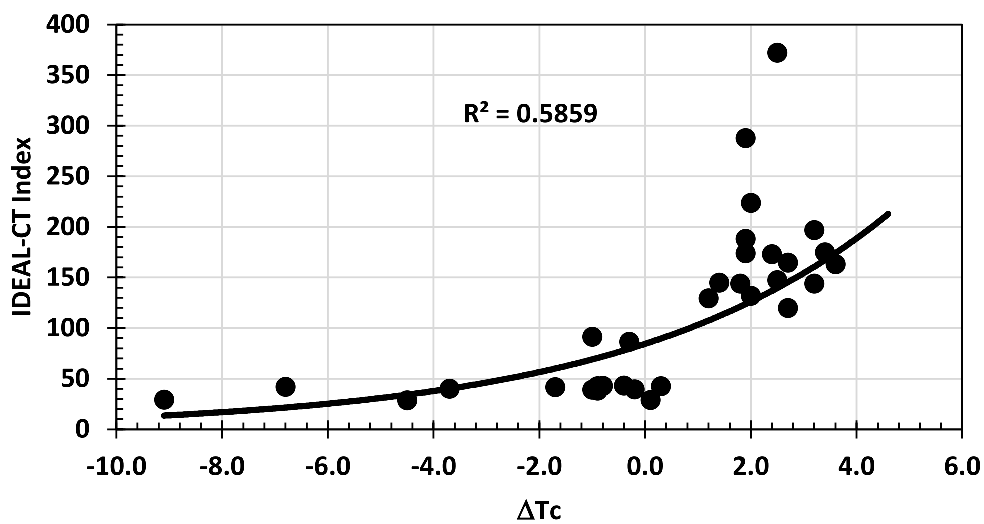


Figure 20 – IDEAL-CT Index vs ΔTc for JFK P401 Asphalt Mixture

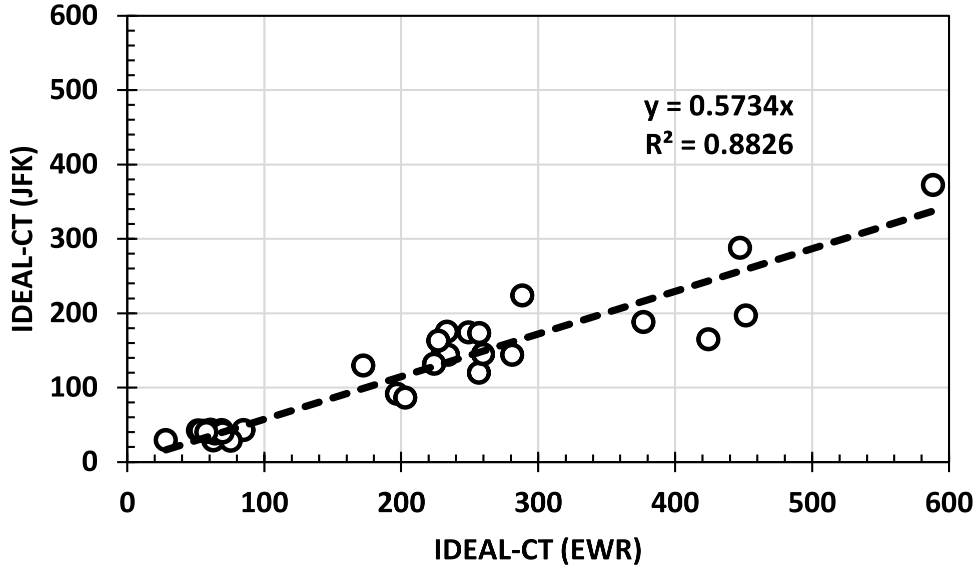


Figure 21 – IDEAL-CT Index Results for EWR and JFK Asphalt Mixtures

<u>Evaluation of Physical Hardening and Oxidative Aging Effects on ΔTc of Asphalt Binders (Yan et al., 2023)</u>

The researchers found that the ΔTc parameter is strongly impacted by the physical hardening and oxidative aging of the asphalt binder. The greater the physical hardening/oxidative aging, the more negative ΔTc becomes. And therefore, the researchers note that ΔTc would be good to help characterize asphalt binders that may age too quickly. The authors also noted that the two aging mechanisms impact ΔTc differently depending on whether or not the asphalt binder has been polymer modified. Through physio-chemical laboratory analysis, the researchers found that the susceptibility of ΔTc to physical hardening is not sensitive to the polymer modification, while polymer modification increases the susceptibility of ΔTc to aging. For neat binders, physical hardening has a stronger effect on decreasing ΔTc than aging does, while for polymer-modified binders, aging has a stronger effect on decreasing ΔTc than physical hardening does.

Physical hardening is a process where the asphalt binder becomes stiffer at low temperatures when exposed to extended low temperatures. This phenomenon is reversible if the asphalt binder is reheated. This can play a role in BBR test results if the test specimens are allowed to sit at low temperatures beyond the times recommended in AASHTO T313. Since the process is reversible, it is not like the increase in stiffness due to aging, which is a chemical change within the asphalt molecular structure that cannot be reversed. Applying this to the above statement from the researchers, the addition of polymers appears to help retard physical hardening, while polymers are more susceptible to oxidative aging.

Field Performance

The Relationship of Binder ΔTc & Other Binder Properties to Mixture Fatigue and Relaxation (Reinke, 2018)

Reinke (2018) presented findings from the CTH 112 Olmsted City, MN test sections where three different crude sources of a PG58-28 were used to construct three 0% RAP test sections (MN1-3, MN1-4, MN1-5). In addition to the PG58-28 sections, one additional 0% RAP section was constructed using a PG58-34 asphalt binder (MN1-2) and one 20% RAP section with the identical PG58-34 asphalt binder (MN1-1). After 4 to 5 years of service life, substantial surface cracking was observed on some of the sections.

Figure 22 shows a comparison of the ΔTc value measured on recovered asphalt binder from the top ½" of the asphalt surface to different observed cracking modes. When combining the results, ΔTc showed an excellent correlation to total cracking in the field (Figure 23). The measured ΔTc from the recovered asphalt binder appeared to be capable of capturing the age hardening of the asphalt field sections and correctly rank the cracking performance of the Olmsted, MN test sections.

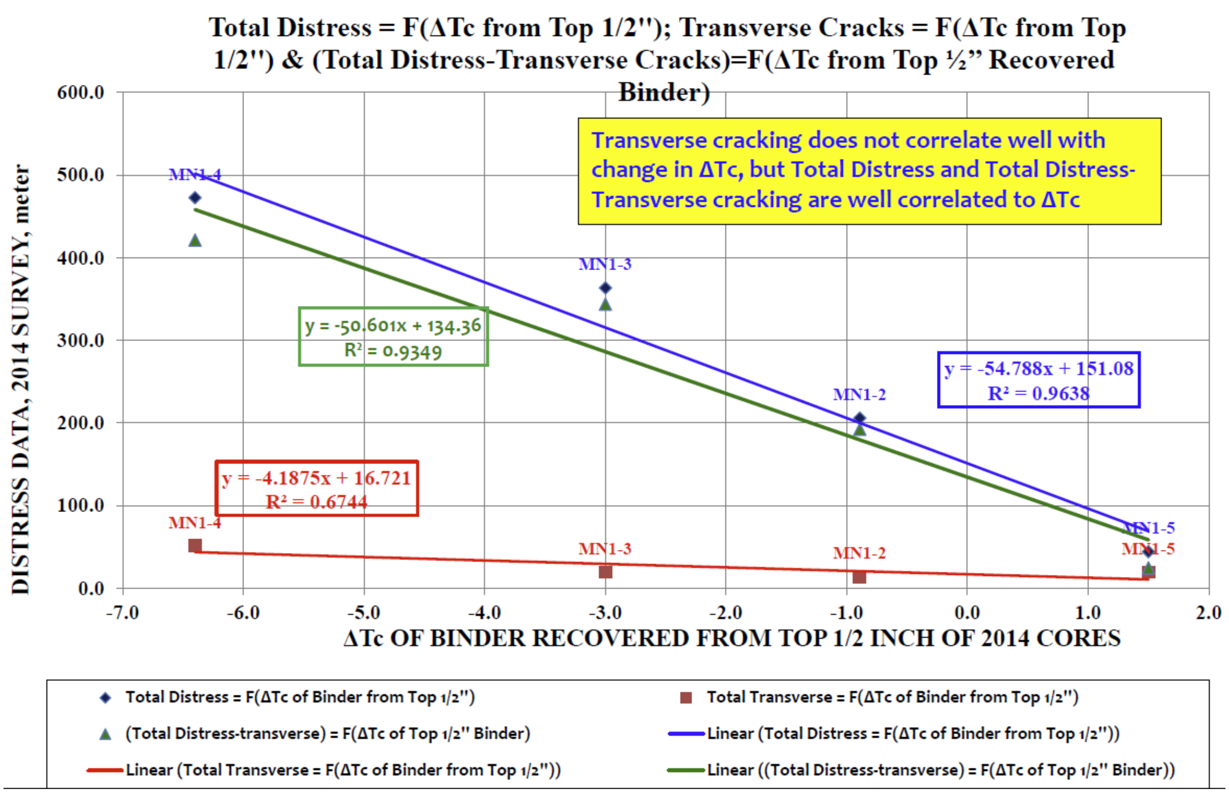


Figure 22 – ΔTc of Recovered Asphalt Binder Compared to Observed Cracking Distress (After Reinke, 2018)

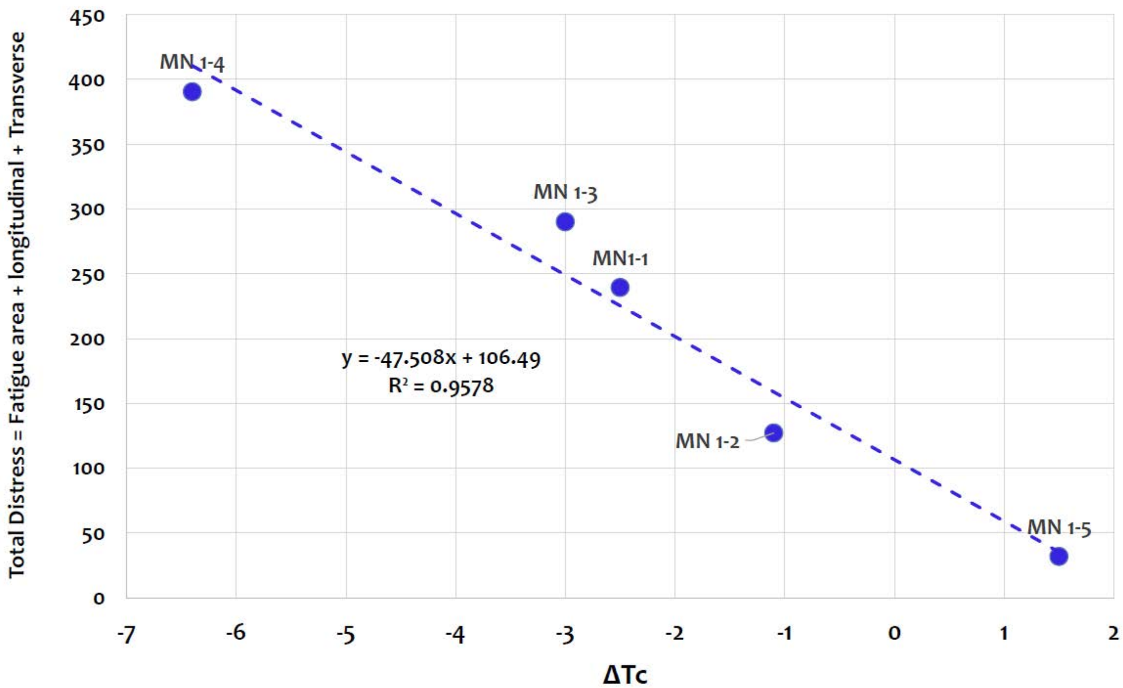


Figure 23 – Total Cracking Distress Compared to ΔTc from Recovered Asphalt Binder (After Reinke, 2018)

Reinke (2018) also presented the results of an MnROAD project where three different pavement sections/cells were produced and constructed using three different asphalt binders; 1) PG58-28; 2) PG58-34; and 3) PG58-40. The pavement sections were constructed in 1999 and trafficked until 2007. Annual distress surveys were conducted and showed little to no cracking for the first 4 years of service. However, after 4 years, cracking quickly progressed in the PG58-40 test section while minimal to no changes were observed in the other two sections. Although the asphalt binder was not recovered for any of the test sections, the field observations were found to trend with the ΔTc of the 40 hr PAV conditioned asphalt binder (Figure 24).

<u>Evaluation of Overlay Tester Test Procedure to Identify Fatigue Cracking Prone Asphalt Mixtures (Bennert et al., 2019)</u>

Bennert et al. (2021) utilized the accelerated loading results from the FHWA's Accelerated Loading Facility (ALF) at the Turner-Fairbanks facility to evaluate different asphalt mixture and binder fatigue cracking test methods and parameters. The ALF contained ten (10) different test lanes consisting of identical pavement structures to evaluate the impact of recycled asphalt and warm mix asphalt (WMA) on the fatigue cracking performance of asphalt pavements. The study entitled, *Advance Use of Recycled Asphalt in Flexible Pavement Infrastructure: Develop and Deploy Framework for Proper Use and Evaluation of Recycled Asphalt in Asphalt Mixtures*, produced the asphalt layers of the testing lanes with varying amounts of recycled asphalt pavement (RAP), recycled asphalt shingles (RAS), WMA technologies and different asphalt binder grades. Table 3 shows how the asphalt mixture was varied for each testing lane.

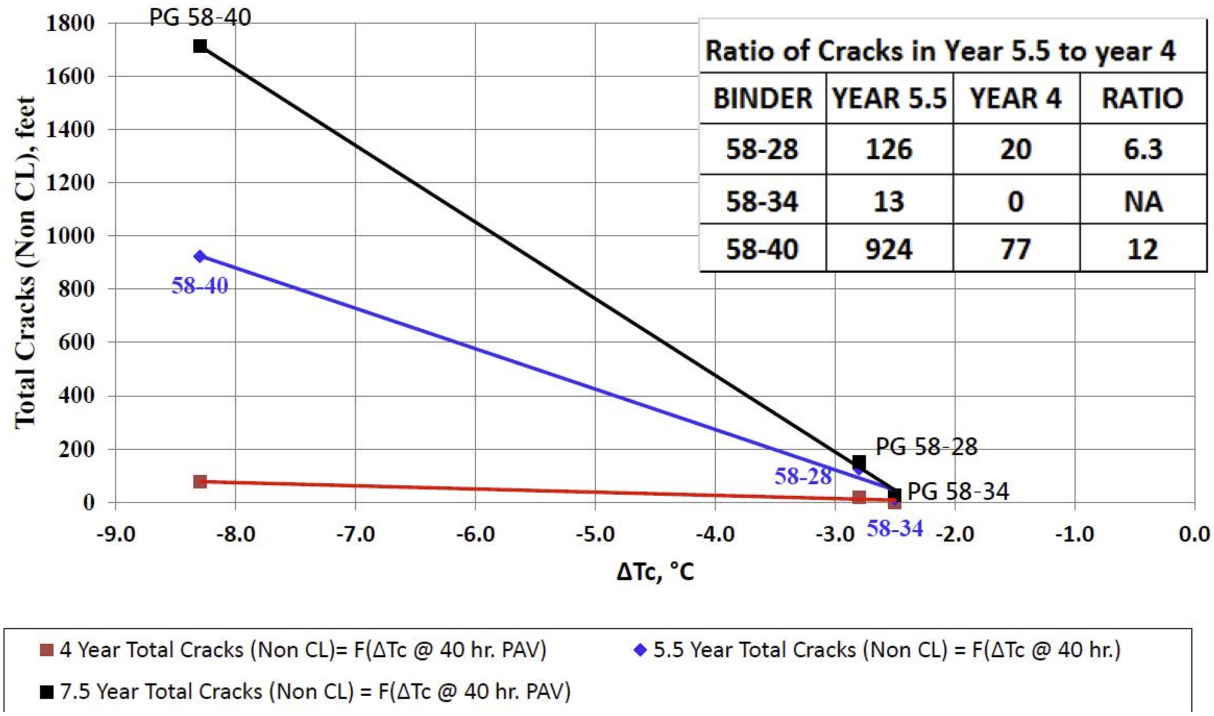


Figure 24 – 40 Hr PAV ΔTc Compared to Field Cracking Performance of MnROAD Test Sections (After Reinke, 2018)

Table 3 – Experimental Design for FHWA ALF Sustainability Study

ALF Lane #	% ABR		Virgin PG Grade	Drum Discharge Temperature	WMA Process	Passes to First ALF Crack
	RAP	RAS				
1	0	--	64-22	300-320	--	368,254
2	40	--	58-28	240-285	Water Foaming	123,035
3	--	20	64-22	300-320	--	42,399
4	20	--	64-22	240-270	Evotherm	88,740
5	40	--	64-22	300-320	--	36,946
6	20	--	64-22	300-320	--	122,363
7	--	20	58-28	300-320	--	23,005
8	40	--	58-28	300-320	--	47,679
9	20	--	64-22	240-285	Water Foaming	270,058
11	40	--	58-28	240-270	Evotherm	81,044

The main ALF parameter used to compare the performance of the different asphalt mixtures was the Number of Passes to 1st Crack. Additionally, the Cracking Rate, which is defined as the measured crack length in inches per ALF pass, was also included in the comparison.

The pavement structure at the FHWA ALF consisted of silty sand subgrade (resilient modulus ≈ 9,000 psi) overlaid by 22 inches of crushed aggregate base (resilient modulus ≈ 12,000 psi). The surface consisted of 4 inches of asphalt. The asphalt layer was placed in two lifts of 2 inches thick. The asphalt mixture type was dependent on the experimental lane, as noted in Table 3.

The test lanes were loaded using a 425/65R22.5 wide base tire at an inflation pressure of 100 psi. A wheel load of 14,200 lbs was applied during the trafficking. The travel speed of the applied tire load was 11 mph (4.9 m/s). The temperature of the test lanes was controlled to maintain a 20°C temperature at the mid-depth (2 inches below the surface) of the asphalt layer.

Cracking was assessed and traced with a planimeter to capture the number of passes associated with the total cracking length. This provided a means for the FHWA to determine Crack Rate. The fatigue cracking results for the FHWA ALF Sustainability Study are shown in Figure 25. Approximately ten (10) field cores were obtained from each of the experimental ALF lanes (Figure 26). Once delivered to the Rutgers Asphalt Pavement Laboratory, the top ½" to ¾" of the surface course was removed with a wet masonry saw. The asphalt binder from the upper ½" to ¾" of the field core underwent a solvent extraction and recovery and tested under a variety of asphalt binder fatigue/durability related performance tests. The asphalt binder performance results were compared with the measured fatigue cracking of the FHWA ALF lanes. Observed fatigue cracking from the FHWA ALF lanes were top-down with none of the experimental lanes undergoing bottom-up fatigue cracking.

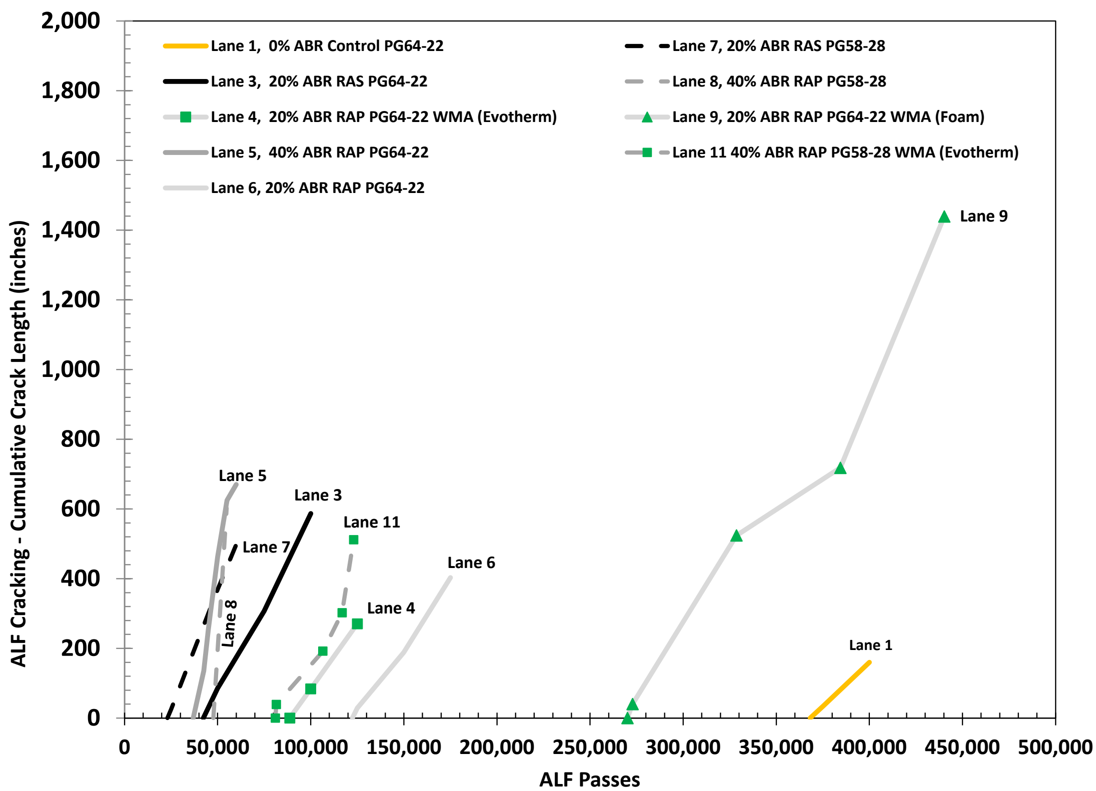


Figure 25 – FHWA ALF Fatigue Cracking from Sustainability Study

Figure 26 – Field Cores Recovered from FHWA ALF for Asphalt Mixture and Binder Testing

The ΔTc was evaluated and compared to asphalt mixture performance on the FHWA ALF test sections. The results of the ΔTc for the ALF asphalt mixtures are shown in Figure 27. The results show a wide range of performance with the 0% recycled asphalt mixture resulting in the warmest (best) ΔTc value. Recovered asphalt binder was not available for Lane 2 (PG58-28, 40% RAP with foamed WMA).

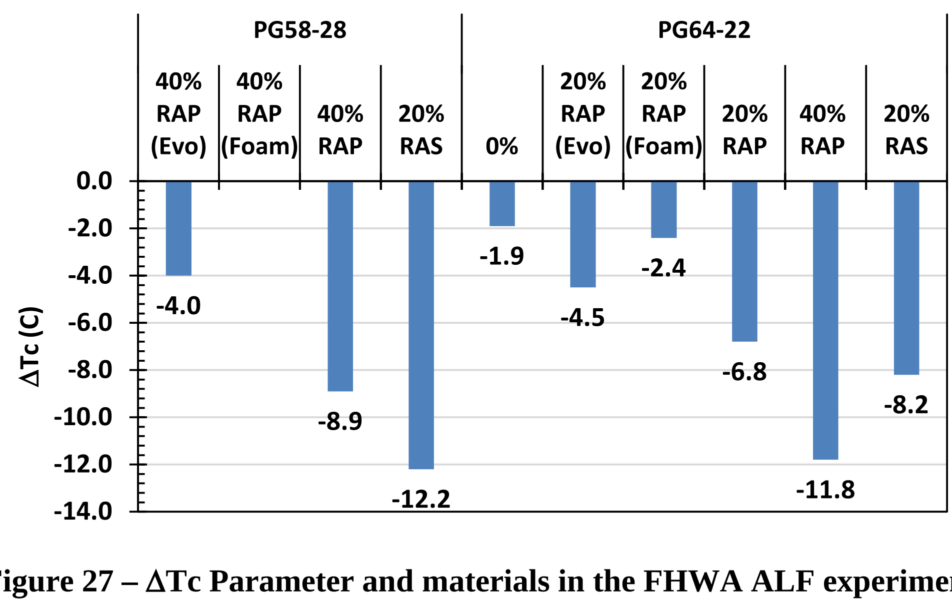


Figure 27 – ΔTc Parameter and materials in the FHWA ALF experiment

The comparison between the ΔTc to the FHWA ALF Number of Passes to 1st Crack is shown as Figure 28. The results show a good comparison between the ΔTc and the FHWA ALF performance, where the number of passes to the 1st crack increases as the ΔTc value becomes warmer (less negative). All asphalt binders used in the FHWA ALF sections were neat and not containing any polymer modification.

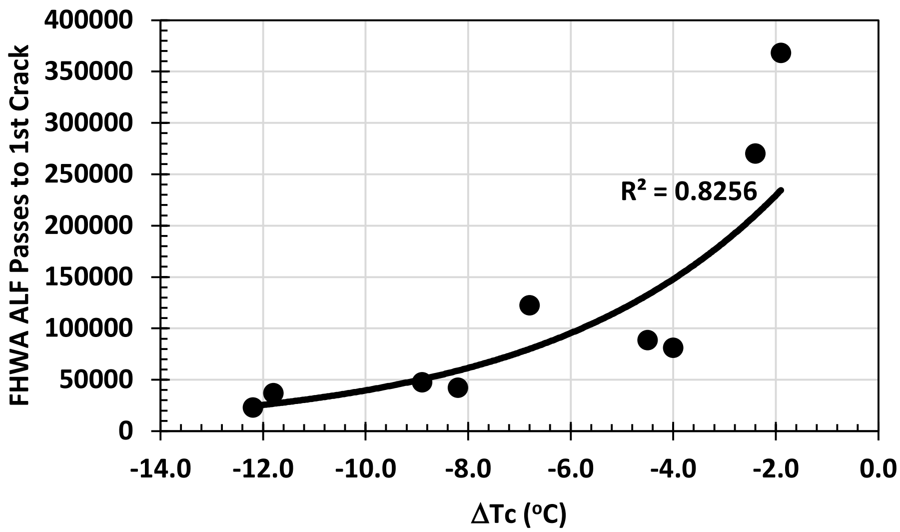


Figure 28 – ΔTc Parameter vs the FHWA ALF Number of Passes to 1st Crack

<u>Evaluation of Test Methods to Identify Asphalt Binders Prone to Surface Initiated Cracking (Bennert et al., 2023)</u>

Rutgers University worked with the FAA Technical Center in Atlantic City, NJ to procure field cores from an on-going study at the FAA Technical Center entitled, *Extended Pavement Life*. The *Extended Pavement Life* study was an effort by the FAA to evaluate various airfield pavements across the country and determine the in-situ and material properties of the pavement structure. The main premise of the study was to collect information to help determine critical airfield pavement parameters that would allow the FAA to improve, and thereby, "extend" the service life of the airfield pavements. Field cores from the following airports were provided to Rutgers University for evaluation; Baltimore-Washington (BWI), Columbus, Greensboro, Kansas City, Salt Lake City, and Tucson.

In addition to the Extended Life Study field cores, Rutgers University included field and laboratory data from a research study conducted for the Port Authority of NY/NJ (PANYNJ) in 2015 (Bennert et al., 2017). Field cores from Newark and JFK International airport were provided to Rutgers University from five (5) different runways of the respective airports. Various levels of top-down field cracking were observed on the runways, ranging from "No Cracking" to "Severe Transverse and Longitudinal Cracking". The airport location and respective PG temperature information for the airfield pavements are summarized in Table 4.

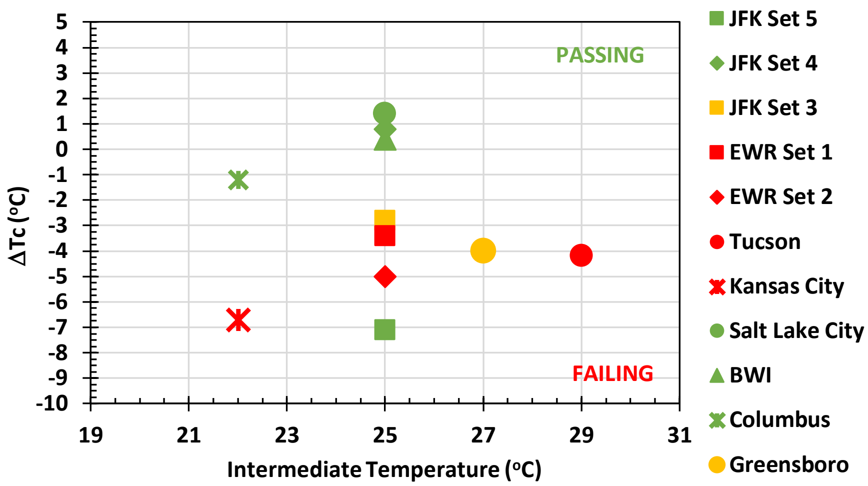
For all of the airfield pavements evaluated, the top ½" to ¾" was removed with a wet masonry saw and underwent a solvent extraction and recovery.

Table 4 – Locations and Intermediate PG Temperatures for Asphalt Binder Fatigue Verification Projects (After Bennert et al., 2023)

Airport Location	Tucson, AZ		Kansas City, KS		Salt Lake City, UT		Newark, NJ	
PG Temperature (°C)	High	Low	High	Low	High	Low	High	Low
PG Temp at 50% Reliability	67.9	-0.5	57.9	-15.6	56.1	-11.3	56.9	-11.8
PG Temp at 98% Reliability	69.0	-6.0	60.3	-23.1	58.7	-18.7	59.4	-17.8
Adjustments for Traffic	0.0		0.0		0.0		0.0	
Adjustments for Depth	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Adjusted PG Temperature	69.0	-6.0	60.3	-23.1	58.7	-18.7	59.4	-17.8
Selected Binder Grade	70.0	-10.0	64.0	-28.0	64.0	-22.0	64.0	-22.0
Intermediate Temp (°C)	29		22		25		25	

Airport Location	Queens, NY		BWI Airport		Columbus Airport		Greensboro, NC Airport	
PG Temperature (°C)	High	Low	High	Low	High	Low	High	Low
PG Temp at 50% Reliability	53.4	-10.5	57.4	-10.3	55.4	-15.2	59.6	-7.9
PG Temp at 98% Reliability	55.8	-16.2	59.7	-16.7	57.8	-23.3	61.3	-14.2
Adjustments for Traffic	0.0		0.0		0.0		0.0	
Adjustments for Depth	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Adjusted PG Temperature	55.8	-16.2	59.7	-16.7	57.8	-23.3	61.3	-14.2
Selected Binder Grade	58.0	-22.0	64.0	-22.0	58.0	-28.0	64.0	-16.0
Intermediate Temp (°C)	25		25		22		27	

The results of the analysis are shown in Figure 29. The **Green** symbols are noted as **Good** performance, **Gold** symbols are noted as **Moderate** performance, and the **Red** symbols are noted as **Poor** performance. The data suggests that a value close to -2.0°C could properly identify the Good to the Moderate/Poor performance. One airfield pavement, JFK Set #5, had good field performance but measured the worst ΔT_c value of -7.1°C. This asphalt mixture utilized a polymer-modified PG76-28 asphalt binder. No information was provided on how the asphalt binder was produced at the terminal.



Airfield Pavement	Intermediate Temp	ΔTc	Cracking Performance	Visual Distress
JFK 4L-22R (Set 5)	25	-7.1	Fail	No Cracking
JFK 4L-22R (Set 4)	25	0.8	Pass	No Cracking
JFK 4L-22R (Set 3)	25	-2.8	Pass	Minor T Cracking
EWR 11-29 (Set 1)	25	-3.4	Pass	Severe L&T Cracking
EWR 11-29 (Set 2)	25	-5	Fail	Severe L&T Cracking
Tuscon	29	-4.2	Pass	Block, L&T Cracking, PCI = 56 (+/- 20)
Kansas City	22	-6.7	Fail	Block, L&T Cracking, PCI = 57 (+/- 17)
Salt Lake City	25	1.4	Pass	No Cracking, PCI = 94 (+/- 4)
BWI	25	0.4	Pass	Low Severity Weathering, PCI = 93 (+/- 3)
Columbus	22	-1.2	Pass	No Cracking, PCI = 100
Greensboro	27	-4.0	Pass	Low Severity Weathering/Cracking, PCI = 89 (+/- 2)

Figure 29 – ΔTc vs Field Distress on Various Airfield Pavements (After Bennert et al., 2023)

Other Considerations for Binder Test Methods/Protocols

Asphalt Parameters and Specification Development: Traveling Through the Jungle of Asphalt Parameter Development (Rowe, 2021)

Dr. Geoff Rowe provided a presentation at the 2021 Asphalt Institute Annual Meeting regarding the different asphalt binder test methods currently available and being proposed. The presentation noted that many of the existing procedures can be categorized into one of three different types; 1) Asphalt binder master curve shape parameters; 2) Asphalt binder master curve point parameters; and 3) Asphalt binder fracture tests.

A shape parameter defines the shape of the asphalt binder master curve (i.e. – shear stiffness and phase angle) tested under a wide range of temperature and loading rates. Meanwhile, a point parameter identifies a value on the master curve that represents a specific value at a specific temperature and loading rate. Figure 30 schematically shows an asphalt binder master curve with theoretical point parameters, while Table 5 identifies the multiple different shape and point parameters currently being evaluated in the industry. And finally, the fracture tests represent the asphalt binder's ultimate strength at a specific temperature and loading rate. Fracture tests currently being evaluated by the industry include, but not limited to; 1) Asphalt Binder Cracking Device (ABCD); 2) Binder Yield Energy Test (BYET); 3) Double Edge Notched Tension Test (DENT); 4) Direct Tension Test; and 5) Linear Amplitude Sweep (LAS).

The emphasis of the presentation was to highlight the fact that many of the test methods/parameters within their same respective category are measuring the same characteristic and that there can be simplifications in the asphalt industry by identifying this. With respect to ΔTc, which is classified as a shape parameter, other parameters such as phase angle (δ) at a designated stiffness, crossover modulus and R-value should all correlate to one another as they represent the shape characteristics of the master curve.

Figures 31 and 32 show the ΔTc parameter compared to the Phase Angle (δ) at G* = 10 MPa (Shape Parameter) and ΔTc compared to the Glover-Rowe Parameter (Point Parameter) for the same set of asphalt binders and laboratory conditioning levels. The figures show a good correlation between the parameters.

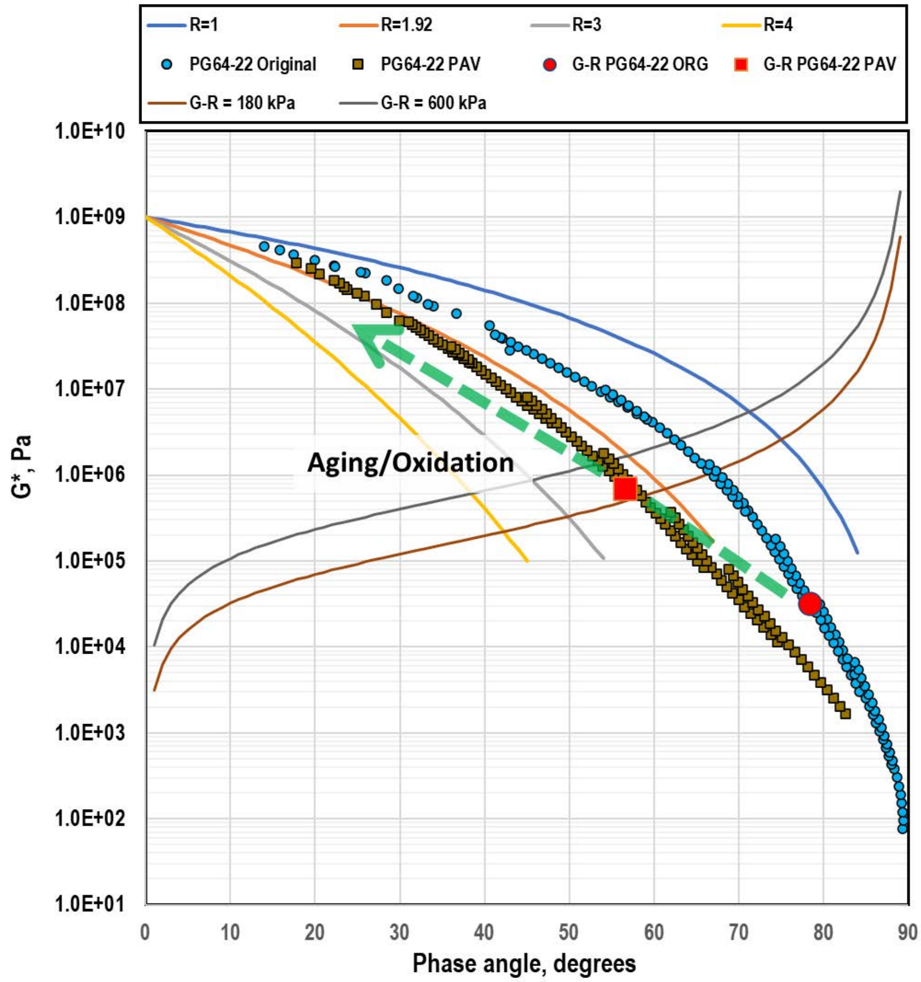


Figure 30 – Idealized Schematic of an Asphalt Binder Master Curve (Shape) and Potential Master Curve Point Parameters (After Rowe, 2021)

Table 5 – Examples of Different Shape and Point Parameters for Asphalt Binder Fatigue Characterization (After Rowe, 2021)

Parameter	Definition	Notes
S(t), t=60 sec.	**Stiffness**	The limiting value for this was specified as ≤300MPa – this effectively corresponds to a phase angle around 26 to 27 degrees (depending on interconversion method). It provides a measure of the binder attract stress. Used to limit cold temperature cracking.
m(t), t=60 sec.	**m-value**	The limiting value for this was specified as ≥0.300 – this effectively corresponds to a G* stiffness of around 100 to 115 MPa (depending on interconversion method). It provides a measure of the binder ability to relax stresses. Used to limit cold temperature cracking.
ΔTc	**Delta Tc** - the grading temperature for S(t) minus the grading temperature for m(t), evaluated in the BBR.	Has been adopted in several USA states on PAV20 or PAV40 binder. Typically needs to be greater than -5, but this requirement changes. The parameter describes the shape of the master curve in the temperature domain. The parameter was introduced in discussions of non-load related cracking.
Gg	The **glassy modulus**	The limiting stiffness at a very high frequency/short loading time at the extreme cold temperature when an asphalt binder is in a glassy state. This is sometimes assumed as 1e9 Pa or it can be calculated from the fit of a rheological model, such as the Christensen-Anderson model.
R-value	The **rheological index** as defined in the Christensen-Anderson model.	Defines the shape of the master curve in the frequency domain. Different methods can be used to obtain. Requires use and/or definition of the glassy Modulus (determined by assumption or extrapolation).
ω_c	**Cross over frequency**	The frequency in a dynamic master curve when the phase angle is equal to 45, or the storage and loss modulus have the same numerical value. The location of the cross over frequency depends upon the reference temperature which must always be given. This parameter is effectively a measure of the hardness of an asphalt binder. For more complex materials several cross over frequencies may exist. This parameter is used in the Christensen-Anderson model.
t_{cr}	**Time corresponding to cross-over frequency**	This is the reciprocal of the cross over frequency (1/radians per second).
Gc or G^*_{VET}	**Cross-over modulus.** The G* modulus at the cross-over condition	The modulus when the phase angle is equal to 45, or the storage and loss modulus have the same numerical value. This parameter can be considered as a function of the R-value and Gg ~ log Gc = log Gg – R. If the Gg is assumed as 1e9, the log Gc = 9 – R. Thus, this parameter defines the shape of the master curve, in the frequency domain in a very similar manner to the R-value. In this context we make use of the log of the parameter. In some areas this as been defined by the parameter G^*_{VET}
T_{Gc} or T_{VET}	**Temperature corresponding to cross-over modulus**	This parameter is similar to the cross over frequency – but in this case in the temperature domain rather than the frequency domain. It provides an indication of the hardness of the material.
PK δ or $\delta_{G^*=8.967MPa}$	**Phase angle at a G* of 8.967 MPa**	This parameter is advocated for use with the fatigue cracking specification and has been implemented by AASHTO. The definition of a phase at a given modulus results in this parameter defining the shape of the master curve in the high stiffness region. In this respect it is similar to ΔTc, R-value, Gc in that it is a shape parameter for the high stiffness rheology. A critical value of 42° is considered.
PK Freq	**Frequency at a reference temperature corresponding to PK δ**	As a master curve is developed at a Tref then the phase stiffness location of 8.967 MPa will correspond to a frequency. We have referenced this as the PK Freq. This parameter like ω_c and T_{Gc} captures the relative hardness. This parameter is not currently used in specifications.
$(\tan\delta)^2_{G^*=10MPa}$	**Loss tangent squared**	A parameter being considered in NJ (Rutgers) with warning and failure criteria of 0.6 and 0.75. In this respect it is similar to ΔTc, R-value, Gc in that it is a shape parameter for the high stiffness rheology. This parameter can be expressed as a phase angle in the same manner as the PK δ value.
$\delta_{G^*=10MPa}$	**Phase angle at a G* of 10 MPa**	Similar to PK δ or $\delta_{G^*=8.967MP}$ – this would be the Loss Tangent Squared parameter being considered in New Jersey. The warning and failure criteria would be 40.9 and 37.8° respectively.
G* sin δ or G″	**Loss modulus**	Specified in the orginal SHRP specification to relate to dissipated energy and fatigue cracking. This combined with the BBR data was considered to provide some control of the R-value (controls a minimum value).
G-R	**Glover-Rowe parameter**	The value of $[G^*(\sin\delta)^2]/\cos\delta$ – measured at 0.005 rads at 15C, or at some other conditions. Can be used for assessment of cracking, thermal, fatigue and/or durability – depending on methods used. This is a point parameter that captures a measure of stiffness and relaxation. Critical values of 180 and 600 kPa have been suggested for the onset and significant durability cracking.

Shape parameter – captures the shape in master curve in the high stiffness region.
Point parameter – indicates a stiffness or combination of stiffness and phase angle

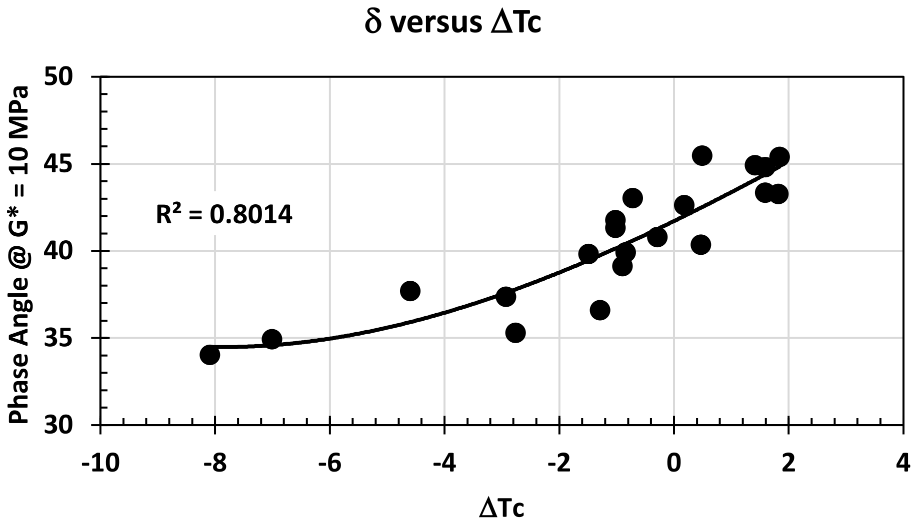


Figure 31 – Shape Parameter (ΔTc) Compared to Another Shape Parameter (δ @ G* = 10 MPa) for the Same Set of Asphalt Binders and Conditioning Levels

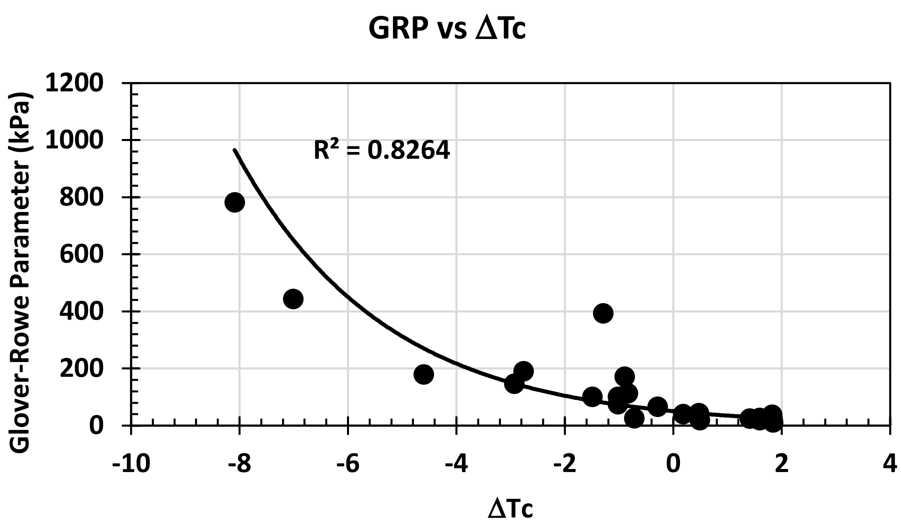


Figure 32 - Shape Parameter (ΔTc) Compared to a Point Parameter (Glover-Rowe Parameter) for the Same Set of Asphalt Binders and Conditioning Levels

Proposed Changes to Asphalt Binder Specifications to Address Binder Quality-Related Thermally Induced Surface Damage (Elwardany et al., 2022)

The researchers noted that linear visco-elastic (LVE) rheological parameters, such as ΔTc, can account for asphalt binder stiffness and relaxation. However, due to the low strain environment in the testing, the LVE parameters do not fully characterize the fracture resistance, especially when elastomeric modifiers are present. Low strain testing does not effectively capture strain tolerance, fracture toughness, and thermal contraction as well as larger strain test methods. To compensate for this, the researchers proposed a binder failure index, ΔTf, which is defined as the difference between the BBR Tc(S) and the critical cracking temperature (Tcr) from the ABCD test. The ΔTf parameter was found to successfully rank asphalt binders' failure strength at low temperature and gives "credit" to well-formulated and compatible modifiers that typically increase binder strength and strain tolerance, such as high-quality, elastomeric polymer-modified asphalt binders. The researchers proposed two different performance spaces, based on both laboratory and field data, one for 20 hr PAV and one for 40 hr PAV conditioned asphalt binders. The researchers noted that a general adoption into AASHTO specifications could look like the following.

1. Conduct conventional BBR data to determine the low temperature performance grade of the asphalt binder.
2. Determine the ΔTc parameter.
    a. If the ΔTc is greater than the critical value based on the PAV conditioning time as noted in the **GREEN** (1) zone in Figure 33, the asphalt binder is considered **Passing**.
    b. If ΔTc is less than a critical value based on the PAV conditioning time as noted in the **RED** (2) zone, the asphalt binder is considered **Failing** for durability issue potential.
    c. If ΔTc falls between the Failing critical value (zone 2) and a Passing value (zone 1), conduct the ABCD test on the same asphalt binder.
3. If the calculated ΔTf of the asphalt binder falls within the **GREEN** (3) zone shown in Figure 33, even though the ΔTc is not passing, the asphalt binder is considered to have sufficient fracture toughness and would be classified as **Passing**.
4. If the calculated ΔTf of the asphalt binder falls in the **RED** (4) zone shown in Figure 33, the asphalt binder is considered to have insufficient fracture toughness and would be classified as **Failing**.

As noted earlier, the thresholds shown in Figure 33 would vary depending on the level of laboratory conditioning used to age the asphalt binder. The proposed limits shown in Figure 33 currently represent the researcher's opinion for 20 hour pressure aging vessel (PAV) conditioning.

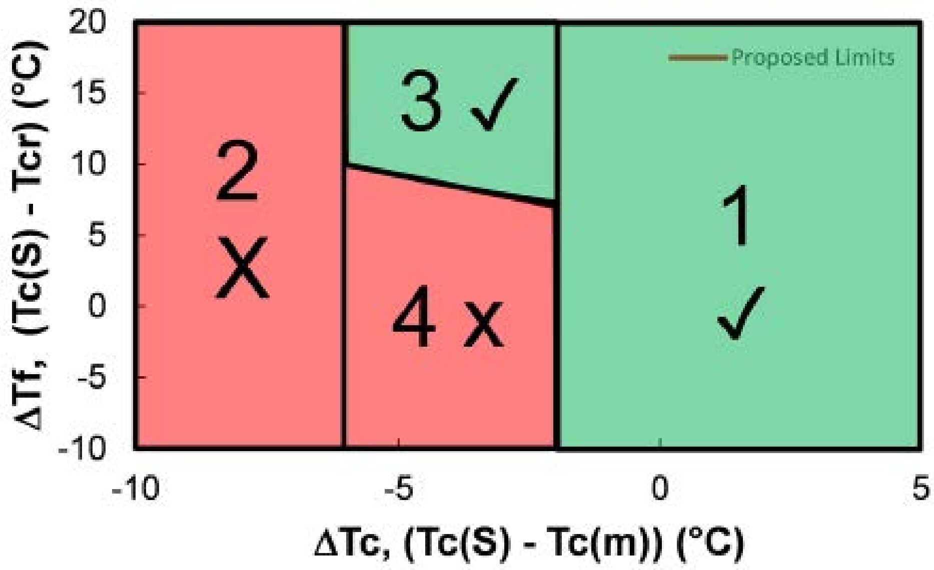


Figure 33 – Proposed AASHTO Specification Inclusion for 20 Hour PAV Conditioned Asphalt Binders to Minimize Durability Issues in Asphalt Pavements

<u>BMD: Does Chemistry Matter? (Planche, 2023)</u>

Presented in the 2023 TRB workshop entitled, *The Effect of Asphalt Supply on Balanced Mix Design*, Planche (2023) proposed that asphalt binder chemistry can help to "fingerprint" different asphalt binder "families" which can be used to help select asphalt binders for different applications. JP Planche (2023) showed how the different binder formulation practices impact several asphalt binder parameters, including ΔTc. Table 6 is from the presentation and highlights the asphalt binder formulation practices and how it impacts ΔTc. As Table 6 notes, most modification practices will have a negative influence on the ΔTc parameter, except for conventional SBS or reacted terpolymers.

<u>Implementing Asphalt Binder Research Results into a BMD Framework (Anderson, 2023)</u>

Presented in the 2023 TRB workshop entitled, *The Effect of Asphalt Supply on Balanced Mix Design*, Anderson (2023) presented an update on the NCHRP funded study to utilize completed NCHRP research to improve the asphalt binder performance grading and test protocols (NCHRP 20-44 (19)). Anderson (2023) provided some interesting insight as to why the current intermediate temperature grading protocol may not be sufficient to indicate whether an asphalt binder is susceptible to cracking and durability issues.

The first issue was selecting the use of dissipated energy in the form of G*(sin δ) to indicate a critical stiffness level of the asphalt binder. According to SHRP Report A-367, the researchers changed the method to dissipated energy after evaluating the field performance of the Zaca-Wigmore Asphalt Test Road. The authors of the Zaca-Wigmore research report noted:

Table 6 – Asphalt Binder Modification Practices and Impact on Binder Properties (After Planche, 2023)

	Wax from asphalts	REOB	Air blowing	Thermal Conversion visbreaking	Polymers Physical blends	Crosslinked SBS or reacted Terpolymers
CII	Neutral	↗	↗ ↗	↗ ↗	↗	Neutral
PI (GPC)	↗	↗	↗ ↗	↘↘	↗↗	↗
Oxidation	Neutral	↗	↗	↗ ↗	Neutral or ↗	Neutral or ↗
PH	↗ ↗	↗	↗	Neutral	Neutral or ↗	Neutral or ↗
PG Low	↗	↘	↗	↗↗	↘	↘↘
Tc(S)	Neutral	↘↘	↗	Neutral	↘	↘
Tc(m)	↗	↗	↗↗	↗↗	↗	Neutral
DT_c	↘	↘	↘↘	↘↘	↘	Neutral
Tcr (ABCD)	Neutral	↘	↗	↗	↗↗	↗
DT_f	↘	↗	↘	↘	↗↗ (SBS)	↗
Failure Modulus (DTT)	↘	Neutral	↘	↘	↗ EVA, SBR (stress) ↗ SBS (strain)	↗ Terpo (stress) ↗ XL SBS (stress + strain)

> "Two main types of failure during service life were encountered on the project. The most prevalent was displayed by wheel track "alligator" type cracking. The other was a large block type cracking together with pitting and raveling. This amount of fatigue type cracking appears to be related to the consistency of the recovered asphalt as a measure of penetration and viscosity. The other form of cracking appears to be related to the gain in shear susceptibility during weathering. This is also indicated by a marked drop in ductility during service life. This form of cracking, as found on this test project appears to be the same as that encountered by P.C. Doyle." (Zube and Skog, 1961)

Based on the SHRP researcher's recommendations, the G*(sin δ) dissipated energy approach was recommended for the "… alligator type cracking…". However, no recommendation was made on how to handle the block cracking with raveling. And with respect to the SHRP researchers attempts to utilize G*(sin δ), the researchers did not understand at the time the magnitude of the impact of aging on the age hardening and resultant cracking.

Figure 34 shows the results of measuring asphalt binders at 25°C in the DSR for their respective shear stiffness (G*) and phase angle (δ). When utilizing the dissipated energy calculation, all the tested asphalt binders fall below the **Green Line**, representing a Passing Intermediate PG at 25°C. However, using the same measured data but with the Glover-Rowe Parameter (GRP) calculation (Equation 4), nine (9) of the data points are above the **Orange Line**, identifying that the binder fails at an intermediate temperature of 25°C.

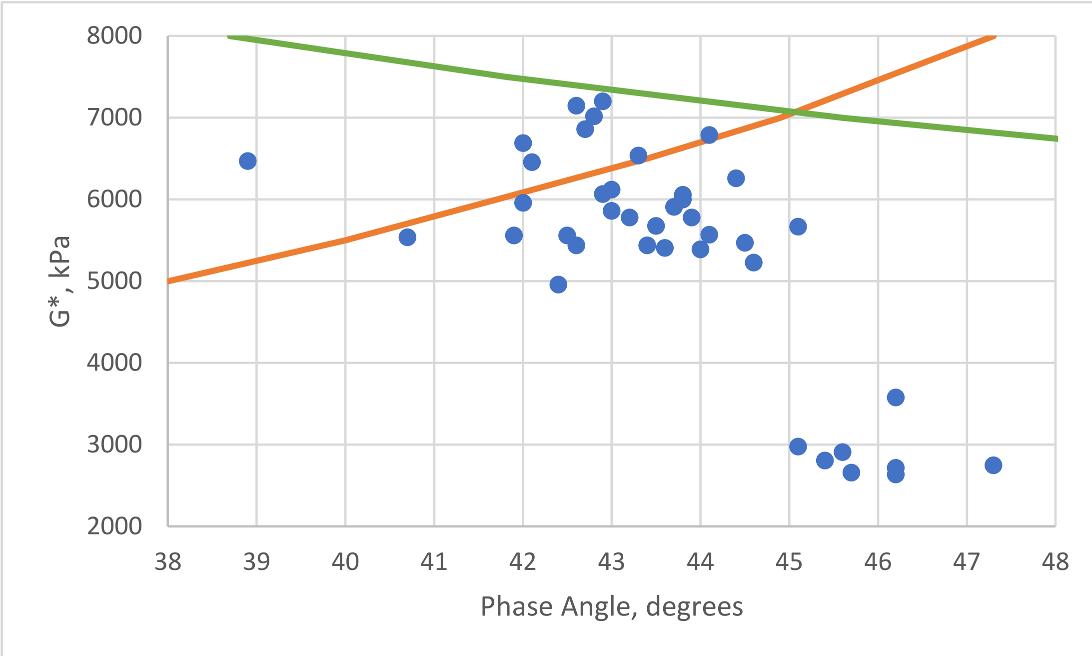


Figure 34 – Final Calculation of AASHTO M320 Intermediate PG Grade and Glover-Rowe Parameter (GRP)Tested at Identical Intermediate Temperature and 10 radians/sec (Orange Line = GRP of 5000 kPa; Green Line = Intermediate PG Grade of 5000 kPa) (After Anderson, 2023)

$$\boldsymbol{G-R=\frac{|G^*|\cdot(\cos\delta)^2}{\sin\delta}} \qquad (4)$$

The main reason the GRP is a better indicator of cracking potential than the Intermediate Temperature PG grade dissipated energy approach is that the GRP calculation penalizes an asphalt binder whose phase angle drops as the asphalt binder ages and stiffens. Whereas in the dissipated energy approach, as δ decreases, the calculated dissipated energy also decreases and stays below the 5,000 kPa threshold. An example of the calculation comparisons is shown in Figure 35. The gray column on the left is the measured phase angle while the gray rows at the top of the second and third column are the measured shear modulus (G*). As Figure 35 shows, the GRP rewards asphalt binders with high phase angle and low shear modulus. Meanwhile, the dissipated energy calculation rewards asphalt binders with a high shear modulus (G*) when the phase angle is low. Therefore, the current intermediate temperature grading system does not identify age hardened binders as well as the GRP approach.

G* (kPa)/δ (degrees)	GRP, kPa			G*sin δ, kPa		
	8000	7000	6000	8000	7000	6000
40	7303	6391	5478	5142	4500	3857
42	6603	5777	4952	5353	4684	4015
45	5657	4950	4243	5657	4950	4243
47	5088	4452	3816	5851	5119	4388
50	4315	3776	3236	6128	5362	4596

Figure 35 – Comparison of the Proposed Glover-Rowe Parameter Calculation and the AASHTO M320 Intermediate Temperature Dissipated Energy Calculation (After Anderson, 2022)

The second issue of the current intermediate PG grading approach to limit fatigue cracking is that no test method was selected to handle the second observed distress from the Zaca-Wigmore test, "…large block type cracking together with pitting and raveling…". To alleviate this, Anderson (2023) recommended the proposed approach under NCHRP 9-60 project that uses ΔTc as a means of identifying asphalt binders with durability issues and supplementing the testing with the ABCD when asphalt binders are "borderline" and may show low ΔTc values due to high elastic behavior of polymer-modified asphalt binders. Figure 36 is test data from Bennert et al. (2021) that shows a strong relationship between phase angle at constant modulus and ΔTc. It is well known the low phase angles represent elastic behavior of asphalt binders. Therefore, ΔTc is influenced by a reduction in the relaxation/phase angle of the asphalt binder due to age hardening and elastomeric modification. The impact of elastomeric modification reducing the phase angle, and therefore what appears to be a reduction in relaxation, is the main reason for the inclusion of the ABCD within the proposed, revised asphalt binder specifications. In the end, Table 7 reflects the recommendations from the NCHRP 20-44(19) research study.

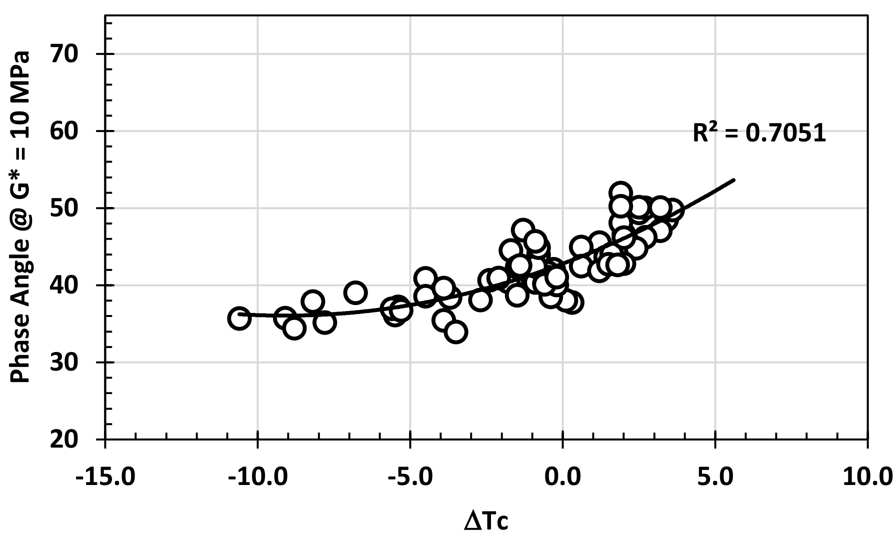


Figure 36 – Relationship Between Measured ΔTc and Phase Angle @ G* = 10 MPa

Table 7 – Proposed Testing Protocol and Criteria for Asphalt Binder Intermediate and Low Temperature Cracking (Bold Black = 20 hr PAV; Bold Red = 40 hr PAV)

PAV Aging Temperature (°C)	**100**						**100 (110)**					
Dynamic Shear, T315 $G^*(\cos\delta)^2/\sin\delta^2$, 10 rad/s, at intermediate temp [a] **< 5,000 kPa** **(< 8,000 kPa)**	29	27	25	22	19	17	29	27	25	22	19	17
Creep Stiffness, T313 Stiffness < 300 Mpa m-value > 0.300 at 60 sec & low temp	0	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	-30
Creep Stiffness, T313 R=log(2)log(S/3000)/log(1-m) at 60 sec & low temp min < R < max	**1.50 < R < 2.50** **1.50 < R < 3.20**											
ΔTc Tc,S - Tc,m	**≥ -2.0** [b] **≥ -3.0** [c]											
ΔTf [b] Tc,S - Tcr	**≥ +8.5** **≥ +4.5**											

[a] - Based on low temperature PG requirement of area (NCHRP 9-59)

[b] - Only determine ΔTf when -6 ≤ ΔTc ≤ -2.0; [c] - Only determine ΔTf when -7.0 ≤ ΔTc ≤ -3.0

WORKPLAN

A research workplan was established to evaluate ΔTc and potential alternative asphalt binder test methods and characterization procedures to improve the Wisconsin Department of Transportation (WisDOT) asphalt binder specifications targeting fatigue cracking performance. The workplan consists of a combination of asphalt binder and asphalt mixture testing to draw correlations between the performance and develop thresholds that could be implemented within a purchase specification platform.

In Phase 1, approved WisDOT asphalt mixtures were collected during plant production. The loose mix was reheated and aged to different levels in accordance with WisDOT's procedures prior to being compacted into test specimens for IDEAL-CT Index (ASTM D8225) and Overlay Tester (NJDOT B-10) test specimens. After testing was completed, the asphalt binder was recovered from the different mixtures and a variety of asphalt binder tests were used to characterize the fatigue cracking performance. During this phase, the asphalt binder was also evaluated to determine how laboratory aging procedures (i.e. – pressure aging vessel) compared to loose mix conditioning to ensure binder and mixture performance was being assessed at the same asphalt binder aged condition.

In Phase 2, using the asphalt binder test methods and their respective thresholds determined during Phase 1, tests were conducted for a variety of asphalt binders currently supplied in Wisconsin. The asphalt binders were supplied by three different asphalt liquid suppliers covering all regions of the state. The purpose of this phase was to determine if ΔTc or any of the proposed asphalt binder criteria would identify existing and approved asphalt binders as potentially poor performers. In addition, Phase 2 identified any currently approved asphalt binder as potentially having issues under any proposed asphalt binder methods and criteria.

In Phase 3, asphalt binders of similar PG grade from the three asphalt liquid suppliers were blended with recovered recycled asphalt pavement (RAP) binder at different percentages to determine the sensitivity of the different asphalt binder test methods to the inclusion of recycled asphalt binder.

In Phase 4 of the study, loose mix collected during the WisDOT Balanced Mix Design (BMD) Implementation test sections underwent solvent extraction and recovery of their respective asphalt binders. The recovered asphalt binders from the six test sections, all designed to achieve a different level of mixture performance, were evaluated under the same testing methods utilized throughout the study. Based on the asphalt binder testing, a ranking of the predicted field performance was provided. It is hopeful that while continuing to monitor the BMD test sections for their respective field performance, the collected data can also be used to validate or modify the proposed asphalt binder test methods and criteria.

To conclude the study, a recommendation was provided to WisDOT on how to improve their existing asphalt binder specifications that is aimed at providing good asphalt mixture performance.

TEST METHODS

The research study utilized both asphalt mixture and asphalt binder test methods to develop data and resultant correlations used within the analysis and final recommendations.

Asphalt Mixture Tests

<u>Overlay Tester (NJDOT B-10)</u>

The Overlay Tester, described by Zhou and Scullion (2007), has shown to provide an excellent correlation to field cracking for both composite pavements (Zhou and Scullion, 2007; Bennert et al., 2009) as well as flexible pavements (Zhou et al., 2007). Figure 37 shows a picture of the Overlay Tester used in this study. Sample preparation and test parameters used in this study followed that of NJDOT B-10, *Overlay Test for Determining Crack Resistance of HMA*. These included:

- 20 and 25°C (77°F) test temperature;
- Opening width of 0.025 inches;
- Cycle time of 10 seconds (5 seconds loading, 5 seconds unloading); and
- Specimen failure defined as 93% reduction in Initial Load.

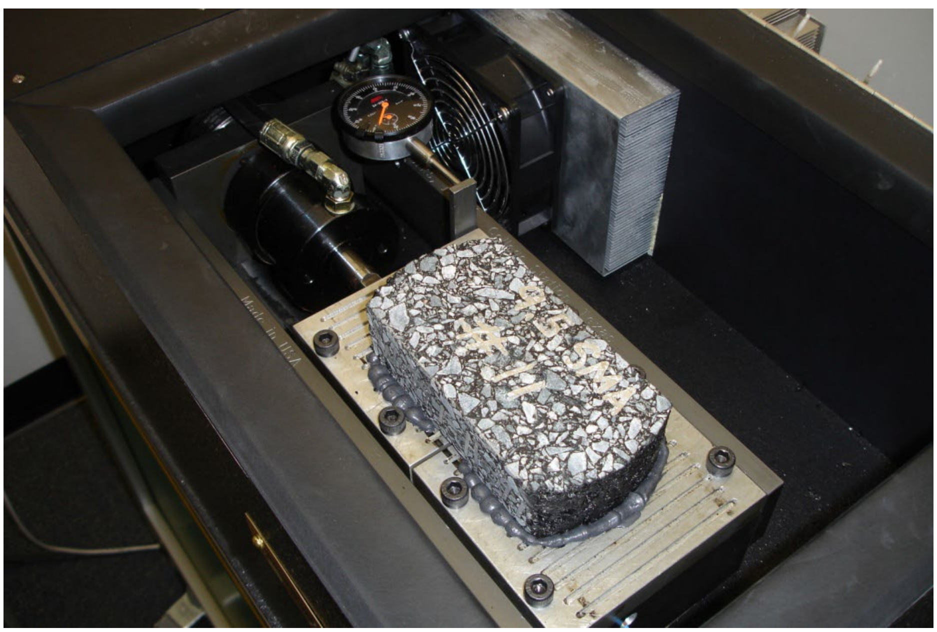


Figure 37 – Picture of the Overlay Tester (Chamber Door Open)

<u>IDEAL-CT Index Cracking Test</u>

The IDEAL-CT is similar to the traditional indirect tensile strength test, and it is run at room temperature with cylindrical specimens at a loading rate of 50 mm/min. in terms of cross-head displacement. Any size of cylindrical specimens with various diameters (100 or 150 mm) and thicknesses (38, 50, 62, 75 mm, etc.) can be tested. For mix design and laboratory QC/QA, the authors proposed to use the same specimen size as the Hamburg wheel tracking test: 150 mm diameter and 62 mm height, since agencies are familiar with molding such specimens. Either lab-molded cylindrical specimens or field cores can be directly tested with no need for instrumentation, gluing, cutting, notching, coring or any other preparation.

Figure 38 shows a typical IDEAL-CT: cylindrical specimen, test fixture, test temperature, loading rate, and the measured load vs. displacement curve.

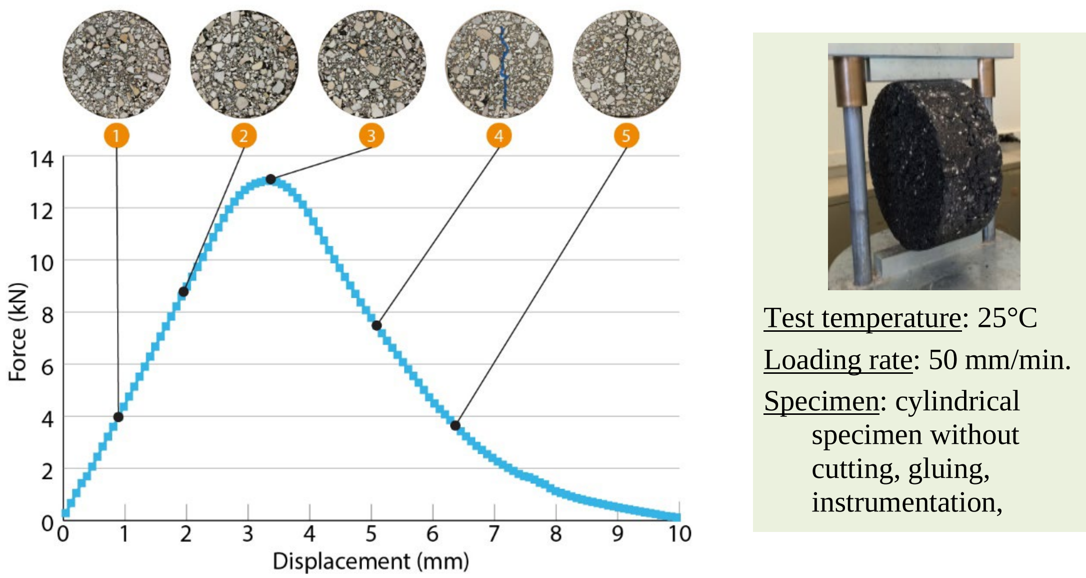


Figure 38 - IDEAL-CT: Specimen, Fixture, Test Conditions, and Typical Result

After carefully examining the typical load-displacement curve and associated specimen conditions at different stages (Figure 38), the authors chose the post-peak segment to extract cracking resistance property of asphalt mixes. Note that with the initiation and growth of the macro-crack, load bearing capacity of any asphalt mix will obviously decrease, which is the characteristic of the post-peak segment. The calculation for the cracking parameter, named CT_{Index}, is shown in Equation 5.

$$CT_{Index} = \frac{G_f}{|m_{75}|} \times \left(\frac{l_{75}}{D}\right) \quad (5)$$

$$|m_{75}| = \left|\frac{P_{85} - P_{65}}{l_{85} - l_{65}}\right| \quad (6)$$

Where,

Gf = the energy required to create a unit surface area of a crack;

|m75| = secant slope is defined between the 85 and 65 percent of the peak load point of the load-displacement curve after the peak; and

l_{75} = deformation tolerance at 75 percent maximum load.

Generally, the larger the Gf , the better the cracking resistance of asphalt mixes. The stiffer the mix, the faster the cracking growth, the faster the load reduction, the higher the $|m_{75}|$ value, and consequently the poorer the cracking resistance. It is obvious that the mix with a larger $\frac{l_{75}}{D}$ and better *strain* tolerance has a higher cracking resistance than the mix with a smaller $\frac{l_{75}}{D}$.

Asphalt Binder Characterization

The asphalt binder from conditioned test specimens were extracted and recovered in accordance with AASHTO T164, *Procedure for Asphalt Extraction and Recovery Process* and ASTM D5404, *Standard Practice for Recovery of Asphalt from Solution Using the Rotary Evaporator* (Figure 39). After the recovery process, the asphalt binder was evaluated for a variety of asphalt binder parameters that have been identified in literature as having a strong relationship to cracking performance. These will be discussed in further detail below. In addition, the recovered asphalt binders were tested for the respective Multiple Stress Creep Recovery (MSCR) properties at 52, 58, and 64°C in accordance with AASHTO T350, *Standard Method of Test for Multiple Stress Creep Recovery (MSCR) Test of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)*. This allowed for an assessment of whether or not the asphalt binder was polymer modified, and if so, the general magnitude of modification. The magnitude of modification was defined as the Z-factor, which is the relative difference between the AASHTO M332 recommended elastomer line and the respective asphalt binder at the measure non-recoverable creep compliance, Jnr, value, as illustrated in Figure 40.

Figure 39 - Asphalt Binder Recovery Equipment at Rutgers University

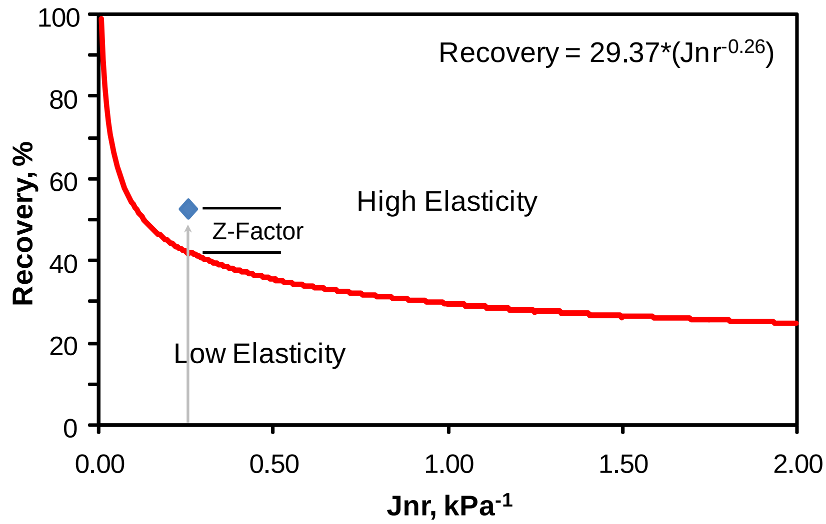


Figure 40 – Schematic Illustration of Z-Factor (Magnitude Polymer Modification)

<u>Asphalt Binder Test Methods</u>

The following describes the asphalt binder test methods utilized in the study to evaluate recovered and sampled asphalt binders.

Bending Beam Rheometer, BBR, Low Temperature Performance

Low temperature PG grading using the Bending Beam Rheometer (BBR) was conducted to ultimately determine the low temperature PG grade. However, a more detailed review of the low temperature grades predicted by the m-slope and Stiffness (S) provides insight as to the general level of oxidative-related aging that has occurred in the asphalt binder. Anderson et al. (2011) identified this difference as a means of indexing the non-load associated cracking potential of asphalt binders and defined it as follows:

$$\Delta T_{cr} = T_{cr\ (Stiffness)} - T_{cr\ (m-slope)} \tag{7}$$

where,

ΔT_{cr} = Difference in critical low temperature PG grade
T_{cr} = Critical low temperature grade predicted using the BBR m-slope
T_{cr} = Critical low temperature grade predicted using the BBR Stiffness (S)

In Equation (7), as the ΔT_{cr} decreases and becomes negative, the asphalt binder is considered to be more prone to non-load associated cracking. Initially, Anderson et al., (2011) set a limit of $\Delta T_{cr} \leq -2.5^{o}C$ for when there is an identifiable risk of cracking and preventative action should be considered. Rowe (2011) further advanced this methodology, eventually developing a new asphalt binder fatigue property termed Glover-Rowe parameter, which will be discussed later, but recommended that at a $\Delta T_{cr} \leq -5^{o}C$ immediate remediation should be considered.

Double Edge Notched Tension (DENT) Test

The Double Edge Notched Tension (DENT) test has also been proposed for characterizing binder fatigue fracture resistance. The DENT test is a monotonic fracture test, similar to the direct tension test (DTT) used in the Superpave PG system with the exception that notches are imposed on the specimen. The test can be conducted in a standard force-ductility instrument, such as that used for the DTT test. The DENT test was developed by Queen's University in Canada (Andriescu et al. 2004) and modified and adapted for intermediate temperature testing by the FHWA (Gibson et al. 2011). The DENT test is formalized in specifications in Ontario, Canada (Ontario Ministry of Transportation Test Method LS-299).

The Double-Edged Notch Tension (DENT) was conducted in accordance with AASHTO TP113, *Determination of Asphalt Binder Resistance to Ductile Failure Using Double-Edge-Notched Tension (DENT) Test*. The DENT test utilizes the concept of fracture mechanics to evaluate the ductility of asphalt binders. The test procedure is based on measuring the energy needed for fracturing ductile materials consists of two parts; an essential portion of work performed in a local region of the advancing crack creating two surfaces and a non-essential work away from the local region of cracking/tearing associated with ductility, plasticity and yielding. To determine the essential work of fracture and critical tip opening displacement (CTOD), the DENT test is performed using similar specimens with different ligament lengths (5, 10, and 15mm). Figure 41 shows a schematic of a typical test specimen showing the notch in the middle of the test specimen, resulting in a "ligament" length (Figure 42a). The test specimens are then pulled using a force-ductility instrument (Figure 42b) and the Force and Displacement is measured (Figure 43). The area under the curve is measured for each ligament length allowing for the determination of the Essential and Non-essential Work. The CTOD is also determined, which has been found to be a good indicator of fatigue resistance. Larger CTOD values indicates better fatigue resistance.

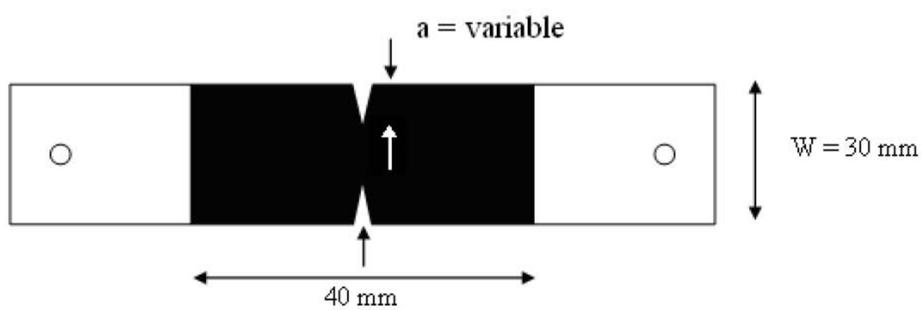
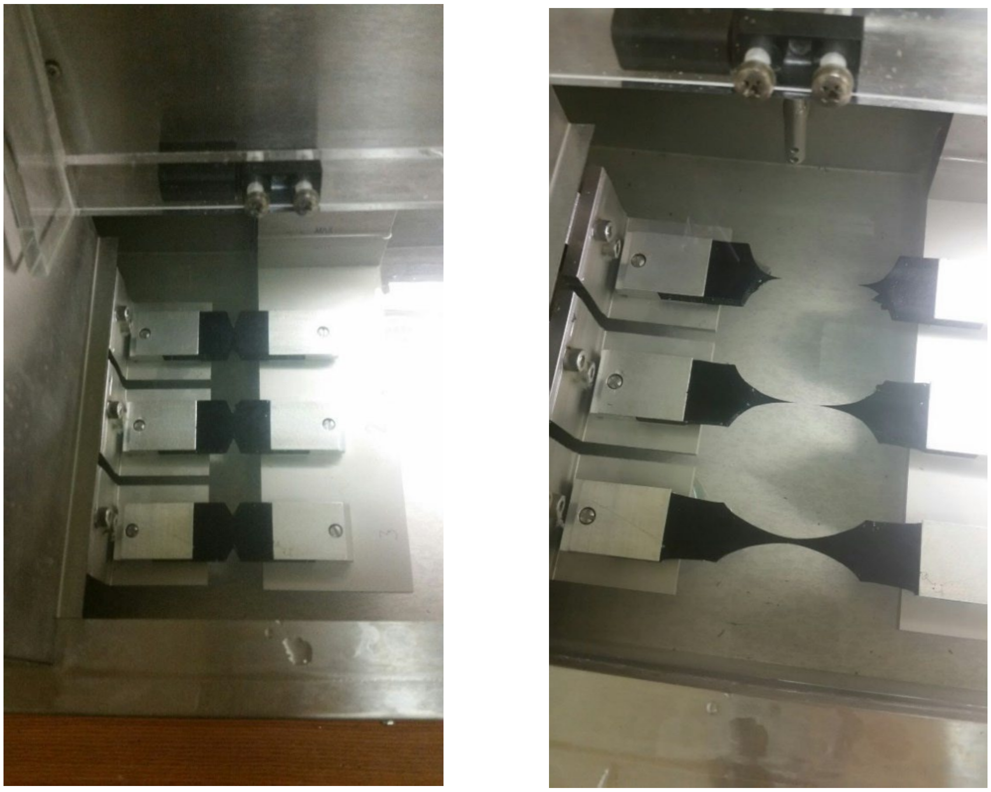


Figure 41 – Double Edged Notched Tension (DENT) Test Specimen



(a) (b)

Figure 42 – DENT Test Specimens; (a) Just Before Starting the Test, (b) Test Specimens of Different Ligament Lengths Failing

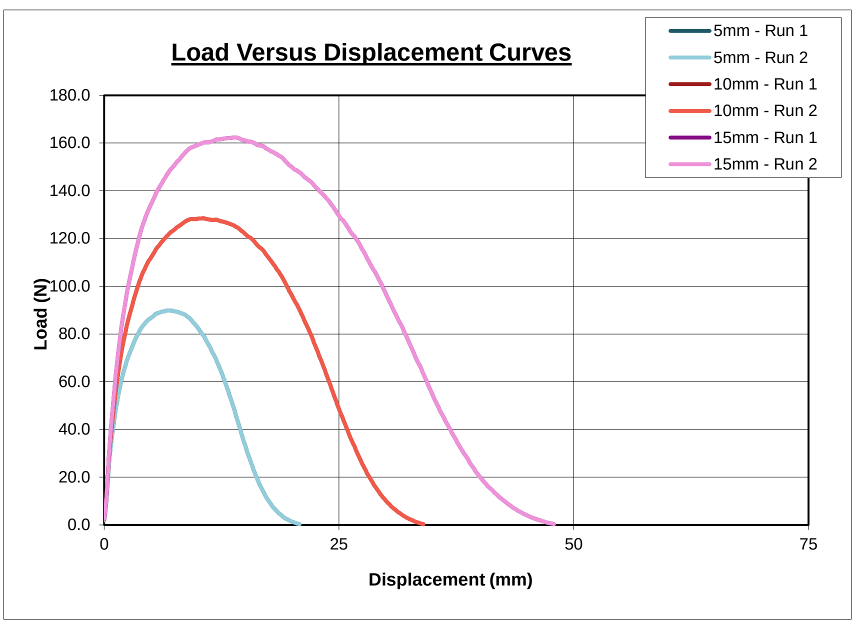


Figure 43 – Example of Load vs Displacement Curves for DENT Test with Different Ligament Lengths

Rheological Indices Related to Brittleness and Durability

To evaluate the various rheological indices in the study, each asphalt binder was tested for their respective master stiffness properties. Both G* and phase angle (δ) was determined at a various test temperature and loading frequencies. As there are currently no standardized procedures in either AASHTO or ASTM for determining the master stiffness properties of asphalt binders, the general procedure used follows that recommended by Rowe (2015) and summarized below;

Run the DSR in the oscillatory mode, within the strain range 0.005 to 0.02 (± 5%) ensuring that the test specimen will be tested over the linear region over the temperature range chosen. The typical range of stiffness being captured in a frequency sweep measurement will be 10 Pa to 10 MPa.

NOTE: Linearity check - This is most conveniently carried out by a torque sweep at both the highest and lowest test temperature to be used for the rheological characterization. For the majority of binders, it has been found that testing within the strain range 0.005 to 0.02 lies within the linear range. However, for PMBs, the linear range may be much less. The linear range available depends upon the stiffness of a binder at the condition being evaluated.

It is recommended to use a strain value of 1% when G is below 1e5 Pa and 2% when G* is above 1e5 Pa. This stiffness has been found to be a convenient for switching plate size with the DSR.*

The value of 1e5 Pa should lie in two isotherms since the value of G is frequency dependent. Ideally, the 1e5 value should be measured with both plate diameters. The majority of the data with a G* below 1e5 should be collected with a 25mm plate size whereas the majority of the data generated with a G* greater than 1e5 Pa should be collected with an 8mm plate.*

Select the test temperatures appropriate to the binder being tested, to define the stiffness in the desired range but including test temperatures of 95, 80, 70, 60, 45, 35, 25, 15 and 5°C. Equilibrate the test specimen before testing.

NOTE: Caution should be taken when testing at the lower test temperatures that the measured shear modulus values are not being affected by possible machine/geometry compliance, or by the test specimen de-bonding from the plates. Also, it may not always be possible to test at the high end of the range since materials will be too fluid.

The recommended range of frequencies (radians per second) for use in the frequency sweep testing is shown in Table 8. The idea of utilizing the selected frequencies shown in Table 8 is that the range covers two decades of loading times, providing five data points per log decade of frequency tested.

Table 8 – Recommended Range of Frequencies for DSR Frequency Sweep Testing

Log Basis (radians/second)	Linear Basis (radians/second)
-1.0	0.100
-0.8	0.159
-0.6	0.251
-0.4	0.398
-0.2	0.631
0.0	1.00
0.2	1.59
0.4	2.51
0.6	3.98
0.8	6.31
1.0	10.0

The data initially should be inspected for quality by plotting the results of G* and phase angle. The objective of this plot is to enable gross errors in the data to be spotted. Some typical examples are shown in Figures 44 and 45. It should be noted that smooth curves may not always exist due to transitions that may occur in materials. However, most asphalt binders when tested in the linear range, without modifiers, generally have a smooth relationship in this plot. Curves as shown in the second figure are generally associated with lower quality testing or utilizing too fast of a loading frequency during testing.

To expedite the testing time, the master curves were abbreviated for the test temperature and frequency range of interest in the study. Therefore, only the test temperatures of 10, 20 and 30^{o}C were utilized for this work.

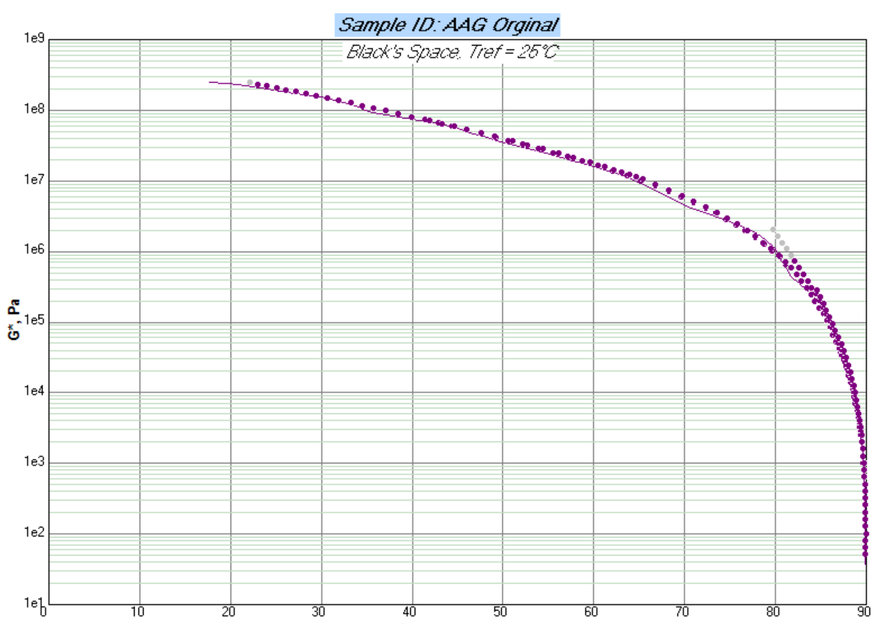


Figure 44 - Example of Acceptable Isotherm Quality from DSR Master Curve Testing

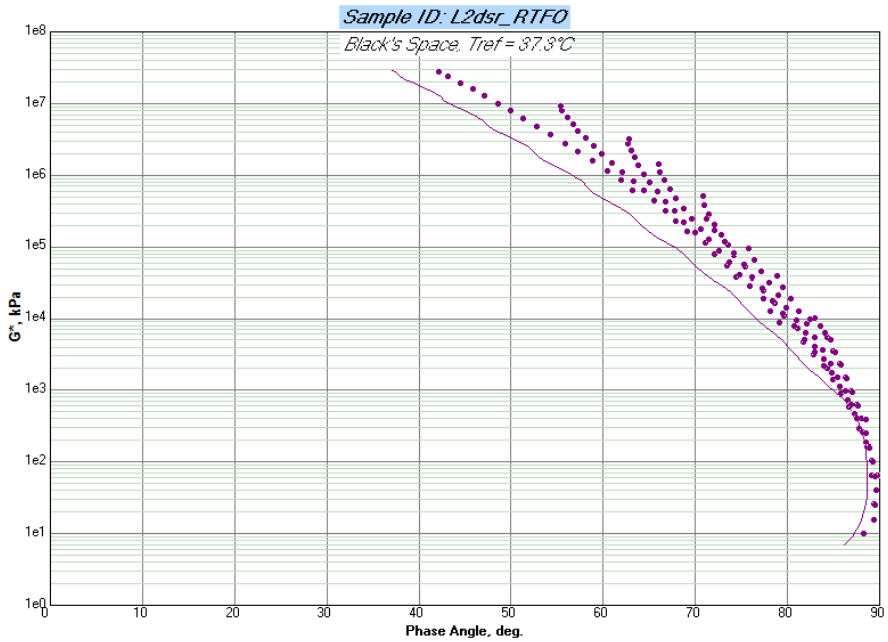


Figure 45 - Lower Quality Data with Isotherms Trending Upwards as Frequency Increases Suggesting Some Compliance Issues

Researchers have demonstrated that several rheological indices can be derived that provide indicators of brittleness and can be easily measured using the DSR. These parameters have been primarily proposed for thermally induced cracking and surface raveling but also have promise for identifying asphalt binders susceptible to fatigue cracking as a result of oxidation induced embrittlement. Glover et al. (2005) proposed the rheological parameter, $G'/(\eta'/G')$, as an indicator of ductility based on a derivation of a mechanical analog to represent the ductility test consisting of springs and dashpots. It has been well demonstrated that the Glover parameter is directly correlated to measured ductility. The Glover parameter can be calculated based on DSR frequency sweep testing results, making it much more practical than directly measuring ductility using traditional methods. Rowe (2011) re-defined the Glover parameter in terms of $|G^*|$ and δ based on analysis of a black space diagram as shown in Equation (8) and suggested use of the parameter $|G^*|\cdot(\cos\delta)^2/\sin\delta$, termed the Glover-Rowe (G-R) parameter in place of the original Glover parameter.

$$\boldsymbol{G-R} = \frac{|G^*|\cdot(\cos\delta)^2}{\sin\delta} \qquad (8)$$

Rowe proposed measuring the G-R parameter based on construction of a master curve from frequency sweep testing at 10°C, 20°C, and 30°C in the DSR and interpolating to find the value of G-R at 15°C and 0.005 rad/sec to assess binder brittleness (Rowe et al. 2014). A higher G-R value indicates increased brittleness. It has been proposed that a G-R parameter value of 180 kPa corresponds to damage onset whereas a G-R value exceeding 600 kPa corresponds to significant cracking based on a study relating binder ductility to field block cracking and surface raveling by Anderson et al. (2011). Applying time-temperature superposition to the test temperature and loading frequency, the 15°C at 0.005 rad/s equals 44.7°C at 10 rad/s. The test temperature of 44.7°C is generally considered too warm to be considered an "intermediate temperature" for fatigue cracking evaluation. However, to correlate to ductility, which is what the research by Glover was originally based on, it is probably somewhat valid.

In addition to the Glover-Rowe parameter, a number of rheological properties that describe the shape and location of the master curve in Black Space were determined and compared to the mixture cracking performance. The shape parameters are determined for the high stiffness area of the master curve. In the Christensen and Anderson (1992) paper it is recommended that values associated with the CA model should be obtained only when the binder stiffness is greater than 1e5Pa. The ΔTc also defines the shape in this higher stiffness region since it is determined from BBR data. At lower stiffness, changes in the behavior result in many shape parameters being more variable and less reliable. Furthermore, modification by polymers, rubber, plastics, etc. all make the higher temperature/lower stiffness more complex and reduce the applicability of using this data in a shape parameter determination. Consequently, researchers often exclude this lower stiffness data from calculations used to determine asphalt binder shape parameters, hence the need for an abbreviated master curve regime.

Figure 46 provides an idealized schematic of an asphalt binder master curve for the complex shear modulus (G*). The shape of the master curve provides an indication of the structure of asphalt binder. Generally, a flatter master curve will be associated with greater oxidation, greater structure (tend to be considered more as a GEL binder versus a SOL binder), lower temperature susceptibility, etc. A GEL binder exhibits a liquid phase that is dispersed in a solid medium, similar in appearance to a jelly-like consistency. Meanwhile, a SOL binder is the opposite of GEL, where there are solid particles randomly dispersed in the liquid medium. Parameters that will be evaluated in the study that pertain to the shape of the master curve, and hence, the unique properties of the asphalt binder are (Christensen and Anderson, 1992):

Glassy Modulus (Gg): where the shear modulus approaches a constant value at low temperatures and high frequencies; often assumed to be 1 GPa (1E9 Pa) and where the phase angle (δ) equals zero

Crossover Modulus (Gc): the modulus value where the asphalt binder transitions from elastic to viscous response; essentially the modulus value where the storage modulus (G') and loss modulus (G") are equal

Crossover Frequency (ωo): the frequency at which the storage modulus (G') and loss modulus (G") are equal; where the measured phase angle (δ) equals 45 degrees and is often considered as a hardness parameter

R-value (Rheological Index): defined as the difference between the log glassy modulus (Gg) and the log shear modulus (G*) at the crossover frequency; considered an indication of rheologic type. R-value increases as asphalt binder stiffness increases

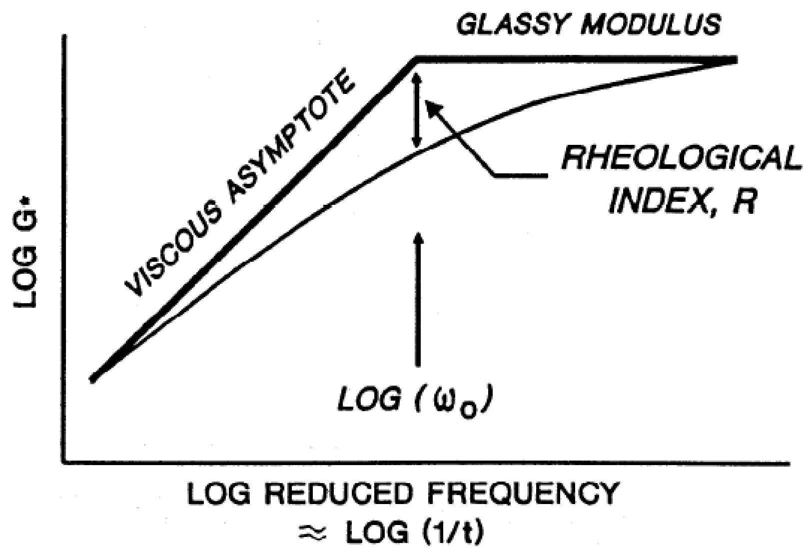


Figure 46 – Idealized Schematic of Master Stiffness Curve for Asphalt Binders (After Christensen and Anderson, 1992)

The R-value can be mathematically determined using data generated during DSR rheological testing. The form of the equation is shown as Equation 9.

$$R = \frac{(\log 2)\times log\left(\frac{G^*(\omega)}{G_g}\right)}{log\left(1-\left(\frac{\delta(\omega)}{90}\right)\right)} \qquad (9)$$

Where,

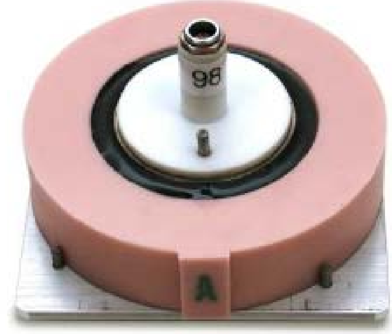
R = rheological index
G*(ω) = complex shear modulus at loading frequency (ω)
Gg = Glassy Modulus, assumed to be 1E9 Pa
δ(ω) = phase angle at loading frequency (ω)

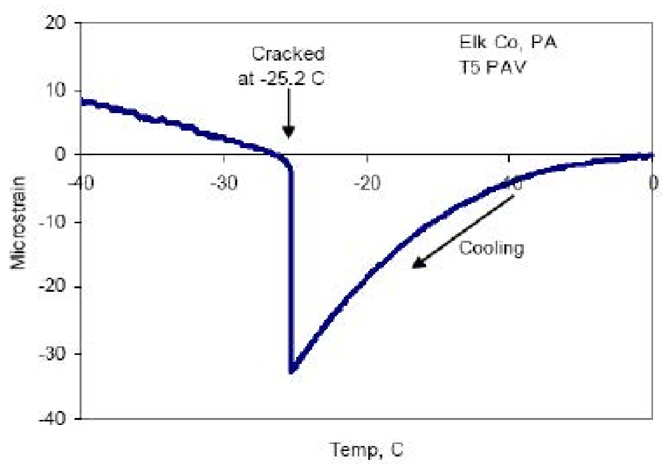
It should noted that the R-value determination is dependent on two parameters – the Gg (glassy modulus asymptote) and the cross-over modulus (Gc) by the log of the two parameters, R-value = log Gg - log Gc.

Since Gc and R-value are closely related, then by default the value of Gc will also be strongly related to the values of ΔTc. Thus, while the research study was aimed at considering the implementation of ΔTc, the value of log Gc could also provide essentially similar information. Furthermore, an additional parameter, the phase angle at a fixed value of G* has been discussed as another alternate method of relating to the shape of the mastercurve in the same manner as R-value and log Gc. Again, like Gc, this is an easy parameter to measure in the DSR in the region of 8.9 to 10 MPa, which has currently been used by researchers. Recent discussion at an Asphalt

Institute meeting (Rowe and Kriz, 2021), it was agreed that this value of phase angle should be measured at 10 MPa. This value is coincidentally similar to the approach presented by Bennert et al. (2023) in the study of asphalt binders for the FAA (using the tangent of the phase angle at 10 MPa). Consequently, the research study evaluated the value of Gc and δ_{10MPa} (phase angle at 10 MPa).

The ABCD is similar in concept to the asphalt mixture test Thermal Stress Restrained Specimen Test (TSRST). The ABCD measures the temperature and strain of a restrained asphalt binder ring subjected to a constant rate cooling. Asphalt binder samples are heated and poured outside of an Invar ring placed in the center of a silicone mold (Figure 47a). The Invar ring includes a strain gauge to record the strain applied to it by contraction of the asphalt binder during cooling and a surface-mounted resistance temperature detector (RTD) to record the temperature of the sample. The ABCD cools an asphalt binder specimen at a rate of 20°C per hour until the asphalt binder specimen "cracks" due to thermal contraction. The crack is determined to occur when a "jump" in the measured strain occurs (Figure 47b). The binder temperature measured when the strain jump occurs is defined as the ABCD critical cracking temperature (T_f).





(a) (b)

Figure 47 – Asphalt Binder Cracking Device (ABCD) Specimen and Data Collection

The ABCD T_f is primarily controlled by the coefficient of thermal contraction (CTC) of the asphalt binder. The CTC controls the rate of volumetric change in the asphalt binder, thereby controlling the rate of thermal stress development. An asphalt binder with a higher CTC may be subjected to larger strains compared to low CTC asphalt binders before cracking failure is observed. Work conducted under NCHRP Project 9-60 showed that the asphalt binder's CTC affects non-load related cracking.

Research from NCHRP Project 9-60 has shown that the results of the ABCD alone may not provide a clear enough picture of an asphalt binder's performance. Figure 48 below shows the ABCD critical cracking temperature (T_{cr}) for four asphalt binders from MNRoad. The results of the ABCD T_{cr} show good comparison to the field results, except for asphalt binder MN 1-4. The MNRoad data, along with other lab data, convinced the NCHRP Project 9-60 researchers that the

ABCD T_{cr} alone cannot adequately rank all asphalt binders. It should be noted that asphalt binder MN 1-4 was modified using re-refined engine oil bottoms (REOB).

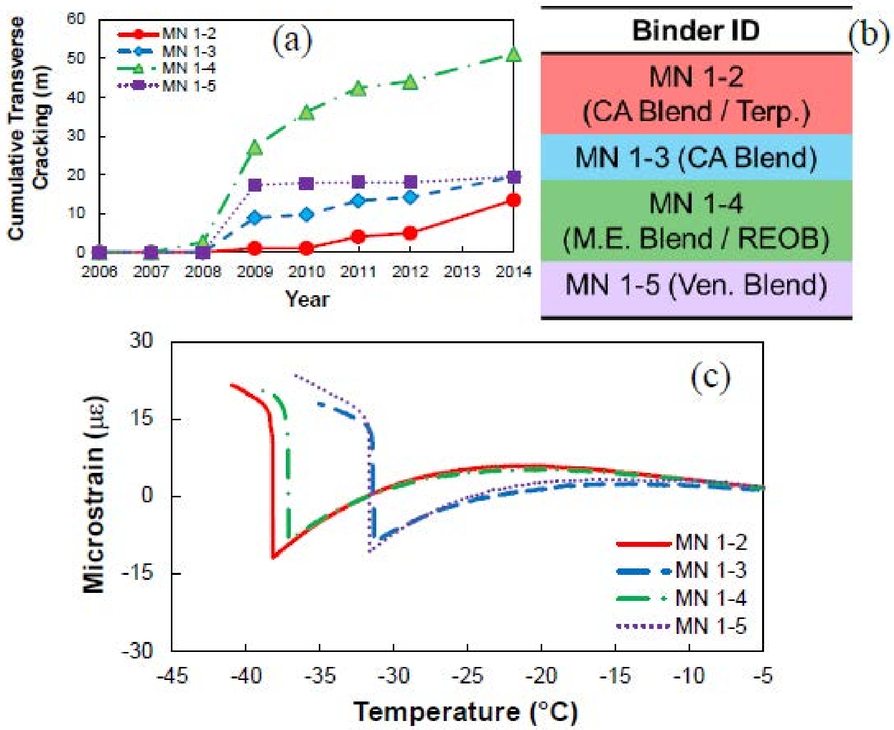


Figure 48 – ABCD T_{cr} Cracking Temperature Compared to Field Cracking Performance (Elwardany et al., 2019)

To better interpret the ABCD data, Elwardany et al., (2019) recommended normalizing the ABCD T_{cr} using the constant stiffness temperature from the bending beam rheometer (BBR) test (Tc = 300 MPa). This was done for two reasons;

1. The T_C (S = 300 MPa) is highly correlated with the glass transition temperature, T_g. Modifiers like REOB will actually help to reduce the T_g. Therefore, normalizing the ABCD T_{cr} using T_C (S = 300 MPa) should improve the sensitivity of the ABCD cracking data to REOB-type modification that reduces the T_g of the asphalt binder.
2. Since the T_C (S = 300 MPa) is already a part of the PG grading system, it is simpler to use than the actual T_g value, which would require sophisticated measurement equipment outside of conventional asphalt binder test equipment.

Therefore, the NCHRP 9-60 researchers proposed a new parameter called, ΔT_f, described below.

$$\Delta T_f = T_C(S) - T_{cr} \quad (10)$$

where,

$T_C(S)$ = low temperature PG grade from BBR Stiffness

T_{cr} = ABCD low temperature critical cracking temperature

Understanding that ΔT_C or ΔT_f were effective at determining some critical characteristics of "poor" asphalt binders yet alone were still not sensitive enough to always separate good vs poor performance, the NCHRP researchers developed a new approach combining the parameters in a performance space. Figure 49 shows the analysis of a number of asphalt binders evaluated during NCHRP Project 9-60. It is not known how the performance level noted in the figure was derived, although no mixture fatigue cracking testing was conducted in the study.

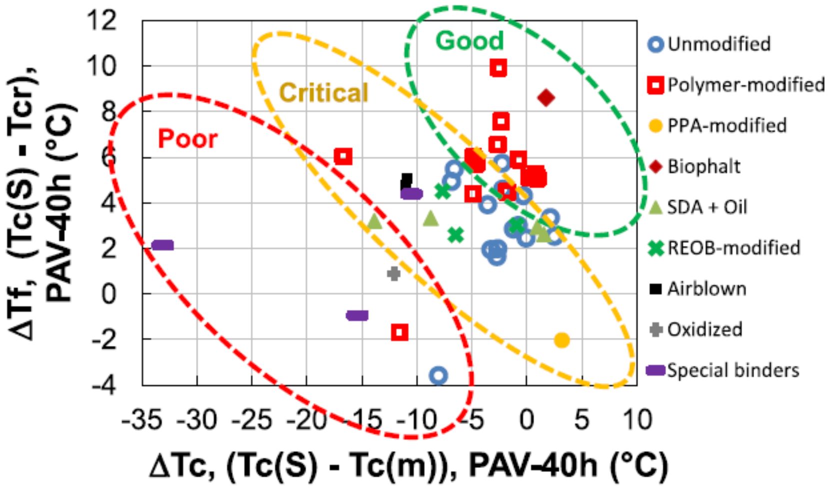


Figure 49 – Measured Performance of Asphalt Binders from NCHRP Project 9-60 (Elwardany et al., 2019)

The advantage of combining the ΔT_C and ΔT_f measurements is in their separate abilities to capture differences in performance. For example, the addition of SBS polymer is known to improve the fatigue performance of asphalt, yet the ΔT_C parameter shows that too much polymer can actually be classified as a poor binder (Figure 50). However, ΔT_C can clearly pick up issues with age hardening additives such as REOB. Meanwhile, ΔT_f can clearly pick up the advantages of the improved strain tolerance of additional polymers but may have difficulty picking up the detrimental impact of REOB-type additives that can reduce the glass transition temperature (T_g).

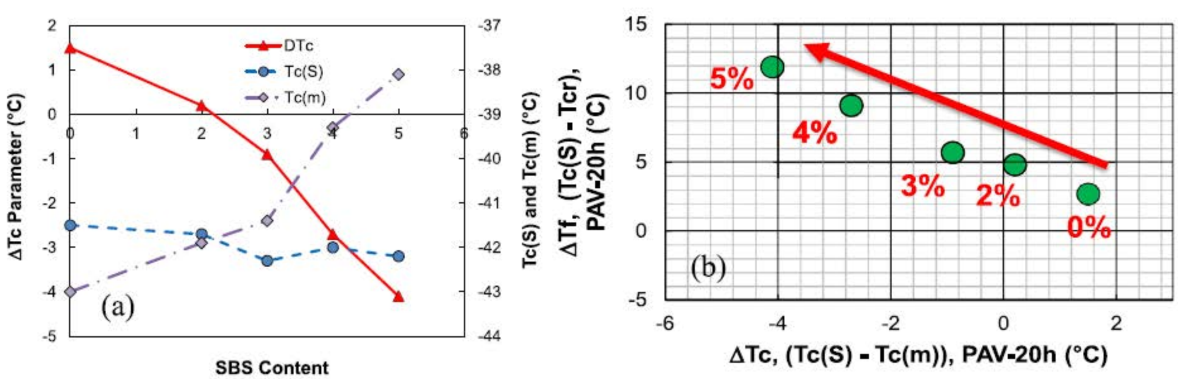


Figure 50 – Impact of %SBS on Low Temperature Performance (Elwardany et al., 2019)

As per Elwardany et al. (2019);

> "Although stiffness and relaxation are related to cracking resistance of binders, these parameters alone do not fully characterize cracking resistance. Other relevant factors relate to thermal contraction, strain tolerance, fracture toughness, crack initiation and propagation, and fatigue resistance. Any measurements in the LVE range, such as stiffness (modulus) and relaxation, without inducing damage to the binder sample, cannot be used to fundamentally and rigorously predict failure properties and strain tolerance for complex and modified binders. The proposed binder failure index, called ΔT_f (defined as the difference between $T_C(S)$ from BBR and T_{cr} from ABCD) rank asphalt binders effectively in terms of failure strength at low temperature and gives credit to well-formulated and compatible modifiers that may increase binder strength and strain tolerance, such as high-quality polymer-modified binders (PMA's) with elastomers in particular. ΔT_C, a BBR rheological parameter measured in the LVE range, can be combined with ΔT_f, a failure index from the BBR and ABCD cracking test, to predict age-induced surface cracking."

In the end, the NCHRP 9-60 researchers proposed recommendation the performance space in Figure 51 as a preliminary means of identifying good vs poor performing asphalt binders regarding their durability/fatigue performance.

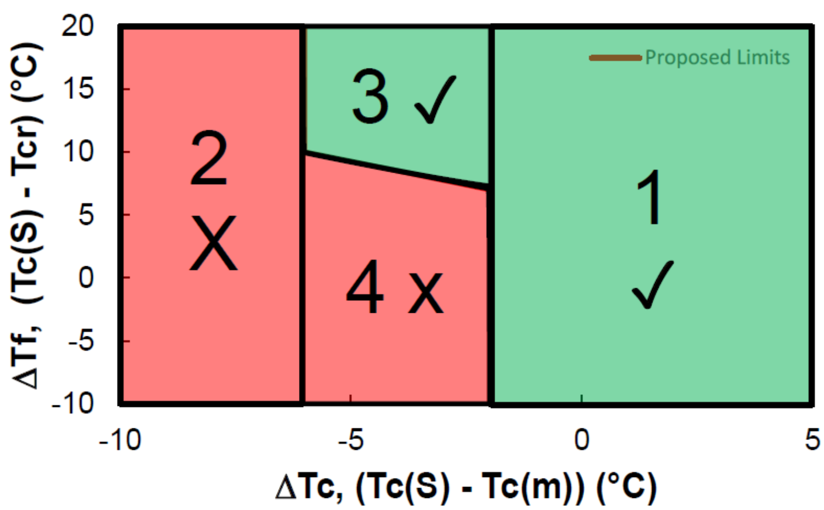


Figure 51 – Proposed Specification Parameters for NCHRP 9-60 Approach (Elwardany et al., 2019)

PHASE 1 – ASPHALT BINDER TO MIXTURE PERFORMANCE

During the 2023 summer paving season, the Wisconsin Highway Research Program (WHRP) organized the collection and shipping of six (6) different asphalt mixtures to the Rutgers Asphalt Pavement Laboratory (RAPL) for characterization. The characterization combined asphalt mixture testing with the characterization of the recovered asphalt binder from the same tested mixture specimens. This was an attempt to compare identically aged/conditioned asphalt mixture and binder specimens. The detailed testing plan is summarized below;

1. Reheat and condition the different approved WisDOT asphalt mixtures according to WisDOT specifications. To provide a wide range of asphalt mixture and binder performance, the loose mix was conditioned at three different levels;
   a. "Short-term" conditioning that consisted of reheating the loose mix for 2 hours at the compaction temperature of the respective asphalt binder;
   b. "Short-term" + 6 hours of additional loose mix conditioning at 135°C – this is the current WisDOT procedure for "Long-term" conditioning; and
   c. "Short-term" + 10 hours of additional loose mix conditioning at 135°C.
2. Compact test specimens to 7% +/- 0.5% air voids for fatigue cracking evaluation. The test specimens were evaluated under the IDEAL-CT Index test (ASTM D8225) and the Overlay Tester (NJDOT B-10) at 20 and 25°C, respectively. The two test methods were selected for their significant difference in specimen loading conditions (i.e. – montonic vs. cyclic). The two test temperatures were selected to compare the current test temperature (25°C) to a test temperature that better represents the intermediate test temperature of United States regional climate of Wisconsin (20°C).
3. The tested IDEAL-CT Index test specimens were used for the asphalt binder recovery and resultant asphalt binder characterization. The asphalt binders were tested at each of the loose mix conditions so a direct comparison could be made between the asphalt mixture and asphalt binder fatigue cracking tests. In addition, the "Short-term" conditioned specimens were also conditioned in the laboratory at 20 hr and 40 hr pressure aging vessel (PAV) conditioning. This was conducted to establish a comparison between the loose mix conditioning and the conventional PAV asphalt binder aging. With respect to a state agency specification protocol, it is important that if an asphalt mixture aging protocol is established for mixture testing, then the same level of aging should be included with the asphalt binder specification. Otherwise, it makes it difficult, and technically not appropriate, to correlate the asphalt binder purchase specification parameter to the asphalt mixture fatigue cracking acceptance parameter.

1.1 - Asphalt Mixture Testing

Two different asphalt mixture test methods were utilized in the study – the IDEAL-CT Index (ASTM D8225) and the Overlay Tester test (NJDOT B-10). The IDEAL-CT Index is a monotonic test that is considered a mixed-mode of crack initiation and crack propagation. The test method is conducted at a loading rate of 50 mm/min and testing time is often completed within one minute. Meanwhile, the Overlay Tester is a cyclic test where a deformation control is applied using a triangular waveform of 5 seconds tensile (straining) loading and 5 seconds of compressive (relaxation) loading. Unlike the IDEAL-CT test which has a finite testing time due to the nature of the loading, the Overlay Tester could potentially run for days depending on the nature of the asphalt materials and how well it resists the propagation of the crack from the bottom of the test specimen to the top. Due to the nature of the loading in each test, the Overlay Tester specimens have the ability to “heal” to a certain extent during the compression portion of the loading while the loading mechanism of the IDEAL-CT test does not allow for any “healing” to take place. Both test methods are described in further detail below.

The IDEAL-CT Index and Overlay Tester were used to characterize the fatigue cracking properties of the six different asphalt mixtures at three different laboratory aged conditions. All test specimens were tested at a test temperature of 25^{o}C, while a test temperature of 20^{o}C was also included at both the STOA and STOA + 10 Hrs Conditioning at 135^{o}C.

1.1.1 - IDEAL-CT Cracking Index

Figures 52 to 54 summarize the IDEAL-CT Cracking Index results for the asphalt mixture evaluated in Phase 1 of the study. The data represents the average of three specimens with the error bars noting the standard deviation +/- the average value. It is immediately noticeable that there appears to be little to no difference in the measured IDEAL-CT Cracking Index when comparing the results at 20^{o}C vs 25^{o}C. In fact, a quick comparison shows that is approximately 10% decrease in IDEAL-CT Index value when decreasing the test temperature 5^{o}C (from 25 to 20^{o}C). This is well within the precision and bias of the test procedure.

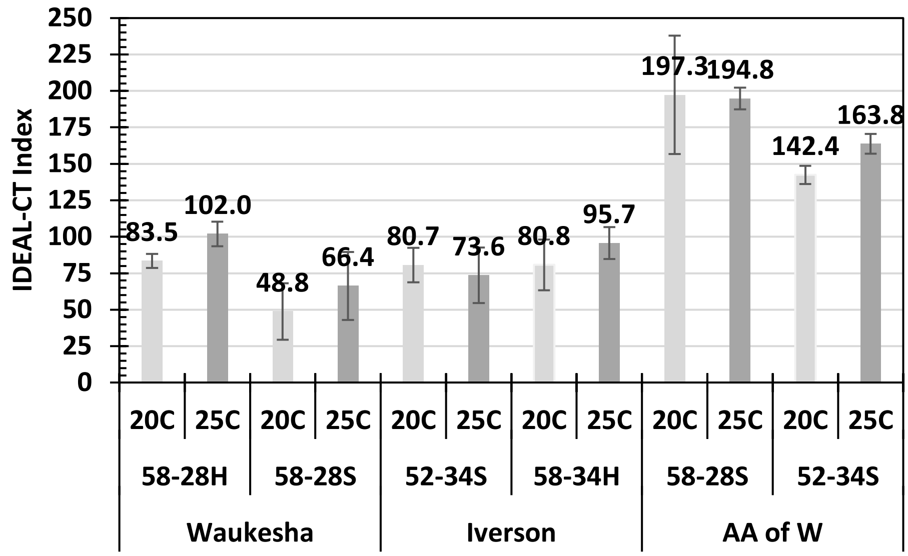


Figure 52 – IDEAL-CT Cracking Index at 2 Hrs STOA Conditioning

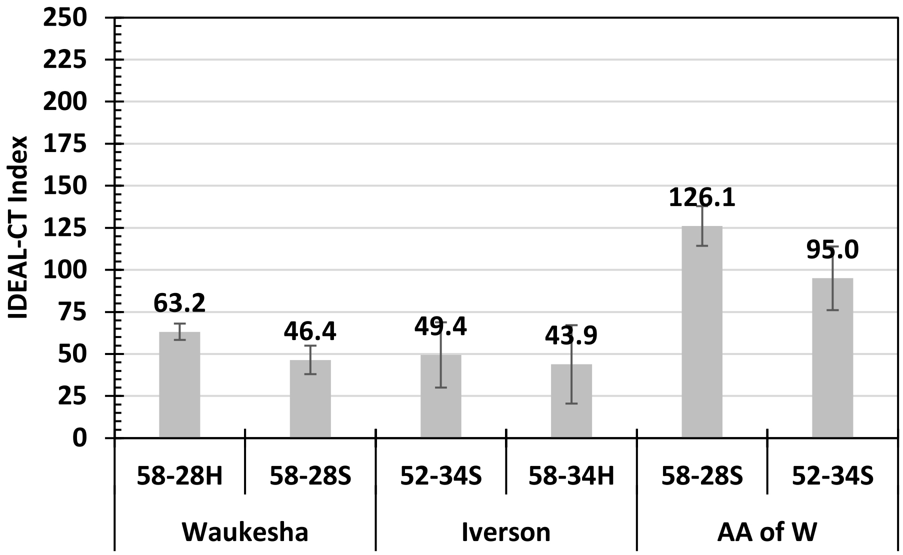


Figure 53 – IDEAL-CT Cracking Index at STOA + 6 Hrs Conditioning at 135ºC

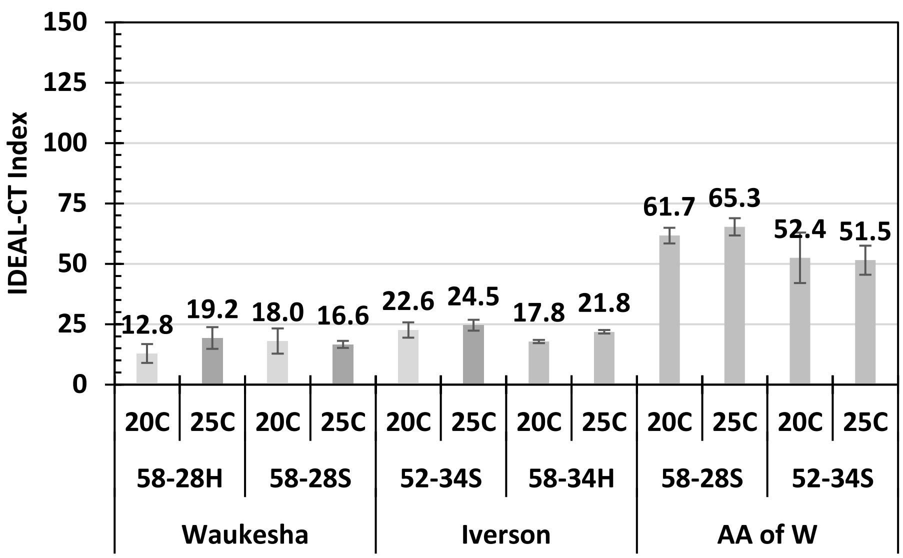


Figure 54 – IDEAL-CT Cracking Index at STOA + 10 Hrs Conditioning at 135ºC

The mixture designs and/or production reports were not provided for each of the asphalt mixtures evaluated in the study. This made it impractical to try to compare volumetrics and recycled asphalt contents to mixture performance.

The impact of extended mixture conditioning compared to the STOA condition is shown in Figure 55 and 56 for the 25ºC test temperature. Only the test data for the 25ºC is shown as all three conditioning levels were tested at this condition. The results in Figure 55 clearly indicate that as conditioning level increased, the resultant IDEAL-CT Index decreased. When compared to the STOA conditioning, which is simply reheating the loose mix for 2 hours at compaction temperature, the reduction in IDEAL-CT Index values is rather consistent. As observed in Figure

56, a strongly, correlated linear relationship between the different aged conditions and the STOA condition exists. This would suggest that the inclusion of an aging requirement for the testing of asphalt mixtures within Balanced Mix Design (BMD) may not be needed. Similar conclusions were also found by Bonaquist (2016) in a previous WisDOT study.

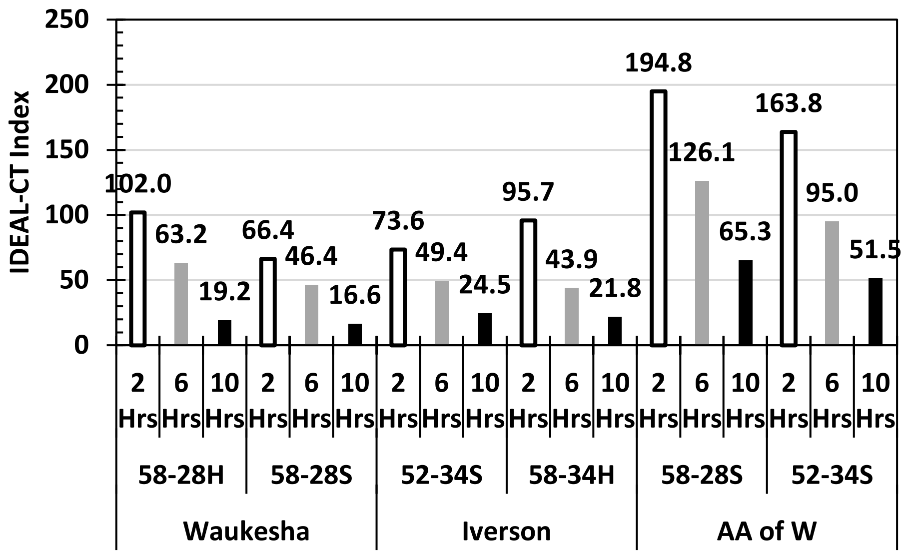


Figure 55 – Measured IDEAL-CT Cracking Index at 25ºC for All Three Conditioning Levels

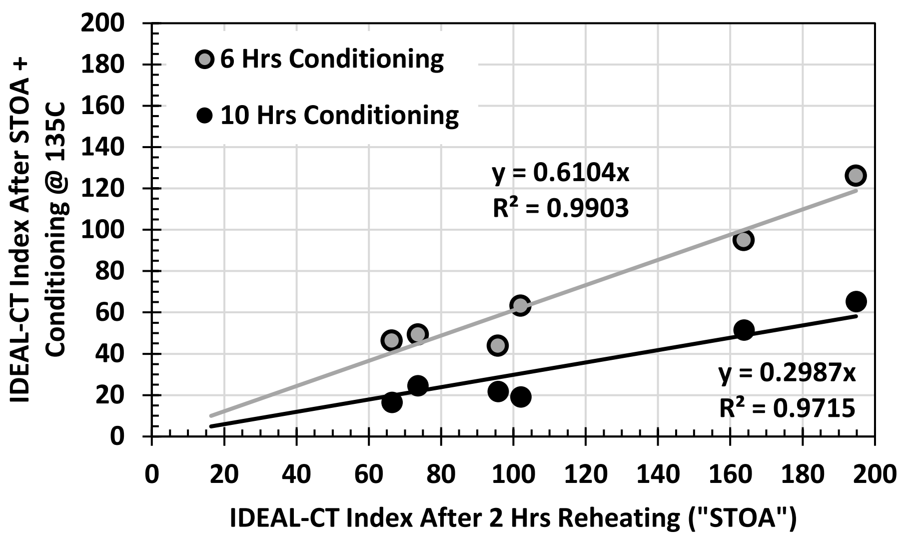


Figure 56 – Relationship of IDEAL-CT Index Values at Different Conditioning Levels to the STOA Conditioning at 25ºC

1.1.2 - Overlay Tester

The results of the Overlay Tester testing are shown in Figures 57 to 59. The results shown are based on the number of cycles to failure, which is defined as when a reduction of 93% of the applied load at the 1st cycle has been achieved. The data represents testing five test specimens, eliminating the high and low values and averaging of middle three specimens with the error bars noting the standard deviation +/- the average value. Unlike the IDEAL-CT Index results, the Overlay Tester results appear to be much more sensitive to the test temperature differences between 20°C and 25°C. On average, there was a 55% reduction in the number of cycles to failure when reducing the testing temperature from 25°C to 20°C. This was almost 4 times the magnitude observed in the IDEAL-CT Index test results.

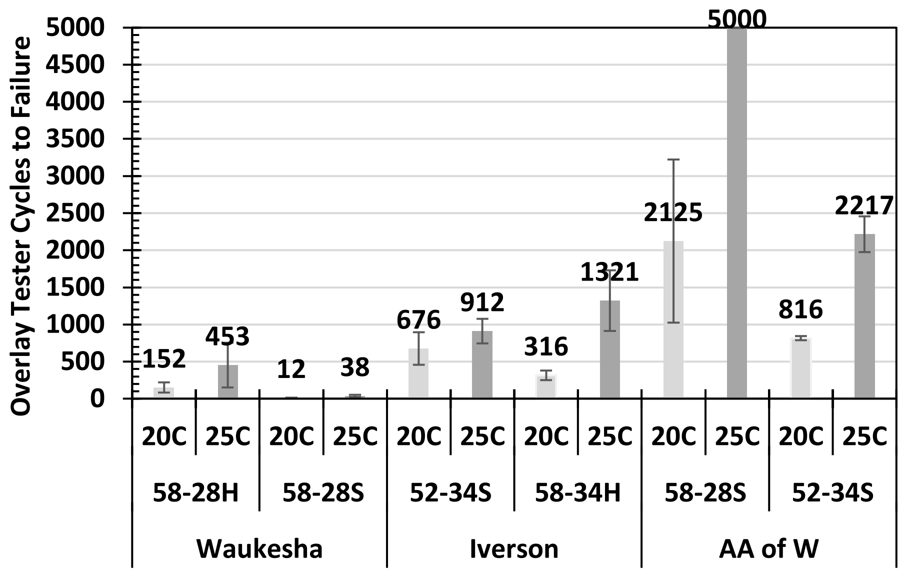


Figure 57 – Overlay Tester Cycles to Failure at 2 Hrs STOA Conditioning

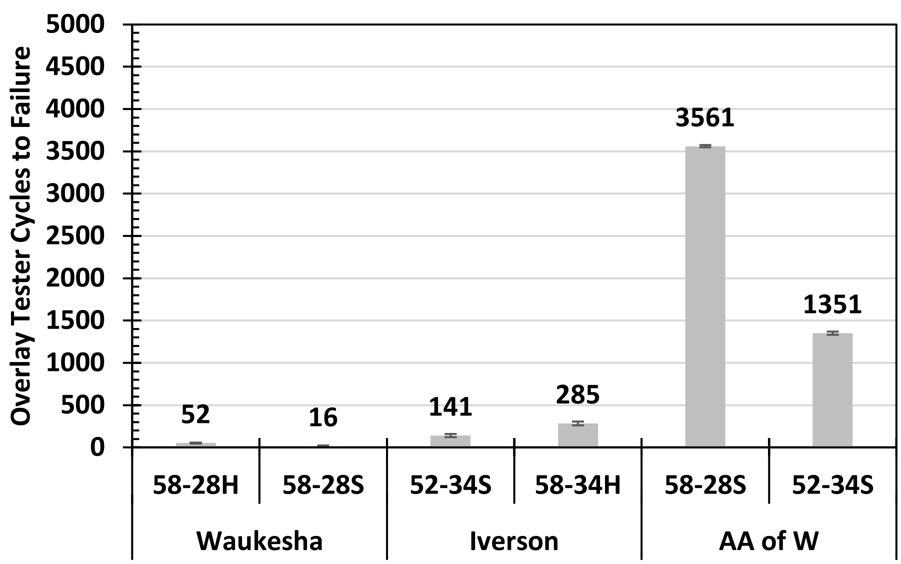


Figure 58 – Overlay Tester Cycles to Failure at STOA + 6 Hrs Conditioning at 135°C

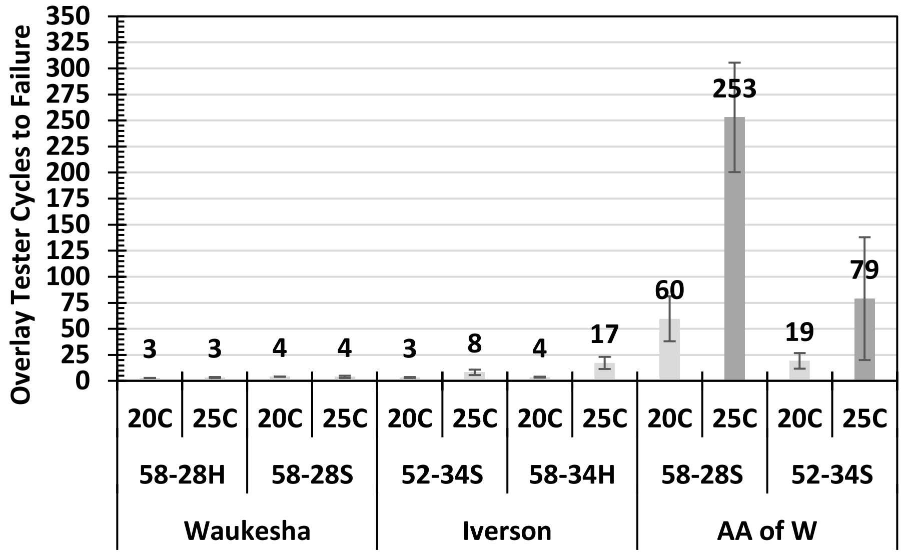


Figure 59 - Overlay Tester Cycles to Failure at STOA + 10 Hrs Conditioning at 135ºC

The impact of mixture conditioning compared to the STOA condition is shown in Figure 60 and 61 for the 25ºC test temperature. Only the test data for the 25ºC is shown as all three conditioning levels were tested at this condition. The results in Figure 60 clearly indicate that as conditioning level increased, the resultant cycles to failure decreased. Similar to the IDEAL-CT Index, the Overlay Tester results clearly decrease as the magnitude of the loose mix conditioning increases. However, as noted earlier, the rate of change due to the increase in conditioning magnitude is far greater than previously observed with the IDEAL-CT Index.

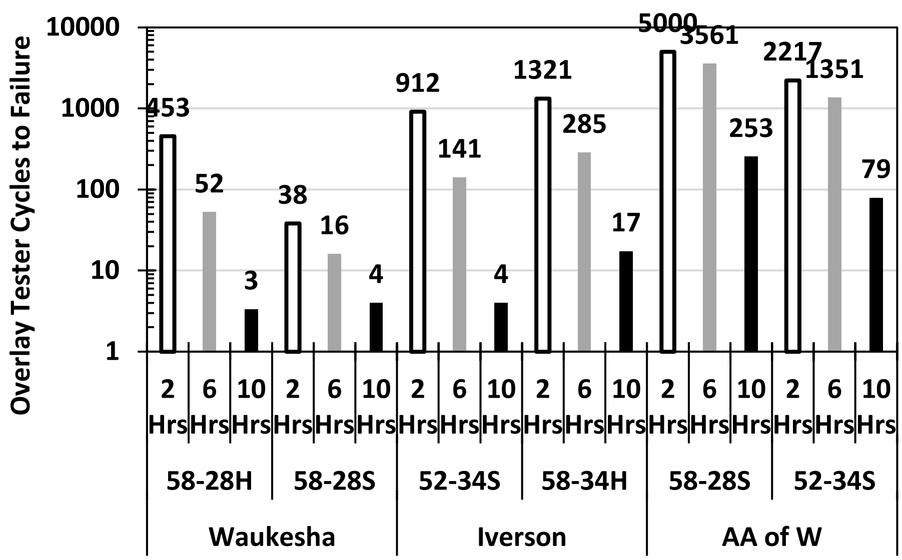


Figure 60 – Measured Overlay Tester Cycles to Failure at 25ºC for All Three Conditioning Levels

Figure 61 illustrates the rate of change in Overlay Tester performance with respect to the STOA conditioning. Similar to the IDEAL-CT Index, there is a strong relationship between the Overlay Tester cycles to failure after STOA conditioning and the STOA + 6 hr and STOA + 10 hr loose mix conditioning. Once again this illustrates that in most cases, the fatigue cracking performance after STOA conditioning is highly related to the final performance after extended loose mix conditioning.

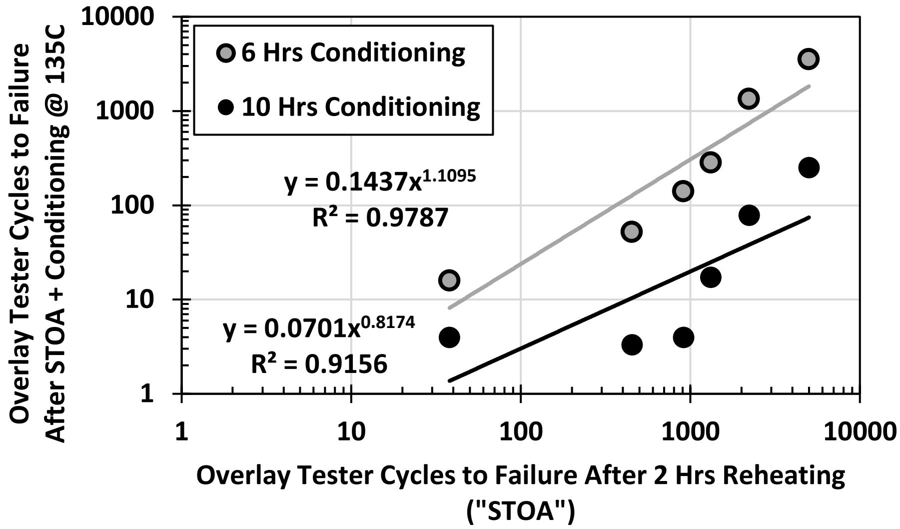


Figure 61 - Relationship of Overlay Tester Cycles to Failure at Different Conditioning Levels to the STOA Conditioning at 25°C

Figure 62 shows the relationship between the IDEAL-CT Index and the Overlay Tester Cycles to Failure at the identical condition (i.e. – test temperature, aged condition, compacted air voids) for the WisDOT asphalt mixes tested. The figure shows a strong relationship between the two test methods, although it is clear that the range of results/performance for the Overlay Tester is far greater than the IDEAL-CT Index, which would be an indication that the sensitivity to the mixture and test conditions is greater in the Overlay Tester than the IDEAL-CT Index. Also shown in the figure is the IDEAL-CT Index criteria of 30 currently being implemented by WisDOT. Based on the relationship shown in Figure 62, an IDEAL-CT Index of 30 would correspond to an Overlay Tester Cycles to Failure value of 10. It should be noted that the IDEAL-CT Index value of 30 is after the STOA + 6 hrs of loose mix conditioning at 135°C, in accordance with WisDOT specifications.

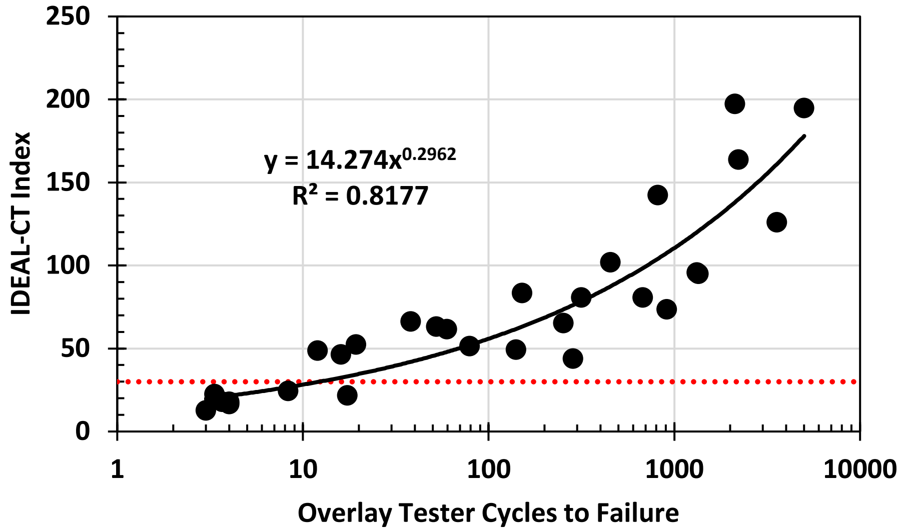


Figure 62 – Relationship Between the Overlay Tester Cycles to Failure and the IDEAL-CT Cracking Index

1.2 - Results on Recovered Asphalt Binders

As discussed earlier, the tested IDEAL-CT Cracking Index specimens were used to recover the asphalt binder providing a direct comparison between the asphalt binder and mixture performance at the identical aged condition. For example, the IDEAL-CT Cracking Index specimens that were loose conditioned for 2, 6 and 10 hrs prior to compaction were recovered and their respective asphalt binder properties determined. The wide range of loose mix conditioning provided IDEAL-CT Index values ranging from 16.6 to 194.8. Overall, a total of 16 data points provided comparisons between the various asphalt binder parameters and the IDEAL-CT Cracking Index.

Table 9 and Figure 63 shows the results of the Multiple Stress Creep Recovery (MSCR) testing with respect to asphalt binder recovered after the 2 hour reheating. This is assumed to have replicated RTFO conditioning, and therefore "age correct" for the MSCR test. The results show that both the Iverson H and Waukesha H recovered binders had the highest levels of elastomer response, although the addition of recycled asphalt binder most likely resulted in the Z-Factor being less than zero (indicating the % Recovery falls below the Elastomer line). With both of the "H" asphalt binders being polymer modified, the fact that the Z-factor is negative suggests that the addition of recycled asphalt during mixture production reduced the elastomeric properties of the polymer modified asphalt binder.

Table 9 – Multiple Stress Creep Recovery Properties at 52 and 58C

Mix Designation	52C			58C		
	Jnr (1/kPa)	% Rec	Z-Factor	Jnr (1/kPa)	% Rec	Z-Factor
Iverson H Oil	0.28	43.2	2.3	0.83	29.4	-1.6
Iverson S Oil	1.44	2.5	-24.3	3.64	0.2	-20.7
Waukesha H	0.14	43.9	-5.3	0.42	29.2	-7.7
Waukesha S	0.09	37.9	-17.1	0.29	23.0	-17.9
American 2A	0.68	22.2	-10.3	1.91	10.8	-13.9
American 2C	0.58	11.1	-22.8	1.68	2.9	-22.8

The identification of elastomeric modification is important as it has been documented to result in a more negative ΔTc value, as well as reducing the phase angle measurements, especially at moderate intermediate/high test temperatures. This mirrors the general impact on the incorporation of recycled asphalt and/or higher levels aging.

The asphalt binder parameters were statistically compared to the IDEAL-CT Cracking Index values using the Correlation function in Microsoft Excel's Data Analysis Toolpak. The quick statistical analysis showed the following parameters best correlated to the IDEAL-CT Cracking Index at 25°C (Table 10);

- Best Point Parameters: BBR Low Temperature PG Grade, m-value; Intermediate Temperature PG Grade (G* x sin (δ)); Glover-Rowe Parameter at 15C, 10 rad/s
- Shape Parameters: ΔTc; δ_{10MPa}; and R-value measured at 15C, 10 rad/s

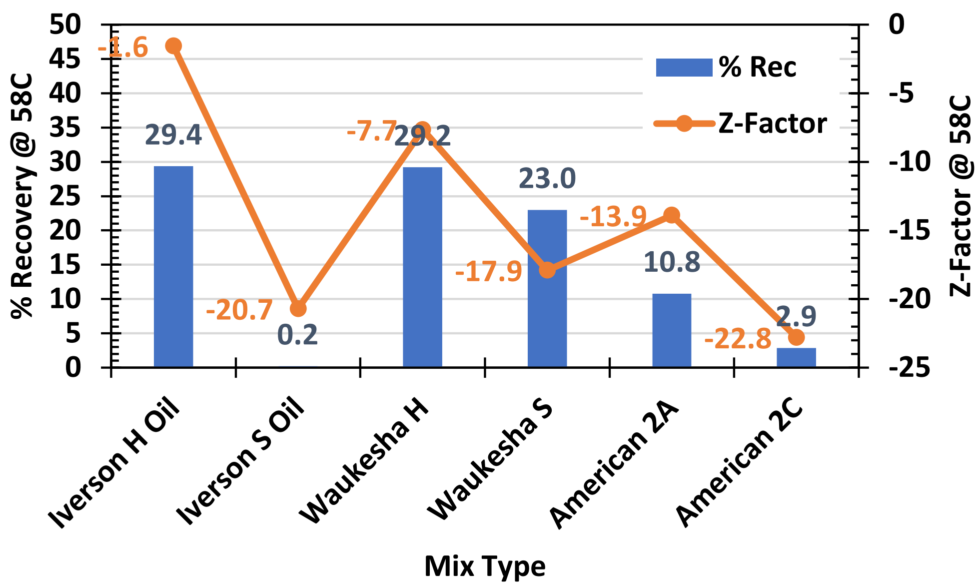
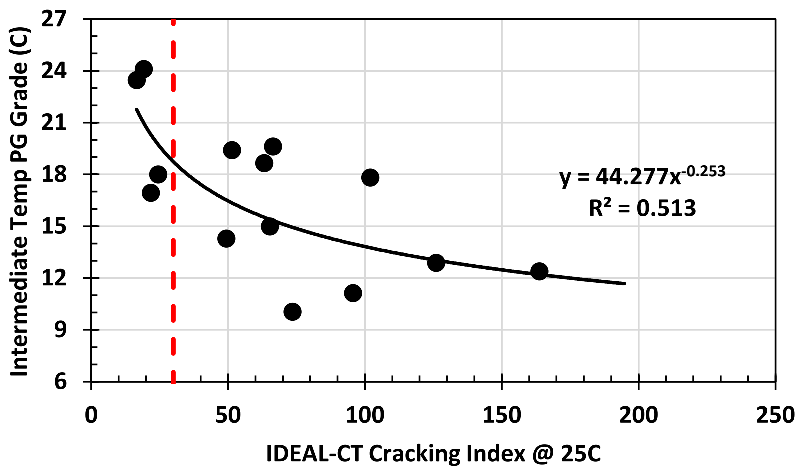


Figure 63 – Multiple Stress Creep Recovery Elastomer Parameters

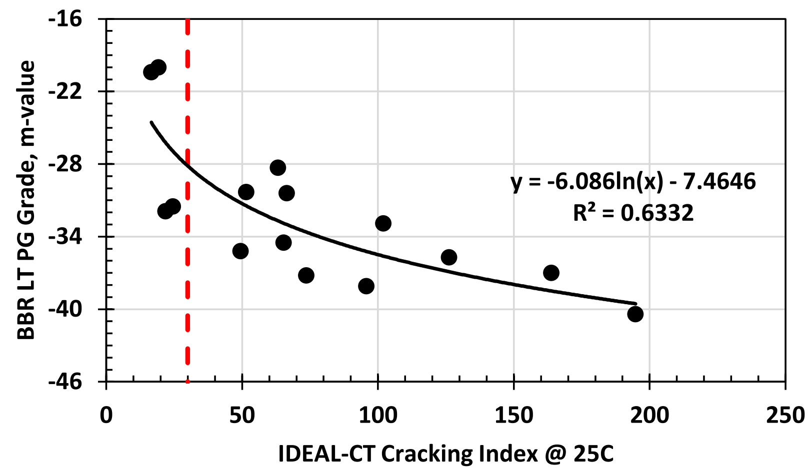
Table 10 – Correlation Coefficient of IDEAL-CT Cracking Index to Recovered Asphalt Binder Parameters

Parameter Type	Binder Parameter	Correlation
Point Parameters	Log G-R kPa (10 rad., 15ºC)	-0.75
	Log G-R kPa (10 rad., 19ºC)	-0.74
	Log G-R kPa (10 rad., 22ºC)	-0.74
	Low Temp PG, m-value	-0.72
	(G* x sin(δ)) (10 rad)	-0.65
	CA ω0 25ºC ref Temp	0.53
	Log G-R G* (0.005 rad., 15ºC)	-0.50
Shape Parameters	ΔTc	0.65
	Log Gc	0.64
	δº (G*= 10 MPa)	0.62
	R value at 15ºC	-0.61
	δº (G*= 8.967 MPa)	0.59
Fracture Test	ABCD Tcr	-0.69
	CTOD (19ºC; 22ºC)	0.59
Asphalt Content (%)		0.57

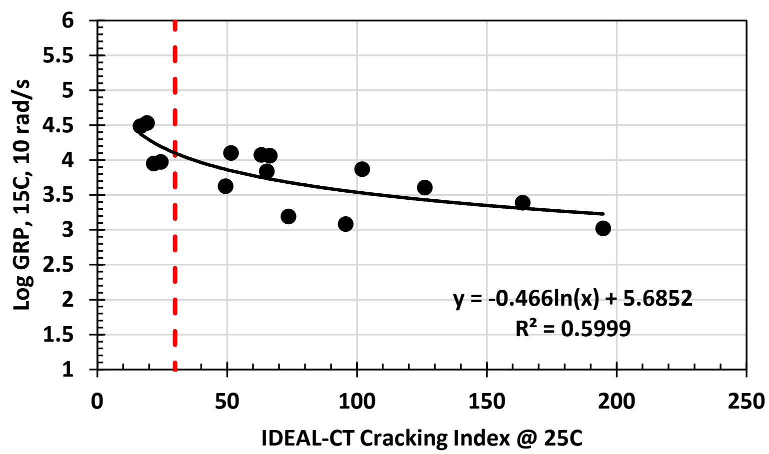
Figure 64 provides the results of the Point Parameters; 1) Intermediate Temperature PG Grade; 2) Low Temperature PG Grade (m-value), and 3) Glover-Rowe Parameter at 15^{o}C, 10 rad/s. Overall, there is some scatter in the results as one would expect considering the data consists of 6 different asphalt mixture with differences in asphalt content, gradations and volumetrics. The two better performing Point Parameters appears to be the Low Temperature PG Grade (m-value) and the Glover-Rowe Parameter at 15C, 10 rad/s (GRP_{15C}). With respect to ease of use, the GRP_{15C} is a simpler parameter to measure and could directly replace the existing Intermediate Temperature PG Grade while only requiring one test temperature, as opposed to a Passing and Failing value for interpolation.



(a)



(b)

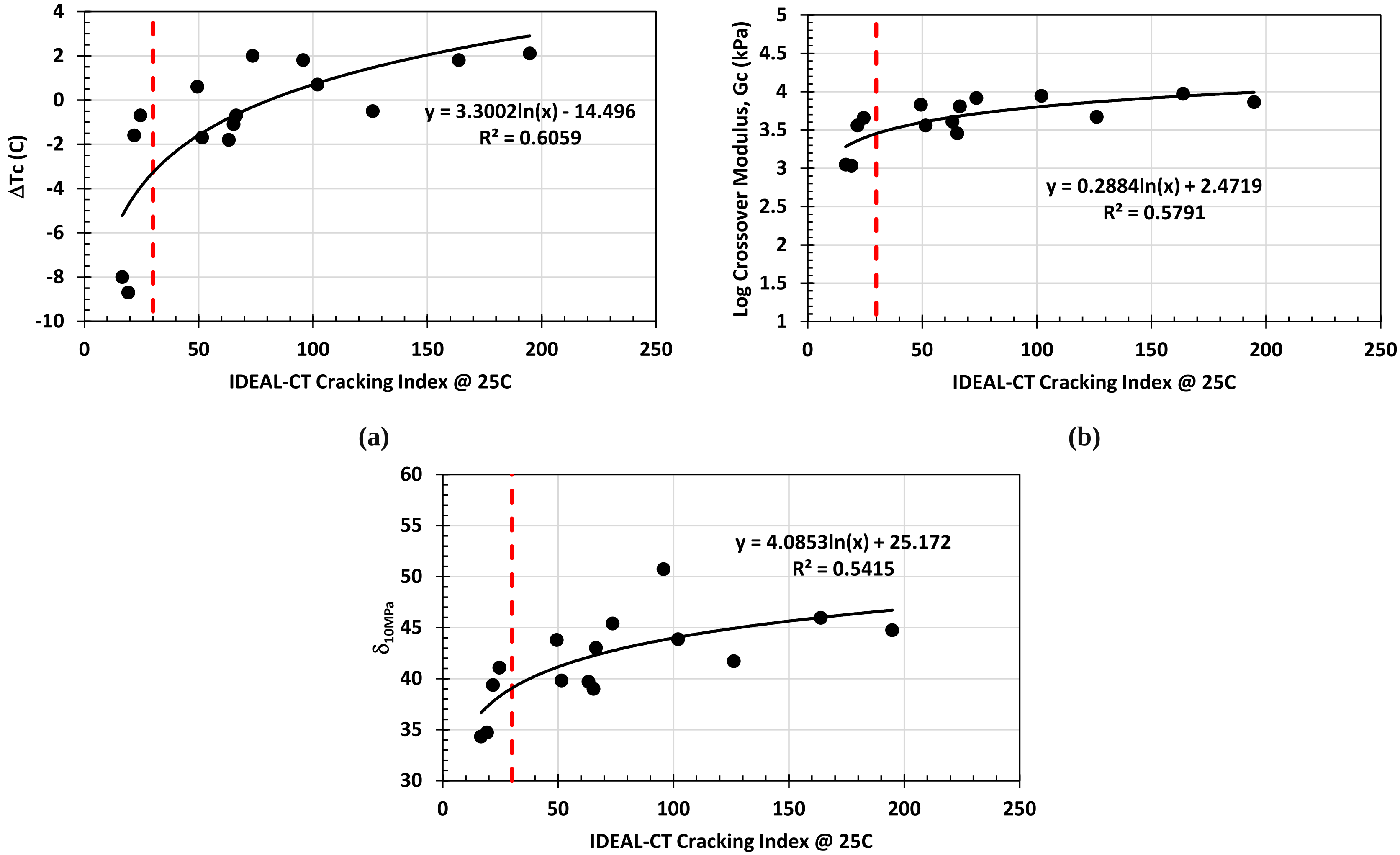


(c)

Figure 64 – Rheological "Point Parameters"; a) Intermediate PG Grade; b) Low Temperature PG Grade from m-value; and c) Glover-Rowe Parameter at 15C, 10 rad/s (shown as Log of Value)

The three best Shape Parameters are shown in Figure 65. Similar to the Point Parameters, there are advantages to utilizing some parameters over the other. For example, the ΔTc value would require testing in the BBR at both Passing and Failing test temperatures. Obviously not optimal with respect to testing time for Quality Control. However, the use of the BBR is commonly required for acceptance, and therefore, the ΔTc can be taken directly from the measured data. Meanwhile, both the Cross-over Modulus (Gc) and the phase angle at a shear modulus of 10 MPa (δ_{10MPa}) can be measured on the DSR, requiring less material, as well as in an expedited fashion, lending itself to both QC and QA testing.

The results of the fracture tests (Double Edge Notched Tension test and Asphalt Binder Cracking Device) are shown in Figure 66. The ABCD Tcr ranked relatively higher compared to some of the other test methods evaluated in the study, while the DENT CTOD was not as strong. Figure 67 further utilized the ABCD Tcr within the NCHRP 9-60 approach that includes the ΔTc parameter. The data labels in the figures notes the IDEAL-CT Cracking Index value for the respective recovered asphalt binder. The NCHRP 9-60 approach in Figure 67a clearly distinguishes between the Poor and Good cracking performers, however, the current criteria for the PASS/FAIL would need to be adjusted to better fit the data measured in the study (Figure 67b).



(c)

Figure 65 - Rheological “Shape Parameters”; a) ΔTc; b) Cross-over Modulus, Gc and c) δ_{10MPa}

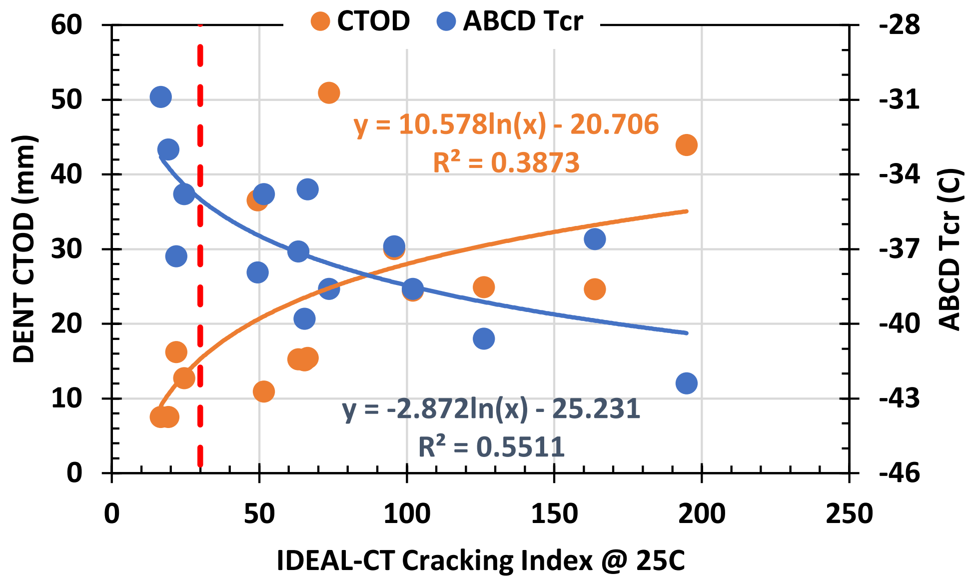


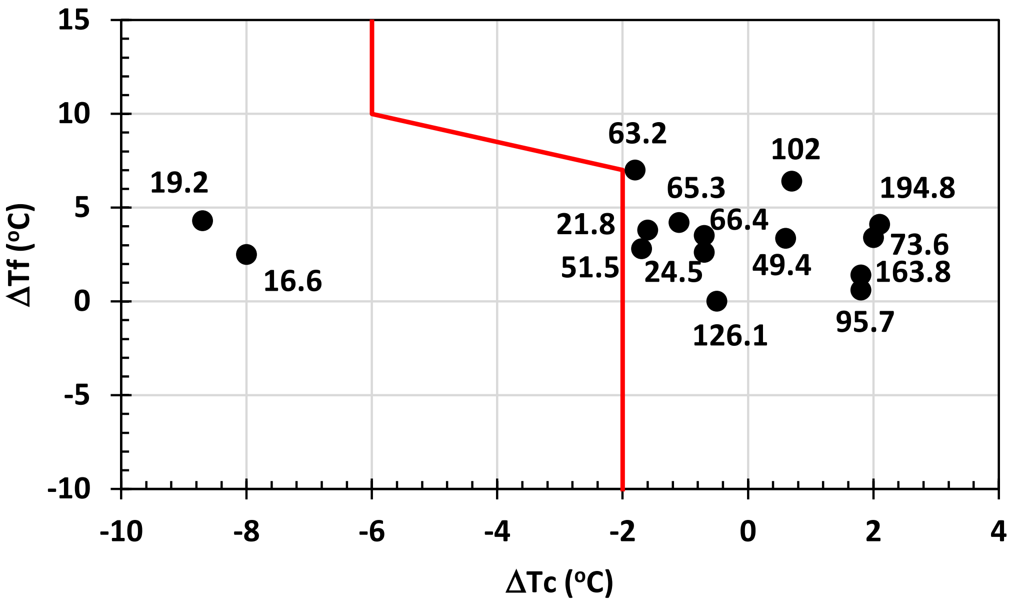
Figure 66 – Results of Fracture-based Asphalt Binder Tests Compared to IDEAL-CT Cracking Index

1.3 - Initial Proposed Fatigue Cracking Binder Specification

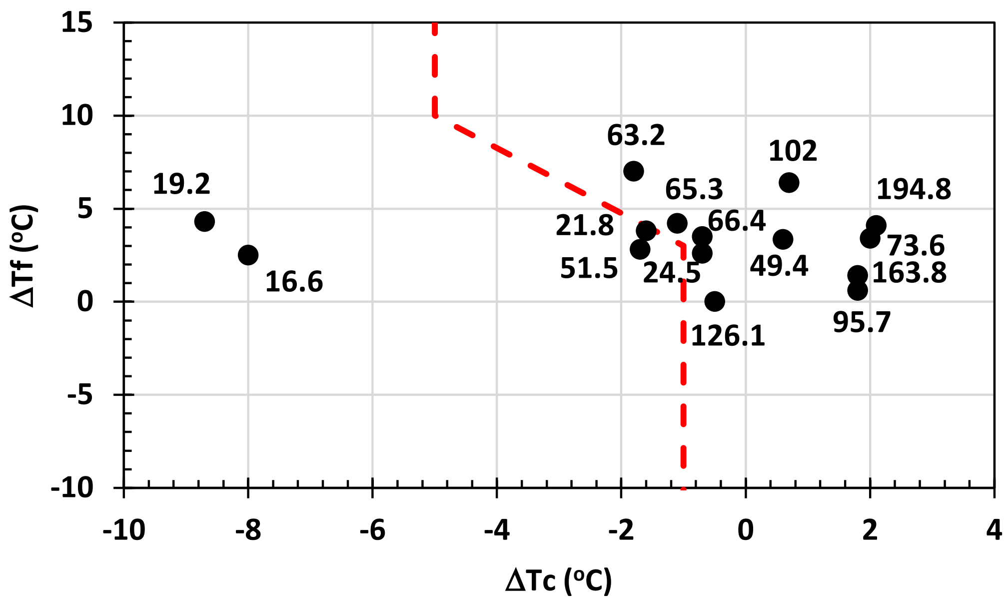
The initial concept of the research study was to look at including the ΔTc parameter within an asphalt binder specification and to substitute the existing intermediate PG grade parameter for the Glover-Rowe parameter. This was the basic approach presented by Anderson (2023). However, as will be shown below, different parameters are recommended based on the work conducted in this study.

First, the ΔTc parameter is recommended to be replaced by the R-value determined in the DSR at 15°C using Equation (9). As noted in the Literature Review, there exists variability in the ΔTc measurement/calculation that could significantly impact confidence in the criteria and enforcement in a specification. The shear modulus and phase angle measurement has been found to be far less variable and there exists a strong relationship between the ΔTc and the R-value (Figure 68). In addition, the R-value can be assessed at a single temperature greatly simplifying the testing protocol. Therefore, R-value (Rheological Index) would become that durability parameter that was never included in the original Superpave specification.

Second, the Intermediate Temperature PG Grade (G* x sin δ) is to be replaced by the Glover-Rowe Parameter at a test temperature of 15°C (GRP_{15C}). As Anderson showed, the current G* x sin δ approach rewards asphalt binders of significantly low phase angles and high shear modulus, characteristics common with aged asphalt binders. Once again, a single test temperature can be utilized as opposed to measuring a Passing and Failing value under current Superpave requirements. In addition, the R-value and the GRP_{15C} can be determined using the identical test data – resulting in a Point and Shape Parameter from a single test. And as Figure 69 illustrates, the methodology identifies a majority of the IDEAL-CT Cracking Index mixture performance.



(a)



(b)

Figure 67 – NCHRP 9-60 Approach of Recovered Asphalt Binders Compared to Measured IDEAL-CT Cracking Index; a) NCHRP 9-60 Proposed Criteria; b) Proposed WisDOT Modified Criteria

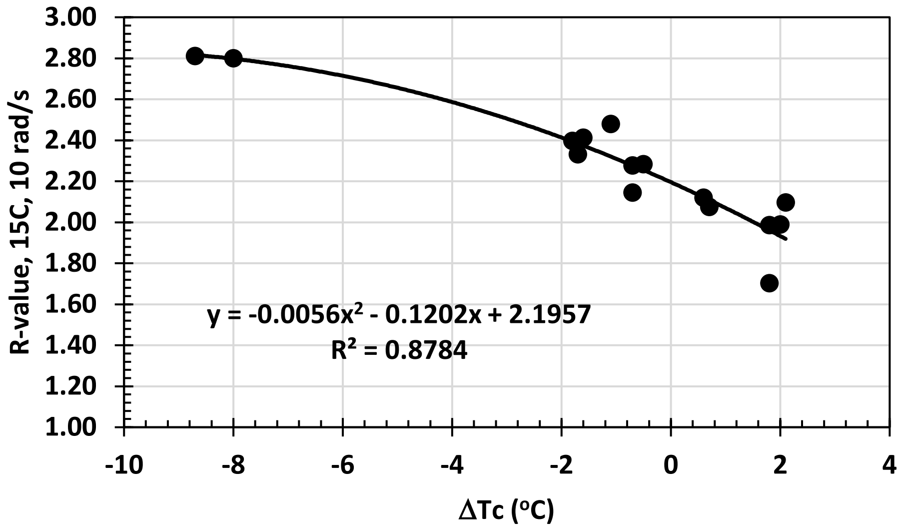


Figure 68 – Relationship Between Two Shape Parameters; ΔTc and R-value

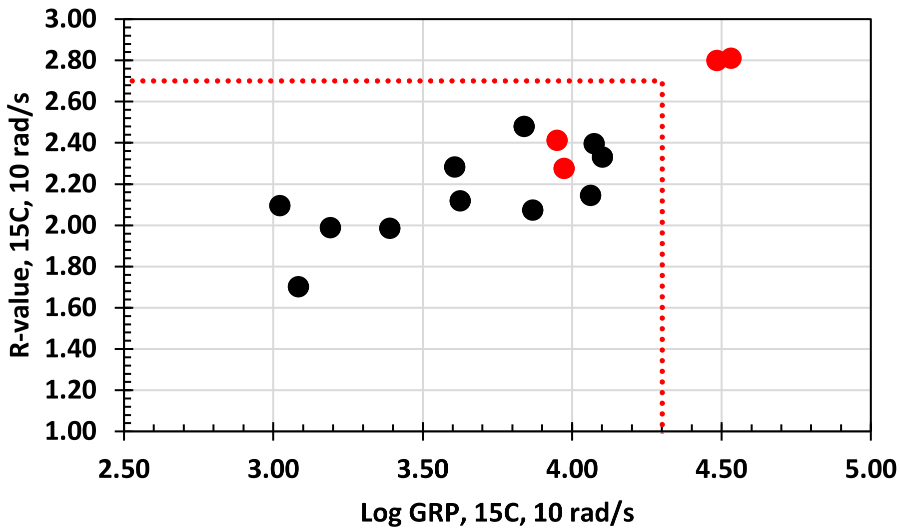


Figure 69 – Recommended Intermediate Temperature Fatigue Cracking Criteria for Wisconsin Based on IDEAL-CT Cracking Index of 30 (Black = PASSING; Red = FAILING)

As noted above, an intermediate test temperature for asphalt binders to evaluate the fatigue cracking potential is recommended to be 15°C for the state of Wisconsin. It was originally planned to utilize the methodology proposed by Christensen and Tran (2022). However, recent work conducted by Rowe and Raposa (2023) suggests that this methodology would result in temperatures too warm to represent intermediate temperature cracking. For example, using the proposed final recommendation from Christensen and Tran (2022), the northern region of Wisconsin would have an intermediate temperature of 19°C (for a -34°C low temperature PG grade), while the southern region would have an intermediate temperature of 22°C (-28°C low temperature PG grade). The approach by Rowe and Raposa approach was originally suggested by Witczak (1972) and implemented in the Asphalt Institute Methods (1982), shown as Equation 11.

$$M_p = M_a\left(1+\frac{1}{z+4}\right)-\frac{34}{z+4}+6 \qquad (11)$$

where,

M_p = mean annual pavement temperature (°F) at depth z
M_a = mean annual air temperature (°F)
z = is depth in inches

This temperature is then increased by 4°C to produce a *Teff (FC)* (effective intermediate temperature for fatigue cracking). Reviewing the Wisconsin database compiled by Rowe and Raposa (2023), a total of 37 weather locations in Wisconsin were collected for the analysis (Figure 70). The analysis resulted in the following;

- Average *Teff (FC)* = 16.0°C
- Standard Deviation *Teff (FC)* = 1.7°C
- 98% Confidence Lower Bound *Teff (FC)* = 15.3°C

For ease of use, an intermediate test temperature of 15°C was selected. It should be noted that the Glover-Rowe parameter at 19°C and 22°C were calculated and compared to the IDEAL-CT Cracking Index results but found to be just slightly less correlated to the mixture cracking when compared to GRP_{15C}. In the end, the 15°C test temperature is more theoretically sound for selection based on the intermediate temperature determination approach described.

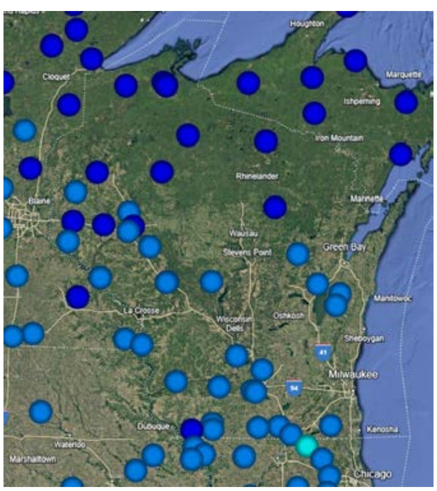
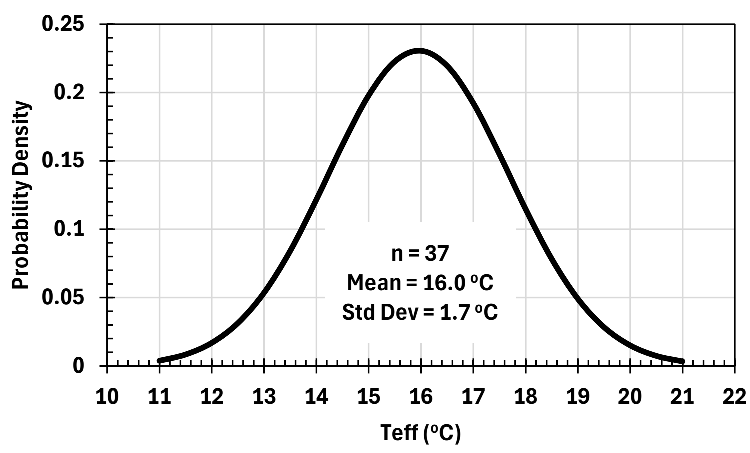



Figure 70 – Weather Data and Statistical Results for Wisconsin *Teff (FC)*

1.4 - Loose Mix Conditioning to Equivalent Binder Conditioning Comparisons

To establish a proper specification framework, the magnitude of the conditioning for all asphalt materials should be consistent. With that being said, WisDOT has incorporated a loose mix conditioning protocol that requires the reheating (2 hours at compaction temperature) and loose mix conditioning (6 hrs at 135°C) prior to determining the IDEAL-CT Cracking Index. Therefore, if WisDOT is to enforce a mixture fatigue cracking parameter, any asphalt binder-based fatigue cracking parameter should also be conditioned to the same relative magnitude.

In order to compare the rheological aging, asphalt binder was recovered from the mixtures after 2 hours of loose mix reheating and then conditioned for 20 hours and 40 hours, respectively, in the pressure aging vessel (PAV). Intermediate and low temperature rheological parameters were the focus of the comparisons. The approach used interpolation of the measured binder properties from the recovered asphalt at each mixture aged condition and determined where the PAV conditioned binders intersected the trendline. Examples are shown in Figure 71 for a binder using the δ_{10MPa} parameter and the ΔTc parameter, respectively. As the figure shows, as loose mix conditioning time increases, the asphalt binders become more prone to hardening and loss of relaxation. The same can be said regarding the PAV conditioning (i.e. – as conditioning goes from 20 to 40 hours, the asphalt binders perform worse).

The following asphalt binder parameters were used for comparison; Glover-Rowe Parameter, δ_{10MPa}, Low Temperature PG Grade from m-value, ΔTc, Asphalt Binder Cracking Device (ABCD) Tcr, and the Double Edge Notched Tension (DENT) CTOD parameter. The tabulated results are shown in Table 11. On average, it would take approximately 2 hours of reheating at compaction temperature plus 8 hours of loose mix conditioning at 135°C to be rheologically equivalent to 20 hours in the PAV. It would take over 16 hours of loose mix conditioning to achieve 40 hours in the PAV. The results of the 20 hour PAV are quite consistent while more variability is associated with the 40 hour PAV comparison due to the data being compared with an extrapolated trendline. Regardless, the results show merit to the fact that the WisDOT loose mix aging protocol for IDEAL-CT Cracking Index testing is relatively consistent, although slightly less severe, to the 20 hour PAV conditioning applied to current asphalt binder purchase specifications. Therefore, when incorporating the 20 hour PAV conditioning for asphalt binders, it will impart a slightly greater aging to the asphalt binder than the loose mix conditioning protocol (i.e. – it will be more conservative when applied within a purchase specification).

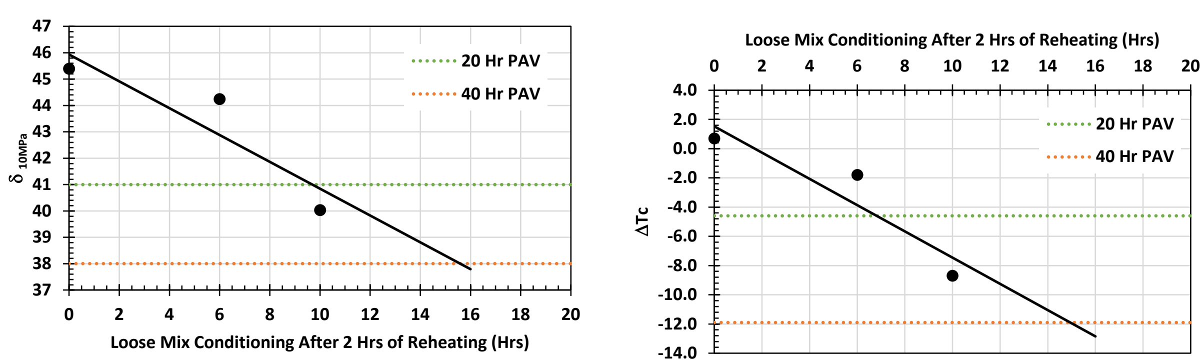


Figure 71 – Interpolation Process to Compare Loose Mix and Pressure Aging Vessel Conditioning Times

Table 11 – Loose Mix Conditioning Time Comparison to PAV Conditioning Time

Asphalt Binder Test Parameter	20 Hr PAV		40 Hr PAV	
	Ave	Std Dev	Ave	Std Dev
GRP, 15C, 0.005 rad/s	8.1	3.4		
GRP, 19C, 10 rad/s	8.5	1.5	20.0	2.3
δ @ G* = 10 MPa	8.5	1.6	15.1	3.7
LT PG Grade, m-value	8.0	1.3	16.6	2.5
ΔTc	7.6	2.4	20.2	6.4
ABCD Tcr	8.3	1.5	13.2	6.1
DENT CTOD	8.7	1.0	13.2	3.5
Average (All Tests)	***8.2***	***0.4***	***16.4***	***3.1***

1.5 - Selection of Proposed Asphalt Binder Fatigue Cracking Specification

With the asphalt binder parameters (Point and Shape Parameters), testing temperature and laboratory conditioning level determined, an asphalt binder specification for fatigue cracking can be proposed for Wisconsin. The methodology would be as follows for fatigue cracking evaluation of asphalt binders:

1. RTFO the respective asphalt binder in accordance to AASHTO and WisDOT specifications;
2. PAV condition the asphalt binder for 20 hours in accordance to AASHTO and WisDOT specifications;
3. Determine the shear modulus (G*) and phase angle (δ) at 15°C and 10 rad/s. Determine the following parameters;
    a. Glover-Rowe Parameter (GRP_{15C})

$$G^* \frac{(cos\delta)^2}{sin\delta} < 20{,}000\ kPa$$

    b. Rheological Index (R-value)

$$1.0 < \frac{(\log 2) \times log\left(\frac{G^*(\omega)}{G_g}\right)}{log\left(1 - \left(\frac{\delta(\omega)}{90}\right)\right)} < 2.7$$

4. Determine Low Temperature PG Grade in accordance to AASHTO and WisDOT specifications and meet the Stiffness and Creep (m-value) requirements;

The above procedure is also captured in Table 12. There is a maximum GRP_{15C} of 20,000 kPa and a range in the calculated R-value that must be less than 2.7 and greater than 1.0. The GRP_{15C} < 20,000 kPa and R-value < 2.7 was shown to result in asphalt mixtures with a loose mix conditioned IDEAL-CT Index greater than 30.0. The minimum R-value of 1.0 is to ensure the asphalt binder is not too soft and highly temperature susceptible, potentially leading to poor pavement performance (Christensen and Tran, 2022).

Table 12 – Proposed Asphalt Binder Fatigue Cracking Specification for Wisconsin Materials and Conditions

<table>
<tr><th>PAV Aging Temperature (°C)</th><th colspan="6">100</th><th colspan="6">100 (110)</th></tr>
<tr><td>Dynamic Shear, T315
G*(cos δ)²/sin δ², 10 rad/s;
< 20,000 kPa</td><td colspan="6">15°C</td><td colspan="6">15°C</td></tr>
<tr><td>Dynamic Shear, T315
R=log(2)log(G*/1E9)/log(1-(δ/90))
at 10 rad/s;
1.0 < R < 2.7</td><td colspan="6">15°C</td><td colspan="6">15°C</td></tr>
<tr><td></td><td colspan="12"></td></tr>
<tr><td>Creep Stiffness, T313
at 60 sec & low temp
Stiffness < 300 MPa
m-value > 0.300</td><td>0</td><td>-6</td><td>-12</td><td>-18</td><td>-24</td><td>-30</td><td>0</td><td>-6</td><td>-12</td><td>-18</td><td>-24</td><td>-30</td></tr>
</table>

1.6 – Comparison of WisDOT Criteria to NCHRP Project 9-59

NCHRP Project 9-59 had similar recommendations to the WisDOT criteria, except the researchers recommended the use of the BBR data to determine the R-value. The researchers utilized Equation 12 to determine the R-value from the BBR data, where "S" is the stiffness at the measured temperature in question and the "m" is the m-value at the same measured temperature in question. The authors noted that the 20 hr PAV data for this measurement is readily available from conventional PG grading procedures and would ultimately be available on the Certificate of Analysis (COA) that is commonly provided to state agencies prior to mixture production.

$$R - value = \log(2)\frac{log\left(\frac{S}{3000}\right)}{log(1-m)} \qquad (12)$$

A comparison was made between the DSR measured R-value proposed in this study to the BBR measured R-value proposed in NCHRP Project 9-59. The entire dataset developed during this study was used to generate Figure 72, which compares the DSR calculated R-value at 15°C and 10 radians per second to the BBR calculated R-value at different test temperatures. The figure clearly shows that the agreement between the DSR and BBR calculated R-value is highly dependent on the BBR test temperature, with better agreement found at warmer test temperatures.

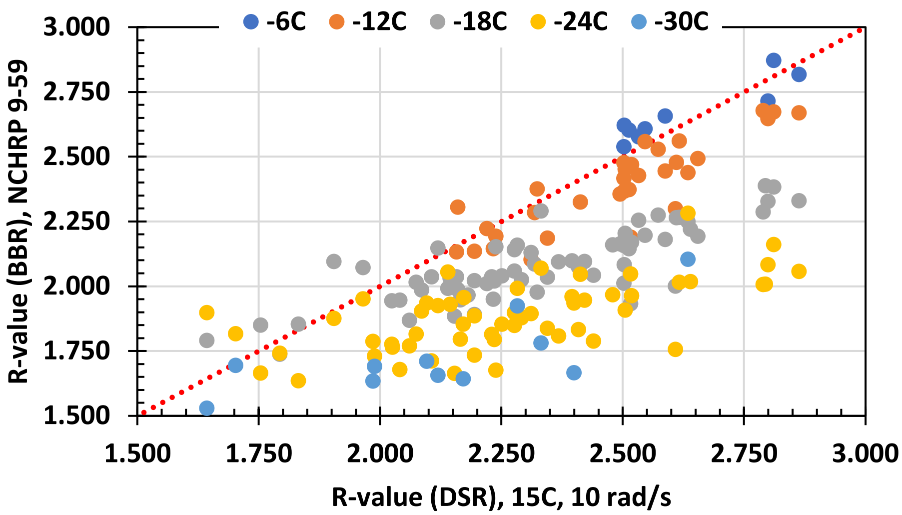


Figure 72 – Calculated R-value from DSR at 15°C and from BBR Data

The percent of the root mean square error (RMSE%) was used to quantify the level of agreement between the temperature measurements. As Figure 73 shows, better agreement is found between two methods as the test temperatures converge.

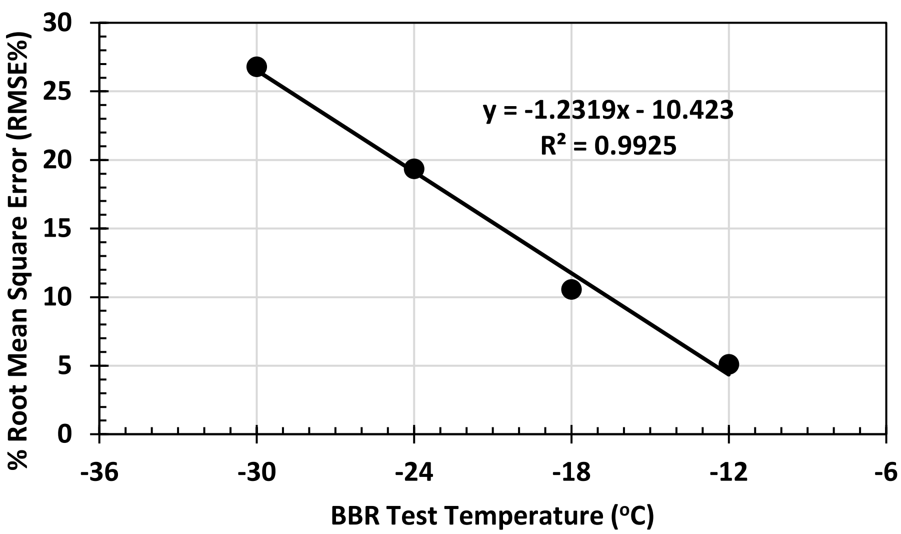


Figure 73 – Percent of Root Mean Square Error (RMSE%) Comparing the DSR Based R-value at 15°C to the BBR Based R-value at Various Temperatures

Figures 72 and 73 show that differences exist between the calculated R-value based on the method and test temperature used. The question is whether or not these differences result in a change in the passing or failing of asphalt binders based on the IDEAL-CT Cracking Index value of 30.0 utilized to calibrate the DSR based criteria shown earlier. Figures 74 and 75 show the resultant performance space using the same Glover Rowe parameter as recommended earlier but now with the NCHRP Project 9-59 R-value calculated at -18°C and -24°C, respectively. It is evident that the resultant figures are extremely similar to the DSR based methodology shown earlier as Figure 69. In fact, visually there appears to be no difference in general ranking whether or not the R-value is measured using the DSR or with the BBR. From a practical perspective, testing in the DSR requires less time and material than testing in the BBR, and hence, a DSR based approach would help to speed up testing time.

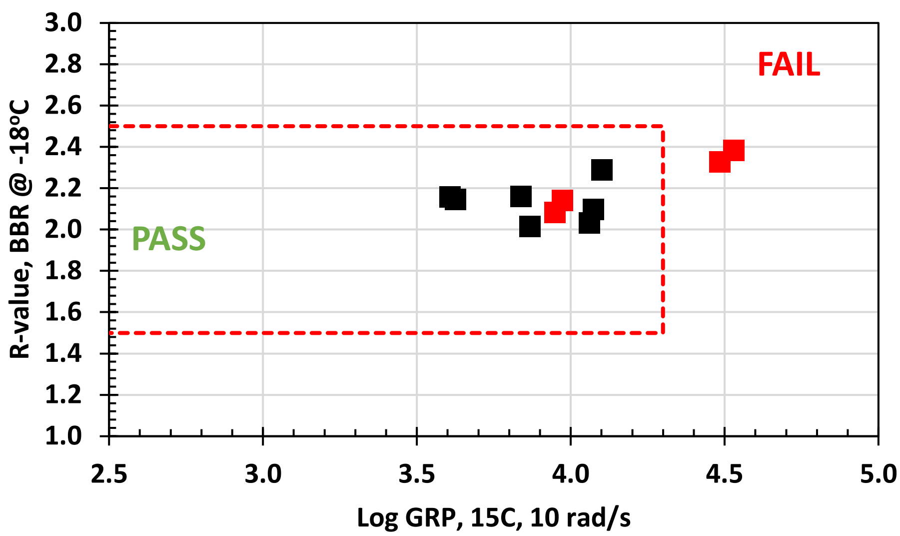


Figure 74 – Fatigue Cracking Criteria Utilizing BBR Based R-value at a Test Temperature of -18°C and Based on IDEAL-CT Cracking Index of 30

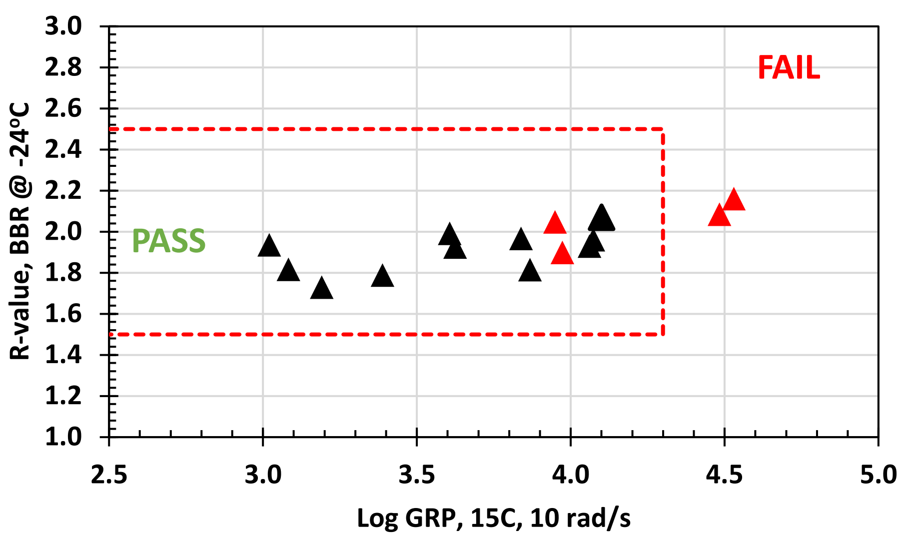


Figure 75 – Fatigue Cracking Criteria Utilizing BBR Based R-value at a Test Temperature of -24°C and Based on IDEAL-CT Cracking Index of 30

1.7 – Comparison of WisDOT Criteria to NCHRP Project 9-60

The on-going NCHRP Project 9-60 had originally recommended the use of the ΔTc parameter, along with the output from the ABCD testing, to validate asphalt binders in a proposed performance space shown earlier in Figure 51. However, as noted in the Literature Review, recent research has raised concerns with not just the repeatability of ΔTc, but also its relative ranking of polymer modified asphalt binders. In general, the ΔTc parameter tends to become more negative (i.e. – indicate poorer binder performance) as polymer modification levels increase. These phenomena can best be explained as the polymer increasing the elastic properties of the asphalt binder, resulting in a lower viscous response under time dependent loading.

As of early 2025, research efforts have moved away from ΔTc approach and instead incorporate phase angle (δ) at the constant modulus of 10 MPa (D'Angelo, 2025). As discussed in the Literature Review, both the ΔTc and δ at Constant Modulus are both classified as rheological Shape parameters. Meaning, both parameters describe the shape of the rheological master stiffness curve of the asphalt binder and should be a measure of the same general characteristic. The figure below shows the test results from the Phase 1 study comparing the recovered asphalt binder ΔTc and δ at G* = 10 MPa. There is a relatively strong correlation between the two Shape parameters of the recovered asphalt binders. Meaning, trends in asphalt binder performance picked up by the ΔTc parameter should also be picked up in the phase angle approach.

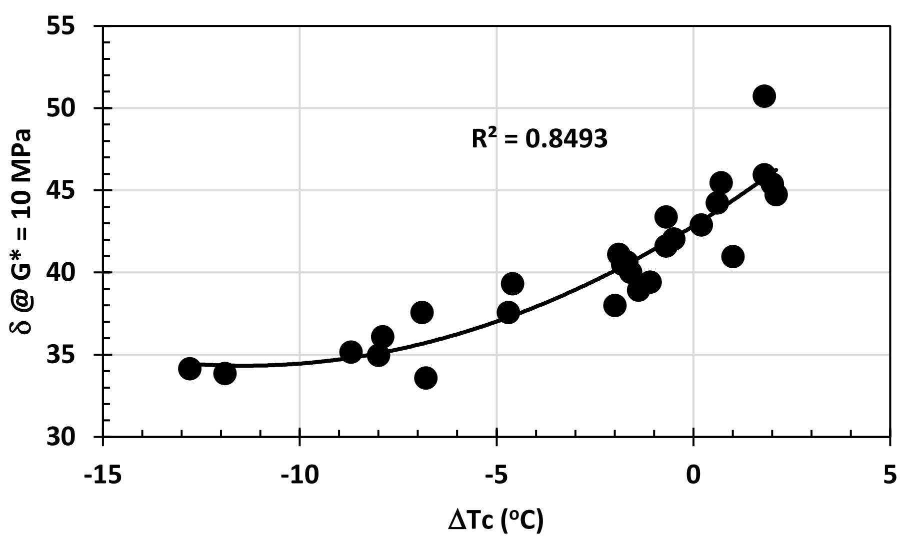


Figure 76 – Comparison of ΔTc to Phase Angle (δ) at Shear Modulus (G*) Equaling 10 MPa

Figure 77 shows the same dataset, but with the calculated R-value at 15C now compared to the phase angle at G* equaling 10 MPa. The correlation between the parameters is excellent. This is most likely attributed to both parameters being measured on the DSR and measuring the same rheological characteristic. The correlation with ΔTc shown in Figure 76 was not as strong most likely due to the variability associated with the BBR specimen prep, conditioning, and testing when compared to the DSR.

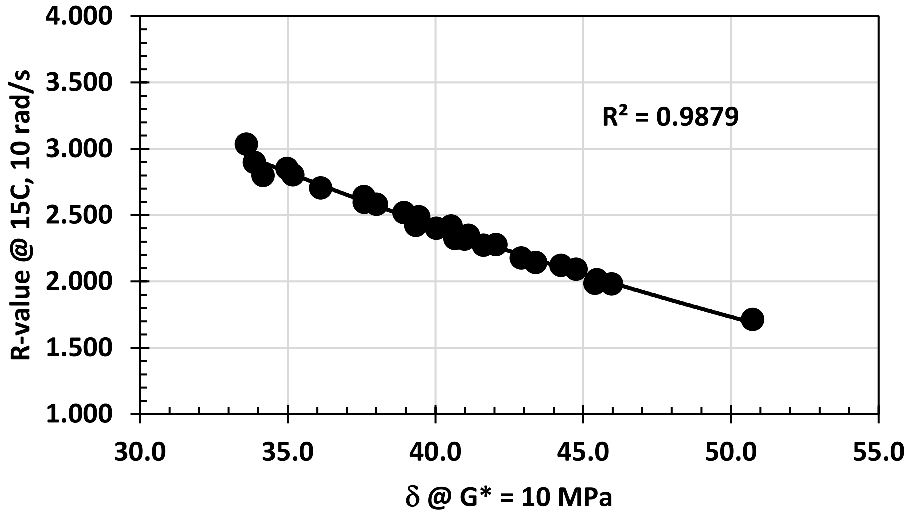


Figure 77 – Comparison of Phase Angle (δ) at Shear Modulus (G*) Equaling 10 MPa and R-value at 15C, 10 rad/s

Currently, NCHRP Project 9-60 has not provided any recommendations on limits with respect to δ at 10 MPa. Regression analysis shown earlier identified a value of 38^{o} correlating with an IDEAL-CT Index of 30. Using that value as a replacement for R-value, Figure 78 was generated. The figure results in an identical ranking as shown in the proposed WisDOT method (Figure 69). Therefore, NCHRP Project 9-60 will not provide any improvement over what has been generated in this study.

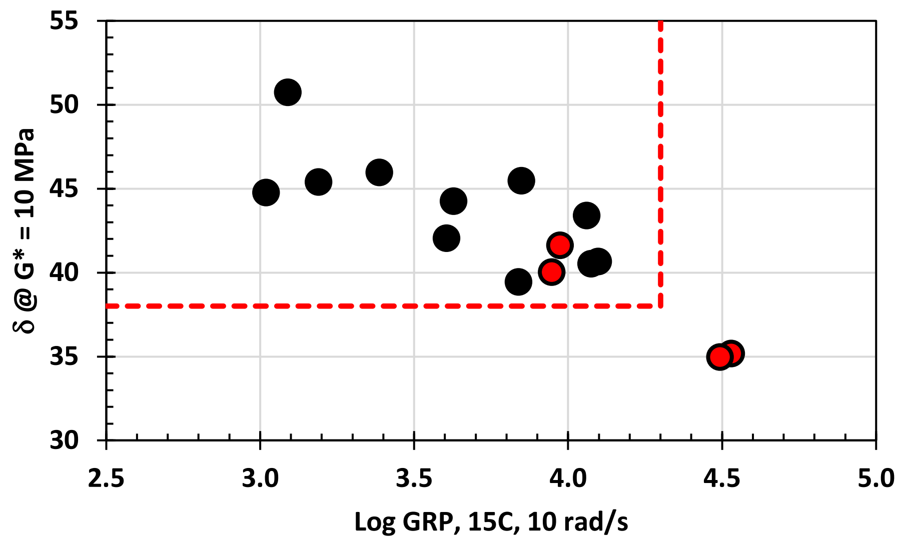


Figure 78 – Performance Space Using δ @ 10 MPa and Log GRP at 15C, 10 rad/s

The comparison of the proposed WisDOT approach to the proposed methods of NCHRP Project 9-59 and 9-60 shows the WisDOT approach matches the conceptual methodology of each of the NCHRP procedures. However, to obtain the phase angle proposed to NCHRP Project 9-60, three test temperatures are required to be run in the DSR using a wide range of testing frequencies to properly construct a master curve and appropriately calculate the phase angle at a shear modulus of 10 MPa. This requires additional testing time, as well as additional analysis to construct the master curve and calculate δ.

PHASE 2 – EVALUATION OF BASELINE ASPHALT BINDERS USED IN WISCONSIN

With a proposed asphalt binder specification in place, the next step was to evaluate how the specification may impact existing asphalt binders currently approved and supplied in Wisconsin. Three asphalt binder suppliers provided samples for evaluation. The names are noted in the study as Supplier A, Supplier B, and Supplier C. Eleven (11) asphalt binders were supplied with five of those binders having polymer modification. Table 13 shows the asphalt binder identification that was supplied for the study. Supplier B provided an asphalt binder labeled PG58S-31, and therefore, it was called this throughout the remainder of the study.

Table 13 – Asphalt Binders Supplied in Study

Supplier A	PG58S-28
	PG58H-28
	PG58V-28
	PG58H-34
Supplier B	PG52S-34
	PG58S-28
	PG58H-28
	PG58V-28
	PG58S-28 (“31”)
Supplier C	PG58S-28
	PG58V-28

As discussed earlier, prior to the intermediate and low temperature asphalt binder characterization, the asphalt binders were evaluated using the Multiple Stress Creep Recovery (MSCR) testing protocols to ensure they met the targeted high temperature of the binder grade, as well as assessing the magnitude of the polymer modification via the Z-factor value. Table 14 provides the MSCR test results of the supplied asphalt binders.

Table 14 – Multiple Stress Creep Recovery (MSCR) Properties of Supplied Baseline Asphalt Binders

Binder Supplier	Target PG Grade	Multiple Stress Creep Recovery								
		52°C			58°C			64°C		
		Jnr	% Rec	Z-Factor	Jnr	% Rec	Z-Factor	Jnr	% Rec	Z-Factor
Supplier A	PG58S-28	1.51	2.15	-22.2	3.81	0	-20.7	8.56	0	-17.5
	PG58H-28	0.28	50.2	9.2	0.80	33.55	2.5	2.15	17.55	-6.5
	PG58V-28	0.16	67.4	19.6	0.44	54.8	18.2	1.26	35.4	7.8
	PG58H-34	0.53	50.5	15.9	1.49	32.2	5.8	3.98	14.7	-5.8
Supplier B	PG52S-34	2.68	1.0	-21.7	6.33	0.0	-18.7	13.43	0.0	-16.2
	PG58S-28	1.09	3.5	-25.2	2.78	0.65	-21.8	6.38	0.0	-18.8
	PG58H-28	0.16	64.1	16.2	0.40	53.4	16.1	1.01	39.2	10.0
	PG58V-34	0.082	87.2	30.4	0.17	85.7	38.5	0.63	70.4	37.3
	PG58S-31	1.51	2.0	-24.4	3.72	0.0	-20.8	8.22	0.0	-17.7
Supplier C	PG58S-28	1.34	2.0	-25.2	3.37	0.0	-21.5	7.35	0.0	-18.1
	PG58V-28	0.014	82.5	-8.2	0.034	77.7	6.1	0.087	69.7	13.8

2.1 - Rheological Evaluation of Baseline Asphalt Binders

After RTFO conditioning, the asphalt binders underwent PAV conditioning for 20 hours and 40 hours respectively. After conditioning, the asphalt binders were evaluated for their respective shear modulus (G*) and phase angle (δ) at different temperature and loading frequencies in order to construct a master stiffness curve (MC). The MC was created using the software RHEA™ which is specifically designed for rheological analysis of visco-elastic materials.

2.1.1 - Shape Parameters

The MC shape parameters evaluated included; ΔTc, R-value at 15°C (R_{15C}), Crossover Modulus (Gc), and phase angle at G* = 10 MPa (δ_{10MPa}).

The ΔTc results are shown in Figure 79. In Figure 79, there is a tentative threshold of -3.0°C that was found to correlate to the IDEAL-CT Cracking Index of 30.0 which shows that all of the asphalt binders evaluated met this criteria after 20 hours of PAV conditioning. Meanwhile, four of the asphalt binders provided would have failed the -3.0°C after 40 hours of PAV conditioning.

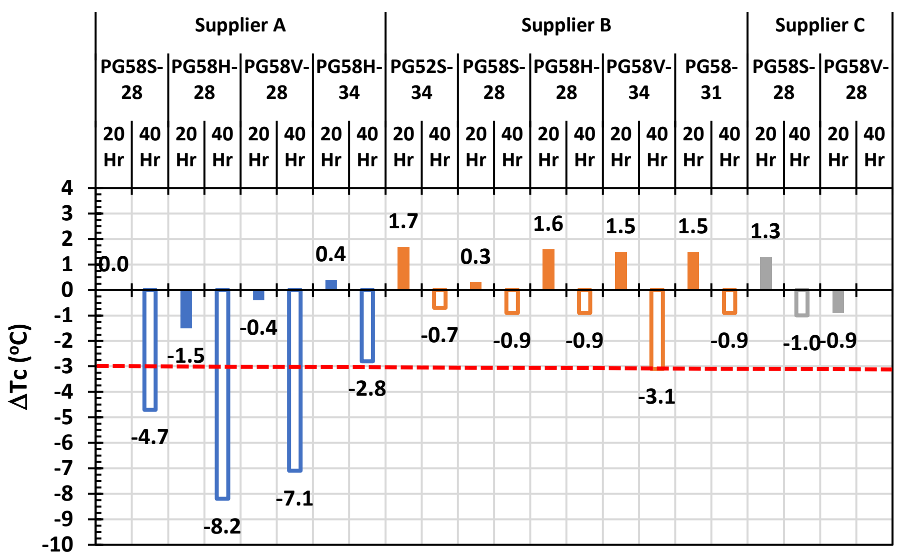


Figure 79 – ΔTc Results for Baseline Asphalt Binders

The δ_{10MPa} parameter measured for the different baseline asphalt binders are shown in Figure 80. Once again, a preliminary criteria was proposed based on the relationship to the IDEAL-CT Cracking Index value of 30.0. This resulted in a Pass/Fail value of 38 degrees, with passing results achieving higher phase angles and failing results having a δ_{10MPa} less than 38 degrees. Similar to ΔTc, all of the baseline asphalt binders, besides Supplier C's PG58V-28, would have passed after 20 hour PAV conditioning. Meanwhile, after 40 hr PAV conditioning, the same asphalt binders that failed the ΔTc also failed the δ_{10MPa}. This is because a good correlation was found between the two Shape parameters, as shown in Figure 81.

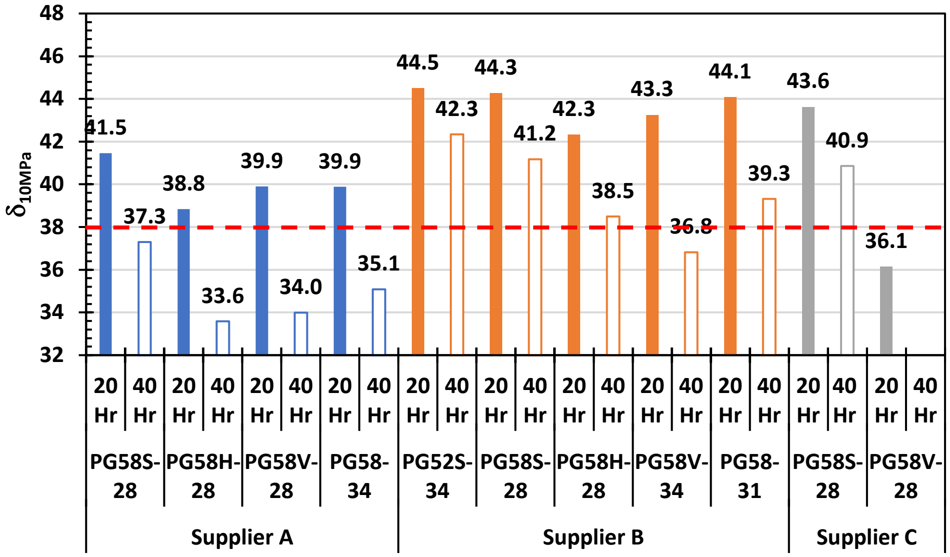


Figure 80 – δ_{10MPa} Asphalt Binder Parameter Results for Baseline Asphalt Binders

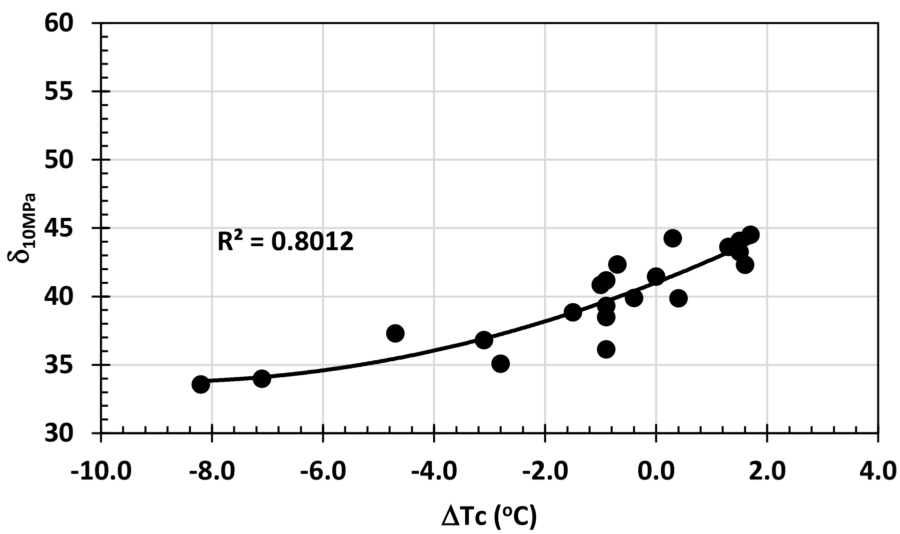


Figure 81 – Relationship Between δ_{10MPa} and ΔTc for Baseline Asphalt Binders

The R-value measured at 15°C (R_{15C}) for the baseline asphalt binders is shown as Figure 82. Once again, a Pass/Fail criteria is shown based on an IDEAL-CT Index of 30.0. The results show that all of the asphalt binders conditioned to 20 hour PAV met the criteria while only three asphalt binders conditioned to 40 hours in the PAV failed the criteria. In addition, a good relationship was found between ΔTc and R_{15C} as noted in Figure 83.

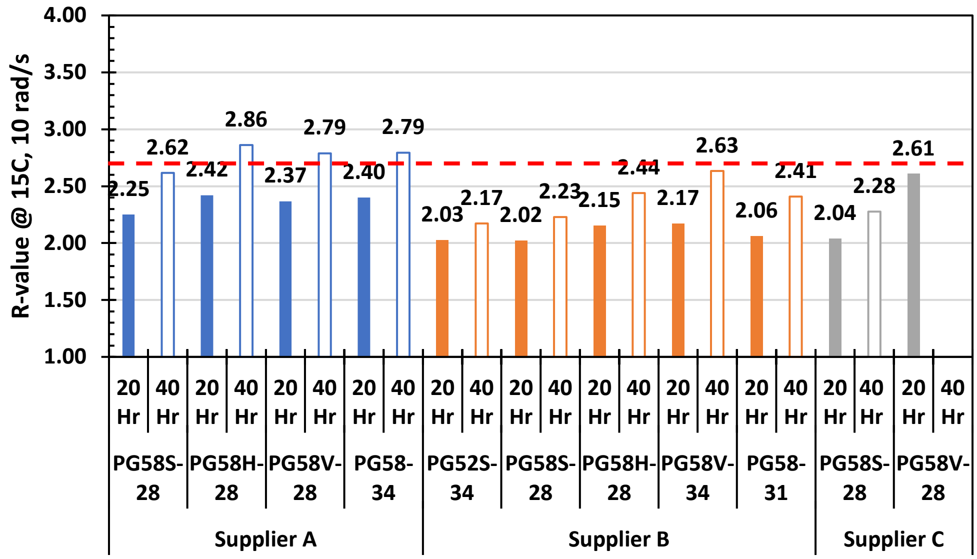


Figure 82 – DSR Derived R-value at 15°C (R_{15C}) for Baseline Asphalt Binders

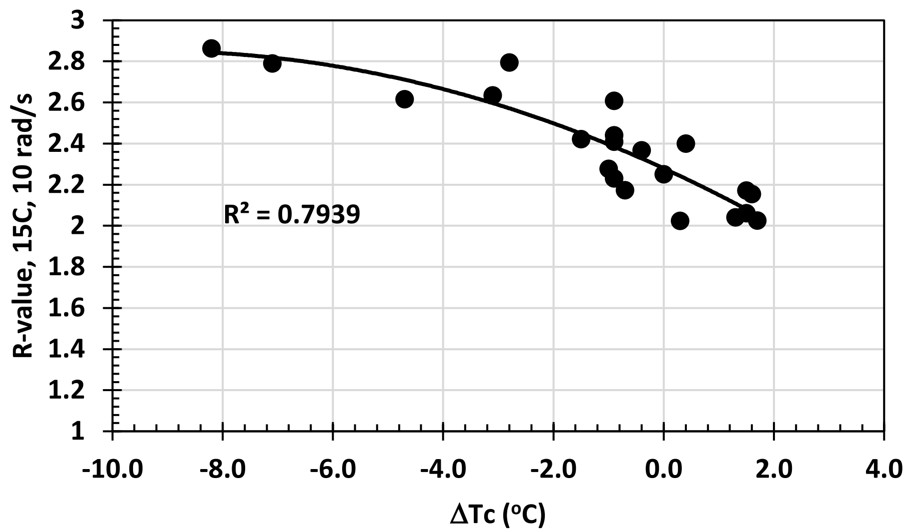


Figure 83 – Relationship Between ΔTc and R_{15C} for Baseline Asphalt Binders

The crossover modulus (Gc) was determined using the Christensen-Anderson model for the 20 hour and 40 hour conditioned baseline asphalt binders. The test results are shown in Figure 84 as the Log Gc. Unfortunately, a good relationship was not able to be developed between the IDEAL-CT Cracking Index and Log Gc, so the data is just shown as measured. Figure 85 shows a moderate relationship between Gc and ΔTc.

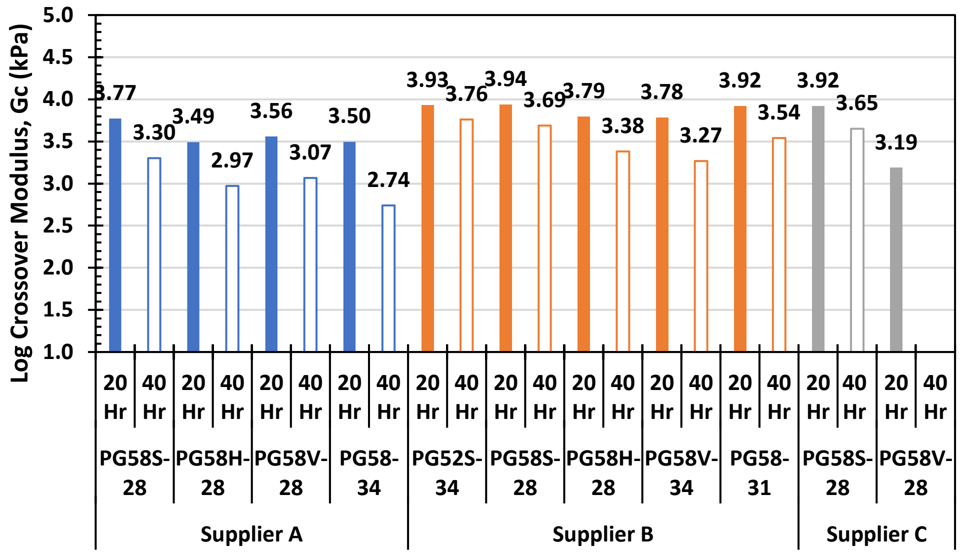


Figure 84 – Crossover Modulus (Log (Gc)) for Baseline Asphalt Binders

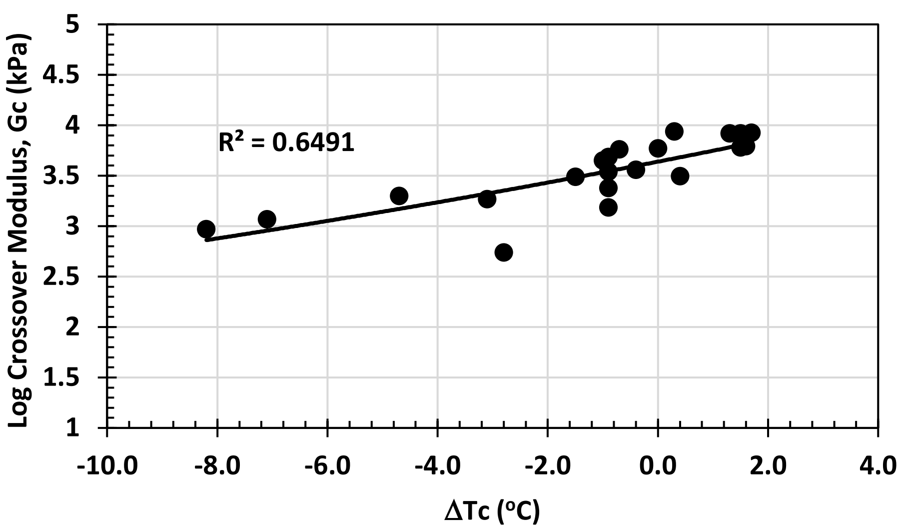


Figure 85 – Relationship Between ΔTc and Log Gc for Baseline Asphalt Binders

Overall, the R_{15C} and δ_{10MPa} had a relatively good correlation to the ΔTc value. This would indicate that both alternative parameters could be included in a purchase specification to enforce the same general asphalt binder performance required by the ΔTc parameter.

2.1.2 - Point Parameters

The MC point parameters evaluated included; intermediate temperature PG grade, low temperature PG grade from BBR m-value, and Glover-Rowe parameter at 15°C. Similar to the Shape Parameters, the Point Parameters were shown with a proposed criteria that is based on the parameter's correlation to the IDEAL-CT Cracking Index from Phase 1.

Figure 86 shows the results of the intermediate PG grade. Based on the Phase 1 work, an intermediate temperature PG grade of 19°C would be equivalent to an IDEAL-CT Index of 30.0 for the mixtures tested. Based on the testing of the baseline asphalt binders, all 20 hour PAV conditioned binders would meet the 19°C criteria except for Supplier C's PG58V-28.

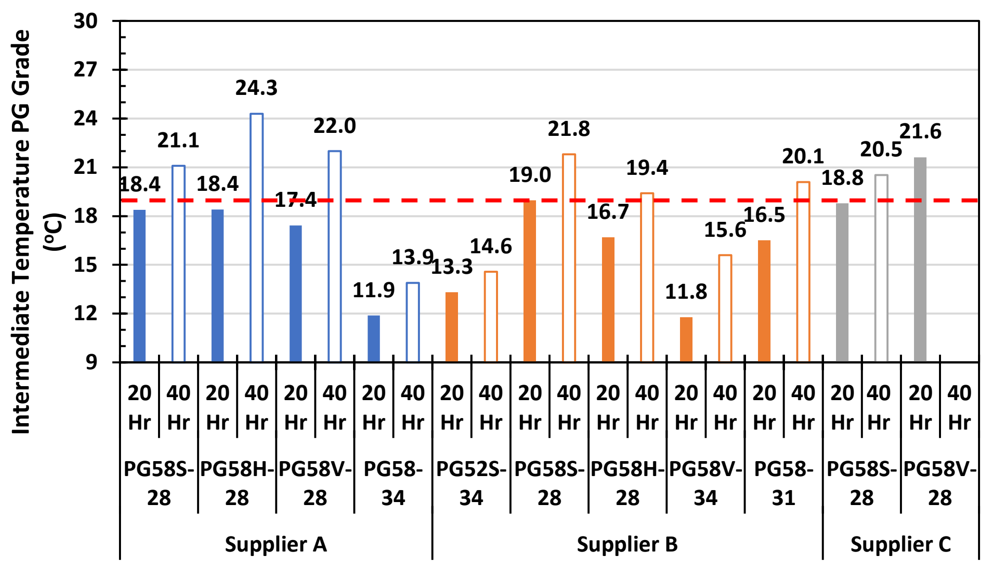


Figure 86 – Intermediate PG Grade for Baseline Asphalt Binders

The Glover Rowe Parameter measured at 15°C and 10 radians per second (GRP_{15C}) results are shown in Figure 87. The proposed specification criteria of 20,000 kPa is superimposed in the graph. The results show that all asphalt binders conditioned in the PAV for 20 hours, besides Supplier C's PG58V-28, would have met the maximum GRP_{15C} value. After 40 hours in the PAV, Supplier B's and C's asphalt binders still met the GRP_{15C} criteria, meanwhile 2 of the 4 asphalt binders from Supplier A (PG58H-28 and PG 58V-28) failed the 20,000 kPa limit.

Finally, the low temperature PG grade, as determined using the m-value, is shown in Figure 88. WisDOT essentially has two different climate regions within the state that requires the northern area to have a -34°C low temperature grade while the southern part of the state utilizes a -28°C low temperature grade. When comparing asphalt binders of the same PG grade, the results are fairly comparable at both the 20 hour and 40 PAV conditions for all three asphalt binder suppliers. It is interesting to note that the GRP_{15C} is a very good predictor of the low temperature PG grade when using the m-value (Figure 89). The relationship was found to be even more robust when pooling all of the test data from the different phases of the study (Figure 90). With the low temperature PG grade commonly controlled by the m-value, or just barely controlled by the Stiffness, the

GRP_{15C} was able to predict with relatively good accuracy the low temperature PG grade of the baseline asphalt binders (Figure 91) using the relationship shown in Figure 90.

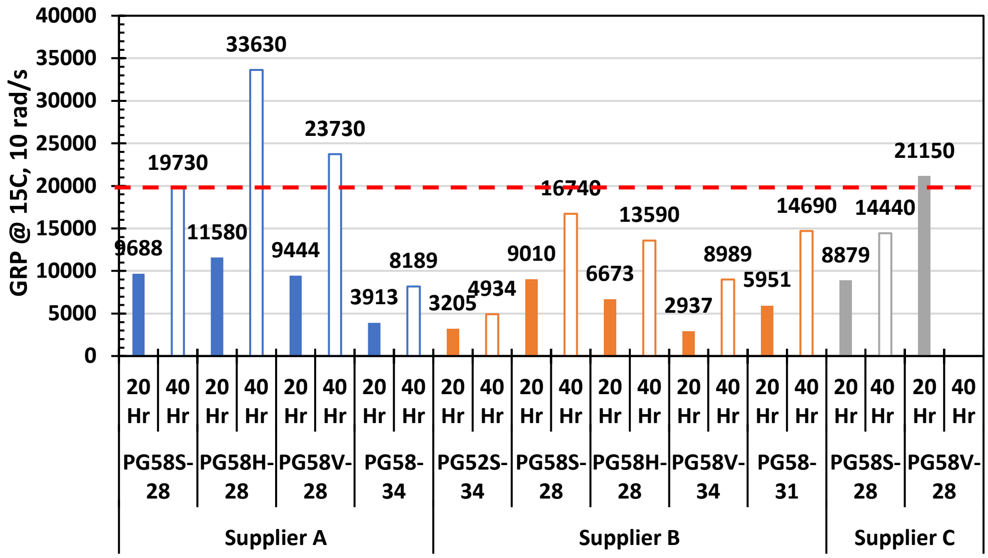


Figure 87 – GRP_{15C} Test Results for Baseline Asphalt Binders

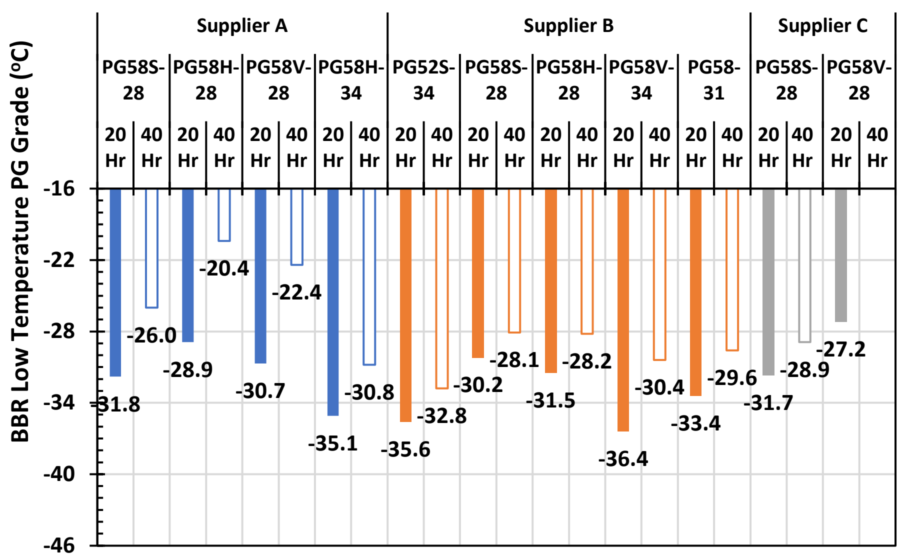


Figure 88 – Low Temperature PG Grade as Determined from BBR m-value for Baseline Asphalt Binders

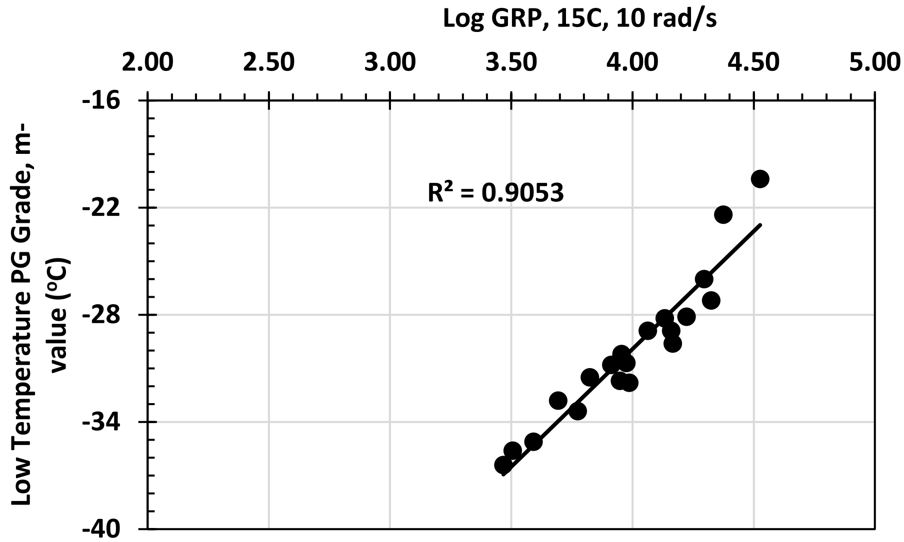


Figure 89 –Relationship Between GRP_{15C} and Low Temperature PG Grade as Determined from the BBR m-value for Baseline Asphalt Binders

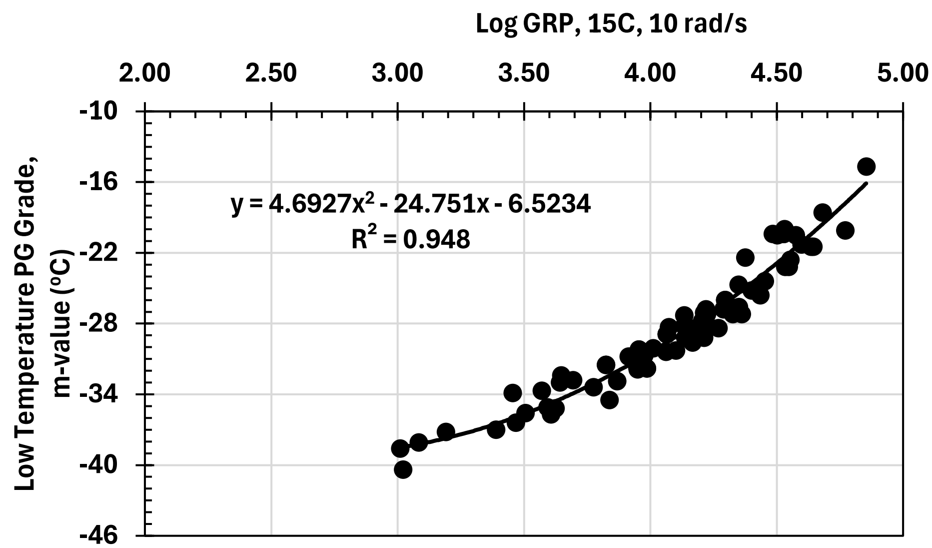


Figure 90 – Relationship Between GRP_{15C} and Low Temperature PG Grade as Determined from the BBR m-value from Entire Study Dataset

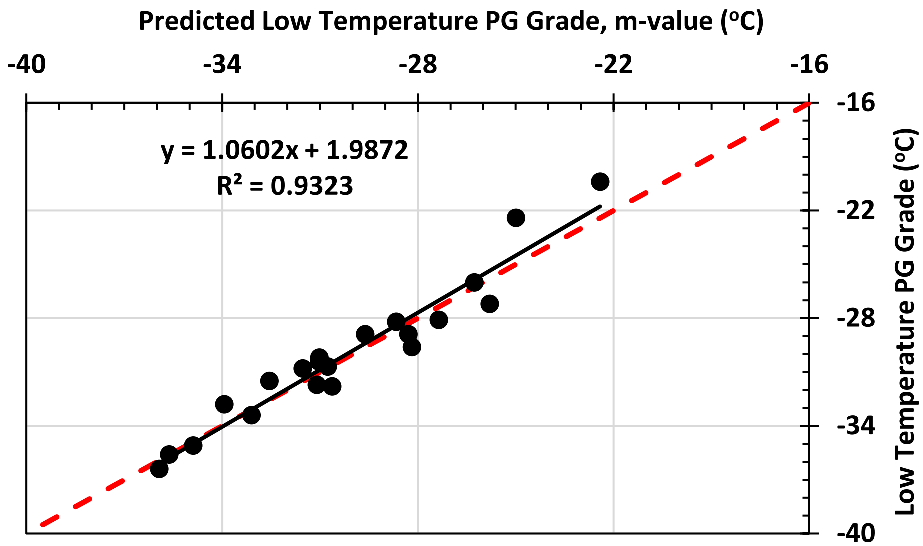


Figure 91 – Low Temperature PG Grade Prediction Using the GRP_{15C} and Relationship Shown in Figure 90

2.2 - Fracture-Based Test Results for Baseline Asphalt Binders

Along with the rheological testing, fractured based testing was also performed on the baseline asphalt binders using the Double Edge Notched Tension (DENT) test at intermediate temperatures and the Asphalt Binder Cracking Device (ABCD) at low temperatures.

The DENT parameter commonly used is the CTOD, which the critical tip opening displacement. This is basically the amount of deformation before failure of the material and is analogous to the strain tolerance of the asphalt binder at the respective temperature and strain rate of the test. It is commonly conducted at the intermediate temperature grade of the climatic region in question. Since Wisconsin contains two different climatic regions, asphalt binders with a -28°C low temperature grade were tested at 22°C. Meanwhile, asphalt binders, with a -34°C low temperature PG grade, were tested at 19°C. The test results are shown in Figure 92. Hollow bars represents the 19°C test temperature while the solid bars represent the 22°C test temperature. The results show relatively good consistency between the different asphalt binder suppliers, except for the two polymer modified binders provided by Supplier B. Both the PG58H-28 and PG58V-28 had much better strain tolerance at the 20 hour PAV conditioning than the rest of the samples tested. The test data also shows that as PAV conditioning increases, the strain tolerance of the asphalt binder decreases. Prior test results presented mainly concentrated on the stiffness increase of the asphalt binder due to conditioning. Therefore, one can make the assumption that as stiffness of the asphalt binder increases due to aging, the strain tolerance will decrease. This is often assumed and now clearly illustrated with this data.

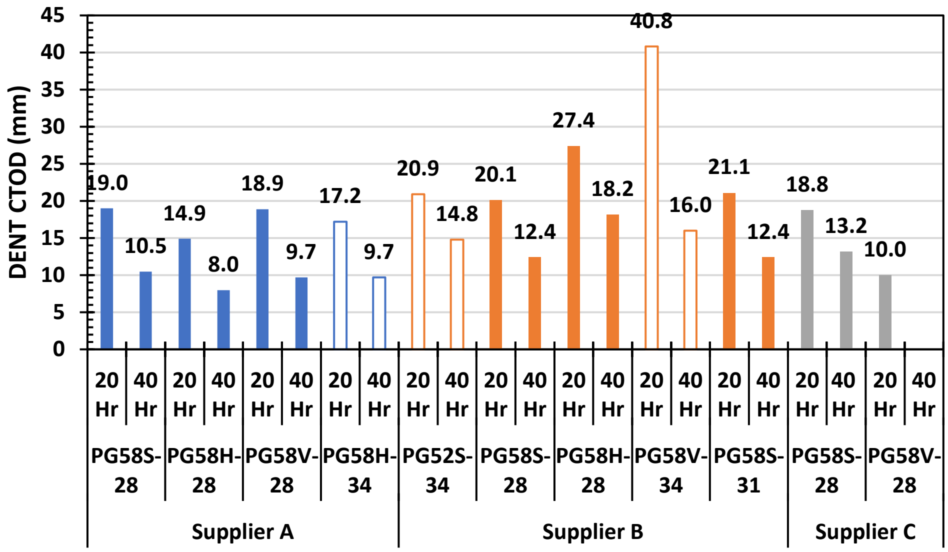


Figure 92 – DENT CTOD Test Results for Baseline Asphalt Binders

The results for the ABCD Critical Cracking Temperature (T_c) are shown in Figure 93. The colder critical cracking temperatures appear to be more associated with the polymer modified asphalt binders than the neat binders, even when the neat binder had a colder low temperature PG grade.

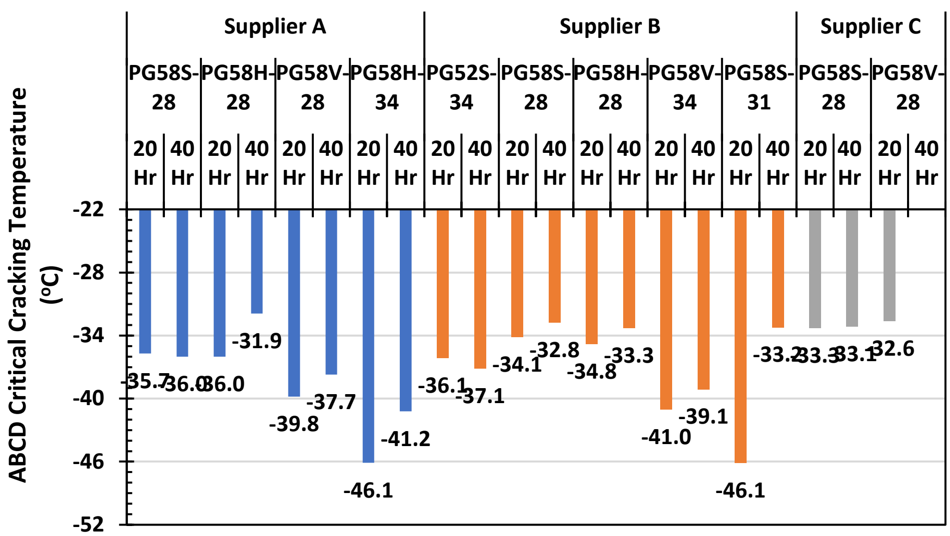


Figure 93 – ABCD Critical Cracking Temperature for Baseline Asphalt Binders

To further look into the low temperature cracking properties of the baseline asphalt binders, the ABCD results were coupled with the ΔTc value within the proposed NCHRP 9-60 analysis. The NCHRP 9-60 approach uses the ΔTc parameter to address the age hardening/relaxation properties of the asphalt binder while including ΔTf to capture the fracture toughness properties. Asphalt binders that achieve better fracture toughness still perform well when less than desirable ΔTc values occur. The NCHRP Project 9-60 researchers also suggest separate criteria for 20 hour and

40 hour PAV conditioned asphalt binders to show that the ΔTc is expected to become more negative with additional PAV aging.

Figure 94 shows the NCHRP 9-60 approach for the 20 hour PAV conditioned baseline asphalt binders. All the tested asphalt binders plot to the right of the red line, indicating that they would be classified as good, performing asphalt binders with respect to cracking/durability. However, after 40 hour PAV conditioning (Figure 95), two of Supplier A's asphalt binders would have failed the proposed NCHRP 9-60 requirements. The trend in results would indicate that the asphalt binder from Supplier A is more susceptible to age hardening than Supplier B or C.

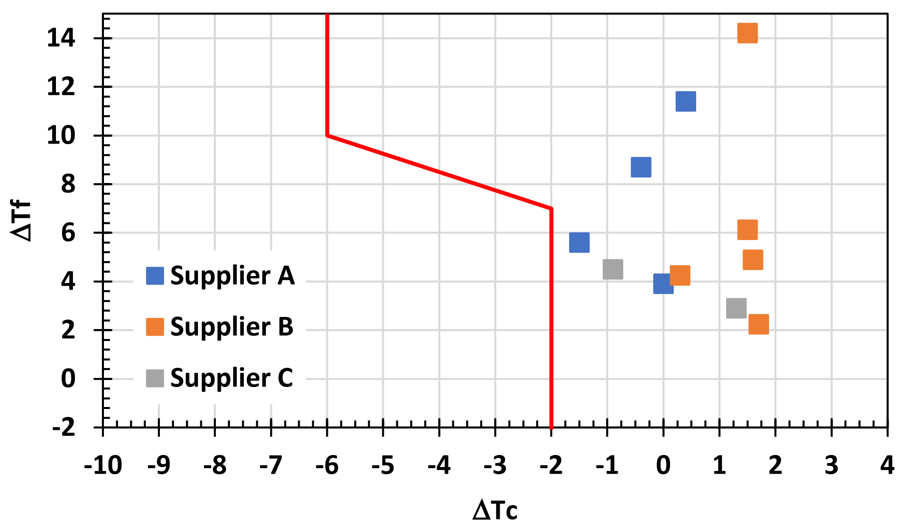


Figure 94 – NCHRP 9-60 Approach for 20 Hour PAV Conditioned Baseline Asphalt Binders

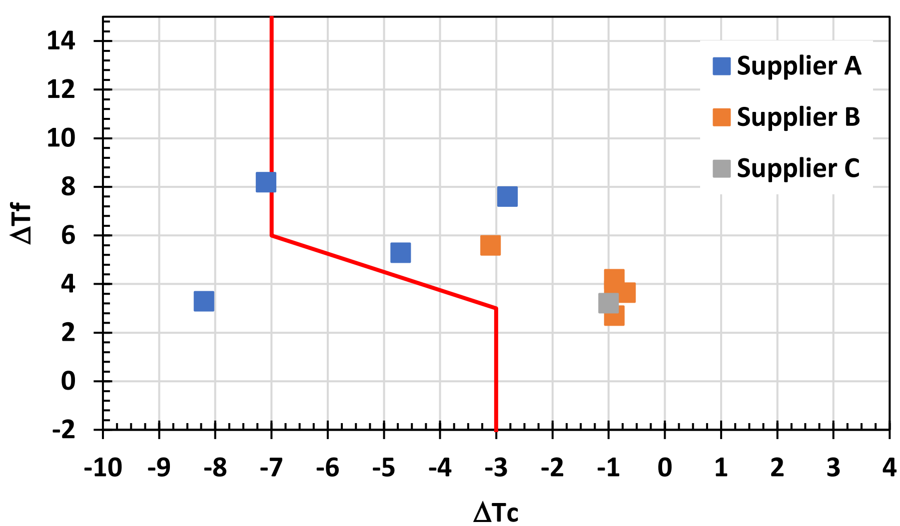


Figure 95 – NCHRP 9-60 Approach for 40 Hour PAV Conditioned Baseline Asphalt Binders

2.3 - Direct Parameter Comparison of "Same" PG Grade

The asphalt industry has a general perception that asphalt binders of the same PG grade have the same performance (i.e. – permanent deformation and cracking). However, with the recent technological advancements of material characterization, the asphalt industry understands that this is not necessarily the case. To demonstrate this, two different PG grades from the three asphalt binder suppliers were selected for comparison; PG58S-28 and PG58H-28. The asphalt binders were compared for the respective ΔTc, Glover-Rowe at 15°C, R-value @ 15°C and NCHRP 9-60 Approach characteristics, respectively. Asphalt binders graded out to a PG58S-28 are shown in blue while the PG58H-28 are shown in orange.

In Figure 96, the ΔTc values are shown and immediately one can observe differences between Supplier A and Supplier B and C. Not only are the ΔTc values more negative at the 20 hour PAV conditioning level, but the ΔTc values result in a more negative value after 40 hour PAV conditioning. This may be interpreted as the asphalt binder being supplied by Supplier A ages and loses its ductility at a greater rate than Supplier B and C's comparative PG grades. Figures 97 and 98 show the GRP_{15C} and R-value at 15°C respectively for the same PG grade. Similar to ΔTc, there are significant differences observed, especially for the PG58H-28 asphalt binders. Lastly, the asphalt binders were compared using the NCHRP 9-60 approach. The asphalt binders plot in a similar area after 20 hour PAV conditioning. However, after 40 hour PAV conditioning, Supplier A's PG58S-28 approaches close to the failing line while, the PG58H-28 transitions from a "Passing" material to a "Failing" material (Figures 99 and 100). Both Supplier B and C show "Passing" material at each long-term conditioning level. Again, this may be interpreted as the asphalt binder of Supplier A undergoes a higher degree of age hardening from the extra PAV conditioning when compared to Supplier B and C, even though they have the identical PG grade.

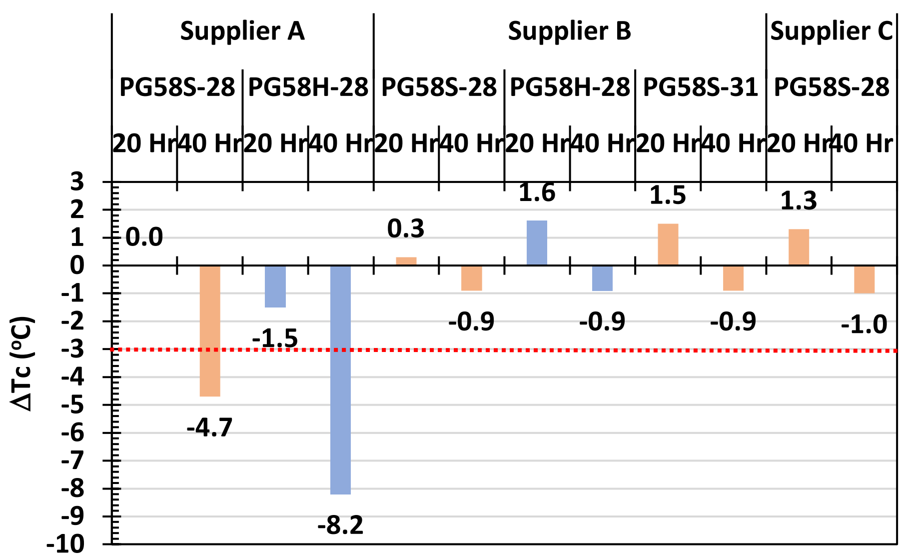


Figure 96 – ΔTc Comparison of Asphalt Binders of Same PG Grade

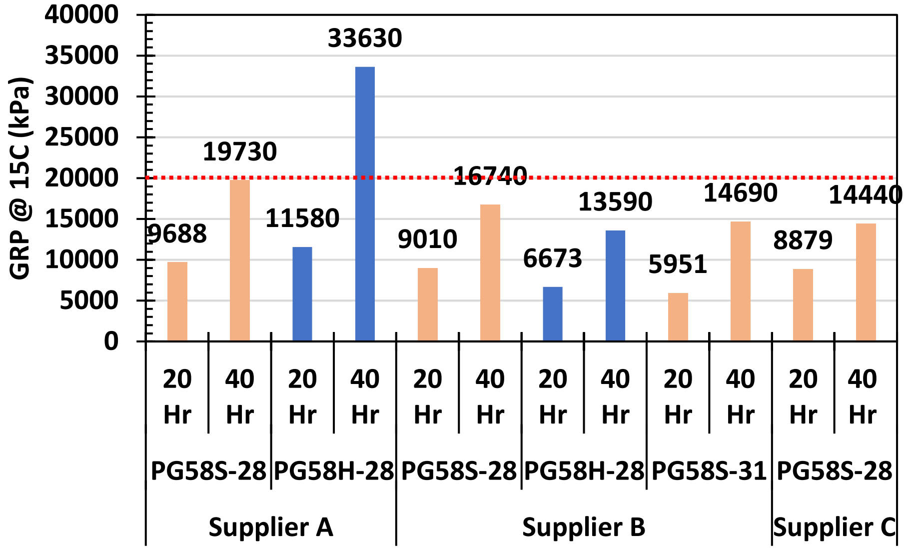


Figure 97 – Glover-Rowe Parameter at 15ºC for "Same" PG Graded Binders

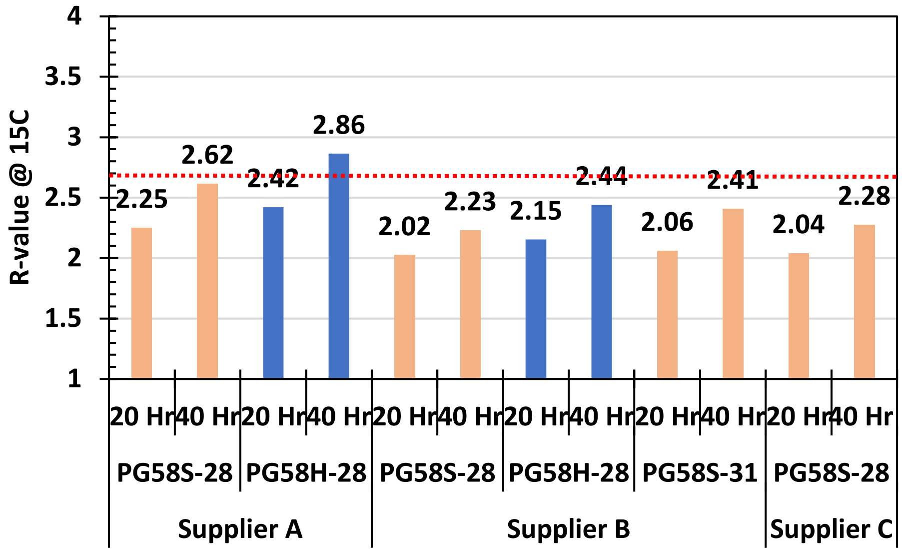


Figure 98 – DSR Measured R-value at 15ºC for "Same" PG Graded Binders

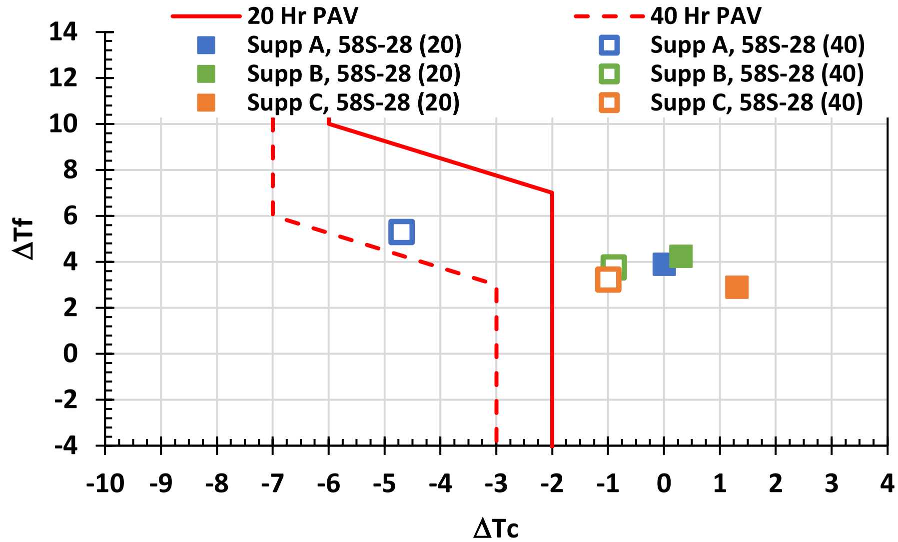


Figure 99 – NCHRP 9-60 Approach Evaluation for PG58S-28 Asphalt Binders

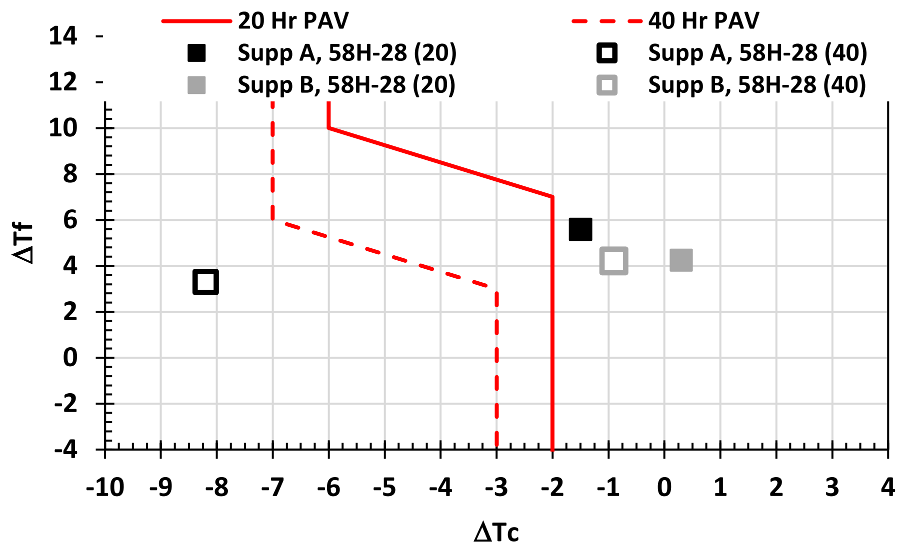


Figure 100 – NCHRP 9-60 Approach Evaluation for PG58H-28 Asphalt Binders

The previous figures showed that there are clear differences in some of the asphalt binder properties of binders with the "same" PG grade. However, it is unclear whether those differences would result in significantly different IDEAL-CT Index values. Using the relationships of recovered binder property to mixture IDEAL-CT Index developed in Phase 1, predicted IDEAL-CT Index values were calculated. The relationships between the mixture IDEAL-CT Index and ΔTc, GRP_{15C} and R-value at 15°C were determined and averaged for illustration purposes and shown in Figure 101. The predicted IDEAL-CT Index would represent the long-term aged mixture condition specified by WisDOT. The error bars shown in the figure represent the standard deviation above and below the average of the three asphalt binder parameters used in the predictions. The results in Figure 101 do indicate that differences in the measured IDEAL-CT Index values should be noticeable, especially for the PG58H-28 asphalt binder. However, the differences between the PG58S-28 asphalt binders from the different suppliers are within what is generally accepted variability of the IDEAL-CT Index. The results in Figure 101 are predictions representing the volumetrics and materials of the mixtures tested in Phase 1. The effective volume of asphalt binder (Vbe) plays a significant role in the cracking performance of the mixture, and therefore, Vbe outside of the range of mixtures tested would result in differences shown in Figure 101.

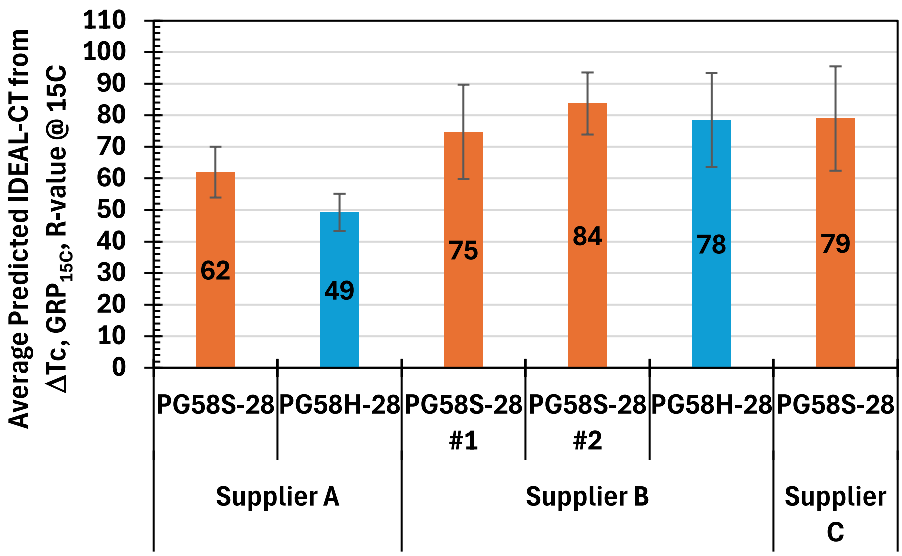


Figure 101 – Predicted IDEAL-CT Index Values from Averaging ΔTc, GRP_{15C} and R-value at 15°C Relationships

PHASE 3 – IMPACT OF RECYCLED ASPHALT ON PROPOSED WISCONSIN BINDER PARAMETERS

The purpose of the proposed specifications was to be included within a purchase specification to ensure good mixture cracking properties. However, during mixture production, Wisconsin mixture suppliers are allowed to utilize recycled asphalt pavement (RAP) in their mixtures. Therefore, it was important to understand how sensitive the proposed parameters are to the addition of recycled asphalt, and if the proposed parameters could be used to identify limits where excessive RAP resulted in poor IDEAL-CT Index values (i.e. – less than 30.0).

To evaluate the impact of RAP on the proposed asphalt binder parameters, RAP from a Wisconsin source was extracted and recovered. A significant quantity of recovered RAP binder was mixed and blended to ensure uniformity during the research phase. Afterwards, the recovered RAP binder was blended to the PG58S-28 and PG58H-28 asphalt binders from Phase 2 at the following percentage per total weight of the asphalt binder; 0, 25, 35, and 45%. The RAP binder graded out to a PG58E-16 with the recovered asphalt binder not capable of meeting the elastomer line, clearly indicating no elastomeric properties in the "E" graded asphalt binder. At the 20 hour PAV conditioning, the RAP binder had a ΔTc value of -3.6°C. Previous research conducted by Bonaquist (2016) noted that the asphalt binder properties of the RAP materials in Wisconsin are fairly uniform, and therefore, the source of RAP asphalt binder used in this study should be a representative RAP binder for the state.

Figures 102 to 104 show the impact of RAP content on the ΔTc parameter. Overall, since the ΔTc value of the RAP is more negative than the asphalt binder at the same aged condition, as the RAP content in the blended asphalt binder increases, the ΔTc will converge towards the value of the RAP. If the ΔTc value of the asphalt binder is relatively close to that of the RAP, minimal changes will occur. This is illustrated by comparing the general ΔTc trend between Asphalt Supplier #1 (Figure 102) and #2 (Figure 103). Asphalt Supplier #1 had PG58S-28 and 58H-28 that had a ΔTc value closer to the magnitude of the RAP. Therefore, only small changes in the blended ΔTc value are observed. Meanwhile, Asphalt Supplier #2 had a more positive ΔTc value, and therefore, larger changes are observed as RAP content increases.

Using a ΔTc minimum value of -3°C based on the IDEAL-CT Index of 30.0, the results show that RAP content up to 45% would provide adequate asphalt binders to achieve the required long-term conditioned IDEAL-CT Index value for all asphalt binder sources tested. The mixture long-term conditioning was found to mirror the 20 hour PAV conditioning used for asphalt binders. It is not until the asphalt binders are 40 hour PAV conditioned that the resultant ΔTc fails the -3°C threshold.

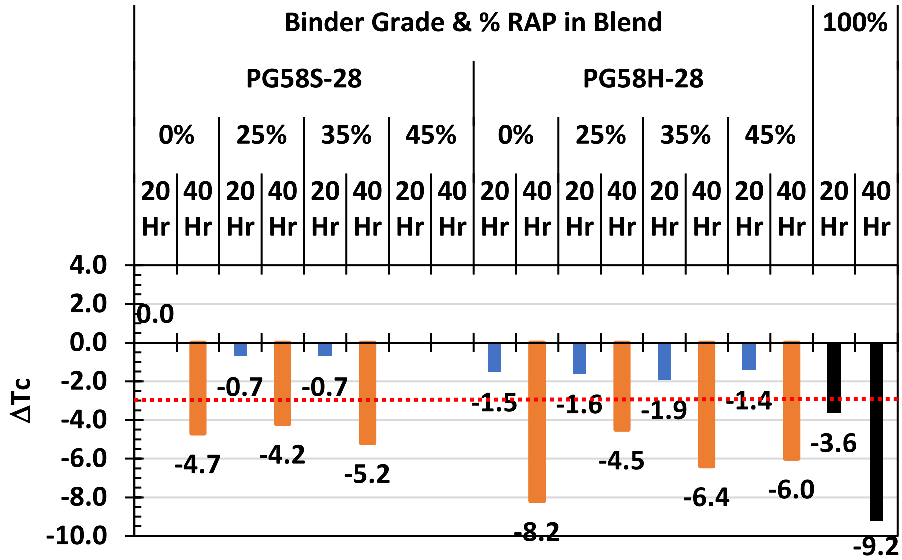


Figure 102 – Measured ΔTc Values for Asphalt Supplier #1 with Varying Percentages of RAP

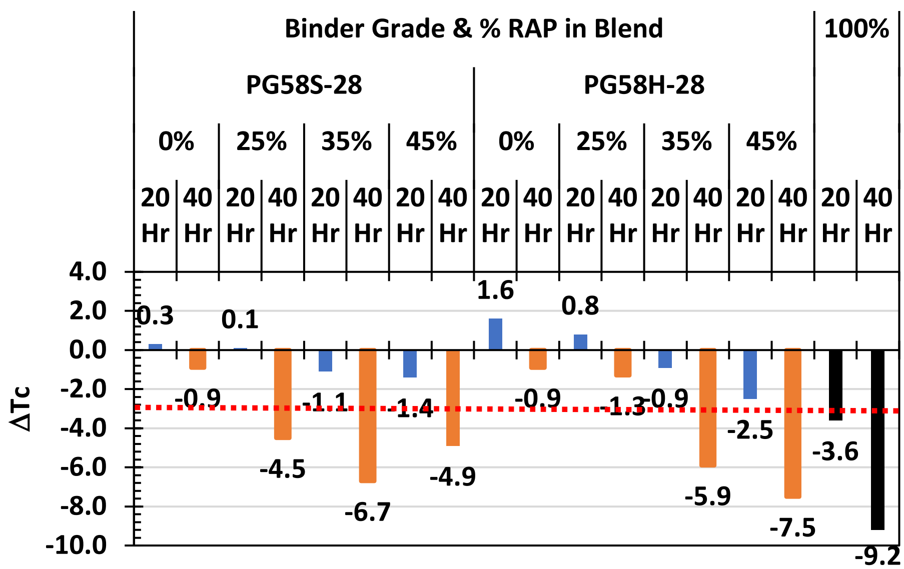


Figure 103 - Measured ΔTc Values for Asphalt Supplier #2 with Varying Percentages of RAP

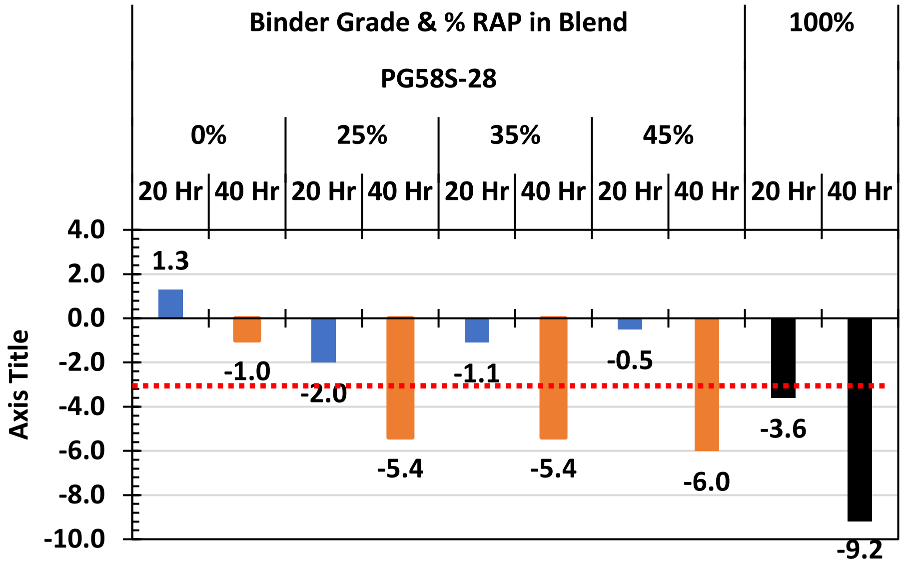


Figure 104 - Measured ΔTc Values for Asphalt Supplier #3 with Varying Percentages of RAP

The Glover Rowe Parameter at 15°C and 10 radians per second (GRP_{15C}) was measured on the same asphalt binder-RAP blends as previously described. The results are shown in Figures 105 through 107 for Asphalt Suppliers #1 to #3, respectively. Using the GRP_{15C} criteria of 20,000 kPa (Log GRP_{15C} = 4.3 kPa) to match an IDEAL-CT Index of 30.0, the figures show that on average, once the RAP content exceeded 35%, the asphalt binder would fail the GRP_{15C} criteria, and therefore, would be susceptible to failing the mixture IDEAL-CT Index criteria. Based on how the GRP_{15C} responds to changes in the dosage rate of the RAP binder, it is far more sensitive to changes in aged asphalt binder than ΔTc.

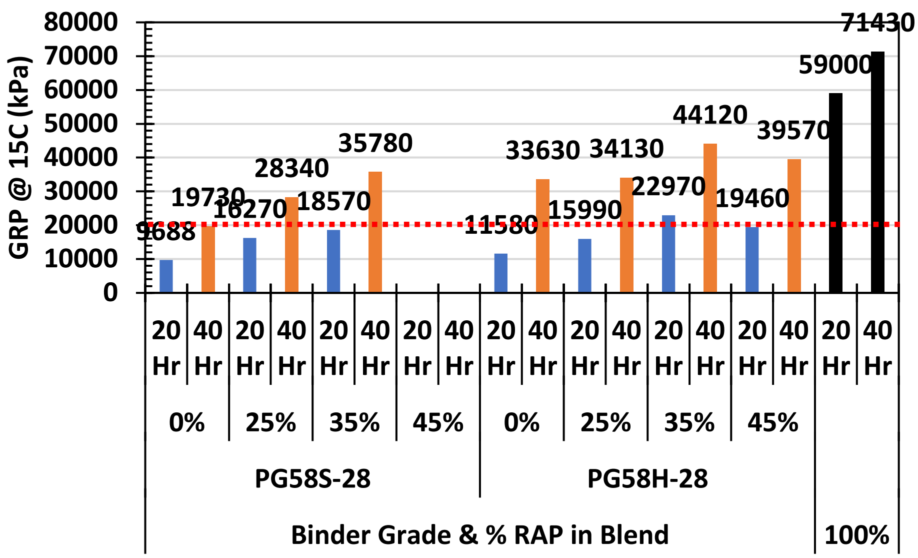


Figure 105 – Measured GRP_{15C} Values for Asphalt Supplier #1 with Varying Percentages of RAP

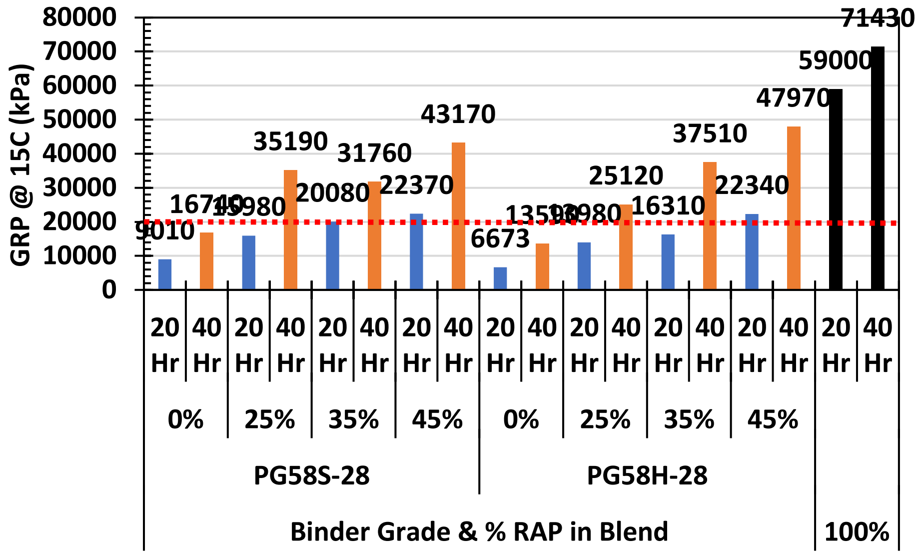


Figure 106 – Measured GRP$_{15C}$ Values for Asphalt Supplier #2 with Varying Percentages of RAP

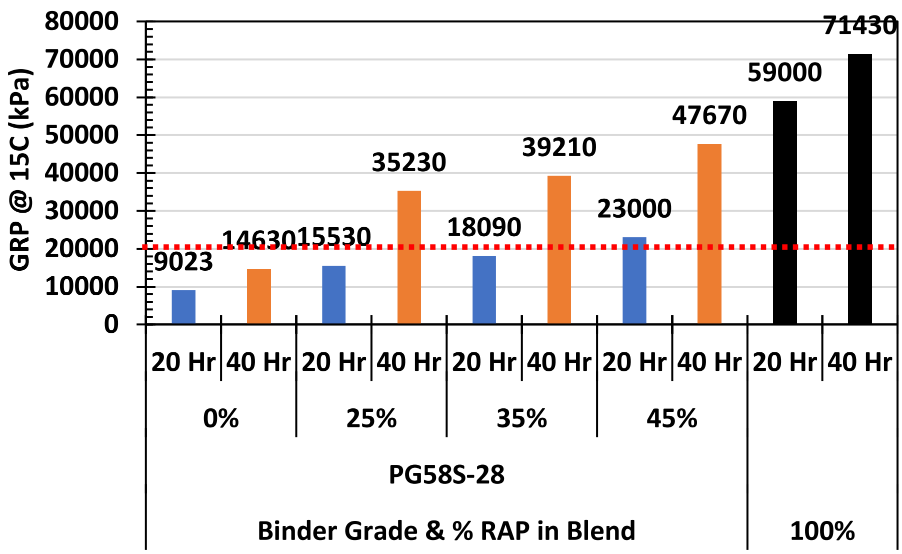


Figure 107 – Measured GRP$_{15C}$ Values for Asphalt Supplier #3 with Varying Percentages of RAP

Similar to the GRP_{15C}, the R-value at 15°C was determined for each asphalt binder supplier, binder grade, and RAP content. The results are shown in Figures 108 to 110. The R-value was not as sensitive to RAP content as the GRP_{15C}. This appears to be very similar to ΔTc, which is a rheological Shape parameter as well. The same can be said regarding the sensitivity of the Phase Angle at a Constant G* = 10 MPa (δ_{10MPa}), as shown in Figures 111 to 113.

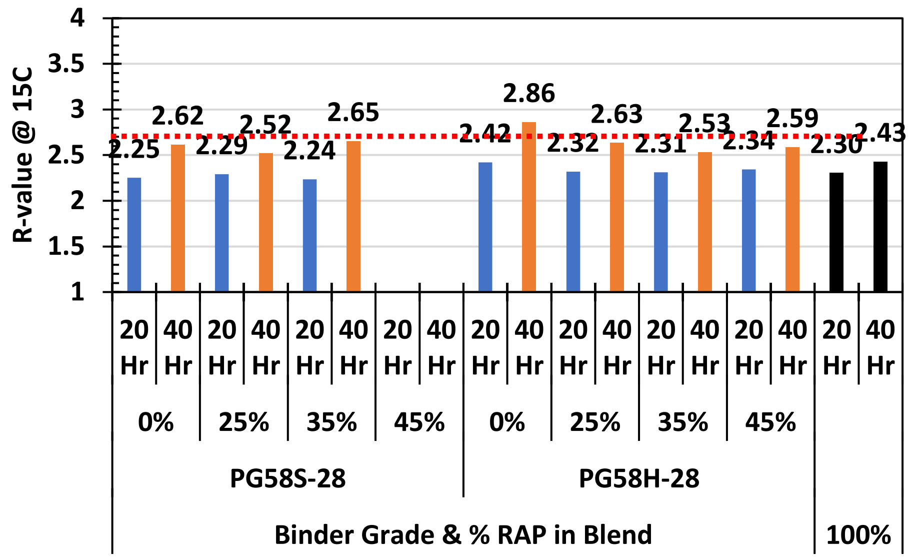


Figure 108 - Measured R-value at 15C Values for Asphalt Supplier #1 with Varying Percentages of RAP

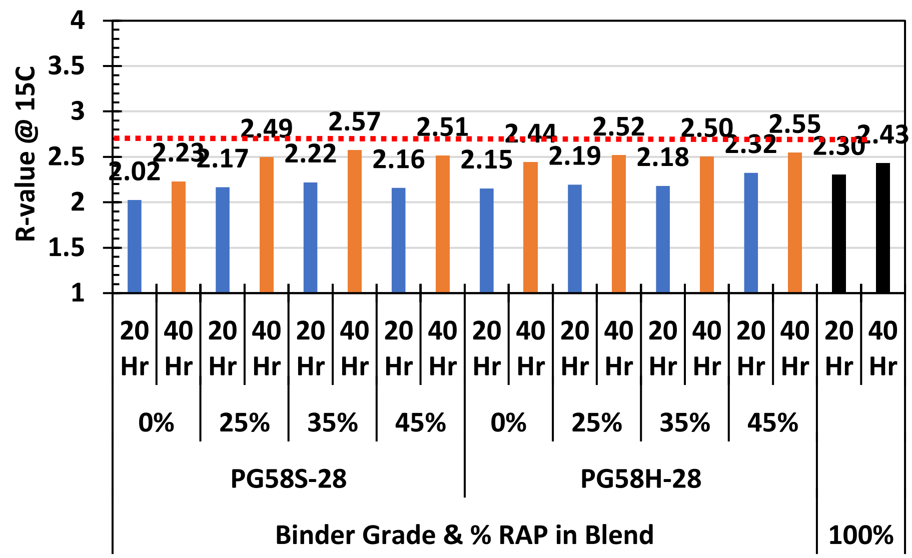


Figure 109 - Measured R-value at 15C Values for Asphalt Supplier #2 with Varying Percentages of RAP

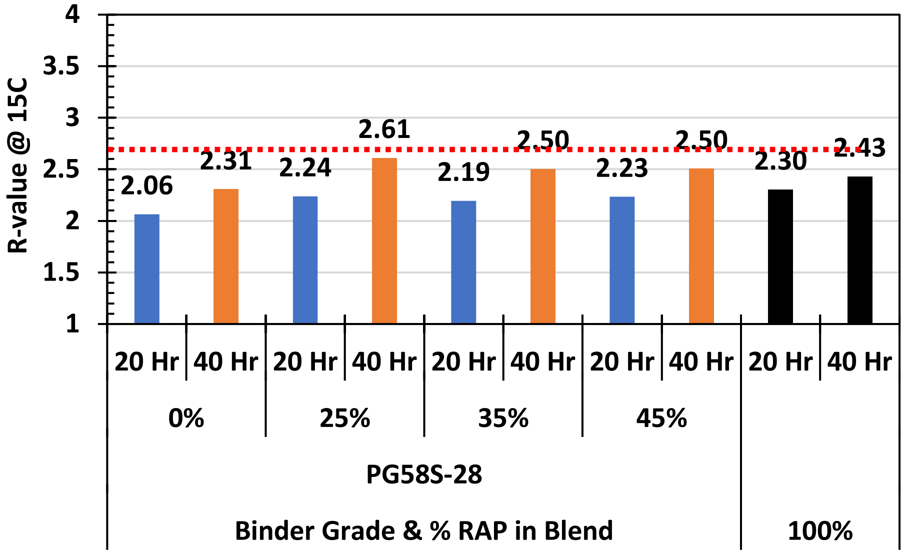


Figure 110 – Measured R-value at 15C Values for Asphalt Supplier #3 with Varying Percentages of RAP

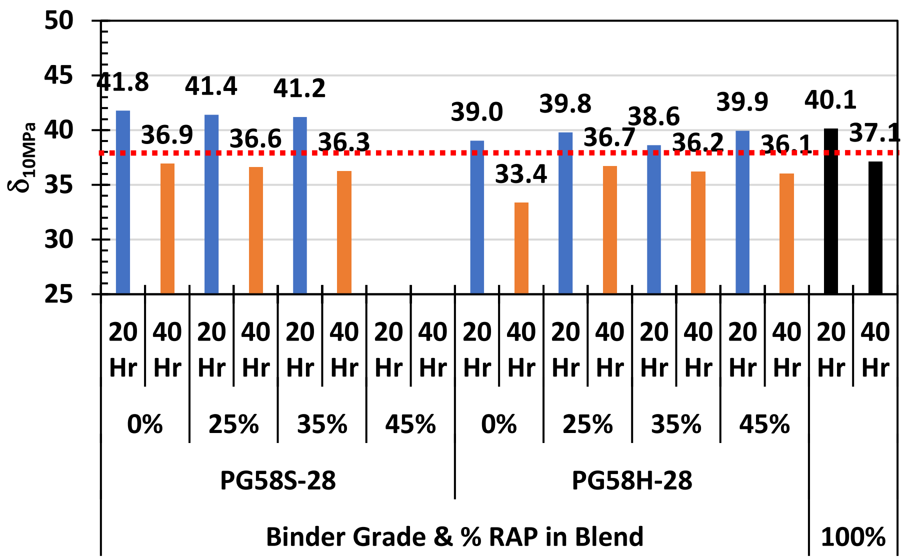


Figure 111 – δ_{10MPa} Values for Asphalt Supplier #1 with Varying Percentages of RAP

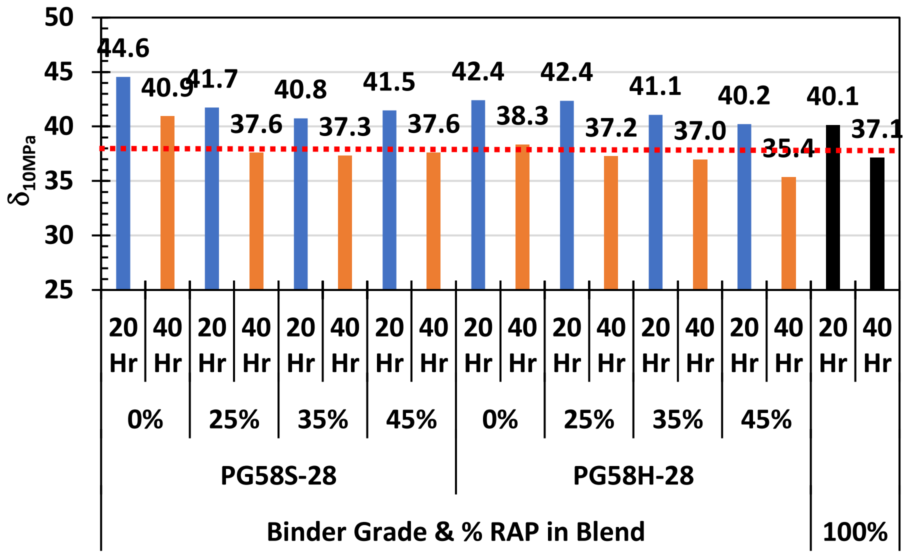


Figure 112 – δ_{10MPa} Values for Asphalt Supplier #2 with Varying Percentages of RAP

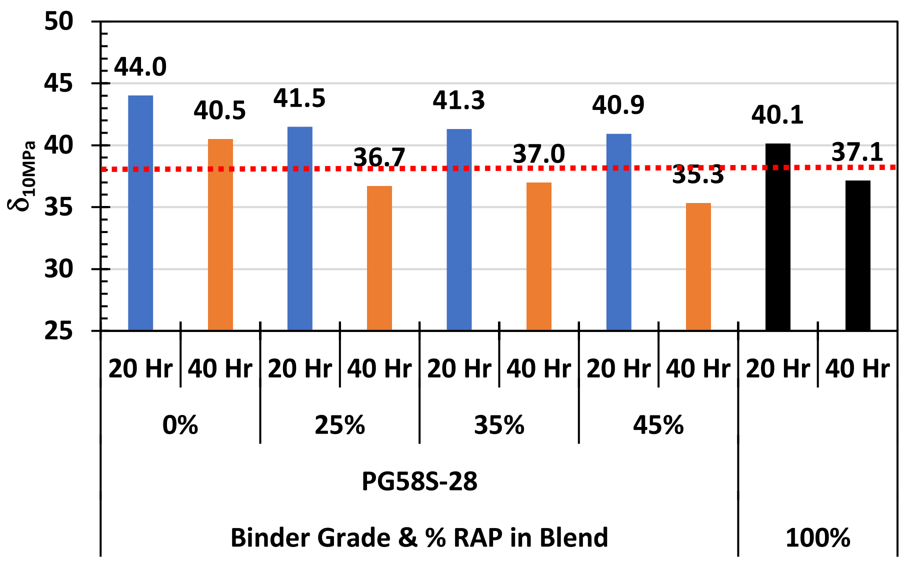


Figure 113 – δ_{10MPa} Values for Asphalt Supplier #3 with Varying Percentages of RAP

To further emphasize the concept that rheological Point parameters are more sensitive to RAP content changes than rheological Shape parameter, the BBR Low Temperature PG Grade using the m-value, which is defined as a Point parameter similar to the GRP_{15C}, Figures 114 to 116 were generated. As shown in the figures, the Low Temperature PG grade as determined by the m-value clearly gets worse (i.e. – warmer) as RAP content increases. Therefore, it appears that the impact of RAP content on rheological Shape parameters is not as sensitive and intuitive as observed with the rheological Point parameters used in this study.

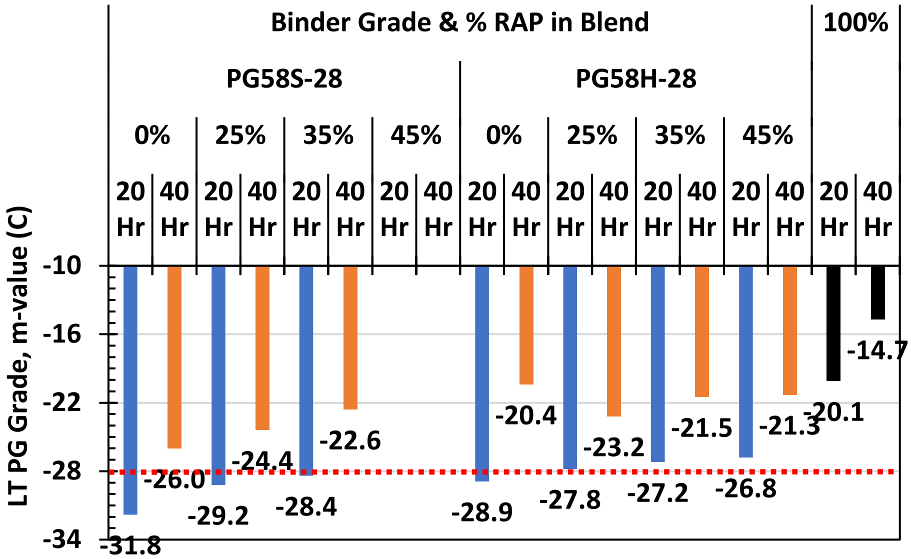


Figure 114 – Low Temperature PG Grade (m-value) Values for Asphalt Supplier #1 with Varying Percentages of RAP

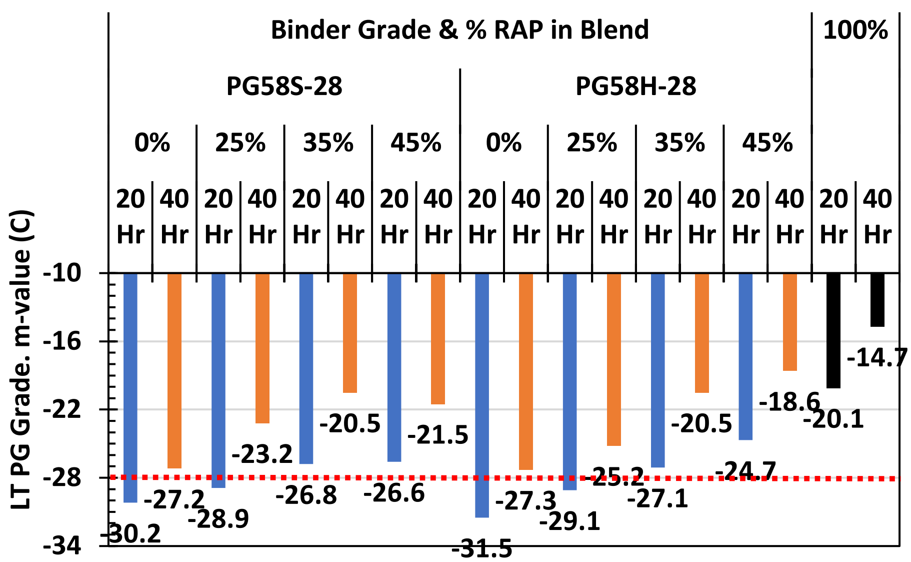


Figure 115 – Low Temperature PG Grade (m-value) Values for Asphalt Supplier #2 with Varying Percentages of RAP

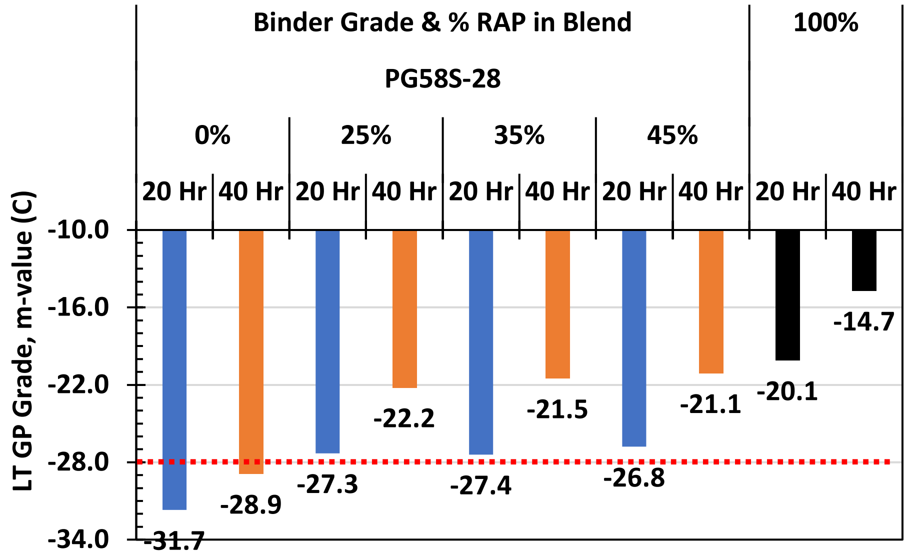


Figure 116 – Low Temperature PG Grade (m-value) Values for Asphalt Supplier #3 with Varying Percentages of RAP

The asphalt binder fracture testing using the Asphalt Binder Cracking Device (ABCD) was conducted for each asphalt binder supplier, PG Grade and RAP content. The results can be found as Figures 117 to 119. The figures present the Critical Cracking Temperature (Tcr) as measured by the ABCD test method. The trend in Tcr is similar to the Point Parameters whereas RAP content increases for the respective asphalt binder supplier and PG graded binder, the Tcr decreases, identifying the asphalt binder is more prone to cracking distress. Although only a moderate correlation was found, the ABCD Tcr value of -34°C correlated to an IDEAL-CT Index of 30.0. The results in Figures 117 to 119 show that only the 0% RAP asphalt binder at the 20 hour PAV conditioning met the proposed Tcr value. Clearly noticeable in the figures is the poor performance of the RAP binder itself with respect to the critical cracking temperature.

The ABCD Tcr parameter was also used within the NCHRP 9-60 Approach that combines the ΔTc measurement with the Tcr normalized using the BBR Low Temperature PG Grade Stiffness, called ΔTf. The results are shown in Figures 120 through 122. The NCHRP 9-60 Approach resulted in almost all of the 20 hour PAV asphalt binders to be classified as good or PASSING, besides the 100% RAP binder and the 45% RAP PG58H-28 asphalt binder from Asphalt Supplier #2. It was not until the asphalt binders were 40 hour PAV conditioned when failing results for a majority of the asphalt binders were classified as FAILING. The NCHRP 9-60 Approach is highly dependent on the ΔTc parameter, so if ΔTc is not sensitive enough to pick up the changes in RAP content, the NCHRP 9-60 Approach will most likely be insensitive as well.

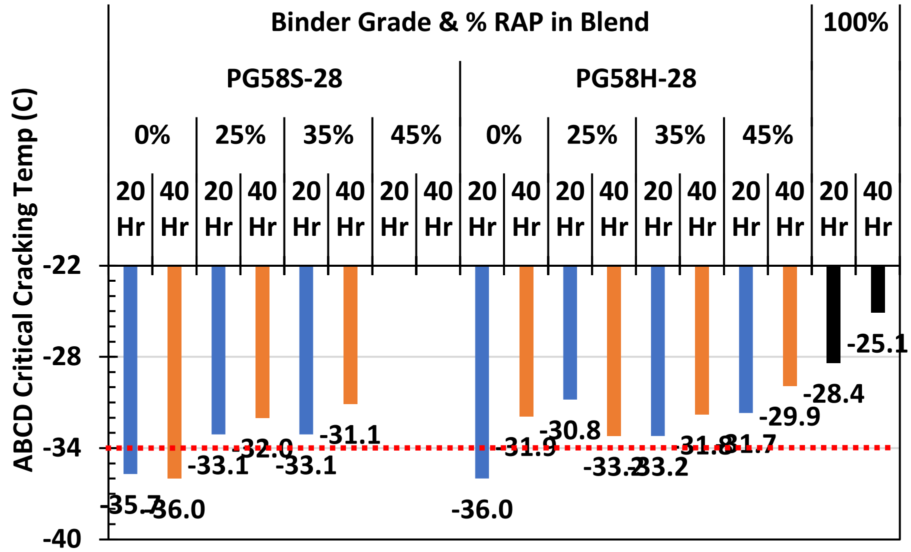


Figure 117 – Asphalt Binder Cracking Device Critical Cracking Temperature Values for Asphalt Supplier #1 with Varying Percentages of RAP

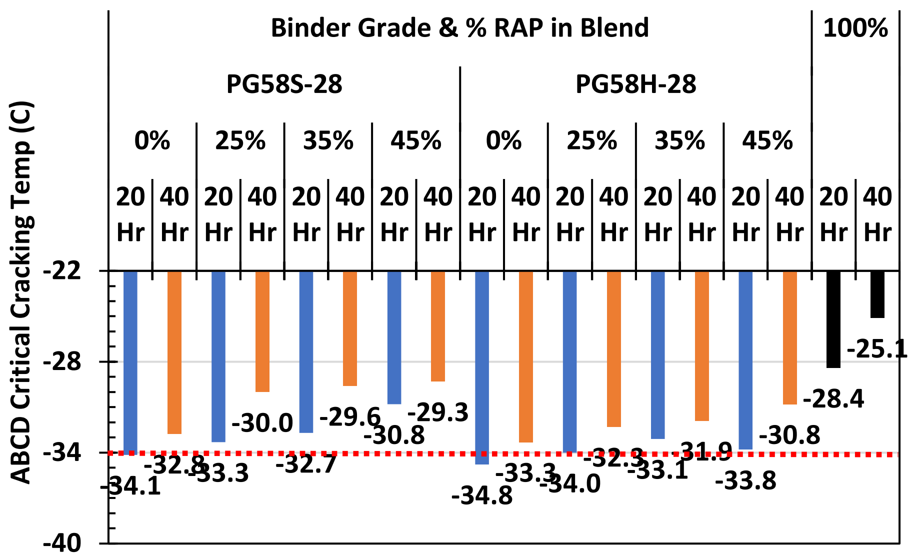


Figure 118 – Asphalt Binder Cracking Device Critical Cracking Temperature Values for Asphalt Supplier #2 with Varying Percentages of RAP

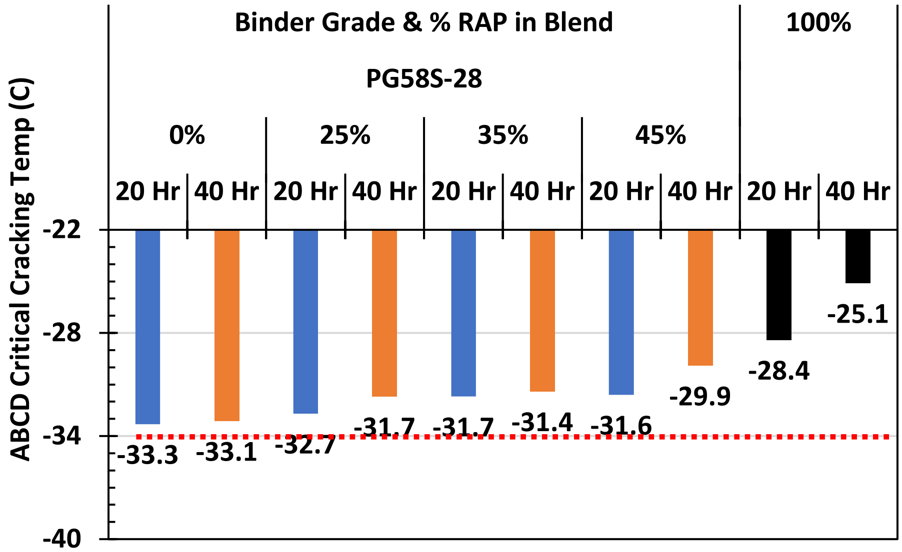
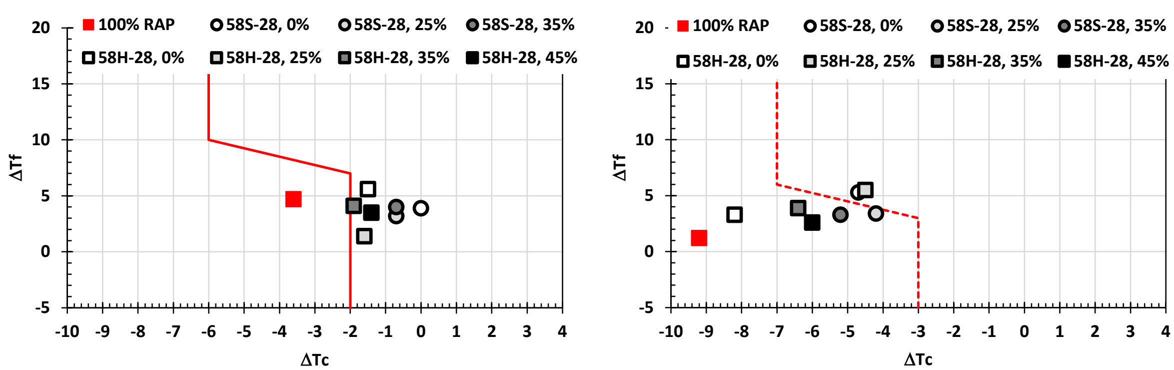
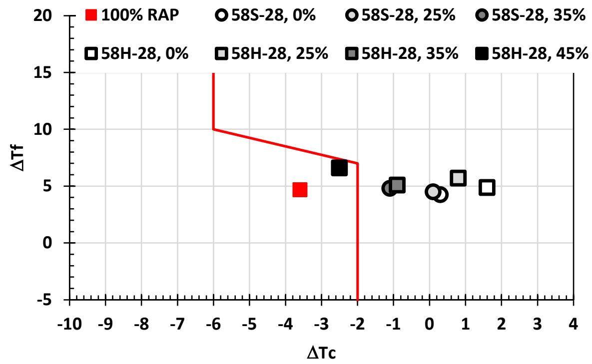


Figure 119 – Asphalt Binder Cracking Device Critical Cracking Temperature Values for Asphalt Supplier #3 with Varying Percentages of RAP

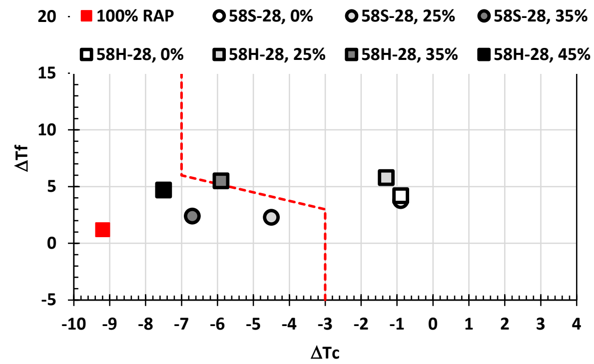


(a) (b)

Figure 120 – NCHRP 9-60 Approach Values for Asphalt Supplier #1 with Varying Percentages of RAP; a) 20 Hour PAV Conditioned; b) 40 Hour PAV Conditioned

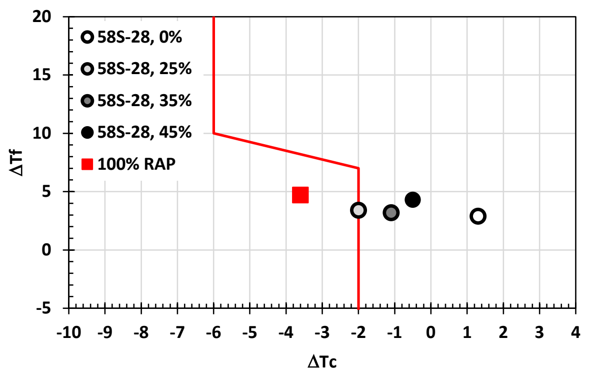


(a)

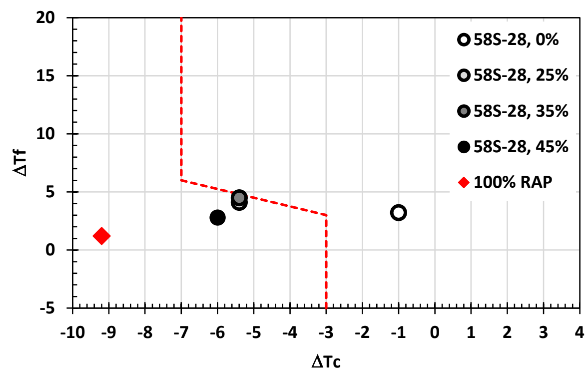


(b)

Figure 121 – NCHRP 9-60 Approach Values for Asphalt Supplier #2 with Varying Percentages of RAP; a) 20 Hour PAV Conditioned; b) 40 Hour PAV Conditioned NCHRP



(a)



(b)

Figure 122 - NCHRP 9-60 Approach Values for Asphalt Supplier #3 with Varying Percentages of RAP; a) 20 Hour PAV Conditioned; b) 40 Hour PAV Conditioned NCHRP

PHASE 4 – ASPHALT BINDER CHARACTERIZATION OF WISCONSIN'S BALANCED MIXTURE DESIGN (BMD) TEST SECTIONS

Whenever establishing a laboratory-based test criteria, asphalt binder or mixture, the ultimate goal is to correlate that laboratory measured property to measured field distress. To help validate the recently proposed Balanced Mixture Design (BMD) mixture performance test criteria, the Wisconsin Department of Transportation (WisDOT) established the WisDOT BMD Test Sections. And even though the intent of the WisDOT BMD Test Sections was to compare field performance to mixture test results, it provides an excellent means of also comparing the proposed asphalt binder protocols to both the mixture test results developed for the test sections, and ultimately, the field performance which will be heavily monitored over the next few years.

4.1 - WisDOT BMD Test Section Mixture Test Results

The WisDOT BMD Test Sections consist of six pavement sections where the surface course is designed for different levels of rutting and fatigue cracking resistance using the test results of the Hamburg Wheel Tracking test and the IDEAL-CT Index tests, respectively. Table 15 shows the design parameters for the different test sections.

Table 15 – Contractor Mix Design Information for Different WisDOT Test Sections

Test Section	CT_{Index}	HWTT Corr. Rut Depth @20k (mm)	Asphalt Content (%)	Binder Grade	Reclaimed Asphalt Pavement (RAP)Content (%)
1	69	10.4	6.5	58-28 S	8
2	99	3.3	6.3	58-28 S	15
3	29	8.1	6.0	58-28 S	0
4	21	2.8	5.3	58-28 S	27
5	56	3.7	6.5	58-28 V	8
6	17	3.2	6.0	58-28 V	0

During production, testing of the different mixes were performed to verify the plant produced asphalt mixtures met the design. The comparison between the mix design and plant produced mixture results are shown in Table 16 and Figure 123 for the IDEAL-CT Index and Table 17 and Figure 124 for the HWTT rutting, respectively. There were some differences between the designed performance and production performance of both the rutting and cracking tests. In some cases, such as Section #2, the plant produced fatigue cracking performance was 50% of the intended mixture design.

Interesting to note that when plotting the volumetrics measured at the plant during quality control testing, the Voids Filled with Asphalt (VFA) had a strong correlation to the IDEAL-CT Index

(Figure 125) while the Voids Filled with Asphalt (VMA) correlated to Hamburg Wheel Track rutting (Figure 126).

Table 16 – Fatigue Cracking Performance Using the IDEAL-CT Index for Both Mix Design and Plant Produced Asphalt Mixtures

Section	QC Test Results - IDEAL-CT Index					Mix Design Results	VMA	VFA
	WisDOT	Walbec	NCAT	Ave	Std Dev			
#1	60	80	51	63.7	14.8	69	15.3	82.4
#2	37	59	38	44.7	12.4	99	14.4	77.1
#3	42	42	33	39.0	5.2	29	16.4	76.2
#4	24	20	22	22.0	2.0	21	13.6	71.3
#5	79	71	63	71.0	8.0	56	15.5	81.3
#6	38	32	28	32.7	5.0	17	15.7	72.6

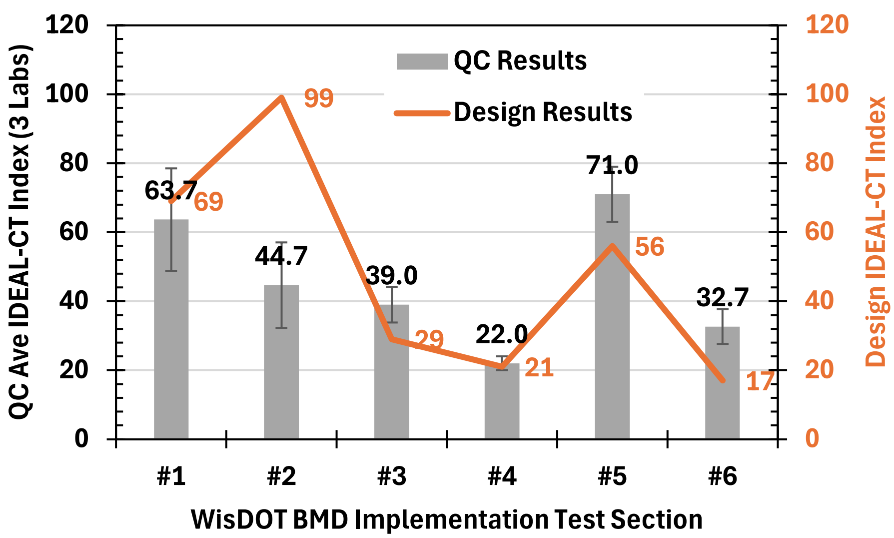


Figure 123 - Fatigue Cracking Performance Using the IDEAL-CT Index for Both Mix Design and Plant Produced Asphalt Mixtures

Table 17 – Rutting Performance Using the Hamburg Wheel Tracking Test for Both Mix Design and Plant Produced Asphalt Mixtures

Section	QC Test Results - HWT Rutting @ 20k Cycles (mm)					Mix Design Results	VMA	VFA
	WisDOT	Walbec	NCAT	Ave	Std Dev			
#1	6.5	12.2	14.4	11.0	4.1	7.5	15.3	82.4
#2	3.4	5.5	6.4	5.1	1.5	3.4	14.4	77.1
#3	6.6	10.5	13.8	10.3	3.6	8.1	16.4	76.2
#4	3.0	4.4	7.1	4.8	2.1	2.8	13.6	71.3
#5	5.8	8.7	11	8.5	2.6	3.7	15.5	81.3
#6	5.2	8.7	12.4	8.8	3.6	3.2	15.7	72.6

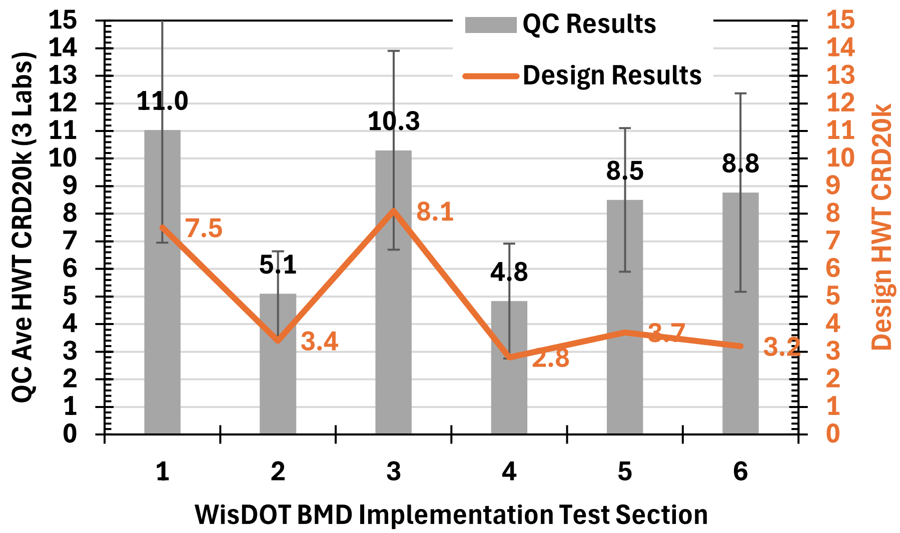
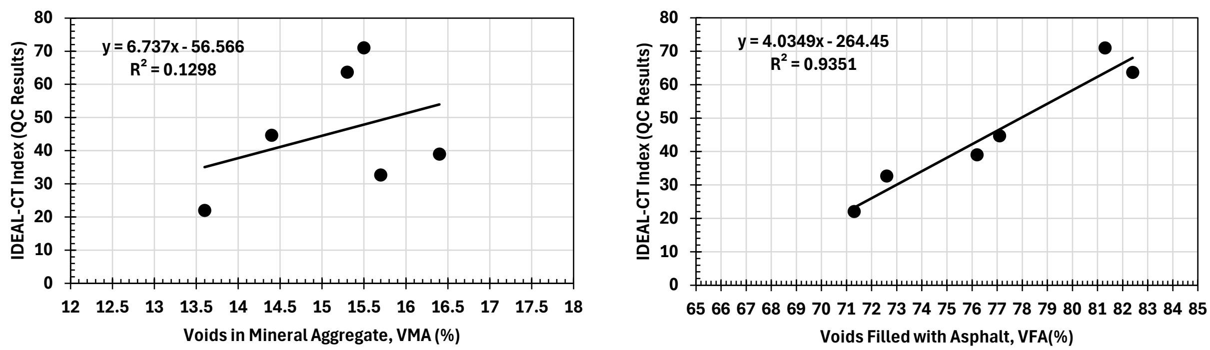
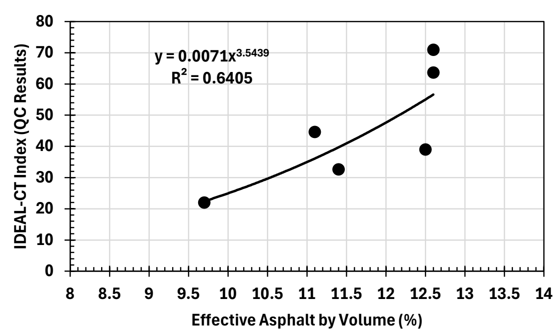


Figure 124 – Rutting Performance Using the Hamburg Wheel Tracking Test for Both Mix Design and Plant Produced Asphalt Mixtures

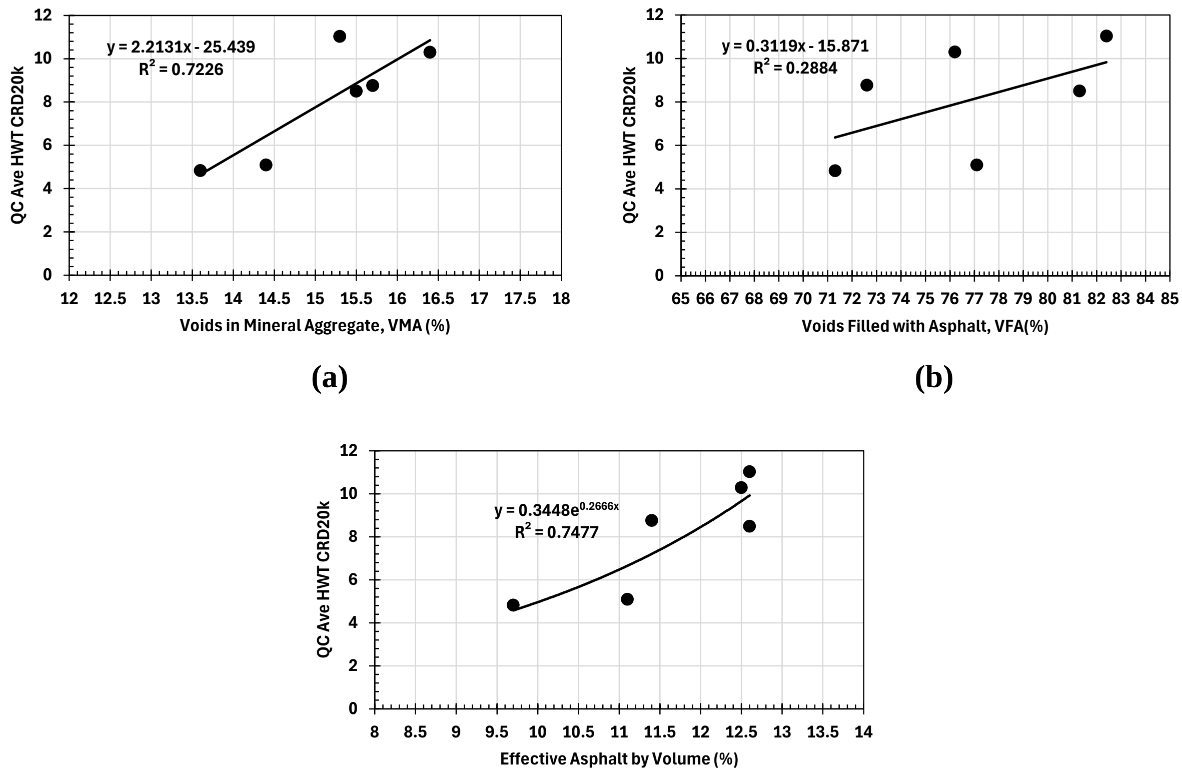


(a) **(b)**



(c)

Figure 125 – Key Mixture Volumetrics Compared to IDEAL-CT Index Cracking Performance; a) Voids in Mineral Aggregate (VMA); b) Voids Filled with Asphalt (VFA); c) Effective Asphalt by Volume (Vbe)



(a) **(b)**

(c)

Figure 126 - Key Mixture Volumetrics Compared to Hamburg Wheel Tracking Rutting Performance; a) Voids in Mineral Aggregate (VMA); b) Voids Filled with Asphalt (VFA); c) Effective Asphalt by Volume (Vbe)

4.2 - Test Results of Recovered Asphalt Binder from WisDOT BMD Test Sections

Asphalt mixture from each of the test sections was provided by The Walbec Group. The loose mix was sampled during production and stored in sample boxes at their QC laboratory. The sample boxes were shipped to the Rutgers Asphalt Pavement Laboratory (RAPL) where the asphalt binder from the different test sections were recovered using the solvent extraction and recovery process described earlier in the report. Once recovered, the asphalt binder was tested at two aged conditions; 1) As-Received to represent the condition of the asphalt binder at the time of construction, and 2) 20 hour PAV conditioned to simulate the long-term field conditioning and the approximate asphalt binder aged condition of the WisDOT long term mix aging for IDEAL-CT Index specimens. The PG Grade properties of each of the recovered asphalt binders and at each conditioning level are shown in Table 18.

Table 18 – PG Grade Properties of Recovered Asphalt Binders from WisDOT BMD Test Sections

BMD Test Section	Asphalt Content (%)		Design PG Grade	Design RAP %	Condition	HT PG			Z-Factor	Int PG	LT PG			LT PG
	JMF	Rec.				PG	Jnr	% Rec			m	S	ΔTc	
#1	6.5	6.3	58S-28	8	As-Rec.	58.3	4.253	-0.1	-20.2	10.9	-38.6	-36.5	2.1	-36.5
					20 Hr PAV	77.9	0.174	30.3	-16.2	20.6	-29.2	-29.6	-0.4	-29.2
#2	6.0	5.9	58S-28	15	As-Rec.	66.9	1.266	3.2	-24.4	16.9	-32.4	-30.6	1.8	-30.6
					20 Hr PAV	78.3	0.169	29.9	-17.0	21.8	-28.1	-28.7	-0.6	-28.1
#3	6.0	6.0	58S-28	0	As-Rec.	63.8	1.892	1.5	-23.3	15.3	-33.9	-32.1	1.8	-32.1
					20 Hr PAV	75.4	0.263	23.4	-18.3	20.4	-30.1	-30.1	0.0	-30.1
#4	5.3	5.3	58S-28	27	As-Rec.	71.1	0.605	9.5	-24.0	18.7	-30.6	-29.2	1.4	-29.2
					20 Hr PAV	82.4	0.076	41.9	-15.9	25.4	-25.6	-27.3	-1.7	-25.6
#5	6.5	6.1	58V-28	8	As-Rec.	67.2	0.813	26.5	-4.5	15.2	-33.0	-32.8	0.2	-32.8
					20 Hr PAV	81.5	0.047	65.1	-0.5	20	-27.2	-30.1	-2.9	-27.2
#6	6.0	6.1	58V-28	0	As-Rec.	67.0	0.839	26.6	-4.2	12.7	-33.7	-33.8	-0.1	-33.7
					20 Hr PAV	82.5	0.041	70.0	1.8	19.9	-26.8	-30.7	-3.9	-26.8

The recovered asphalt binders were evaluated using the proposed test protocols and respective criteria, as well as some of the other asphalt binder indices evaluated throughout the study.

No mixture testing was conducted in this phase of the study. Comparisons to mixture test results are based on the average QC values recorded.

4.3 - ΔTc and NCHRP 9-60 Approach

The ΔTc and the NCHRP 9-60 Approach were used to characterize the recovered asphalt binder from the different asphalt mixtures of the WisDOT BMD Test Sections. The ΔTc results are shown in Figure 127 at both aged conditions with the relationship between ΔTc and IDEAL-CT Index shown in Figure 128. The As-Received condition shows that most of the recovered asphalt binders had a positive ΔTc and it was not until 20 hour PAV conditioning until some of the recovered asphalt binders were measured to have a negative ΔTc. Figure 128 shows a poor correlation between the 20 hour PAV conditioned recovered asphalt binders and IDEAL-CT Index.

Figure 129 shows the Asphalt Binder Cracking Device (ABCD) Critical Cracking Temperature (Tcr) for the recovered asphalt binder at As-Received and 20 Hour PAV conditioned levels. Figure 130 shows a moderate relationship between the ABCD Tcr and the measured IDEAL-CT Index. Lastly, the NCHRP 9-60 Approach is shown in Figure 131. The results are somewhat mixed as the methodology captured the marginal cracking performance of Section #6, which was the 2nd worst mixture performer, but also classified Section #5 as "Failing", even though it was the best IDEAL-CT Index mixture placed on the WisDOT BMD Test Sections. Figure 131 includes the test section and respective IDEAL-CT Index value within the figure.

Sections #5 and #6 utilized polymer modified binder while Sections #1 to #4 were unmodified. In addition, the PG58V-28 used in Sections #5 and #6 were provided by a different asphalt binder supplier than the PG58S-28 in Sections #1 through #4.

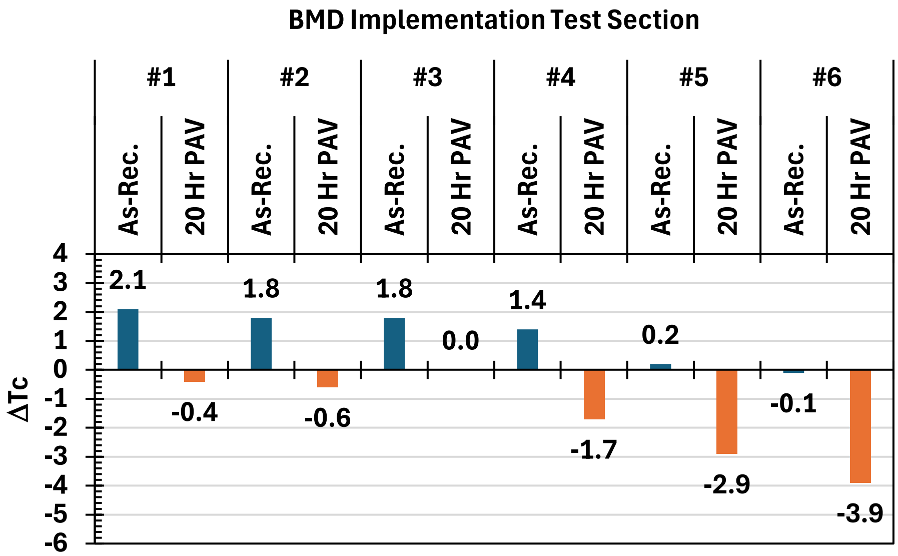


Figure 127 – Recovered Asphalt Binder ΔTc Measured Values for WisDOT BMD Test Section Asphalt Mixes

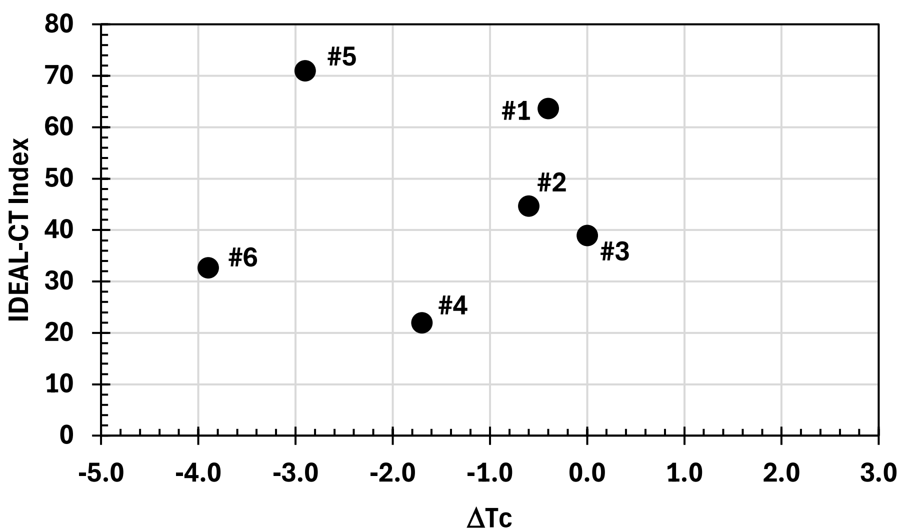


Figure 128 – Recovered Asphalt Binder ΔTc Measured Values Compared to Measured IDEAL-CT Index (data point label indicates test section #)

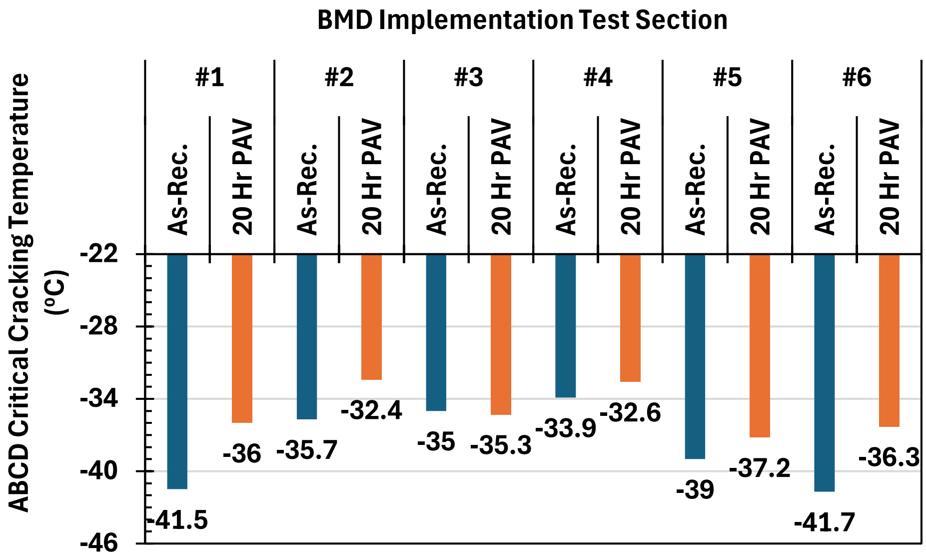


Figure 129 – Recovered Asphalt Binder ABCD Critical Cracking Temperature Measured Values for WisDOT BMD Test Section Asphalt Mixes

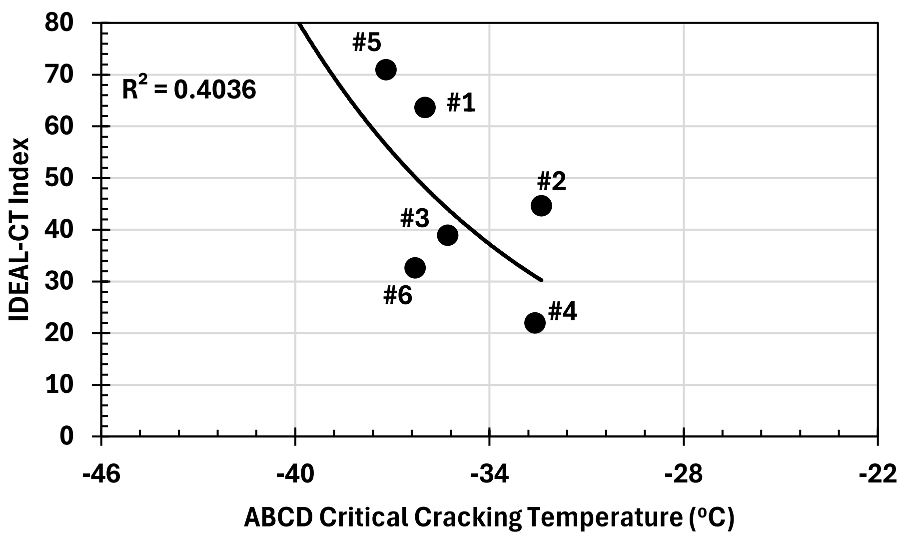


Figure 130 – Recovered Asphalt Binder ABCD Critical Cracking Temperature Compared to Measured IDEAL-CT Index (data point label indicates test section #)

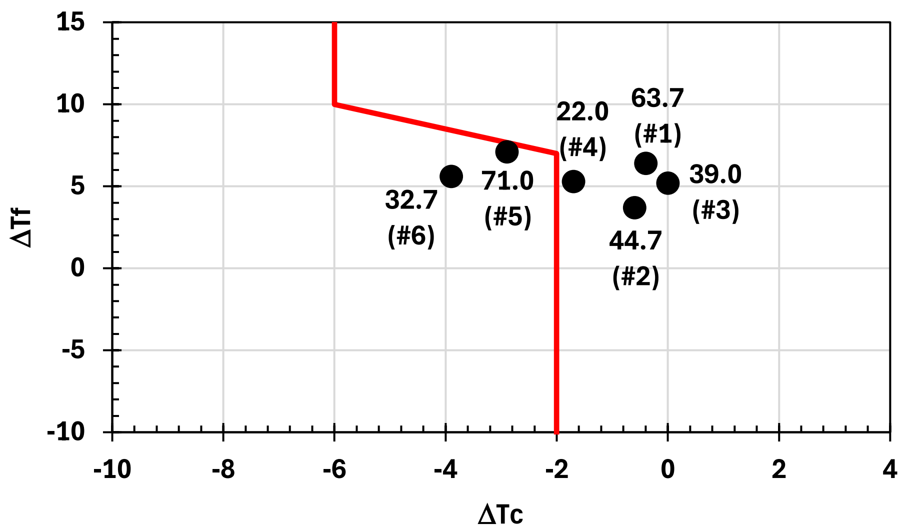


Figure 131 – NCHRP 9-60 Approach for WisDOT BMD Test Sections Recovered Asphalt Binder (data point label indicates CT Index result and test section #)

4.4 - Proposed WisDOT Binder Protocol – GRP_{15C} and R-value at 15C

The research study recommended the use of the Glover-Rowe Parameter (GRP) and the Rheological Index (R-value) at 15°C using the DSR at 10 radians per second. The calculated GRP at 15 and R-value at 15°C was shown to be sensitive to asphalt binder modification and aged condition and criteria was developed based on an IDEAL-CT Index of 30.0 after long term conditioning.

Figure 132 shows the results of the GRP_{15C} for the As-Received and 20 Hour PAV conditioned recovered asphalt binders. The results show that the GRP_{15C} was able to identify Section #4 as being the worst IDEAL-CT performing asphalt mixtures. The R-value at 15°C is shown in Figure 133. Although the R-value at 15C for Section #6 approached the threshold value, none of the asphalt binders tested failed the R-value at 15C criteria.

Lastly, the proposed WisDOT asphalt binder fatigue cracking protocol combines the GRP15°C and R-value at 15°C and was compared to the WisDOT BMD Test Section IDEAL-CT mixture performance (Figure 134). The results show a good relationship to the mixture fatigue cracking performance and were sensitive enough to identify the Section #4 asphalt mixture had a low fatigue cracking resistance as determined by the IDEAL-CT Index.

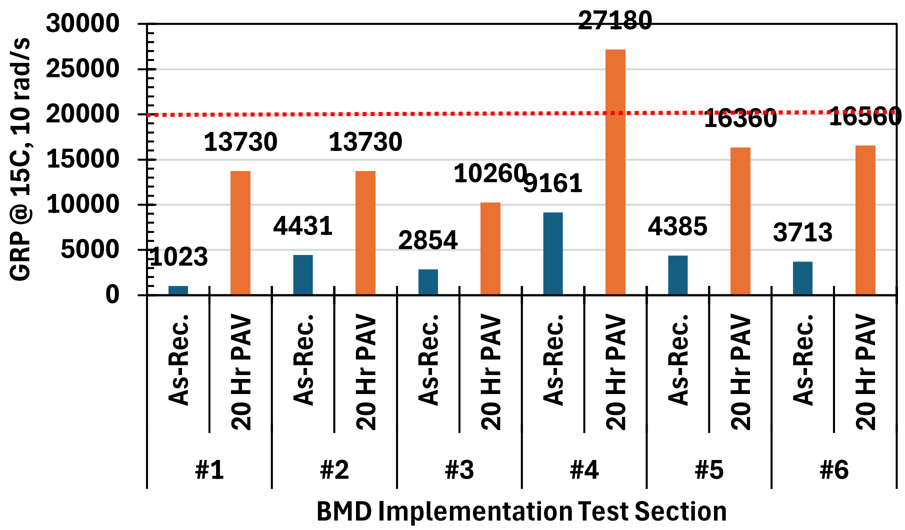


Figure 132 – GRP$_{15^{\circ}C}$ for As-Received and 20 Hour PAV Conditioned Recovered Asphalt Binders

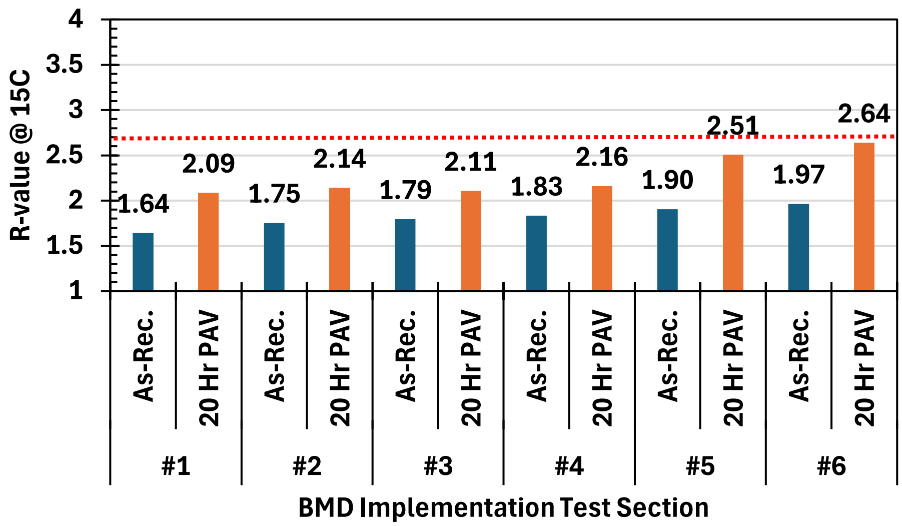


Figure 133 – R-value at 15C for As-Received and 20 Hour PAV Conditioned Recovered Asphalt Binders

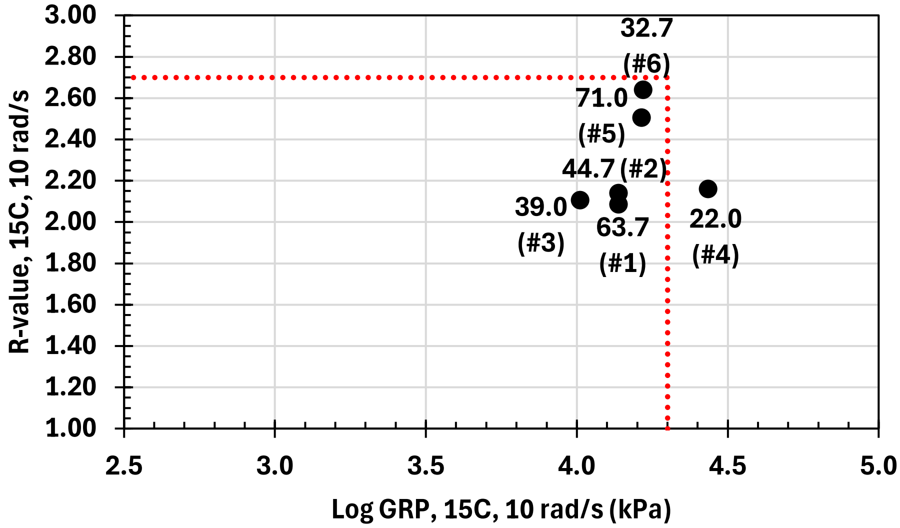


Figure 134 – Proposed WisDOT Testing Protocol for Fatigue Cracking Performance for WisDOT BMD Test Sections Recovered Asphalt Binder (data point label indicates CT Index result and test section #)

SUMMARY OF WORK EFFORT

The literature review has clearly indicated that overall, the ΔTc parameter is sensitive to the relaxation properties of the asphalt binder, and therefore, may help in identifying asphalt binders/additives/modification practices that accelerate the aging of asphalt materials. However, the ΔTc measurement at times incorrectly ranked polymer modified asphalt performance. The literature review identified the following conclusions important to this study:

- Ignoring any potential for testing error itself, one of the main factors controlling the final calculation of the ΔTc value is the potential to extrapolate the Tc(m) and Tc(S) values when only using two test temperatures. Depending on the condition, an error of up to 20% could be achieved by ensuring both a passing and failing test temperature is achieved during the BBR testing.
- In the most recent and comprehensive interlaboratory study, the average multiple laboratory allowable range of two (2ds) for the ΔTc measurement after 20 hour PAV conditioning was;
    - Single Operator = 0.8°C
    - Multiple Operator = 2.4°C

  Reasoning for the variability of the test procedure has not been well documented, although work conducted by Asphalt Institute suggests the following are the most common reasons; (1) poor equipment calibration resulting in variable levels of RTFO and PAV conditioning as well as issues with the BBR measurements; (2) calculation of true grade conducted via extrapolation instead of measuring actual passing and failing temperatures; and (3) lack of use of replicates during BBR testing. The Asphalt Institute study did clearly indicate that extended PAV conditioning, 40 hour continuous vs two 20 hour cycles, also had an impact on the ΔTc variability.
- Laboratory studies involving the use of recycled asphalt, which is known to have poor relaxation properties, showed ΔTc parameter was sensitive to the level and type of recycled asphalt when blended with a base asphalt binder. However, when mixture testing was included, relationships were not as strong, most likely due to the differences between 100% blending of liquids and variable blending of the recycled asphalt material in the asphalt mixture. Laboratory studies also indicated that there is a potential for poor ΔTc values when utilizing elastomeric polymer modification. This is primarily due to the strengthening of the asphalt binder that reduces the magnitude of relaxation and not due to age hardening. The addition of the ABCD test and protocols under NCHRP 9-60 project were developed to evaluate the fracture toughness of asphalt binders after exhibiting a poor ΔTc measurement to ensure the use of strong, elastic asphalt binders are not prohibited.
- Field studies have shown a good relationship between field cracking and ΔTc for neat asphalt binders but marginally for modified asphalt binders. Studies utilizing the recovery of the asphalt binder from the top ½” to ¾” of the pavement surface have found measured ΔTc values generally correlate with the field performance, and over time, as one would expect, the ΔTc values continue to get worse as the pavement ages and oxidizes.
- The ΔTc parameter can be compared to other master curve related Shape parameters. Research has shown that ΔTc correlates to parameters such as phase angle at constant modulus, crossover modulus, and R-value. Therefore, other parameters may also be able

to be utilized as a potential replacement for ΔTc that require less material for testing and possibly show better repeatability.

- With there potentially being an issue correctly characterizing elastomer modified asphalt binders with ΔTc alone, recommendations have been introduced to supplement the ΔTc with the ABCD based ΔTf parameter to make sure asphalt binders that exhibit good fracture toughness properties will not be disqualified within a purchase specification. The ΔTc & ΔTf performance space would be used to as an asphalt binder durability check while the Glover-Rowe parameter would be used to replace the current intermediate PG grade.

An extended laboratory characterization program was conducted to first develop a proposed methodology relating ΔTc and other asphalt binder parameter(s) to the IDEAL-CT Index value of 30. The parameter(s) was then evaluated using WisDOT approved asphalt binders, as well as how the addition of RAP binder would influence the overall trend in parameter(s). The testing program showed:

- Mixture characterization using the monotonic IDEAL-CT Index and the cyclic Overlay Tester showed a strong correlation to one another. However, results of the Overlay Tester showed far more sensitivity with respect to aging, test temperature and modification with respect to fatigue cracking performance. The results of the IDEAL-CT Index showed much less sensitivity to test temperature changes (25 to 20°C), and little to no benefit by using a polymer modified binder over a neat asphalt binder regarding fatigue cracking performance. It was also determined that an IDEAL-CT Index value of 30.0 was equivalent to an Overlay Tester cycles to failure of 10 cycles.
- Re-evaluating the method for determining an intermediate test temperature, a proposed version of the original Strategic Highway Research Program (SHRP) approach was selected. At a 98% confidence interval, the SHRP approach resulted in an intermediate test temperature of 15°C for asphalt binder testing. This is a reduction in temperature compared to the PG grade approach that resulted in Wisconsin having two different intermediate test temperatures of 19°C (northern climate) and 22°C (southern climate).
- Work conducted to compare the aged asphalt binder condition from loose mix aging to PAV conditioning showed that 20 hour PAV conditioning was equivalent to approximately 8 hours of loose mix conditioning at 135°C. This would indicate that the laboratory 20 hour PAV conditioning is slightly more severe than the current WisDOT loose mix aging, resulting in a slightly more conservative purchase specification approach (i.e. – purchase specification aging would be greater, and thereby, require slightly better asphalt binder performance).
- Final recommended approach to asphalt binder fatigue cracking assessment utilizes the Glover-Rowe Parameter (GRP_{15C}) and the Rheological Index (R-value) measured in the DSR at 15°C and 10 radians per sec. The benefit of using this approach over ΔTc and other methods evaluated in this study are:
    - A single DSR test is used to measure both GPR and R-value. A laboratory technician only needs to test the asphalt binder sample at one temperature (single point test) and record shear modulus (G*) and phase angle (δ) just like the current intermediate temperature PG grade. However, both a passing and failing temperature is not required.

- Some researchers have supported using the phase angle at a constant modulus approach to address asphalt binder fatigue cracking performance. This rheological shape parameter was found to correlate well with the R-value at 15°C and ΔTc. However, multiple test temperatures in the DSR would be required to determine the phase angle at constant modulus, increasing testing time over the single point R-value.
  - The GRP_{15C} is defined as a rheological Point parameter and was found to be highly correlated to the low temperature PG grade as determined by the BBR m-value and may even be a proxy for the quick determination of the low temperature PG grade of the asphalt binder.
  - The combination of a rheological Point and Shape parameter within an asphalt binder specification provides a proper evaluation of an asphalt binder to not only ensure that good fatigue properties are maintained, but also that the asphalt binder is sufficiently stiff so that the final mixture is stable.
- The GRP_{15C} and R-value at 15°C approach was "calibrated" to an IDEAL-CT Index value of 30.0 to ensure the asphalt binder and mixture fatigue cracking requirements were consistent within the WisDOT specifications. Ultimately, this resulted in a final recommended asphalt binder fatigue cracking method shown in Table 12 of this report. The methodology was able to capture a majority of the recovered asphalt binders that had an IDEAL-CT Index value below 30.0. As shown in Figure 135, the mixture test results from Phase 1 and Phase 4 were pooled together to show the resultant asphalt binder parameters vs IDEAL-CT Index. Symbols that are **RED** did not meet the IDEAL-CT Index value of 30.0, while **BLACK** symbols achieved a value greater than 30.0. Solid symbols are test results from Phase 1 (WisDOT Mixture Study), while hollow symbols are test results from Phase 4 (WisDOT BMD Test Sections). The two data points that are RED but had passing asphalt binder results were noted to have asphalt contents below 5% based on solvent extraction, and thereby, may have simply been under-asphalted compared to the rest of the asphalt mixtures tested.
- The proposed WisDOT method was found to be sensitive to the different asphalt binder suppliers' "same" PG grades. Both a neat PG58S-28 and polymer modified PG58H-28 were compared between three asphalt binder suppliers and the proposed GRP_{15C} and R-value at 15°C were capable of distinguishing differences among the three suppliers' products.
- The proposed method was also sensitive to RAP binder content with respect to the total asphalt content. The results showed that a maximum RAP content without the aid of a recycling agent/WMA additive would be approximately 35% before failing the proposed criteria in Table 12.
- The proposed WisDOT method was used to evaluate the recovered asphalt binders from the WisDOT BMD Test Sections. It was able to correctly identify the lowest IDEAL-CT Index Section (Test Section #4) and identified Test Section #6 as also having a low, but still passing, IDEAL-CT Index value.

Overall, the proposed method is a significant improvement over the existing intermediate PG grade, as well as the proposed ΔTc. A single point test (i.e. – one test temperature at only one loading frequency) provides the necessary information for the GRP_{15C} and R-value at 15°C calculation. In addition, the method was calibrated to the existing WisDOT IDEAL-CT Index mixture cracking performance criteria, providing continuity across WisDOT asphalt binder and mixture specifications.

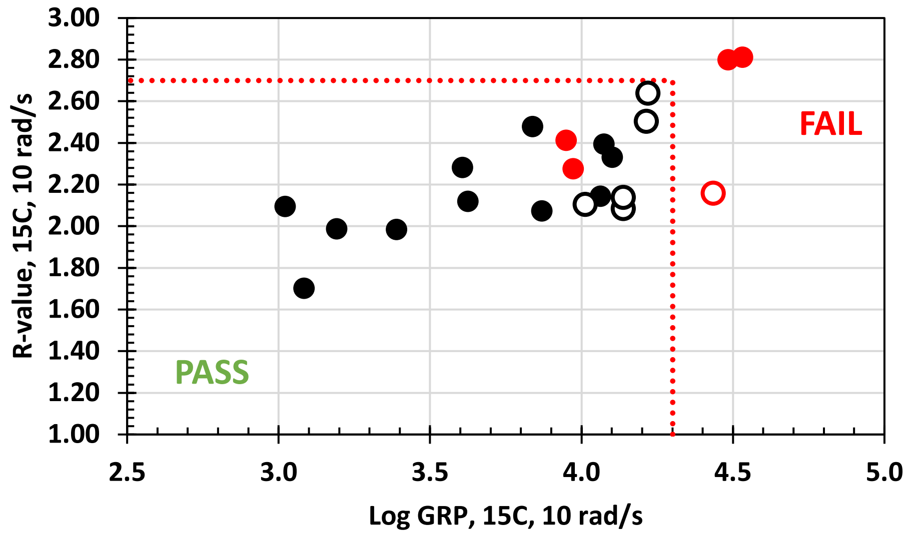


Figure 135 – Proposed WisDOT Asphalt Binder Parameters Compared to IDEAL-CT Index Values in Study (All Data n = 21)

CONCLUSIONS

A research effort was conducted to evaluate the potential use of the ΔTc parameter within an asphalt binder specification framework for WisDOT. Information generated during the literature review noted the associated variability with the ΔTc measurement and calculations. This has raised concerns regarding its use within a specification. This study also found that the ΔTc parameter was not as sensitive to the mixture fatigue cracking results from the IDEAL-CT Index testing as some of the other test parameters, or that the ΔTc correlated well enough to other rheological Shape parameters that could be incorporated with more confidence due to lower variability associated with the testing and calculations.

The final recommended approach using the GRP_{15C} and R-value at 15°C was recommended to be included with the WisDOT specification framework because;

1. The parameters correlated as well as or better than ΔTc to the mixture cracking tests;
2. The parameters have the benefit of simply testing at one temperature and one loading rate to ascertain the required data for calculations. Not only is this much faster than the ΔTc testing required, but it also increases the current testing speed regarding intermediate temperature PG grade determination;
3. Other researchers have proposed a BBR-based approach to calculating R-value, but this study found no significant improvement in predicting mixture fatigue cracking, and;
4. The GRP_{15C} was found to correlate highly to the low temperature PG grade determined using the BBR m-value. Since a majority of laboratory conditioned/aged asphalt binders

are commonly m-value dependent, the approach could be used to predict the low temperature PG grade without the use of conventional BBR testing.

In Phase 2 of the study, currently approved WisDOT asphalt binders were found to successfully meet the proposed test methodology and respective limits after 20 hr PAV conditioning. However, some of the tested asphalt binders would fail under 40 hr PAV conditioning. The proposed approach was also found to be sensitive enough to differentiate asphalt binder performance of the same PG grade but from different suppliers. In Phase 3 of the study, asphalt binder recovered from a recycled asphalt pavement (RAP) source and blended with a PG58S-28 and PG58H-28 from separate asphalt binder suppliers showed that recycled binder replacement (RBR) greater than 35% would require some type of modification in order to meet the proposed methodology. Therefore, it appears that adopting the proposed methodology, which has been calibrated to mixture cracking performance using an IDEAL-CT Index of 30, would not adversely disrupt the use of currently approved asphalt binders and still allow asphalt mixture suppliers to utilize RAP contents up to 35% binder replacement with minimal to no modification requirements (i.e. – recycling agents, etc.).

RECOMMENDATIONS

Although the original intent of the study was to assess ΔTc and its potential implementation into WisDOT's asphalt binder specifications, the study found that the combined use of the Glover-Rowe Parameter at 15^{o}C and 10 radians per second (GRP_{15C}) and the Rheological Index at 15^{o}C (R-value at 15C) provided the best means to characterize the asphalt binders evaluated in this study and is summarized in Table 12, shown below for convenience. The respective criteria are based on the asphalt mixture IDEAL-CT Index fatigue cracking performance of 30.0 after long-term conditioning as per the WisDOT specifications. In addition, the intermediate temperature for the asphalt binder testing was recommended to be revised to 15^{o}C for the entire state of Wisconsin based on the in-situ climate temperatures and the methodology described in the report and first introduced by Dr. Matthew Witczak in 1972 (Figure 70, again shown below for convenience) at a 98% confidence interval.

Although a significant amount of work was conducted in this study, there are still some gaps that should be evaluated in more detail. First, the asphalt mixtures utilized in the study were somewhat limited. Only 6 asphalt mixtures were evaluated, although at three different conditioning levels. It is recommended to include a larger database of asphalt mixtures across the state and evaluate their respective fatigue cracking mixture and binder properties. In particular, polymer modified asphalts should be included as the literature has shown their improved field performance over unmodified asphalt mixtures.

Second, the research study showed the limitations and lack of sensitivity with the IDEAL-CT Index test with respect to polymer modification and conditioning. The Overlay Tester method showed a greater impact of both polymer modification and conditioning. WisDOT may want to reevaluate their approach on performance testing methodology and utilize a mixture test method like the Overlay Tester that better captures these parameters.

It is also recommended to continue to assess the potential use of the GRP_{15C} correlation to the low temperature PG grade as determined using the BBR m-value. Figure 90, shown below for convenience, illustrates a strong relationship between these measurements. The GRP_{15C} could potentially be used as a quick determination for low temperature PG grade without the need for additional asphalt binder and testing time in the BBR.

Lastly, there needs to be more work to look at the interrelationship between volume of effective asphalt binder and the asphalt binder properties on the fatigue cracking performance of asphalt mixtures. A purchase specification only provides a starting point for selecting an asphalt binder, but ultimately it is the coupled effect of the volume of effective asphalt binder and the quality of asphalt binder that will support the cracking resistance of the asphalt mixture.

Table 12 - Proposed Asphalt Binder Fatigue Cracking Specification for Wisconsin Materials and Conditions

PAV Aging Temperature (°C)	**100**						**100 (110)**					
Dynamic Shear, T315 $G^*(\cos\delta)^2/\sin\delta^2$, 10 rad/s; **< 20,000 kPa**	**15°C**						**15°C**					
Dynamic Shear, T315 R=log(2)log(G*/1E9)/log(1-(δ/90)) at 10 rad/s; **1.0 < R < 2.7**	**15°C**						**15°C**					
Creep Stiffness, T313 at 60 sec & low temp **Stiffness < 300 MPa** **m-value > 0.300**	0	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	-30

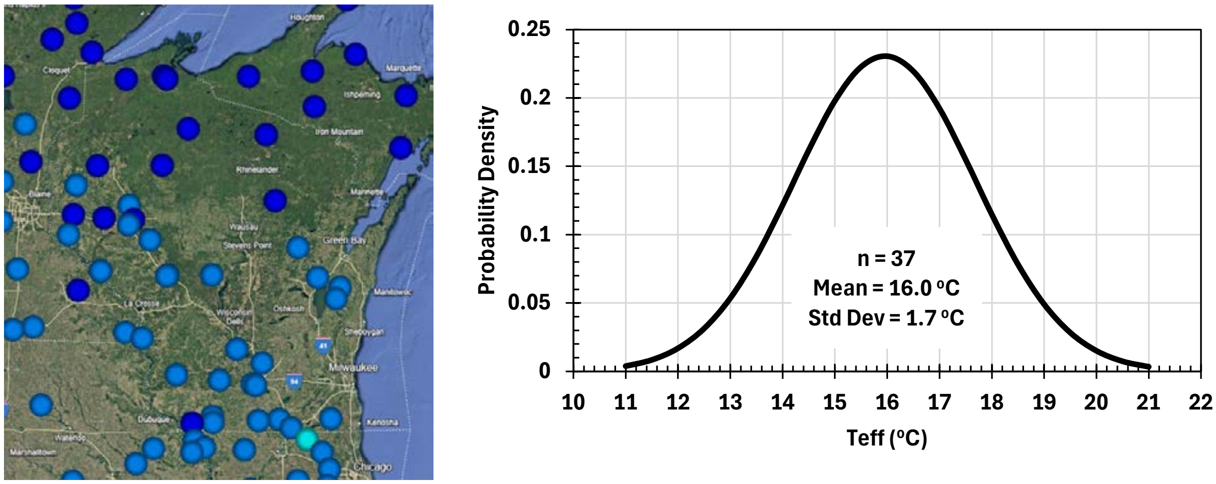


Figure 70 - Weather Data and Statistical Results for Wisconsin *Teff (FC)*

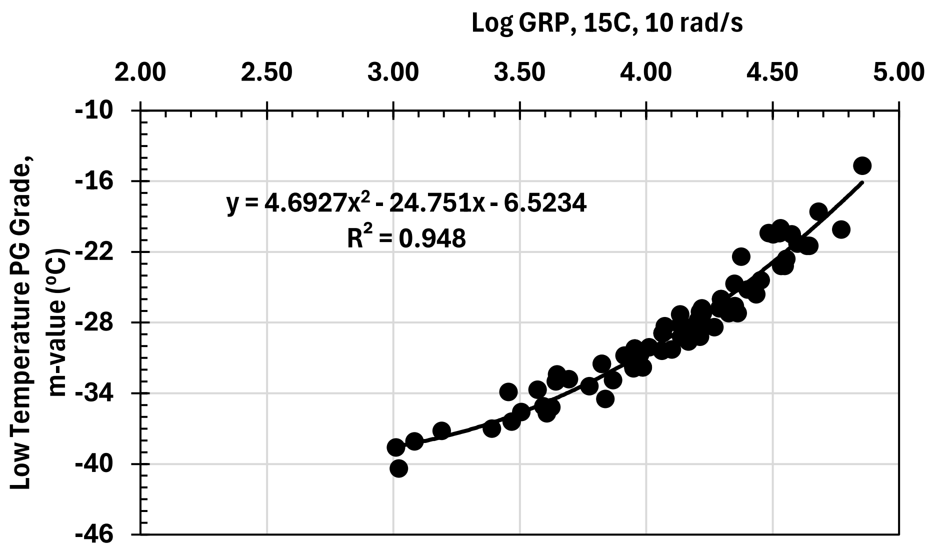


Figure 83 - Relationship Between GRP$_{15C}$ and Low Temperature PG Grade as Determined from the BBR m-value from Entire Study Dataset

REFERENCES

Anderson, R.M., 2017, “Delta Tc: Concept and Use”, *Past, Present and Future of Asphalt Binder Rheological Parameters, Transportation Research Board E-Circular E-C241*.

Anderson, R.M., 2022, *Developments in Asphalt Binder Tests and Specifications Resulting from National Research*, Presented at the 54th Annual Mid-Atlantic Quality Assurance Workshop.

Anderson, R.M., 2023, *Implementing Asphalt Binder Research Results into a BMD Framework*, Presented at the 102nd Annual Meeting of the Transportation Research Board.

Asphalt Institute, 2019, *State-of-the-Knowledge: Use of the Delta Tc Parameter to Characterize Asphalt Binder Behavior*, Asphalt Institute Technical Advisory Committee, Lexington, KT, 81 pp.

Asphalt Institute, *Research and Development of the Asphalt Institute Thickness Design Manual (MS-1) Ninth Edition*, Asphalt Institute, RR-82-2, 1982.

Azari, H., and C. Akisetty, 2024, *Recommended Changes to the Precision Estimates of T313*, Presented at the 56th Mid-Atlantic Quality Assurance Workshop, Washington, D.C.

Bennert, T., C. Ericson, D. Pezeshki, E. Haas, R. Shamborovskyy, and R. Corun, 2016, "Laboratory Performance of Re-Refined Engine Oil Bottoms (REOB) Modified Asphalt", *Journal of the Association of Asphalt Paving Technologists (AAPT*), Vol. 85, p. 163 – 208.

Bennert, T., C. Ericson, D. Pezeshki, R. Shamborovskyy, and C. Bognacki, 2017, "Moving Toward Asphalt Binder and Mixture Protocols to Minimize Fatigue Cracking on Asphalt Airfields", *Transportation Research Record: Journal of the Transportation Research Board*, No. 2633, pp. 177 – 126.

Bennert, T., E. Haas, C. Ericson, and E. Wass, 2019, *Evaluation of the Overlay Tester Test Procedure to Identify Fatigue Cracking Prone Asphalt Mixtures*, Center for Advanced Infrastructure and Transportation (CAIT), University Transportation Center (UTC), U.S. Department of Transportation (USDOT), 42 pp.

Bennert, T., C. Ericson, E. Haas, and E. Wass, 2021, *Investigation into the Identification and Potential Specification for the Fatigue Performance of Asphalt Binders*, U.S. Department of Transportation, Federal Aviation Administration (FAA), 83 pp.

Bennert, T., N. Garg, C. Ericson, and Nick Cytowicz, 2023, "Evaluation of Test Methods to Identify Asphalt Binders Prone to Surface-initiated Cracking", *Transportation Research Record: Journal of the Transportation Research Board*, No. 2677 (3), pp. 897 – 910.

Blankenship, P., R. M. Anderson, G. King, and D. Hanson, 2010, *A Laboratory and Field Investigation to Develop Test Procedures for Predicting Non-Load Associated Cracking of Airfield HMA Pavements*, Airfield Asphalt Pavement Technology Program (AAPTP) Project 06-01, 163 pp.

Bonaquist, R., 2016, *Critical Factors Affecting Asphalt Concrete Durability*, WisDOT ID no. 0092-14-06, Wisconsin Highway Research Program (WHRP), Wisconsin Department of Transportation (WisDOT), 128pp.

Christensen, D. W., and D. A. Anderson (1992). "Interpretation of Dynamic Mechanical Test Data for Paving Grade Asphalt Cements," *Journal of the Association of Asphalt Paving Technologists*, 61, pp. 67-116.

Christensen D.W., and Tran, N., 2022, *Relationships Between the Fatigue Properties of Asphalt Binders and the Fatigue Performance of Asphalt Mixtures,* NCHRP Research Report 982, Washington, DC: The National Academies Press.

D'Angelo, J., 2025, *Personal Communication*.

Elwardany, M., J.P. Planche, and G. King, 2020, "Universal and Practical Approach to Evaluate Asphalt Binder Resistance to Thermally-Induced Surface Damage", *Construction and Building Materials*, 255, 119331.

Elwardany, M., J.P. Planche, and G. King, 2022, "Proposed Changes to Asphalt Binder Specifications to Address Binder Quality Related Thermally Induced Surface Damage", *Transportation Research Record: Journal of the Transportation Research Board*, No. 2676 (5), pp. 176 – 191.

Glover, C., R. Davison, C. Domke, Y. Ruan, P. Juristyarini, D. Knorr, and S. Jung, 2005, *Development of a New Method for Assessing Asphalt Binder Durability with Field Validation*, Project No. 0-1872, FHWA.TX-05/1872-2, 334 pp.

Planche, J.P., 2023, *BMD: Does Chemistry Matter?,* Presented at the 102nd Annual Meeting of the Transportation Research Board.

Reinke, G., 2018, *The Relationship of Binder Delta Tc (ΔTc) & Other Binder Properties to Mixture Fatigue and Relaxation*, Presentation at the FHWA's Expert Task Group (ETG), Fall River, MA, May 2018.

Reinke, G., A. Hanz, R. Zhang, J. Sias, and E. Dave, 2022, "Laboratory Evaluation of Rheological, Chemical, and Compositional Properties of Bitumen Recovered from RAP Mixtures Treated with Seven Different Recycling Additives (RA) with Aging", Presented at the *101st Meeting of the Transportation Research Board (TRB),* National Academy of Sciences, Washington D.C., 20 pp.

Rodenzo, C., R. Moraes, F. Yin, and M. Fortunatus, 2021, *Recycled Asphalt Binder Study*, WHRP 0092-19-04, Wisconsin Department of Transportation, 125 pp.

Rowe, G., 2021, *Asphalt Parameters and Specification Development: Traveling Through the Jungle of Asphalt Parameter Development*, Presented at the Asphalt Institute Annual Meeting, Tucson, AZ.

Rowe, G., and S. Raposa, 2023, "Asphalt Binder Specification for Intermediate Temperatures and Climatic Conditions", Presented at the 61st Annual Petersen Asphalt Research Conference, Laramie, Wyoming.

Witczak, M.W., *Design of Full-Depth Asphalt Airfield Pavements,* Proceedings, Third International Conference on the Structural Design of Asphalt Pavements, London, 1972, pp. 550-567.

Yan, T., E. Mariette, M. Turos, and M. Marsteanu, 2023, "Evaluation of Physical Hardening and Oxidative Aging Effects on Delta Tc Asphalt Binders", *Road Materials and Pavement Design*, Vol. 24, pp. 626 – 639.

Zhang, R., J. Sias, E. Dave, A. Hanz, G. Reinke and Z. Wang, 2022, "Comprehensive Laboratory Evaluation of Recycling Agent (RA) Treated Plant-Produced Asphalt Mixtures", Presented at the *101st Meeting of the Transportation Research Board (TRB),* National Academy of Sciences, Washington D.C., 28 pp.

Zube, E. and J. Skog, 1961, "Final Report on the Zaca-Wigmore Asphalt Test Road", Proceedings of the Association of Asphalt Paving Technologists (AAPT), Vol. 38,